TECHNICAL REPORT STANDARD PAGE

1. Report No. LA.01/349	2. Government Accession No.	3. Recipient's Catalog No.
4. Title and Subtitle Assessment of Mitigating Embankment Settlement with Pile Supported Approach Slabs	5. Report Date December 1999 6. Performing Organization Code Tulane University Civil and Environmental Engineering Department	
7. Author(s) Sankar C. Das, Reda Bakeer, Jianqiang Zhong, and Mark Schutt	8. Performing Organization Report No.	
9. Performing Organization Name and Address Tulane University Civil and Environmental Engineering Department 6823 St. Charles Avenue New Orleans, LA 70118	10. Work Unit No. 11. Contract or Grant No. 97-4GT	
12. Sponsoring Agency Name and Address Louisiana Transportation Research Center 4101 Gourrier Avenue Baton Rouge, LA 70808	13. Type of Report and Period Covered March 1, 1997 through September 30, 1999 Final Report 14. Sponsoring Agency Code Louisiana Transportation Research Center	
15. Supplementary Notes		

16. Abstract

Problems involving highway bridge approach settlement have been observed at many sites in Louisiana. In Southeastern Louisiana, where subsoil settlement potential is the greatest, the bridge structures are usually lengthened in order to reduce the height of the approach embankment on some major structures, pile supported approach slabs have been used to improve the settlement profile of the approach slab. However, during the years, many pile supported approach slabs have performed well, while others have settled enough to create bumps at the interface with the bridge or roadway. In many other areas of the state, DOTD has implemented accelerated settlement techniques such as preloading in association with wick drains, with some promising results.

This research study has identified the factors that contribute to total approach settlement in pile supported approach slabs in Southeast Louisiana. The study involved examination of over 100 pile supported bridge approach slabs. Results of the study indicated that the main parameters that influence the performance of pile supported approach slabs are the height of embankment, subsoil conditions, surcharge height and duration and the length of the piles used for support. The main factor affecting slab settlement is downdrag, or negative skin friction, load imposed on the pile due to the weight of the roadway embankment. Therefore, settlement performance of a pile supported approach slab to yield an "ideal" settlement profile. The study has developed a database (LAPS) and a spreadsheet program (TU Drag) which may be used by DOTD Road and Bridge Engineers for design of approach slabs. In addition, a rating system was developed for assessment of the conditions of the interfaces along the approach slab based on the international roughness index (IRI).

17. Key Words bridges, foundations, approach slab, abutment, embankment, settlement, consolidation, concrete, soil		18. Distribution Statement **Unrestricted. This document is available through the National Technical Information Service, Springfield, VA 21161.**	
19. Security Classif. (of this report) Unclassified	20. Security Classif. (of this page) Unclassified	21. No. of Pages	22. Price

ASSESSMENT

OF

MITIGATING EMBANKMENT SETTLEMENT

WITH

PILE-SUPPORTED APPROACH SLABS

Final Report

by

Sankar C. Das, Ph.D., P.E.
Reda M. Bakeer, Ph.D., P.E.
Jianqiang Zhong
Mark A. Schutt

Civil and Environmental Engineering Department
Tulane University
New Orleans, Louisiana 70118

LTRC PROJECT NO. 97-4GT
STATE PROJECT NO. 736-99-0454

Conducted for

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
LOUISIANA TRANSPORTATION RESEARCH CENTER

December, 1999

ABSTRACT

Bridge approaches are the roadway portion that immediately follows the end of a bridge structure and form the transition between the bridge deck and the adjacent roadway. Occasionally, this transition element, regardless of pavement type, has developed rough rideability problems with time due to differential settlement between the highway pavement and bridge abutment. In southeastern Louisiana, where soil conditions are generally poor and highly compressible, pile supported approach slabs have been used to improve the transition between the roadway and bridge.

The first task of the study was to evaluate performance of pile supported approach slabs in southeastern Louisiana and identify the possible factors that contribute to their settlement. This was achieved by:

- Performing a parametric study on a large number of pile supported approach slabs to determine the factors that could possibly affect their performance.
- Performing field tests at a group of representative test sites.
- Developing a rating system using a modified system based on the International Roughness Index (IRI).

Based on the results of the parametric study and field tests, it was concluded that, as expected, factors such as embankment height, surcharge amount and period have the most influence on approach slabs performance. Factors such as speed limit, type of ramp, traffic count, etc. had no distinguishable impact.

A rating system using the IRI was developed using the information from the representative test sites and was used in the parametric study. The IRI slab rating system was also used to predict the condition of other approach slabs within the studied geographical area, by examining their IRI plots.

In the case of a pile-supported approach slab, the piles are typically embedded in a consolidating soil mass and no significant point support is typically available. This condition results in the subsoils both supporting the structure through "skin friction" along the embedded portion of the pile and yet allowing settlement of the structure to occur because of the consolidating mass in which they are embedded. It was also concluded that the problem of settlement of pile supported approach slabs is due to drag load imposed on the piles caused by negative skin friction. If piles are installed before most of the consolidation is complete,

the movement of the soil would cause negative friction (downdrag) load with the pile and subsequent downward movement.

At present, pile supported approach slabs are empirically designed. However, performance of existing pile supported approach slabs has varied significantly from one site to another. Therefore, design of pile supported approach slabs needs to be improved to account for site specific conditions. The effect of downdrag needs to be taken into consideration in the design of pile supported approach slabs by selecting the appropriate pile length or increasing the amount or duration of surcharge.

The second task of the study was made to develop an analytical method to accurately predict the settlement profile of a pile supported approach slab. This task was accomplished by:

- Developing a spreadsheet program (TU-DRAG) using soil/structure interaction methods to predict the required pile length based on the estimated downdrag loads.
- Using the developed spreadsheet program to predict settlement of the piles at test sites and compare the calculated pile settlements with those measured in the field.
- Performing a parametric study by selecting design parameters such as pile length, pile spacing, embankment height and approach slab dimension, so that the ideal approach slab settlement profile could be achieved.

The spreadsheet program which is user friendly and time effective, may be used directly by bridge design engineers to estimate the long-term performance of bridge/embankment approach system and to select the most cost-effective approach slab/embankment design.

ACKNOWLEDGEMENTS

Work of this project was conducted by the Tulane University Department of Civil and Environmental Engineering, under the sponsorship of the Louisiana Transportation Research Center and in cooperation with the Louisiana Department of Transportation. Their financial support is greatly appreciated.

Field work has been conducted by the principal investigators, graduate students of Tulane University and the LTRC/DOTD personnel under the administrative direction of Mark Morvant, Geophysical Research Manager of Louisiana Transportation Research Center.

Appreciation is also expressed to other personnel of the Louisiana Transportation Research Center (LTRC) and Louisiana Department of Transportation and Development (DOTD), especially to Gary Keel, Kevin Gaspard, Bill Tierney, Fred Wetekamm, Bob Roth and Ken Doyle for their technical support.

Thanks also to Tulane students Xin Miao, San Hla Aung and Sean Mclaure for their contribution effort in this project.

IMPLEMENTATION STATEMENT

The intent of this research has yielded a simplified design procedure that could be used to estimate the long-term settlement profile of a pile supported bridge embankment/approach slab system based on downdrag loads imposed on the piles used for support. The design should be based on selecting embankment height, pile length, pile arrangement, and maximum allowed-settlement that achieve an acceptable level of rideability. This procedure will likely benefit DOTD design engineers and will provide a tool for systematic evaluation of the most cost-effective approach slab/embankment system design.

TABLE OF CONTENTS

PAGE NO.

TABLE OF CONTENTS (Cont'd)

PAGE NO.

TABLE OF CONTENTS (Cont'd)

TABLE OF CONTENTS (Cont'd)

PAGE NO.

LIST OF TABLES

LIST OF TABLES (Cont'd)

TABLE NO. **PAGE NO.**

LIST OF FIGURES

LIST OF FIGURES (Cont'd)

FIGURE NO. PAGE NO.

LIST OF FIGURES (Cont'd)

FIGURE NO. **PAGE NO.**

LIST OF FIGURES (Cont'd)

FIGURE NO. PAGE NO.

INTRODUCTION

BACKGROUND

A bridge approach is the roadway portion that immediately follows the end of a bridge structure and provides a transition between the bridge's deck and the adjacent roadway. Generally this transition element, regardless of pavement type, has occasionally developed rough ridability problems due to differential settlement between the highway pavement or bridge abutment. This differential settlement originates from the fact that the bridge approach connects two different types of structures with different support systems. Settlement of a bridge abutment, usually supported on relatively firm soil or rock, point-bearing piles driven to a dense or stiff deep soil stratum, or long friction piles, is typically negligible compared to the settlement of a highway pavement, which is typically constructed over a natural soil subgrade.

Various factors have been reported to contribute to the differential settlement between a bridge deck and highway pavement *[1], [2]*:

- Compression of embankment fill material (primary and secondary compression as well as shear strain).
- Settlement (primary and secondary) of the soil subgrade (native soil under the embankment).
- Poor construction practices such as improper compaction of the approach embankment.
- Poor quality fill material.
- Loss of material from or around the abutment and approach slab due to erosion.
- Poor construction joints.
- Extreme temperature variations.
- Lateral deformation of the bridge approach embankment.
- Longitudinal or rotational movement of the abutments.

The first two factors are the most important elements that may cause the change in the approach slab's elevation *[3]*.

The approach slabs used in the United States are typically made of reinforced concrete. The typical approach slab has a span of 20 to 40 ft (6.10 to 12.19 m), which is applicable to embankments with relatively small differential settlements as shown in Figure 1.

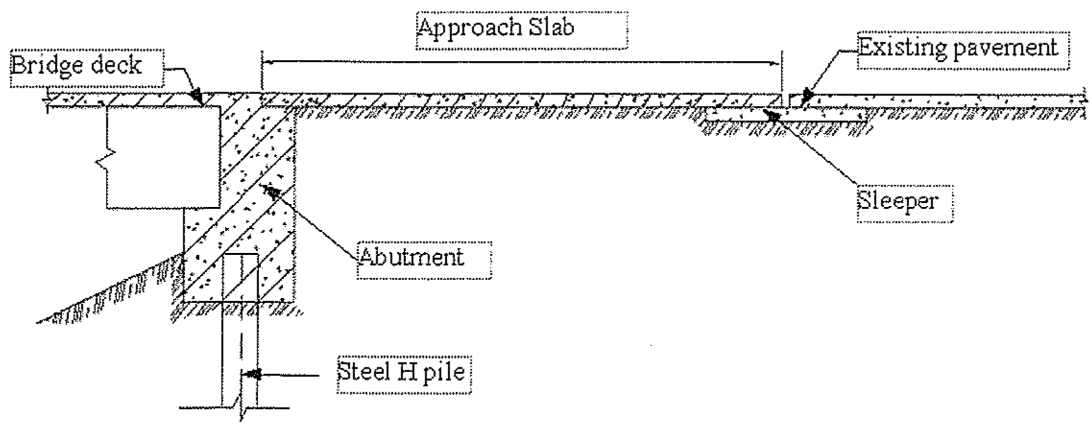


Figure 1
Typical non-pile supported approach slab system.

In southern Louisiana, excessive settlements are expected in the approach embankments due to the presence of soft and organic subsoils. From a geological standpoint, the upper soil strata encountered in southeastern Louisiana, which is the focus area of this study, comprise Holocene deposits overlying Pleistocene deposits. The much older Pleistocene soils consist mainly of massive dense sand and over-consolidated cohesive deposits and are typically encountered at depths ranging from few feet to about 50 to 100 ft (15.24 to 30.48 m) (fig. 2).

The much younger Holocene deposits, on the other hand, consist primarily of soft compressible cohesive or loose sandy strata. Thick humus and organic clay deposits are also encountered near the ground surface in many parts of the region. Therefore, the near surface soils in the study area consist mainly of massive formations of normally consolidated, and even under-consolidated, cohesive soils of extremely poor vertical drainage. Consequently, the ground surface in the region has been subsiding with time due to:

- Consolidation of the soft and compressible soils under their own weight.
- Lowering of groundwater table and related man-made activities.
- Oxidation and decomposition of organic soils.
- Global rise of sea water level.

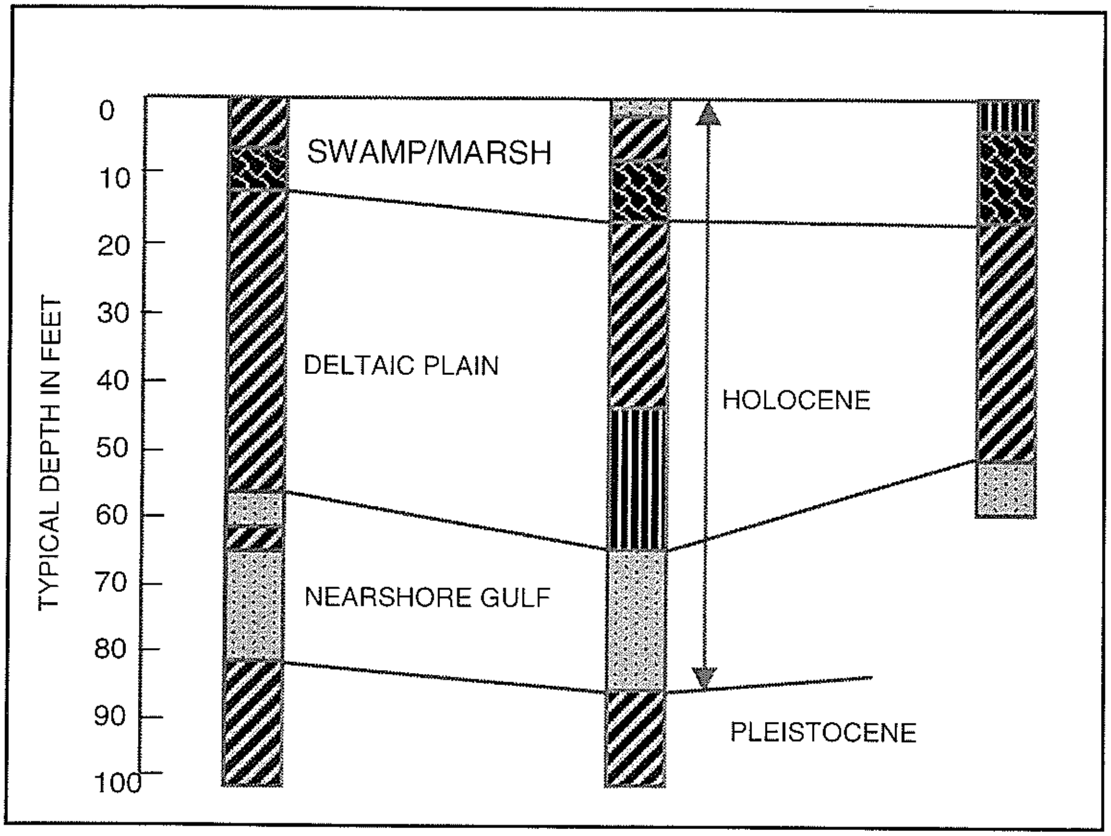


Figure 2
Typical south Louisiana geological soil strata

Due to the unique geology of the region, piles are frequently used for support of major and sensitive structures, including highway bridges. Due to load requirements and in order to minimize settlement, bridge piers and abutments are typically supported on relatively long piles with tips driven into stiff or dense Pleistocene Age soils.

Pile supported approach slabs were suggested for use by the DOTD to yield a more gradual transition between the bridge and roadway. In this selection, the approach slab is supported on piles of variable length where the longest pile is installed near the bridge abutment and the shortest near the roadway (fig.3). Since the pile-supported approach slab contains piles of variable lengths, it is expected that they would experience variable settlement under a constant load of a uniform height roadway embankment.

At the time of construction, the pile tops (butts) will be installed at a given design elevation. It was desired that these piles would gradually undergo variable long-term settlement to form a smooth transitional curve between the bridge abutment and roadway. With time, it was hoped that settlement of the shortest row of piles near the roadway would be comparable to that of the soil-supported roadway. On the other hand, settlement of the longest row of piles near the bridge abutment should be practically negligible. With intermediate piles settling differentially between these two extremes, it was hoped that the resulting settlement pattern of the approach slab would yield the desired gradual transition between the bridge and roadway.

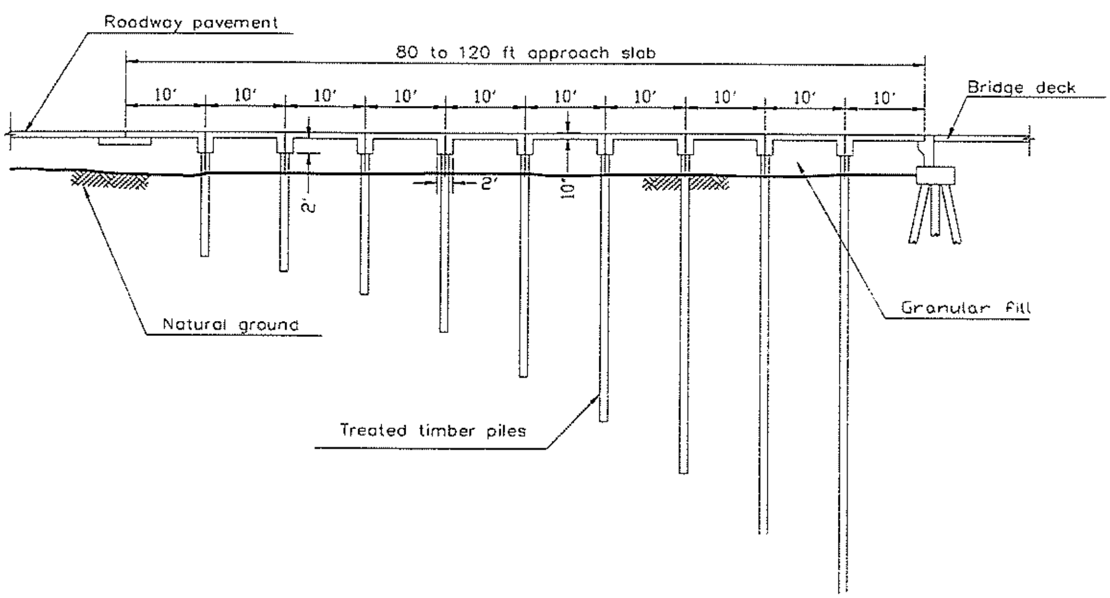


Figure 3
Typical configuration of a pile-supported approach slab system.

Figure 3 shows a typical configuration of a pile supported approach slab system. Over the years, a large number of pile supported approach slabs have performed relatively well, while others have settled enough to create bumps, or gaps, at the bridge end. Systematic evaluation of the current approach slab embankment system in Louisiana will likely benefit DOTD designers by providing an insight into bridge approach design.

Field observations have shown that some pile-supported approach slabs have performed adequately while others have developed abrupt or sudden differential settlement with time. It is suspected that this inconsistent performance is a result of variations in soil and design conditions at the various sites that are not accounted for in the current empirical design practice. In addition, the undesirable settlement profiles could be attributed to the effect of "downdrag" load imposed on the variable length piles due to surcharge loads and weight of the highway embankment, as will be discussed later in the chapter.

LITERATURE REVIEW

In 1957, a study was made to evaluate the settlement of the friction pile supported butment of the Aggersund Bridge in Denmark *[4]*. The study showed that the abutment had settled 800 mm (31.5 in), of which half was believed to be due to secondary time-effect. The reported settlement has occurred over a period of 15 years. Settlement calculations were made assuming a load transfer at the two-thirds point of the length and was in good agreement with measured values. Vertical settlement was of minor consequence, but horizontal movement was significant. The abutment tilted due to the difference in stress increase in the compressible clay stratum below the pile group. Consequently, the rear pile group carried a much smaller load than the others which resulted in differential settlement.

Another study conducted by West Virginia University *[5]* showed that perched bridge abutments tend to rotate and move laterally away from the bridge superstructure. The magnitude of movement is dependent on several factors:

- relative stiffnesses of the embankment and foundation soil
- depth of the compressible foundation soil, relative to the height of the approach embankment
- nature of the provided pile support

The backward rotation and horizontal displacement of this type of abutment is not prevented by the pile support.

In 1987, the Colorado Department of Highways conducted a study to identify the actors responsible for the settlement of pavements at bridge approaches and to suggest solutions for eliminating or minimizing such occurrences *[6]*. The following conclusions were made:

- Settlement within the foundation soil is mainly due to consolidation and is one of the major contributing factors to the settlement at the bridge approaches.
- Settlement due to consolidation is especially noticeable in approaches where embankments are mainly composed of compressible materials.
- A major factor in the settlement of bridge approaches is poor compaction of the backfill material.
- Erosion behind the abutment backwall can cause loss of subgrade and consequently causes the approach to settle.
- Before construction, the compressible foundation could be improved to reduce the approach settlement. Adequate time must be given for consolidation to occur.
- The embankment could be surcharged to preconsolidate the foundation soil.
- Sand drains and wick drains could be used with the surcharge to reduce the time of consolidation.
- The backfill behind the abutment should be well-graded to provide better compaction and higher densities.
- Proper drainage should be provided to prevent erosion along the abutment faces.

In a study conducted by the University of Nebraska, state highway department and agencies involved in bridge design, construction and maintenance were surveyed. The surveyed agencies were generally in agreement that high traffic volume and high embankments increase the degree of settlement. Most agencies reported the use of asphalt overlays and slab-jacking methods once settling has occurred *[7]*. Use of a sleeper slab, specifying select backfill material and use of wick drains to accelerate consolidation rate were the most common recommendations made by the organizations.

A large number of bridge approaches in Oklahoma had experienced substantial settlements and their maintenance costs had increased excessively. Among the major factors that caused this settlement was consolidation of the subsoils. Zaman *[8]* presented an analysis of the consolidation settlement of a bridge-approach foundation based on a nonlinear finite-element method (FEM) type analyses. The analyses included the formulation of an infinite element to accurately represent the lateral boundaries of the finite-element mesh. A bridge-approach site in Oklahoma was analyzed for time-settlement history and pore-pressure dissipation characteristics.

Another study that used finite element analysis was a study made by the Engineering Research Institute of Iowa State University *[9]*. A state-of-the-art, three dimensional, nonlinear finite element algorithm was developed and used to study pile stresses and pile-soil interaction in bridge abutments. One of the conclusions of this study was that thermal expansion of the bridge introduced a vertical load on the piles and reduced its vertical load-carrying capacity.

A finite element study was conducted on a non-pile supported approach slab at the University of Maryland *[10]*. Nonlinear analyses were performed to model the soil: Portland Cement Concrete (PCC) approach slab and sleeper slab using quadratic isoparametric elements and two-dimensional interface elements with two nodes and two degrees of freedom at each node. The interface elements were used to allow for separation and sliding between the approach slab and the embankment fill and between the embankment fill and the abutment. An elastic-plastic model with Drucker-Prager yield criteria and the Coulomb condition for failure were used to model the soil.

In the same study, several parameters that significantly affect the performance of approach slabs such as slab length, fill height, fill density and slab-abutment connection were investigated. The research showed that the most important parameter was the fill height. Fill density and slab-abutment connection were also found to have a significant effect on the approach slab performance.

One of the various methods available for the treatment of soft soil foundation is the wick drains method which can reduce the time required for the foundation soil to consolidate, perhaps by 50 to 75 percent over surcharging alone *[11]*.

The design of wick drains is theoretically simple, though practically difficult because sound field data regarding consolidation has to be available *[12]*. Wick drains have been used successfully in many projects in California including the structure approach fills in Eureka at Elk River Road on Route 101, structure approach fills at Elkhorn Slough on Route 101 in Moss Landing, and the structure approach fills on Route 101 at the junction with Route 92. *[13]*. These have also been used in many other projects including the construction of the New Istana for the Sultan of Brunei *[14]* and for the U.S Navy's Home Port Facility in Pascagoula, Mississippi.

In many areas of Louisiana, DOTD has also implemented accelerated settlement techiques such as preloading in associated with wick drains with generally favorable results.

EVALUATION OF BRIDGES

Highways and bridges are usually evaluated on a regular basis to determine their performance and condition from structural and ride-comfort stand points. When evaluating the condition of a bridge, it is difficult to figure out if the experienced movements are tolerable or not. Several factors have to be considered in the assessment:

- amount of movement.
- type of structure.
- effect of each component in the structure.
- cost of alternative choices (if the problem calls for repair or replacement).
- impact on traveling public.

Sometimes large movements which may cause minimal damage to the structure have to be tolerated because other alternatives could be too costly and prohibitive *[15]*. On the other hand, a small differential settlement between the various bridge components may cause discomfort to drivers and even safety concerns.

When evaluating a bridge, many factors have to be taken into consideration to identify the cause(s) of a problem, if one exists. In California, 820 approaches to highway structures were evaluated with respect to ride comfort and frequency of maintenance being performed *[3]*. This data was compiled along with many parameters such as age of the structure, fill height, abutment skew, preload surcharge period, geographical region, ingress or egress approach end, length of slab and average daily traffic. The parameters that appeared to have an effect on the need for at least one-time maintenance for the approaches were age and the geographical region. Other parameters such as ingress or egress approach end, skew, fill height, settlement period, traffic volume, and length of slab did not have any significant effect on the need for at least one time maintenance.

In addition to the factors mentioned above, a study done by the University of Kentucky also mentions that lateral and vertical deformations (or creep) of the bridge and approach pavements are also causes of differential movement. The findings of this study were similar to those of the other studies mentioned above *[1]*.

CURRENT CONDITION RATING SYSTEM

Current condition rating of the approach slabs is routinely performed by New Orleans DOTD personnel by visually inspecting the slab and assigning a rating value based on its condition. This rating is between one and ten, where ten is excellent and five or less is very poor. Table 1 shows the system of rating used by DOTD. For the 112 approach slabs investigated in this project, the assigned ratings ranged between five and eight. Therefore, the condition of the approach slabs under investigation ranged from poor to very good.

Table 1
Current condition rating system used by
New Orleans District 02 Office

Current Condition	Rating
<5	Very Poor
5	Poor
6	Fair
7	Good
8	Very Good
9-10	Excellent

RESEARCH ON PILE SUPPORTED APPROACH SYSTEM

Pile-supported approach slabs were used mostly in Louisiana *[16]*. The current practice of pile supported approach slab design in Louisiana is based on an empirical methodology and past experience. Timber piles of decreasing length are commonly driven at specified spacing along the span of the approach slab to provide a gradual transition from the abutment to the roadway embankment.

Holmberg *[17]* described the design, construction and performance of pile supported bridge approaches at 11 sites in Thailand and reached the conclusion that use of embankment piles beneath bridge approaches on soft ground is a suitable technique to eliminate the traditional problem of differential settlement.

In their paper entitled "Embankment Piles", Broms and Wong *[18]* documented the use of embankment piles for supporting fill and structures, deep excavation and slope protection. Design considerations for each application were also introduced.

Performance of pile supported approach slabs remains unknown since no assessment of this type of construction has been made. However, some analysis criteria and design considerations for end bents and approach slabs are provided by DOTD as a general policy statement and a supplement to AASHTO specifications.

DOWNDRAG

Negative skin friction is a downward force exerted by a consolidating soil mass on a pile. This can occur under certain conditions *[19]*:

- A cohesive fill is placed over a cohesionless soil deposit.
- A cohesionless fill is placed over a compressible, cohesive deposit.
- The water table is lowered causing ground subsidence.
- A pile-driving operation produces negative stresses in the upper part of the shaft when the load is released and the pile shaft expands upwards.

A stress transfer exists between the pile and the soil. There is movement in the soil which is restrained by the pile. This restraint builds up force in the pile and there is no accelerating movement in the pile. The forces on the pile are therefore in equilibrium. There is negative friction in the upper part of the pile which results in a load on the pile that increases from zero at the pile top to a maximum at the depth of equilibrium. This point of equilibrium is referred to as the "neutral point or neutral plane" (fig. 4). At this depth no relative displacement occurs between the pile and the soil. Below the neutral point the load decreases by being transferred to the soil by the positive shaft resistance in addition to the tip resistance *[20]*. There are several methods that could be used to reduce negative skin friction *[21]*:

- Installing a casing around the pile shaft to prevent direct contact with the settling soil.
- Placing the pile in a pre-drilled hole of larger diameter than the pile shaft and filling the gap with bentonite slurry, which limits the negative friction to a relatively low value.
- Applying bitumen of appropriate consistency on the pile shaft to reduce friction.
- Increasing the pile length to generate more positive skin friction.

Approach slabs can be designed to take downdrag into consideration by varying the length of piles along the approach slab. Currently, pile supported approach slabs built in Louisiana are designed this way, except that the design is purely empirical. Therefore, some slabs have performed well while others have not.

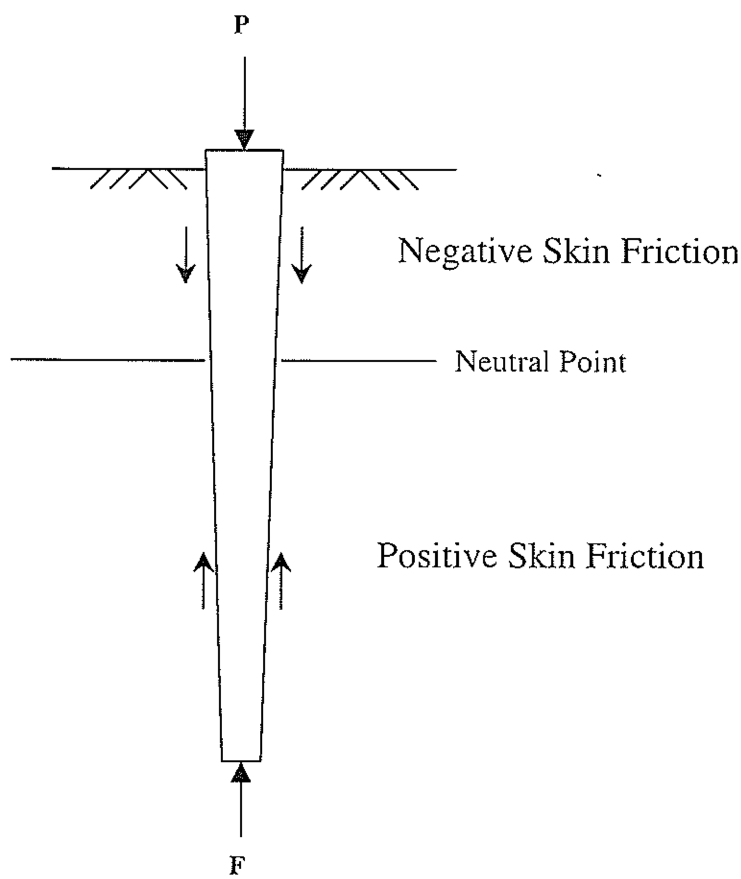


Figure 4
Negative skin friction

OBJECTIVE

The objective of this research was to identify the factors that contribute to the settlement of pile supported approach slabs in southeastern Louisiana. This objective has been achieved by performing a parametric study based on a database of actual bridges within the subject geographical area and computer simulations using the soil/structure interaction method to develop a design tool for bridge approach slabs. This methodology is intended to improve settlement profile instead of minimizing it. This objective has been accomplished through the following tasks:

1. A large number of pile supported, and a few non-pile supported, approach slab sites in south Louisiana were identified in coordination with DOTD and LTRC personnel. The design, soil information and traffic data for these sites were compiled from their as-built drawings and maintenance records available at DOTD offices.

2. A computer database containing all pertinent information of these bridges including the parameters that could potentially affect the performance of their approach slabs was developed.

3. A parametric study using the information compiled in the database was performed.

4. Some representative pile-supported approach slabs were selected in coordination with DOTD and LTRC personnel.

5. Performance of the approach slabs at these representative sites was evaluated via field tests.

6. Simplified soil/structure interaction methods were used to examine the effects of various parameters on the performance of a pile supported approach slab.

SCOPE

The researchers have identified and located about ninety sites of bridges with pile-supported approach slabs across southeastern Louisiana. Seven representative sites were selected for thorough in-situ investigations and sampling, the results of which have been compiled in a computer database.

Field work was done by Tulane researchers in collaboration with DOTD and LTRC personnel at the representative sites that included visual inspection of pavement, bridge, approach slabs and ramps, settlement measurements, rideability, etc. Detailed information of all the identified sites has been compiled in the database.

Performance of a given approach slab was assessed based on visual inspection, surveys and assessment of road surface conditions. Field instruments used included a walking profiler, Dynatest, laser profiler, geodetic total station, soil wash borings and cone penetrometer.

A simplified soil/structure interaction method was employed to assess the performance of pile supported approach slabs as mentioned above. A design procedure was developed to determine the most effective bridge embankment approach slab design. The proposed design considers embankment height and maximum allowed settlement to determine the required pile lengths and distribution along the approach slab length. It is anticipated that this selection will improve the long-term settlement of the approach slab and achieve an acceptable level of rideability.

METHODOLOGY

IDENTIFICATION OF SITES

Over 100 bridge structures with Portland Cement Concrete (PCC) approach slabs in the southeastern Louisiana area were identified by LTRC and Tulane University, about 80 percent of which have pile supported approach slabs. A list of the selected sites is given in Appendix A. Approach slabs in Sites 1 through 90 are pile supported, while approach slabs in Sites 91 through 112 are non-pile supported. Most of the sites were selected on highways I-310, I-10, I-510, I-610, LA 3139 and US 90.

The identified sites included almost all pile supported approach slabs in southeastern Louisiana except for those located in the Houma/Thibodeaux area, where the approach slabs were constructed over light-weight aggregate fill (shell). Only few non-pile supported approach slabs were selected for comparative purposes. Therefore, no conclusions can be made in regard to the performance of these non-pile supported slabs versus pile-supported approach slabs due to the limited number of non-pile supported approach slabs contained in the database.

One hundred and four sites were identified and their related drawings were re-produced either from microfilm archives at the DOTD office in Baton Rouge or from their blue prints available at the DOTD New Orleans district office. The current condition ratings and maintenance records of the bridge sites located in the New Orleans district were also collected. Out of the 90 pile-supported approach slabs, 63 sites were identified in Orleans, Jefferson and St. Charles Parishes that were targeted for thorough review and evaluation.

The collected information, such as approach slab dimension, approach slab reinforcement, pile spacing, pile length, embankment dimensions, embankment material, soil conditions, etc., was compiled into a database named LAPS, which is presented in the next section.

DATABASE COMPILATION

The collected information for all one hundred and four identified sites was compiled in the database LAPS for use in the assessment of possible causes for the approach slab settlement and for future use by DOTD, if so desired. The database was developed as part of this study at Tulane University using FoxPro software (Microsoft – 1985) and a personal computer platform.

Information was collected from the approach slab records available at the DOTD microfilm office in Baton Rouge and the New Orleans District 02 office over a period of three months. The related drawings for each site were either printed from the microfilms or copied from the blueprints kept in the New Orleans district office. All maintenance records of the bridge sites selected for the study and available at the New Orleans district office were also collected.

The database was organized into 12 sections as follows:

1) Identification of Approach Slab Site

Information concerning the structure number and site location.

2) General Information

Year of construction, approach foundation type, joint type, number of lanes and whether or not the slab is an on-ramp or off-ramp.

3) Current Condition

Current condition and ranking of the approach slab.

4) Approach Slab Dimensions

Approach slab dimensions such as length, width and thickness.

5) Approach Slab Materials

Quality of common approach slab materials (concrete and steel).

6) Approach Slab Rehabilitation History

Rehabilitation history of the various sites.

7) Embankment Information

Embankment information such as dimensions, material and placement techniques.

8) Soil Conditions

Predominant soil material and properties at the site.

9) Approach Slab Piling System

Approach slab embankment piles such as pile material, diameter, length range, spacing and reinforcement.

10) Main Bridge General Information

General information, such as bridge type, material, foundation type, number of ramps, number of approach slabs and whether the bridge crosses over a river or a roadway.

11) Road Condition

Road maintenance records such as current condition, rehabilitation history and design modifications.

12) Traffic Conditions

Traffic count, design speed limit and design moving loads.

Operating instructions for the database LAPS are given in Appendix B.

INTERNATIONAL ROUGHNESS INDEX (IRI) RATING SYSTEM

The International Roughness Index (IRI) information was obtained by the DOTD personnel for roadway / approach slab / bridge using the laser profiler. The information was used to plot graphs of IRI data for 90 of the 104 approach slabs under investigation. A sample of such graphs is shown in Figure 5. The location of the approach slab, roadway and bridge are shown on the graph. Relevant graphs are given later of the International Roughness Indices sections for the various test sites. The remaining graphs are available in reference *[22]*. The graphs indicate that the transition between the bridge and the approach slab and the transition between the roadway and the approach slab generally yield high IRI values ranging between 3 and 27.

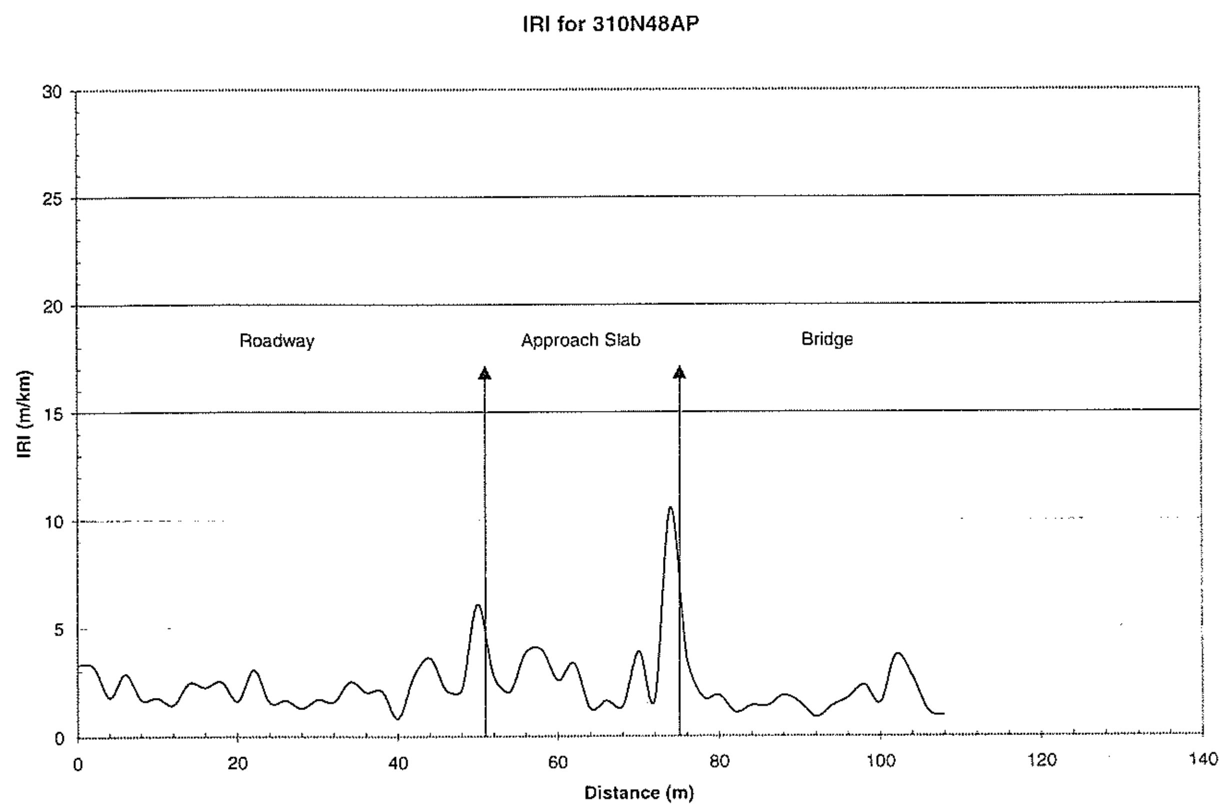


Figure 5
IRI Graph

The recognized standard rating for pavement using the IRI is shown in Table 2. According to the pavement evaluation criterion, all approach slabs investigated with the laser profiler would be rated as poor to very poor. Therefore, a new IRI rating system, was developed and is shown in Table 3. The development of this system is explained in the next section.

Table 2
IRI Pavement Ratings Used by LTRC
for Roadway Pavement

Range (IRI)	Rating
0.9 to 1.26	Very Good
1.26 to 1.90	Good
1.90 to 2.37	Fair
2.37 to 3.16	Poor
3.16 and higher	Very Poor

Table 3
IRI Approach Slab Rating System Developed
by Tulane University for Approach Slabs

IRI Range	Rating
0 to 4	Very Good
5 to 8	Good
9 to 12	Fair
13 to 16	Poor
17 and above	Very Poor

LASER PROFILER

The Federal Highway Administration (FHWA) requires the states to report road roughness according to the IRI scale for inclusion in the Highway Performance Monitoring System (HPMS). Almost every automated road profiling system contains a software for calculating the IRI. The IRI is defined as a property of the true profile, and therefore can be measured with any valid profiler *[23]*.

The laser profiler used by LTRC consists of a van equipped with an onboard computer interfaced with three height sensors, two accelerometers and a distance sensor. The height sensors are attached to a replacement bumper assembly or special mounting bar in the center of the van and at both sides in each wheel track (fig. 6). The height sensor is a Selcom laser with a resolution of 0.001 inches (0.0254 mm). It provides continuous coverage of the roadway at a constant operating speed of 60 mph (96.54 kmph). The accelerometers are mounted over the wheel path on each side of the van with the respective height sensor. These three devices function together to allow for road profile and surface height data to be collected which is used to calculate the road roughness.

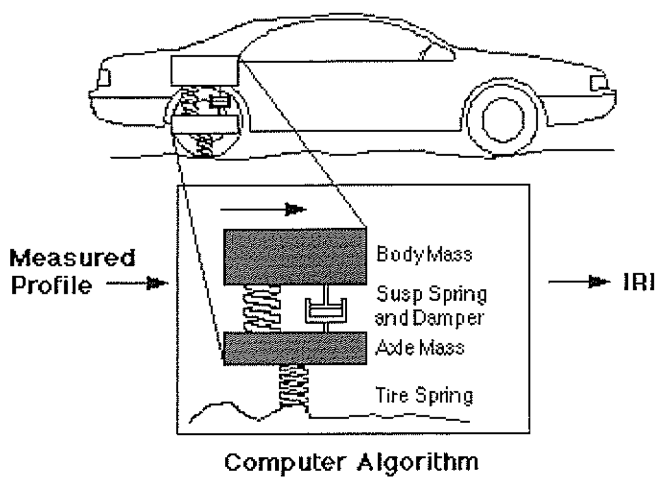


Figure 6
Quarter-car model

POSSIBLE CAUSES FOR APPROACH SLAB SETTLEMENT

Using the information compiled in the database, analyses were made to determine the possible causes for approach slab settlement. Bar graphs and pie charts were used to compare various parameters of concern for both pile-supported and non-pile supported approach slabs selected for this study. Ratings from the current condition records as well as the newly developed rating system using the IRI were used to compare performance of the different approach slabs. Samples of the bar graphs and pie charts are shown in Figures 7 and 8. The entire set of graphs and charts is available in the Tulane University Civil and Environmental Engineering Department.

