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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 southeastern 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 could be mitigated by selecting the length of piles along the 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).

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ASSESSMENT

OF

MITIGATING EMBANKMENT SETTLEMENT

WITH

PILE-SUPPORTED APPROACH SLABS

Summary Report

by

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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

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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.

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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.

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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 rideability 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]*.

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). 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. 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.

Literature Review

In 1957, a study was made to evaluate the settlement of the friction pile supported abutment 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.

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 investigation 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). Out of the 90 pile-supported approach slabs, 63 sites were identified in Orleans, Jefferson and St. Charles Parishes and were targeted for thorough review and evaluation. Only few non-pile supported approach slabs were selected for comparative purposes.

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.

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.

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 1. 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 *[15]*. 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.

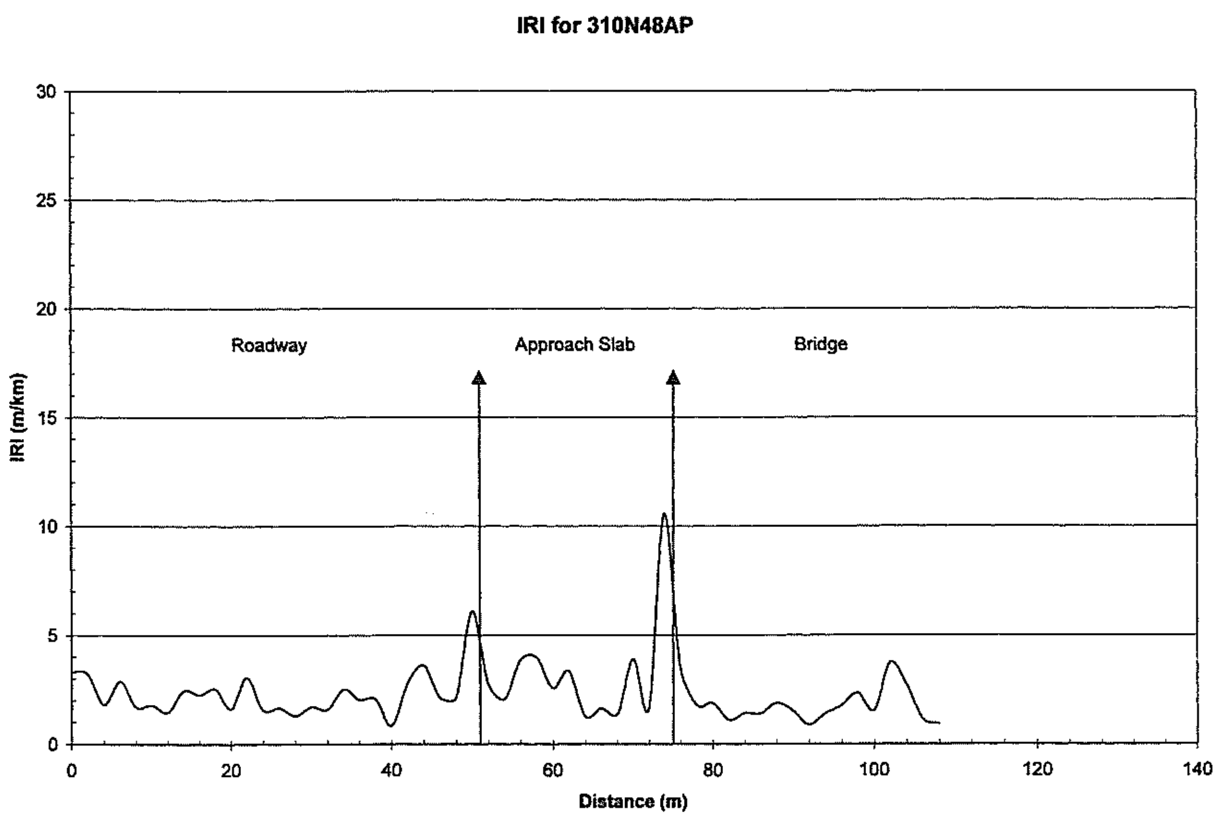


Figure 1
IRI Graph

The recognized standard rating for pavement using the IRI is shown in table 1. 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 2.

Table 1
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 2
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

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 2 and 3. The entire set of graphs and charts is available in the Tulane University Civil and Environmental Engineering Department.

Figure 2 shows a bar graph comparison of current condition ratings versus length of the 63 pile supported approach slabs in Orleans, Jefferson, and St. Charles Parishes. 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.

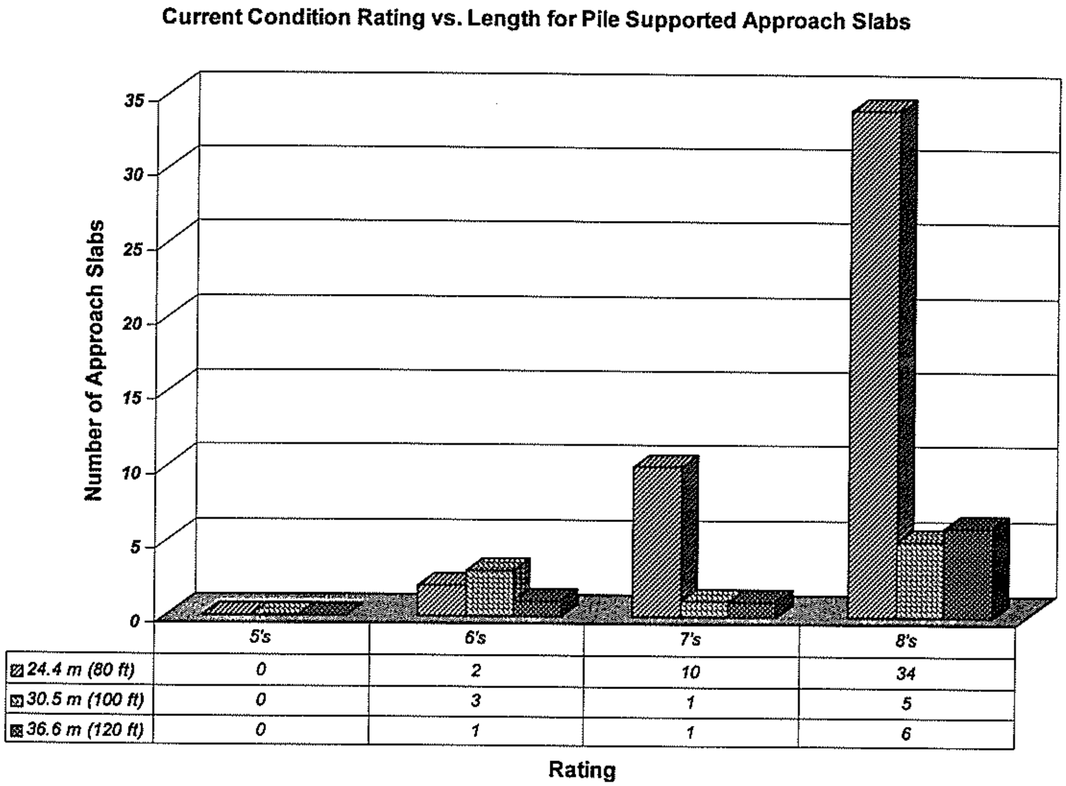


Figure 2
Effect of approach slab length

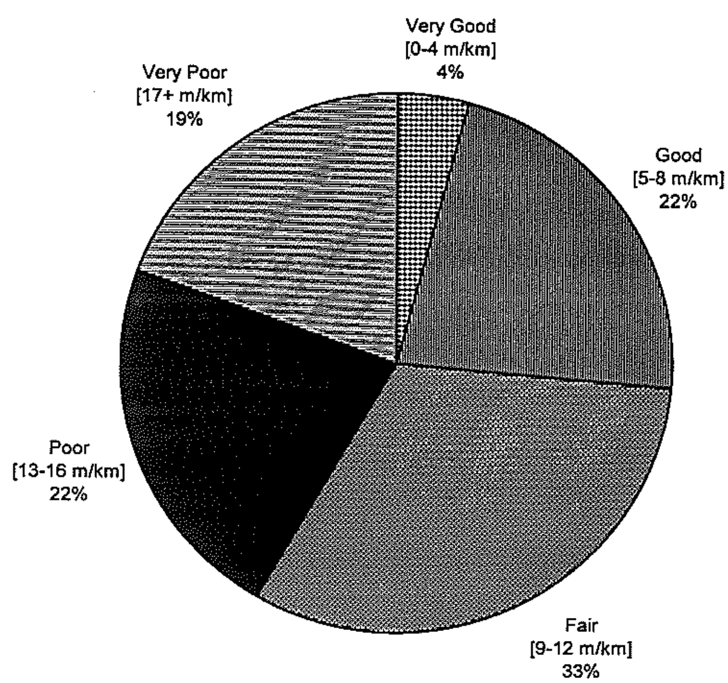


Figure 3
IRI ratings of pile approach slabs

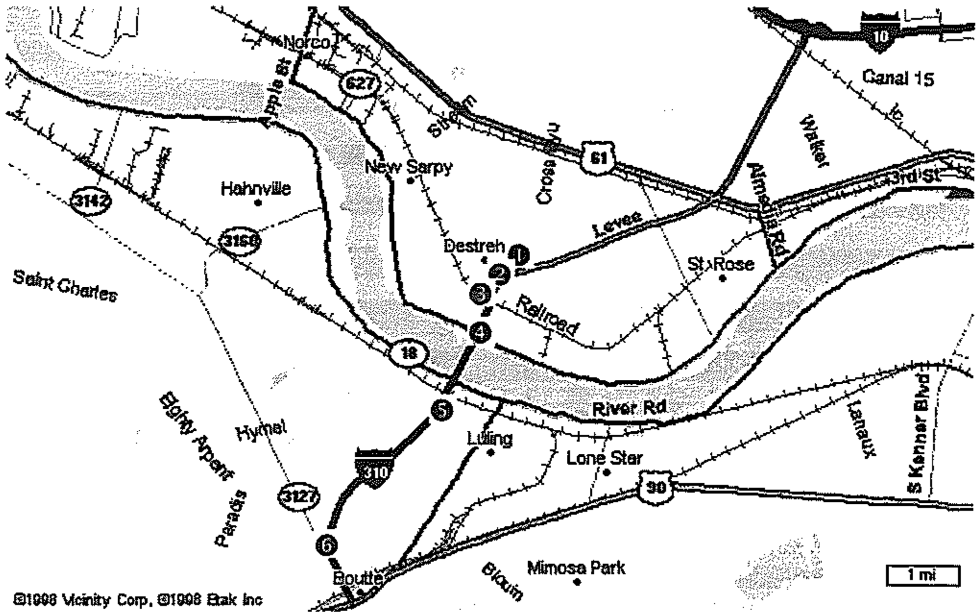
Selection of Representative Testing Sites

