Beckett Bridge

Project Development & Environment (PD&E) Study

from **Chesapeake Drive** *to* **Forest Avenue**
Tarpon Springs, Pinellas County, FL

Pinellas County Project ID: PID 2161 • ETDM #: 13040
FDOT Financial Project ID: 424385-1-28-01

July 2012

Cover Updated January 2016

Geotechnical Technical Memorandum

Prepared for:
Pinellas County
Department of Environment & Infrastructure
14 S Ft Harrison Avenue
Clearwater, FL 33756

Prepared by:
Tierra, Inc.
7351 Temple Terrace Highway
Tampa, Florida 33637

TABLE OF CONTENTS

LIST OF TABLES

LIST OF FIGURES

LIST OF APPENDICES

1.0 PROJECT SUMMARY

1.1 PURPOSE

Pinellas County, in coordination with the Florida Department of Transportation (FDOT) District Seven, is conducting a Project Development and Environment (PD&E) Study to evaluate alternatives to remove, rehabilitate or replace the existing Beckett Bridge (Bridge No. 154000) in Tarpon Springs, Pinellas County, Florida.

1.2 PROJECT DESCRIPTION

The existing bridge was originally constructed in 1924 as a timber structure with a steel movable span. The fixed timber approach spans were replaced with concrete approach spans in 1956. The bridge is considered historic, and is the only highway single-leaf rolling-lift bascule bridge remaining in Florida. Major repairs were performed in 1979, 1998 and in 2011. Major rehabilitation or replacement of the bridge is needed to keep the bridge open and operating efficiently.

The project limits extend along Riverside Drive from Chesapeake Drive across Whitcomb Bayou to Forest Avenue, a distance of approximately 0.3 mile. The existing two-lane bridge connects areas west and north of the Bayou to downtown Tarpon Springs. The bridge is also located on a popular route for access to Fred Howard Park, a Pinellas County park located approximately 3.1 miles west on the Gulf of Mexico. Riverside Drive/North Spring Boulevard is an extension of Tarpon Avenue, which is a designated evacuation route. (See Figure 1, Project Location.) Beckett Bridge provides access to major north/south arterials including Alternate US 19 and US 19 for coastal residents during hurricane evacuation. The bridge also provides access for emergency vehicles, including police, ambulance and fire.

Beckett Bridge is owned and operated by Pinellas County. A bridge tender is only present when required to open the drawbridge for a vessel; there are no full-time bridge tenders. US Coast Guard drawbridge opening regulations (33CFR117.341) states that “The draw of the Beckett Bridge, mile 0.5, at Tarpon Springs, Florida shall open on signal if at least two hours’ notice is given.” Whitcomb Bayou connects to the Gulf of Mexico via the Anclote River to the north. Boats docked along Whitcomb, Spring and Minetta Bayous, and along artificial canals which connect to the southeastern portion of the Whitcomb Bayou, must pass the Beckett Bridge to access the Gulf of Mexico.

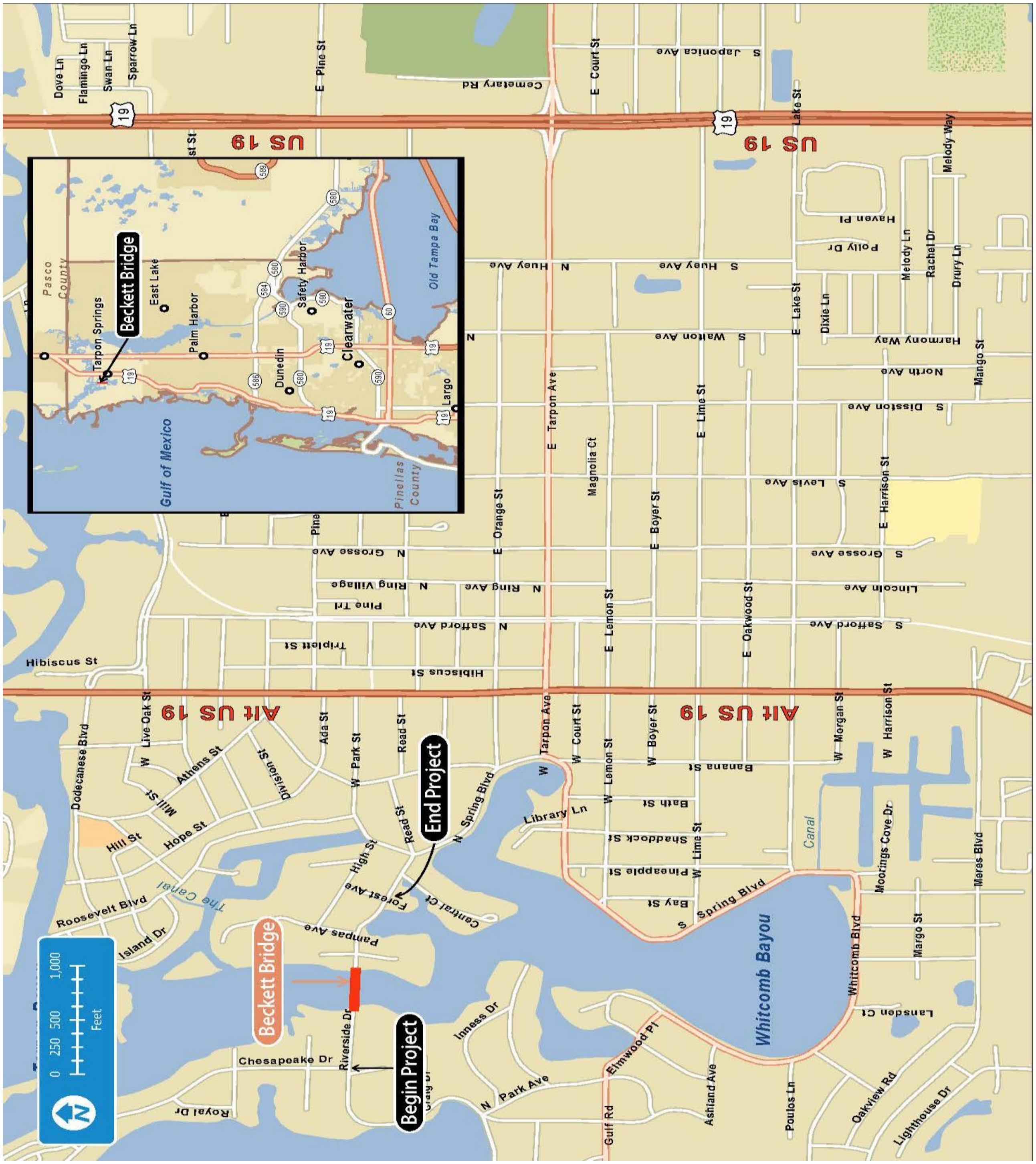


Figure 1 – Project Location Map

Project Need

The bridge is considered functionally obsolete. This designation is based primarily on the substandard clear roadway width of only 20 feet and substandard roadway safety features. The existing typical section consists of one, 10-foot wide travel lane in each direction and 2-foot 2-inch-wide sidewalks separated by a curb on both sides of the bridge. (See Figure 2 – Existing Bridge Typical Section.)

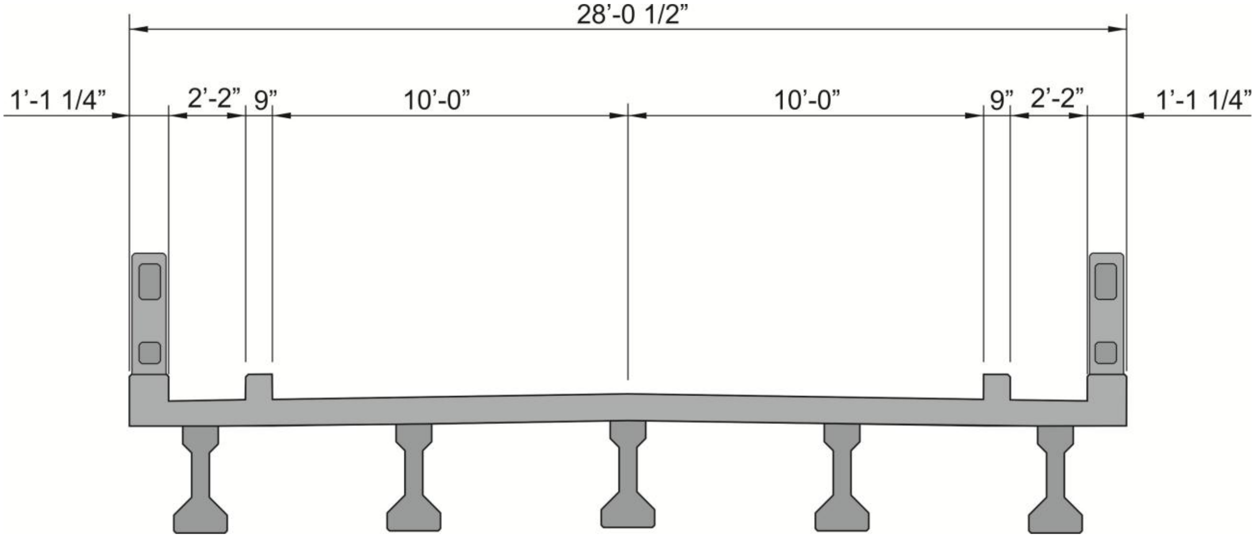


Figure 2 – Existing Bridge Typical Section

Minimum required lane and shoulder widths prescribed by the American Association of State Highway and Transportation Officials (AASHTO) are not met. The sidewalks on the bridge are narrow and do not meet current accessibility requirements established by the Americans with Disabilities Act (ADA). The bridge railings do not meet current standards for pedestrian safety or geometric and crash testing safety standards for vehicles. Approach guardrail and transitions and end treatments also do not meet current safety standards.

According to recent (10/27/09) FDOT inspection reports, the existing bridge has an overall Structure Inventory and Appraisal Sufficiency Rating of 44.9 out of 100. (Sufficiency ratings are a method of evaluating highway bridges by calculating a numeric value between 0 and 100, indicative of bridge sufficiency to remain in service). Bridges with a sufficiency rating less than 50 are eligible for federal replacement funds.