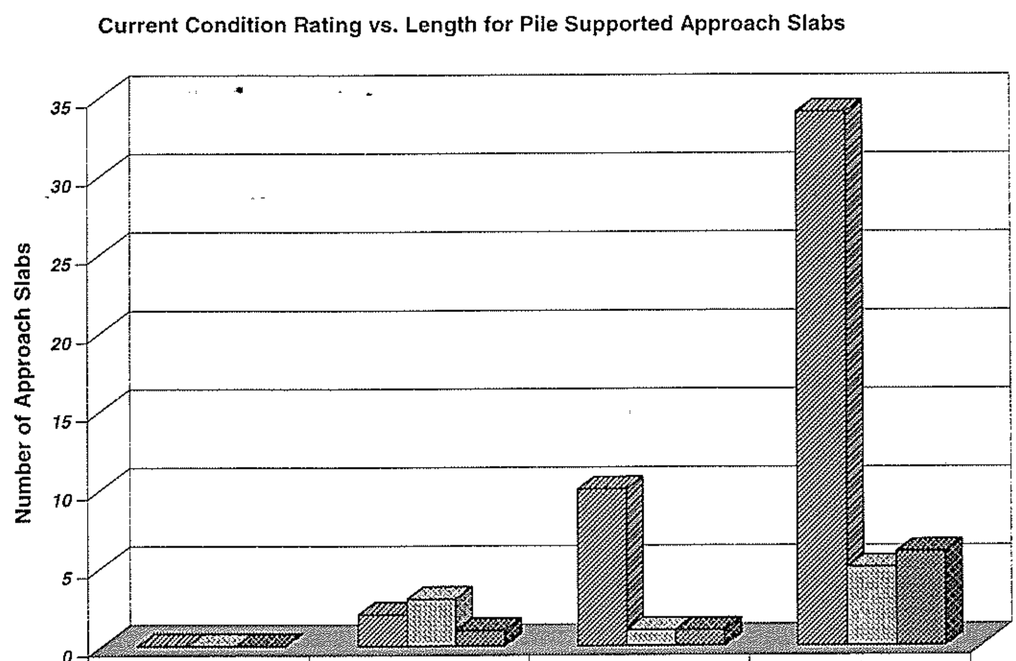
Figure 7 shows a bar graph comparison of current condition ratings versus length of the pile supported approach slabs investigated in this study. The graph shows that most of the 80 ft (24.38 m) approach slabs were rated as seven or eight and most of the 120 ft (36.58 m) approach slabs were given a rating of eight.

RESULTS OF PARAMETRIC STUDY

Many parameters were compared to identify the cause for settlement of approach slabs. Most of the sites are located in three parishes: Orleans, St. Charles and Jefferson. There are 63 pile supported approach slabs and 21 non-pile supported approach slabs in this study for these three parishes, of which 34 were located in Orleans parish, 32 in Jefferson parish and 18 in St. Charles parish. According to the charts which are given in the reference *[22]*, the current condition ratings indicate that performance of the approach slabs in Orleans Parish was the best and that of those in St. Charles Parish were the worst. The current condition ratings show that pile supported approach slabs performed well. The bar graph for the IRI rating given in Figure 9 shows a relatively normal distribution.

Effect of age on an approach slab is another parameter that was studied. The same 84 approach slabs were used for this comparison. There are 23 approach slabs that were built in the 1990s, 15 that were built in the 1980s, 24 that were built in the 1970s and 5 that were built in the 1960s. According to the pie charts and bar graphs, newer approach slabs are typically in better condition for both pile and non-pile supported approach slabs. However, the IRI ratings show that pile supported approach slabs built in the 1980s have performed better than those built in the 1990s. On the other hand, the IRI ratings do show that the approach slabs built in the 1990s performed better than those built in the 1970s. Based on these observations, it can be concluded that the newer approach slabs are performing better than the older ones. However, more long-term consolidation settlement is expected to occur in the newer slabs with time.

Of the 63 pile-supported approach slabs, 46 were 80 feet (24.38 meters) long. The 80 feet (24.38 meters) long pile supported approach slabs performed the best based on the current condition ratings and based on the IRI ratings the 80 feet (24.38 meters) and 100 feet (30.48 meters) approach slabs were best. Some of the approach slabs studied in detail in later sections showed that most of the settlement in the approach slabs occurred close to the abutment. This might be the reason why the longer approach slabs were not rated as high as the shorter ones. Therefore, it appears that it is more beneficial to design shorter slabs. It can



	5's	6's	7's	8's
24.4 m (80 ft)	0	2	10	34
30.5 m (100 ft)	0	3	1	5
36.6 m (120 ft)	0	1	1	6

Rating

Figure 7
Effect of approach slab length

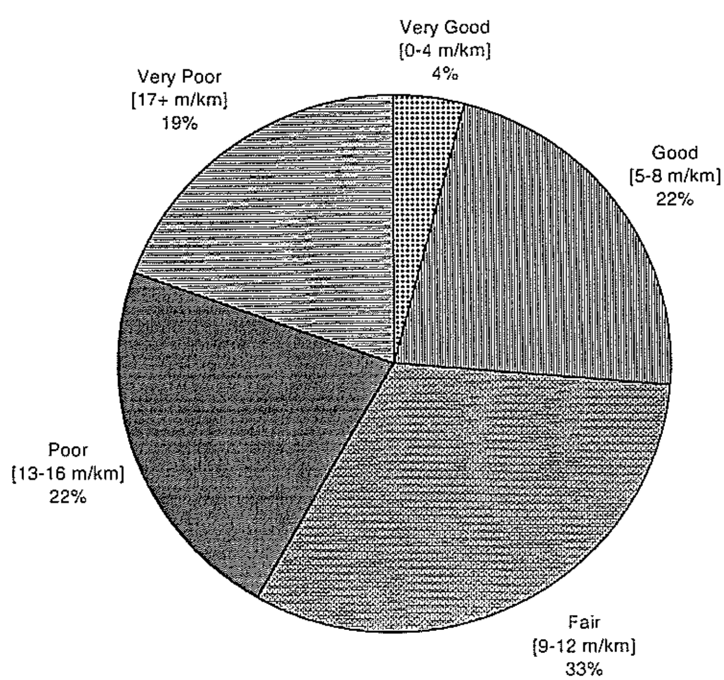


Figure 8
IRI Ratings of pile approach slabs

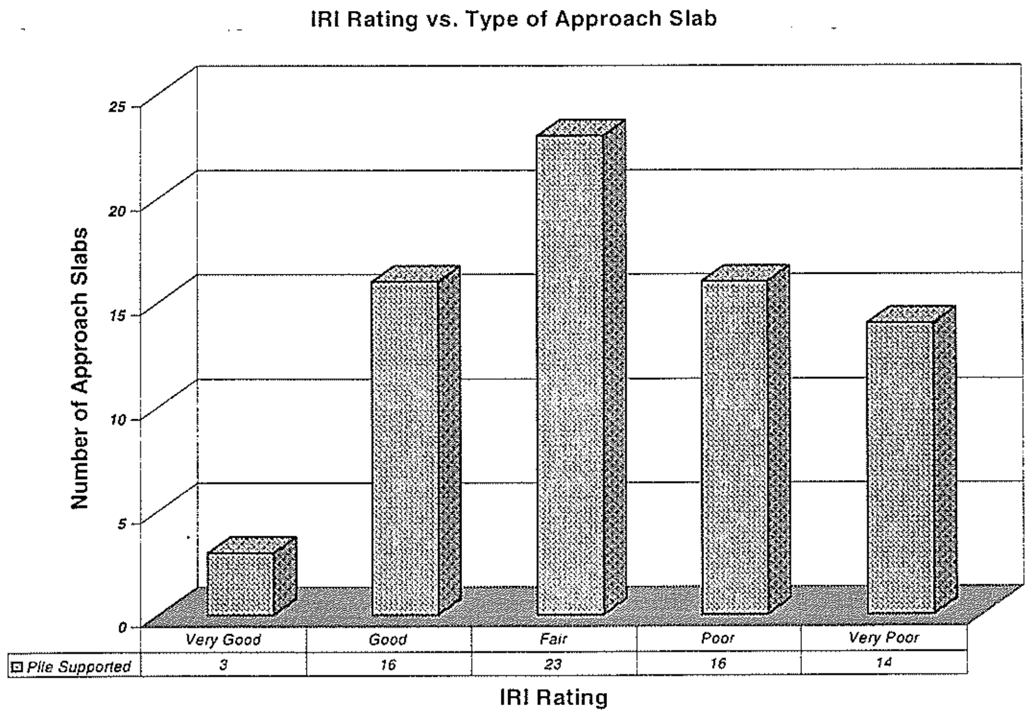


Figure 9
Effect of approach slab type

be concluded that the 80 feet (24.38 meters) long approach slabs performed the best.

Another parameter examined was whether the approach slab was on an approach or exit to the bridge structure. This parameter does not appear to affect the approach slab's performance.

The charts for the various embankment heights and the daily traffic count were inconclusive. The same was the case with the daily traffic count. Speed limit was another parameter investigated. No conclusion can be drawn from the speed limit data. The comparison between curved and straight approach slabs was also inconclusive since there was only a limited number of curved approach slabs.

When comparing the pie charts and bar graphs for IRI ratings and current condition ratings, it was found that for the most part the current condition rating yields better ratings for the approach slabs than the IRI system. It is believed that this is due to the subjective nature of the current condition rating whereas IRI ratings are based on a visual inspection of the approach slab. It is also important to visually inspect a bridge to make sure the structure is in a safe condition.

Based on the results of the parametric study, it appears that no conclusive factors were identified to be impacting the performance of pile-supported approach slabs. Therefore, it was concluded that since the construction of these slabs involves use of various heights of fill and surcharges of variable durations, focus should be given to investigate the downdrag phenomenon and its possible impact on performance of the variable length piles used in approach slabs. Representative sites were also identified to establish some quantitative data for use in the downdrag study.

SELECTION OF REPRESENTATIVE TESTING SITES

Seven representative sites were selected for thorough in-situ investigations. Figures 10 a and 10 b show the map location of six of these test sites located along I-310 and the remaining test site located on LA 3139. The sites located on I-310 were built in the 1990's and are built over a swamp area. Site 7 located on LA 3139 was built in 1982 and is located in an urban area. A summary of information collected for the seven test sites chosen for detailed field studies is shown in Table 4. This table includes information such as slab dimensions, travel direction, concrete grade, pile information, site location, age, fill height, geometry, daily traffic count, speed limit, calculated settlement and two types of ratings for each slab. These specific sites were selected for the following reasons:

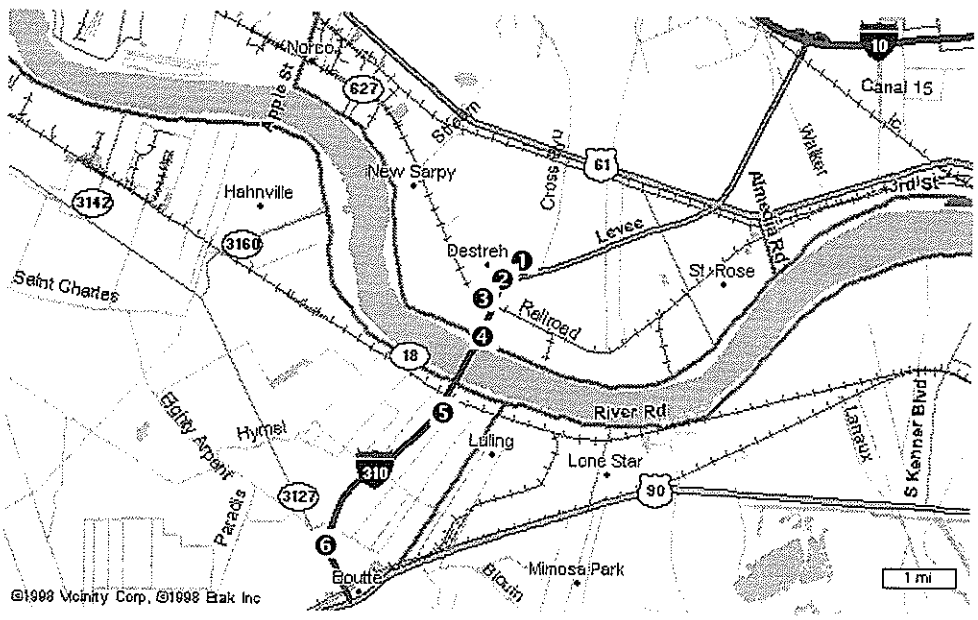
- The sites are relatively close to New Orleans and Baton Rouge which reduces travel time and cost,
- Traffic control is possible for an extended period of time,
- Relatively new bridges with complete records, and
- Difference in performance of the various slabs along I-310.

FIELD TESTING OF REPRESENTATIVE TEST SITES

Various methods were employed in this project to assess the current conditions of the approach slab profile, settlement and contact with soil as well as soil condition at the selected test sites. The deployed methods included: total station survey, walking profiler test, laser profiler test, Dynatest, cone penetration test (CPT) and wash-type soil boring. Table 5 lists the seven different test sites and the specific field tests performed at each site. A brief review of each of the in-situ test methods is presented in the following sections.

TOTAL STATION SURVEY

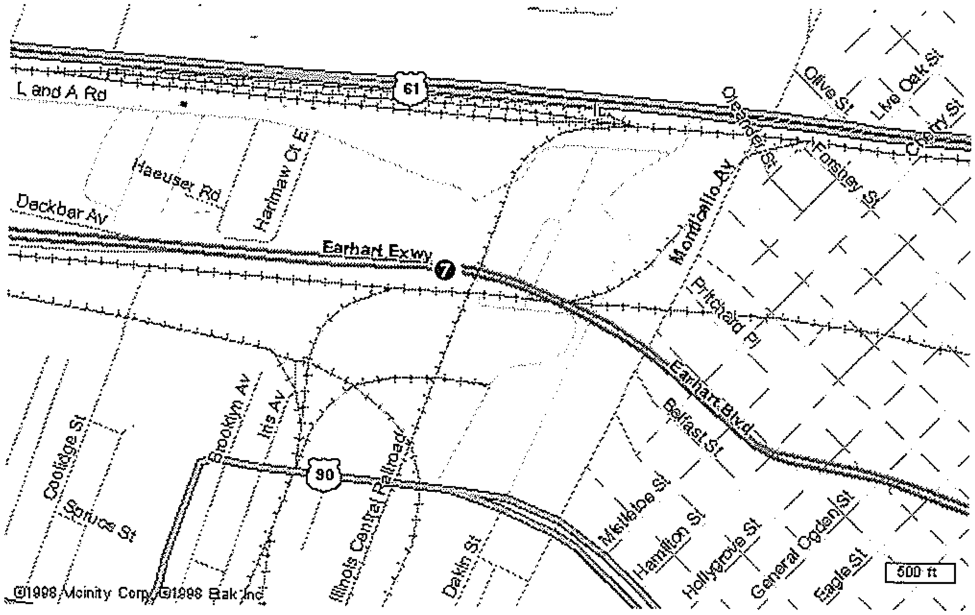
A Nikon DTM-A20LG total station with data logger was used to perform elevation surveys in all seven selected representative sites. The objective of the survey is to obtain the elevations of selected points on the approach slab, adjacent bridge deck and roadway pavement. With this information, the roadway profile along longitudinal lines on the bridge approach slab could be obtained. By comparing the current profile with the original profile obtained from the as-built drawings, the amount of settlement along these longitudinal lines can be calculated. This test was performed by Tulane University personnel. The survey was conducted for all seven test sites. Three longitudinal profiles were done for Sites 1 (on each approach slab), 2, 3, 4 and 6; two profiles for Site 5 (on each approach slab); and four profiles for Site 7.



Legend
❶ I-310 Elevated Structure- South Approach Northbound and Southbound Approach Slabs
❷ I-310 Pipeline Bridge - North Approach Northbound and Southbound Approach Slabs
❸ I-310 Pipeline Bridge - South Approach North bound and Southbound Approach Slabs
❹ I-310 Hale Boggs Bridge - North Approach Northbound and Southbound Approach Slabs
❺ I-310 Hale Boggs Bridge - South Approach Northbound and Southbound Approach Slabs
❻ I-310 Over LA 3127 - South Approach Southbound Approach Slab

Figure 10a

Location map of representative test sites selected along I-310



Legend
❼ Parish line bridge West Approach

Figure 10b
Location map of the representative test site selected on LA 3139

Table 4

Summary of items collected for each test site

	SITE 1		SITE 2	SITE 3	SITE 4	SITE 5		SITE 6	SITE 7
Parish	St. Charles		St. Charles	St. Charles	St. Charles	St. Charles		St. Charles	Jefferson
Year Built	1992		1991	1991	1991	1991		1991	1982
Geometry	Straight		Trapezoid	Trapezoid	Straight	Straight		Curved	Trapezoid
Daily Traffic Count	28230		28230	28230	28230	28230		28230	N/A
Travel Direction	N/B	S/B	S/B	S/B	S/B	N/B	S/B	S/B	W/B
On or Off Ramp	On	Off	On	Off	On	On	Off	Off	Off
Slab Length (ft)	100	100	100	100	100	120	120	80	80
Slab Width (ft-in)	42-10	42-10	42-8	42-8	40-11	38-0	38-0	42-10	35-0
Thickness (in)	10	10	10	10	10	10	10	10	10
Concrete Grade	AA	AA	AA	AA	AA	AA	AA	AA	A
Pile Type	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber
Pile Diameter (in)	12	12	12	12	12	12	12	12	12
Pile Lengths (ft)	15-60	15-60	15-60	15-60	15-60	9-54	9-54	15-60	15-60
Fill Height (ft)	9	9	8	8	10	12	12	8	2.5
Speed Limit (mph)	70	70	70	70	70	70	70	70	50
Maximum Settlement (ft)	0.8	0.8	N/A	N/A	N/A	1.0	1.0	N/A	1.0
Current Condition Rating	6	6	8	8	6	6	6	8	6
Highest IRI	16	13	6	9	10	20	13	4	16

N/B = northbound S/B = southbound

Metric Equivalents:

1 ft = 0.3048 m

1 in = 25.4 m

1 mph = 1.609 kmph

Table 5

The seven selected sites for field tests

Site No.	Structure No.	Highway	Year Built	Location	Description	Field tests performed*
1	02-4503600412 02-4503600401	I-310	92	NB I-310 to Airline (US 61) SB I-310 from Airline (US 61)	Elevated Structure	CPT,Core Boring, Survey, Dynatest, Walking Profiler test, Laser Profiler test
2	02-4503605981	I-310	91	0.2m North of Luling Bridge	Pipeline Crossing	Survey, Walking Profiler test, Dynatest, Laser Profiler test
3	02-4503605982	I-310	92	0.2m North of Luling Bridge	Pipeline Crossing	Survey, Walking Profiler test, Dynatest, Laser Profiler test
4	02-4503606221	I-310	84	ICG,RR, Ramp E F &H, LA	Southbound, North Approach to Luling Bridge	Survey, Walking Profiler test, Dynatest, Laser Profiler test
5	02-4503800361 02-4503800362	I-310	79	South Approach, Luling Bridge	Northbound and Southbound Approaches	CPT,Core Boring, Survey, Dynatest, Walking Profiler test, Laser Profiler test
6	02-4503802411	I-310	86	1.57m North of US 90, I-310 over LA3127	Southbound, South Approach	Survey, Walking Profiler test, Dynatest, Laser Profiler test
7	02-4300104581	LA 3139	82	4.58 m East of Hickory Ave	Westbound, West Approach	Survey, Dynatest, Walking Profiler test, Core Boring, Laser Profiler test

* CPT: Cone Penetrometer Test
Survey: Eletronic Total Station

WALKING PROFILER TEST

The equipment used for this test is the Australian Road Research Board (ARRB) Walking Profiler (Model APR1) shown in Figure 11a. This is a high quality precision instrument which provides efficient collection and presentation of continuous paved surface information, including distance, profile and grade. Figure 11b shows a walking profiler being used at one of the representative test sites. LTRC was responsible for this field test. Measurements were made along the same established survey lines. This test was performed in all test sites except Site 7.

Figure 11a
ARRB Walking Profiler

Figure 11b
A walking profiler test on I-310

DYNATEST

A testing machine commonly known as a falling-weight deflectometer (FWD) was used for non-destructive testing of pavement sections at the selected representative sites. The FWD is an impulse-type testing device which imparts a transient load upon the pavement surface. Both the duration and magnitude of the force are representative of the load pulse induced by a truck tire moving at moderate speeds. The load is generated by dropping a set of masses from selected heights onto a system of rubber buffers. Both the duration of the load pulse and magnitude of the maximum load can be varied based on the drop height and buffer configuration. The loading mechanism is housed in a trailer pulled by a special vehicle which is also used for power generation and data acquisition. The test is static when measurements are made at a given location and then the vehicle is moved to the next measuring station. These stations were located at the same survey points established earlier on the roadway. Machines that are suitable for highway pavement testing typically can generate maximum load pulses in the range of 1500 lbf (680.4 kg) to 25,000 lbf (11340 kg). The tests were performed at load pulses of approximately 16,750 lbf (7597.8 kg). Figure 12 shows a FWD test at a representative test site. These tests were all performed by LTRC.

Figure 12
A FWD machine working at a test site

CONE PENETROMETER TEST

The Louisiana Electric Cone Penetrometer System (LECOPS) was used to acquire and reduce data for soil investigations.

Figure 13a
General exterior view of Louisiana electric cone penetrometer system (LECOPS)

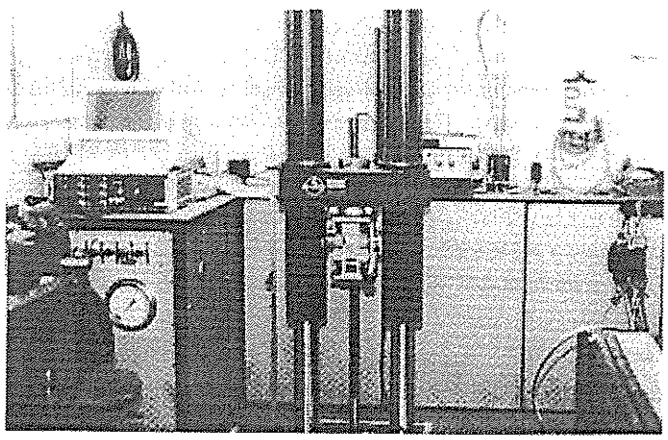


Figure 13b
General interior view of Louisiana electric cone penetrometer system (LECOPS)

The penetrometers used by LECOPS are shown in Figure 13. The tests were performed at the representative sites in the median or off the shoulder of the highway. This test was performed at Sites 1, 5 and 7. Three tests were made in Site 1 and five were made in Site 5.

CONTINUOUS WASH BORING

The borings were made at the representative test sites with a truck mounted drill rig at designated locations, typically in the median or off the soulder of the highway. Using a three-inch diameter thin wall tube sampler. Undisturbed samples were taken out for any cohesive on semi-cohesive materials encountered. Moisture proof containers were used to store representative samples cut from the cores. Afterwards, laboratory testing was performed on some of the retrieved samples.

The Standard Penetration test was used when cohesionless material was encountered. To perform this test, a two inch (50.8 mm) diameter splitspoon sampler is driven in with a 140 lb (63.5 kg) hammer falling 30 inches (762 mm). The density of the material can be estimated from the number of blows it takes to drive the sampler one foot (0.3048 meters) [after first seating it in six inches (152.4 mm)].

The retrieved undisturbed soil samples were mainly used to perform soil mechanics laboratory tests to determine the physical properties of the soils. The tests performed included Natural Moisture Content, Unit Weight and Unconfined Compression. Grain Size (percent passing the No. 200 Sieve) tests were performed on the granular soils and Atterberg Limits were performed on some cohesive samples. Figure 14 shows a core boring sampling truck working at Site 5 on I-310. These tests were performed by Gore Engineering, Inc. of Metairie, Louisiana. Some of the laboratory tests were perfomed by Gore Engineering, Inc. and some others were performed by Tulane University.

Soil borings were taken at Sites 1, 5 and 7. Eighty feet (24.38 m) and twenty five feet (7.62 m) borings were drilled in Site 1 and a 100 feet (30.48 m) boring was drilled in both Site 5 and Site 7.

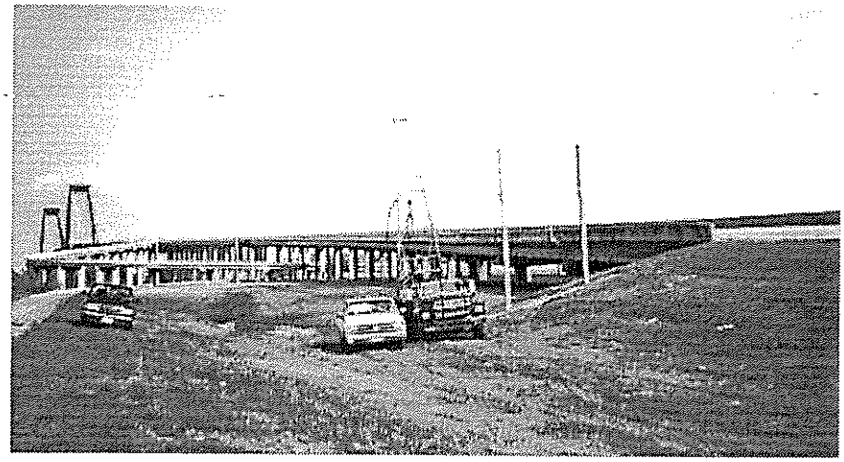


Figure 14
Core boring sampling truck working at site 5 on I-310

SOIL-STRUCTURE INTERACTIION METHOD

A simplified soil/structure interaction method was employed to examine the effects of various parameters on the performance of pile-supported approach slabs. Detailed analysis was performed to examine the effects of the various parameters identified in the selection of representative testing sites, field testing of representative testing sites, and the laboratory testing of soil samples collected on the performance of the pile supported approach slabs. Findings from this analytical and numerical study have resulted in a set of guidelines and recommendation for future design and maintenance of bridge embankment approach system.

DISCUSSION OF RESULTS

FIELD INVESTIGATION OF SELECTED TEST SITES

Table 6 and Table 4 summarize details of the approach slabs studied in more depth (Sites 1-7). Table 6 gives the dimensions of the approach slabs as well as the concrete grade, maximum settlement, pile diameter, and fill height. Due to space limitations, detailed results of Site 1 along with a portion of other sites results are included in this report. Detail results of the other sites are given in Schutt *[22]*.

Table 6

Details of approach slabs studied (Sites 1-7)

Site #	Approach Slab					Treated Timber Pile Butt Diameter (in)	Fill Height (ft)
	Length (ft)	Width (ft-in)	Thickness (in)	Concrete Grade	Maximum Settlement (ft)		
1	100	42-10	10	AA	0.8	12	9
2	100	42-8	10	AA	N/A*	12	8
3	100	42-8	10	AA	N/A*	12	8
4	100	40-11	10	AA	N/A*	12	10
5	120	38	10	AA	1.0	12	12
6	80	42-10	10	AA	N/A*	12	8
7	80	35	10	A	1.0	12	7

*N/A = Not Available

Metric Equivalents:

1 ft = 0.3048 m

1 in = 25.4 mm

SITE 1: I-310 ELEVATED STRUCTURE

Site 1 selected for this project is about one mile away from the north side of Hale Boggs (Luling) bridge. It is the south approach of the elevated bridge structure (see Figure 10a for map location) and includes both the southbound and northbound approaches. Figure 15 shows a view of the elevated bridge structures and the embankment median of Site 1 looking in the north direction. As indicated in Table 4, field tests performed at this site included: survey, profiler test, dynatest, core boring and CPT.

This bridge was constructed in 1992. The information concerning the approach slab, embankment and piles on both the northbound and southbound approaches were obtained from the DOTD District 02 office in New Orleans and is described in the following sections.

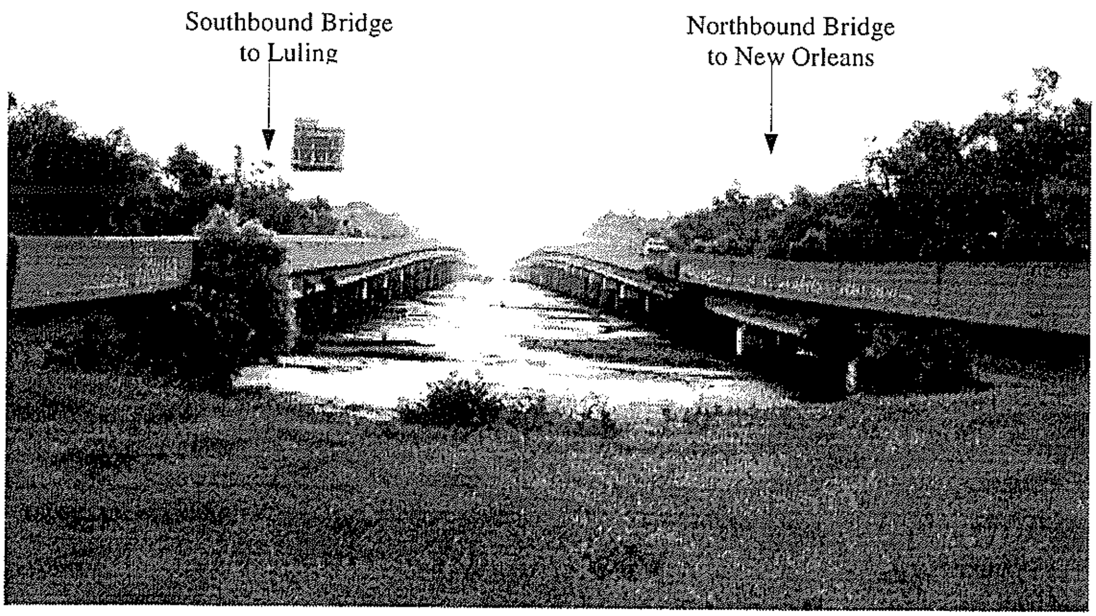


Figure 15
Site 1: I-310 Elevated structure approaches

APPROACH SLAB INFORMATION

Figure 16 shows a plan view of the southbound approach slab at representative Site 1. The design elevations at typical points of the slab surface are listed in Table 7. Figure 17 shows a plan view of the northbound approach slab. The design elevations at specific points along the slab surface are listed in Table 8.

The design thickness of this approach slab is 10 in (254 mm). The approach slab was made of grade AA concrete and reinforced with two layers of grade 60 rebars. Both top and bottom rebar layers consist of 401 bars in the transverse direction and 701 bars in the longitudinal direction. The approach slab is supported by nine rows of timber piles, with each row consisting of seven 12-in (304.8 mm) diameter butt timber piles capped by a 2 ft (0.61 m) wide and 2 ft (0.61 m) deep reinforced concrete beam.

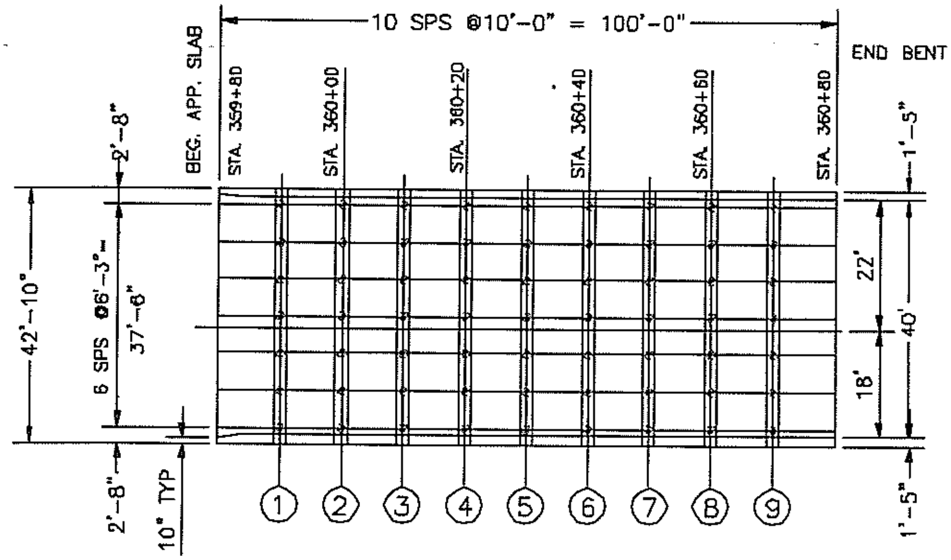


Figure 16
Plan of southbound approach at Site 1

Table 7

Elevations of Site 1 southbound approach slab (mean sea level)

Superelevation Runoff Approach Slab at Bridge						
STATION	ELEV. C ℄ RDWY	RUNOFF (%)	ELEV. M	ELEV. N	ELEV. H	ELEV. J
359+80	9.00	4.49	10.05	10.01	8.17	8.13
360+00	9.01	3.866	9.92	9.86	8.31	8.25
360+20	9.05	3.23	9.81	9.76	8.47	8.42
360+40	9.10	2.59	9.71	9.67	8.63	8.60
360+60	9.17	1.95	9.63	9.60	8.82	8.79
360+80	9.26	1.31	9.57	9.55	9.02	9.01

*Elevations in feet, National Geodetic Vertical Datum (NGVD)

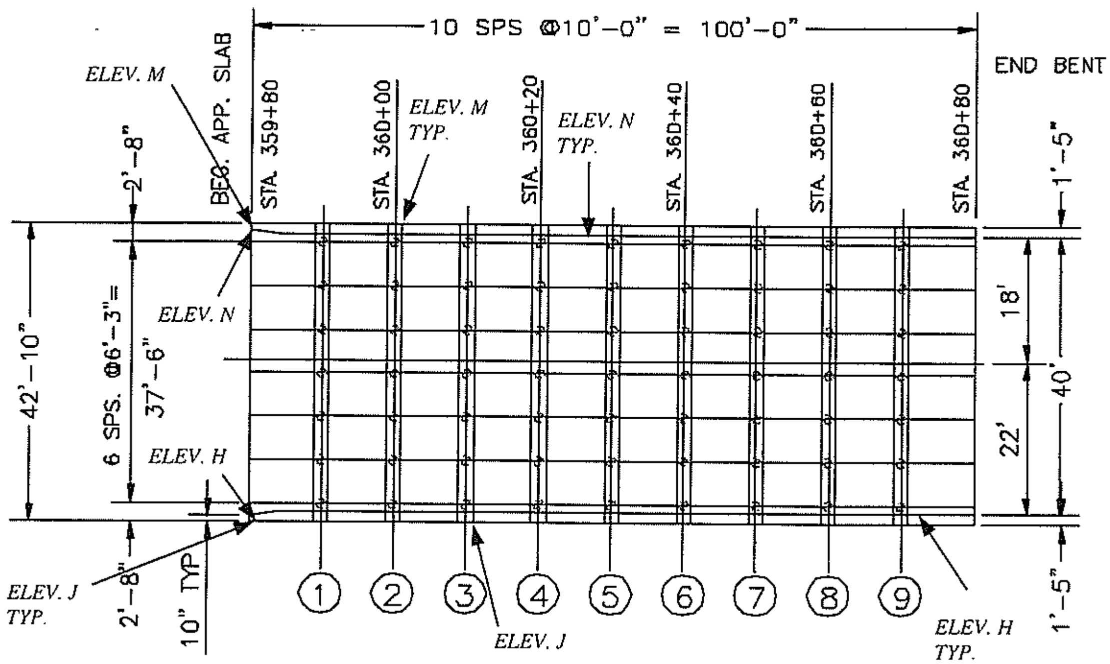


Figure 17
Plan of northbound approach at Site 1

Table 8

Elevations of Site 1 northbound approach slab (mean sea level)

Superelevation Runoff Approach Slab at Bridge						
STATION	ELEV. C ℄ RDWY	RUNOFF (%)	ELEV. M	ELEV. N	ELEV. H	ELEV. J
359+80	9.00	5.03	9.98	9.93	7.86	7.82
360+00	9.01	4.78	9.94	9.87	7.96	7.89
360+20	9.05	4.52	9.93	9.86	8.06	7.99
360+40	9.10	4.27	9.93	9.87	8.16	8.10
360+60	9.17	4.02	9.95	9.89	8.29	8.23
360+80	9.26	3.76	9.99	9.94	8.43	8.38

*Elevations in feet, National Geodetic Vertical Datum (NGVD)

EMBANKMENT INFORMATION

Before construction of the roadway pavement and approach slab, the embankment was surcharged for period of six months. The purpose of surcharging was to minimize the amount of detrimental settlement subsequent to paving service. The surcharge used for this site was about three feet (0.91 m) above the final design profile grade. Figure 18 shows the surcharge profile of grade at station 360+80. The cross section of the final embankment at station 360+80 for both the southbound and northbound approach slabs at representative Site 1 are also shown in Figure 19. Figure 19 shows that the average height of the final embankment is nine feet (2.74 m) above natural ground and is about 208 feet (63.40 m) wide.

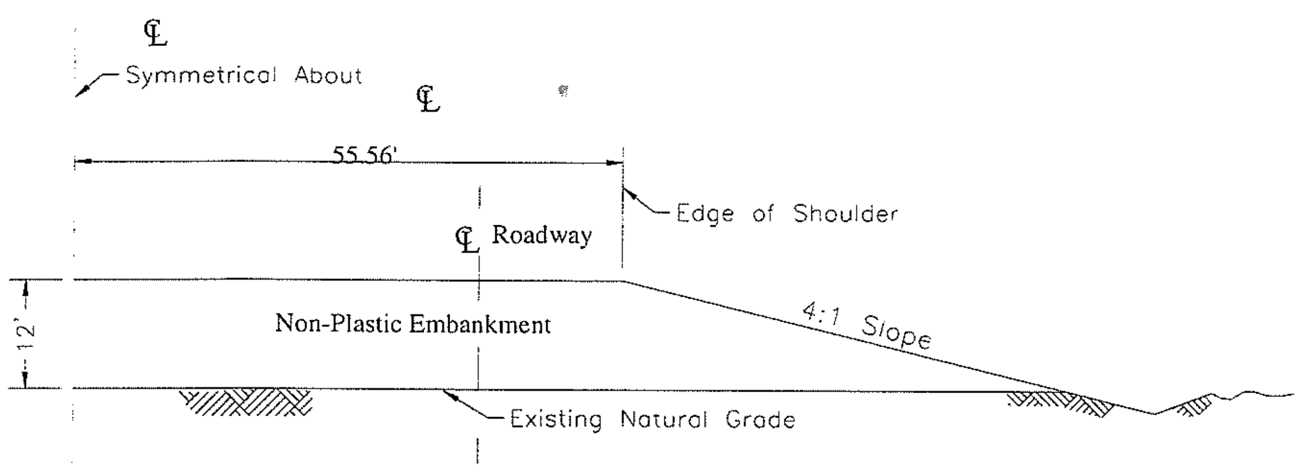


Figure 18
Surcharge profile of grade at station 360+80

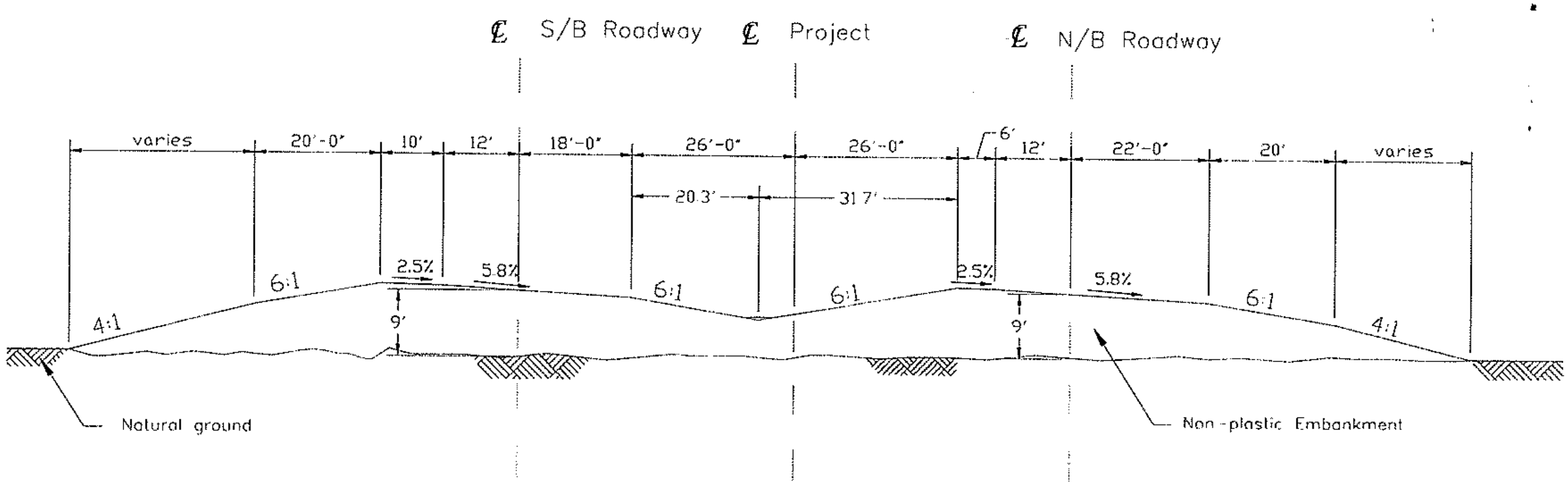


Figure 19
Typical embankment section

SOIL INFORMATION

A continuous soil boring was performed at a designated location at the center of the median for Site 1. The drilling was made at station 360+80. The boring log and test results are shown in Table 10. Table 10 shows that there are about six ft (1.83 m) of sand and then about fifty five feet (16.76 m) of very soft to soft gray clay stratum. Most of the piles in the approach slab extend into these clays. The geologically identified Pleistocene Age soils are reached at about 65 ft (19.81 m). Table 9 shows a summary of the types of soils encountered. The average unconfined compression strength for the thick soft clay stratum is about 500 psf (46.45 psm) and the average compression index (Cc) is 0.5. The water content of this soil is about 55 percent. Substantial organic matter was also found in the shallow depths of this stratum between the depths of ten ft (3.05 m) and twenty five feet (7.62 m). This particular stratification is considered typical for this area where the top sand stratum is part of the fill used to build the roadway embankment. This is underlain by the original near surface soils which were part of the surrounding wetland and swamp area.