Seven representative sites were selected for thorough in-situ investigations. Figures 4 a and 4 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 3. 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
- Difference in performance of the various slabs along I-310.
-

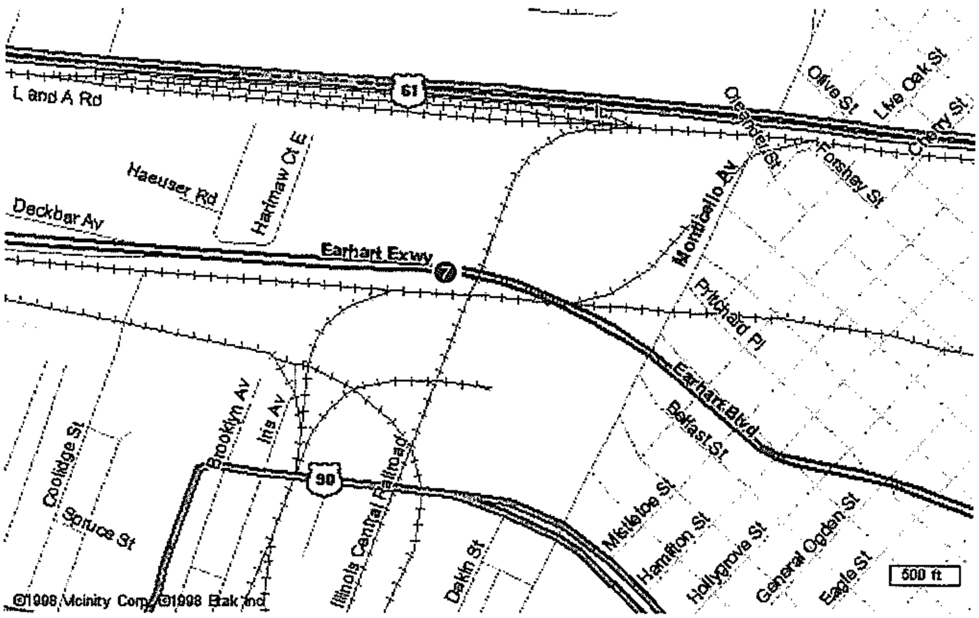
Field Testing of Representative Test Sites

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 4 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.



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 4a
Location map of representative test sites selected along I-310



Legend

❼ Parish line bridge West Approach

Figure 4b
Location map of the representative test site selected on LA 3139

Table 3

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 4
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

Soil-Structure Interaction 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 5 and table 3 summarize details of the approach slabs studied in more depth (sites 1-7). Table 5 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 *[15]*.

Table 5

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 the Hale Boggs (Luling) bridge. It is the south approach of the elevated bridge structure (see figure 4a for map location) and includes both the southbound and northbound approaches. Figure 5 shows a view of the elevated bridge structures and the embankment median of site 1 looking in the north direction. As indicated in table 3, 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.

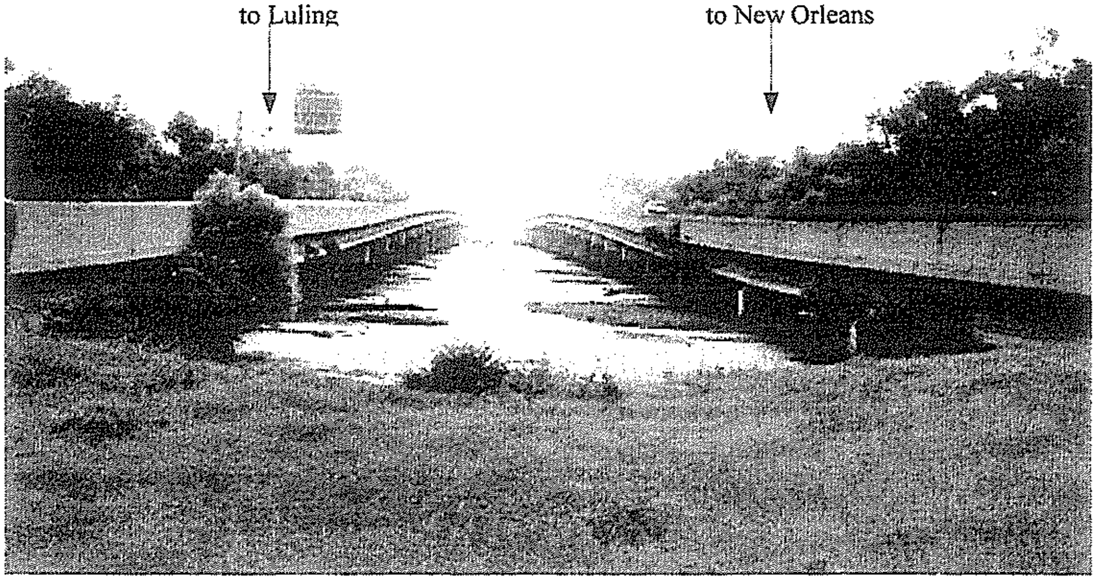


Figure 5
Site 1: I-310 Elevated Structure Approaches

Approach Slab Information

Figure 6 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 6. Figure 7 shows a plan view of the northbound approach slab. The design elevations at specific points along the slab surface are listed in table 7.

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.

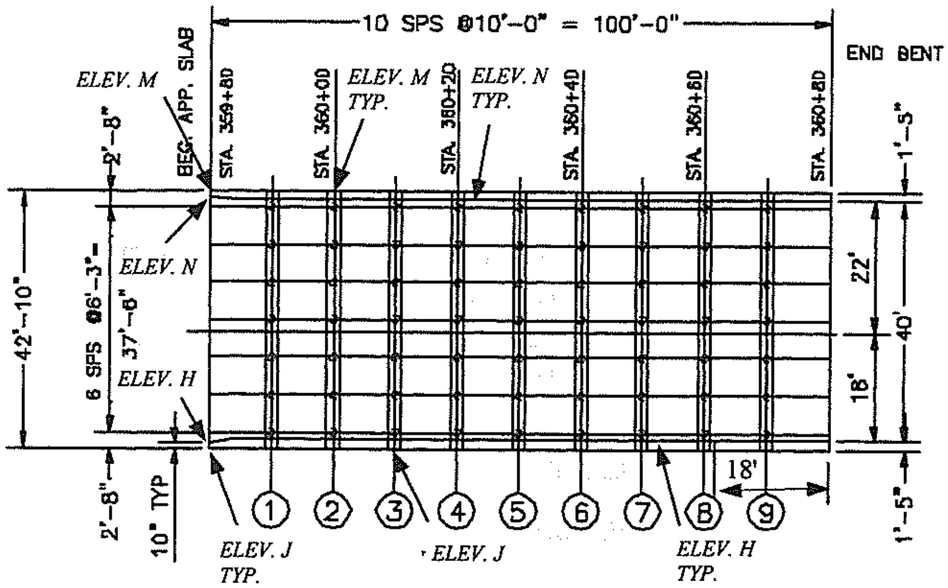


Figure 6
Plan of southbound approach at site 1

Table 6

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)

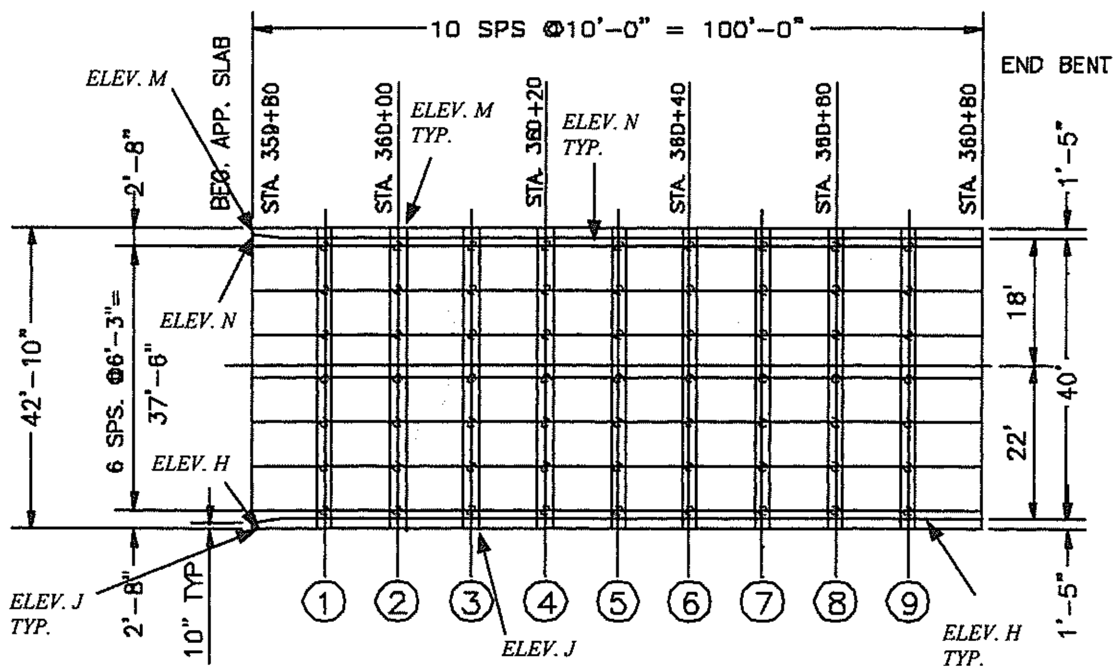


Figure 7
Plan of northbound approach at site 1

Table 7

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 8 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 9. Figure 9 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.