Although the bridge is not considered Structurally Deficient, the bridge has a substandard load carrying capacity requiring weight restrictions. The bridge is currently posted for legal loads limited to two-ton Single Unit Trucks and 15-ton Combination Trucks. Repairs in 1979 and 1988 included installation of crutch bents due to settlement and lateral stability concerns. Repairs in 2011 were performed to correct issues with the operating machinery and bascule leaf alignment.

The existing vertical clearance at the fenders is six feet. The tip of the bascule leaf overhangs the fender with the leaf fully raised and does not provide unlimited vertical clearance between the fenders. The existing horizontal clearance between the fenders is 25 feet.

Alternatives Considered

The following alternatives will be evaluated during the study:

- No-Build - Maintain Existing Bridge
- No-Build - Remove Existing Bridge (includes alternate routing of traffic)
- Rehabilitation of the Existing Bridge
- Replace with a new Movable Bridge
- Replace with a new Fixed Bridge

The “No-Build” alternative includes only routine maintenance to keep the bridge open to traffic until safety issues would require it to be closed. Evaluation of future improvements would occur at a later date. The “No Build with Removal of the Existing Bridge” would result in routine maintenance in the near future with the intent to demolish the bridge when it is no longer safe for traffic, with no plans to replace it with a new one. All bridge replacement alternatives considered will be constructed in approximately the same location as the existing bridge to minimize impacts.

Alternate corridors for bridge location will not be evaluated due to the extent of development in the vicinity of the existing bridge. Capacity improvements will not be considered. The complete removal alternative will examine alternative traffic routes and potential impacts to the community and on traffic operations.

Proposed Typical Sections

The proposed bridge typical section was based on a 35 mph design speed. The governing specifications include design criteria specified by the American Association of State Highway and Transportation Officials (AASHTO), the Florida Green Book and the FDOT Plans and Preparation Manual. A detailed discussion of design criteria will be included in the Preliminary Engineering Report, published separately for this project. The typical section has a total out-to-out width of 47 feet 1 inch as shown in Figure 3. The typical section includes two, 11-foot wide travel lanes with 5.5-foot shoulders that can function as undesignated bicycle lanes. Sidewalks, 5.5 feet wide, are proposed on both sides of the bridge. Proposed sections on the roadway approaches were developed to avoid acquisition of additional right-of-way.

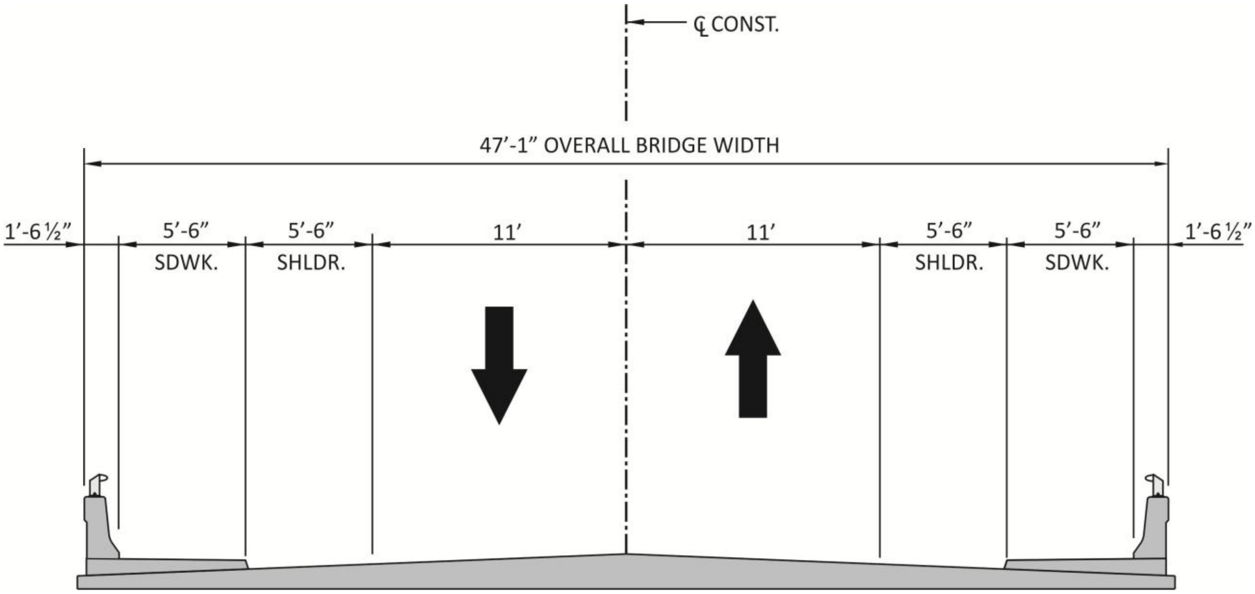


Figure 3 – Proposed Bridge Typical Section

2.0 SCOPE OF SERVICES

The geotechnical portion of the PD&E study was to obtain and evaluate information on the existing subsurface conditions within the project limits to assist in the preparation of the PD&E Report for the project. The following services were provided for this summary:

- Reviewed published information on topographic, soils and groundwater conditions. Soil, groundwater and regional geology information was obtained from the Web Soil Survey of Pinellas County, Florida published by the United States Department of Agriculture (USDA) – Natural Resource Conservation Service (NRCS). Topographic information was obtained from appropriate topographic maps published by United States Geological Survey (USGS).
- Reviewed previous geotechnical explorations and reports and summarized the collected data to support the PD&E study for the project.
- Prepared this Geotechnical Memorandum for the project.

3.0 SUBSURFACE CONDITIONS

3.1 USGS TOPOGRAPHIC SURVEY

The USGS topographic survey map titled "Tarpon Springs, Florida" was reviewed. The natural ground surface elevations appear to be within a range of about +5 feet to +10 feet National Geodetic Vertical Datum of 1929 (NGVD29). A reproduction of the USGS maps is presented on **Figure 4.0**.

3.2 REGIONAL GEOLOGY

The regional geology presented below is as presented in the USDA Soil Survey of Pinellas County, Florida.

The two major geologic formations in Pinellas County are the Hawthorn Formation of the lower Miocene and Caloosahatchee Marl of the lower Pliocene. The border between these formations extends across the peninsula north of the Cross Bayou Canal through Safety Harbor and Oldsmar. The Hawthorn Formation underlies soils north of this line.

The Hawthorn Formation consists of interbedded sand, clay, marl, limestone, lenses of fuller's earth, and land-pebble phosphate. Soils that occur on the side slopes of depressions northeast of Clearwater and in cuts made by Curlew Creek north of Dunedin contain phosphatic material from this formation.

During the Pleistocene, marine deposits that formed four terraces covered these formations. A mantle of sand that ranges from two to 35 feet in thickness covered these terraces. These terraces are described below:

The Pamlico terrace occurs at an elevation of 0 to 25 feet above mean sea level. It is mainly sand, one to 15 feet thick. In areas near Oldsmar, St. Petersburg, and Pinellas Park, the sand is only one to 4 feet thick and is underlain by Caloosahatchee Marl.

Soils of the Oldsmar and Wabasso series that have acidic sand upper horizons and nonacidic, loamy subsoil formed on this terrace.

The Talbot terrace is 25 to 42 feet above mean sea level. It is fine sand not more than 16 feet thick. In a few places, the sand mantle is thin and soils have been affected by phosphatic material from underlying Hawthorn Formation. Most soils of the Talbot terrace are acidic. Soils of Astatula, Immokalee, Myakka, and Pomello series formed this terrace.

The Penholoway terrace is 42 to 70 feet above mean sea level. It is mostly fine sand as much as 28 feet thick. The Hawthorn Formation underlies it. On sides of depressions the sand mantle is thin, and materials from the Hawthorn Formation have affected the soils. Most soils on this terrace are acidic. A few nonacid soils occur in small isolated areas in depressions and along streams. Soils of the Astatula, Immokalee, Myakka, Paola, Pomello, and St. Lucie series formed this terrace.

The Wicomico terrace is 70 to 97 feet above mean sea level. It is mainly fine sand as much as 27 feet thick. The Hawthorn Formation underlies it. The soils on this terrace are dominantly acid sands of the Astatula, Immokalee, Paola, Pomello, and St. Lucie series.

A few pockets of recently deposited muck and freshwater marl occur in low areas. With few exceptions, individual soils are confined to a particular geologic formation or marine terrace. For example, Pinellas soil that formed in fresh-water alkaline deposits on upland terraces are very similar to Pinellas soil that formed in alkaline sediments of Caloosahatchee Marl. Though variations in characteristics of the parent material are apparent in the field, they do not affect soil classification.

3.3 PINELLAS COUNTY SOIL SURVEY

Based on a review of the Pinellas County Soil Survey published by USDA-NRCS, it appears that there are three soil-mapping units noted within the project limits. A detailed soil survey map is shown on **Figure 4.** The general soil descriptions are presented in the sub-sections below, as described in the Web Soil Survey. Table 3-1 summarizes information on the soil mapping units obtained from the Web Soil Survey.

3.3.1 Astatula Soils and Urban Land (Unit 4)

The Astatula component makes up 50 percent of the map unit. Slopes are 0 to five percent. This component is on ridges on marine terraces on coastal plains. The parent material consists of eolian or sandy marine deposits. This soil is not flooded. It is not ponded. There is no zone of water saturation within a depth of 72 inches.

Generated brief soil descriptions are created for major soil components. The Urban land is a miscellaneous area.

3.3.2 Matlacha and St. Augustine Soils and Urban Land (Unit 16)

The Matlacha component makes up 32 percent of the map unit. Slopes are 0 to two percent. This component is on fills on ridges on marine terraces on coastal plains. The parent material consists of sandy mine spoil or earthy fill.

This soil is not flooded. It is not ponded. A seasonal zone of water saturation is at 30 inches during June, July, August, September, and October.

The St. Augustine component makes up 32 percent of the map unit. Slopes are 0 to 2 percent. This component is on ridges on marine terraces on coastal plains. The parent material consists of sandy mine spoil or earthy fill. This soil is not flooded. It is not ponded. A seasonal zone of water saturation is at 27 inches during June, July, August, September, and October.

Generated brief soil descriptions are created for major soil components. The Urban land is a miscellaneous area.