Table 9

Types of soils present in Site 1

Predominant soil Type	DEPTH (FT) Below Ground Surface
Sand (embankment fill)	0-6
Soft Clay w/ silt lenses and organics	6-60
Stiff Clay	60-80

Metric Equivalent:

1 ft = 0.3048 m

Table 10

Soil boring at Site 1

Boring No. B-1 **LOG OF BORING AND TEST RESULTS** Date Boring Drilled: 8 December 1997

Project: SOIL BORINGS & LABORATORY TESTS - LOUISIANA HIGHWAY 310 BRIDGE ABUTMENTS - ST. CHARLES PARISH, LOUISIANA
FOR: TULANE UNIVERSITY

Recorded By: Don Tusa

Sample No.	Sample Depth in Feet From	Sample Depth in Feet To	Stratum Depth in Feet	Visual Classification	*Blows per Foot	Symbol Log	Scale (feet)	Unconfined Compression (Qu) (lbs./sq.ft.)	Water Content (percent)	Unit Weight (lbs./cu.ft.) Dry	Unit Weight (lbs./cu.ft.) Wet	Atterberg Limits L.L.	Atterberg Limits P.L.	Atterberg Limits P.I.
			.0				0							
1	.0	.5	1.0	MEDIUM STIFF TAN & GRAY CLAY W/ SILT										
2	1.3	1.8	1.8	MEDIUM STIFF GRAY & TAN CLAY W/ SILT				1875	18.5	92.2	109.3			
3	1.8	2.0		LOOSE TO MEDIUM DENSE TAN SILTY FINE SAND					8.3					
4	2.0	3.0	3.5	W/ MUCH SHELL (PETROLEUM ODOR)	19		5		12.6					
5	3.5	5.0	6.0	DENSE TAN & GRAY SILTY FINE SAND	30 = .8'				20.0					
6	6.0	7.5		SOFT TO MEDIUM STIFF GRAY CLAY	3									
7	9.5	10.0	10.0	W/ TRACE ORGANIC (4" TAR IN SAMPLE)			10	1190	52.3	65.5	99.7	102	32	70
8	11.5	12.0												
9	14.5	15.0		VERY SOFT TO SOFT GRAY CLAY W/ WOOD			15	405	71.9	52.2	89.7			
10	19.5	20.0	20.0				20							
				WOOD W/ SOME CLAY										
			25.0				25							
11	26.0	26.5												
12	29.5	30.0					30	495	50.1	67.8	101.8	58	48	10
13	34.5	35.0					35							
				VERY SOFT TO SOFT GRAY CLAY W/ SILT LENSES										
14	39.5	40.0					40	555	51.3	68.3	103.3			
				(W/ SHELL FRAGMENTS @ 39.5'-40.0')										
15	44.5	45.0					45							
16	49.5	50.0					50	785	46.8	69.6	102.2			
17	53.5	54.0	54.5											
18	54.5	55.0	55.0	LOOSE GRAY SILTY FINE SAND W/ TRACE ORGANIC			55		32.1					
19	55.5	56.0		SOFT GRAY CLAY				920	48.4	70.3	104.3	63	23	40
20	59.5	60.0	62.5	W/ SHELL FRAGMENTS			60							
21	62.5	63.0	64.5	SOFT GRAY SANDY CLAY W/ SHELL FRAGMENTS				550	28.9	86.8	111.9	34	21	13
22	64.5	65.0	67.5	MEDIUM STIFF GREENISH GRAY SILTY CLAY			65	1520	22.9	100.6	123.6	36	14	22
23	69.5	70.0	73.0	VERY STIFF GREENISH GRAY & REDDISH TAN CLAY W/ SILT			70	4520	21.1	104.3	126.3			
24	74.5	75.0	76.0	VERY STIFF REDDISH TAN & LIGHT GRAY CLAY W/ SAND LAYERS			75	3100	32.4	85.0	112.5	67	26	41
25	76.0	76.5	79.0	MEDIUM STIFF LIGHT GRAY & REDDISH TAN SILTY CLAY				1120	27.5	92.2	117.5			
26	79.5	80.0	80.0	LOOSE LIGHT GRAY & REDDISH TAN SILTY FINE SAND			80		22.1					
							85							
							90							
							95							
							100							

CLAY SILT SAND ORGANIC
Predominant Type Bold. Modifying Type Light.

*140 lb. hammer dropped 30 inches on 2 inch splitspoon sampler after first being seated 6 inches

REMARKS: Water Table Depth = 2.8 ft (See Text)
Free Water Depth = 5.0 ft (See Text)

TEST RESULTS

Specific survey points were marked on each approach slab, on the adjacent roadway and on the bridge deck. These points were established at a constant pitch and were used to identify the locations of the survey, Dynatest and walking profiler measurements. The location of these data points is shown in Figure 21 for the southbound approach slab and in Figure 22 for the northbound approach slab.

VISUAL INSPECTION

Based on a visual inspection, it appears that the approach slab at Site 1 has performed poorly. There is significant differential settlement along the approach slab. At the joint between the bridge and the approach slab there is also noticeable differential settlement. A view of the northbound slab and abutment is shown in Figure 20.

Figure 20
View of northbound bridge of Site 1

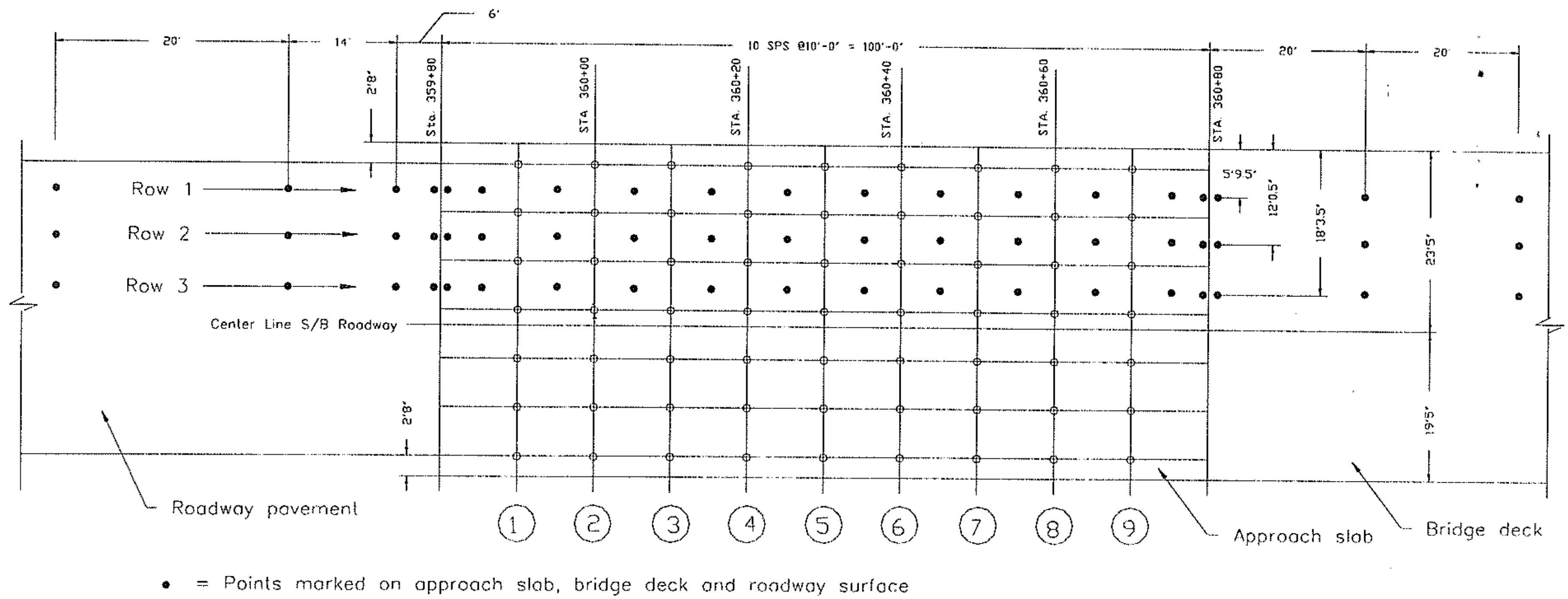


Figure 21
Data points marked to perform survey, profiler and Dynatest at southbound approach slab of Site 1

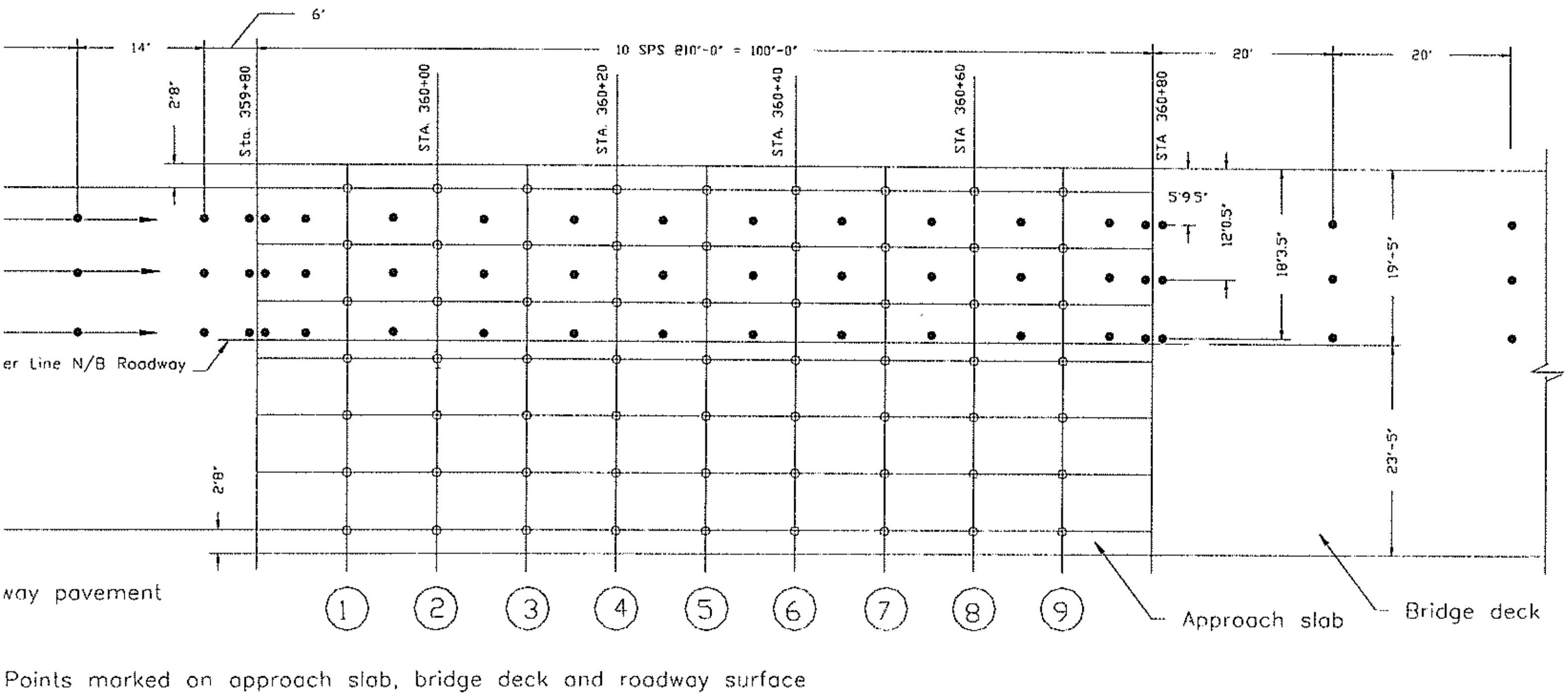


Figure 22

ta points chosen to perform survey, profiler and Dynatest at northbound approach slab of Site 1

SURVEY RESULTS

The relative elevations of all of the points shown in Figures 21 and 22 were determined by an electronic total station. Thus the profile along each longitudinal row of the three rows surveyed was developed. Because the bridge abutment settlement could be considered insignificant in comparison with the settlement of the bridge approach, it was assumed that the bridge abutment settlement was zero. Based on this assumption and utilizing the original elevations of the specific points along the approach slab listed in Tables 7 and 8, the settlement along each longitudinal row of the three rows could be determined by interpolation. Figures 23 and 24 show the approach slab settlement along each longitudinal row along the northbound and southbound roadways, respectively.

Figure 23 shows that the maximum settlement of the northbound approach slab at Site 1 is around 0.7 ft (0.21 m), and Figure 24 shows that the maximum settlement of the southbound approach slab at this test site is about 0.8 ft (0.24 m). Both measurements were recorded near the roadway/approach slab interface joint. At a distance of 60 ft (18.29 m) away from bridge/approach slab interface, the northbound approach is currently at the same elevation as the edge of the approach slab/roadway interface. Therefore, nearly hundred percent of the differential settlement between the bridge abutment and the roadway has occurred in the first 60 ft (18.29 m) segment of the 100 ft (30.48 m) long northbound approach slab. Hence, it can be concluded that the approach slab at this test site did not perform adequately as a sudden bump would be felt by the driver at the end of the bridge.

The settlement profile of the southbound approach slab at this test site is less severe than the settlement profile of the northbound approach slab. The entire length of the southbound approach slab was utilized to gradually distribute the settlement between the bridge abutment and the roadway pavement.

By examining the data shown in Table 6 for the southbound approach slab along the right edge in the south direction, the elevation (Elev. M) at the approach slab/roadway edge is 0.5 ft (0.15 m) higher than the elevation at the approach slab/bridge edge. But for the northbound approach slab along the right edge in the south direction (Table 7), the elevation (Elev. M) at the approach slab/roadway edge is nearly the same as the elevation at the approach slab/bridge edge. The difference in settlement profiles for the southbound and northbound approach slabs could be attributed to this variation.

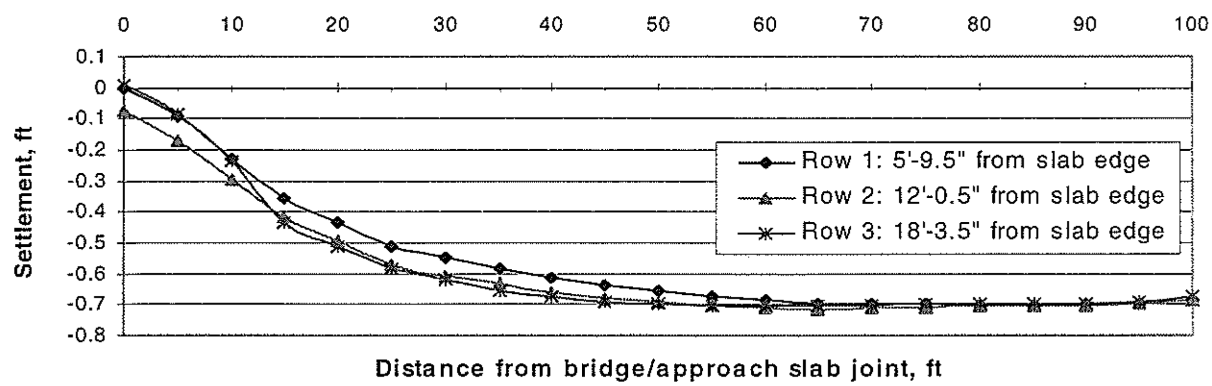


Figure 23
Northbound approach slab settlement for Site 1

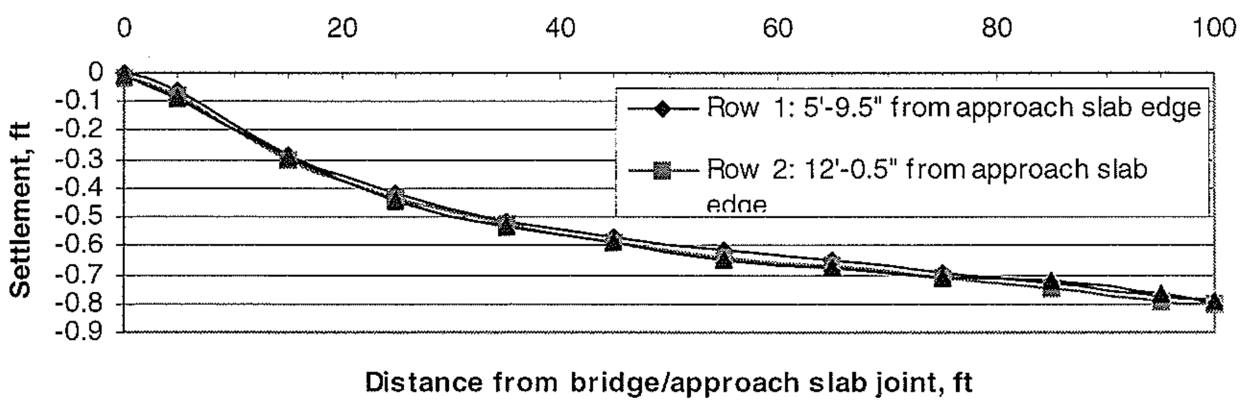


Figure 24
Southbound approach slab settlement for Site 1

PROFILER AND DYNATEST RESULTS

The walking profiler and the Dynatest tests were performed on both the northbound and southbound approach slabs of Site 1. The results of these tests were compared with those of the survey for both the longitudinal and transverse directions. A sample of the results of the tests for the northbound and southbound approach slabs is shown in Figures 25 and 26, respectively. A complete set of graphs for the different locations are available in the Tulane University Civil and Environmental Engineering Department.

As shown by the graphs, the data obtained from the walking profiler and the survey are in good agreement. This shows that the walking profiler yields the necessary data for evaluating the performance of approach slabs. The graphs also show that the approach slab is bent at a distance of 5 ft (1.52 m) to 10 ft (3.05 m) from the abutment. Towards the roadway, the approach slab is relatively flat, but near the bridge, there is an abrupt change in slope due to excessive settlement.

The Dynatest measured deflection is higher towards the two ends of the approach slab. Since the roadway is relatively soft in comparison with the reinforced concrete approach slab, higher deflection is expected on the roadway. A considerable deflection was also observed at the roadway end of the approach slab. This is probably due to the fact that the end of the approach slab is directly ground-supported while the remainder of the approach slab is supported by piles in addition to the ground. Towards the abutment, however, relatively high deflection is displayed. This is probably due to the loss of soil support under the approach slab due to erosion, and/or settlement.

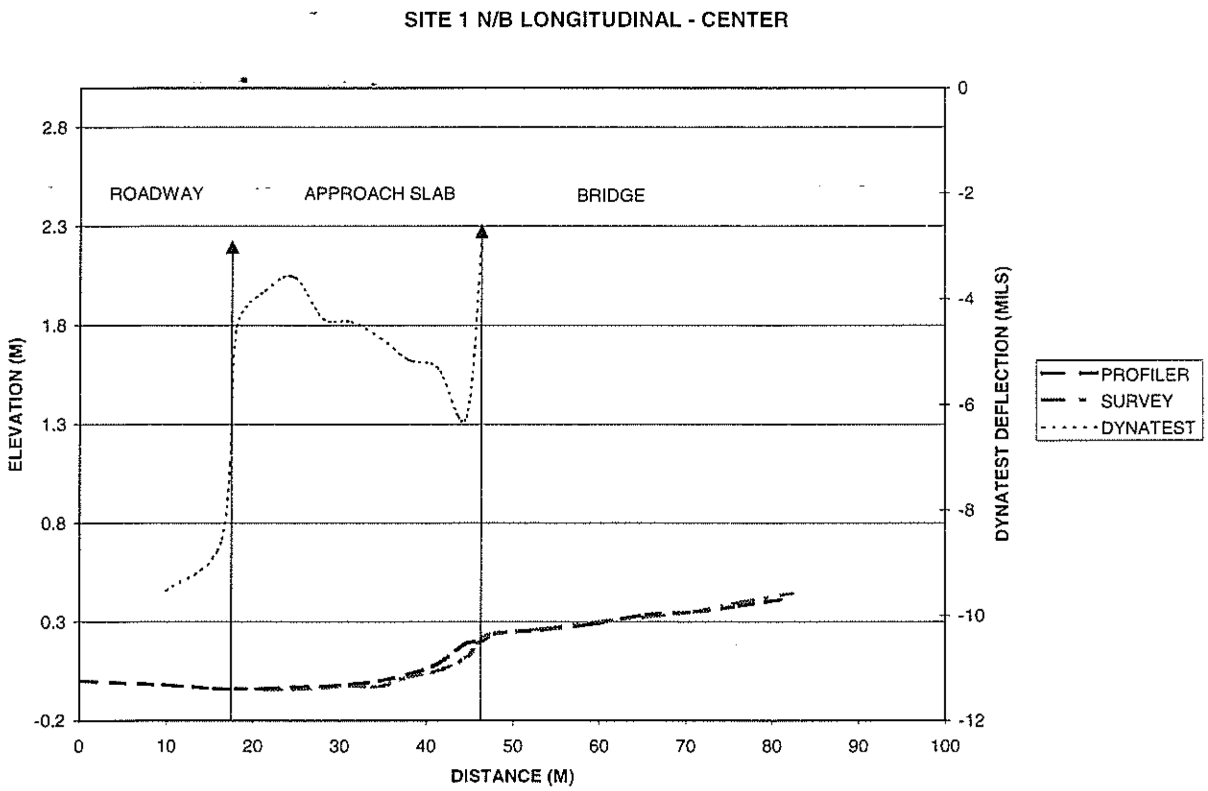


Figure 25
Sample graph of walking profiler, Dynatest, and survey for Site 1 northbound

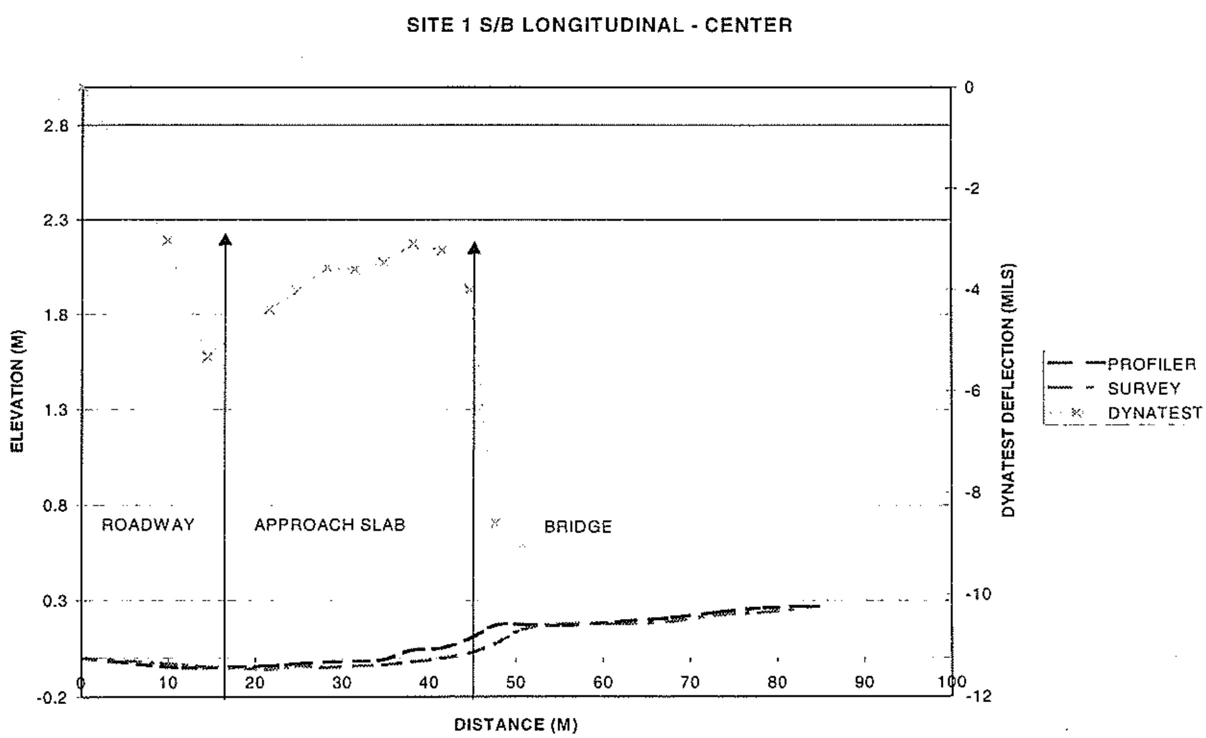


Figure 26
Sample graph of walking profiler, Dynatest and survey for Site 1 southbound

SOIL BORINGS

Soil borings were drilled at Site 1 and some of the obtained parameters were compared with the original borings made at the time of construction. The comparison is shown in Table 11. The unconfined compression values determined from the samples retrieved from the new borings were somewhat better in the top thirty five feet (10.67 m). At deeper depths they were closer to the original values. This indicates that some consolidation has occurred in the upper layers due to the effect of construction. Water content values were generally higher in the original borings, but the unit weights were close for the two conditions.

CONE PENETROMETER TEST (CPT)

LTRC performed cone penetrometer tests (CPT) on the median of Site 1. The readings of the CPT were compared with the results of the soil borings. As shown in Table 13, the CPT results were generally in good agreement with the results of the soil borings. There are slight variations in the layers, but this could be due to the fact that the CPT was not performed at exactly the same locations of the soil borings as well as human errors in the tests on their interpretation. The detailed graphs of the CPT data are available in the reference *[22]*.

Table 11

Comparison of soil properties between current soil boring and original soil boring for Site 1

Depth (ft)		Unconfined Compression (psf)		Water Content (%)		Wet Unit Weight (pcf)		Atterberg Limits			
								L. L.		P. I.	
From	To	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring
0	1										
1	2	1875		18.5		109.3					
2	3			12.6				GWT			
3	4			20							
4	5			20							
5	6										
6	7		Original	Ground	Surface						
7	8										
8	9										
9	10	1190		52.3		99.7		102		70	
10	11		760		90		98		100		57
11	12										
12	13										
13	14										
14	15	405	340	71.9	69	89.7	108		95		66
15	16										
16	17										
17	18										
18	19										
19	20		100		60		106		36		13
20	21										
21	22										
22	23						122				
23	24										
24	25										
25	26		280		48		110		48		31
26	27										
27	28										
28	29										
29	30	495		50.1		101.8		58		10	
30	31		580		52		102				
31	32										
32	33										
33	34										
34	35										
35	36										

Table 11

Comparison of soil properties between current soil boring and original soil boring for Site 1 (continued)

Depth (ft)		Unconfined Compression (psf)		Water Content (%)		Wet Unit Weight (pcf)		Atterberg Limits			
								L. L.		P. I.	
From	To	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring
36	37										
37	38										
38	39		780		63		106				
39	40	555		51.3		103.3					
40	41		920		55		104		69		38
41	42										
42	43										
43	44										
44	45										
45	46		960		65		102				
46	47										
47	48										
48	49										
49	50	785		46.8		102.2					
50	51		520		62		102		105		79
51	52										
52	53										
53	54										
54	55			32.1			112				
55	56	920		48.4		104.3		63		40	
56	57										
57	58										
58	59										
59	60		900		77		98		124		87
60	61										
61	62		840		36		120		54		35
62	63	550		28.9		111.9		34		13	
63	64										
64	65	1520		22.9		123.6		36		22	
65	66		6080				132		26		10
66	67										
67	68										
68	69										
69	70	4520		21.1		126.3					
70	71										
71	72										

Table 11

Comparison of soil properties between current core boring and original core boring for Site 1(continued)

Depth (ft)		Unconfined Compression (psf)		Water Content (%)		Wet Unit Weight (pcf)		Atterberg Limits			
								L. L.		P. I.	
From	To	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring	1997 Boring	1983 Boring
72	73										
73	74										
74	75	3100		32.4		112.5		67		41	
75	76		1200				122		26		9
76	77	1120		27.5		117.5					
77	78										
78	79										
79	80			22.1							

LABORATORY CONSOLIDATION TESTS

Consolidation tests were performed on selected undisturbed soil samples obtained from Site 1. The selected samples were numbers 8, 13 and 17 obtained from the soil borings at the 12 ft (3.66 m), 35 ft (10.67 m) and 54 ft (16.46 m) depths, respectively (refer to Figure 20). Results of the consolidation tests are shown in Table 12. The layer closest to the top of the soil boring has a higher over consolidation ratio and low permeability. This is probably due to the effect of the weight of the embankment fill placed at the time of construction. The permeability of the deepest layer is also low. The reason for this may be that this sample is the older of these deeper soils.

Table 12

Consolidation results for Site 1

Sample No.	Depth of Sample (ft)	Original Void Ratio (e)	Compression Index (C_c)	Recompression Index (C_r)	Over Consolidation Ratio (OCR)	Average Permeability (k) m/s
8	12	0.96	0.417	0.115	4.03	8.4×10^{-6}
13	35	1.88	0.443	0.106	1.40	2.4×10^{-5}
17	54	2.14	0.681	0.218	1.13	4.53×10^{-6}

Metric Equivalent:

1 ft = 0.3048 m

Table 13

Comparison of soil properties obtained from current boring and CPT at Site 1

<table>
<tr><th colspan="2">Depth(ft)</th><th colspan="2">Soil Classification</th></tr>
<tr><th>From</th><th>To</th><th>1997 Boring</th><th>CPT</th></tr>
<tr><td>0</td><td>1</td><td rowspan="2">Medium Stiff Tan & Gray Clay w/Silt</td><td rowspan="2">Very Soft Inorganic Clay</td></tr>
<tr><td>1</td><td>2</td></tr>
<tr><td>2</td><td>3</td><td rowspan="2">Loose to Medium Dense Tan Silty Fine Sand w/Much Shell (Petroleum Odor)</td><td>Sandy Clay</td></tr>
<tr><td>3</td><td>4</td><td rowspan="5">Dense or Cemented Sand</td></tr>
<tr><td>4</td><td>5</td><td rowspan="2">Dense Tan & Gray Silty Fine Sand</td></tr>
<tr><td>5</td><td>6</td></tr>
<tr><td>6</td><td>7</td><td rowspan="4">Soft to Medium Stiff Gray Clay w/Trace Organic (4" Tar in Sample)</td></tr>
<tr><td>7</td><td>8</td></tr>
<tr><td>8</td><td>9</td><td>Sandy Clay</td></tr>
<tr><td>9</td><td>10</td><td>Organic Clay</td></tr>
<tr><td>10</td><td>11</td><td rowspan="10">Very Soft to Soft Gray Clay w/Wood</td><td>Sandy Clay</td></tr>
<tr><td>11</td><td>12</td><td rowspan="6">Medium Inorganic Clay</td></tr>
<tr><td>12</td><td>13</td></tr>
<tr><td>13</td><td>14</td></tr>
<tr><td>14</td><td>15</td></tr>
<tr><td>15</td><td>16</td></tr>
<tr><td>16</td><td>17</td></tr>
<tr><td>17</td><td>18</td><td>Sandy clay</td></tr>
<tr><td>18</td><td>19</td><td rowspan="5">Soft Inorganic Clay</td></tr>
<tr><td>19</td><td>20</td></tr>
<tr><td>20</td><td>21</td><td rowspan="5">Wood w/Some Clay</td></tr>
<tr><td>21</td><td>22</td></tr>
<tr><td>22</td><td>23</td></tr>
<tr><td>23</td><td>24</td><td>Sandy clay</td></tr>
<tr><td>24</td><td>25</td><td>Sand</td></tr>
<tr><td>25</td><td>26</td><td rowspan="19">Very Soft to Soft Gray Clay w/Silt Lenses (w/Shell Fragments @ 39.5'-40.0')</td><td>Clayey Sand</td></tr>
<tr><td>26</td><td>27</td><td>Medium Inorganic Clay</td></tr>
<tr><td>27</td><td>28</td><td rowspan="2">Sandy clay</td></tr>
<tr><td>28</td><td>29</td></tr>
<tr><td>29</td><td>30</td><td>Sand</td></tr>
<tr><td>30</td><td>31</td><td rowspan="14">Sandy clay</td></tr>
<tr><td>31</td><td>32</td></tr>
<tr><td>32</td><td>33</td></tr>
<tr><td>33</td><td>34</td></tr>
<tr><td>34</td><td>35</td></tr>
<tr><td>35</td><td>36</td></tr>
<tr><td>36</td><td>37</td></tr>
<tr><td>37</td><td>38</td></tr>
<tr><td>38</td><td>39</td></tr>
<tr><td>39</td><td>40</td></tr>
<tr><td>40</td><td>41</td></tr>
<tr><td>41</td><td>42</td></tr>
<tr><td>42</td><td>43</td></tr>
<tr><td>43</td><td>44</td></tr>
</table>

Table 13

Comparison of soil properties obtained from current boring and CPT at Site 1 (continued)

44	45	Very Soft to Soft Gray Clay w/Silt Lenses (w/Shell Fragments @ 39.5'-40.0')	Sandy Clay
45	46		
46	47		
47	48		
48	49		
49	50		
50	51		
51	52		
52	53		Sand
53	54		
54	55	Loose Gray Silty Fine Sand w/Trace Organic	
55	56	Soft Gray Clay w/Shell Fragments	Sandy Clay
56	57		
57	58		
58	59		
59	60		
60	61		
61	62		
62	63		
63	64	Soft Gray Sandy Clay w/Shell Fragments	
64	65		
65	66	Medium Stiff Greenish Gray Silty Clay	
66	67		
67	68		
68	69	Very Stiff Greenish Gray & Reddish Tan Clay w/Silt	
69	70		
70	71		
71	72		Clayey Sands and Silts
72	73		Stiff Inorganic Clay
73	74	Very Stiff Reddish Tan & Light Gray Clay w/Sand Layers	Sandy clay
74	75		
75	76		Stiff Inorganic Clay
76	77	Medium Stiff Light Gray Reddish Tan Silty Clay	Sandy clay
77	78		Clayey Sands and Silts
78	79		
79	80	Loose Light Gray & Reddish Tan Silty Fine Sand	

INTERNATIONAL ROUGHNESS INDICES (IRIs)

The IRIs for Site 1 were calculated using a laser profiler. The IRIs for the northbound and southbound approach slabs were graphed and are shown in Figures 27 and 28, respectively. The IRIs are significantly higher on the approach slabs, especially at their ends. This is due to the bad condition of the approach slab in comparison to the condition of the bridge and roadway which indicates that there is a riding problem at the approach slab. The recognized standard rating for pavement using the International Roughness Index (IRI) is shown in the previous section in Table 3. The graphs show that the approach slabs have a rating of poor to very poor.

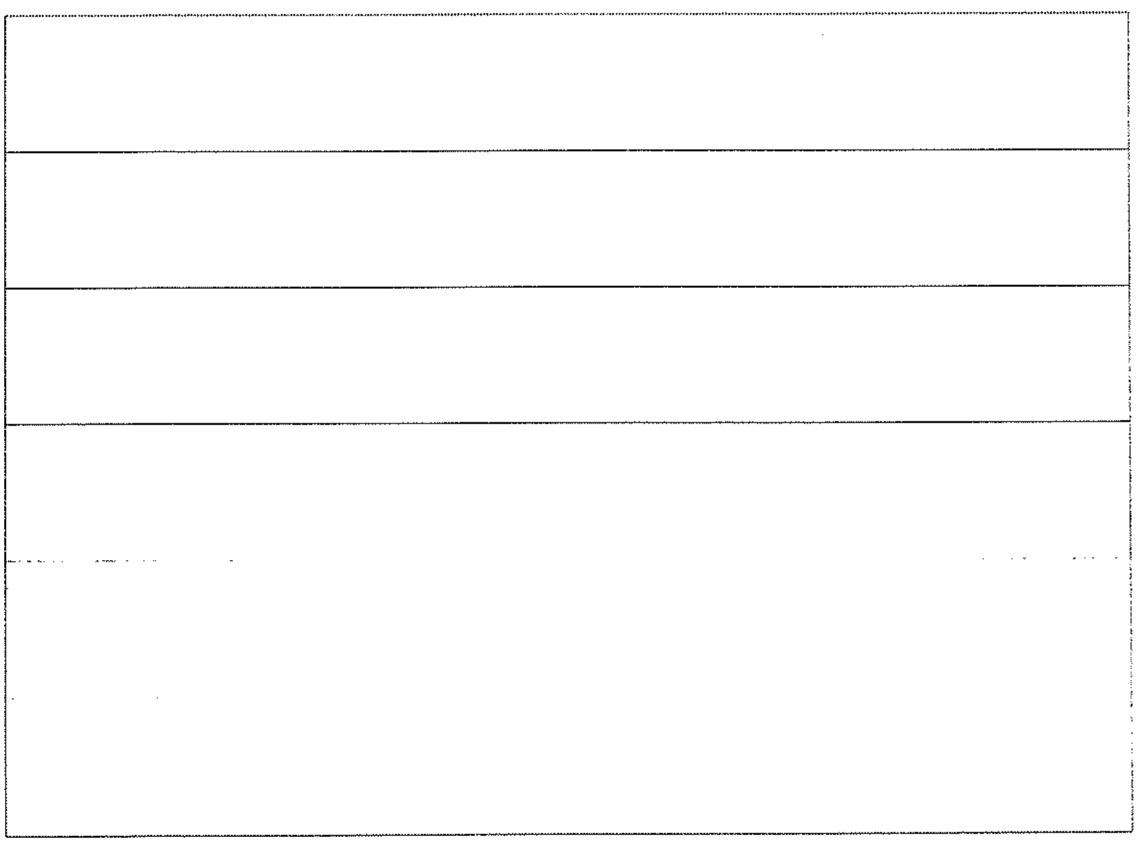


Figure 27
IRIs for Site 1 northbound approach slab

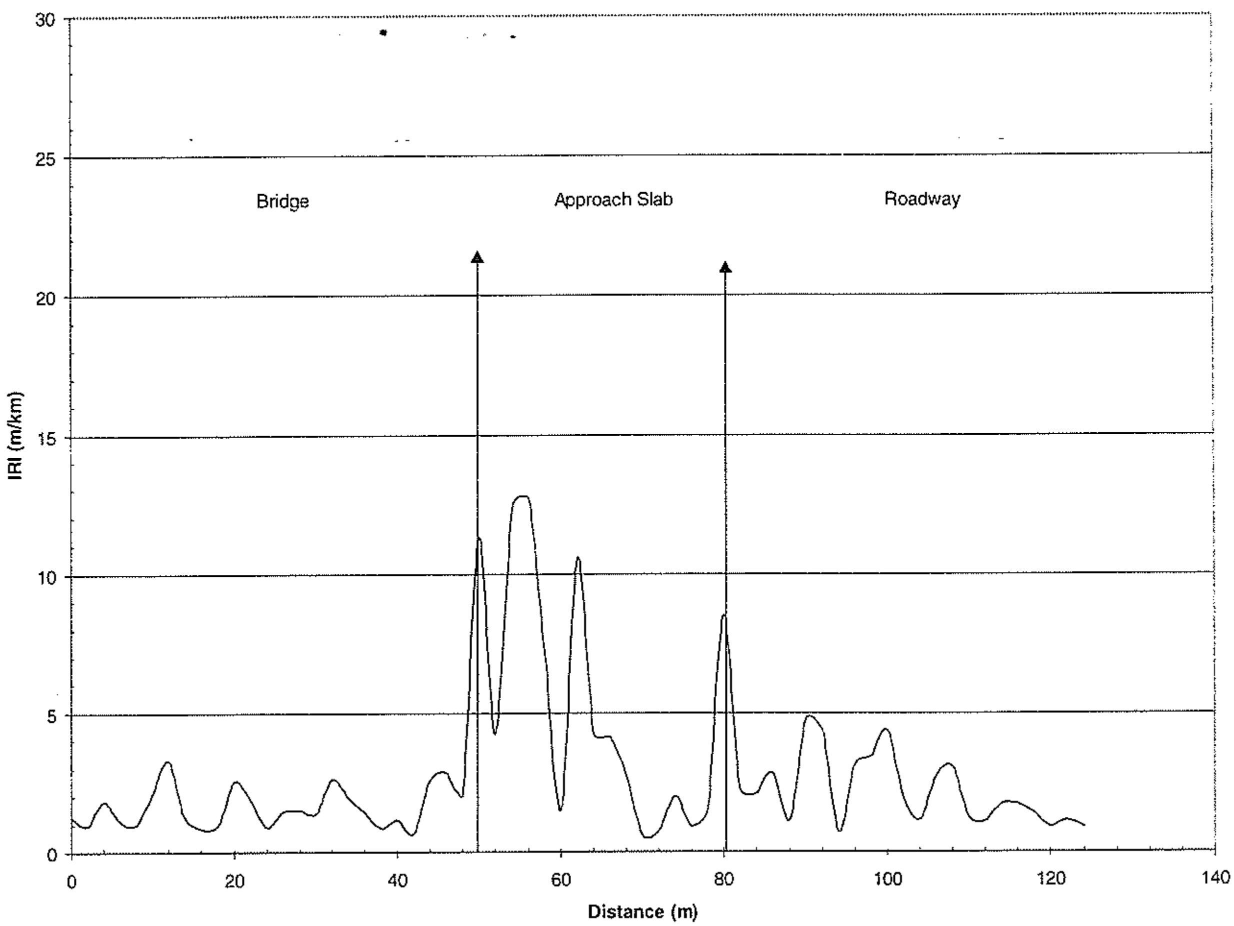


Figure 28
IRIs for Site 1 southbound approach slab

SITE 2 AND 3: I-310 PIPELINE BRIDGE

Sites 2 and 3 considered in this project are about half a mile south of Site 1. Site 2 is the southbound north approach of the pipeline bridge on I-310 while Site 3 is the southbound south approach of the same bridge (see Fig. 10a for the location map). These approach slabs were constructed in 1991 in accordance with the 1982 DOTD office of Highway Standard Specifications for Roads and Bridges and the 1983 AASHTO Standard Specifications for Highway Bridges. Only walking profiler, laser profiler, Dynatest and survey measurement tests were performed at Site 2 and Site 3.

TESTS RESULTS

The approach slabs at Site 2 and Site 3 have performed very well. There is no significant differential settlement at the joints between the bridge abutment and the approach slab, or between the approach slab and the roadway at both sites.

SURVEY RESULTS

The relative elevations of the points marked on slabs were determined with an electronic total station. Due to the fact that traffic was not blocked for an extended period of time, the number of data points was less than in the previous site. The differential settlements at the bridge/slab and slab/roadway joints are evaluated by comparing two corresponding points along each row at each joint (see Table 14). When the difference is positive it means that the approach slab is higher than the bridge or roadway. It can be seen from Table 14 that the differential settlements are not significant at both the approach slab/roadway joint and the bridge/approach slab joint in Site 2 and Site 3. These small differential settlements indicate that the approach slabs at both sites have performed relatively well.

Based on the geodetic survey of Sites 2 and 3, the approach slabs are about 0.1 in (2.54 mm) lower than the roadway at their interface joint. Meanwhile, the approach slab in Site 2 is lower by about 0.188 in (4.78 mm) than the bridge at their interface joint. On the other hand, the approach slab in Site 3 is higher by 0.248 in (6.30 mm) than the bridge at their interface joint.

Figures 29 and 30 show the average height differences between the approach slab and the roadway and bridge.

Table 14

Differential settlement of points at joints for Site 2 and Site 3

	Site 2		Site 3	
	Slab/roadway joint (in.)	Bridge/slab joint (in.)	Slab/roadway joint (in.)	Bridge/slab joint (in.)
Row 1 *	-0.228	-0.192	-0.156	-0.12
Row 2	-0.012	-0.156	-0.324	0.768
Row 3	-0.06	-0.216	0.18	0.096
Average	-0.1	-0.188	-0.1	0.248

* Row 1: Located along the outer edge of the outside lane.
Row 2: Located along the center of the outside lane.
Row 3: Located along the inside edge of the outside lane.