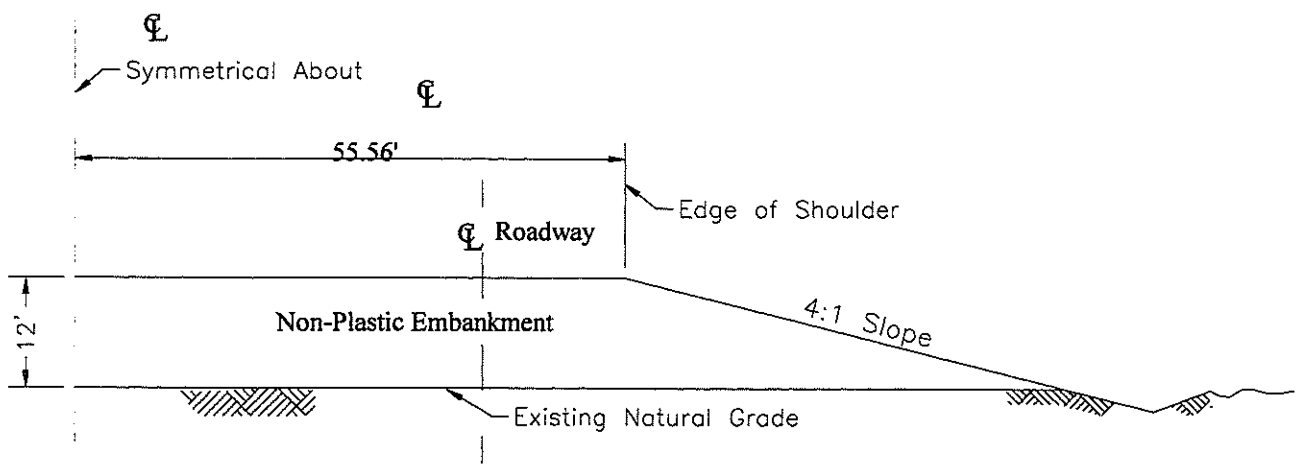


Figure 8
Surcharge profile of grade at station 360+80

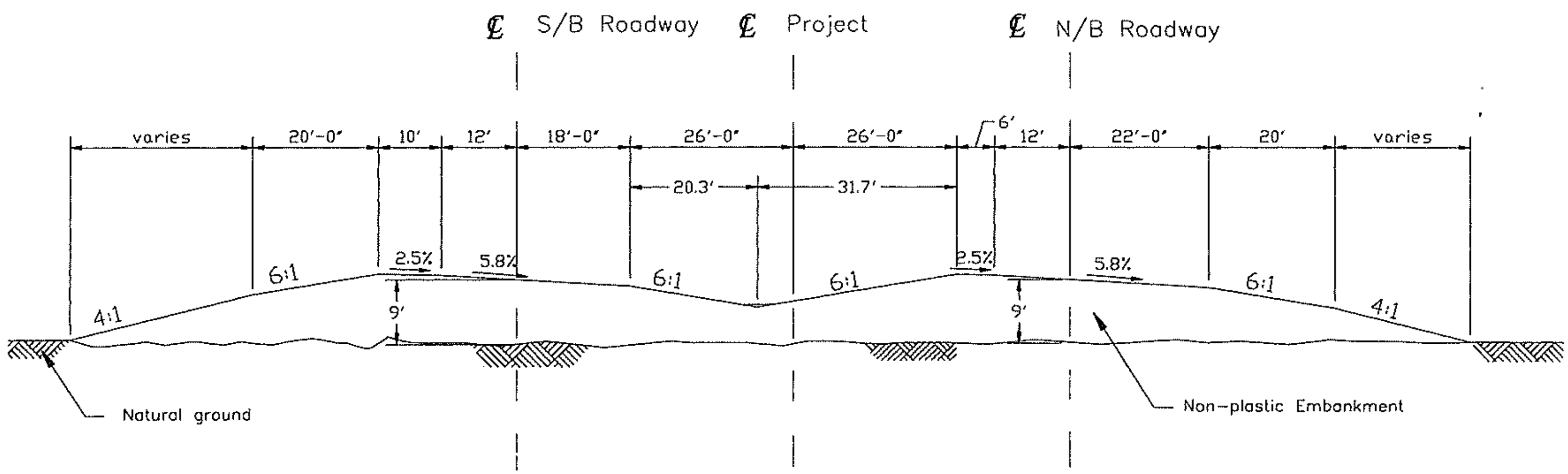


Figure 9
Typical embankment section

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 9. Table 9 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 8 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 8

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 9

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	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	WET	ATTERBERG LIMITS L.L.	P.L.	P.I.
							0							
1	.0	.5	.0 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 W/ MUCH SHELL (PETROLEUM ODOR)					8.3					
4	2.0	3.0	3.5		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 W/ TRACE ORGANIC (4" TAR IN SAMPLE)	3									
7	9.5	10.0	10.0				10	1190	52.3	65.5	99.7	102	32	70
8	11.5	12.0		VERY SOFT TO SOFT GRAY CLAY W/ WOOD										
9	14.5	15.0					15	405	71.9	52.2	89.7			
10	19.5	20.0	20.0				20							
			25.0	WOOD W/ SOME CLAY			25							
11	26.0	26.5		VERY SOFT TO SOFT GRAY CLAY W/ SILT LENSES (W/ SHELL FRAGMENTS @ 39.5'-40.0')										
12	29.5	30.0					30	495	50.1	67.8	101.8	58	48	10
13	34.5	35.0					35							
14	39.5	40.0					40	555	51.3	68.3	103.3			
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 W/ SHELL FRAGMENTS				920	48.4	70.3	104.3	63	23	40
20	59.5	60.0	62.5				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 11 for the southbound approach slab and in figure 12 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 10.

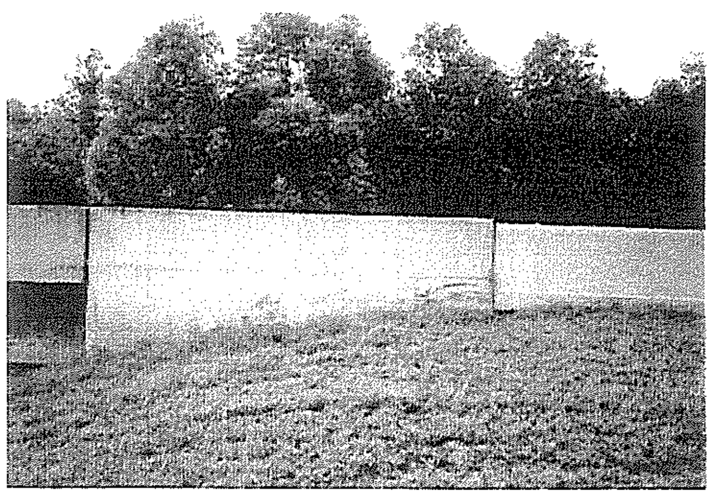


Figure 10
View of northbound bridge of site 1

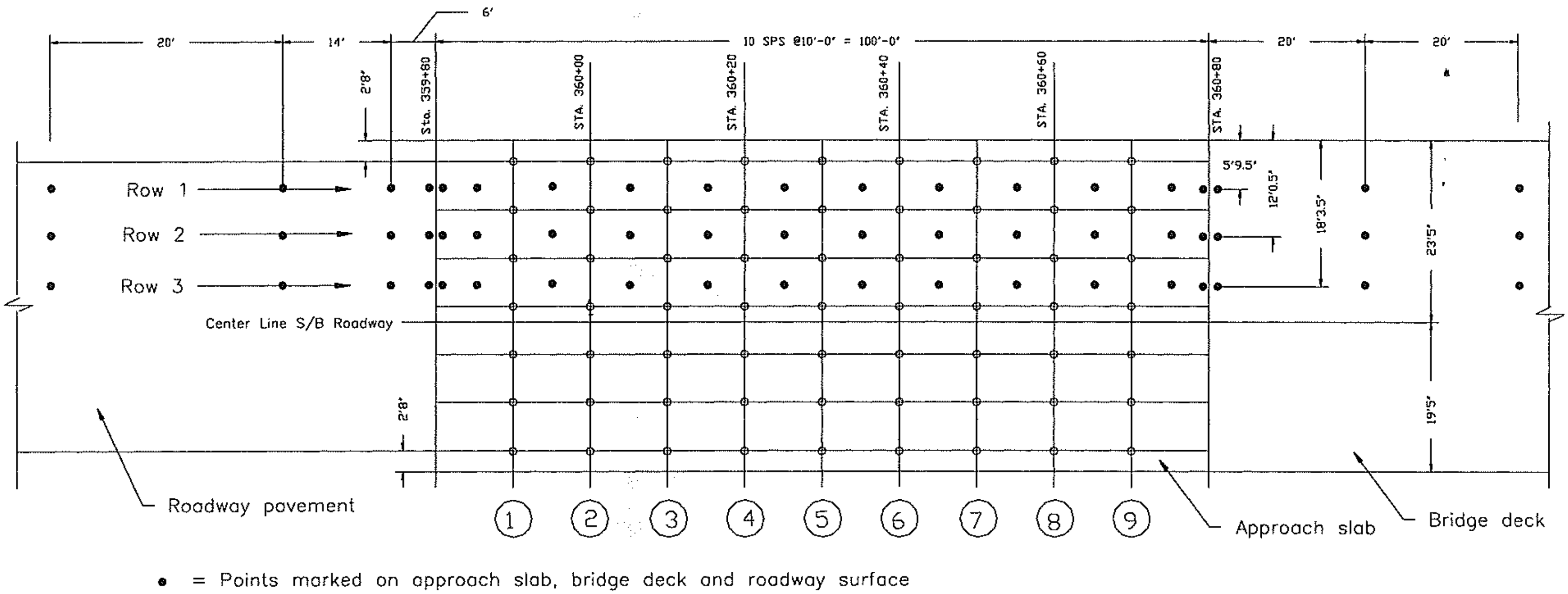


Figure 11
Data points marked to perform survey, profiler and Dynatest at southbound approach slab of site 1