3.3.3 Tavares Soils and Urban Land (Unit 29)

The Tavares component makes up 50 percent of the map unit. Slopes are 0 to 5 percent. This component is on knolls on marine terraces on coastal plains, ridges on marine terraces on coastal plains. The parent material consists of eolian or sandy marine deposits. This soil is not flooded. It is not ponded. A seasonal zone of water saturation is at 57 inches during June, July, August, September, October, November, and December.

Generated brief soil descriptions are created for major soil components. The Urban land is a miscellaneous area.

Table 3-1
Pinellas County USDA NRCS Soil Survey Information

USDA Map Unit and Soil Name	Depth (in)	Soil Classification		Permeability (in/hr)	pH	Seasonal High Water Table	
		USCS	AASHTO			Depth (feet)	Months
(4) Astatula-Urban land	0-3	SP, SP-SM	A-3	20.0 - 49.9	4.5-6.5	---	Jan-Dec
	3-80	SP, SP-SM	A-3	20.0 - 49.9	4.5-6.5		
	---	---	---	0.0 - 0.0	---	---	Jan-Dec
(16) Matlacha St. Augustine-Urban land	0-42	SP, SP-SM	A-3	2.0 - 6.0	6.1-8.4	2.0-3.0	June-Oct
	42-80	SP, SP-SM	A-3	6.0 - 20.0	6.1-8.4		
	0-8	SP, SP-SM	A-3	6.0 - 20.0	6.1-8.4	1.5-3.0	June-Oct
	8-33	SP-SM	A-2-4	2.0 - 20.0	6.1-8.4		
	33-48	SP, SP-SM	A-3	6.0 - 20.0	6.1-8.4		
	48-63	SM, SP-SM	A-2-4	2.0 - 20.0	6.1-8.4		
	63-80	SP, SP-SM	A-3	6.0 - 20.0	6.1-8.4		
	---	---	---	0.0 - 0.0	---	---	Jan-Dec
(29) Tavares-Urban Land	0-5	SP, SP-SM	A-3	6.0 - 20.0	3.5-6.5	3.5->6.0	June-Dec
	5-80	SP, SP-SM	A-3	6.0 - 20.0	3.5-6.5		
	---	---	---	0.0 - 0.0	---	---	Jan-Dec

3.4 GROUNDWATER CONDITIONS

Riverside Drive and the Beckett Bridge crosses the Whitcomb Bayou/Minetta Branch of the Anclote River. Based on the USDA Soil Survey of Pinellas County, Florida, the seasonal high groundwater table ranges from about 1½ to greater than six feet below grade. Due to the proximity of the project to the river and Bayou it is anticipated that the water table is tidally influenced.

3.5 REVIEW OF POTENTIOMETRIC SURFACE MAPS

Based on a review of the "Potentiometric Surface of the Upper Floridan Aquifer, West Central Florida" maps published by the USGS, the potentiometric surface elevation at the bridge site ranges from approximately +5 feet to +10 feet NGVD 29. As indicated in Section 3.1, the project site elevations range from approximately +5 feet to +10 feet, NGVD 29. It should be noted that artesian conditions were not noted within test borings completed by others at the project site.

4.0 PRELIMINARY ENGINEERING EVALUATIONS

4.1 SHALLOW SOIL SUITABILITY

Based upon the USDA-NRSC Soil Survey for Pinellas County, sandy soils to depths of 80 inches below the natural ground surface are reported along the entire project limits. In general, these sandy soils are suitable for supporting the proposed improvements after proper subgrade preparation and removal of unsuitable materials.

The near surface soils within 80 inches are reported to consist of A-3 and A-2-4 select sandy soils. These soils are anticipated to be suitable for roadway subgrade and roadway fill materials. It is recommended that soil test borings be completed during final design activities to verify soil suitability.

4.2 ROADWAY CONSTRUCTION

Site preparation should consist of normal clearing and grubbing followed by compaction of subgrade soils. Subgrade preparation will include the removal of plastic soils and top-soils and organic soils in accordance with FDOT Design Standard Index 500. Backfill embankment materials should consist of materials conforming to FDOT Design Standard Index 505. Clearing and grubbing and compaction should be accomplished in accordance with the latest FDOT Standard Specifications for Road and Bridge Construction (SSRBC).

The overall site preparation and mechanical densification work for the construction of the proposed roadway should be in accordance with the FDOT SSRBC and Standard Index requirements. In general, the existing subsurface soils appear capable of supporting the construction of the proposed roadway improvements subject to the above geotechnical considerations and after proper subgrade preparation.

4.3 GEOTECHNICAL BRIDGE CONSIDERATIONS

The Beckett Bridge is a multi-spanned bridge that has been reported to have experienced lateral movement and subsidence. The bridge is a two- lane bascule bridge about 20 feet across and 360 feet in length with two-foot 2 inch wide sidewalks on both sides. We understand the approach span structures are constructed on 14- inch square prestressed concrete piles. There are four spans on the east approach and five spans on the west approach. The bascule is approximately 40 feet long and is supported on a concrete pier. The bridge was originally constructed in 1924 using timber piling and timber bents. The bridge approach spans were reconstructed in 1956 using reinforced concrete, however, the original bascule span remained. Structural repairs were performed between 1979 and 2011 including the installation of crutch bents.

4.3.1 Previous Geotechnical Studies

Williams Earth Sciences provided a report dated November 10, 1994, which provided recommendations for the installation of crutch bents using H-Piles. During the 1994 study, Williams preformed three Standard Penetration Tests (SPT) borings; one was performed at the west abutment, one at the east abutment, and one was performed in the vicinity of the Bent 5, adjacent to the bascule. The two abutment borings were performed from land and the Bent 5 boring was performed from the bridge (as opposed to a barge over water). Two SPT borings were also performed by Professional Service Industries (PSI). These two borings were performed at Bent 6 from the bridge. One was performed in the westbound lane and the other was performed in the eastbound lane. The report for this study, as submitted to the E.C. Driver team, is attached as **Appendix A**.

An additional geotechnical study was completed in 2009 by Williams Earth Sciences which included an Electrical Resistivity Geophysical Report by Subsurface Evaluations, Inc. (SEI). The Williams report along with the SEI report is provided as **Appendix B** and the soil descriptions and discussion is summarized below.

During the 2009 study, Electrical Resistivity Imaging (ERI) was conducted. The purpose of the ERI testing was to determine the vertical extent and lateral continuity of soil layers and to identify possible karst hazards within the river along the sides of the bridge. The ERI testing was performed by "Subsurface Evaluations, Inc." (SEI) and their report, dated April 28, 2009, is included in Appendix B.

The results of the ERI testing indicated several features and anomalies within the vicinity of the bridge footprint. First, there appears to be an anomaly near Bent 6, with the center approximated just north of the bridge, as depicted on Figure 1 of the SEI report. In addition, there appears to be a shelf at about 20 to 40 feet in depth indicating a change in soil material and/or density, as indicated on Figure 1 of the 2009 report.

Boring B-1 (PSI) was performed very close to the ERI anomaly indicated at Bent 6. PSI Boring B-1 indicates that there is a dense grading to medium dense dark brown to brown fine sand with trace of silt from the mud-line to about 10 feet below the mud-line, followed by a nine foot thick layer of stiff dark gray sandy silt, from 10 to 19 feet below the mud-line.

The silt layer was underlain by a relatively thin layer of hard limestone, from 19 to 24 feet below the mud-line. From 24 to 40 feet below the mud-line, a medium dense grading to very loose layer of brown sand with trace of silt (SP-SM) was encountered.

A second layer of hard limestone was present from 40 to 45 feet below the mud-line, followed by a medium dense brown fine sand with trace of silt (SP-SM) to the termination depth of the boring at about 57 feet below the mud-line.

Boring B-1 (PSI) and the ERI results correlate at Bent 6. In addition, this anomaly can be considered indicative of Karst conditions and potential weathering/ solutioning of the limestone. Boring B-2 was also performed at Bent 6, on the opposite side of the bridge (eastbound lane). This boring indicated somewhat similar soils to Boring B-1, however, there was no evidence of the stiff silt layer at 10 to 19 feet below the mud-line.

The borings conducted by Williams in the 1994 study indicated a soil stratigraphy that was quite dissimilar to the borings conducted at Bent 6 by PSI. These borings generally indicate a surficial layer of sands to silty sands or clayey soils, followed by very hard limestone to the full depth of the borings. There were a few minor variations in the subsurface soils, such as a thin layer of clay (CH) material in boring B-1 at a depth of 47 to 58 feet below the ground surface; a very loose shelly fine sand layer from 77 to 84 feet below the mud-line at boring B-2; and a possible void from 69 to 71 feet below the ground surface at boring B-3. The medium dense fine sand with trace of silt soil was not encountered in the SPT borings conducted by Williams.

Encountering highly dissimilar soils in a relatively short distance indicates that this area potentially has localized karst features. The Anclote River area is known for variable subsurface conditions and karst features. The subsurface is characterized by a sand layer overlying a shallow limestone. There is a lack of clay layering in this area and this condition can promote localized subsidence and raveling of the surficial soils into the karst limestone. Review of the ERI results indicates that the surficial karst solution features, or surficial relic sinkhole features, may be more prevalent near the center of the bridge. There also appears to be an apparent shelf, as indicated on ERI transects T3 and T4. Review of ERI transects T3, T4 and T5 indicate the possibility of a solution zone near to and below the bridge footprint that may be located in a southwest orientation. However, it should be noted that the bascule bridge footing and the piles may be providing interference of the ERI data and therefore **additional geotechnical exploration is warranted to verify subsurface conditions.**

The Williams report indicates that there has been settlement and rotation of the bents and/or bascule pier. There are a number of potential causes for this, both structurally and geotechnically, however, from a geotechnical standpoint, the causes may be due to subsidence of the piles due to 1) active solutioning of the limestone, or 2) insufficient pile bearing both axially and laterally, or a combination of both. Another consideration is the age of the timber piles supporting the bascule pier, which are more than 85 years old. The timber piles could be in poor condition due to fatigue, rot or some other form of deterioration.

HP 14x73 crutch bent piles were installed in 1996. The 1996 plans indicate crutch bents at Bent 6 and Bent 7, and pier stabilizers for the bascule. The lengths of the crutch bent piles varied dramatically from tip elevations of about -30 to -200 feet. These lengths were taken from old facsimile correspondence between Williams and DSA.