Metric Equivalent:
1 in = 25.4 mm

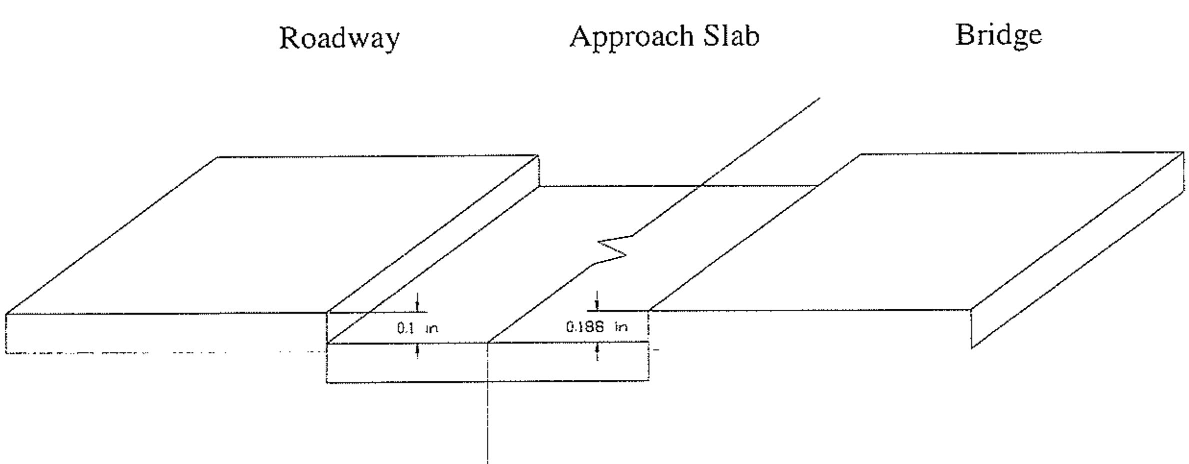


Figure 29
Average height difference between the approach slab and the roadway and bridge joints for Site 2

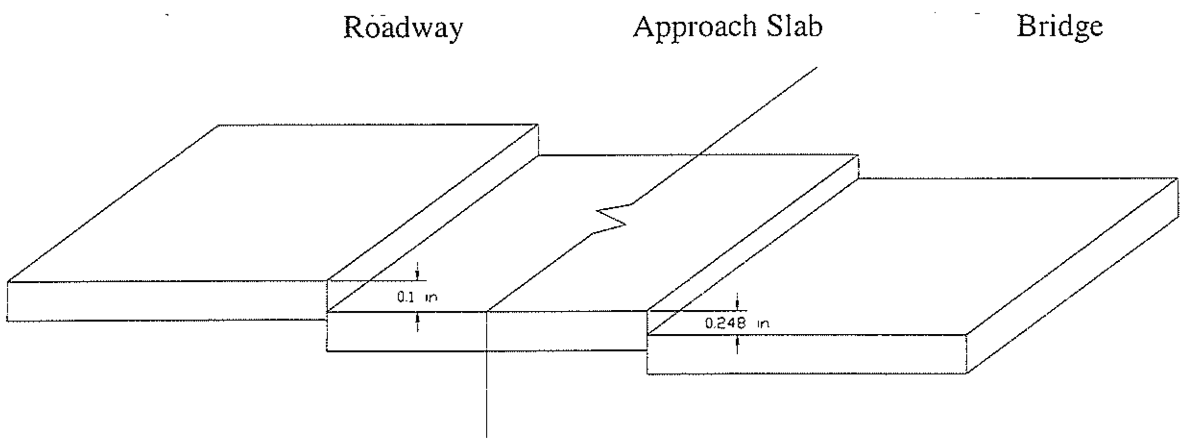


Figure 30
Average height difference between the approach slab and the roadway and bridge joints for Site 3

IRIs

The IRIs for Sites 2 and 3 were calculated based on the readings of a laser profiler. The IRI graphs are shown in Figures 31 and 32. The IRIs are significantly higher at both ends of the approach slabs, which shows that the ridability at the end of the approach slab is not as good. This is probably due to the presence of the construction joints at these locations which would impact the IRI. as is the case in Site1. However, the high value is limited to the joints and is not followed by a noticeable bump.

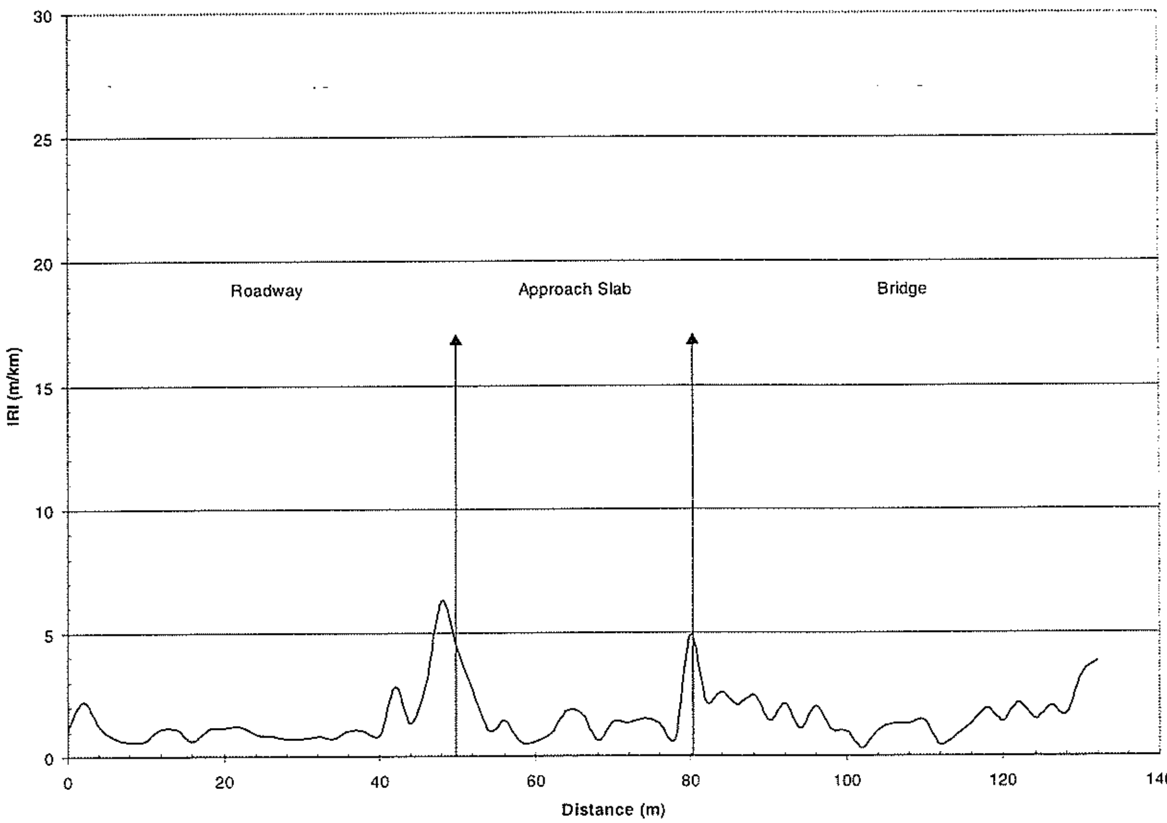


Figure 31
IRIs for Site 2

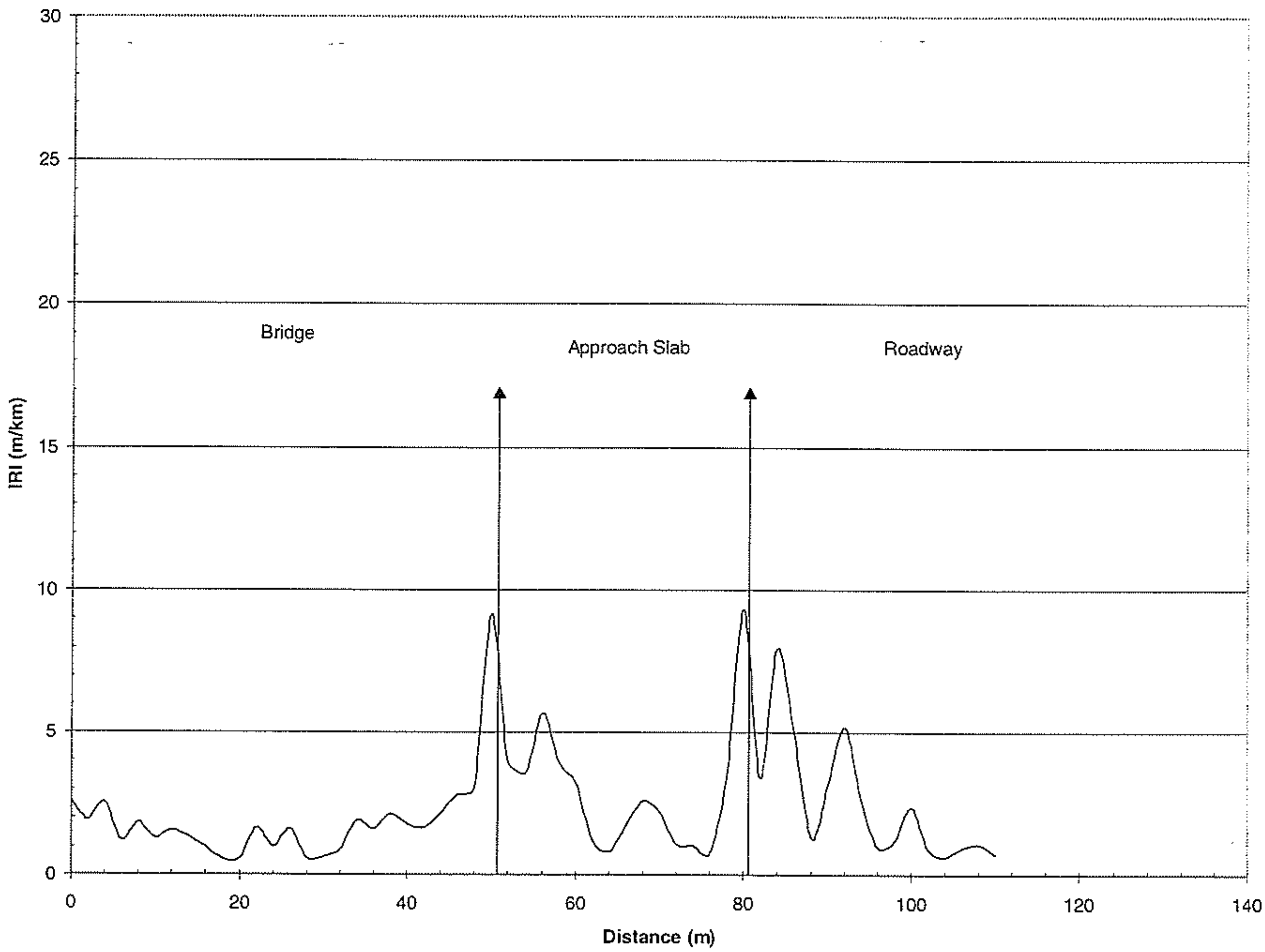


Figure 32
IRIs for Site 3

SITE 4: I-310 HALE BOGGS BRIDGE NORTH APPROACH

Site 4 is the north approach of the Hale Boggs (Luling) Bridge and includes southbound approach (see Fig. 10a for the location map). The approach slab was constructed in 1991 in accordance with the 1982 DOTD office of Highway Standard Specifications for Roads and Bridges and the 1983 AASHTO Standard Specifications for Highway Bridges. Only the profile, Dynatest and geodetic survey tests were performed at this site.

TESTS RESULTS

The approach slab at Site 4 has generally performed well. There is no obvious differential settlement at the joint between the bridge abutment and the approach slab or between the approach slab and the roadway.

SURVEY RESULTS

The differential settlements in the bridge/slab and slab/roadway interfaces were evaluated by comparing two corresponding points along each row at each joint. Positive differences indicate that the approach slab is generally higher than the bridge or roadway. From Table 15, the differential settlement at the bridge/approach slab interface is insignificant. For the approach slab/roadway joint, however, the differential settlement along Row 2 was about 2 in (50.8 mm) which is much more than was observed in Sites 2 and 3. It seems as if the roadway settled towards the center of the lane. This may be due to loss of soil support under the roadway due to organic decay and/or settlement. Figures 33 shows the average height differences between the approach slab and the roadway and bridge.

Table 15

Differential settlement of points at joints for Site 4

	Site 4	
	Slab/roadway joint differential settlement (in.)	Bridge/slab joint differential settlement (in.)
Row 1 *	0.672	-0.756
Row 2	2.016	0.696
Row 3	0.672	-0.036
Average	1.12	-0.032

* Row 1: Located along the outer edge of the outside lane.
Row 2: Located along the center of the outside lane.
Row 3: Located along the inside edge of the outside lane.

Metric Equivalent:
1 in = 25.4 mm

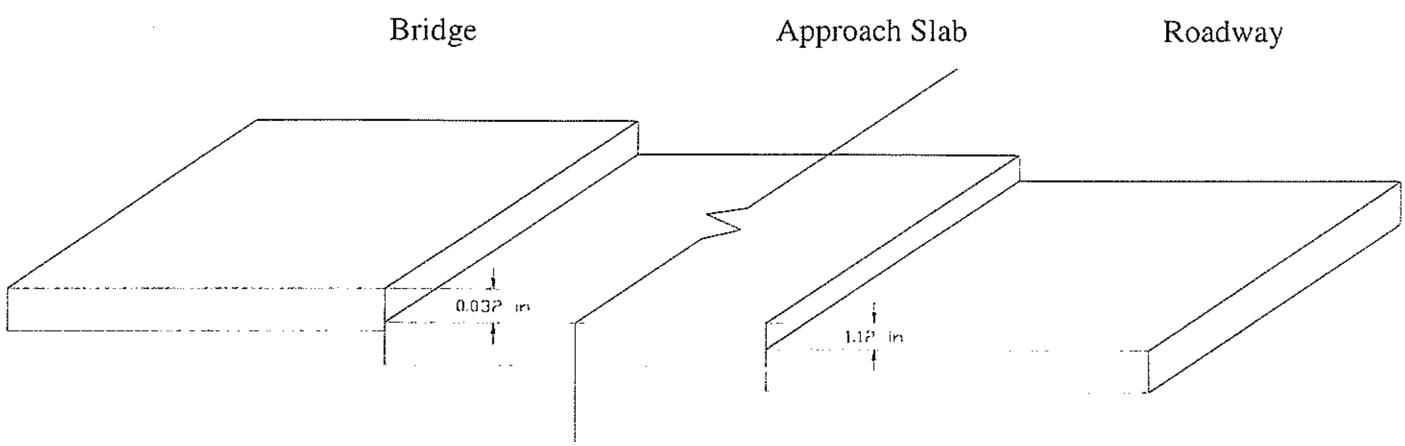


Figure 33
Average height difference between the approach slab and the roadway and bridge joints for Site 4

PROFILER AND DYNATEST RESULTS

The walking profiler and the Dynatest tests were performed for Site 4. The results of these tests were compared with those of the geodetic survey for the longitudinal and transverse directions. The data obtained from the profiler was graphed separately as well. The profiler graphs show a slight bent in the approach slab, but it is not as drastic as in Site 1.

IRIs

The IRIs for Site 4 were determined using a laser profiler and are shown in Figure 34.

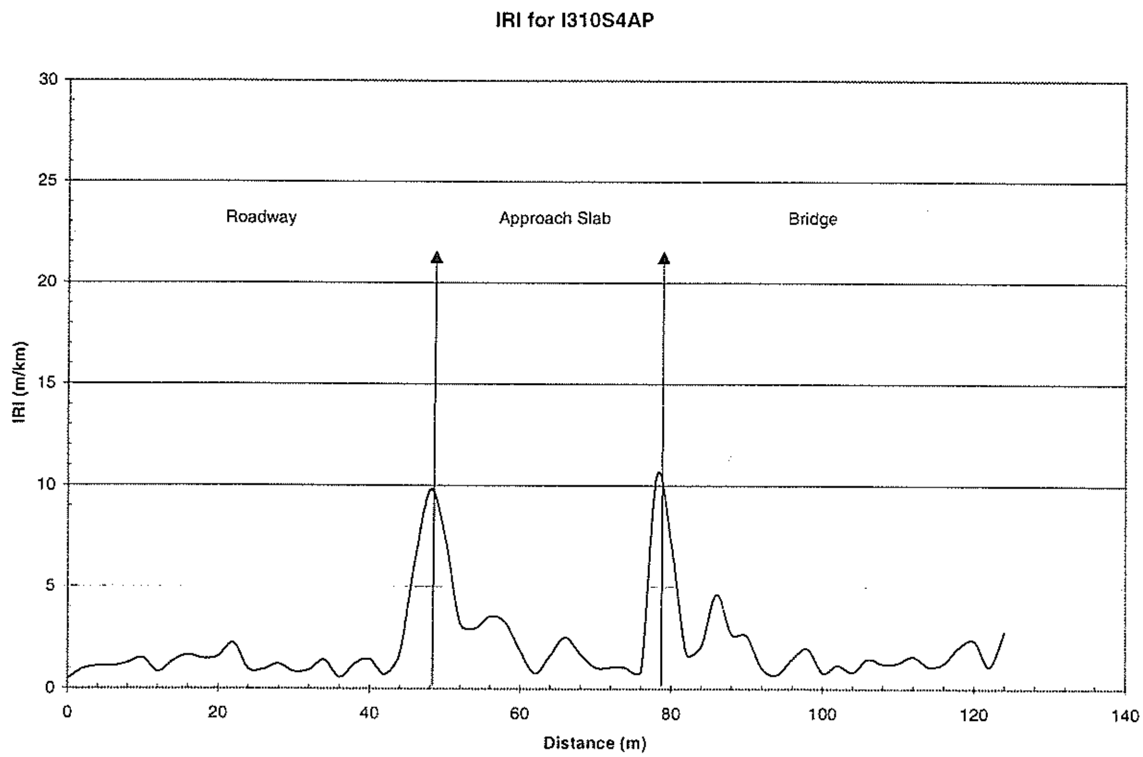


Figure 34
IRIs for Site 4

SITE 5: I-310 HALE BOGGS BRIDGE SOUTH APPROACH

Site 5 is the south approach to the Hale Boggs (Luling) Bridge and includes the southbound and northbound lanes (see fig. 10a for map location). Figure 35 shows a view of this site looking in the north direction from the southbound approach.

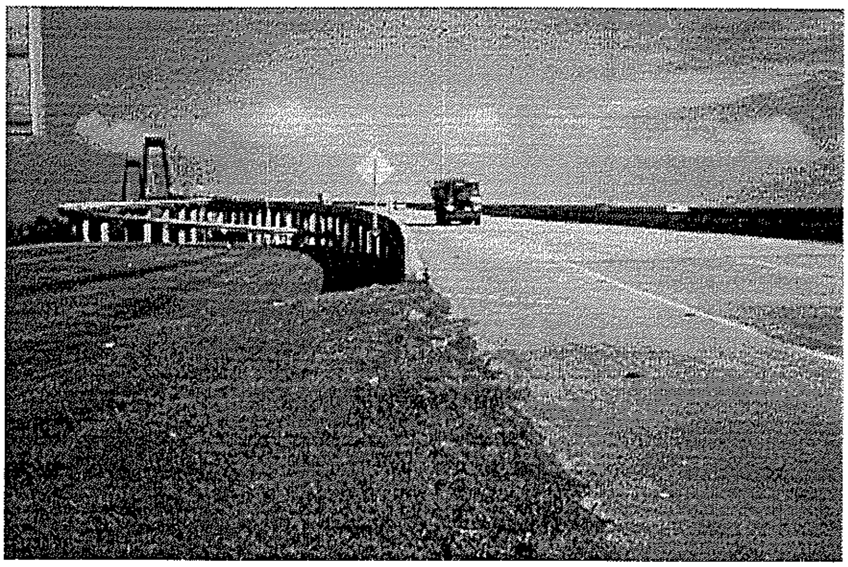


Figure 35
Site 5: Hale Boggs bridge south approach

TEST RESULTS

Points were marked on each approach slab, on the adjacent roadway and on the bridge deck. These points were used to mark the location of the survey, Dynatest and walking profiler measurements.

VISUAL INSPECTION

There was no significant vertical differential settlement at the interface between the approach slab and the roadway and the interface between the bridge and the approach slab. However, at the interface between the bridge deck and the approach slab, there exists severe longitudinal displacement (fig. 36). The gap between the bridge abutment and the approach slab reached about six in (152.4 mm) and the joint sealer fell off. This horizontal movement is mainly due to the rotation of the bridge abutment, which often happens when there is a high embankment height. Figure 37 shows a bolt connection damaged by the excessive movement of the abutment on the northbound approach.

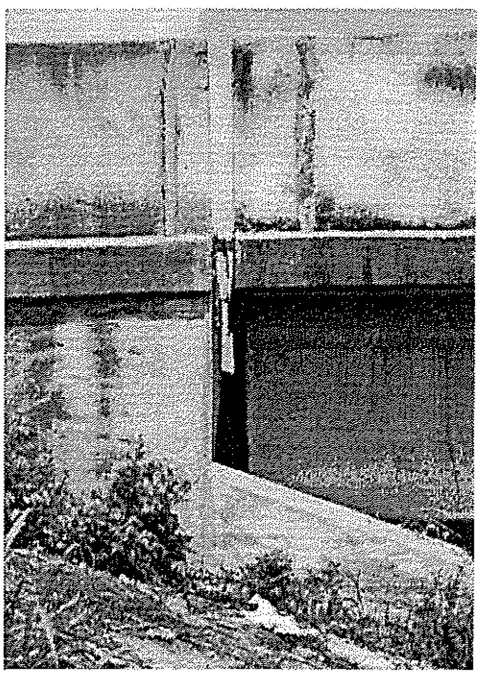


Figure 36
Gap between abutment and approach slab (S/B)

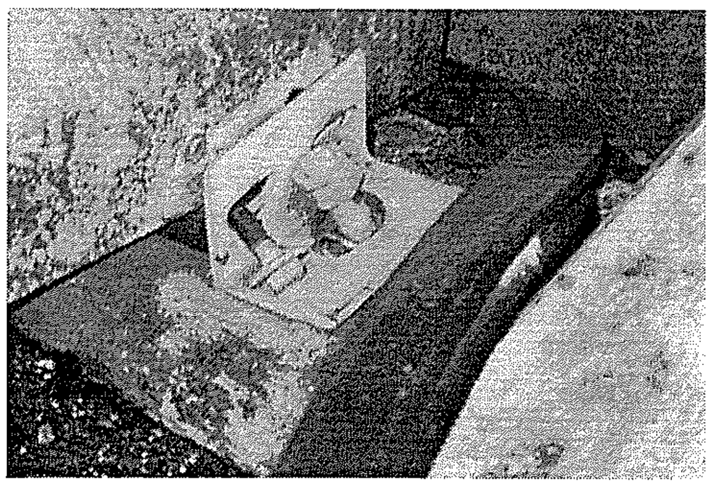


Figure 37
Bolt connection damaged due to abutment horizontal movement at northbound

IRIs

The IRIs for Site 5 were calculated based on the reading of the laser profiler. The IRIs for the northbound and southbound approach slabs were graphed and are shown Figures 38 and 39, respectively. The IRIs are significantly higher on the approach slabs, especially at the ends. The graphs show that the approach slabs have a rating of poor to very poor according to the recognized standard pavement rating scale shown in Table 2.

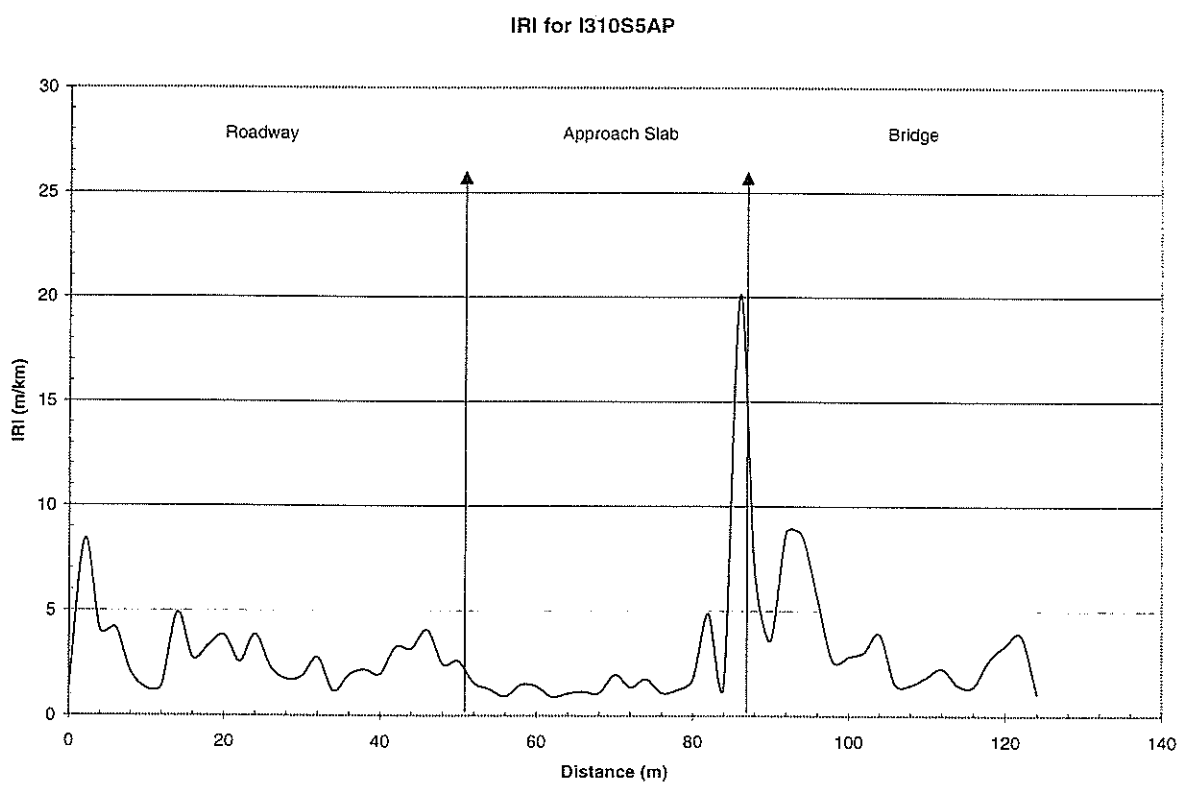


Figure 38
IRIs for Site 5 northbound approach slab

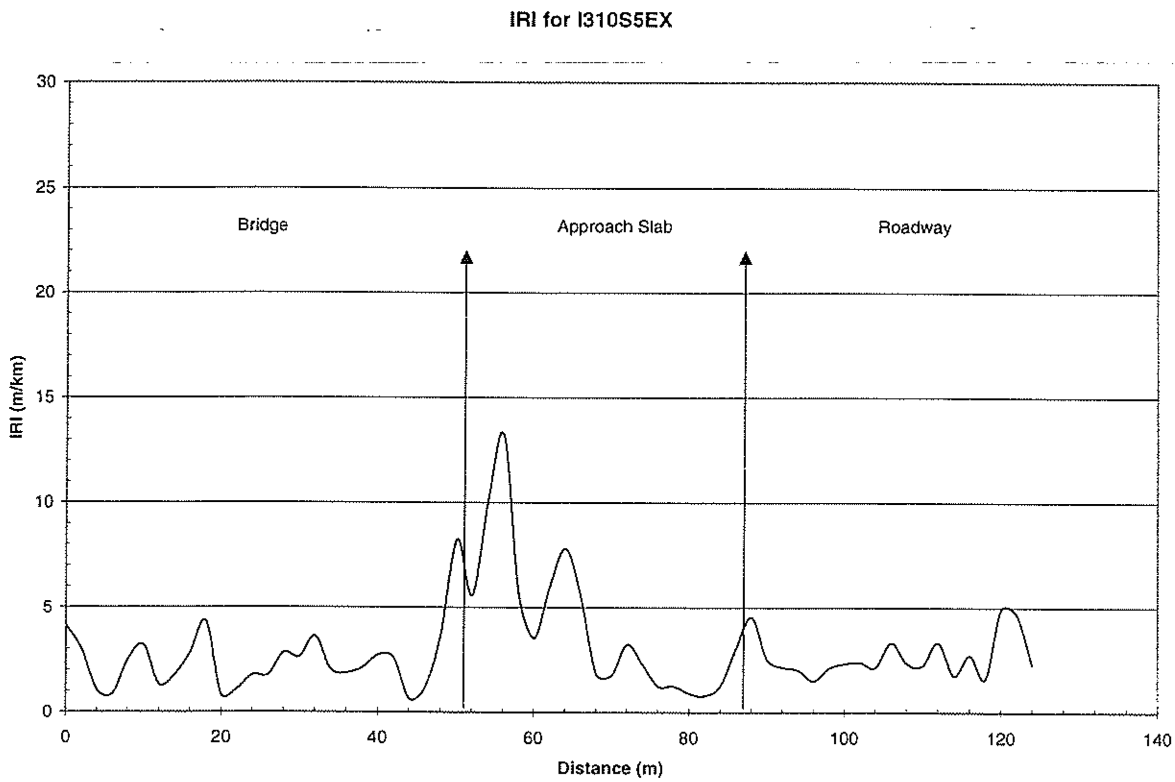


Figure 39
IRIs for Site 5 southbound approach slab

SITE 6: I-310 OVER LA 3127

Site 6 in this project is the southbound south approach of I-310 over LA 3127, (see Figure 10a for the location map).

TEST RESULTS

The approach slab at Site 6 performed very well. There is no significant differential settlement at the joint between the bridge abutment and the approach slab, or that between the approach slab and the roadway.

SURVEY RESULTS

The relative elevations of the points marked on the bridge and approach slab were determined using an electronic total station.

Table 16

Differential settlement of points at the joints for Site 6

	Site 6	
	Slab/roadway joint differential settlement (in.)	Bridge/slab joint differential settlement (in.)
Row 1:	-0.828	0.072
Row 2:	-0.06	-0.18
Row 3:	0.012	-0.084
Average:	-0.216	-0.064

Metric Equivalent:

1 in = 25.4 mm

It can be seen from Table 16 that the differential settlement is relatively small at both the approach slab/roadway joint and the bridge/approach slab joint for this site. The maximum average differential settlement was –0.216 in (-5.49 mm). Figures 40 shows the average height differences between the approach slab and the roadway and bridge.

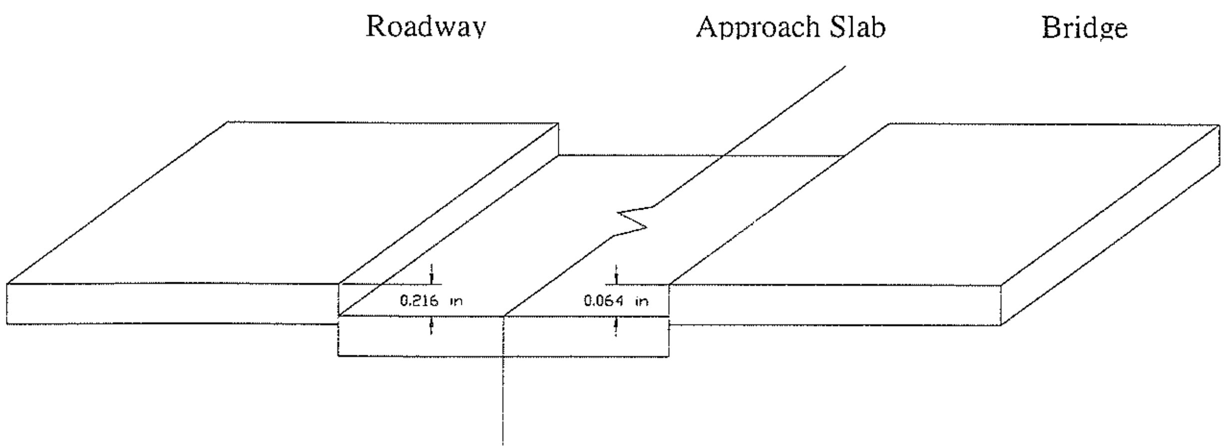


Figure 40
Average height difference between the approach slab and the roadway and bridge joints for Site 6

SITE 7: LA 3139 PARISH LINE BRIDGE

Site 7 is the westbound west approach of the Jefferson/Orleans parish line bridge (see Figure 10b for map location). Figure 41 shows a view of this site looking in the east direction. As shown in Figure 41, a serious bump has developed at the start of the approach slab on the bridge side. A void can be seen under the slab in this area, which indicates that the soil under the approach slab has significantly settled and accordingly no contact is present between the slab and the foundation soil.

Figure 41
Site 7: LA 3139 parish line bridge westbound west approach

This bridge was constructed in 1982 in accordance with the 1977 DOTD Standard Specifications for Roads and Bridges and with the 1973 AASHTO Standard Specifications for Highway Bridges. Information concerning the approach slab, embankment, and piles at both the northbound and southbound approaches is described in the following sections.

TEST RESULTS

Survey points were marked on the approach slab, adjacent roadway and bridge deck. These points were used to mark the locations of the survey, Dynatest and walking profiler measurements. As indicated in Table 4, field tests performed at this site included: survey, profiler test, Dynatest and soil borings.

VISUAL INVESTIGATION

There is no significant differential settlement at the joint between the approach slab and the roadway, and the joint between the bridge and the approach slab. The approach slab, however, has settled significantly near the bridge abutment. Cracks were observed in the approach slab at this site. About eight inches (203.2 mm) of differential settlement was observed between the approach slab outer edge and the non-pile supported shoulder pavement at the bridge abutment station. This differential settlement decreased to zero in the direction of traffic (westbound) at the roadway/slab interface. Figure 42 shows the longitudinal differential settlement between the approach slab and its side shoulder pavement.

To investigate the embankment condition under the approach slab near the bridge abutment, part of the shoulder concrete slab, which was already broken, was removed. Figure 43 shows that a gap as large as three feet (0.91 m) has developed between the approach slab and the foundation soil. The large gap can be attributed to the consolidation settlement in the foundation soil under the weight of the highway embankment as well as the erosion of the granular embankment material.

By removing the shoulder to examine under the approach slab, only the section between the abutment and the first row of piles could be observed. In order to investigate if there were voids under other sections of the approach slab, one inch (25.4 mm) holes were drilled between pile rows. A total of five holes were drilled. A metal rod was dropped through the hole and a mark was made on the rod where the top of the approach slab met the rod. The distance from the bottom of the rod to the mark was measured and the thickness of the approach slab was then subtracted to determine the depth of the void. As expected, the voids became smaller away from the bridge. No void was detected at a distance of about forty five feet (13.72 m) from the bridge.

An attempt was made to measure the vibration of the approach slab as the traffic went by using an oscilloscope with a recording plotter attached to it. A sample of the recording paper is given in Schutt, which shows the large difference in the vibrations when a large truck drives by in comparison to a regular automobile *[22]*. When a 3 axle truck drives by, the amplitude of the response is about five times larger than that of a regular automobile.

A two axle truck causes the response amplitude to increase three times the amount of that of an automobile.

When conducting this vibration experiment it was discovered that the approach slab was loose at the outside edge corner of the abutment. Any time a large vehicle drove by, the approach slab would push down and hit the abutment. This separation was probably forced by the excessive settlement of the approach slab.

SURVEY RESULTS

The relative elevation of each of the points marked on the approach slabs was determined using an electronic total station. Thus the profile along each longitudinal direction of the four rows surveyed was obtained. The original elevations of the specific points along the approach slab were calculated from pile cutoff elevations (Table 16), the settlements along each longitudinal direction were determined by interpolation. Figure 44 shows the approach slab settlement along each longitudinal row.

Figure 44 indicates that the maximum settlement at this test site is about 1.2 ft (0.37 m) near the roadway/approach slab joint. At a distance of 40 ft (12.19 m) away from the bridge/approach slab interface, the approach slab is close to the elevation of the edge of the approach slab at the roadway. Nearly 85 percent [1 ft (0.30 m)] of the differential settlement between the bridge abutment and the roadway has occurred in the first 40 ft (12.19 m) segment of the 80 ft (24.38 m) long approach slab. It can be concluded that the approach slab at this test site did not perform adequately. This large differential settlement over such a short distance not only produced an abrupt transition which caused discomfort to the driver but also resulted in the cracking of the approach slab as observed in the field.

Figure 42
Longitudinal differential settlement between approach slab and its adjacent side shoulder pavement

Figure 43
View of embankment under approach slab

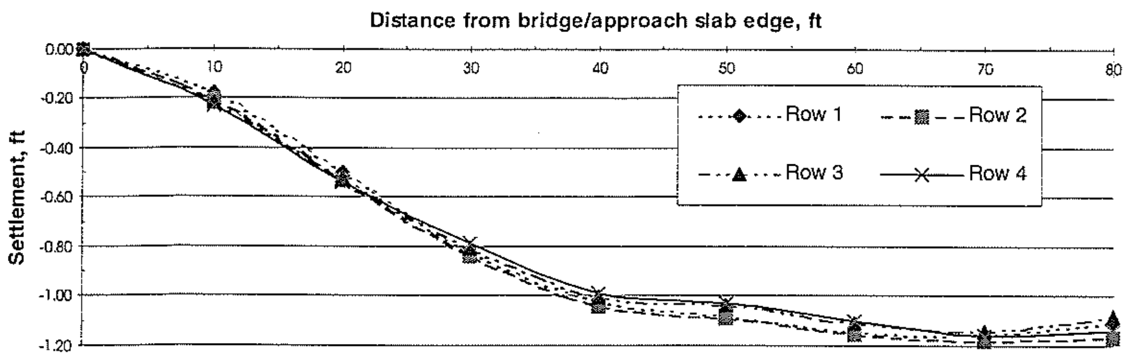


Figure 44
Approach slab settlement for Site 7

IRIs

The IRIs for Site 7 were calculated using the readings of a laser profiler. The IRIs were graphed and are shown in Figure 45. IRIs were significantly higher on the approach slab. This is due to the poor condition of the approach slab in comparison with the condition of the bridge and roadway which indicates the poor condition of the approach slab. The graph shows that the approach slab has a rating of very poor, according to the pavement rating scale shown in Table 17. The highest IRI occurred where the change in the slope of the survey is the highest, which indicates that the IRI could be used to identify the worst locations on the approach slab with respect to deflection.

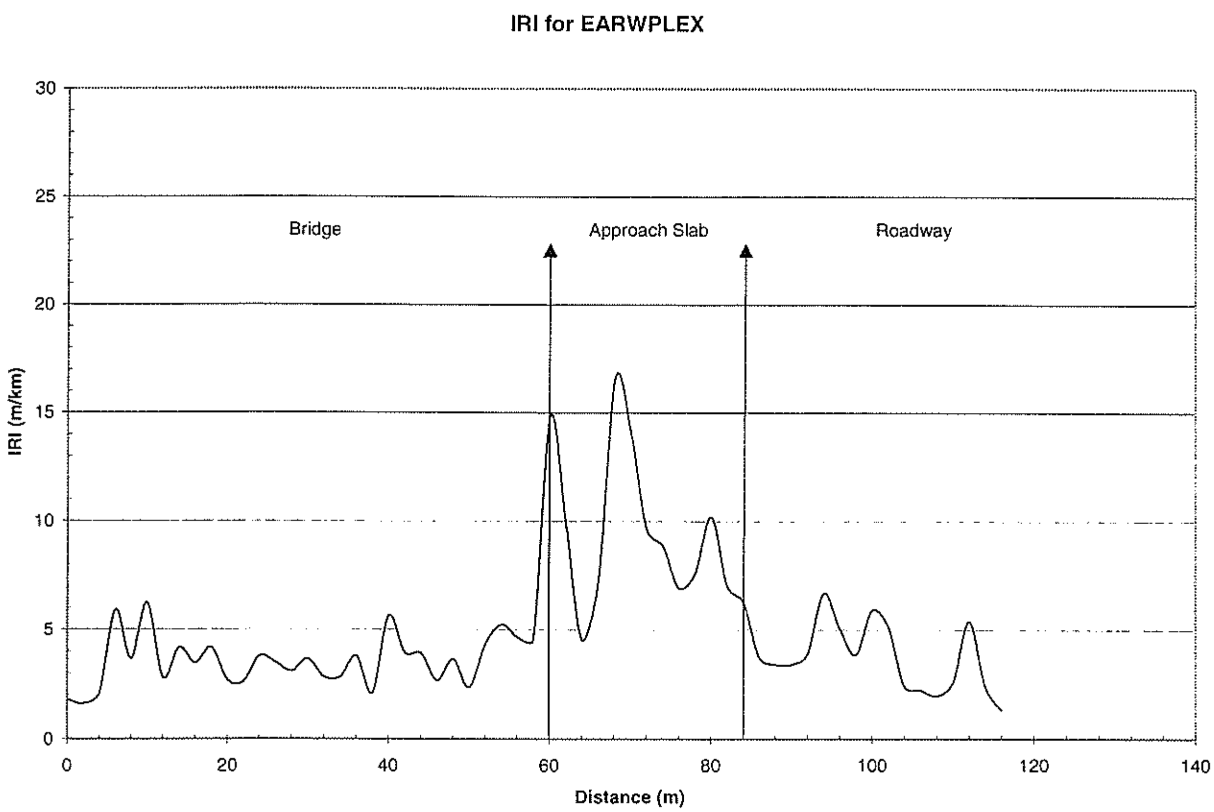


Figure 45
IRIs for Site 7

DEVELOPMENT OF THE INTERNATIONAL ROUGHNESS INDEX SLAB (IRIS) RATING SYSTEM

The highest IRIs for the approach slabs ranged from 3 to 27 (Table 3). In order to better evaluate the performance of the approach slabs, a new refined approach slab rating system (IRIS) was developed *[22]*. The system is shown in Table 17.

Table 17

Refined IRI Approach Slab Rating System (IRIS)

IRI Range	Rating
0 to 3.9	Very Good
4.0 to 7.9	Good
8.0 to 9.9	Fair
10.0 to 11.9	Poor
12 and above	Very Poor

It was felt that a more objective method is needed for evaluating approach slabs in lieu of the existing subjective rating system based on visual inspection. Since the IRI was originally developed for pavement evaluation, it was necessary to modify the system for use in approach slabs assessment. The new rating system was developed by evaluating the specific test sites where comprehensive testing has been performed to identify their condition. For example, Site 1 (elevated structure to Airline Hwy) was considered to be in poor condition. Sites 2 and 3 (pipeline bridge) were considered to be in good condition. Once the seven sites that were evaluated and assigned a rating value, the highest IRI for each of these approach slabs was retrieved. Using these values, the IRI approach slab rating system was developed. In order to rate all the approach slabs the highest IRI on each of the approach slabs was identified and the approach slab was rated according to this value. The IRI values were rounded off to the nearest whole number.

A list was made containing the sites where IRI information was available. This list included the file number, length, highest IRI, current condition rating and whether it was pile or non-pile supported slabs. The current condition ratings and the IRI ratings corresponding to each approach slab were compared. Of these, 47 percent were very close, 29 percent were close and 24 percent were not close. It seems that, for the most part, the IRI rating and the current condition rating seem to match up.

According to the results, it seems that the scale developed in this research study matches closely to the current condition rating.

SUMMARY OF FINDINGS

In order to compare the results of the seven test sites, the information was summarized in three tables. Table 11 gives the general site information. The details of the approach slabs are shown in Table 12. Table 13 contains the soil conditions of the sites in which soil borings were performed (Sites 1, 5, and 7). All three tables contain approach slab condition information for comparison. Actual settlements were calculated for approach slabs in Sites 1, 5, and 7.