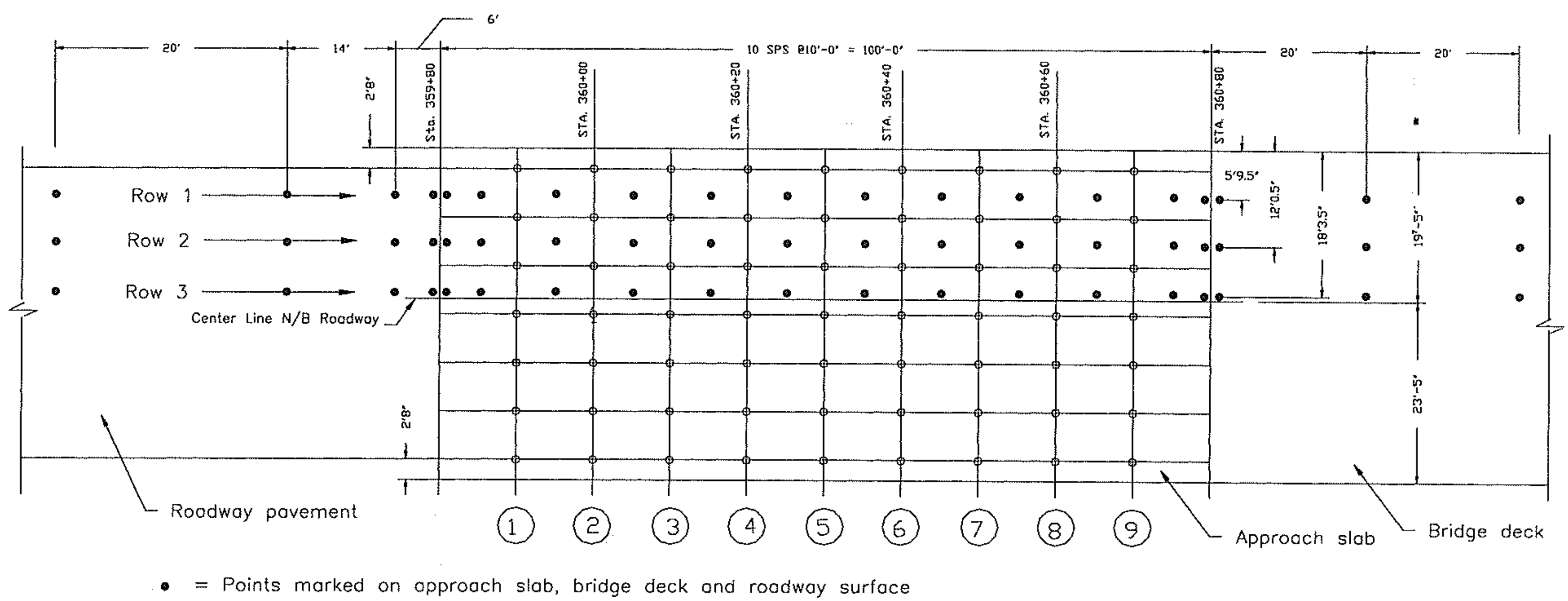


Figure 12
Data points chosen to perform survey, profiler and Dynatest at northbound approach slab of site 1

Survey Results

The relative elevations of all of the points shown in figures 11 and 12 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 6 and 7, the settlement along each longitudinal row of the three rows could be determined by interpolation. Figures 13 and 14 show the approach slab settlement along each longitudinal row along the northbound and southbound roadways, respectively.

Figure 13 shows that the maximum settlement of the northbound approach slab at site 1 is around 0.7 ft (0.21 m), and figure 14 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 (Elve. 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 6), the elevation (Elve. 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.

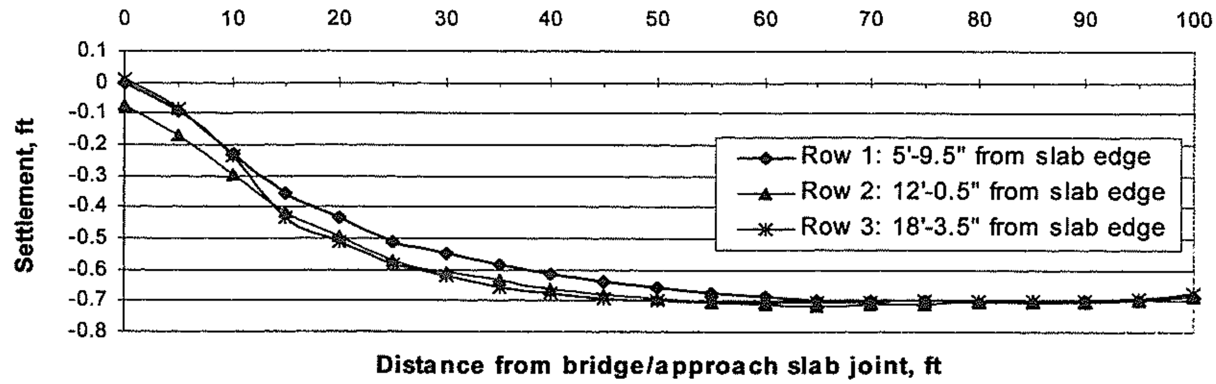


Figure 13
Northbound approach slab settlement for site 1

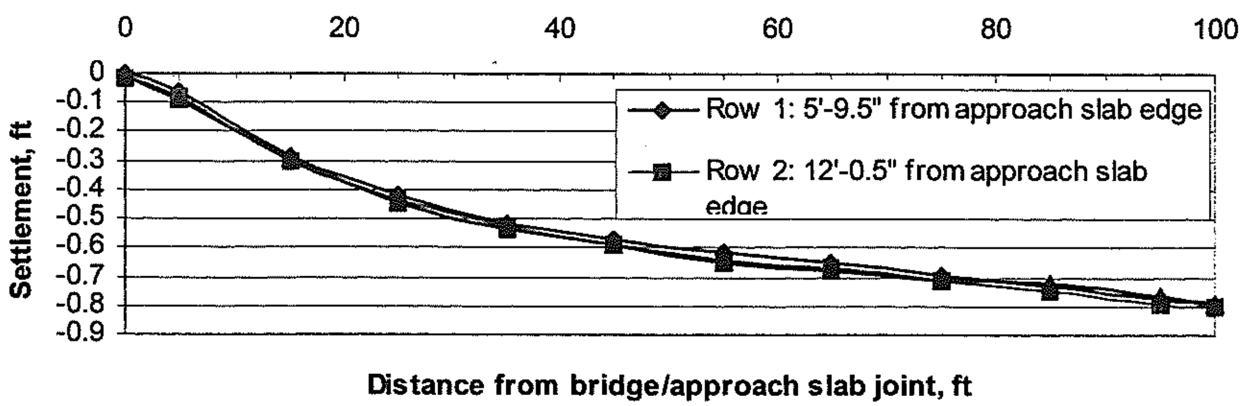


Figure 14
Southbound approach slab settlement for site 1

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 15 and 16, 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.

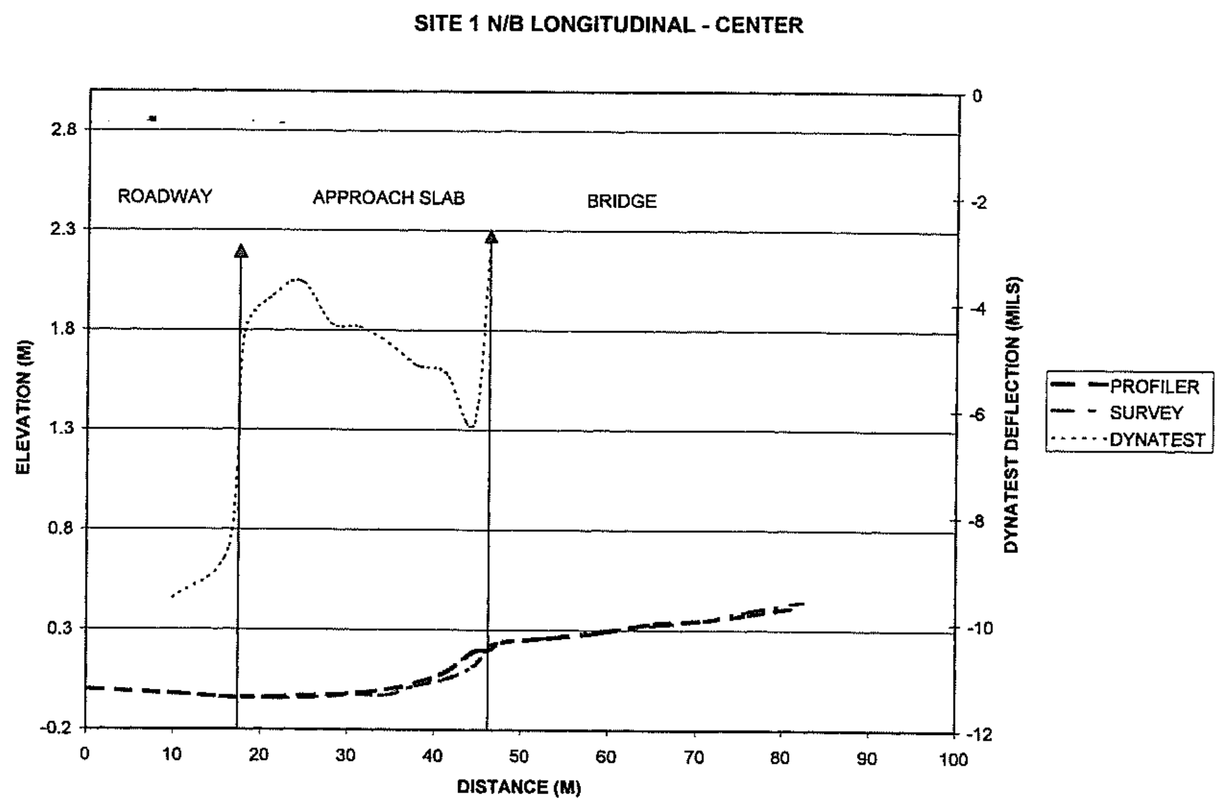


Figure 15
Sample graph of walking profiler, Dynatest, and survey for site 1 northbound