There was a minimum tip elevation of -35 feet indicated on the plans; therefore, one of the piles did not achieve the minimum tip elevation in accordance with the plans. The piles were also supposedly preformed to an elevation of -27 feet, and the preformed hole was supposed to be grouted. The HP crutch bent piles were also planned to be jacketed using an epoxy mix from elevation -4 to +4 feet, at the splash zone of the piles. Based on the 2007 Bridge Inspection Report, performed by Volkert & Associates, Inc., the "jackets are in good condition with no washouts or exposed base pile".

4.4 GEOTECHNICAL BRIDGE RECOMMENDATIONS

Tierra understands that the bridge is under evaluation for:

- No-Build - Maintain Existing Bridge
- No-Build - Remove Existing Bridge (includes alternate routing of traffic)
- Rehabilitation of the Existing Bridge
- Replace with a new Movable Bridge
- Replace with a new Fixed Bridge

For the maintenance and rehabilitation alternatives, settlement and rotation monitoring of the bents and piers is recommended to determine the location and rate of movement that it is occurring so that the bents and/or piers can be shored to stabilize the settlement and rotation. Evaluation of how to shore the bents and/or piers can then be made.

Additional test borings will be required if settlement and rotation is ongoing to use as part of the design and construction of repair/modifications.

4.4.1 Geotechnical Bridge Replacement Considerations

If it is determined that the bridge will be replaced, then additional soil borings will be required as part of the design process.

Evaluations of foundation alternatives for a bridge replacement were based on the results of subsurface conditions encountered in the borings performed by others at the bridge site. Based on our experience with similar projects, we initially considered the following foundation alternatives:

- Shallow Foundations
- Steel Piles, including Pipe and H Sections

- Pre-stressed Square Concrete (PSC) Piles (18 and 24 inch square)
- Drilled Shafts

The following paragraphs discuss each of these alternatives briefly.

4.4.2 Shallow Foundations

With shallow foundation systems, the structure loads are supported by the bearing capacity of the foundation soils. The design of shallow foundations is typically governed by the soil bearing capacity and the total and differential settlement criteria. Based on the soil boring profiles, loose/soft soil zones at shallow depths and potential Karst/solutioned limestone were encountered in some of the borings performed.

The surficial soils throughout the project site would likely require soil improvement to achieve an adequate bearing resistance and minimize the potential for differential settlements. Shallow foundations can also be undermined by scour unless the foundations are constructed at depths that are too deep to be practical. Therefore, considering the scour effects, impacts of the soil improvement operations and associated costs, shallow foundations were not considered further for this preliminary bridge geotechnical report.

4.4.3 Steel Piles

Steel pile types include pipe and H-piles. Previous experience has shown that steel piles are generally more expensive per lineal foot than PSC piles. Steel piles may more easily penetrate dense layers to achieve a desired penetration depth. Typical sizes of pipe piles range from 18 to 24 inches in diameter. Steel pipe piles do not develop as much capacity for similar penetration depths as PSC piles. Steel H-piles often provide lower capacities than pipe piles at similar costs. Steel piles although structurally viable, are susceptible to corrosion in aggressive – high chloride content environments as is present at the Beckett Bridge site.

Steel piles are well suited to conditions with high variability in anticipated penetration depths where frequent splicing is expected. The environment of the substructure at the bridge site is extremely aggressive due to saltwater and high chloride contents. Steel piles are therefore not typically considered appropriate for a bridge replacement project in an extremely aggressive saltwater environment.

4.4.4 Drilled Shafts

Drilled cast-in-place straight-sided concrete shafts have the ability to develop high axial and lateral capacities. One drilled shaft could potentially take the place of several driven piles. The quality control of drilled shaft installation requires more engineering judgment and precaution compared with driven piles to ensure that the construction is in accordance with the specifications. This type of foundation system is often the chosen alternative for sites where competent limestone or very dense bearing strata are present at a relatively shallow depth with a sufficient thickness. Drilled shafts are also considered for sites where limiting vibrations and noise are important as is applicable to the Beckett Bridge project.

Drilled shafts are considered to be feasible for this project and therefore warrant further evaluation as the project proceeds into design. It should be noted that the potential potentiometric head pressure (potential artesian head) is reported at an elevation +0 to +10 NGVD. The potential for artesian conditions will need to be evaluated as part of the planned design of the bridge substructure. Drilled shaft cut-off elevations should ideally be set above the potential artesian head elevation to avoid construction problems with artesian flow. Benefits of a drilled shaft foundation include reduced noise and vibrations when compared to a driven pile system.

4.4.5 PSC Piles

Prestressed concrete pile foundations are a feasible foundation alternative. They are a widely used and proven foundation system in central Florida. PSC pile foundations are readily available and generally have a lower cost per ton of capacity than other pile types. Based on the environmental corrosion tests performed on recovered water samples obtained from the bridge site, the environment of the substructure at the bridge site is classified as extremely aggressive due to the chlorides content of the water. As a result it is recommended that the minimum size for PSC pile foundations be 24 inches square as referenced in the FDOT Structures Design Guidelines. Benefits of a driven pile system include typical Contractor familiarity and experience with driven pile installation.

It should be noted that the pile installation process creates both noise and induces vibrations to the surrounding environment. Vibration considerations are the primary concern with a driven pile foundation at the project site.

5.0 PRELIMINARY CONSTRUCTION CONSIDERATIONS

5.1 GENERAL

The overall site preparation and construction should be in accordance with the FDOT Standard Specifications for Road and Bridge Construction (SSRBC) and Standard Index Requirements.

5.2 TEMPORARY SIDE SLOPES

Side slopes for temporary excavations above the water table may stand near 1.5H:1V for short dry periods of time; however, it is recommended that temporary excavations that are deeper than 4 feet be cut on slopes of 2H:1V or flatter. Where restrictions will not permit slopes to be laid back as recommended above, the excavation should be shored in accordance with OSHA requirements. Furthermore, open-cut excavations exceeding 10 feet in depth should be properly dewatered and sloped 2H:1V or flatter or be benched using a bracing plan approved by a professional engineer licensed in the State of Florida. During foundation construction, excavated materials should not be stockpiled at the top of the slope within a horizontal distance equal to the excavation depth.

5.3 GROUNDWATER CONTROL

Depending upon groundwater levels at the time of construction, some form of dewatering may be required to achieve the required compaction. Due to groundwater levels during the wet season of the year, seepage may enter the bottom and sides of excavated areas. Such seepage will act to loosen soils and create difficult working conditions. Groundwater levels should be determined immediately prior to construction. Shallow groundwater should be kept below the lowest working area to facilitate proper material placement and compaction in accordance with the FDOT SSRBC.

5.4 PROTECTION OF EXISTING STRUCTURES

FDOT, SSRBC Section 455-1 should be followed for the protection of existing structures during foundation construction operations. It should be noted that some of the proposed bridge pier foundation locations will likely be situated in close proximity (distances less than 100 feet) to existing structures.

5.5 DYNAMIC LOAD TESTING FOR DRIVEN PILE FOUNDATIONS

In the event a driven pile foundation is considered for the project, we recommend that a test pile program be conducted for the proposed bridge construction including testing of at least 10% of the total piles, and that the test piles be monitored dynamically utilizing the Pile Driving Analyzer (PDA). The monitoring will provide estimates of pile capacity versus pile penetration, stresses in the pile, and other relevant parameters used to evaluate the pile driving process. CAPWAP analyses should be performed on selected conditions for evaluation of the PDA results. The results of the CAPWAP analyses will provide information for developing production pile length and driving criteria recommendations. The installation of the piles should be carried out in accordance with the FDOT SSRBC Section 455.

5.6 DRILLED SHAFT CONSTRUCTION

In the event a drilled shaft foundation is considered for the project FDOT requires that non-production test-hole shafts be installed to determine if the Contractor’s methods and equipment are sufficient for the project. It is recommended that the Contractor perform one test hole for each shaft size proposed to be completed. The test hole should be installed in accordance with the FDOT SSRBC Section 455.

To verify the integrity of drilled shafts, Cross-hole Sonic Logging tubes should be installed in all drilled shafts in accordance with the FDOT SSRBC Section 455. It is our recommendation that Cross-hole Sonic Logging testing be performed on all test-hole shafts, and selected production shafts on the project. Recommended general notes for drilled shaft construction would occur during project design.

6.0 ENVIRONMENTAL CLASSIFICATION

Corrosion tests were performed as part of one of the previous geotechnical explorations on both soil and water samples from the site. The results of the tests are included in Appendix A and summarized below:

Table 6-1
Environmental Testing

Sample ID	Sample Date	Sample Location	Sample Type	Sample Depth	pH	Chlorides ppm	Sulfates ppm	Resistivity ohm-cm
S-1	10/20/94	North Side	Soil	1.0	8.8	300	<2	1440
W-1	10/20/94	Middle of Channel	Water	1.0	7.9	14,000	7,920	41

Based on the above laboratory test results and the FDOT Structures Design Guidelines, the environmental classification of the bridge site is shown in the following table.

Table 6-2
Environmental Classification

Description	Superstructure Environmental Classification	Concrete Substructure Environmental Classification	Steel Substructure Environmental Classification
Beckett Bridge	Extremely Aggressive	Extremely Aggressive	Extremely Aggressive

APPENDIX A

Williams Earth Science Report for Crutch Bent Foundations, Dated 1994

TABLE OF CONTENTS

1.0 PROJECT INFORMATION

1.1 Introduction

As requested by Mr. Timothy Farrell, P.E. of DSA Group, Inc., in his request for services dated October 3, 1994, Williams Earth Sciences, Inc. has analyzed crutch bent foundations for Beckett Bridge Repairs. The project is located in Township 27 South, Range 15 East, Sections 11 and 12, on the Anclote River in Pinellas County, Florida. Figure 1, shown in Appendix A, illustrates the location of the project.

The Beckett Bridge is a two-lane bascule bridge 20 feet across and 358 feet long with two 2 foot wide sidewalks on each side. The approach span foundations structures are constructed of 14 inch square prestressed concrete piles. Plans provided to us by DSA Group show that the existing bridge consists of four spans on the east approach and five spans on the west approach. The bascule is approximately 40 feet long and rests on a concrete pier.