The approach slabs in Sites 2, 3 and 6 showed no significant deflection and the average elevation differences between the roadway and the approach slab joints were very low (Table 11).

The approach slabs in Site 5 showed severe longitudinal displacement. These approach slabs were the only ones of the seven sites that displayed this phenomenon. As shown in Table 12, these approach slabs also happened to have the highest embankment.

As discussed, Site 5 is the site with the highest embankment. However, as is also shown in Table 12, it is the site with the shortest piles. Site 5 settled more than Site 1, even though they both have similar soil conditions. Therefore, the higher settlements observed in Site 5 could be attributed to the higher embankment weight and use of shorter pile for support of the approach slab.

Another factor that could affect settlement is permeability of the soil. As shown in Table 13, Site 5 the consolidating soils are embedded between less permeable strata. With lower permeability, consolidation would take longer time to occur due to the longer drainage path. Therefore, given the above assumption, if both sites were surcharge for the same time period, Site 1 would be closer to reaching full consolidation than Site 5 and Site 1 should experience less settlement after the surcharge period.

Table 18

General Site Information

	SITE 1		SITE 2	SITE 3	SITE 4	SITE 5		SITE 6	SITE 7
Travel Direction	N/B	S/B	S/B	S/B	S/B	N/B	S/B	S/B	W/B
Visual Observations	Significant Deflection	Significant Deflection	No Significant Deflection	No Significant Deflection	Slight Deflection	Severe Longitudinal Displacement	Severe Longitudinal Displacement	No Significant Deflection	Significant Deflection
Survey and Walking Profiler Observations	Abrupt Change in Slope about 10 feet from Abutment	Abrupt Change in Slope about 5 feet from Abutment	No Abrupt Change in Slope	No Abrupt Change in Slope	No Abrupt Change in Slope	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	No Abrupt Change in Slope	Abrupt Change in Slope about 5 feet from Abutment
Length of Section of Approach Slab Spanning Most of the Settlement (ft)	60	100	100	100	100	60	60	80	40
Current Condition Rating	6	6	8	8	6	6	6	8	6
Highest IRI	16	13	6	9	10	20	13	4	16
Maximum Actual Settlement (ft)	0.7	0.8	N/A	N/A	N/A	1.0	1.0	N/A	1.2
Settlement Calculated (ft)	0.95	0.95	N/A	N/A	N/A	1.20	1.20	N/A	N/A
Parish	St. Charles	St. Charles	St. Charles	St. Charles	St. Charles	St. Charles	St. Charles	St. Charles	Jefferson
Year Built	1992	1992	1991	1991	1991	1991	1991	1991	1982
Geometry	Straight	Straight	Trapezoid	Trapezoid	Straight	Straight	Straight	Curved	Trapezoid
Daily Traffic Count	28230	28230	28230	28230	28230	28230	28230	28230	N/A
On or Off Ramp	On	Off	On	Off	On	On	Off	Off	Off
Speed Limit (mph)	70	70	70	70	70	70	70	70	50
Location	I-310 Elevated Structure	I-310 Elevated Structure	I-310 Pipeline Bridge	I-310 Pipeline Bridge	I-310 Hale Boggs Bridge	I-310 Hale Boggs Bridge	I-310 Hale Boggs Bridge	I-310 Over LA 3127	LA 3139Parish Line Bridge
Entity Traversed	Swamp	Swamp	Small Canal	Small Canal	Mississippi River	Mississippi River	Mississippi River	Roadway	Canal
Design Loads	HST-18 Truck	HST-18 Truck	HST-18 Truck	HST-18 Truck	HST-18 Truck	HST-18 Truck	HST-18 Truck	HST-18 Truck	HS20-44
Slab/Roadway Joint Average Elevation Difference (in)	N/A	N/A	0.1	0.1	1.12	N/A	N/A	0.216	N/A
Bridge/Slab Joint Average Elevation Difference (in)	N/A	N/A	0.188	0.248	0.032	N/A	N/A	0.064	N/A

N/B = northbound S/B = southbound N/A = not available

Table 19

Details of Approach Slabs

	SITE 1		SITE 2	SITE 3	SITE 4	SITE 5		SITE 6	SITE 7
Travel Direction	N/B	S/B	S/B	S/B	S/B	N/B	S/B	S/B	W/B
Visual Observations	Significant Deflection	Significant Deflection	No Significant Deflection	No Significant Deflection	Slight Deflection	Severe Longitudinal Displacement	Severe Longitudinal Displacement	No Significant Deflection	Significant Deflection
Survey and Walking Profiler Observations	Abrupt Change in Slope about 10 feet from Abutment	Abrupt Change in Slope about 5 feet from Abutment	No Abrupt Change in Slope	No Abrupt Change in Slope	No Abrupt Change in Slope	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	No Abrupt Change in Slope	Abrupt Change in Slope about 5 feet from Abutment
Length of Section of Approach Slab Spanning Most of the Settlement (ft)	60	100	100	100	100	60	60	80	40
Current Condition Rating	6	6	8	8	6	6	6	8	6
Highest IRI	16	13	6	9	10	20	13	4	16
Maximum Actual Settlement (ft)	0.7	0.8	N/A	N/A	N/A	1.0	1.0	N/A	1.2
Settlement Calculated (ft)	0.95	0.95	N/A	N/A	N/A	1.20	1.20	N/A	N/A
Slab Length (ft)	100	100	100	100	100	120	120	80	80
Slab Width (ft-in)	42-10	42-10	42-8	42-8	40-11	38-0	38-0	42-10	35-0
Thickness (in)	10	10	10	10	10	10	10	10	10
Concrete Grade	AA	AA	AA	AA	AA	AA	AA	AA	A
Pile Type	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber	Treated Timber
Pile Diameter (in)	12	12	12	12	12	12	12	12	12
Pile Lengths (ft)	15-60	15-60	15-60	15-60	15-60	9-54	9-54	15-60	15-60
Pile Transverse Spacing (ft-in)	6-3	6-3	6-3	6-3	6-0	8-9	8-9	7-6	9-7
Pile Longitudinal Spacing (ft)	10	10	10	10	10	10	10	10	10
Fill Height (ft)	9	9	8	8	10	12	12	8	2.5

N/B = northbound S/B = southbound N/A = not available

Table 20

Soil Conditions in Sites 1, 5 and 7

	SITE 1		SITE 5		SITE 7
Travel Direction	N/B	S/B	N/B	S/B	W/B
Visual Observations	Significant Deflection	Significant Deflection	Severe Longitudinal Displacement	Severe Longitudinal Displacement	Significant Deflection
Survey and Walking Profiler Observations	Abrupt Change in Slope about 10 feet from Abutment	Abrupt Change in Slope about 5 feet from Abutment	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	Abrupt Change in Slope at Midway Point Between Abutment and Roadway	Abrupt Change in Slope about 5 feet from Abutment
Length of Section of Approach Slab Spanning Most of the Settlement (ft)	60	100	60	60	40
Current Condition Rating	6	6	6	6	6
Highest IRI	16	13	20	13	16
Maximum Actual Settlement (ft)	0.7	0.8	1.0	1.0	1.2
Settlement Calculated (ft)	0.95	0.95	1.20	1.20	N/A
Surcharge Time (months)	6	6	N/A	N/A	N/A
Details of Top Layers	Sand (fill) 6 feet	Sand (fill) 6 feet	First 30 feet consist of small layers of soft to medium clay, stiff organic clay, loose sandy silt and loose clayey sand		Sand (fill) 7 feet
Predominant Soil	Soft Clay with Silt Lenses and Organics	Soft Clay with Silt Lenses and Organics	Soft Clay with Sand Pockets	Soft Clay with Sand Pockets	Soft Clay with Organics
Thickness of Predominant Soil (ft)	55	55	35	35	50
Average Unconfined Compression Strength of Predominant Soil (psf)	500	500	640	640	645
Average Compression Index (C_c) of Predominant Soil	0.5	0.5	0.7	0.7	0.6
Average Permeability (k) (m/s) of Predominant Soil	1.4×10^{-5}	1.4×10^{-5}	4.9×10^{-6}	4.9×10^{-6}	N/A
Depth of Pleistocene Age Soils (ft)	65	65	65	65	65

N/B = northbound S/B = southbound N/A = not available

SOIL-STRUCTURE INTERACTION STUDY

The static capacity of a pile consists of the summation of mobilized shaft (skin) resistance and end bearing (toe or point) resistance. A positive shaft resistance is mobilized during compression loading or when the pile is being pushed downward into the ground. A negative shaft resistance, on the other hand, develops along the pile shaft when it is being loaded in tension or subjected to uplift. Shear stresses generally develop along the pile shaft as the surrounding soils move relative to the pile itself. In the later case, an additional negative (downward) skin friction could develop along the pile shaft when the soil settles relative to the pile, such as the case of a pile installed in a consolidating soil mass. In turn, an additional positive skin friction could develop along the pile shaft when the soil expands, such as the case of a pile installed in a swelling soil.

For piles installed in a layered soil medium, the upper strata may settle due to a surcharge load or a general groundwater lowering. When a surcharge load or fill (fig. 46) is placed, the underlying compressible soil strata consolidate, resulting in surface subsidence and possible damage to surface structures. Theoretically, this should extend to a significant depth if the fill area is relatively large and no pre-consolidated stiff clay or dense sand stratum exists to limit its effect. Drag load develops when consolidating soils impose "negative skin friction" or "downdrag load" on the piles and create an extraneous downward load on the piles. In general, drag load and its effects are primarily a function of the thickness of fill, compressibility of the soils, time-rate of consolidation, pile length and sustained pile load. In southeastern Louisiana, consolidation is greatest in the upper Holocene deposits (fig. 2), primarily due to the greater compressibility of these highly organic or soft normally consolidated soils.

When piles with tips embedded in dense sand or stiff clay are used for support, significant "point" or "tip" support would be achieved. In this case, drag load should be considered in the structural design of the pile member itself for fear of possible overstressing of the pile member itself. This type of pile is typically used for support of the bridge abutment and, therefore, relatively negligible settlements are typically experienced in these structures.

In the case of a pile-supported approach slab, the piles are embedded in the consolidating soil mass and no significant point support is typically available. This condition results in the subsoils both supporting the structure through "skin friction" along the embedded portion of the pile and yet allowing settlement of the structure to occur because of the consolidating mass in which they are embedded.

Equilibrium of forces on a pile (fig. 20) is defined as the sum of sustained load applied at the pile head (P) and dragload (Q_d), and the sum of the positive shaft resistance (Q_s) and point resistance (Q_p). The sustained pile load is defined as the summation of applied dead loads plus any permanent long-term live load residual. Calculated settlement of the subsoil strata should be estimated on the fill load and any additional surcharge load. The location where equilibrium of forces occurs is called the "neutral plane" or "neutral point". It is generally defined as the depth at which the shear stress along the shaft changes from negative skin friction to positive shear resistance *[16]*. It is also defined as the location along the pile shaft where there is no relative displacement between the pile and surrounding soil *[17]*.

A Microsoft Excel spreadsheet with Visual Basic Application (VBA) macros was developed for use in design of pile-supported bridge approach slabs. The spreadsheet is based on a numerical model that accounts for downdrag and site specific conditions including soil settlement and approach slab design. The spreadsheet could be used to perform a parametric study to select the desired pile lengths throughout the slab length that yield an acceptable long-term settlement profile.

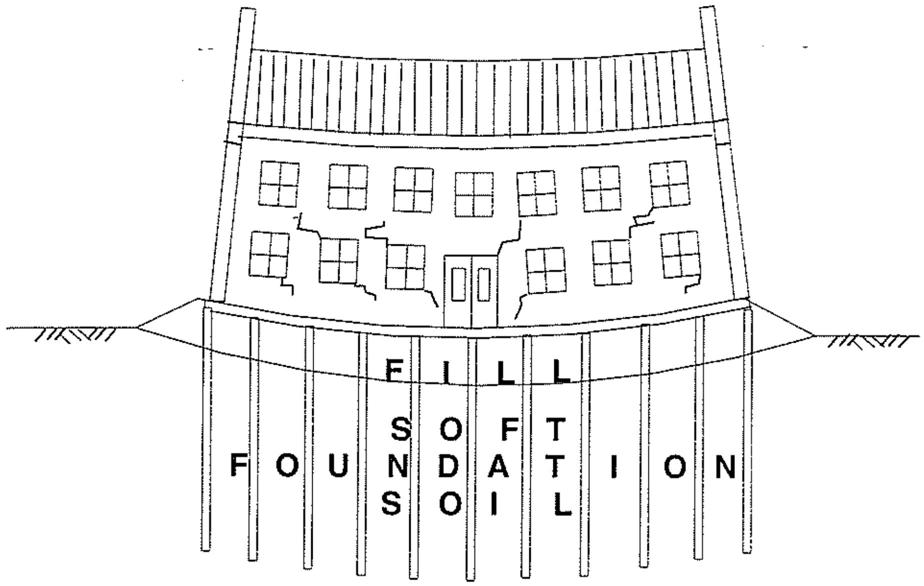


Figure 46
An example of negative friction on piles *[18]*

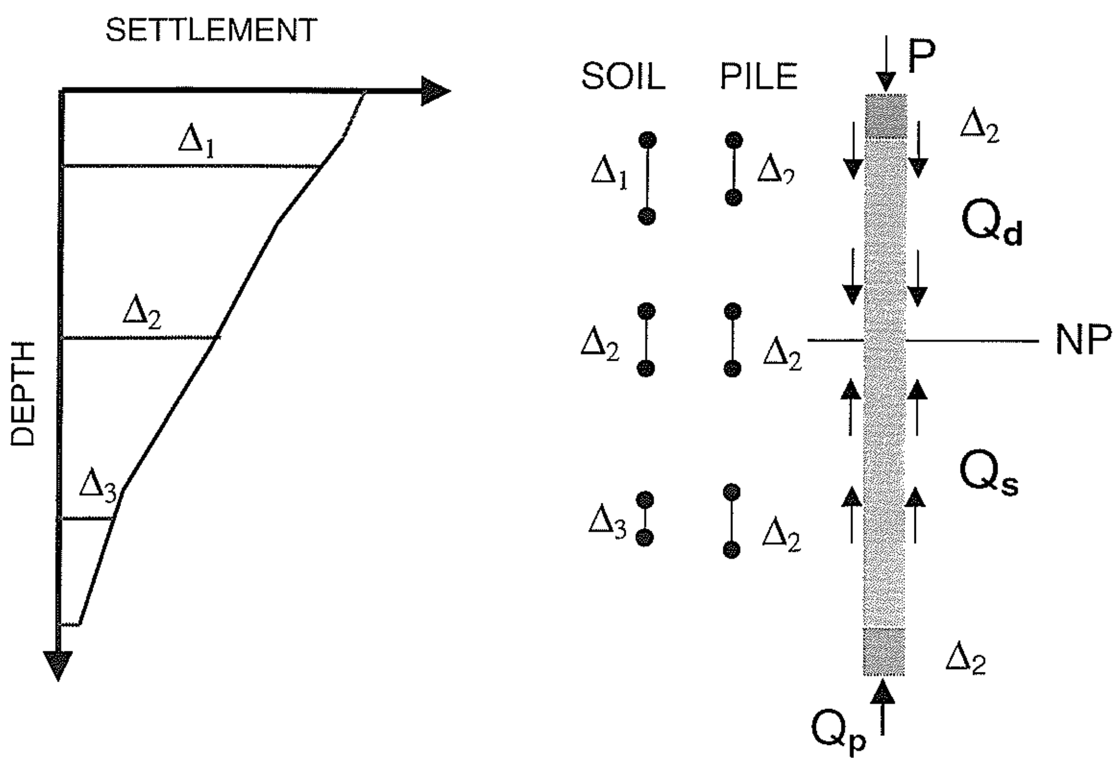


Figure 47
Equilibrium of forces on a pile *[25]*

ANALYTICAL METHOD

The proposed analytical method to estimate the long-term settlement profile of the pile-supported approach slabs involves the following steps:

1. Select a "preliminary" design for the approach slab that includes its length, number and spacing of transverse rows of piles, pile type and size and pile length along each transverse row.
2. Establish the embankment, surcharge and foundation characteristics at each pile row location. This includes embankment height, surcharge height and stratification of the underlying soils. Also establish the necessary soil properties needed to estimate the consolidation settlement and to calculate pile head capacity at each pile row location (γ, C_c, OCR, LL, PL, etc.).
3. Estimate the soil settlement profile along each transverse row of piles.
4. Estimate the mobilized friction stiffness of a single pile of length L in each transverse row of piles.
5. Estimate the longitudinal settlement profile of the approach slab based on the estimated settlement of the typical single pile within each row and the other characteristics established in steps 1 through 4.
6. Compare the estimated settlement profile and the ideal settlement profile.
7. Repeat steps 1 through 6 until an acceptable estimate of the approach slab settlement profile is achieved.

In the proposed approach, it is assumed that the response of any single pile in a given transverse row of piles would represent that of the entire pile row. This further assumes that all piles in the row have the same length, applied load and load bearing capacity. It is also assumed that surcharge and embankment properties are the same along each transverse row and, therefore, all piles along a given row would experience the same amount of settlement. However, these parameters could be different at each pile row along the length of the approach slab. In view of this, different settlement should be expected at each row of piles if piles of various lengths are used along the slab length. Therefore, an "ideal" design profile could be achieved through a trial and error process where the settlement profile is adjusted by changing the selected pile lengths. This further assumes that for the same surcharge, embankment and subsoil conditions, drag load, location of the neutral point and settlement of the single pile would only depend on the pile length, as discussed earlier.

In general, consolidation settlement of a structure results from long-term loads, such as dead load, fill load and any sustained portion of the live load. Additional settlement could also occur due to temporary or short-term loads; such as due to a surcharge load or lowering of the water table, but magnitude of the resulting settlement would depend on the duration of these temporary loads. Therefore, settlement analyses should include the effect of surcharge loads depending on the degree of consolidation achieved during the surcharge period. On the other hand, other short-term loads, such as live load or traffic load, do not induce appreciable consolidation settlement since they do not apply for an extended period of time.

Since drag load is a result of consolidation settlement, consideration should only be given in design to long-term sustained loads, such as dead loads and fill loads, and short-term loads of relatively extended duration, such as surcharge loads. In case of an approach slab, it is recommended that only these loads be considered in estimating the settlement profile and the required pile lengths. In regard to pile head load, it is recommended that the analyses should account only for the dead load of the approach slab and the pile cap (beam) (fig. 50).

The proposed approach was programmed into a spreadsheet type computer program TU-DRAG. The spreadsheet was developed using Microsoft Excel and Visual Basic. The results of the test sites described earlier in this report were used to develop and verify the spreadsheet and are presented in the next sections. A user's manual for the spreadsheet is given in Appendix C and typical example runs are given in Appendix D.

The subsoils deposit underneath the highway embankment could be divided into a finite number of soil strata. A typical subsoil profile in southeastern Louisiana would consist of a relatively thick deposit of soft alluvial cohesive soils underlain by stiffer, or denser, Pleistocene Age soils. The consolidation settlement of these strata could be determined using Terzaghi's one-dimensional consolidation theory. A computer program could be used to obtain the variation of the estimated settlement along the approach slab embankment. At present, an embankment settlement analysis program *[26]* is used by DOTD for calculating consolidation settlement of soils.

A typical shape of the soil settlement curve is shown in Figure 48.As shown in Figure 48, settlement of the soil strata diminishes with depth and the cumulative maximum settlement will be realized at the ground surface. For the extremely soft cohesive soils typically encountered in southeastern Louisiana, the cumulative settlement under a typical highway embankment could be on the order of one to two feet (0.30 to 0.61 m). This settlement is also time dependent and may take several years to fully develop under a sustained load. Therefore, performance of a given approach slab may change with time as more settlement occurs.

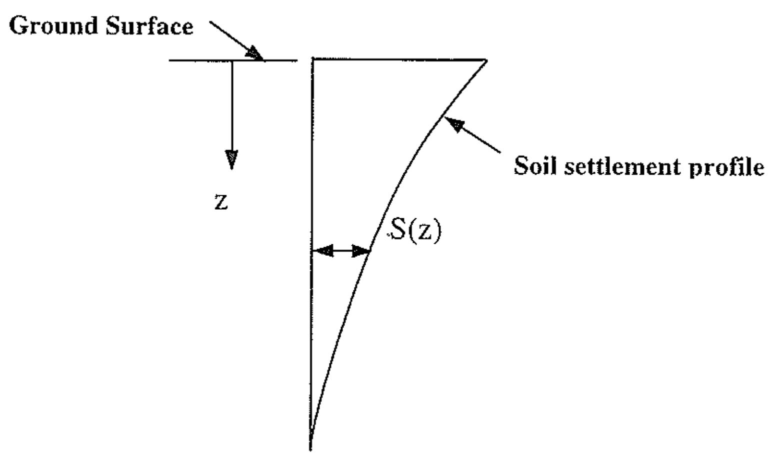


Figure 48
Settlement distribution along soil depth

In order to determine the friction stiffness of a pile, a displacement ΔL at the pile head of length L has to be assumed. The pile shaft is assumed to deform elastically by an amount of ΔL_{shaft}. Thus the displacement at the pile tip is:

$$\Delta L_{tip} = \Delta L - \Delta L_{shaft} \quad (1)$$

ΔL_{shaft} is typically insignificant and thus could be ignored in the analysis. Therefore at any depth along the pile shaft

$$\Delta L(z) = \Delta L_{tip} = \Delta L \quad (2)$$

where $\Delta L(z)$ = pile displacement at depth z

The relative displacement $\Delta D(z)$ between the soil and pile at any depth could also be defined as:

$$\Delta D(z) = S(z) - \Delta L(z) \quad (3)$$

where $S(z)$ is the estimated soil consolidation settlement at depth z.

Along the pile shaft, there often exists a point where the relative displacement between the pile and the surrounding soil is almost zero ($\Delta D(z) = 0$). As discussed earlier, this point is defined as the neutral point. Figure 49 illustrates the determination of the neutral point for a given pile head displacement.

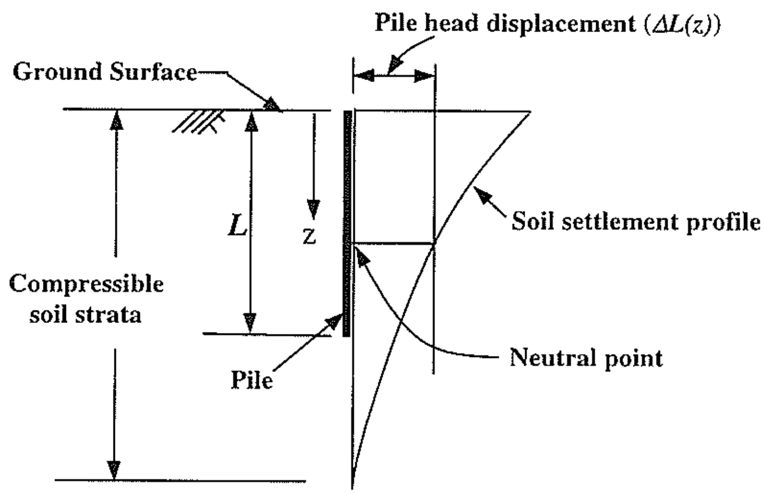


Figure 49
Neutral point illustration

According to the FHWA[27], frictional resistance per unit surface area of pile length at depth z, $f(z)$ is calculated from relative displacement $\Delta D(z)$ as:

$$f(z) = \begin{cases} -f_s & \text{If } \Delta D(z) \geq 0.5\,in(12.7mm) \\ -f_s \dfrac{\Delta D(z)}{0.5} & \text{If } 0 \leq \Delta D(z) \leq 0.5\,in(12.7mm) \\ f_s \dfrac{\Delta D(z)}{0.5} & \text{If } -0.5\,in(-12.7mm) \leq \Delta D(z) \leq 0 \\ f_s & \text{If } \Delta D(z) \leq -0.5\,in(-12.7mm) \end{cases} \quad (4)$$

where f_s = Pile shaft skin resistance.

Thus, the pile head force ΔF is calculated as:

$$\Delta F = \int_0^L f(z) A_s dx \tag{5}$$

where A_s = Effective pile surface area on which $f(z)$ acts.

From the Eqn.5, the friction force distribution along a pile shaft could be obtained, which produces a resultant frictional force, ΔF. Theoretically, the pile friction stiffness S_p is then calculated as:

$$S_p = \frac{\Delta F}{\Delta L} \tag{6}$$

where S_p is a function of ΔL.

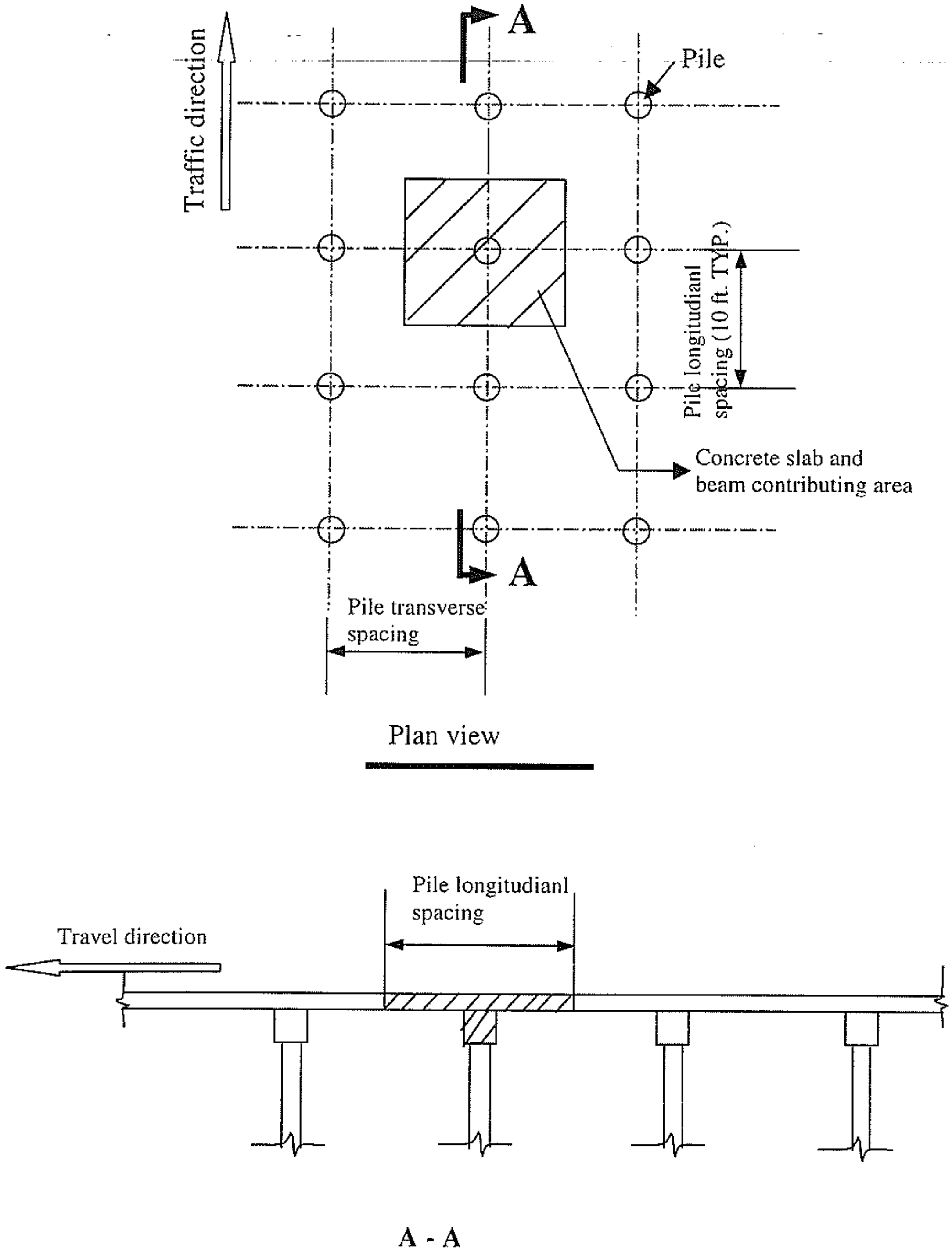


Figure 50
Pile head load

APPLICATION OF PROPOSED METHOD

For a given embankment height, the desired settlement profile could be achieved by selecting piles of variable lengths. To calculate the estimated approach slab settlement profile, a strip of the approach slab is selected along the longitudinal direction that includes one pile from each transverse row of piles. The design pile head settlement for each pile of a given length is obtained from the chart of pile head displacement. After settlement of each single pile in the longitudinal set of piles has been determined, the approach slab settlement profile could be obtained by plotting each pile head settlement at each pile location. If the calculated approach slab settlement profile is not close to the desired or ideal shape, such as that shown in Figure 54, the design parameters such as pile length, embankment fill height, etc. could be modified until an acceptable estimated profile is achieved.

This approach is demonstrated with typical examples using the data collected in the field study as described earlier in this report. Four of the representative sites were selected for evaluation based on the above procedure. Detailed data of these sites were given in an earlier section. These sites are:

- Site 1: I-310 elevated structure
- Site 5: I-310 Luling bridge south approach
- Site 7: LA 3139 Earhart Blvd (Orleans/Jefferson Parish line) bridge west approach, west bound.
- Site H: A hypothetical site by assuming extra 20 ft (6.10 m). soil stratum underlaying Site 1 bottom stratum.

These sites were selected because of the availability of relatively more comprehensive soil data. In addition, exact field settlement data was available from in-situ tests and surveys.

The subsoils settlement distribution curve for each site was computed using the embankment settlement software developed by DOTD *[26]*, and each pile stiffness was calculated by using a spreadsheet computer program TU-DRAG developed at Tulane University and the DOTD pile capacity program *[28]*. Only weight of the embankment (fill) and surcharge loads and duration were considered in the settlement analyses. The pile head load was considered as the weight of the approach slab strip and the pile cap (beam) (fig. 50).

VERIFICATION EXAMPLES

EXAMPLE 1 ---- SITE 1 - I-310 ELEVATED STRUCTURE

Details of this site are given in the earlier section of this report. The required soil properties at this site are listed in Table 21. These properties were obtained from the soil boring made at this site by Gore Engineering, Inc. as part of this study.

The properties of the structure at this site are as listed below:

Piles = timber/driven
Pile butt diameter = 12 in (304.8 mm)
Pile tip diameter = 8 in (203.2 mm)
Average pile diameter = 10 in (254mm)
Surcharge period = five months
Uniform surcharge fill height = 3 ft (0.91 m)

Existing pile length and embankment fill height at each pile position are tabulated in Table 22. The 9 ft (2.74m) uniform embankment cross section is approximately as shown in Figure 51. This information was obtained from the design/construction documents available at the DOTD offices which was collected during the study.

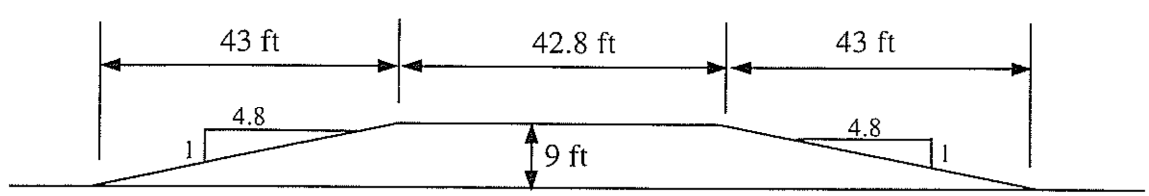


Figure 51

Site 1 approximate embankment dimensions

Table 21

Site 1 soil properties

Soil Stratum No.	Stratum Thickness (ft.)	Unit Weight (pcf), γ	Initial Void Ratio, e_0	C_c	P_{max}* (tsf)	C_V $1x10^{-5}$ $in.^2/sec$	Average Cohesion (psf), c
1	4	99.7	1.213	0.828	0.08	2.95	595
2	5	89.7	1.506	0.63	0.11	6.00	202.5
3	5	89.7	1.506	0.63	0.18	6.00	202.5
4	5	70.0	1.200	0.00	0.22	345.26	0
5	5	101.8	1.197	0.432	0.28	15.36	247.5
6	5	101.8	1.197	0.432	0.38	15.36	247.5
7	5	103.3	1.277	0.432	0.48	15.36	277.5
8	5	103.3	1.277	0.432	0.58	15.36	277.5
9	5	102.2	1.094	0.414	0.68	17.02	392.5
10	5	102.2	1.094	0.414	0.78	17.02	392.5
11	0.5	110.0	0.851	0.00	0.84	345.26	0
12	7.5	104.3	1.197	0.477	0.92	12.06	460
13	2	111.9	0.673	0.216	1.02	73.18	275
14	3	123.6	0.584	0.234	1.10	61.92	760
15	5.5	123.6	0.448	0.27	1.89	45.51	2260
16	3	112.5	0.788	0.513	1.35	10.07	1550
17	3	117.5	0.682	0.45	1.43	13.91	560

*P_{max} is the maximum past pressure
C_c is the compression index
C_v is the coefficient of consolidation
Metric Equivalents:
1 ft = 0.3048 m
1 pcf = 16.02 kg/m^3
1 tsf = 10.76 ton/m^2
1 in^2/sec = 6.451 cm^2/sec
1 psf = 4.8827 kg/m^2

Table 22

Site 1 Pile length and embankment fill height at each pile position

Element	Bridge Abutment	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9	Roadway
Pile length (ft)	N/A	60	52	44	37	31	25	20	17	15	N/A
Fill Height (ft)	9	9	9	9	9	9	9	9	9	9	9

Metric Equivalent:
1 ft = 0.3048 m

The calculated embankment foundation soil settlement curve was obtained using DOTD embankment settlement program *[26]* and is shown in Figure 52. Detailed results of the analysis are given in Appendix D.

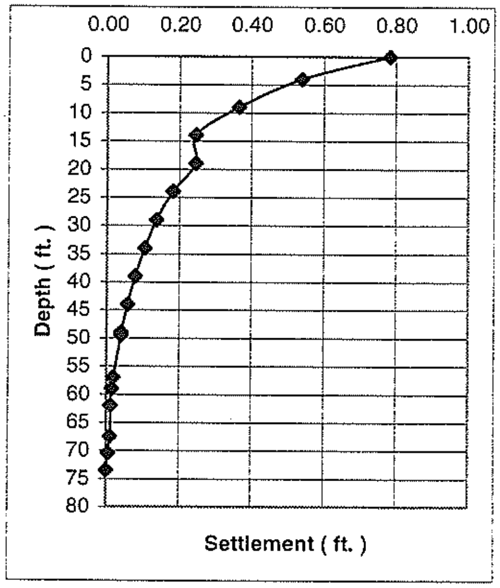


Figure 52
Site 1 calculated soil settlement curve

Pile head load (fig. 50).

Concrete slab: (10/12)×6.25×10×150/1000	= 7.81kips (3.54 tons)
Concrete beam: 2×2×6.25×150/1000	= 3.75 kips (1.70 tons)
Total load:	11.56 kips (5.24 tons)

Based on the above calculations, a maximum design pile head load of 11.56 kips (5.24 tons) was used in the analysis.

A comparison between measured and predicted settlement profiles of the northbound and southbound approach slabs are plotted in Figure 53. The predicted profile was obtained by the spreadsheet program (TU-DRAG). Appendix C lists the program. The predicted settlements are generally less than the measured values. Settlement of the piles are calculated independently without taking into account the effect of the approach slab stiffness.

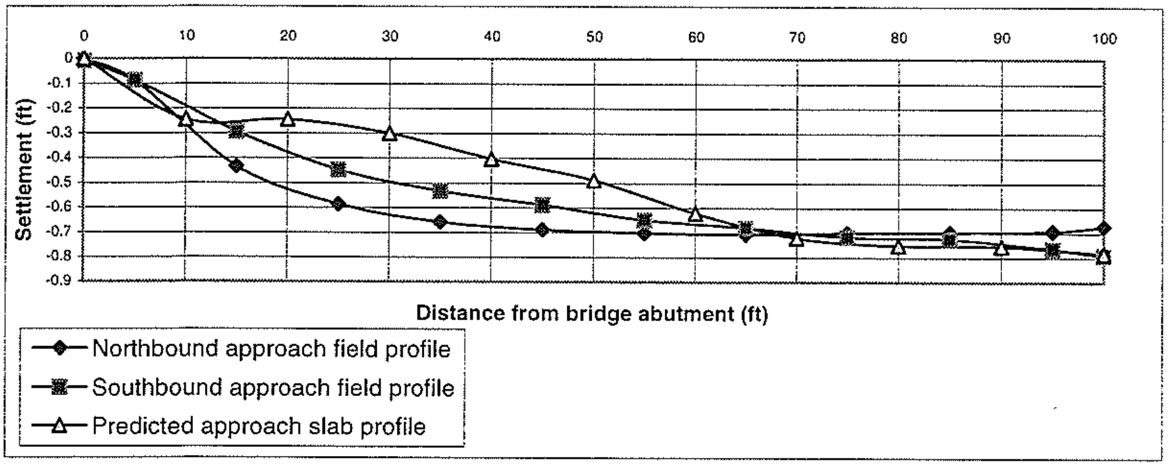


Figure 53
Measured and predicted approach slab settlement for Site 1

By varying the length of piles along the longitudinal approach slab profile, in a trial-and-error process, the desired approach slab settlement profile can be obtained. Based on the results of such a parametric study, it was determined that the pile length arrangement which yields a close agreement with the ideal curve for site 1 should be as that of tabulated in Table 23.

Table 23

Required length of each transverse row of piles

Pile Row No.	Distance from Abutment (ft)	Required Pile Length (ft)			
		Exact	Approximate	Actual	Required increase %
1	10	81	80	60	33.3
2	20	80	80	52	53.8
3	30	72	70	44	59.1
4	40	47	45	37	21.6
5	50	38	40	31	29.0
6	60	30	30	25	20.0
7	70	26	25	20	25.0
8	80	23	25	17	47.1
9	90	14	15	15	0
Total			410	301	36.2

* Each transverse row contains 9 piles
Metric Equivalent: 1 ft = 0.3048 m

Figure 54 illustrates the ideal settlement and calculated settlement curves based on the approximate pile lengths listed in Table 23. As shown in Figure 54, the estimated design profile is in close agreement with the hypothetical ideal profile. This particular profile should offer the desired smooth transition between the bridge and roadway.

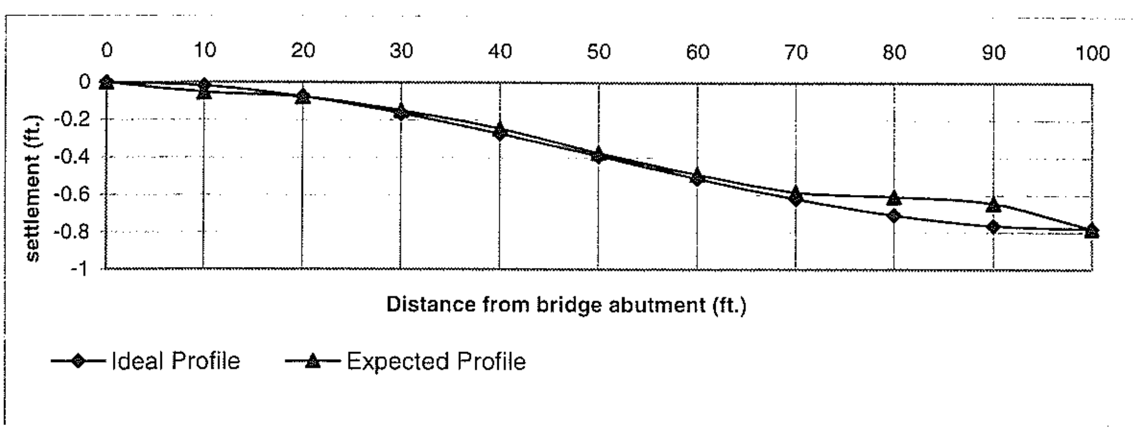


Figure 54

Ideal profile and calculated profile

EXAMPLE 2 ---- SITE 5 - LULING BRIDGE SOUTH APPROACH

Details of this site are given in the earlier section of this report. Soil properties at this site are listed in Table 24. The properties of the structure are as listed below:

Piles = timber/driven
Pile butt diameter = 12 in (304.8 mm)
Pile tip diameter = 8 in (203.2 mm)
Average pile diameter = 10 in (254mm)
Embankment height = Varies from 9 ft (2.74 m) to 12 ft (3.66 m)
Surcharge period = twelve months

Design pile length and embankment fill height at each pile position along the longitudinal profile of the approach slab are tabulated in Table 25. The variable height embankment cross section is approximately shown in Figure 55.

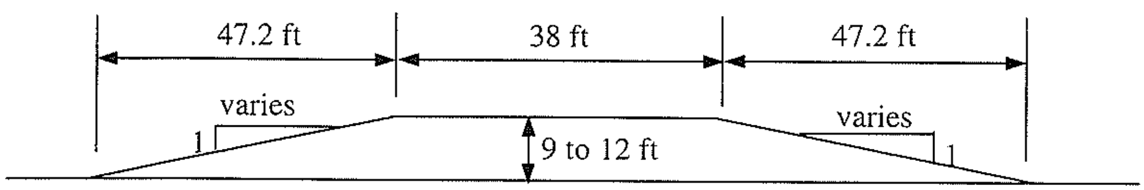


Figure 55

Site 5 approximate embankment dimensions

Table 24

Site 5 soil properties

Soil Stratum No.	Stratum Thickness (ft.)	Unit Weight (pcf)	Initial Void Ratio	C_c	PMAX (tsf) {PMAX}	C_V $1x10^{-5}$ in.^{2}/sec	Average Cohesion C (psf)
1	3	106.4	1.311	0.45	0.16	13.91	587.5
2	3.5	99.7	1.457	0.783	0.23	3.41	435.0
3	3	112.3	0.908	0.36	0.33	23.70	500.0
4	5	110.0	0.951	0.00	0.27	345.26	0.0
5	7.5	93.5	1.836	1.071	0.44	1.48	540.0
6	3.5	85.3	2.429	1.08	0.86	1.45	1022.5
7	2.5	110.0	0.739	0.00	0.51	345.26	0.0
8	5	102.0	1.039	0.378	0.59	21.13	367.5
9	10	105.5	1.019	0.36	0.75	23.70	297.5
10	10	110.8	0.852	0.288	0.98	39.46	317.5
11	8	110.8	0.852	0.288	1.19	39.46	317.5
12	2	94.4	1.600	0.864	1.31	2.63	727.5
13	6.5	122.9	0.479	0.324	1.71	30.24	1812.5
14	1.5	122.9	0.479	0.324	1.72	30.24	1812.5
15	7	113.7	0.700	0.495	1.66	11.01	1335
16	8.5	116.7	0.891	0.522	2.05	9.65	2140
17	6	106.9	0.819	0.603	2.04	6.71	1312.5
18	0.5	110	0.742	0.00	2.12	345.26	0
19	5	112.7	0.938	0.603	2.18	6.71	782.5

*P_{max} is the maximum past pressure
C_c is the compression index
C_v is the coefficient of consolidation

Metric Equivalents:

1 ft = 0.3048 m
1 pcf = 16.02 kg/m^3
1 tsf = 10.67 ton/m^2
1 in^2/sec = 6.451 cm^2/sec
1 psf = 4.8827 kg/m^2

Table 25

Site 5 Pile length and embankment fill height at each pile position
(Metric Equivalent: 1 ft = 0.3048 m)

Element	Bridge Abut	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9	Pile 10	Pile 11	Road way
Pile length (ft)	N/A	54	49.5	45	40.5	36	31.5	27	22.5	18	13.5	9	N/A
Fill Height (ft)	12	11.8	11.6	11.3	11.1	10.8	10.6	10.3	10.1	9.9	9.7	9.5	9.2

The calculated embankment settlement curves at the beginning and end of the approach slab are shown in Figure 56. Detailed results of the analyses are attached in Appendix D.