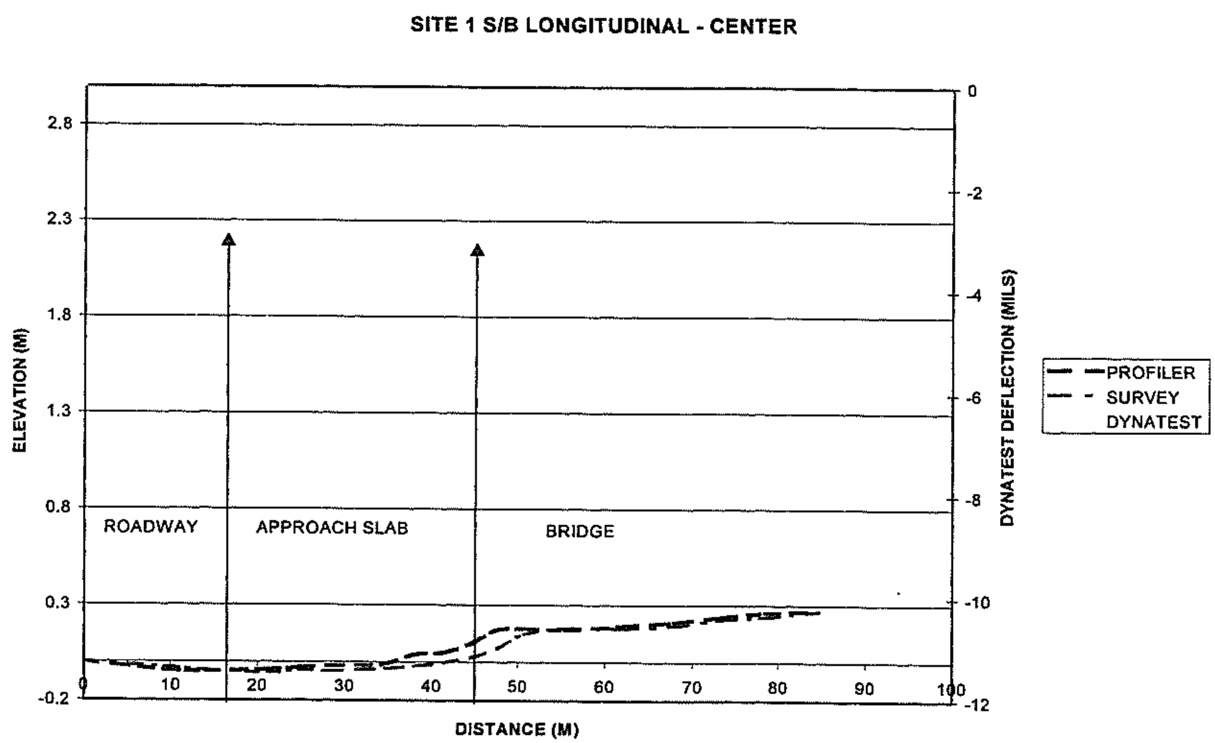


Figure 16
Sample graph of walking profiler, Dynatest and survey for site 1 southbound

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 17 and 18, 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 2. The graphs show that the approach slabs have a rating of poor to very poor.

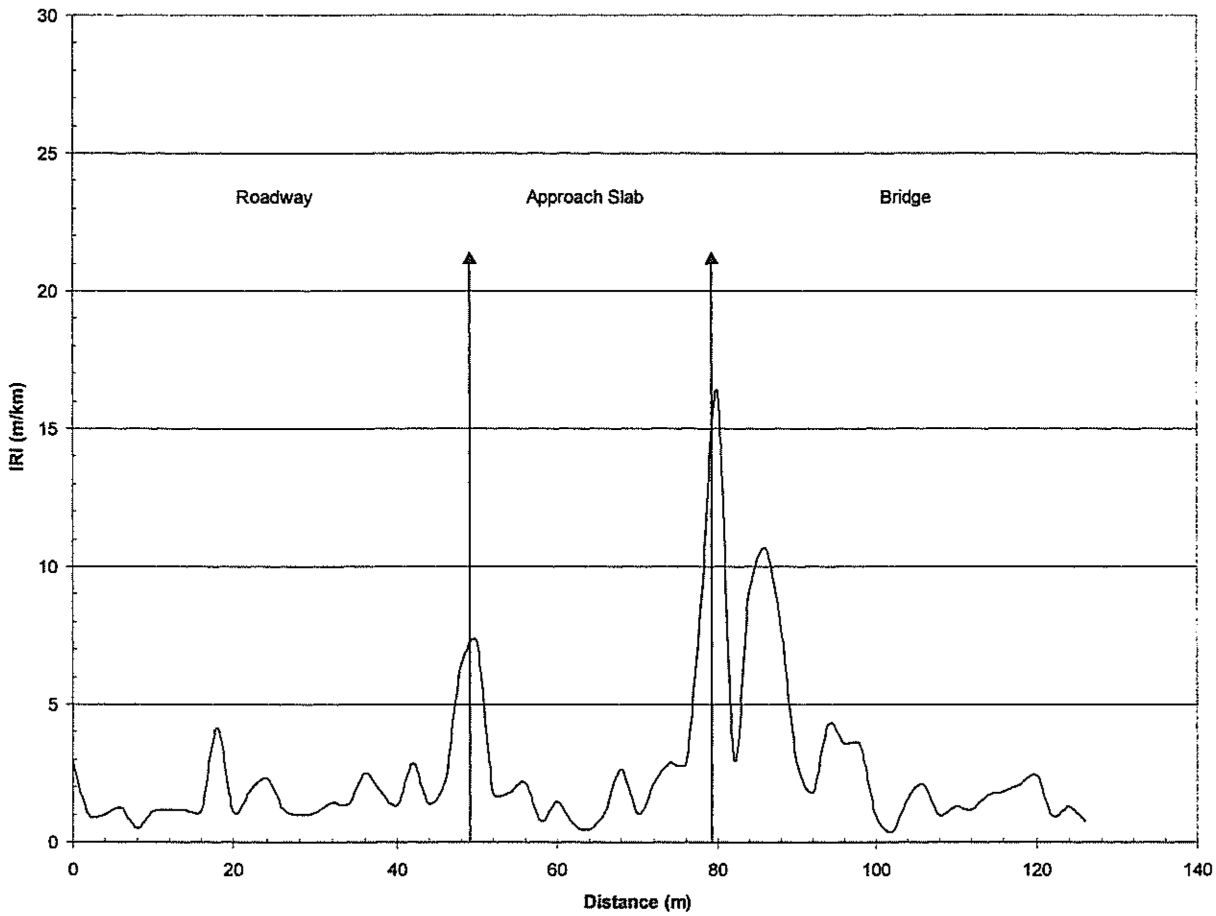


Figure 17
IRIs for site 1 northbound approach slab

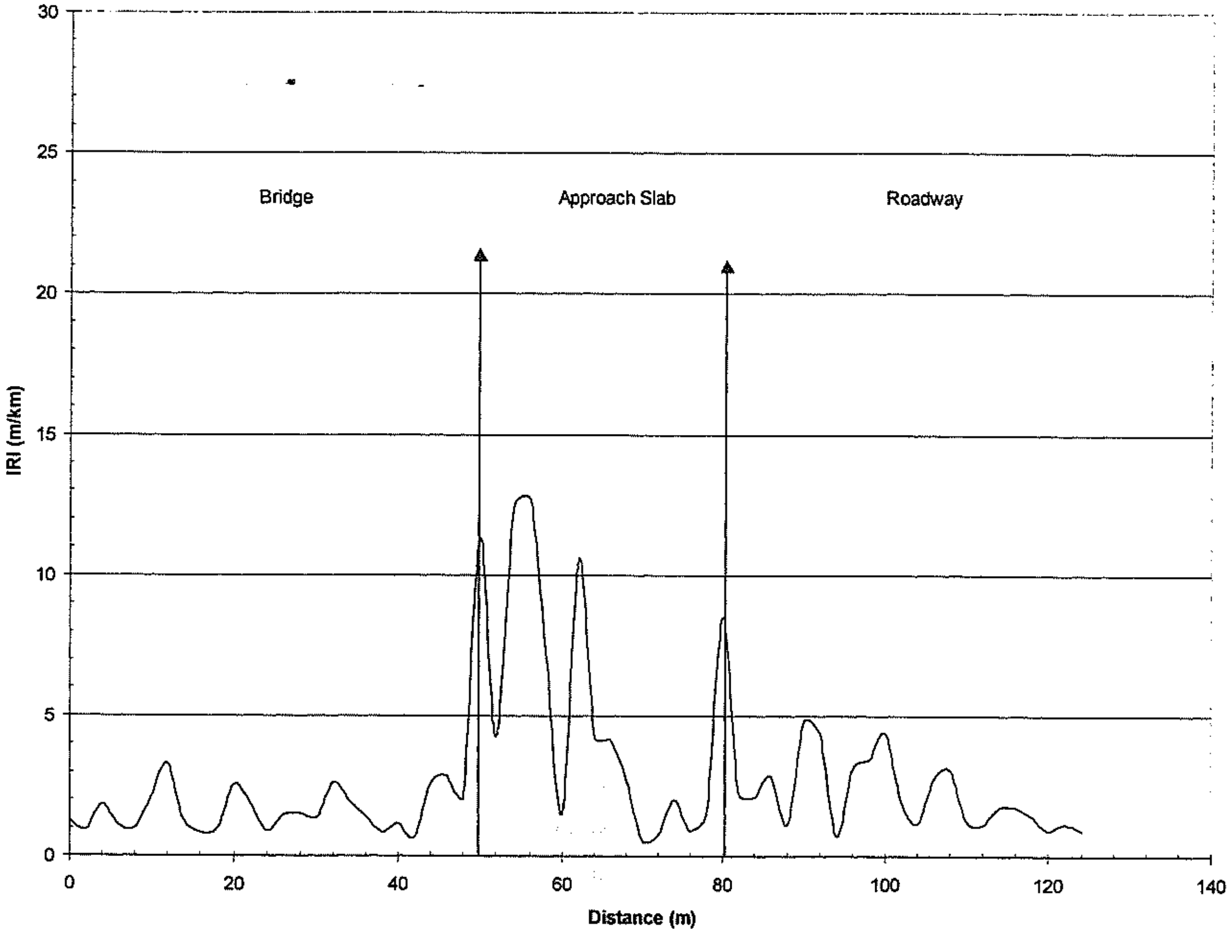


Figure 18
IRIs for site 1 southbound approach slab

International Roughness Index Slab (IRIS) Rating System

The highest IRI values for the approach slabs ranged from 3 to 27 (table 2). In order to evaluate the performance of the approach slabs, a new refined approach slab rating system (IRIS) was developed *[15]*. The proposed IRIS system is shown in table 10.

Table 10

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 was 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 was 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. In view of the results, it appears 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 11

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 12

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 13

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.