1.2 Information Provided

Williams Earth Sciences, Inc. has reviewed the Subsurface Exploration Report provided to DSA Group by Professional Services Industries, Inc., (PSI) dated January 7, 1994. Also reviewed was the Preliminary Investigation Report by David Volkert and Associates, Inc., dated February 2, 1994. A Bridge Inspection Report prepared by Kisinger, Campo and Associates Corp. was also made available. These items were sent to us in a Letter of Transmittal dated November 4, 1994, from DSA Group, Inc. along with a plan and elevation sheet of the bridge. The Letter of Transmittal requested Williams Earth Sciences, Inc. to perform capacity analyses on HP 14 x 73 and HP 14 x 89 steel piles. The letter also requested Williams Earth Sciences, Inc. to provide estimated settlements of the existing 14-inch square prestressed concrete piles. The settlement analysis however will be submitted in a separate report.

2.0 FIELD EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Our field exploration consisted of performing three Standard Penetration Test (SPT) borings. Two borings were performed near the abutments on the east and west approaches to the existing bridge and one boring was performed on the westbound lane of the bridge deck adjacent to the west side of the bascule. The test boring locations are shown on Figure 2 in Appendix A. In addition, a Report of Core Borings has been included. The test location of the SPT borings performed by PSI are also shown on Figure 2 and the Report of Core Borings.

A lane closure and Maintenance of Traffic (MOT) was necessary for the borings performed on the bridge. The bridge deck was cored with a 6-inch barrel for drilling purposes and the hole was patched using Quickrete after completion of the test boring.

While on site, the drill crew retrieved both a soil and water sample for corrosion testing at the laboratory. The water sample was taken from the middle of the Anclote River and the soil sample was taken 1 foot below the ground surface adjacent to Boring B-3.

2.2 Laboratory Testing

Grain size determination and natural moisture content tests were performed on selected samples to assist in soil classification and to provide a general indication of the engineering properties of the soils. The grain size test was performed in general accordance with ASTM D-442.

Corrosion testing was performed on one soil and one water sample to determine the environmental classification. The environmental classifications have been summarized in Table 1 and the results are reported in Appendix B.

Table 1: Summary of Environmental Classification for Soil and Water Samples

Sample ID	Sample Date	Sample Location	Sample Type	Sample Depth	pH	Chlorides ppm	Sulfates ppm	Resistivity ohm-cm
S-1	10/20/94	EAST West approach, north side	Soil	1.0	8.8	300	<2	1440
W-1	10/20/94	Middle of channel	Water	1.0	7.9	14,000	7,920	41

3.0 SUBSURFACE CONDITIONS

3.1 Subsurface Conditions

3.1.1 Abutment Borings

The major subsurface conditions encountered in our exploration are outlined below. A more detailed description of the subsurface soils is provided in the form of individual boring logs in Appendix B. Subsurface conditions may vary across the site and between boring locations.

Borings B-1 and B-3 were performed on land on the east and west sides of the bridge respectively. The soils types and strata depths encountered on these borings were fairly similar. Generally, very loose to medium dense fine sands were found from ground surface to approximately 13 feet below ground surface. The sands were slightly shelly and silty from 8 to 13 feet below the ground surface in Boring B-1. From 13 to approximately 19 feet, the soils encountered were very loose to loose, clayey to very clayey fine sands. Boring B-3 encountered firm green clay with limestone fragments from 18 to 21 feet below ground surface.

In Borings B-1 and B-3, limestone with blow counts ranging from 50=5 inches to 50=1 inch was encountered to termination depths of 75.3 feet and 81.5 feet respectively. However, at Boring B-1 a hard sandy clay with limestone pebbles was encountered from 47 to 58 feet below ground surface. At Boring B-3, the strata from 47 to 53 feet contained interpocketed silty limestone and green sandy clay. There was also a possible void at 69 to 71 feet at this boring location as evidenced by a 2 foot drop in the drill rod.

3.1.2 Bridge Borings

Boring B-2 was performed through the bridge over the Anclote River. The water depth was measured to be approximately 5 feet deep to the top of the mudline. The mudline was measured to be approximately 18 feet below the top of the bridge deck where drilling commenced. From 18 (mudline) to 25 feet below the top of the bridge deck, very loose fine sand was encountered. From 25 to 95 feet limestone was found with blow counts ranging from 50=5 inches to 50=1 inch. The strata from 68 to 75 feet, however, had blows on the order of 55 blows per foot. At 95 feet below the top of the bridge deck a very loose shelly fine sand was encountered. Below this stratum the blows increased to 50=1 inch. However, there was no recovery of the samples. The boring was terminated at 108 feet.

3.2 Groundwater and Surface Water

The groundwater depths at the time of drilling for Borings B-1 and B-3 were measured to be 5.5 and 3.5 feet below ground surface. The groundwater depth for Boring B-2 was found to be 5 feet to the mudline.

4.0 EVALUATION AND RECOMMENDATIONS

4.1 General

The evaluations that follow were performed under the assumption that steel piles HP 14 x 73 or HP 14 x 89 are to be used as crutch bents. Therefore, driven square prestressed piles, drilled shafts, steel pipe piles and shallow foundations have not been evaluated in this report.

As previously stated, the settlement predictions on the existing 14-inch square prestressed concrete piles will be provided in a separate report. Our analysis for future settlement assumes that construction of a new bridge will not influence the piles on the existing bridge. That is, the existing bridge will be demolished prior to constructing the replacement bridge. If this is not the case, a vibration and settlement monitoring program should be implemented to ensure the safety of motorists during the foundation installation. In addition, vibration monitoring might be necessary during the installation of crutch bent piles.

4.2 Analysis of Steel HP Piles

The computer program "SPT94" was used to analyze HP 14 x 73 and HP 14 x 89 steel piles as crutch bents for the existing bridge. Both steel sections were analyzed at each of the three test borings performed by Williams Earth Sciences, Inc. and as a result, six capacity curves were generated. The curves are shown in Appendix C along with the output created by the computer program. The section properties used as input for the computer runs are as follows:

	HP 14 x 73	**HP 14 x 89**
Unit Weight	490 pcf	490 pcf
Width	14.0"	14.0"
Depth	13.61"	13.83"
Area	21.4 sq. in.	26.1 sq. in.

For computer analysis purposes the elevations of the borings were assumed to be +5.5 feet for Boring B-1 and +3.5 feet for Boring B-3. Similarly, for Boring B-2 the mudline elevation was assumed to be at -5.0 feet. The elevations assumed were based on water levels at the time of drilling. The elevations shown on the capacity curves should be taken only as estimates.

As previously stated, the required design capacity of the steel piles has not been provided as of this writing, therefore, we can not make recommendations for pile lengths at this time. In addition, when selecting pile lengths and the corresponding allowable capacities from the curves, it should be recognized that the relatively hard limestone can cause buckling of the steel members during driving operations. Therefore, we recommend that a test pile program be considered using the Pile Driving Analyzer (PDA). The PDA offers driving resistance values during driving operations and can detect damage of the member. In addition, the data collected from the PDA can be used to determine driving criteria for production piles. The number of test piles will be determined based on number of crutch piles necessary to support the structure. Also, to minimize damage to the H-pile during installation, we recommend using commercially available H-pile tips with teeth. This device will improve driving alignment, reduce skidding on sloping rock and helps penetrate hard layers of soil and obstruction.

5.0 LIMITATIONS

Evaluations and recommendations presented in this report were prepared for the exclusive use of DSA Group, Inc., their clients, and consultants for the specific application to the Beckett Bridge Repairs Project. These evaluations and recommendations were prepared using generally accepted standards of geotechnical engineering practices. No other warranty is expressed or implied. Also, these evaluations and recommendations are based on design information provided and discussed earlier.

If the structural conditions vary from those stated or should the structure location be changed, the geotechnical engineer should be notified for review of the foundation recommendations.

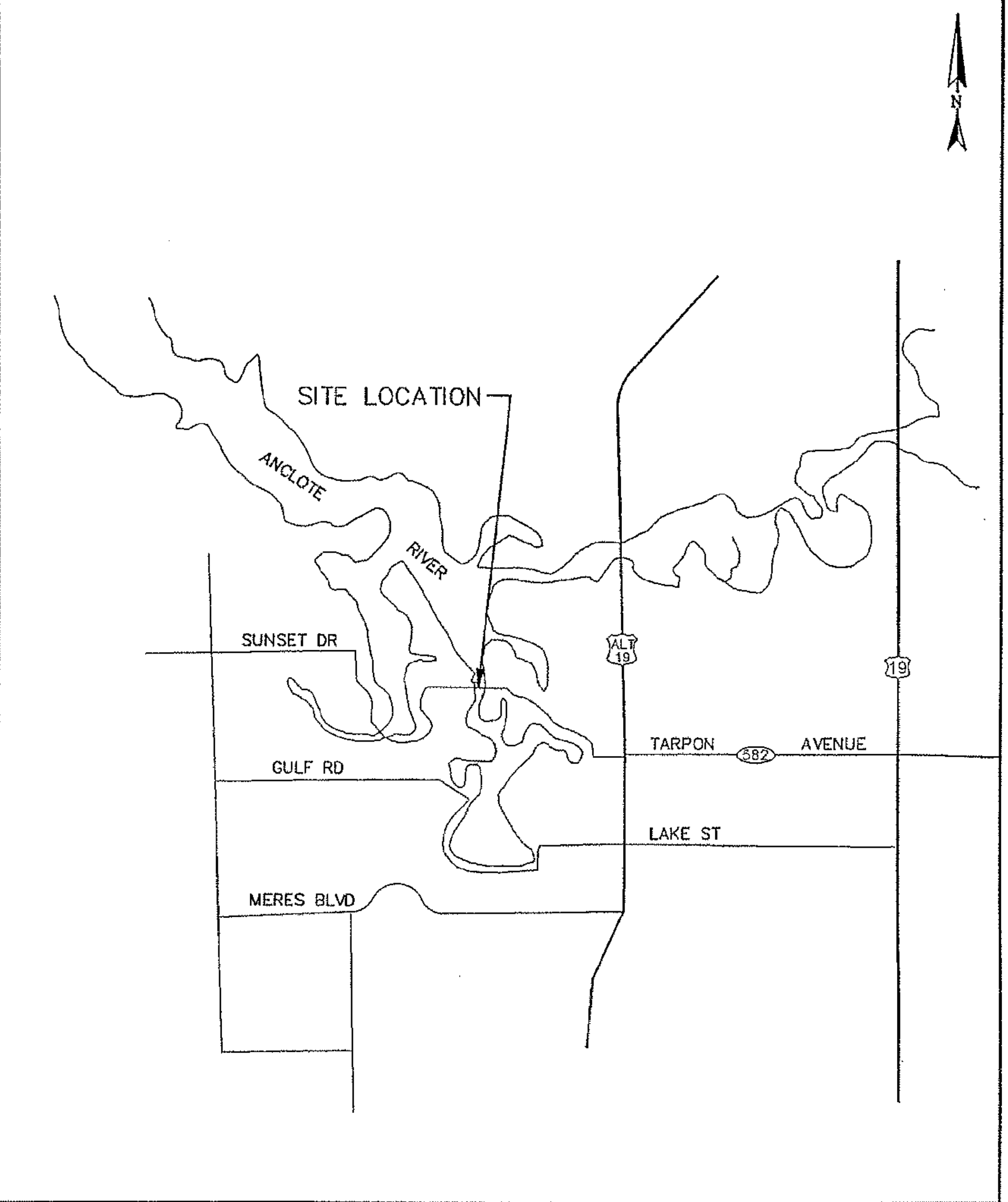
Furthermore, upon discovery of any site or subsurface condition during construction which appears to deviate from the data obtained during this geotechnical exploration as documented herein, please contact us immediately so that we may visit the site, observe the differing conditions, and thus evaluate this new information with regards to our evaluation and recommendations contained herein.