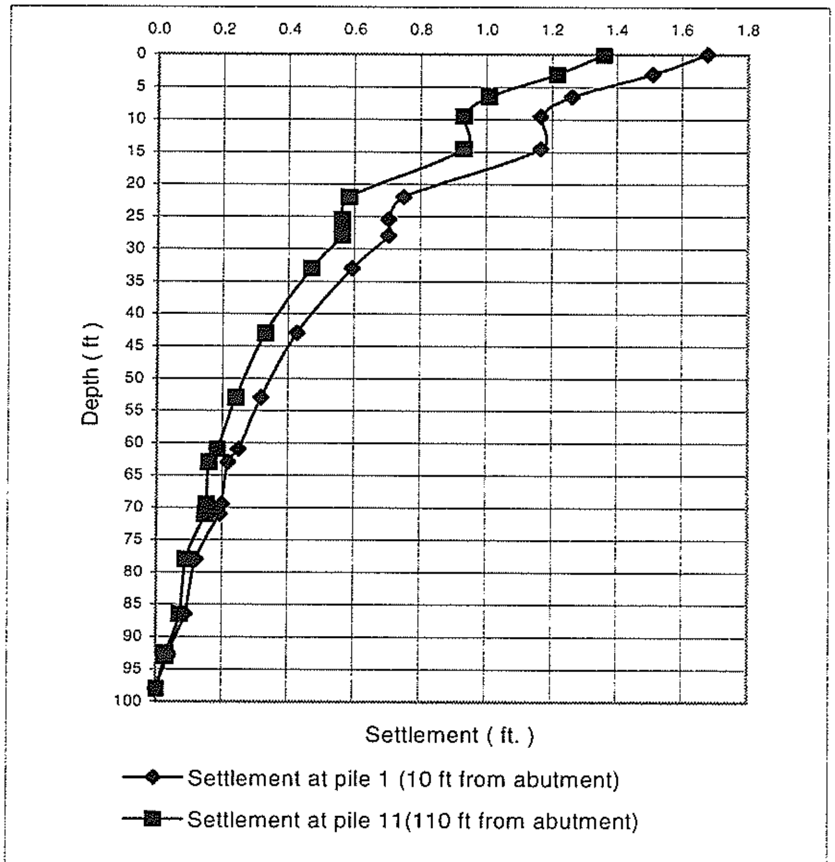


Figure 56
Site 5 calculated soil settlement curve

Pile head load (fig. 50).

Concrete slab: (10/12)×8.75×10×150/1000	= 10.94 kips (4.96 tons)
Concrete beam: 2×2×8.75×150/1000	= 5.25 kips (2.38 tons)
Total load:	16.19 kips (7.34 tons)

The maximum design pile head load was selected to be 16.19 kips (7.34 tons).

A comparison between measured approach slab settlement and predicted settlement profiles of the approach slab is plotted on Figure 57. The shape of both settlement profiles are quite similar, except at the first pile position near the abutment. It should be noted that settlement of the piles are calculated independently without taking into account the effect of the approach slab stiffness. Therefore, actual settlement of the first row of piles that follow the abutment may be overestimated by the program since they would actually be influenced by the stiffness of the slab.

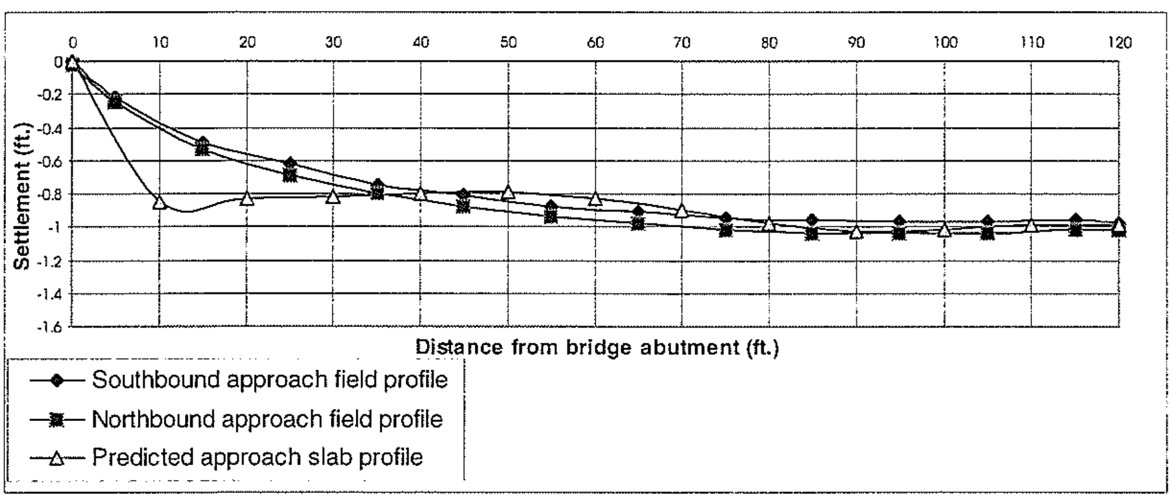


Figure 57
Measured and predicted approach slab settlement profiles at Site 5

Table 26

Required length of each transverse row of piles

Pile Row No.	Distance from Abutment (ft)	Required Pile Length (ft)			
		Exact	Approximate	Actual	Required increase %
1	10	108	110	54	103.7
2	20	108	110	49.5	122.2
3	30	105	105	45	133.3
4	40	100	100	40.5	146.9
5	50	95	95	36	163.9
6	60	80	80	31.5	154.0
7	70	60	60	27	122.2
8	80	25	25	22.5	11.1
9	90	15	15	18	-16.7
10	100	12	10	13.5	-25.9
11	110	10	10	10	0
Total			720	347.5	107.2

* Each transverse row contains 11 piles

Metric Equivalent:

1 ft = 0.3048 m

From the parametric study, it was determined that the optimum pile length arrangement which yields the ideal curve for site 5 is as tabulated in Table 26. Figure 58 illustrates the ideal and calculated settlement profiles based on the calculated pile lengths listed in Table 26. Figure 58 also shows a close agreement between the estimated design profile and the ideal profile that should offer the smooth transition.

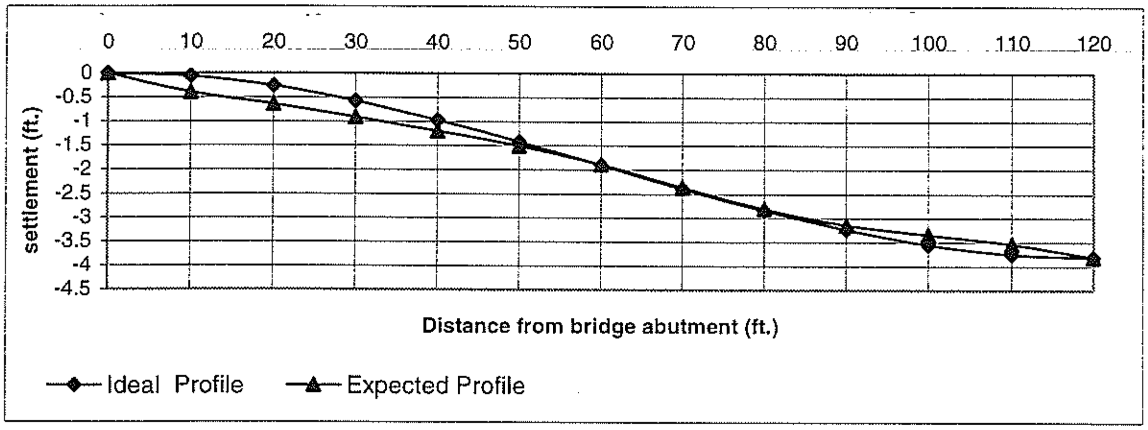


Figure 58
Ideal profile and calculated profile

EXAMPLE 3 ---- SITE 7 - LA 3139 (EARHART BLVD, PARISH LINE)

Details of this site are given in the earlier section of this report. Soil properties at this site are listed in Table 27. The properties of the structure at this site are as listed below:

Piles = timber/driven
Pile diameter = 12 inches (304.8 mm)
Pile tip diameter = 8 in (203.2 mm)
Average pile diameter = 10 in (254mm)
Surcharge period = three months

Existing pile length and embankment fill height at each pile position are tabulated in Table 28. The embankment cross section is approximately as shown in Figure 59.

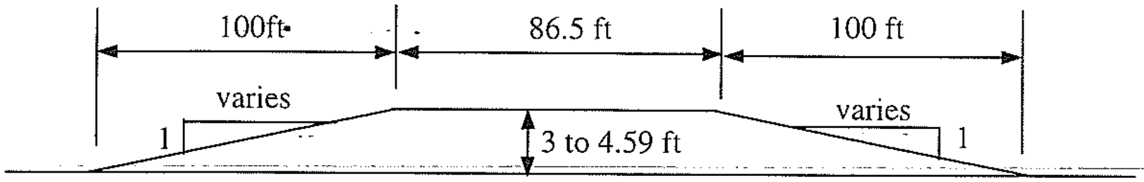


Figure 59
Site 7 approximate embankment dimension

Table 27
Site 7 soil properties

Soil Layer No.	Soil Thickness (ft.)	Unit Weight (pcf)	Initial Void Ratio	C_c	PMAX (tsf) {PMAX}	C_V 1x10^{-5} in.^{2}/sec	Average Cohesion C (psf)
1	3	87.7	0.783	0.36	1.16	23.70	1162.5
2	4	110.0	0.663	0.00	0.19	345.26	0
3	2	66.4	8.313	4.14	0.26	0.31	222.5
4	3.5	83.5	3.105	1.179	0.28	1.14	87.5
5	3.5	80.7	3.394	1.35	0.31	0.79	152.5
6	9	100.2	1.320	0.54	0.41	8.86	152.5
7	8	97.3	1.824	0.666	0.57	5.20	327.5
8	6.5	95.7	1.798	0.684	0.69	4.85	442.5
9	8.5	115	0.663	0.00	0.86	345.26	0
10	5.5	98.5	1.400	0.486	1.02	11.52	177.5
11	7	99.2	1.690	0.54	1.13	8.86	640.0
12	6	99.2	1.690	0.54	1.25	8.86	640.0
13	7	125.0	0.537	0.162	3.66	129.10	3660.0
14	4.5	110.0	0.663	0.00	1.58	345.26	0
15	2.5	115.6	0.668	0.162	1.67	129.10	335.0

*P_{max} is the maximum past pressure
C_c is the compression index
C_v is the coefficient of consolidation
Metric Equivalents:
1 ft = 0.3048 m
1 pcf = 16.02 kg/m^3
1 tsf = 10.76 ton/m^2
1 in^2/sec = 6.451 cm^2/sec
1 psf = 4.8827 kg/m^2

Table 28
Site 7 Pile length and embankment fill height at each pile position
(Metric Equivalent: 1 ft = 0.3048 m)

Element	Bridge Abut	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Roadway
Pile length (ft)	N/A	60	60	55	45	35	25	15	N/A
Fill Height (ft)	4.59	4.37	4.14	3.92	3.72	3.52	3.35	3.18	3

The calculated embankment foundation soil settlement curve is shown in Figure 60. Only settlement curves for piles 1 and 11 are plotted in Figure 60. Settlement curves for pile 2 through 10 fall between the two curves shown in Figure 60 and have similar shapes.

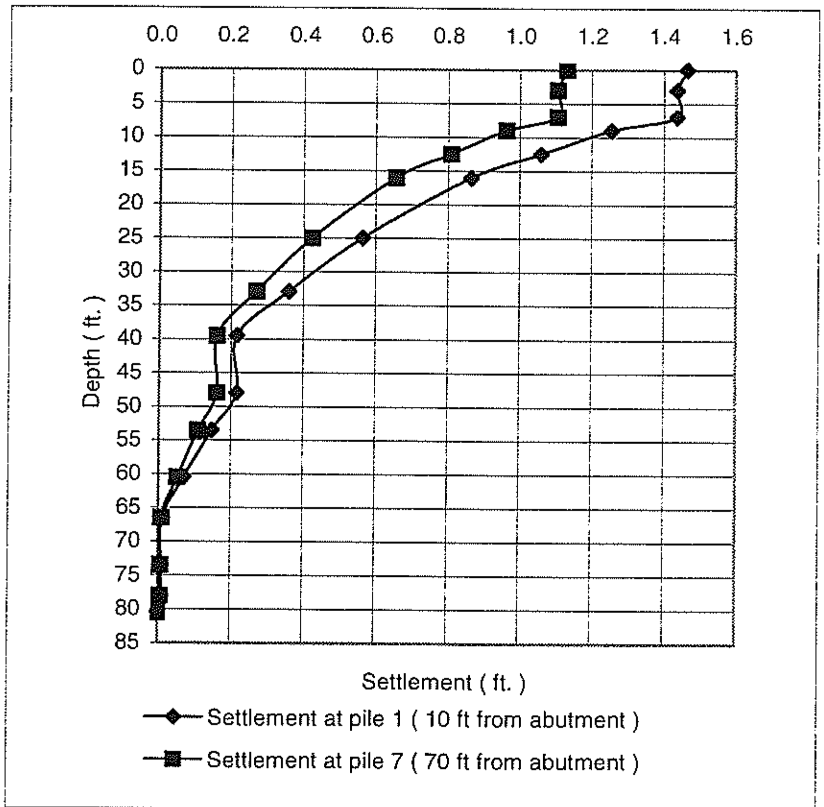


Figure 60
Site 7 calculated soil settlement curve

Pile head load (fig. 50).

Concrete slab: (10/12)×9.58×10×150/1000	= 11.98 kips (5.43 tons)
Concrete beam: 2×2×9.58×150/1000	= 5.75 kips (2.61 tons)
Total load:	17.73 kips (8.04 tons)

A maximum design pile head load of 17.73 kips (8.04 tons) was used in the analysis.

A comparison between measured and predicted settlement profiles of this site approach slab is plotted in Figure 61. Just as concluded in the previous two sites, the predicted curve was calculated using the simplified method which does not consider slab stiffness and assumes single free piles. As shown in Figure 62, the length of each pile used in the field was found to be inadequate.

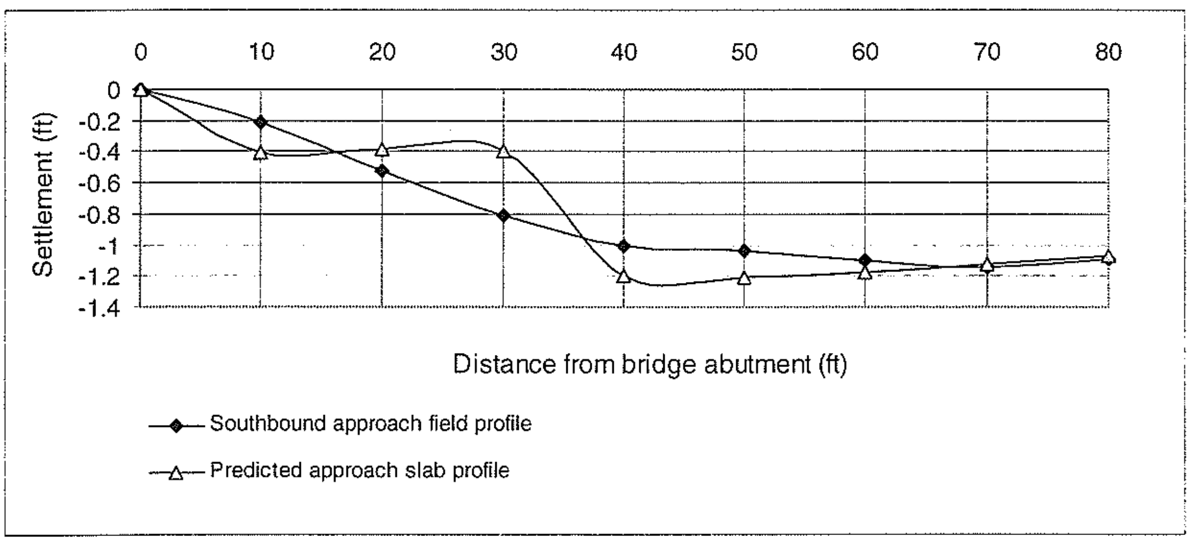


Figure 61
Measured and predicted approach slab settlement at Site 7

The optimum pile length arrangement which yields the ideal curve for site 7 was found to be as tabulated below in Table 29.

Table 29

Required length of each transverse row of piles at Site 7

Pile Row No.	Distance from Abutment (ft)	Required Pile Length (ft)			
		Exact	Approximate	Actual	Required increase %
1	10	83	80	60	33.3
2	20	83	80	60	33.3
3	30	65	65	55	18.2
4	40	57	55	45	22.2
5	50	48	50	35	42.9
6	60	46	45	25	80
7	70	16	15	15	0
Total			390	295	32.2

* Each transverse row contains 7 piles

Metric Equivalent:

1 ft = 0.3048 m

Figure 62 illustrates the ideal settlement curve and calculated settlement curve based on the pile lengths listed in Table 29.

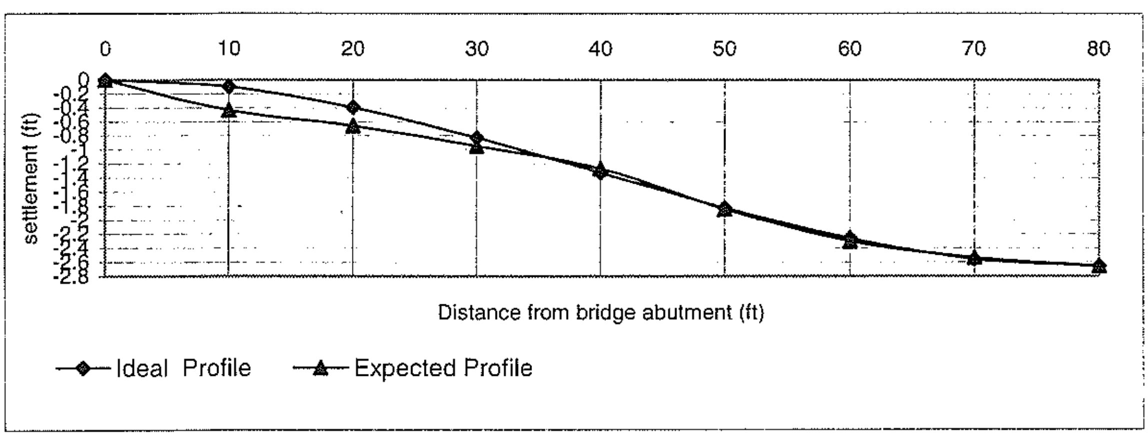


Figure 62
Ideal and calculated profile at Site 7

EXAMPLE 4.---- HYPOTHETICAL SITE

This site is a modified configuration of site 1 by extending the bottom soil stratum by an additional 20 ft. The purpose of selecting this hypothetical site is to examine if a deeper soft soil under the embankment would affect the estimated pile length determined earlier for Site 1. The embankment cross section is the same as Site 1.

Piles = timber/driven
Pile diameter = 12 inches (304.8 mm)
Pile tip diameter = 8 in (203.2 mm)
Average pile diameter = 10 in (254mm)
Surcharge period = five months
Uniform surcharge fill height = 3 feet (0.91 m)

The calculated embankment foundation soil settlement curve is shown in Figure 63. Detailed results of the analysis are given in Appendix D.

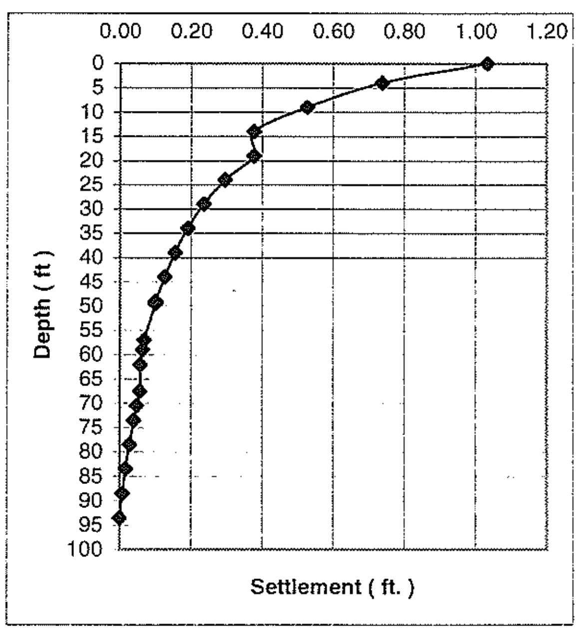


Figure 63
Hypothetical site calculated soil settlement curve

Maximum design pile head load is the same as that of Site 1 [28.4 kips (12.88 tons)]. For this site, the optimum pile length arrangement which yields the ideal curve was found to be as tabulated below in Table 30.

Table 30

Required length of each transverse row of piles at hypothetical site

Pile Row No.	Distance from Abutment (ft)	Required Pile Length (ft)			
		Exact	Approximate	Actual	Required increase %
1	10	100	100	60	66.7
2	20	80	80	52	53.8
3	30	72	70	44	59.1
4	40	50	50	37	35.1
5	50	40	40	31	29.0
6	60	32	30	25	20.0
7	70	26	25	20	25.0
8	80	22	20	17	17.6
9	90	10	10	15	-33.3
Total			425	301	41.2

* Each transverse row contains 9 piles

Metric Equivalent:

1 ft = 0.3048 m

COST BENEFIT ANALYSIS

The examples given in this section indicate that the piles used at the various sites do not have the required length to yield the ideal approach slab settlement profile. Table 31 shows a cost estimate of using longer piles to withstand downdrag compared to the actual lengths used in design. Unit prices for piles of different materials were based on the 1998 unit prices for DOTD projects. These unit prices are as follow:

Treated timber unit price (noncoastal treatment) = $9.23 per linear foot

Precast concrete pile (14" square) =$23.28 per linear foot

Price was not available for a 12 inch square concrete piles. Therefore, a cost of $17.10 was assumed for the purpose of analysis based on the ratios of the cross sectional areas of the 12 and 14 inch square piles.

Table 31

Cost Benefit Analysis

Site No.	Pile Material	Original Piles		Modified piles		Ratio
		Quantity (linear ft.)	Total Cost ($)	Quantity (linear ft.)	Total Cost (Dollars)	Longer Piles / Original Piles
Site 1	Treated timber pile	2,107	19,448	1,260	39,161	2.01
	Precast concrete pile			1,610		
Site 5	Treated timber pile	1,732.5	15,991	600	56,838	3.55
	Precast concrete pile			3,000		
Site 7	Treated timber pile	2,950	27,229	2,250	53,705	1.97
	Precast concrete pile			1.650		

Metric Equivalent:-

1 ft = 0.3048 m

The above table shows that additional cost of about 200 to 350 percent should be expected due to the required increase in pile length to offset drag load. However, it should be noted that significant savings could be achieved by improving the performance of bridge approach slabs. These costs include

- Inspection and maintenance costs.
- Costs of multiple overlays required to repair the approach slab during its service design life.
- Some approach slabs have performed so poorly that they had to be demolished and reconstructed before reaching their service design life.

Other indirect economical losses are also incurred because of the problem, but those are much harder to quantify. These costs include damage to vehicles and discomfort to drivers using the highway and economical losses incurred due to traffic delays experienced during repair or reconstruction. Some of the extremely poor approach slabs could also present a hazard to drivers who slow down or lose control of their vehicle due to these severe bumps. Therefore, it is our opinion that these costs would be generally higher than the cost of using longer piles. It should be noted that the proposed approach would not require any other costs or modifications to the existing DOTD design practice.

CONCLUSIONS

1. This research has identified and located about ninety bridges with pile-supported approach slabs across southeastern Louisiana. The identified sites included almost all of the pile-supported approach slabs in southeastern Louisiana except for those located in the Houma/Thibodeaux area where the approach slabs were constructed over lightweight aggregate fill (shell).

    - Seven representative sites were selected for through in-situ investigation and sampling. Performance of a given approach slab was assessed based on visual inspection, surveys and assessment of road surface conditions. Field instruments used included a walking profiler, Dynatest, laser profiler and geodetic total station. Soil borings and cone penetrometer tests were performed at three of the seven sites.

    - A rating system based on IRI values obtained from the laser profiler was developed and used to assess the condition of the sixty-three approach slabs. Even though this was not part of the original scope of the project, but this was developed because it offers a more accurate, consistent and objective method than a subjective rating system based on visual inspection. IRI rating system as developed for the approach slabs indicate that 4 percent of the slabs were in very good condition, 22 percent in good condition, 33 percent in fair condition, 22 percent in poor condition, and 19 percent in very poor condition.

    - These results of the study indicate that the standard design being used by DOTD for design of pile-supported approach slabs does not always produce acceptable field performance.

    - Data obtained from the walking profiler and geodetic survey was generally in good agreement. The walking profiler yields the necessary data for evaluating the performance of approach slabs. The Dynatest method can be used effectively to detect voids under approach slabs.

    - Factors such as speed limit, type of ramp, traffic count, etc. have no distinguishable impact on the settlement of approach slabs.

2. Based on evaluation of approach slab data and field evaluation and testing at the representative sites, it was concluded that the variable performance of pile-supported

approach slab is mainly due to differences in drag load and site conditions from one site to another.

3. Drag load could be accounted for by increasing the surcharge height and/or period, improving site conditions, or use of longer piles. It is also recommended that approach slab piles be driven only after allowing for a sufficient degree consolidation to occur under the weight of the surcharge. This process may require longer surcharge period or height.

4. A simplified soil/structure interaction procedure has been developed for the design of pile-supported approach slabs that accounts for specific site characteristics. The procedure takes into consideration effects of downdrag, embankment height, pile length, pile arrangement, and maximum allowed settlement to achieve an acceptable level of rideability. The predicted settlements were compared with those of the existing settlements of several approach slabs with good correlation. By varying the length of piles along the longitudinal approach slab profile in a trial and error process, the desired "ideal" approach slab settlement profile could be obtained.

5. A Microsoft Exel spreadsheet program with Visual Basic Application (VBA) macros has been developed for use in the parametric study of pile- supported approach slabs using a personal computer platform. The software accounts for downdrag in the selection of pile lengths and also takes into account various site conditions. The proposed methodology provides a pile-supported approach slab with an estimated settlement based on anticipated drag loads and specific site characteristics. This program input consists of pile characteristics and will accept the input from other computer programs directly involving pile load capacity with depth and soil and embankment settlement profile with depth. It will also accept data from appropriate hand calculations.

6. The collected information of one hundred and four identified sites, such as approach slab dimension, approach slab reinforcement, pile spacing, pile length, embankment dimensions, embankment material, soil conditions, etc. was compiled into a database LAPS for future use by DOTD, if so desired. The current condition ratings and maintenance records of the bridge sites located in the New Orleans district were also collected and recorded. This database was developed as part of this research study using Fox Pro software (Microsoft – 1985) and a personal computer platform.

RECOMMENDATIONS

1. The design procedure and computer program presented in this report could be used by DOTD engineers to design pile-supported approach slabs. The results should be compared with other ground improvement methods such as wick drains, sand drains, etc. A cost benefit analysis could be performed to determine the appropriate solution.

2. LTRC laser profiler could be used to access the conditions of the approach slabs in lieu of the visual rating system currently being used by DOTD.

3. Future research is needed to evaluate the proposed numerical model for the selection of pile lengths along pile-supported approach slabs. An approach slab could be designed using the spreadsheet then monitored over an extended period of time to evaluate its performance with time. This should include a thorough soil investigation of the subject site and settlement monitoring using settlement plates, surveys and IRI measurements.

4. Future research is needed to investigate soil modification techniques that could be used to improve or to reduce the effect of downdrag site conditions. These include use of lightweight aggregate, wick drains, cement-lime columns, bitumen coating of piles, ideal surcharge programs, etc.

LIST OF REFERENCES

1. Hopkins, T. C. "Long-term Movements of Highway Bridge Approach Embankments and Pavements," Research Report UKTRP-85-12, Kentucky Transportation Research Program, 1985, 149 pp.

2. TRB, NCHRP Synthesis of Highway Practice 234, "Settlement of Bridge Approches (The Bump at the End of the Bridge)," Transportation Research Board, National Research Council, Washington D. C., 1997, 75 pp.

3. Stewart, Carl F. "Highway Structure Approaches," Report No. FHWA/CA/SD-85-05, California Department of Transportation, Sacramento, CA, July 1985, 188 pp.

4. Bjerrum, L., Johnson, W., and Ostenfeld, C. "The Settlement of a Bridge Abutment on Friction Piles," Proceedings of the 4th International Conference of Soil Mechanics & Foundation Engineering, Vol 2, 1957, pp. 14-19.

5. Siriwardane, H.J., Moulton, L.K., and Chen, R-J., "Prediction of Lateral Movement of Bridge Abutments on Piles," in Transportation Research Record 998: Testing and Modeling Soils and Soil Stabilizers, Transportation Research Board, National Research Council, Washington, D.C., 1984, pp. 14-24.

6. Ardani, A. "Bridge Approach Settlement," *Report No. CDOH-DTP-R-87-06*, Colorado Department of Highways, Denver, April 1987. 53 pp.

7. Nowak, A.S., Nassif, H., and DeFrain, L. "Effect of Truck Loads on Bridges," *Journal of Transportation Engineering,* Vol. 119, November/December 1993, pp. 853-67.

8. Zaman, M., Gopalasingam, A., and Laguros, J.G. "Consolidation Settlement of Bridge Approach Foundation," *Journal of Geotechnical Engineering*, Vol. 117, No. 2, February 1991.

9. Greimann,L.F.,Yang, Pe-Shen, Wolde-Tinsae, and Amde M. "Nonlinear Analysis of Integral Abutment bridges," *Journal of Structural Engineering,* Vol. 112, No.10, October 1986, pp. 2263-2280.

10. Moumena, L., Wolde-Tinsae, A.M., and Aggour, M.S. "Analysis of Approaches to Bridges," Proceedings of the Second Workshop on Bridge Engineering Research in Progress, University of Nevada, Reno, Oct 29-30, 1990, pp. 283-286.

11. Tadros, M. and Benak, J. "Bridge Abutment and Approach Settlement, Phase 1," Nebraska Department of Roads, Dec. 1989.

12. McDonald, P. "Settlement of Fills on Soft Clay with Drains," Proceedings of the 11th International Conference on Soil Mechanics and Foundation Engineering, Vol. 4, San Francisco, August 12-16, 1985, pp. 2213-2216.

13. Hoover, Thomas P. "Caltrans' Wick Drain Experiences," Soil Improvement: a Ten Year Update; proceedings of a symposium", ASCE, New York, 1987, pp. 184-196.

14. Davie, J.R., Lewis, M.R., and Young, L.W. Jr. "Accelerated Consolidation of Soft Clays Using Wick Drains," Proceedings of the Second International Conference on Case Histories in Geotechnical Engineering, St. Louis, MO, June1-5.

15. Keene, P. "Tolerable Movements of Bridge Foundations," Transportation Research Record 678: Tolerable Movement of Bridge Foundations, Sand Drains, K-test, Slopes, and Culverts, Transportation Research Board, National Research Council, Washington, D.C., 1978, pp. 1-6.

16. Allen, D.L. "A Survey of the States on Problems Related to Bridge Approaches," *Report No. UKTRP-85-25*, Kentucky Transportation Research Program, Lexington, KY, October, 1985, 188 pp.

17. Holmberg, Soren. "Bridge Approach on Soft Clay Supported by Embankment Piles," *Geotechnical Engineering*, Vol. 10, 1978, pp. 77-89.

18. Broms,B.B. and Wong, I.H. "Embankment Piles," Soil Improvement Methods, Proceedings of the 3rd International Geotechnical Seminar, Nanyang Technological Institute, Singapore, 1985, pp. 167-178.

19. Bowles, Joseph E. Foundation Analysis and Design. McGraw-Hill, Inc., New York, NY, 1988, pp. 843-848.

20. Vesic, A.S., "Design of Pile Foundations," TRB, NCHRP Synthesis of Highway Practice 42, National Research Council, Washington, D. C., 1977, pp. 68.

21. Fang, Hsai-Yang. "Foundation Engineering Handbook," Chapman & Hall, New York, 1991.

22. Schutt, Mark. "Performance of Pile Supported Approach Slabs in Southeastern Louisiana.," M.S. Thesis, Tulane University, New Orleans, LA, April 1999.

23. Sayer M. W. "On the Calculation of IRI from Longitudinal Road Profile," Transportation Research Record 1501, Transportation Research Board, National Research Council, Washington, D.C., 1995, pp. 1-12.

24. Fellenius, B. H. "Unified design of piles and pile groups," Transportation Research Board, Washington, D. C., 1989, TRB Record 1169, pp. 75-82.

25. Held, L. A., Gilbert,L., Gwyn, W.W., “Non-structural Loads on Pile Foundations,” Presentation materials in New Orleans Branch, March, 1996.

26. Louisiana Department of Transportation and Development, “Manual of Embankment Settlement Analysis Program,” 1989

27. Hannigan, P. J., Goble, G. G., Thendean, G., Likins, G. E., and Rausche, F., “Design and Construction of Driven Pile Foundations,” FHWA Report, FHWA-HI-97-013, December 1996, pp. 9-136.

28. Louisiana Department of Transportation and Development, “Pile Capacity Guide,” 1989.

APPENDIX A

SELECTED COMPLETE SITE LIST

Table A.1
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
1	02 4190100811	419-01-14	14.644	LA3021	85	Elysian Fields Overpass		*
2	02 4190100812	419-01-14	14.644	LA3021	85	Elysian Fields Overpass		*
3	02 4503602456	450-36-0002	Blue print	I-310	92	0.2m North of US61	Ramps/US61	*
4	02 4503602567	450-36-0002	Blue print	I-310	92	0.2m North of US61	Ramps/US61	*
5	02 4503602646	450-36-0002	Blue print	I-310	92	0.1m North of US61	Swamp	*
6	02 4503603505	450-36-0002	Blue print	I-310	92	0.1m North of US61	I-310	*
7	02 4503602777	450-36-0002	Blue print	I-310	92	0.15m North of US61	Swamp	*
8	02 4503603388	450-36-0010	Blue print	I-310	92	I-310 Ramp over I-310 0.25m South of US61	Ramps/RR/Canal/US61	*
9	02 4503603158	450-36-0010	Blue print	I-310	92	0.25m South of US61	RR/Canal	*
10	02 4503603325	450-36-0010	Blue print	I-310	92	0.1m South of US61	RR/Canal	*
11	02 450360388	450-36-0010	Blue print	I-310	N/A	RR/Canal		*
12	02 4501400001	450-14-0002	B-4(4)	I-10	72	10010	Bonnet Carere Spillway	*
13	02 4501400002	450-14-0002	B-4(4)	I-10	72	02-45450140002	Bonnet Carere Spillway	*
14	02 4503605981	450-36-0006	Blue print	I-310	91	0.2m North of Luling Bridge	Pipeline	*
15	02 4503605982	450-36-0006	Blue print	I-310	92	0.2m North of Luling Bridge	Pipeline	*
16	02 4501503305	450-15-0068	Blue print	I-10	87	I-10 at Power Blvd		*
17	02 4501500452	450-15-0012	14.75(2,3)	I-10	71	10010	I-10 westbound over Loyola Ave	*
18	02 2830802441	283-08-0019	N/A	US90-B	N/A	Mississippi River/City		X

* = data compiled   XX = don't have approach slab   X = no microfilm or blue print

Table A.1 (Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
19	02 2830802442	283-08-0048	CD000002	US90-B	85	02-362830802442	Mississippi River GNO #	X
20	02 4300100182	430-01-0001	14.298(3,4)	LA3139	77	02-264300100182	I.C.G. RR	*
21	02 4300101201	430-01-0002	SM14.645	LA3139	86	1.2m East of Hickory Ave	LA3152 Ramps A B D &E	*
22	02 4300101568	430-01-0002	SM14.645	LA3139	86	Ramp C		*
23	02 4300101416	430-01-0008	14.298(3,4)	LA3139	78	ICC, G,RR		*
24	02 4300101511	430-01-0002	SM14.645(3,4)	LA3139	86	1.51 m East of Hickory Ave	St. Peter's Ditch	*
25	02 4300101358	430-01-0002	SM14.645(3,4)	LA3139	86	1.35 m East of Hickory Ave	Clearview Pkwy	*
26	02 4300101147	430-01-0008	14.298(3,4)	LA3139	78	Clearview Pkwy		*
27	02 4300101801	430-01-0002	SM14.645(3,4)	LA3139	86	1.8 m East of Hickory Ave	Central Ave. IC,KCS,SP,R	*
28	02 4300102821	430-01-0003	14.590(4)	LA3139	84	2.82 m East of Hickory Ave	LA South & KCS Railroad	*
29	02 4300104068	430-01-0004	14.591(1,2)	LA3139	83	4.06m East of Hickory Ave	Hwys, STS,RR,CA	*
30	02 4300104115	430-01-0004	14.591(1,2)	LA3139	83	4.11 m East of Hickory Ave	RR,CA	*
31	02 4300104581	430-01-0004	14.591(1,2)	LA3139	82	4.58 m East of Hickory Ave		*
32	02 4300104671	430-01-0004	14.591(1,2)	LA3139	82	4.67 m East of Hickory Ave	St., R/R	*
33	02 4300104672	430-01-0004	14.591(1,2)	LA3139	82	4.67 m East of Hickory Ave	STS., R/R	*
34	02 4300200001	430-01-0004	14.591(1,2)	LA3139	82	Jeff./Orleans Parish Line		*
35	02 4503606221	450-36-0007	14.578(1,2)	I-310	84	ICG,RR, Ramp E F &H, LA	Main street on/off, no settlement	*
36	02 4503800001	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		LA18 & M.P. RR	XX

* = data compiled   XX = don't have approach slab   X = no microfilm or blue print

Table A.1(Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
37	02 4503800055	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		MP RR	*
38	02 4503800058	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		MP RR	*
39	02 4503800206	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		MP RR	*
40	02 4503800207	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		Ramp "A"	*
41	02 4503800361	450-38-0004	14.438(2-4) 14.439(1)	I-310	79		Ramp "C" from I-310 SB Settlement	*
42	02 4503800362	450-38-0001	SM000865	I-310	79	Ramp "C" from I-310 SB	major settlement	*
43	02 4280310815	450-38-0001	SM000865	I-310	86	1.87m North of US90	LA 3127 & I-310	*
44	02 4503802411	450-38-0001	SM000865	I-310	86	1.57m North of US 90	I-310 over LA 3127	*
45	02 4503802412	450-38-0001	SM000865	I-310	86		I-310 over LA3127	*
46	02 4504300552	450-43-0056	Blue print	I-510	92	0.55m North of Gulf Outlet	RR/US90(Chef Hwy)	*
47	02 4504300581	450-43-0056	Blue print	I-510	92	0.58m North of Gulf Outlet	RR/US90(Chef Hwy)	*
48	02 4504300706	450-43-0056	Blue print	I-510	N/A		Railroad	*
49	02 4504300757	450-43-0056	Blue print	I-510	92	0.75m North of Gulf Outlet	Railroad	*
50	02 4504301026	450-43-0056	Blue print	I-510	N/A		Ground	*
51	02 4504301832	450-43-0057	Blue print	I-510	92	0.95m North of US90	City Streets	*
52	02 4504301851	450-43-0057	Blue print	I-510	92	1.0m North of US90	City Streets	*
53	02 4504302205	450-43-0057	Blue print	I-510	92		Ground	*
54	02 4504302278	450-43-0057	Blue print	I-510	92	1.27m North of US90	Ground	*
55	02 4504305341	450-16-0047	7-28(2)	I-510	67	LA0047	I-510(Paris Rd) over I-10	*

* = data compiled XX = don't have approach slab X = no microfilm or blue print

Table A.1 (Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
56	02 4504305342	450-16-0047	B-6(2)	I-510	67	LA0047	I-510(Paris Rd) over I-10	*
57	02 4504300241	450-43-0059	CD000022	I-510	92	0.24m North of Gulf Outlet	I-510	*
58	02 4504300242	450-43-0059	CD000022	LOC-RD	92	0.26m North of Gulf Outlet	I-510	*
59	02 4504301401	450-43-0057	Blue print	I-510	92	0.55m North of US90	City Streets	*
60	02 4504301402	450-43-0057	Blue print	I-510	92	0.55m North of US90	City Streets	*
61	03-5104240517891	424-05-0058	SM00.704	US90	87	Jct. US90 & LA317	US90 over LA317	*
62	03-5104240536941	424-05-0087	N/A	LA3052 (US90)	94	3.21m East of Morgan City	Bayou Ramos/LOC Road	X
63	03-5104240537191	424-05-0087	N/A	LA3052 (US90)	94	5.18m East of Morgan City	Texas Gas Pipeline	X
64	62-5304529000997	SP452-02-45	14.403(3)	I-55	79	Manchac Interchange	Ponchatoula-Manchac Ramp A	*
65	62-5304529001315	SP452-02-45	14.403(3)	I-55	79	Manchac Interchange	Ponchatoula-Manchac Ramp B	*
66	62-5304529001318	SP452-02-45	14.403(3)	I-55	79	Manchac Interchange	Ponchatoula-Manchac Ramp C	*
67	62-5304529000996	SP452-02-45	14.403(3)	I-55	79	Manchac Interchange	Ponchatoula-Manchac Ramp C	*
68	62-5304529000001	SP452-02-85	14.404(2)	I-55	77	North Bound on Elevated Roadway	South of Ponchatoula	*
69	62-5304529000002	SP452-02-85	14.404(2)	I-55	77	Ponchatoula-Manchac Past Manchac	South Bound off Elevated Roadway	*
70	07-1201930208531	SP193-02-24	14.281(1,2)	LA0027	76	ICWW Gibbstown	Pile Supported Approach Slab	*
71	07-1000310403151	SP31-04-03	13.31(1)	LA0027	79	Choupique Bayou		*
72	61-0304501100002	736-99-0454	N/A	Bayou Manchac	93		Concrete Overlay	X

* = data compiled XX = don't have approach slab X = no microfilm or blue print

Table A.1 (Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
73	61-0302860108102	736-99-0454	N/A	Bayou Conway	90	0.1m North of I-10 & LA22	Concrete Slab	X
74	61-0302680108101	736-99-0454	N/A	Bayou Conway	90	0.1m North of I-10 & LA22	Concrete Slab	X
75	02-2904120201201 829-28-04101	412-02-0016 412-02-0025	SM00.692 SM000864	Hollyw'd Canal	88	1.20 in South of Jct. LA 3087 US 90		*
76	02-2904120201202 829-28-04101	412-02-0016 412-02-0025	SM00.692 SM000864	Hollyw'd Canal	88	1.20 in South of Jct. LA 3087 US 90		*
77	02-554240718181	424-07-0011	14.494(3,4)	St. Louis Street	81	02-554240718181		*
78	02-554240718182	424-07-0011	14.494(3,4)	St. Louis Street	81	02-554240718182		*
79	4240716541	424-07-0013	14.495(1,2)		80	US 90 Bypass over LA 24		*
80	4240716542	424-07-0013	14.495(1,2)		80	US 90 Bypass over LA 24		*
81	4240800001	424-08-0013	14.495(1,2)	LA3052	80	US 90 Bypass over LA 316		*
82	4240800002	424-08-0013	14.495(1,2)	LA3052	80	US 90 Bypass over LA 316		*
83	4240806631	424-08-0012	14.442(2,3)	LA3052	80	US 90 Bypass over LA 3198		*
84	4240806632	424-08-0012	14.442(2,3)	LA3052	80	US 90 Bypass over LA 3198		*
85	4509008403	450-90-0050	CD000021	I-10	92	Louisa & Almonaster		*
86	4503600412	450-36-0001	CD000035	I-310	92	NB I-310 to Airline (US 61)		*
87	4503600401	450-36-0001	SM000880-1	I-310	92	SB I-310 from Airline (US 61)		*
88	2830904708	283-09-0070	Blue print		92	Barataria off ramp E/B		*
89	2830904595	283-09-0070	Blue print		92	WB on-ramp from Barataria Blvd.		*
90	006-3000001	006-02-0027	14.540	LA 48	82	Eastbank Traffic Circle		*

* = data compiled XX = don't have approach slab X = no microfilm or blue print

Table A.1 (Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
91	4503400001	450-34-0004	7.23(4),14.120	I-610	63	17th St Canal & I-10 WB		*
92	4503400481	450-34-0004	7.23,14.120(1)	I-610	63	Ponchartrain Blvd over I-610 & I-10		*
93	4503400781	450-34-0005	14.303(3,4) 14.304(1)	I-610	73	EB over Canal Blvd.		*
94	4503400782	450-34-0005	14.303(3,4) 14.304(1)	I-610	73	WB over Canal Blvd.		*
95	4503402121	450-34-0005	14.303(3,4) 14.304(1)	I-610	73	I-610 over Golfer's underpass		*
96	4503404078	450-34-0006	14.305(1,2)	I-610	75	I-610 (off ramp) to Elysian Fields		*
97	4503403816	450-34-0006	14.305(1,2)	I-610	75	I-610 West on-ramp from Elysian Fields		*
98	4503403615	450-34-0006	14.305(1,2)	I-610	75	I-610 EB on-ramp from Broad		*
99	4503400491	450-34-0004	7.23(3)	I-610	65	West End Blvd over I-610		*
100	4503400485	450-34-0004	7.23(3)	I-610	63	I-610 East on-ramp from Pontch. Blvd		*
101	4503401221	450-34-0005	14.303(3,4) 14.304(1)	I-610	73	Orleans Outfall Canal		*
102	4280311571	428-03-0001	14.246(3)	LA3127	75	LA 3127 @ 80 Arpent Canal		*
103	4501505711	740-00-0033	14.219	I-10	67	Clearview S.B.over I-10		*
104	4501505715	740-00-0033	14.219	I-10	67	Clearview S.B. off ramp to I-10 E		*
105	4501505718	740-00-0033	14.219	I-10	67	I-10 W.B. off ramp to Clearview S.B.		*
106	4501505731	740-00-0033	14.219	I-10	67	Clearview N.B. over I-10		*

* = data compiled　　XX = don't have approach slab　　X = no microfilm or blue print

Table A.1 (Continued)
Selected complete site list.