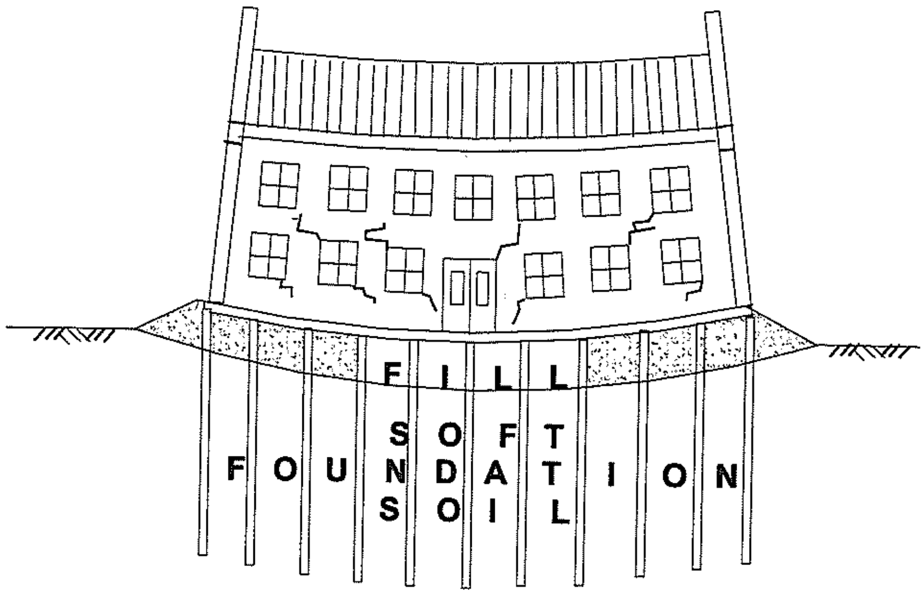


Figure 19
An example of negative friction on piles *[18]*

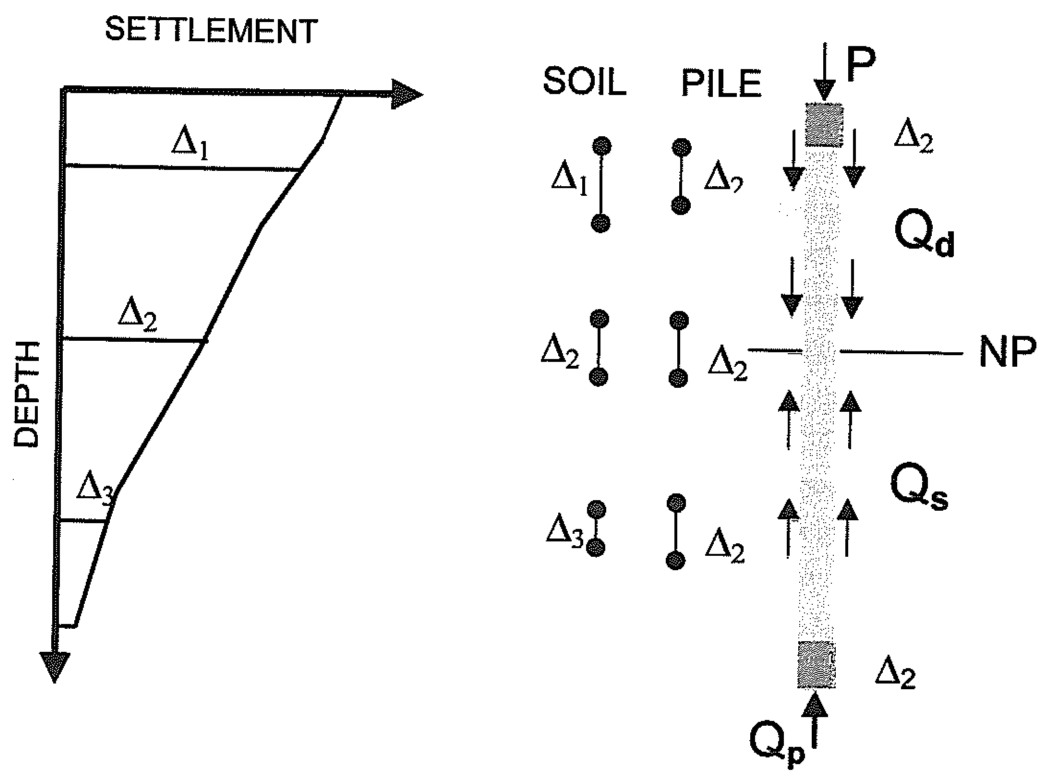


Figure 20
Equilibrium of forces on a pile *[18]*

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. 23).

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 this section.

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 *[19]* is used by DOTD for calculating consolidation settlement of soils. A typical shape of the soil settlement curve is shown in figure 21.As shown in figure 21, 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.

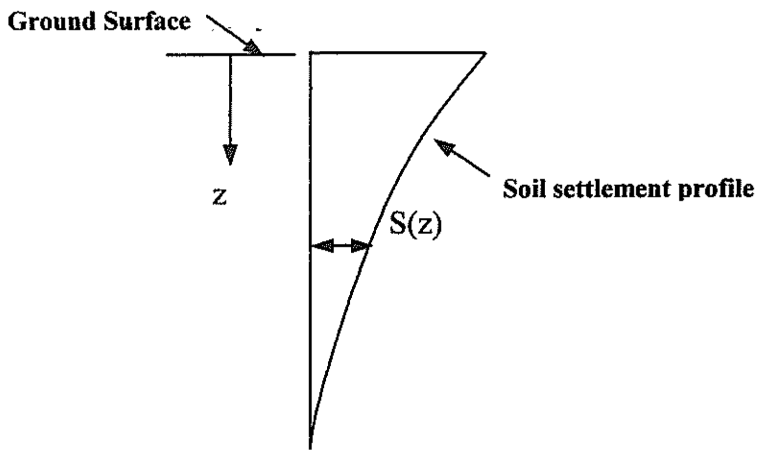


Figure 21
Settlement distribution along soil depth

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:

$$\Delta L_{tip} = \Delta L - \Delta L_{shaft} \tag{1}$$

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

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

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:

$$\Delta D(z) = S(z) - \Delta L(z) \tag{3}$$

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 22 illustrates the determination of the neutral point for a given pile head displacement.

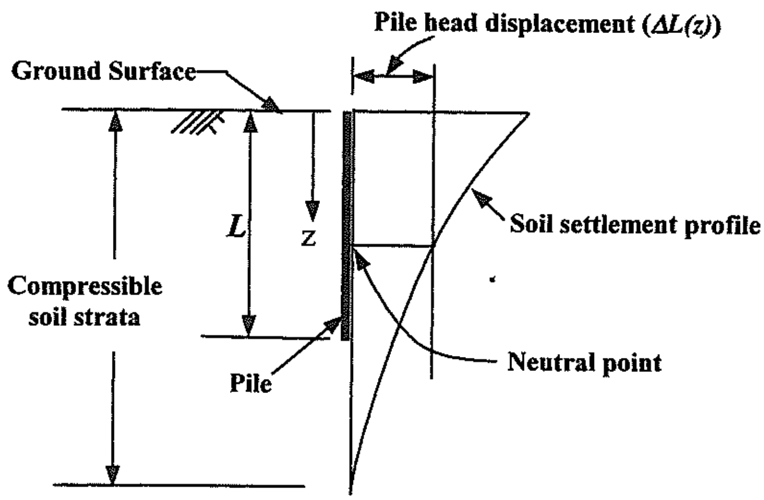


Figure 22
Neutral point illustration

According to the FHWA, frictional resistance per unit surface area of pile length at depth z, $f(z)$ is calculated from relative displacement $\Delta D(z)$ as [20]:

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

where f_s = Pile shaft skin resistance.

Thus, the pile head force ΔF is calculated as:

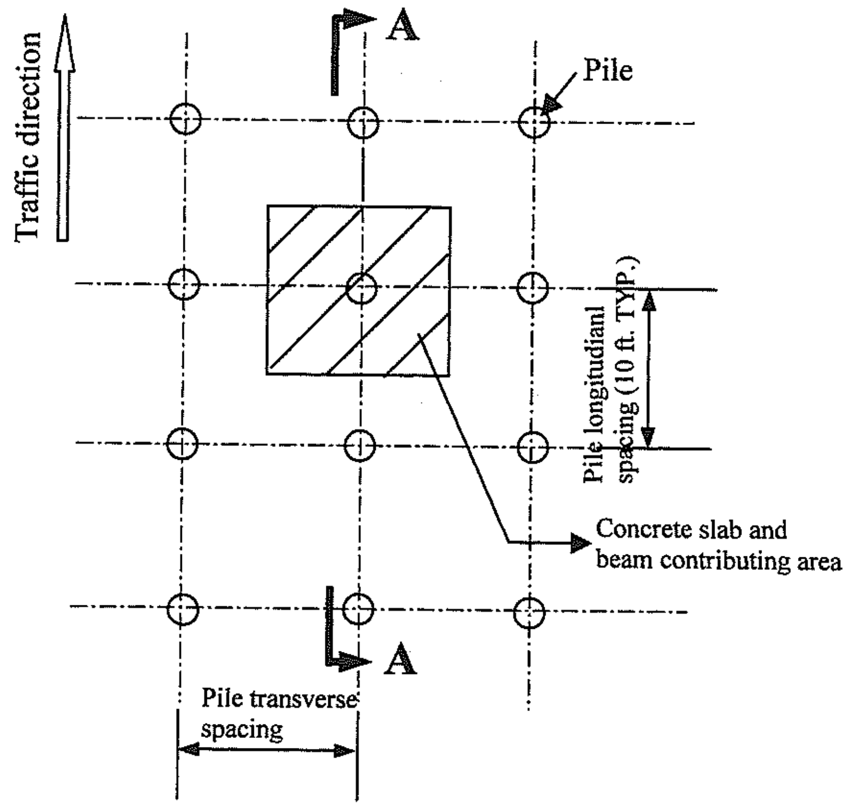
$$\Delta F = \int_0^L f(z) A_s dx \tag{5}$$

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

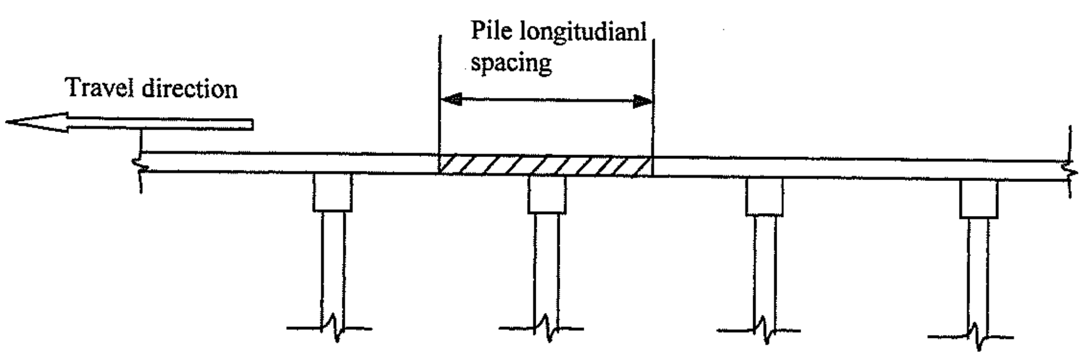
From the Eqn.5, the friction force distribution along a pile shaft could be obtained, which produces a resultant frictional force, ΔF. Theoretically, the pile friction stiffness S_p is then calculated as:

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

where S_p is a function of ΔL.