The recommendations presented previously represent design and construction techniques which we feel are both applicable and feasible for the planned construction. It is our recommendation that Williams Earth Sciences, Inc. be provided the opportunity to review the final foundation plans construction specification to evaluate whether the recommendations have been properly interpreted and implemented.

Involvement of the geotechnical engineer during construction is vitally important to ensure the project is constructed in accordance with the geotechnical report. In addition, if varying subsurface conditions are encountered, resolutions can be obtained quickly. Therefore, we recommend that Williams Earth Sciences, Inc. provide inspection services for the foundation elements of this project.

APPENDIX A

Figure 1 - Site Location Map
Figure 2 - Boring Location Map
Report of Core Borings
Soil Test Borings



WILLIAMS EARTH SCIENCES, INC.

CORPORATE OFFICE:
10800 Endeavour Way, Largo, FL 34647

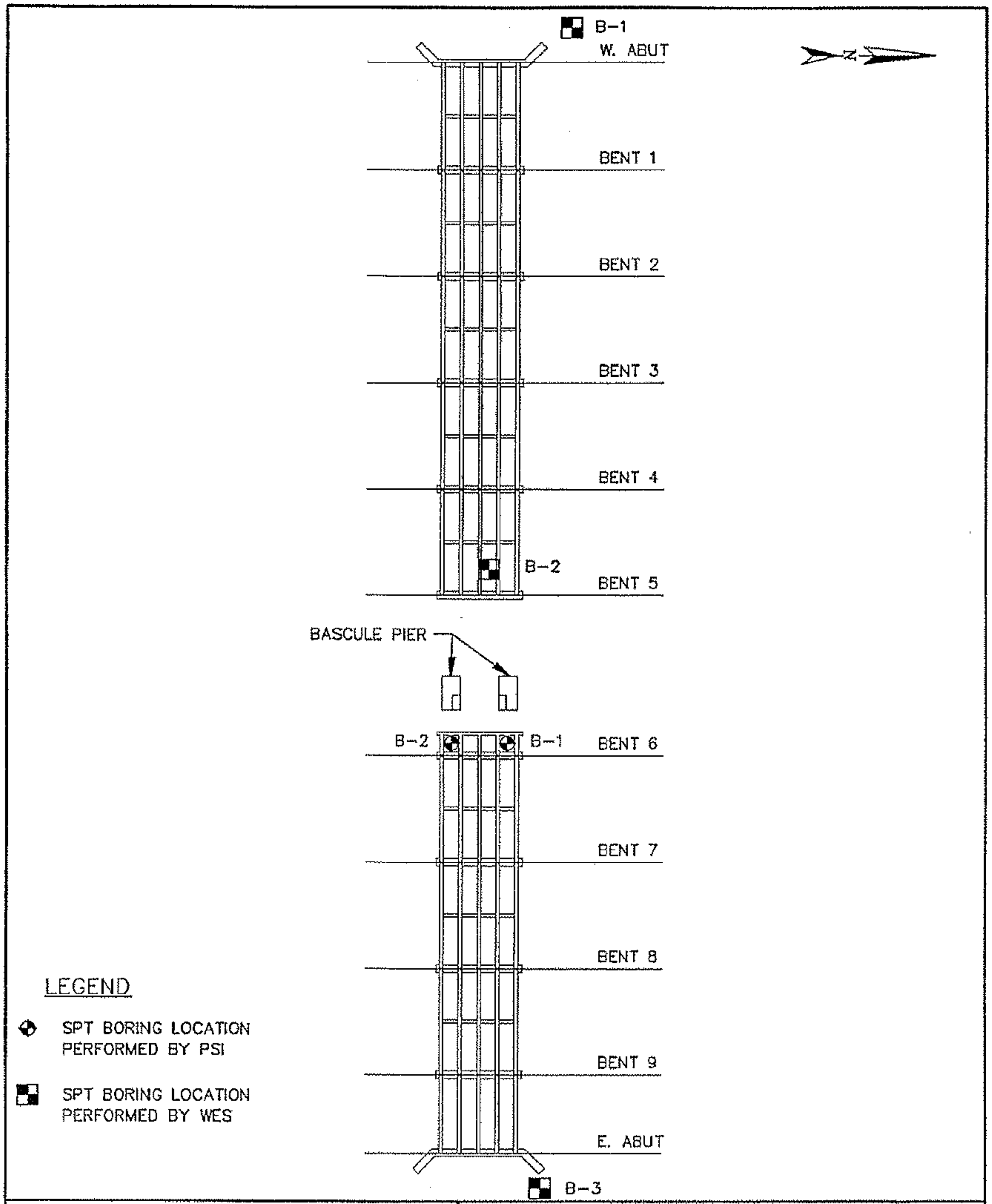
Largo: (813) 541-3444 FAX: (813) 541-1510
Jacksonville: (904) 262-8852 FAX: (904) 262-8854
Panama City: (904) 747-9419 FAX: (904) 763-2454

BECKETT BRIDGE REPLACEMENT
PINELLAS COUNTY, FLORIDA

SITE LOCATION MAP

Drawn By: TEJ
Checked By: LDS
Date: 9/11/94
Report No. C394348
Scale: N.T.S.
Figure No. 1

site



WILLIAMS EARTH SCIENCES, INC.

CORPORATE OFFICE:
10600 Endeavour Way, Largo, FL 34647

Largo:	(813) 541-3444	FAX: (813) 541-1510
Jacksonville:	(904) 262-8852	FAX: (904) 262-8864
Panama City:	(904) 747-9419	FAX: (904) 763-2454

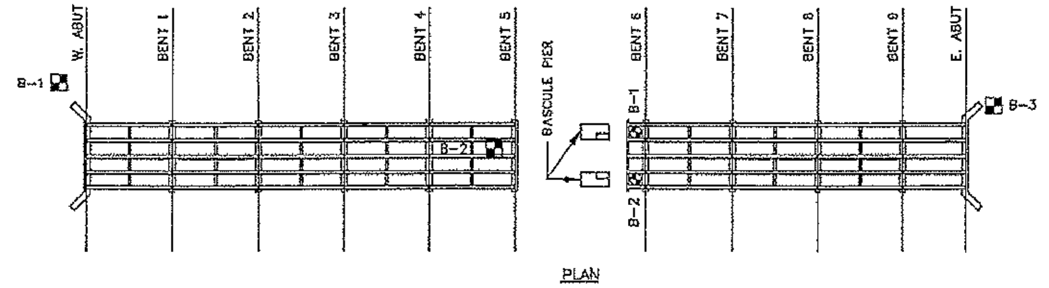
BECKETT BRIDGE REPLACEMENT
PINELLAS COUNTY, FLORIDA

BORING LOCATION MAP

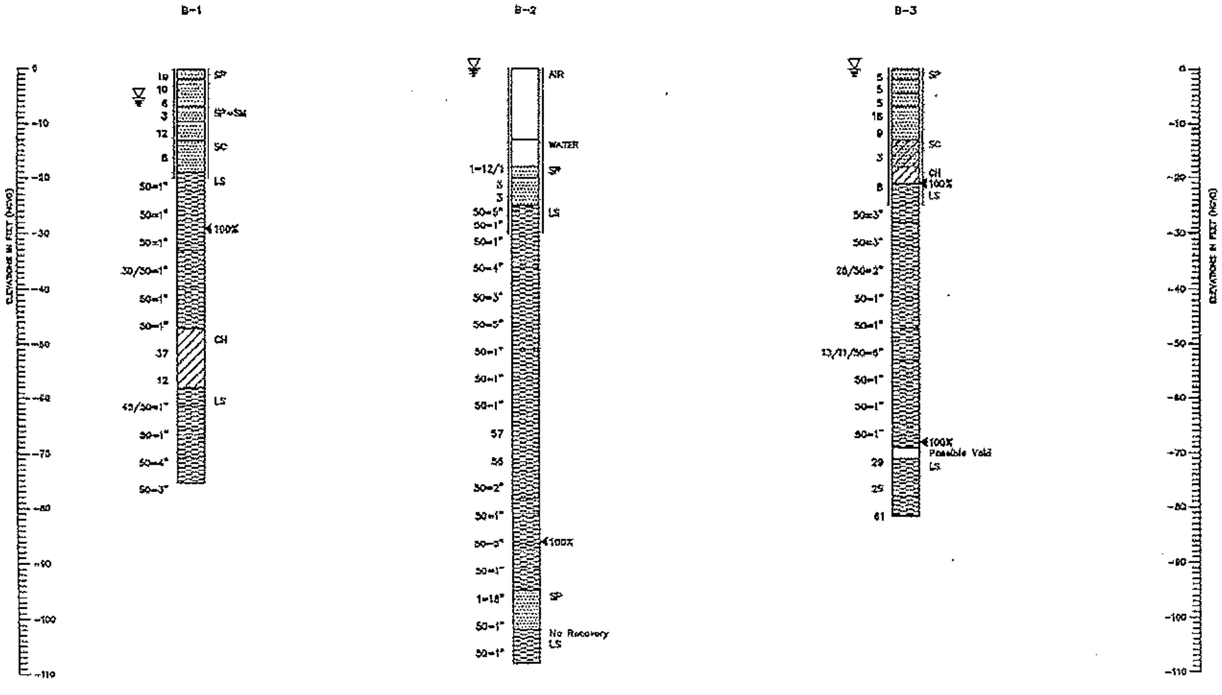
Drawn By: TEJ	Date: 9/11/94	Scale: N.T.S.
Checked By: LDS	Report No. C394348	Figure No. 2

bore

FISCAL YEAR	ROAD NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
		FLA			



PLAN



BOTTOM OF BORING @ 75.3'
DATE DRILLED 10/27/94

BOTTOM OF BORING @ 108.1'
DATE DRILLED 10/22/94

BOTTOM OF BORING @ 81.5'
DATE DRILLED 10/20/94

LEGEND

- ▤ = SP, SP-SM and SP-SC, Sands and slightly clayey sands
- ▨ = CH, Inorganic clays of low plasticity
- ▨ = SC, Clayey sands and very sandy clays
- ▤ = LS, Limestone