Site No.	Structure No.	Project No.	Roll No.	Route #	Year	Location	Description	Status
107	4501505736	740-00-0033	14.219	I-10	67	NB Clearview off ramp to I-10 W		*
108	4501505737	740-00-0033	14.219	I-10	67	I-10 EB off-ramp to NB Clearview		*
109	4501503956	740-00-0033	14.219	I-10	67	Canal #3		*
110	006-3000002	006-02-0018	1.24,1.25	LA. 48	61	Jefferson Hwy @ Clearview Pkwy(EB)		*
111	4501505051	740-00-0034	8.49(4),8.50(1)	I-10	67	Transcontinental drive		*
112	4501505032	740-00-0034	8.49(4),8.50(1)	I-10	67	Transcontinental Blvd		*

* = data compiled or blue print   XX = don't have approach slab   X = no microfilm

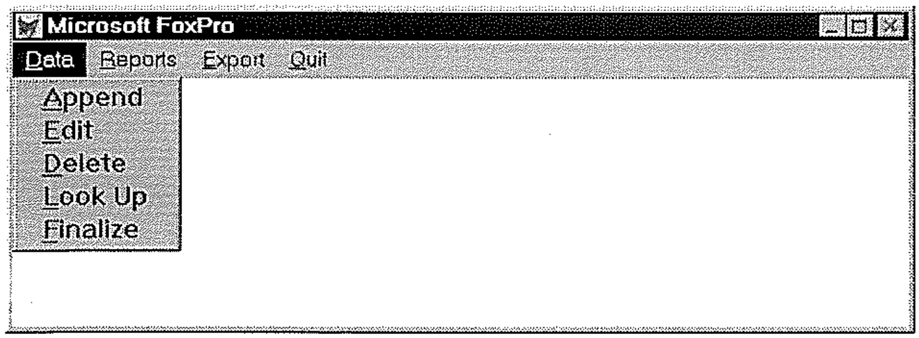
APPENDIX B

USER'S MANUAL FOR LAPS DATABASE

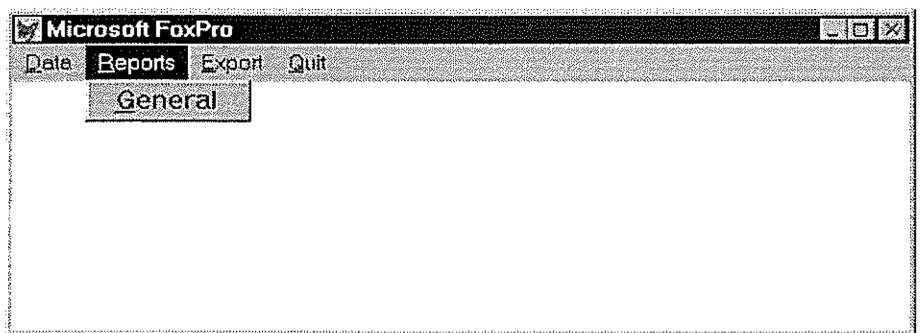
B.1 MAIN MENU SYSTEM

When the database for Louisiana bridge approach slabs *LAPS* is loaded by running the stand-alone executable file laps.exe, a main menu appears on the screen. Figure B.1 shows the main menu with each of its menu pads open one at a time.

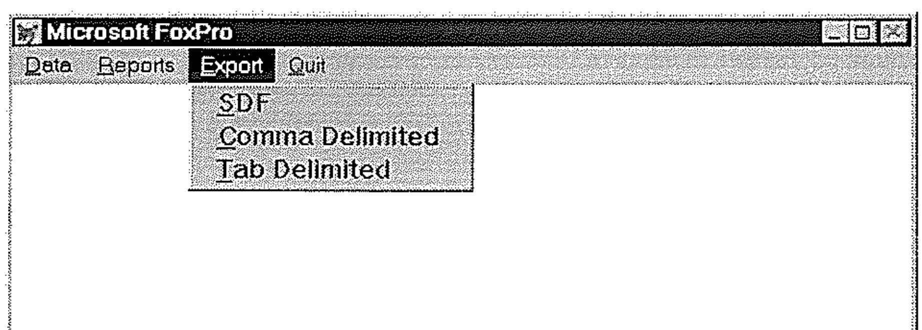
As shown in Figure B.1, the main menu has four menu pads. They are the Data menu, Reports menu, Export menu and Quit Menu.



(a). Menu screen with Data menu open.



(b). Menu screen with Report menu pad open.



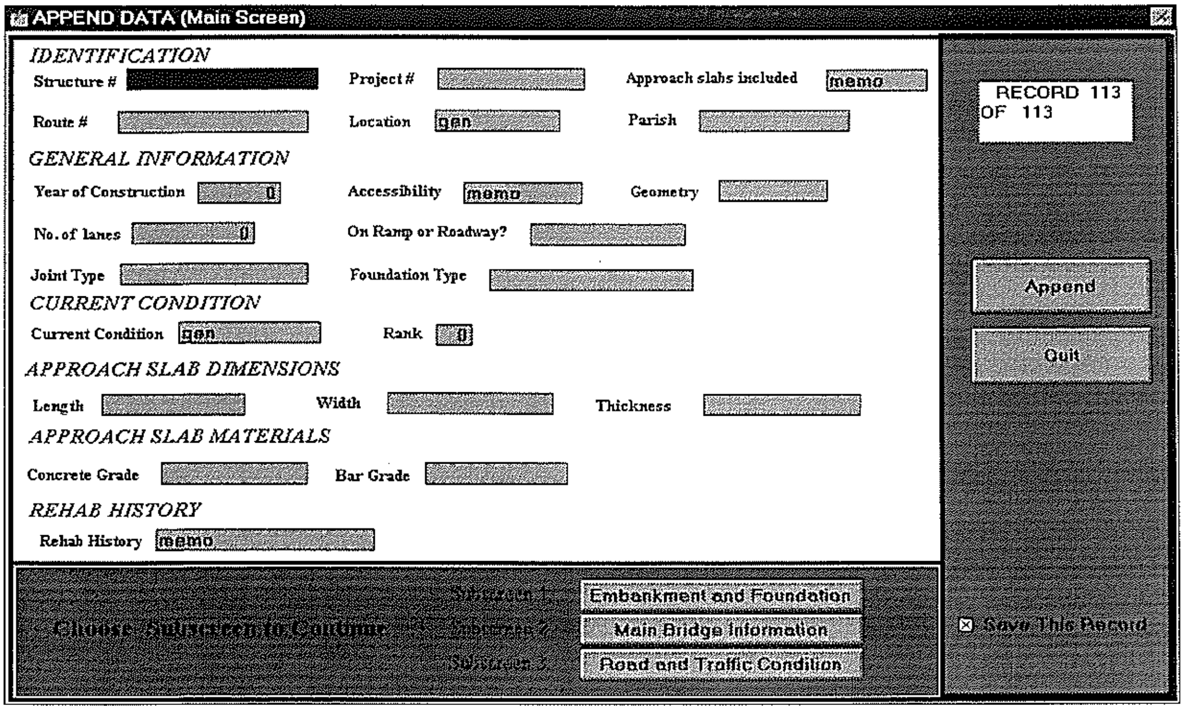
(c). Menu screen with Export menu pad.

Figure B.1.
Main Menu Screen Design.

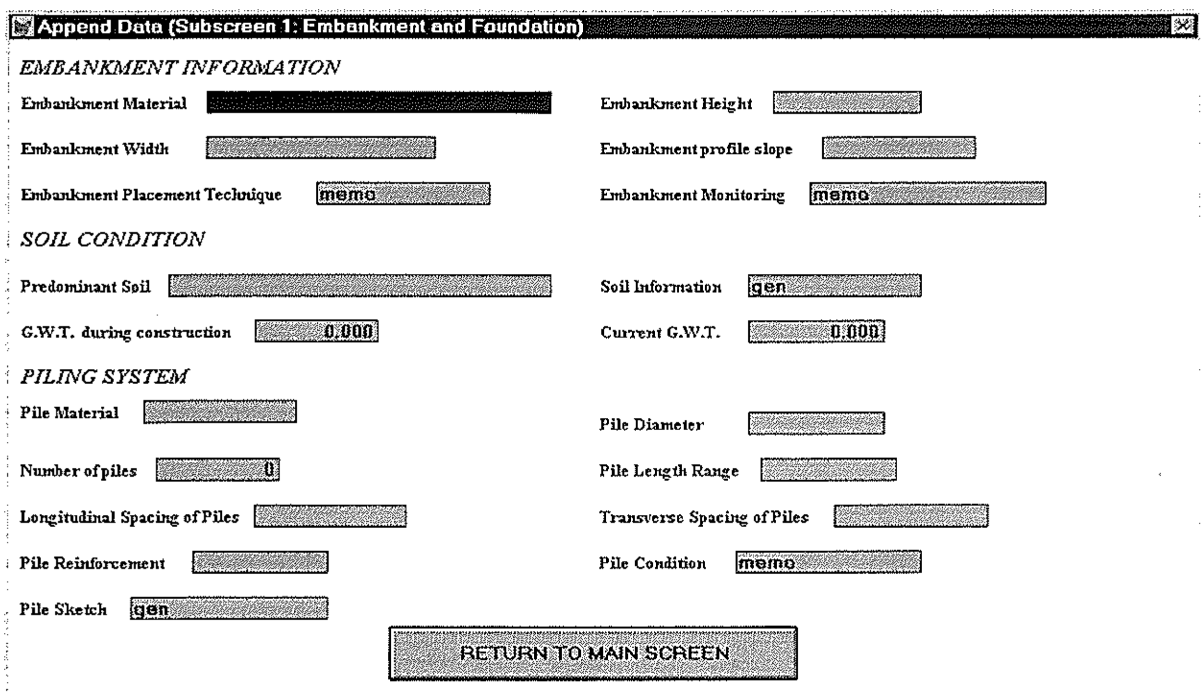
B.2 DATA MENU PAD

The data menu pad includes Append, Edit, Delete, Look up and Finalize options.

Append Option: When the option Append is chosen, a popup screen named "APPEND DATA (Main Screen)" appears so that the user can append a new record to the database. Figure B.2 shows this "APPEND DATA (Main Screen)" and its three subscreens which are named "Embankment and Foundation", "Main Bridge Information Records" and "Road and Traffic Condition Records" respectively. Once open, all fields in a given screen appear empty and ready to be entered. The user can navigate easily between the main screen and its subscreens by clicking on either the desired subscreen button or the "RETURN TO MAIN MENU" button. To exit from the "APPEND DATA (Main Screen)", the user has to click on the "Quit" button.

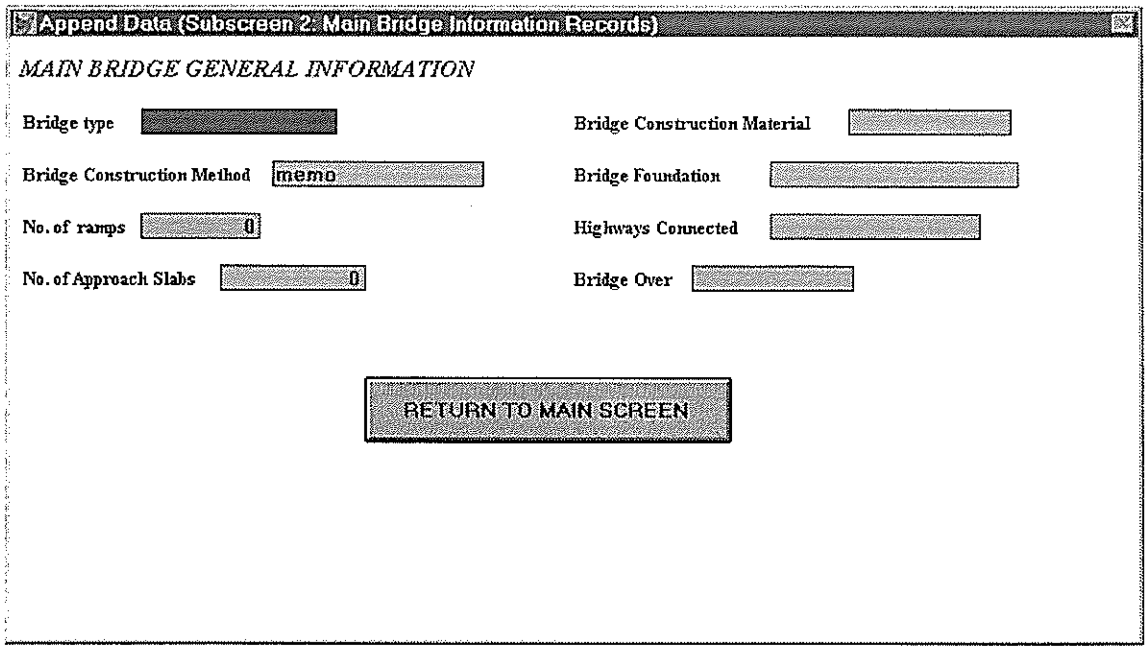


(**a**). Main Append Screen.

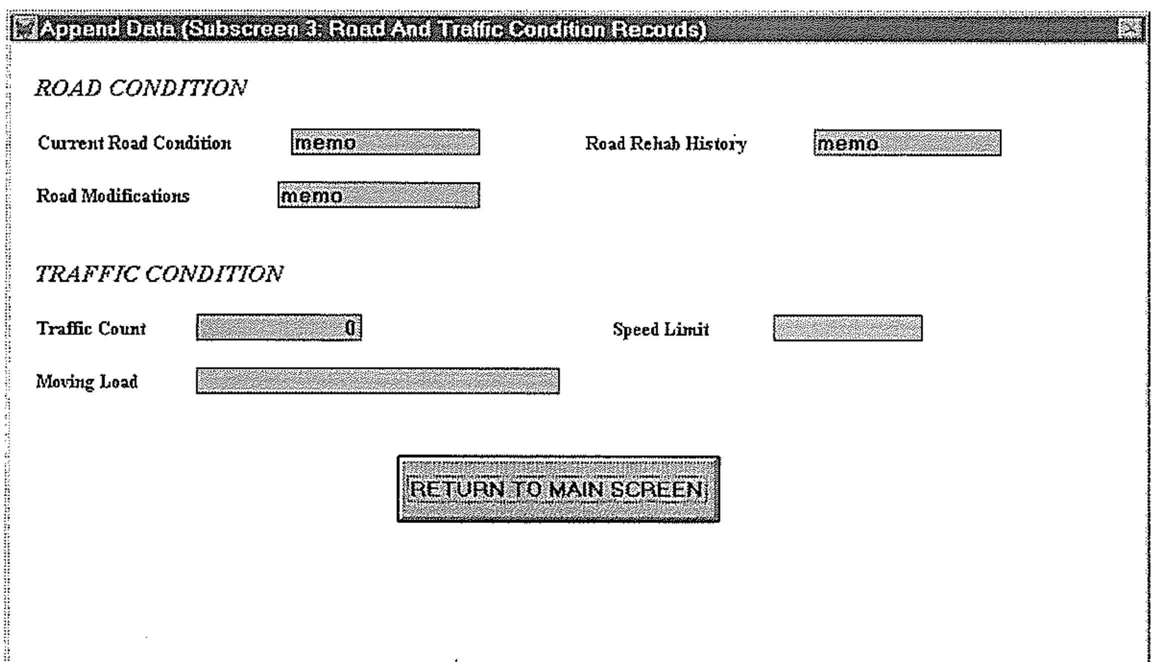


(**b**). Append Data Subscreen 1 "Embankment and Foundation".

Figure B.2.
Append Data Screen Design



(**c**). Append Data Subscreen 2 "Main Bridge Information Records".



(**d**). Append Data Subscreen 3 "Road And Traffic Condition Records".

Figure B.2 (Continued).
Append Data Screen Design

Edit Option: When the option Edit is chosen, a popup search (fig. B.3) screen appears requesting the user to input the structure number to load its records. The format of the structure number is 99-9999999999. Once the keyed structure number matches one of the records in the database, the edit data screen for that record appears so that the user can edit, modify or add new information to the record. Figure B.4 shows this "Edit Data (Main Screen)" and its three subscreens entitled "Embankment and Foundation", "Main Bridge Information Records" and "Road and Traffic Condition Records" respectively. The fields for an existing record in all screens are already entered, but can be edited. The main screen and its subscreens can be navigated across freely by clicking on either the requested subscreen button or the "RETURN TO MAIN MENU" button. To exit from the "APPEND DATA (Main Screen)", the user needs to click on the "Quit" button.

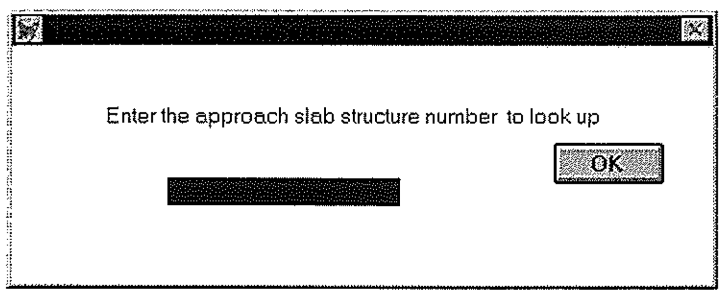
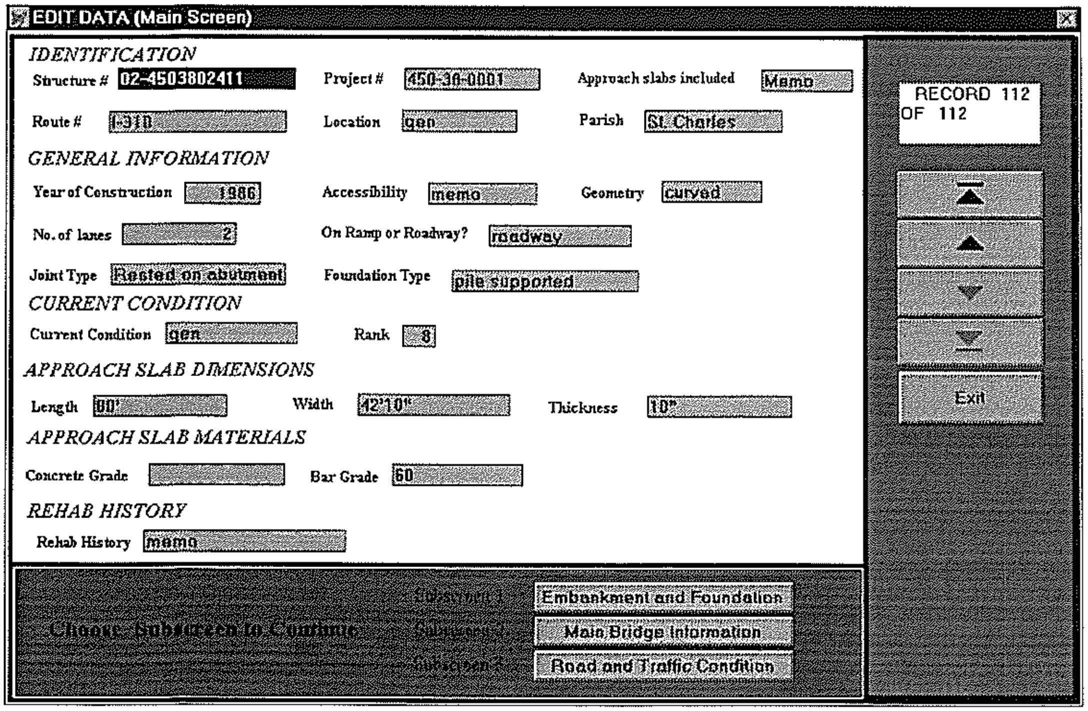
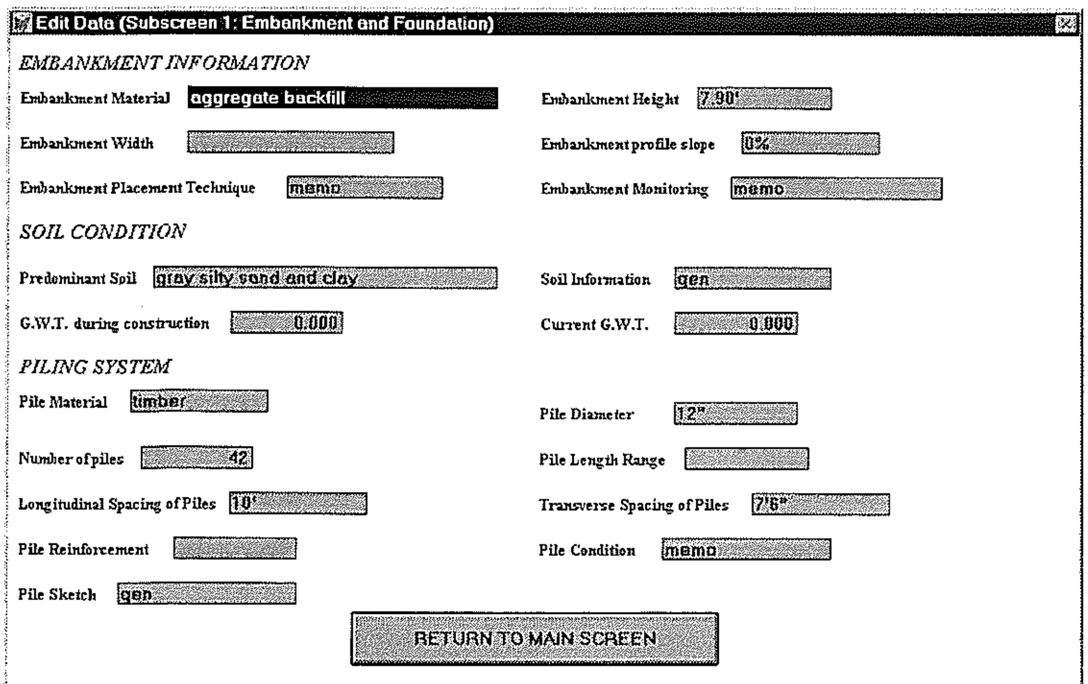


Figure B.3.
Lookup pop up screen when performing data edit.

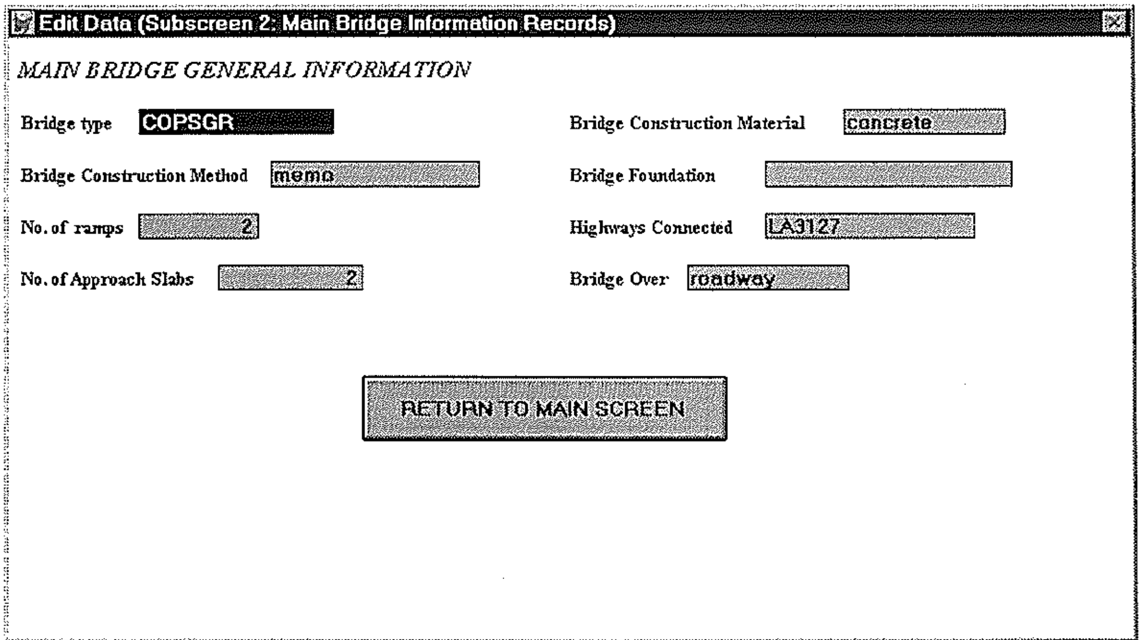


(a). Main Edit Screen.

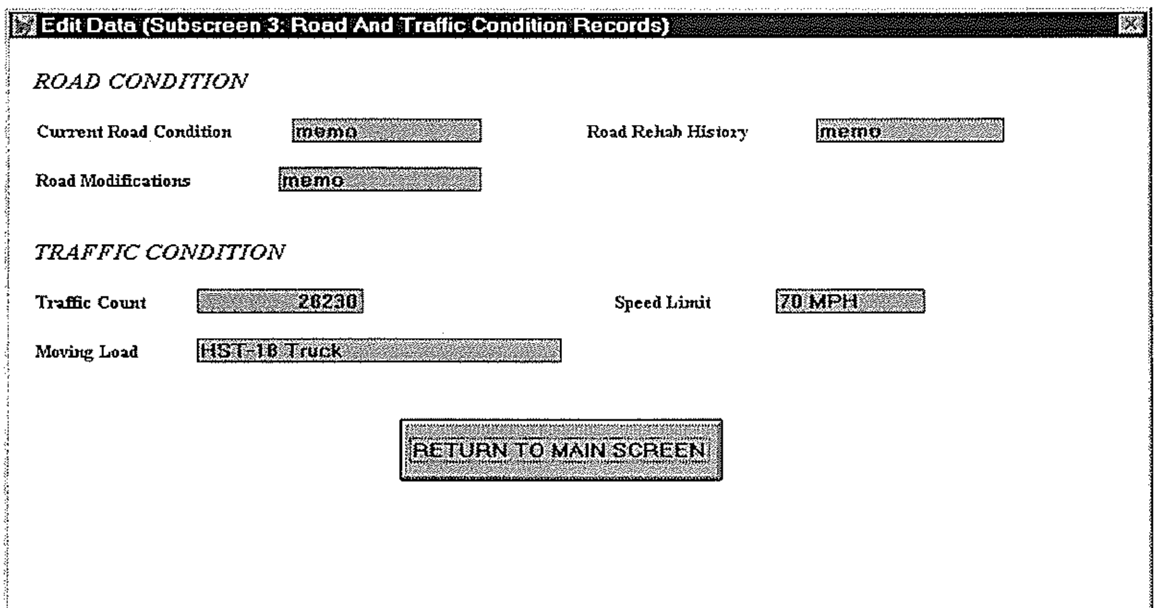


(b). Edit Data Subscreen 1 "Embankment and Foundation".

Figure B.4
Edit Data Screen Design



(**c**). Edit Data Subscreen 2 "Main Bridge Information Records".



(**d**). Append Data Subscreen 3 "Road And Traffic Condition Records"

Figure B.4 (Continued).
Edit Data Screen Design.

Delete option: When this option is chosen, the popup lookup screen shown in Figure B.3 appears, requesting the structure number whose record is to be deleted. Once this record is loaded, it can be removed from the database.

Finalize option: The purpose of a Delete option is to permanently delete a selected record from the database.

Search option: This option is designed to provide the user with a way to access a specific record in the database. When the Search option is chosen, the popup screen shown in Fig. B.3 appears. After entering the structure number for the desired approach site, the information of this site can be retrieved.

B.3 REPORTS MENU PAD

Since it is impractical to print all the information in such a long database, this menu pad is designed to provide a printing facility for the information contained in the database *LAPS*. The General option in this Reports menu pad is an example which lists the general information for the approach slabs contained in *LAPS*. Figure B.5 shows a sample general report. A report with specific inquires can be designed and entered into the Reports menu pad.

B.4 EXPORT MENU PAD

This menu pad has SDF, Comma Delimited and Tab Delimited options. When the option SDF is chosen, all data will be exported to a simple text file named export.txt. When the option Comma Delimited is chosen, all data in *LAPS* will be exported to a comma delimited type text file. Similarly, when the Tab Delimited option is chosen, all data in *LAPS* will be exported to a Tab type delimited text file.

B.5 QUIT MENU PAD

When Quit menu pad is chosen, the main menu will be closed.

APPENDIX C

TU-DRAG PROGRAM

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
BATON ROUGE - LOUISIANA

PROGRAM:	TU-DRAG A COMPUTER PROGRAM FOR DETERMINING PILE LENGTHS CONSIDERING DOWNDRAG FOR USE IN DESIGN OF PILE-SUPPORTED BRIDGE APPROACH SLABS
VERSION:	1.00 1999
DEVELOPED BY:	DEPARTMENT OF CIVIL AND ENVIRONMENTAL ENGINEERING - TULANE UNIVERSITY NEW ORLEANS, LOUISIANA 70118 PHONE: (504) 865-5778 FAX: (504) 862-8941 e-mail: bakeer@mailhost.tcs.tulane.edu
DEVELOPED FOR:	LOUISIANA TRANSPORTATION RESEARCH CENTER (LTRC)
LTRC PROJECT:	97-4GT
STATE PROJECT:	736-99-0454
LTRC CONTACT:	MARK MORVANT PHONE: (225) 767-9124 FAX (224) 767-9108 e-mail: mmorvant@dotdmail.dotd.state.la.us

Select worksheet "Instructions" for instructions or proceed to "Input" to enter project data.

LOUISIANA DEPARTMENT OF TRANSPORTATION AND DEVELOPMENT
BATON ROUGE - LOUISIANA

PROGRAM: TU-DRAG
A COMPUTER PROGRAM FOR DETERMINING PILE LENGTHS CONSIDERING DOWNDRAG
FOR USE IN DESIGN OF PILE-SUPPORTED BRIDGE APPROACH SLABS

VERSION: 1.00 1999

BACKGROUND:
* The Program Includes Six Worksheets:
  - General
  - Instructions
  - Input
  - Pile
  - Chart_curve
  - Profile
* Worksheets are Locked Except for Input Cells.
* Primary Input Includes Pile Capacity and Soil Settlement at Various Depths.
* The Program is Independent of the Method Used to Calculate Pile Capacity or Soil Settlement.

Scroll Down for More Instructions. >>

INSTRUCTIONS:

Step 1 Select Worksheet "Input" and Enter the Following Values in the Gray Boxes Only!

Enter General Project Information:

ROW	COLUMN	
2	B	Date
2	E	User's Name
3	B	Project Title
4	C	Number of Soil Strata (n, Maximum 40)
5	C	Pile Type ("C" for Circular and "S" for Square Piles)
6	C	Pile Diameter or Width (inches) Use Average Diameter for Timber Piles
7	E	Ground Surface Elevation (feet, NGVD)

For Each Soil Stratum Enter:

ROW	COLUMN	
11 to n	A	Thickness of Soil Stratum (feet, n is No. of Strata in Cell 4-C)
11 to n	G	Cumulative Pile Friction Force (pounds)

Scroll Down for More Instructions. >>

Step 2 Select Worksheet "Pile" and Enter the Following Values in the Gray Boxes Only!

Enter

ROW	COLUMN	
6	C	Number of Transverse Pile Rows (m, Maximum 11 Rows)
7	C	Abutment Height above Ground Surface (feet)
7	H	Roadway Height above Ground Surface (feet)
		The Height Difference will be Considered in the Final Slab Profile

For Each Row of Piles Enter

ROW	COLUMN	
9	B to m	Embankment/Fill Height (feet)
10	B to m	Pile Length (feet)

Enter

ROW	COLUMN	
11 to n	A	Depth of Soil Stratum Change (feet)
11 to n	B to m	Cumulative Soil Strata Settlement at Each Pile Location (feet)

Scroll Down for More Instructions. >>

Step 3 Return to Worksheet "Input"

Click on the Button **Press to Calculate Pile Frictional Stiffness** on Row 1

Wait Until the Screen Stops Flashing ....

Scroll Down for More Instructions. >>

Step 4 The Program will Select Worksheet "Chart_curve"

The Program Will Display the Load/Settlement Curves for Various Pile Lengths

Wait Until the Screen Stops Flashing ....

If Desired, Click on File and Select *Print* to Print the Chart

Scroll Down for More Instructions. >>

Step 5 Select Worksheet "Profile"

ROW	COLUMN	
5	C	Enter Longitudinal Pile Spacing along Approach Slab Span (feet)
7	D	Enter Total Settlement of Roadway at End of Approach Slab (feet)

For Each Row of Transverse Piles Enter

ROW	COLUMN	
11 to 21	C	Single Pile Head Load at each Row (kips)

Click on the Button **Press to Plot Approach Slab Profile** on Row 3
Examine the Calculated Approach Slab Profile Versus the Ideal Profile
The Profile Accounts for the Height Difference between Abutment and Roadway
Change Pile Head Load to Adjust Settlement Profile

Step 6 Go to Step 2
Change Pile Length to Adjust Settlement Profile
Repeat the Above Steps Until an Acceptable Settlement Profile is Achieved
To Print a Worksheet, Select it then Click on File and Select *Print*

Louisiana Department of Transportation and Development
Pile Supported Bridge Approach Slab Design

PILE INPUT DATA

Date: 12/05/1999 User: RMB

Project: Site 5 (I-310 Luling Bridge Southbound South Approach)

No. of Soil Strata: 19 (n, Maximum 40 Strata to be Entered in Column A)

Pile Type: c (Enter "S" for Square, "C" for Circular)

Pile Diameter (inch): 12.0 (Enter Average Diameter for Timber Piles)

Natural Ground Elevation (ft): -1 NGVD (National Geodetic Vertical Datum)

Stratum Thickness ΔZ (ft)	Depth Z (ft)	Mobilized Friction (lb/ft)	Pile Skin Capacity (psf)	Elev. NGVD (ft)	Cumulative Pile Friction Force (lbs)
	0.00			-1.00	
3.00	3.00	2307.1	734.4	-4.00	6921.3
3.50	6.50	1708.2	543.8	-7.50	12900.2
3.00	9.50	1963.5	625.0	-10.50	18790.7
5.00	14.50	2118.2	674.3	-15.50	29381.8
7.50	22.00	1662.5	529.2	-23.00	41850.8
3.50	25.50	2373.1	755.4	-26.50	50156.6
2.50	28.00	2852.3	907.9	-29.00	57287.3
5.00	33.00	1154.5	367.5	-34.00	63060.0
10.00	43.00	934.6	297.5	-44.00	72406.2
10.00	53.00	997.5	317.5	-54.00	82380.8

Louisiana Department of Transportation and Development
Pile Supported Bridge Approach Slab Design

8.00	61.00	997.5	317.5	-62.00	**90360.4**
2.00	63.00	2025.5	644.7	-64.00	**94411.5**
6.50	69.50	2847.1	906.3	-70.50	**112917.4**
1.50	71.00	2847.1	906.3	-72.00	**117188.0**
7.00	78.00	2443.0	777.6	-79.00	**134289.2**
8.50	86.50	3361.5	1070.0	-87.50	**162862.0**
6.00	92.50	2448.2	779.3	-93.50	**177551.4**
0.50	93.00	7152.9	2276.9	-94.00	**181127.8**
5.00	98.00	2111.1	672.0	-99.00	**191683.2**

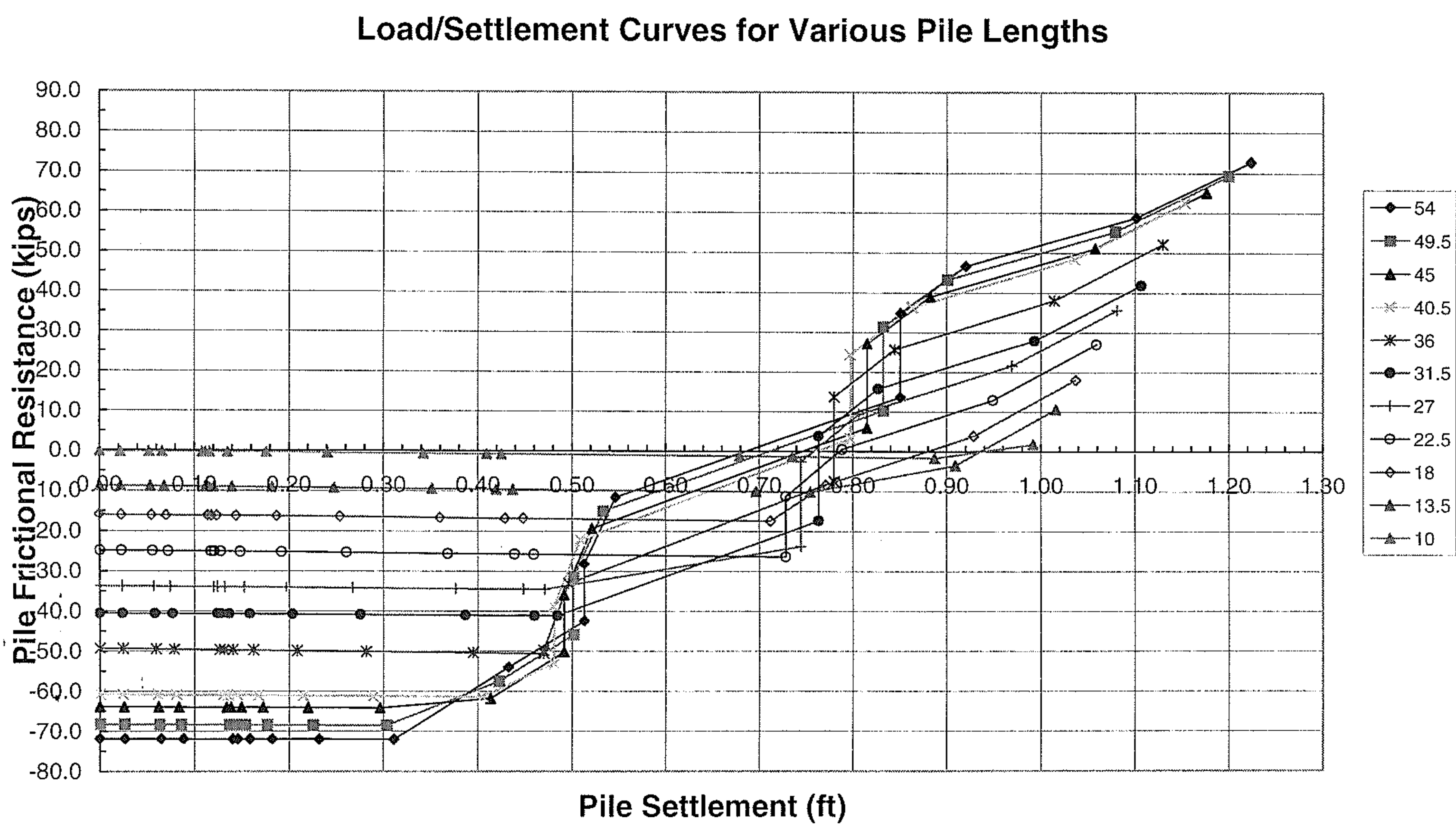
PILE LOCATION SOIL SETTLEMENT DISTRIBUTION

Project: Site 5 (I-310 Luling Bridge Southbound South Approach)

No. of Pile Rows: 11 (m, Maximum 11 Rows)

Abutment Height (ft): 12.00 **Roadway Height (ft):** 9.18 **Height Diff. (ft):** 2.82