Plan view



A - A

Figure 23
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 27, 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. Three 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.

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, and each pile stiffness was calculated by using the spreadsheet computer program TU-DRAG developed at Tulane University and the DOTD pile capacity program *[19],[21]*. 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. 23).

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 14. 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 15. The 9 ft (2.74m) uniform embankment cross section is approximately as shown in figure 24. This information was obtained from the design/construction documents available at the DOTD offices which was collected during the study.

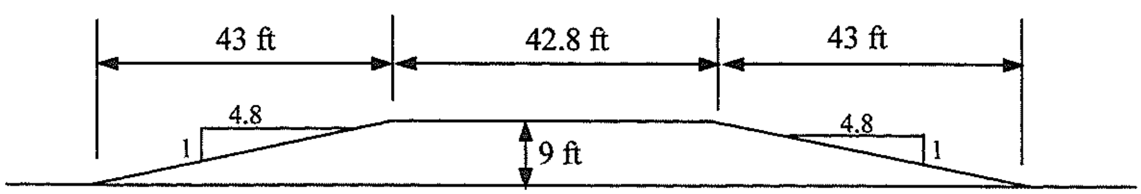


Figure 24

Site 1 approximate embankment dimensions

Table 14

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 1×10^{-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 15

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 and is shown in figure 25*[19]*.

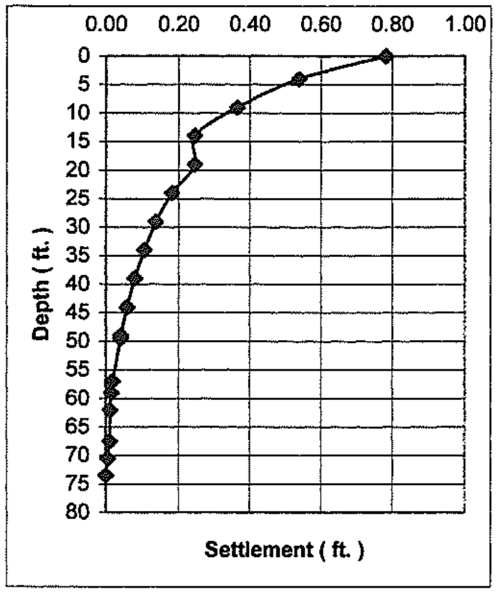


Figure 25
Site 1 calculated soil settlement curve

Pile head load (fig. 23).

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 26. The predicted profile was obtained by the spreadsheet program (TU-DRAG). 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.

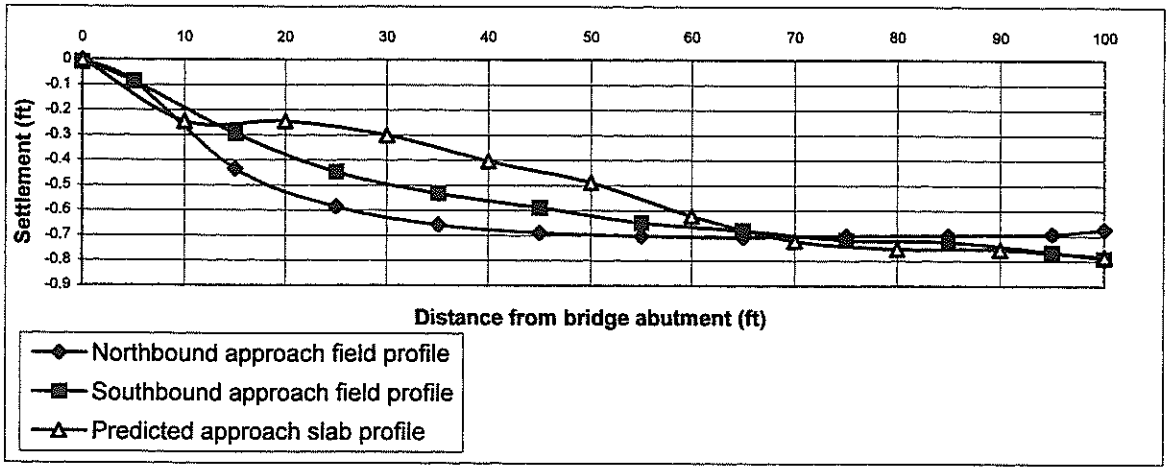


Figure 26
Measured and predicted approach slab settlement for site 1

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 16.

Table 16

* 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 27 illustrates the ideal settlement and calculated settlement curves based on the approximate pile lengths listed in table 16. As shown in figure 27, 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.

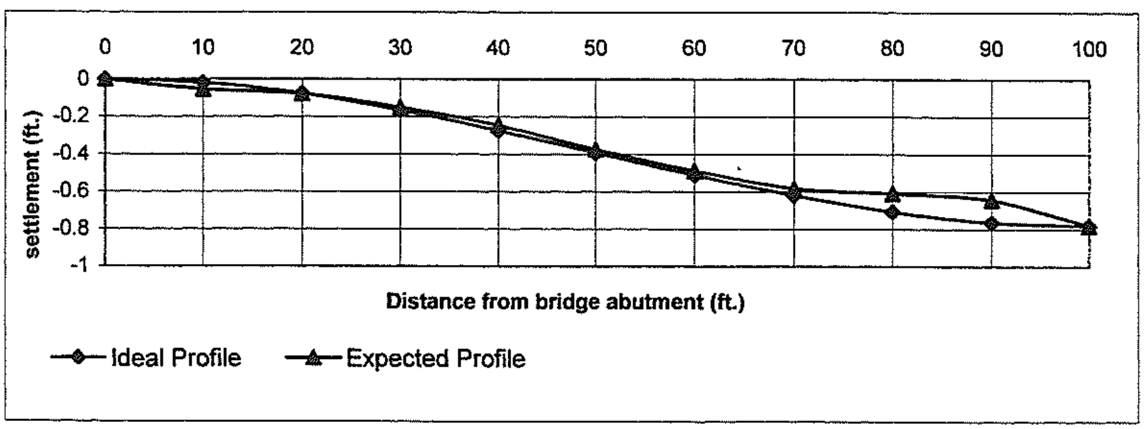


Figure 27

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 17. 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 18. The variable height embankment cross section is approximately shown in figure 28.

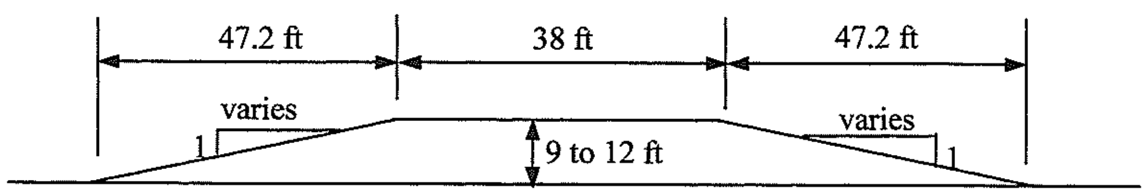


Figure 28

Site 5 approximate embankment dimensions

Table 17

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 18

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 29.

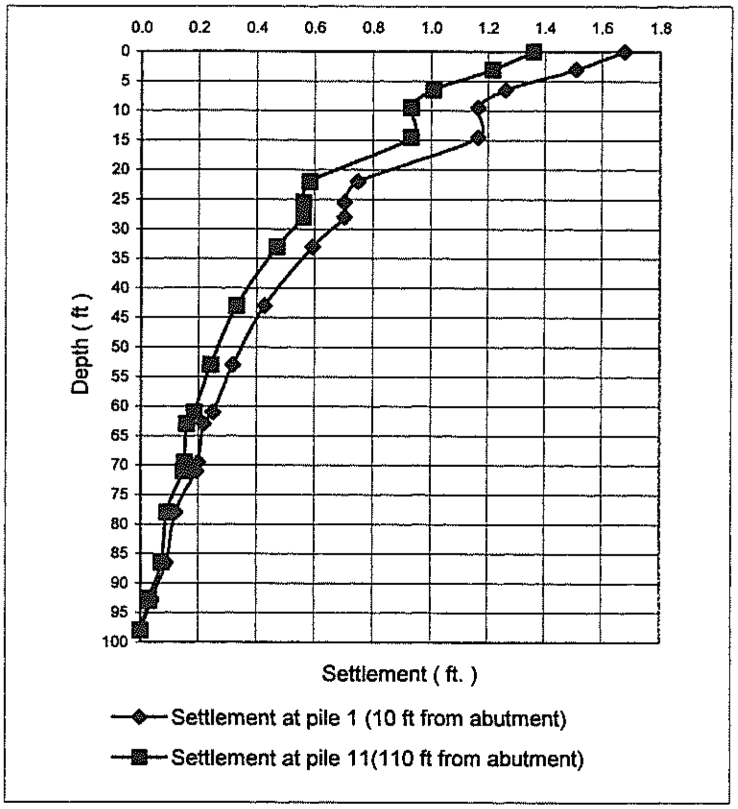


Figure 29
Site 5 calculated soil settlement curve

Pile head load (fig. 23).

Concrete slab:	$(10/12)\times8.75\times10\times150/1000$	= 10.94 kips (4.96 tons)
Concrete beam:	$2\times2\times8.75\times150/1000$	= 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 30. 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

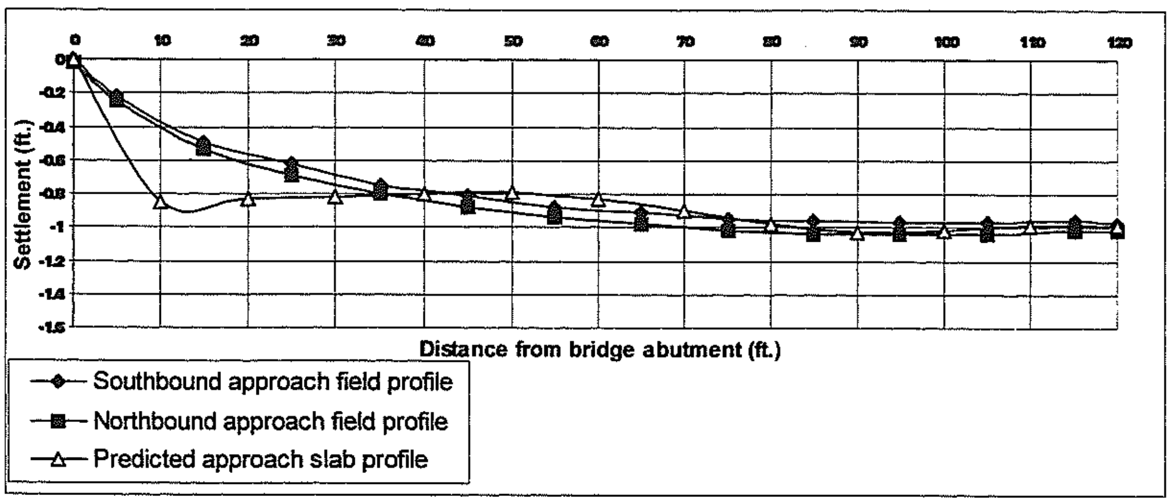


Figure 30
Measured and predicted approach slab settlement profiles at site 5

Table 19

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 19. Figure 31 illustrates the ideal and calculated settlement profiles based on the calculated pile lengths listed in table 19. Figure 31 also shows a close agreement between the estimated design profile and the ideal profile that should offer the smooth transition.