GENERAL NOTES

DRILL AND PENETRATION TESTING WERE PERFORMED IN ACCORDANCE WITH ASTM D 1586. NUMBER TO LEFT OF BORING INDICATES BLOWS OF 1 3/8" I.D., 2" O.D. SPLIT-SPOON FOR 12" OF PENETRATION (UNLESS OTHERWISE NOTED) WITH A 140 LB. HAMMER DROPPED 30 INCHES

THE BORING LOGS SHOWN REPRESENT SUBSURFACE CONDITIONS WITHIN THE BOREHOLE AT THE TIME OF DRILLING. NO WARRANTY AS TO THE SUBSURFACE CONDITION, STRATA DEPTH OR SOIL CONSISTENCY BETWEEN OR OUTSIDE BORING LOCATIONS IS EXPRESSED OR IMPLIED BY THIS DRAWING.

REFER TO FINAL REPORT FOR ADDITIONAL BORING INFORMATION.

CREW CHIEF: SPOON
DRILLER: PATTERSON
DRILL RIG TYPE: FAILING 250

LEGEND

- ▽ = Water Table @ end of drilling
- ▯ = Casing Used
- ▮ = Shelby Tube
- ◀100% = Percent Loss of Circulation

ENVIRONMENTAL CLASSIFICATION

SUBSTRUCTURE: CORROSIVE (EXTREMELY AGGRESSIVE)
SUBSTRUCTURE: CORROSIVE (EXTREMELY AGGRESSIVE)

Granular Materials-Relative Density	SPT (Blows/Ft.)
Very Loose	Less than 4
Loose	4 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	Greater than 50

Silts and Clays-Consistency	SPT (Blows/Ft.)
Very Soft	Less than 2
Soft	2 - 4
Firm	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	Greater than 30

REVISIONS					
Date	By	Description	Date	By	Description

	Name	Date
Drawn by	YD	11/3/94
Checked by	LDS	11/3/94
Designed by	LDS	11/8/94
Checked by	KDB	11/8/94
Approved by	K. D. BONETT	

ENGINEER OF RECORD:
WILLIAMS EARTH SCIENCES, INC.
10500 ENDEAVOUR WAY
LARGO, FLORIDA 34647

LOGO:
WILLIAMS EARTH SCIENCES, INC.

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

ROAD NO.	COUNTY	PROJECT NO.
SR936	PINELLAS	

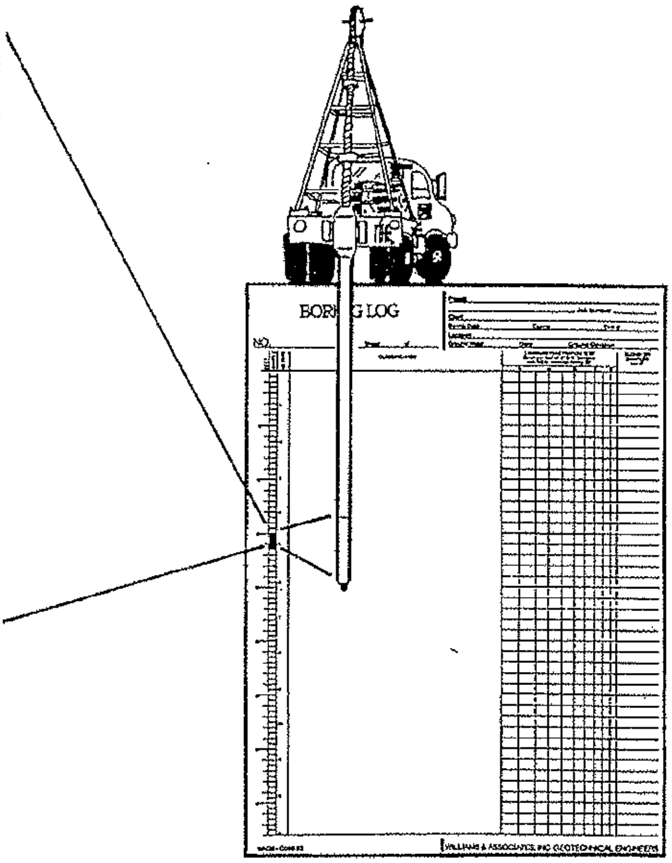
SHEET TITLE:
REPORT OF CORE BORINGS

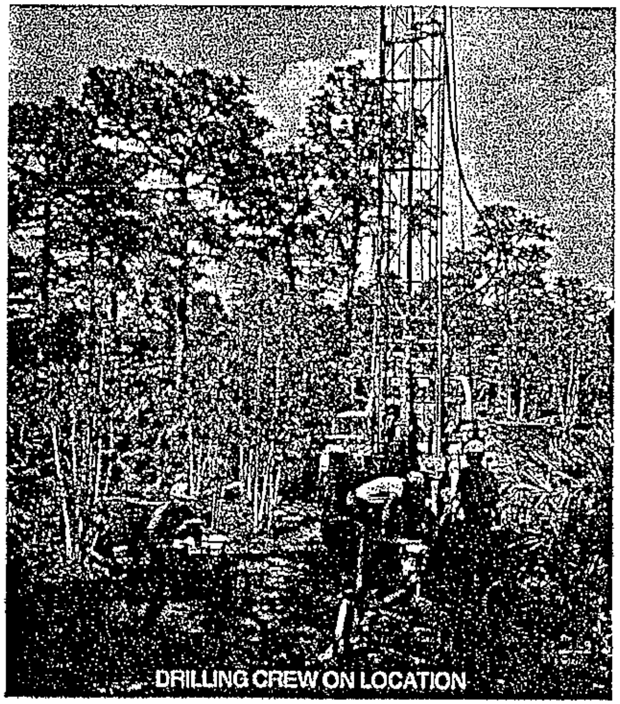
PROJECT NAME:
BECKETT BRIDGE REPLACEMENT

Drawing No.

Sheet No.

SPLIT B EL SAMP





DRILLING CREW ON LOCATION

SSIFICATION

STANDARD PENETRATION TESTING

Watching a soil test boring drill crew is a prime example of man and machine working together to explore our environment. The testing process begins with the mixing of a slurry called "drill mud". A mixture of powdered clay and water is used to flush cuttings from the borehole. The mud also stabilizes the hole walls.

For each project, there are drilling and sampling criteria. Most test borings for engineering purposes utilize an industry standard described in ASTM D1586. This procedure requires a sample be obtained using a driven tube-shaped sampler. The sampler is constructed in such a way that the barrel portion splits to allow visual examination of the soil sample. To drive the sampler, a 140-pound hammer is placed on top of the drill rods. The hammer is raised mechanically using a rope (catline) and wench (cathead), then dropped a standard 30 inches. This operation continues until either 100 blows occur or the sampler is driven 18 inches, whichever occurs first. The number of blows required to advance the sampler each 6-inch increment is recorded. The total number of blows for the last 12 inches of penetration is termed the blow count (N-value).

After the sampler is dislodged and brought to the ground surface, the soil retained in the split barrel is immediately examined and classified. A representative portion of the sample is sealed in a glass jar and labeled. All samples are returned to the laboratory where they are reviewed. Selected samples are chosen for laboratory testing. Samples are stored for a minimum of 60 days.

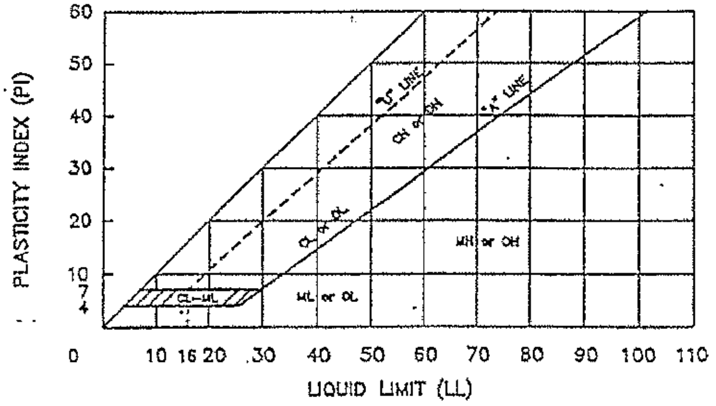
WILLIAMS
EARTH SCIENCES, Inc.

UNIFIED CLASSIFICATION SYSTEM

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE-GRAINED SOILS More than 50% retained on NO. 200 sieve *	GRAVELS 50% or more of coarse friction retained on NO. 4 sieve	CLEAN GRAVELS	GW	Well-graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS More than 50% of coarse friction passes No. 4 sieve	CLEAN SANDS	SW	Well-graded sands and gravelly sands, little or no fines
			SP	Poorly graded sands and gravelly sands, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS 50% or more passes NO. 200 sieve *	SILTS AND CLAYS Liquid limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAYS Liquid limit greater than 50%		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
			CH	Inorganic clays or high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
Highly Organic Soils			PT	Peat, muck and other highly organic soils

* Based on the material passing the 3-in. (75-mm) sieve.