	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9	Pile 10	Pile 11
Embankment/Fill (ft)	11.80	11.55	11.30	11.05	10.81	10.57	10.33	10.11	9.89	9.67	9.45
Pile Length (ft) / Depth (ft)	54.0	49.5	45.0	40.5	36.0	31.5	27.0	22.5	18.0	13.5	10.0
0.00	1.223	1.199	1.176	1.153	1.129	1.107	1.081	1.059	1.037	1.016	0.992
3.00	1.101	1.079	1.057	1.036	1.014	0.993	0.969	0.949	0.929	0.909	0.887
6.50	0.920	0.901	0.882	0.863	0.844	0.827	0.806	0.789	0.771	0.755	0.735
9.50	0.850	0.832	0.815	0.798	0.780	0.764	0.744	0.728	0.712	0.697	0.679
14.50	0.850	0.832	0.815	0.798	0.780	0.764	0.744	0.728	0.712	0.697	0.679
22.00	0.546	0.533	0.521	0.509	0.496	0.484	0.471	0.460	0.448	0.437	0.425
25.50	0.513	0.502	0.491	0.481	0.470	0.460	0.448	0.439	0.429	0.420	0.409
28.00	0.513	0.502	0.491	0.481	0.470	0.460	0.448	0.439	0.429	0.420	0.409
33.00	0.433	0.423	0.414	0.404	0.395	0.387	0.376	0.368	0.359	0.351	0.342
43.00	0.311	0.303	0.296	0.289	0.282	0.275	0.267	0.261	0.254	0.248	0.241
53.00	0.232	0.226	0.220	0.215	0.208	0.204	0.197	0.192	0.186	0.182	0.176
61.00	0.182	0.177	0.172	0.167	0.162	0.158	0.153	0.148	0.144	0.140	0.135
63.00	0.158	0.154	0.150	0.145	0.141	0.137	0.132	0.128	0.124	0.120	0.116
69.50	0.146	0.142	0.139	0.135	0.131	0.128	0.124	0.121	0.118	0.115	0.111
71.00	0.141	0.137	0.134	0.130	0.127	0.124	0.120	0.117	0.114	0.112	0.108
78.00	0.088	0.086	0.083	0.081	0.079	0.077	0.074	0.072	0.070	0.068	0.066
86.50	0.065	0.063	0.062	0.061	0.059	0.058	0.057	0.056	0.054	0.053	0.052
92.50	0.027	0.026	0.026	0.025	0.025	0.024	0.023	0.023	0.022	0.022	0.022
93.00	0.027	0.026	0.026	0.025	0.025	0.024	0.023	0.023	0.022	0.022	0.022
98.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000


Load/Settlement Curves for Various Pile Lengths
Pile Frictional Resistance (kips)
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
-40.0
-50.0
-60.0
-70.0
-80.0
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
Pile Settlement (ft)
54
49.5
45
40.5
36
31.5
27
22.5
18
13.5
10

Louisiana Department of Transportation and Development
Pile Supported Bridge Approach Slab Design

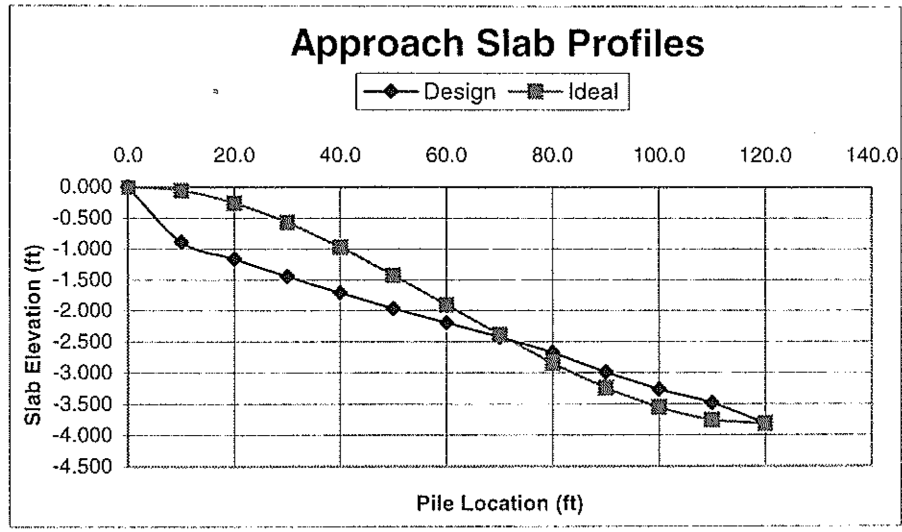
APPROACH SLAB SETTLEMENT PROFILE

Pile Rows: 11

Pile Spacing (ft): 10.00

Roadway Settlement (ft): 0.992

Element	Pile Location (ft)	Pile Head Load (kips)	Pile Head Settlement (ft)	Ideal Profile (ft)	Design Profile (ft)
Abutment	0.0	0	0.000	0.000	0.000
Pile 1	10.0	0	0.685	0.059	0.885
Pile 2	20.0	0	0.711	0.259	1.161
Pile 3	30.0	0	0.745	0.572	1.445
Pile 4	40.0	0	0.761	0.969	1.711
Pile 5	50.0	0	0.780	1.424	1.970
Pile 6	60.0	0	0.764	1.906	2.194
Pile 7	70.0	0	0.758	2.388	2.428
Pile 8	80.0	0	0.785	2.843	2.675
Pile 9	90.0	0	0.878	3.240	2.988
Pile 10	100.0	0	0.935	3.553	3.265
Pile 11	110.0	0	0.936	3.753	3.486
Roadway	120.0	0	0.992	3.812	3.812



APPENDIX D

EXAMPLES

EXAMPLE 1

PILE INPUT DATA

Date: 12/5/99 User: RMB

Project: Site 1 (I-310 Elevated Structure)

No. of Soil Strata: 17

Pile type: c (Enter "S" for Square, "C" for Circular)

Pile Diameter (inch): 12.0 (Enter Average Pile Diameter for Timber Piles)

Natural Ground Elevation (ft): -1 NGVD (National Geodetic Vertical Datum)

Stratum Thickness Δz (ft)	Depth (ft)	z	Mobilized Friction (lb/ft)	Pile skin Capacity (psf)	Elev. NGVD (ft)	Cumulative pile friction force (lbs)
	0.00				-1.00	
4.00	4.00		2336.6	743.8	-5.00	9346
5.00	9.00		795.2	253.1	-10.00	13322
5.00	14.00		795.2	253.1	-15.00	17298
5.00	19.00		2217.8	706.0	-20.00	28388
5.00	24.00		777.5	247.5	-25.00	32275
5.00	29.00		777.5	247.5	-30.00	36163
5.00	34.00		871.8	277.5	-35.00	40522
5.00	39.00		871.8	277.5	-40.00	44881
5.00	44.00		1233.1	392.5	-45.00	51046
5.00	49.00		1233.1	392.5	-50.00	57212
0.50	49.50		3579.4	1139.4	-50.50	59001
7.50	57.00		1445.1	460.0	-58.00	69840
2.00	59.00		863.9	275.0	-60.00	71568
3.00	62.00		2077.2	661.2	-63.00	77799
5.50	67.50		3550.0	1130.0	-68.50	97324
3.00	70.50		2434.7	775.0	-71.50	104629
3.00	73.50		1706.5	543.2	-74.50	109748

EXAMPLE 1

PILE LOCATION SOIL SETTLEMENT DISTRIBUTION

Project: Site 1 (I-310 Elevated Structure)

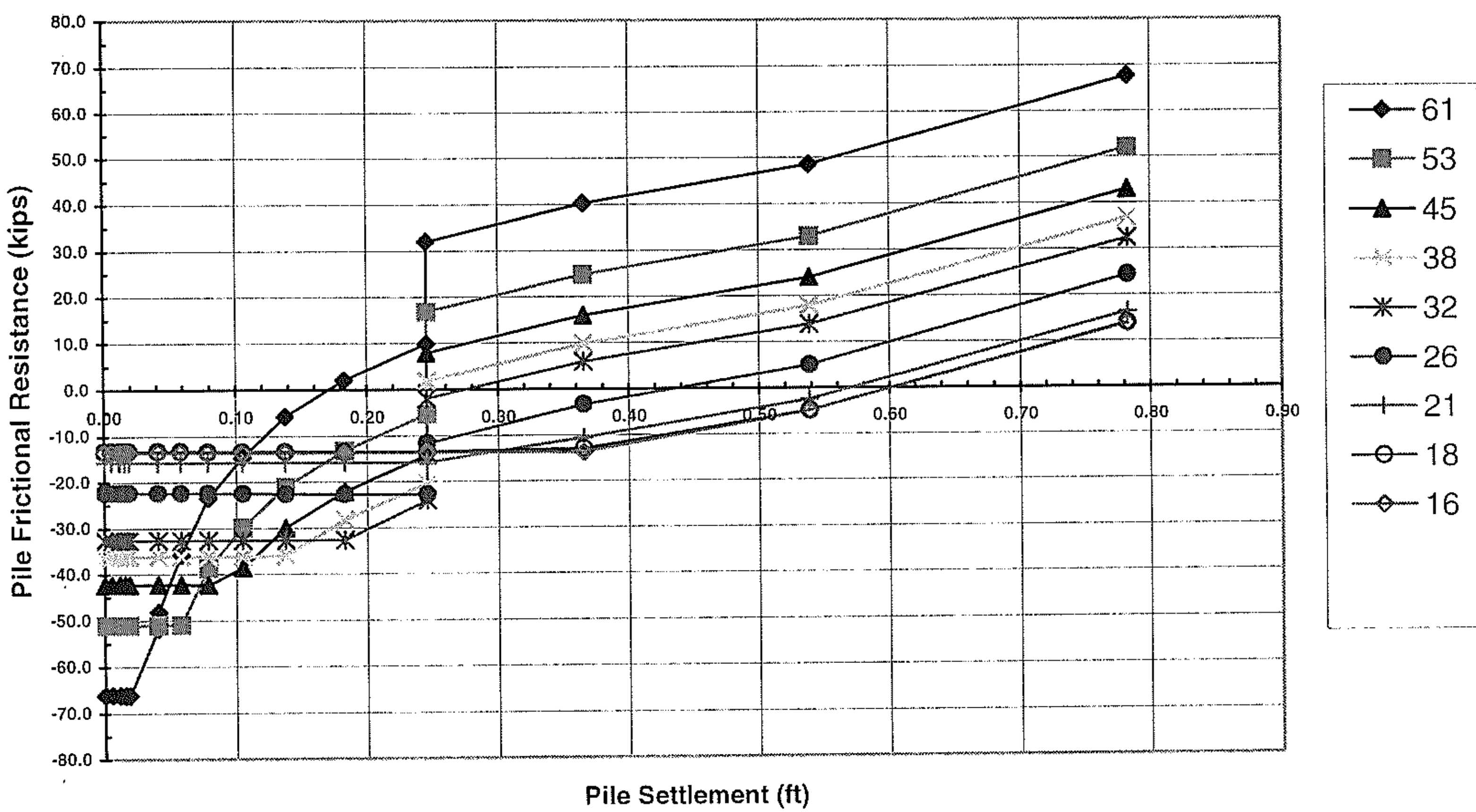
Pile Rows: 9

Abutment Elevation (ft): 9.00 Roadway Fill Height: (ft) 9

	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9
Embankment Fill (ft)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
Depth (ft) \ Pile Length (ft)	61	53	45	38	32	26	21	18	16
0.00	0.783	0.783	0.783	0.783	0.783	0.783	0.783	0.783	0.783
4.00	0.539	0.539	0.539	0.539	0.539	0.539	0.539	0.539	0.539
9.00	0.365	0.365	0.365	0.365	0.365	0.365	0.365	0.365	0.365
14.00	0.245	0.245	0.245	0.245	0.245	0.245	0.245	0.245	0.245
19.00	0.245	0.245	0.245	0.245	0.245	0.245	0.245	0.245	0.245
24.00	0.183	0.183	0.183	0.183	0.183	0.183	0.183	0.183	0.183
29.00	0.137	0.137	0.137	0.137	0.137	0.137	0.137	0.137	0.137
34.00	0.105	0.105	0.105	0.105	0.105	0.105	0.105	0.105	0.105
39.00	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079	0.079
44.00	0.058	0.058	0.058	0.058	0.058	0.058	0.058	0.058	0.058
49.00	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040
49.50	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040
57.00	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019	0.019
59.00	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015	0.015
62.00	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012	0.012
67.50	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011	0.011
70.50	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005
73.50	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

EXAMPLE 1

Pile stiffness curves



EXAMPLE 1

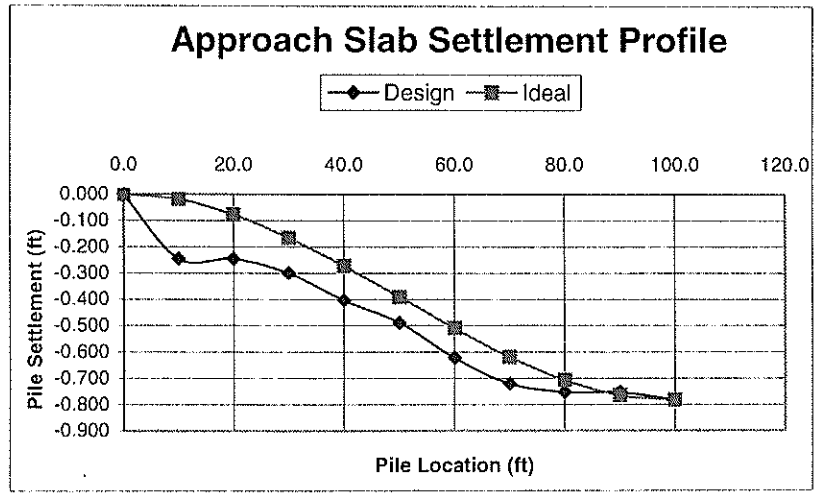
APPROACH SLAB SETTLEMENT PROFILE

Pile Rows: 9

Pile Spacing (ft): 10.00

Roadway Settlement (ft) 0.783

Element	Pile Location (ft)	Pile Head load (kips)	Pile Head Settlement (ft)	Ideal Settlement (ft)	Design Profile
Abutment	0.0	0	0.000	0.000	0.000
Pile 1	10.0	11.56	0.245	0.018	0.245
Pile 2	20.0	11.56	0.245	0.076	0.245
Pile 3	30.0	11.56	0.300	0.165	0.300
Pile 4	40.0	11.56	0.403	0.273	0.403
Pile 5	50.0	11.56	0.489	0.391	0.489
Pile 6	60.0	11.56	0.621	0.510	0.621
Pile 7	70.0	11.56	0.721	0.618	0.721
Pile 8	80.0	11.56	0.752	0.706	0.752
Pile 9	90.0	11.56	0.754	0.764	0.754
Roadway	100.0	0	0.783	0.783	0.783
	100.0				
	100.0				



EXAMPLE 2

PILE INPUT DATA

Date: 12/5/99 **User:** RMB

Project: Site 5 (I-310 Luling Bridge Southbound South Approach)

No. of Soil Strata: 19

Pile type: c (Enter "S" for Square, "C" for Circular)

Pile Diameter (inch): 12.0 (Enter Average Pile Diameter for Timber Piles)

Natural Ground Elevation (ft): -1 NGVD (National Geodetic Vertical Datum)

Stratum Thickness Δz (ft)	Depth (ft)	z	Mobilized Friction (lb/ft)	Pile skin Capacity (psf)	Elev. NGVD (ft)	Cumulative pile friction force (lbs)
	0.00				-1.00	
3.00	3.00		2307.1	734.4	-4.00	6921
3.50	6.50		1708.2	543.8	-7.50	12900
3.00	9.50		1963.5	625.0	-10.50	18791
5.00	14.50		2118.2	674.3	-15.50	29382
7.50	22.00		1662.5	529.2	-23.00	41851
3.50	25.50		2373.1	755.4	-26.50	50157
2.50	28.00		2852.3	907.9	-29.00	57287
5.00	33.00		1154.5	367.5	-34.00	63060
10.00	43.00		934.6	297.5	-44.00	72406
10.00	53.00		997.5	317.5	-54.00	82381
8.00	61.00		997.5	317.5	-62.00	90360
2.00	63.00		2025.5	644.7	-64.00	94411
6.50	69.50		2847.1	906.3	-70.50	112917
1.50	71.00		2847.1	906.3	-72.00	117188
7.00	78.00		2443.0	777.6	-79.00	134289
8.50	86.50		3361.5	1070.0	-87.50	162862
6.00	92.50		2448.2	779.3	-93.50	177551
0.50	93.00		7152.9	2276.9	-94.00	181128
5.00	98.00		2111.1	672.0	-99.00	191683

EXAMPLE 2

PILE LOCATION SOIL SETTLEMENT DISTRIBUTION

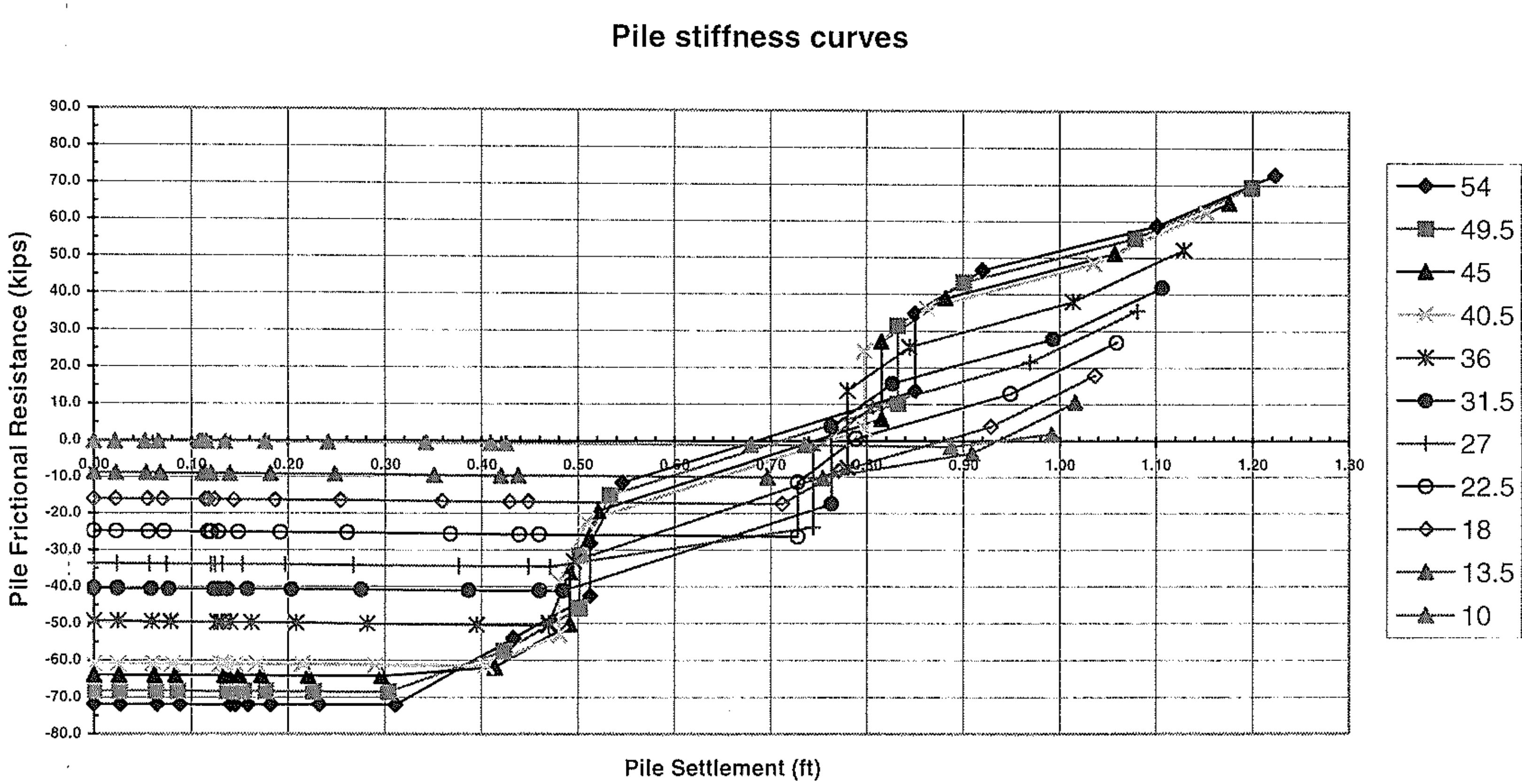
Project: Site 5 (I-310 Luling Bridge Southbound South Approach)

Pile Rows: 11

Abutment Elevation (ft): 12.00 **Roadway Fill Height: (ft)** 9.18

	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9	Pile 10	Pile 11
Embankment Fill (ft)	11.80	11.55	11.30	11.05	10.81	10.57	10.33	10.11	9.89	9.67	9.45
Pile Length (ft) / Depth (ft)	54.0	49.5	45.0	40.5	36.0	31.5	27.0	22.5	18.0	13.5	10.0
0.00	1.223	1.199	1.176	1.153	1.129	1.107	1.081	1.059	1.037	1.016	0.992
3.00	1.101	1.079	1.057	1.036	1.014	0.993	0.969	0.949	0.929	0.909	0.887
6.50	0.920	0.901	0.882	0.863	0.844	0.827	0.806	0.789	0.771	0.755	0.735
9.50	0.850	0.832	0.815	0.798	0.780	0.764	0.744	0.728	0.712	0.697	0.679
14.50	0.850	0.832	0.815	0.798	0.780	0.764	0.744	0.728	0.712	0.697	0.679
22.00	0.546	0.533	0.521	0.509	0.496	0.484	0.471	0.460	0.448	0.437	0.425
25.50	0.513	0.502	0.491	0.481	0.470	0.460	0.448	0.439	0.429	0.420	0.409
28.00	0.513	0.502	0.491	0.481	0.470	0.460	0.448	0.439	0.429	0.420	0.409
33.00	0.433	0.423	0.414	0.404	0.395	0.387	0.376	0.368	0.359	0.351	0.342
43.00	0.311	0.303	0.296	0.289	0.282	0.275	0.267	0.261	0.254	0.248	0.241
53.00	0.232	0.226	0.220	0.215	0.208	0.204	0.197	0.192	0.186	0.182	0.176
61.00	0.182	0.177	0.172	0.167	0.162	0.158	0.153	0.148	0.144	0.140	0.135
63.00	0.158	0.154	0.150	0.145	0.141	0.137	0.132	0.128	0.124	0.120	0.116
69.50	0.146	0.142	0.139	0.135	0.131	0.128	0.124	0.121	0.118	0.115	0.111
71.00	0.141	0.137	0.134	0.130	0.127	0.124	0.120	0.117	0.114	0.112	0.108
78.00	0.088	0.086	0.083	0.081	0.079	0.077	0.074	0.072	0.070	0.068	0.066
86.50	0.065	0.063	0.062	0.061	0.059	0.058	0.057	0.056	0.054	0.053	0.052
92.50	0.027	0.026	0.026	0.025	0.025	0.024	0.023	0.023	0.022	0.022	0.022
93.00	0.027	0.026	0.026	0.025	0.025	0.024	0.023	0.023	0.022	0.022	0.022
98.00	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

EXAMPLE 2



Pile stiffness curves
Pile Frictional Resistance (kips)
Pile Settlement (ft)
90.0
80.0
70.0
60.0
50.0
40.0
30.0
20.0
10.0
0.0
-10.0
-20.0
-30.0
-40.0
-50.0
-60.0
-70.0
-80.0
0.00
0.10
0.20
0.30
0.40
0.50
0.60
0.70
0.80
0.90
1.00
1.10
1.20
1.30
54
49.5
45
40.5
36
31.5
27
22.5
18
13.5
10

EXAMPLE 2

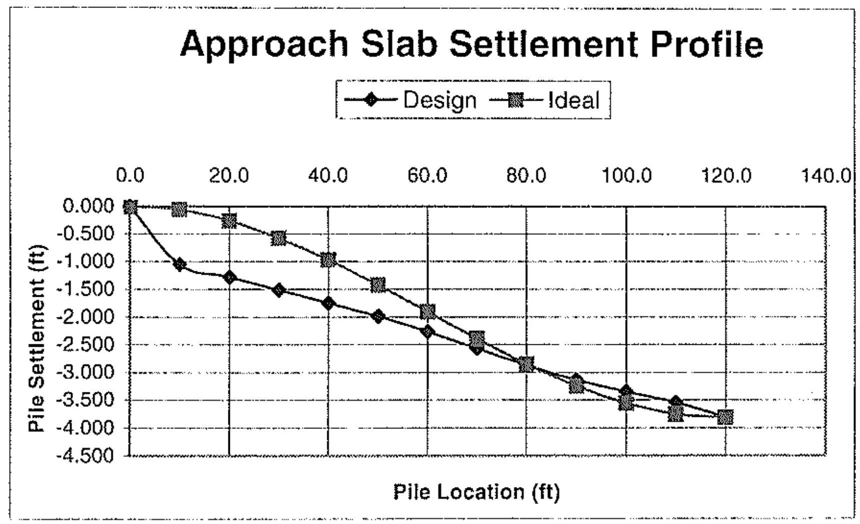
APPROACH SLAB SETTLEMENT PROFILE

Pile Rows: 11

Pile Spacing (ft): 10.00

Roadway Settlement (ft) 0.992

Element	Pile Location (ft)	Pile Head load (kips)	Pile Head Settlement (ft)	Ideal Settlement (ft)	Design Profile
Abutment	0.0	0	0.000	0.000	0.000
Pile 1	10.0	16.19	0.850	0.059	1.050
Pile 2	20.0	16.19	0.832	0.259	1.282
Pile 3	30.0	16.19	0.815	0.572	1.515
Pile 4	40.0	16.19	0.798	0.969	1.748
Pile 5	50.0	16.19	0.793	1.424	1.983
Pile 6	60.0	16.19	0.833	1.906	2.263
Pile 7	70.0	16.19	0.897	2.388	2.567
Pile 8	80.0	16.19	0.975	2.843	2.865
Pile 9	90.0	16.19	1.023	3.240	3.133
Pile 10	100.0	16.19	1.016	3.553	3.346
Pile 11	110.0	16.19	0.992	3.753	3.542
Roadway	120.0	16.19	0.992	3.812	3.812



EXAMPLE 3

PILE INPUT DATA

Date: 12/5/99 **User:** RMB

Project: Site 7 (LA 3139 Parish Line West Approach West Bound)

No. of Soil Strata: 15

Pile type: c (Enter "S" for Square, "C" for Circular)

Pile Diameter (inch): 12.0 (Enter Average Pile Diameter for Timber Piles)

Natural Ground Elevation (ft): -1 NGVD (National Geodetic Vertical Datum)

Stratum Thickness Δz (ft)	Depth z (ft)	Mobilized Friction (lb/ft)	Pile skin Capacity (psf)	Elev. NGVD (ft)	Cumulative pile friction force (lbs)
	0.00			-1.00	
3.00	3.00	4565.1	1453.1	-4.00	13695
4.00	7.00	1007.4	320.7	-8.00	17725
2.00	9.00	873.8	278.1	-10.00	19473
3.50	12.50	343.6	109.4	-13.50	20675
3.50	16.00	598.9	190.6	-17.00	22771
9.00	25.00	479.1	152.5	-26.00	27083
8.00	33.00	1028.9	327.5	-34.00	35314
6.50	39.50	1390.2	442.5	-40.50	44350
8.50	48.00	3847.8	1224.8	-49.00	77057
5.50	53.50	557.6	177.5	-54.50	80124
7.00	60.50	1869.9	595.2	-61.50	93213
6.00	66.50	1869.9	595.2	-67.50	104432
7.00	73.50	5749.1	1830.0	-74.50	144676
4.50	78.00	5129.9	1632.9	-79.00	167760
2.5	80.50	1052.4	335.0	-81.50	170392

EXAMPLE 3

PILE LOCATION SOIL SETTLEMENT DISTRIBUTION

Project: Site 7 (LA 3139 Parish Line West Approach West Bound)

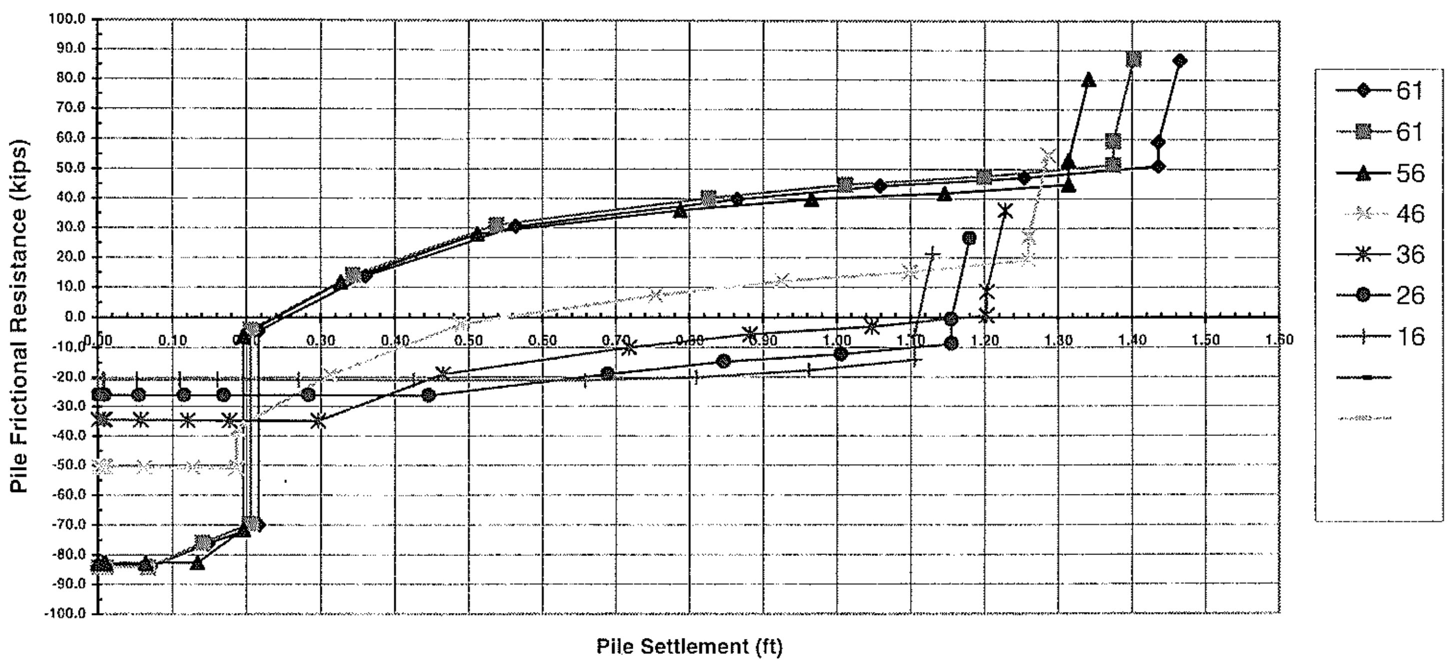
Pile Rows: 7

Abutment Elevation (ft): 4.59   Roadway Fill Height: (ft) 3.00

	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7
Embankment Fill (ft)	4.37	4.14	3.92	3.72	3.52	3.35	3.18
Pile Length (ft) / Soil Depth (ft)	61	61	56	46	36	26	16
0.0	1.465	1.403	1.342	1.287	1.228	1.180	1.130
3.0	1.436	1.375	1.314	1.260	1.203	1.155	1.106
7.0	1.436	1.375	1.314	1.260	1.203	1.155	1.106
9.0	1.254	1.199	1.146	1.098	1.047	1.006	0.962
12.5	1.059	1.012	0.966	0.925	0.882	0.846	0.809
16.0	0.865	0.826	0.788	0.754	0.718	0.689	0.658
25.0	0.564	0.538	0.512	0.489	0.465	0.446	0.425
33.0	0.360	0.343	0.327	0.312	0.296	0.283	0.270
39.5	0.216	0.205	0.196	0.187	0.177	0.169	0.161
48.0	0.216	0.205	0.196	0.187	0.177	0.169	0.161
53.5	0.148	0.141	0.134	0.127	0.121	0.116	0.110
60.5	0.071	0.067	0.064	0.061	0.057	0.055	0.052
66.5	0.011	0.011	0.010	0.010	0.009	0.009	0.009
73.5	0.009	0.008	0.007	0.007	0.007	0.007	0.006
78.0	0.009	0.008	0.007	0.007	0.007	0.007	0.006
80.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000

EXAMPLE 3

Pile stiffness curves



EXAMPLE 3

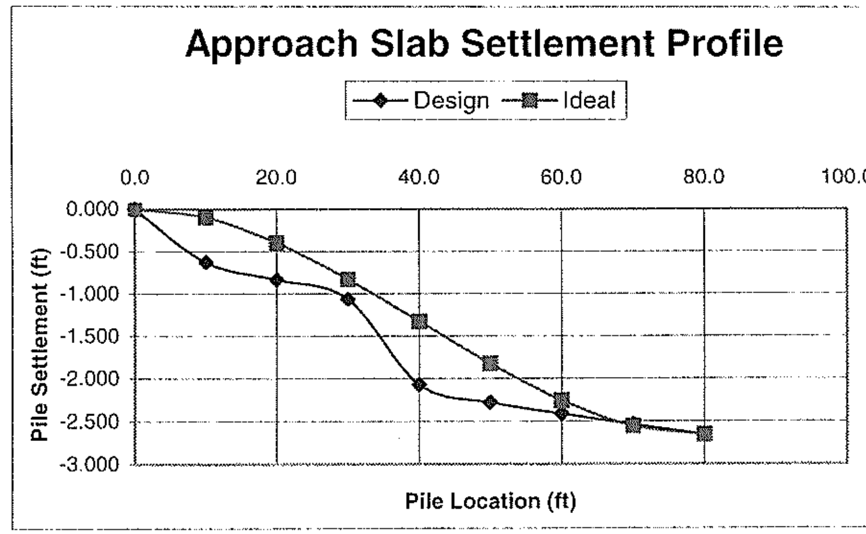
APPROACH SLAB SETTLEMENT PROFILE

Pile Rows: 7

Pile Spacing (ft): 10.00

Roadway Settlement (ft) 1.066

Element	Pile Location (ft)	Pile Head load (kips)	Pile Head Settlement (ft)	Ideal Settlement (ft)	Design Profile
Abutment	0.0	0	0.000	0.000	0.000
Pile 1	10.0	17.73	0.409	0.100	0.629
Pile 2	20.0	17.73	0.385	0.398	0.835
Pile 3	30.0	17.73	0.395	0.830	1.065
Pile 4	40.0	17.73	1.200	1.328	2.070
Pile 5	50.0	17.73	1.211	1.826	2.281
Pile 6	60.0	17.73	1.172	2.258	2.412
Pile 7	70.0	17.73	1.127	2.556	2.537
Roadway	80.0	17.73	1.066	2.656	2.656
	80.0				
	80.0				
	80.0				
	80.0				



EXAMPLE 4

PILE INPUT DATA

Date: 12/5/99 **User:** RMB

Project: Site F (I-310 Elevated Structure)

No. of Soil Strata: 21

Pile type: c (Enter "S" for Square, "C" for Circular)

Pile Diameter (inch): 12.0 (Enter Average Pile Diameter for Timber Piles)

Natural Ground Elevation (ft): -1 NGVD (National Geodetic Vertical Datum)

Stratum Thickness Δz (ft)	Depth (ft)	z	Mobilized Friction (lb/ft)	Pile skin Capacity (psf)	Elev. NGVD (ft)	Cumulative pile friction force (lbs)
	0.00				-1.00	
4.00	4.00		2336.6	743.8	-5.00	9346
5.00	9.00		795.2	253.1	-10.00	13322
5.00	14.00		795.2	253.1	-15.00	17298
5.00	19.00		2217.8	706.0	-20.00	28388
5.00	24.00		777.5	247.5	-25.00	32275
5.00	29.00		777.5	247.5	-30.00	36163
5.00	34.00		871.8	277.5	-35.00	40522
5.00	39.00		871.8	277.5	-40.00	44881
5.00	44.00		1233.1	392.5	-45.00	51046
5.00	49.00		1233.1	392.5	-50.00	57212
0.50	49.50		3579.4	1139.4	-50.50	59001
7.50	57.00		1445.1	460.0	-58.00	69840
2.00	59.00		863.9	275.0	-60.00	71568
3.00	62.00		2077.2	661.2	-63.00	77799
5.50	67.50		3550.0	1130.0	-68.50	97324
3.00	70.50		2434.7	775.0	-71.50	104629
3.00	73.50		1706.5	543.2	-74.50	109748
5.0	78.50		1706.4	543.2	-79.50	118280
5.0	83.50		1706.4	543.2	-84.50	126812
5.0	88.50		1706.4	543.2	-89.50	135344
5.0	93.50		1706.4	543.2	-94.50	143876

EXAMPLE 4

PILE LOCATION SOIL SETTLEMENT DISTRIBUTION

Project: Site F (I-310 Elevated Structure)

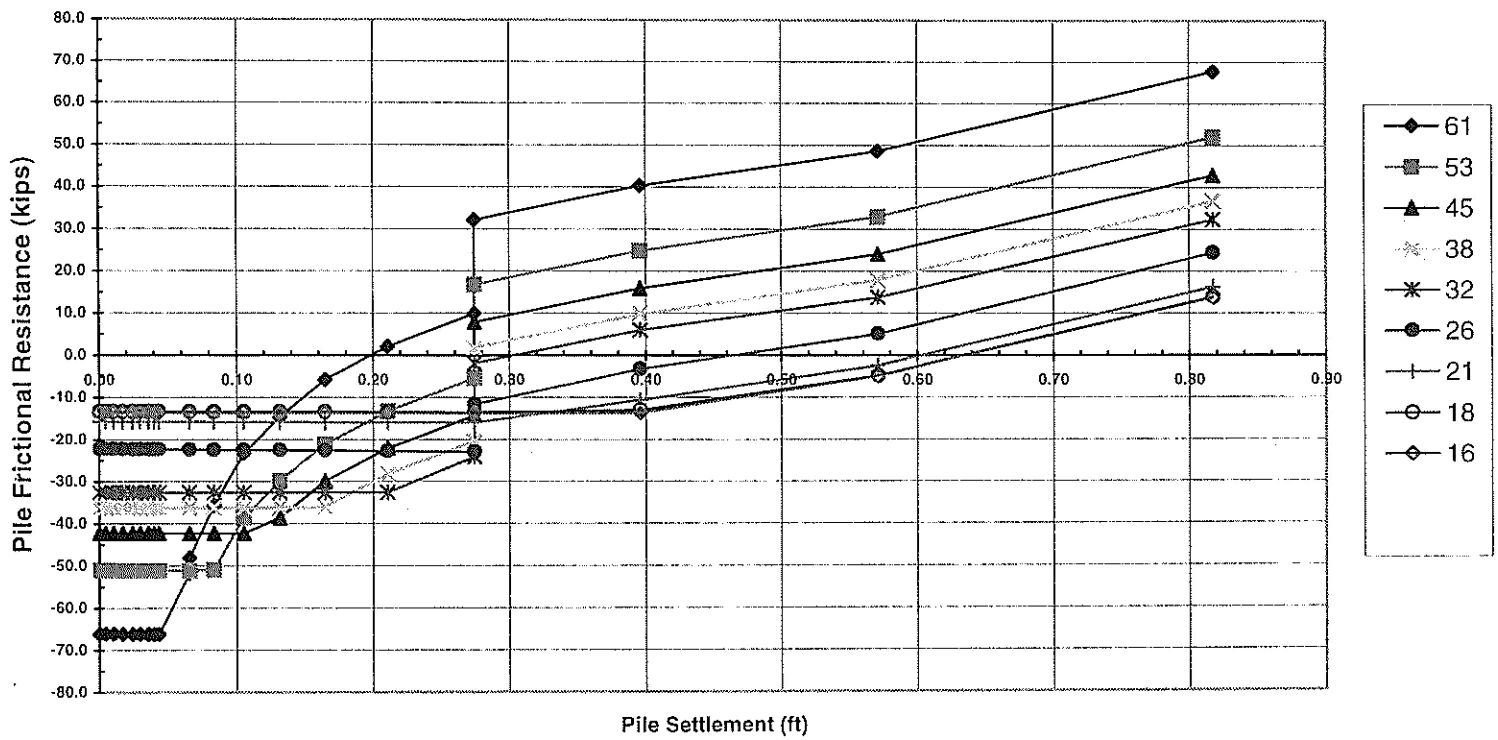
Pile Rows: 9

Abutment Elevation (ft): 9.00 **Roadway Fill Height: (ft)** 9

	Pile 1	Pile 2	Pile 3	Pile 4	Pile 5	Pile 6	Pile 7	Pile 8	Pile 9
Embankment Fill (ft)	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00	9.00
Pile Length (ft) / Soil Depth (ft)	61	53	45	38	32	26	21	18	16
0.0	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82	0.82
4.0	0.571	0.571	0.571	0.571	0.571	0.571	0.571	0.571	0.571
9.0	0.396	0.396	0.396	0.396	0.396	0.396	0.396	0.396	0.396
14.0	0.274	0.274	0.274	0.274	0.274	0.274	0.274	0.274	0.274
19.0	0.274	0.274	0.274	0.274	0.274	0.274	0.274	0.274	0.274
24.0	0.211	0.211	0.211	0.211	0.211	0.211	0.211	0.211	0.211
29.0	0.165	0.165	0.165	0.165	0.165	0.165	0.165	0.165	0.165
34.0	0.131	0.131	0.131	0.131	0.131	0.131	0.131	0.131	0.131
39.0	0.105	0.105	0.105	0.105	0.105	0.105	0.105	0.105	0.105
44.0	0.084	0.084	0.084	0.084	0.084	0.084	0.084	0.084	0.084
49.0	0.066	0.066	0.066	0.066	0.066	0.066	0.066	0.066	0.066
49.5	0.066	0.066	0.066	0.066	0.066	0.066	0.066	0.066	0.066
57.0	0.043	0.043	0.043	0.043	0.043	0.043	0.043	0.043	0.043
59.0	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040	0.040
62.0	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036
67.5	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036	0.036
70.5	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.030	0.030
73.5	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024	0.024
78.5	0.017	0.017	0.017	0.017	0.017	0.017	0.017	0.017	0.017
83.5	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010	0.010
88.5	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005	0.005
93.5	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000	0.000

EXAMPLE 4

Pile stiffness curves



EXAMPLE 4

APPROACH SLAB SETTLEMENT PROFILE

Pile Rows: 9

Pile Spacing (ft): 10.00

Roadway Settlement (ft) 0.818

Element	Pile Location (ft)	Pile Head load (kips)	Pile Head Settlement (ft)	Ideal Settlement (ft)	Design Profile
Abutment	0.0	0	0.000	0.000	0.000
Pile 1	10.0	11.56	0.274	0.019	0.274
Pile 2	20.0	11.56	0.274	0.080	0.274
Pile 3	30.0	11.56	0.329	0.172	0.329
Pile 4	40.0	11.56	0.433	0.285	0.433
Pile 5	50.0	11.56	0.521	0.409	0.521
Pile 6	60.0	11.56	0.653	0.532	0.653
Pile 7	70.0	11.56	0.755	0.645	0.755
Pile 8	80.0	11.56	0.786	0.738	0.786
Pile 9	90.0	11.56	0.788	0.799	0.788
Roadway	100.0	11.56	0.818	0.818	0.818
	100.0				
	100.0