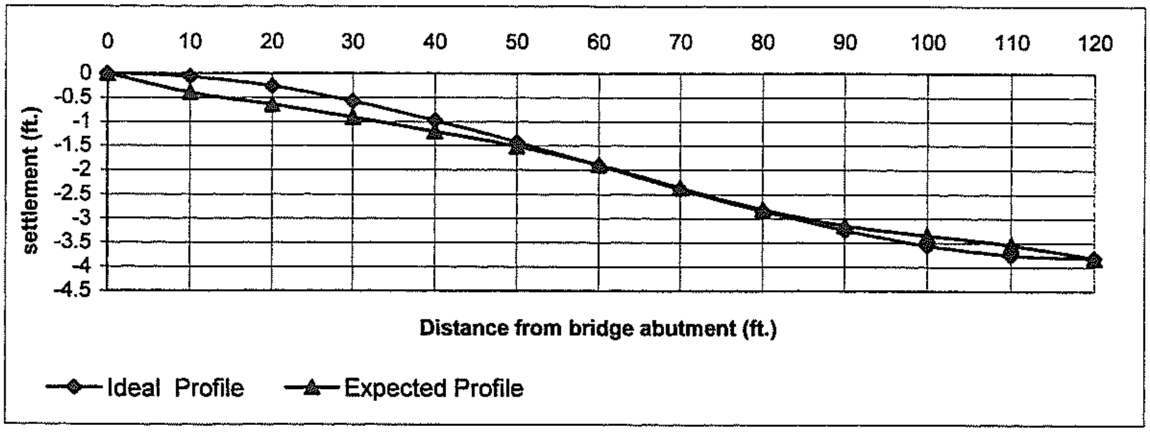


Figure 31
Ideal profile and calculated profile

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 20. 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 21. The embankment cross section is approximately as shown in figure 32.

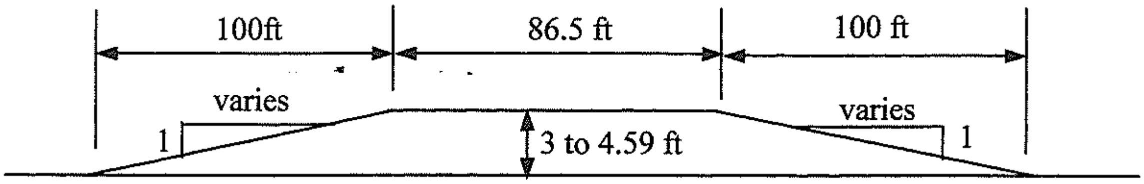


Figure 32
Site 7 approximate embankment dimension

Table 20
Site 7 soil properties

Soil Layer No.	Soil Thickness (ft.)	Unit Weight (pcf)	Initial Void Ratio	C_c	PMAX (tsf) {PMAX}	C_V 1×10^{-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 21
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 33. Only settlement curves for piles 1 and 11 are plotted in figure 33. Settlement curves for pile 2 through 10 fall between the two curves shown in figure 33 and have similar shapes.

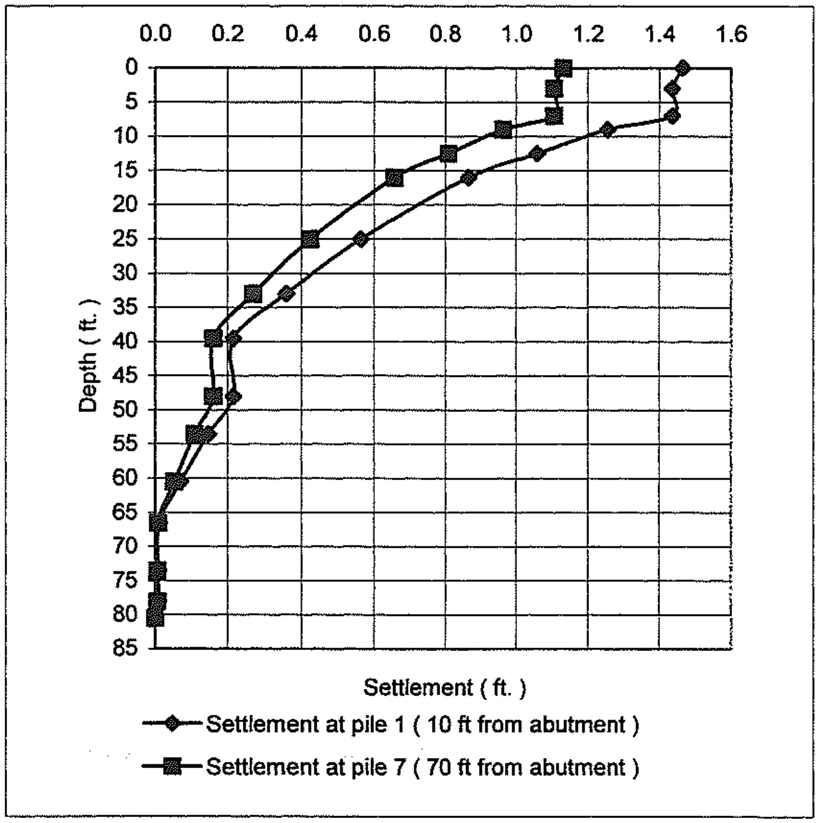


Figure 33
Site 7 calculated soil settlement curve

Pile head load (fig. 23).

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 34. 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 35, the length of each pile used in the field was found to be inadequate.

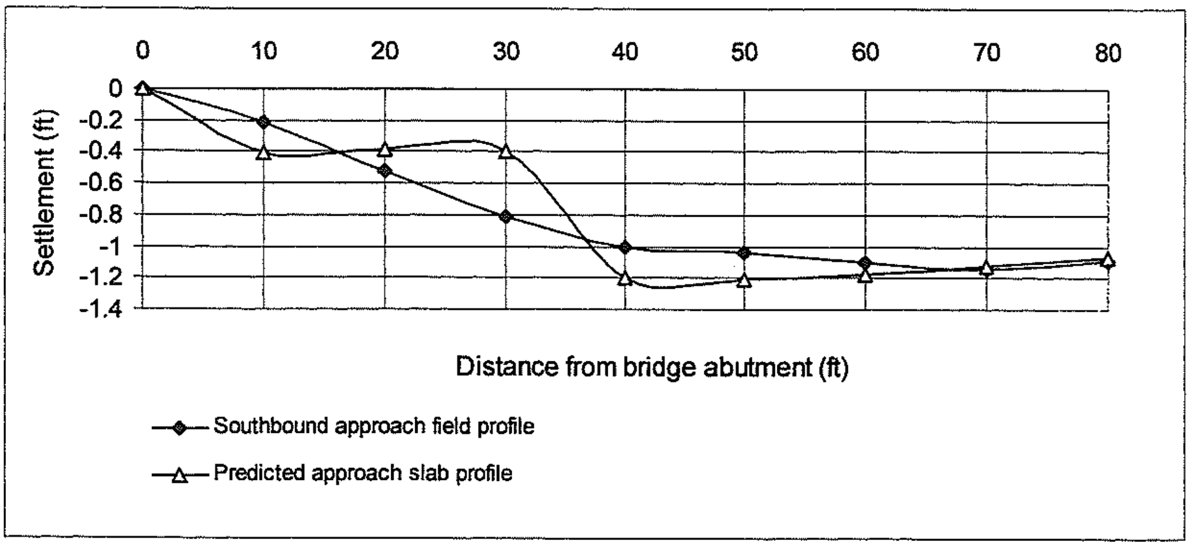


Figure 34
Measured and predicted approach slab settlement at site 7

The optimum pile length arrangement which yields the ideal curve for site 7 was found to be as tabulated below in table 22.

Table 22

*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 35 illustrates the ideal settlement curve and calculated settlement curve based on the pile lengths listed in table 22.

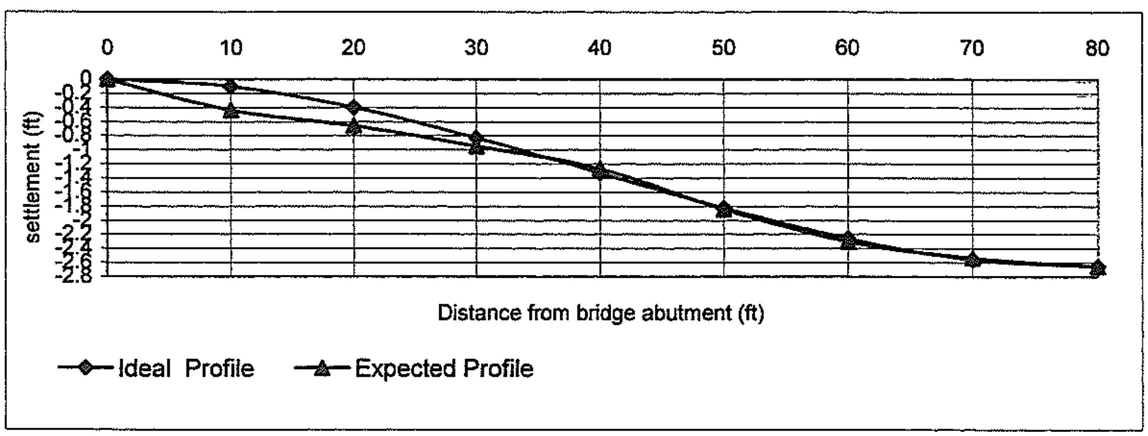


Figure 35
Ideal and calculated profile at site 7

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 23 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 23

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

Table 23 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 ninety 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 Excel 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.

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