PLASTICITY CHART



WILLIAMS
EARTH SCIENCES, INC.

CLASS

TEST BORING LOG

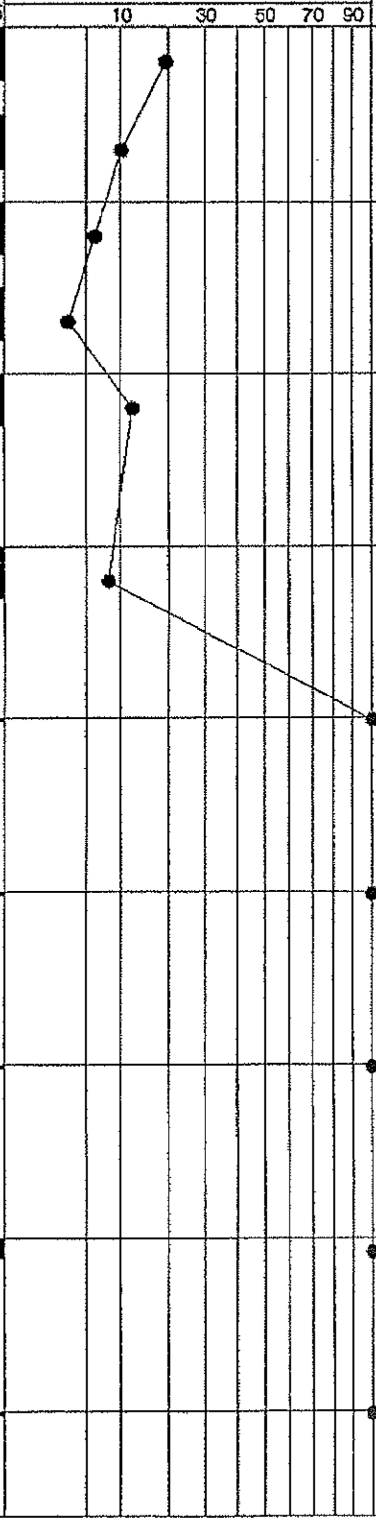
WILLIAMS
EARTH SCIENCES, INC.

Project BECKETT BRIDGE REPLACEMENT
Boring Location SEE FIELD EXPLORATION PLAN
Ground Elevation N/A
Groundwater Depth ∇ 5.5'
Length of Casing Set 20'

Boring No. B-1
Sheet 1 of 2
Job No. C394348
Boring Completed 10/27/94
Driller J. SPOON
Engineer L. SPEARS

DEPTH FEET	CLASSIFICATION	BLOWS ON SAMPLER PER 6"
	MEDIUM DENSE light brown slightly shelly fine SAND (SP)	5/7/12
	LOOSE brown fine SAND (SP)	5/5/5
5		2/2/4
	VERY LOOSE dark brown slightly shelly slightly silty fine SAND (SP-SM)	4/2/1
10	MEDIUM DENSE brown slightly silty fine SAND (SP-SM)	2/5/7
	LOOSE green very clayey fine SAND (SC)	
15		3/4/4
	Light brown sandy LIMESTONE	
20		50=1"
25		50=1"
	100% Loss of Circulation @ 29.0'	
30		50=1"
	White fossiliferous silty LIMESTONE	
35		30/50=1"
40		50=1"

STANDARD PENETRATION TEST
Blows per foot on 2" O.D. Sampler
with 140 lb. hammer falling 30"



WILLIAMS
EARTH SCIENCES, INC.

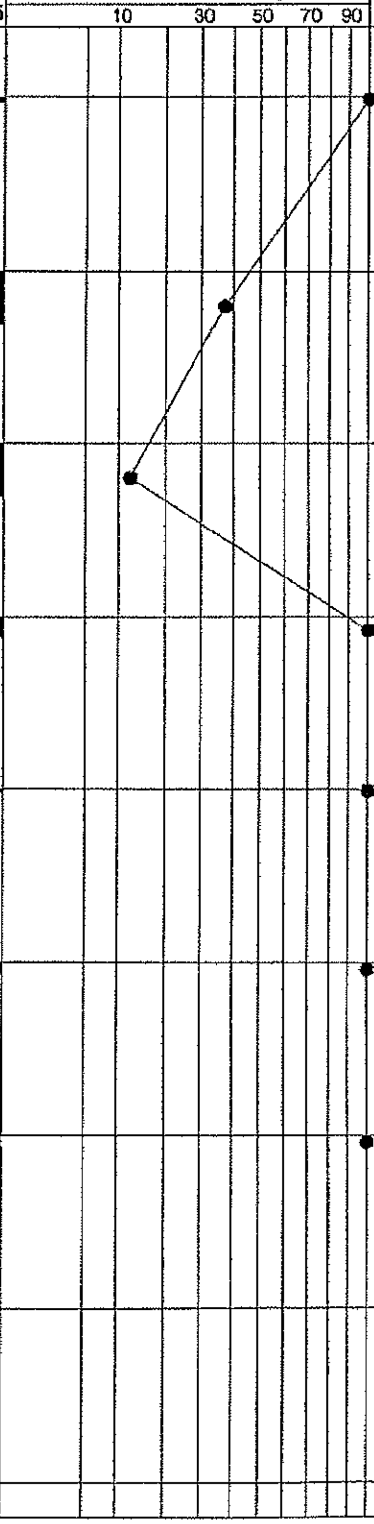
TEST BORING LOG

Project BECKETT BRIDGE REPLACEMENT

Boring No. B-1
Sheet 2 of 2
Job No. C394348

DEPTH FEET	CLASSIFICATION	BLOWS ON SAMPLER PER 6"
	White fossiliferous silty LIMESTONE	
45		50=1"
	HARD TO STIFF green sandy CLAY with limestone pebbles (CH)	
50		8/18/19
55		3/5/7
	Cream-colored silty LIMESTONE with pockets of green sandy clay	
60		45/50=1"
65		50=1"
70		50=4"
75		50=3"
	Boring Terminated @ 75.3 feet	
80		
85		

STANDARD PENETRATION TEST
Blows per foot on 2" O.D. Sampler
with 140 lb. hammer falling 30"

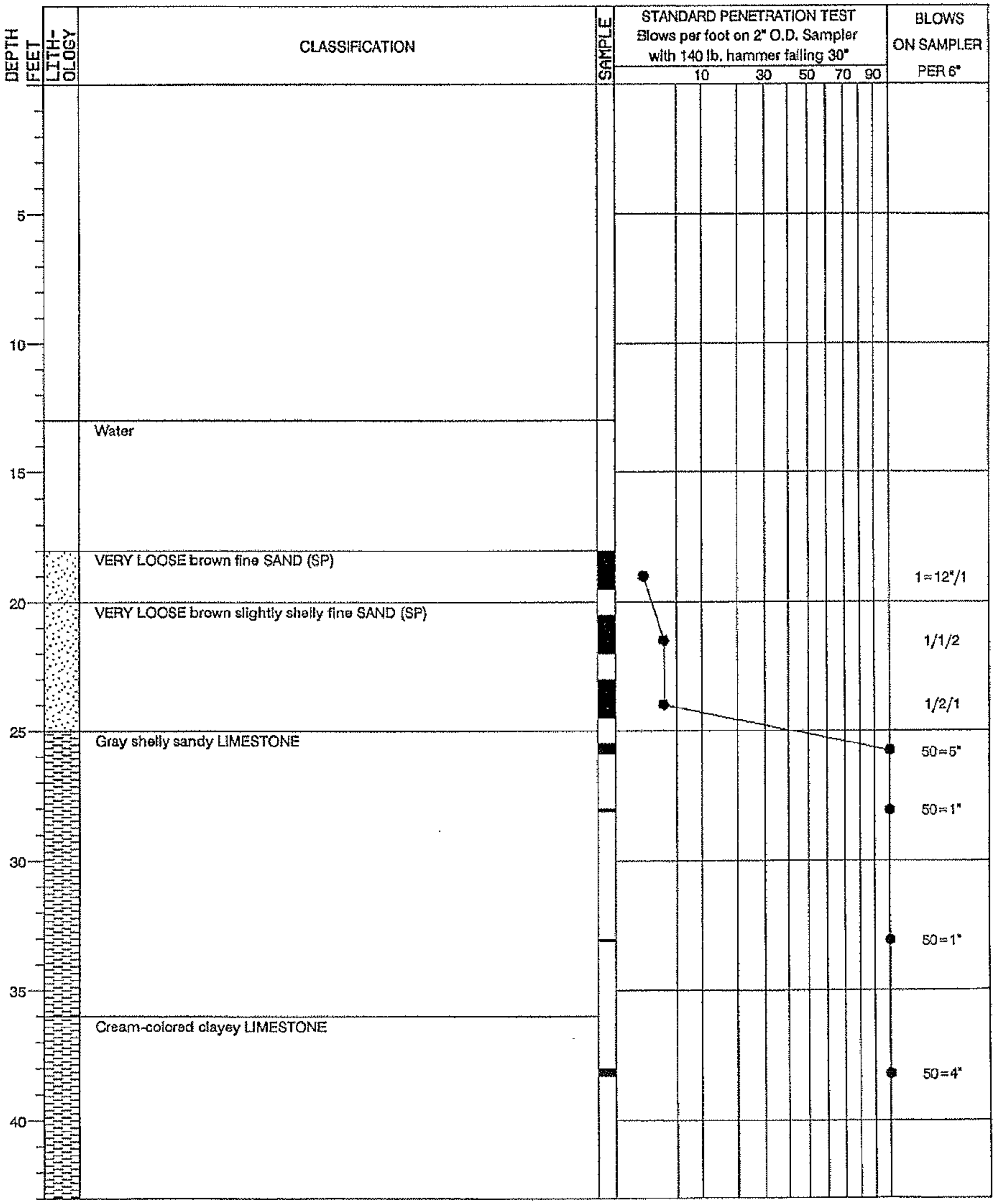


TEST BORING LOG

WILLIAMS
EARTH SCIENCES, INC.

Project BECKETT BRIDGE REPLACEMENT
Boring Location SEE FIELD EXPLORATION PLAN
Ground Elevation N/A
Water Depth 5.0'
Length of Casing Set 30'

Boring No. B-2
Sheet 1 of 3
Job No. C394348
Boring Completed 10/22/94
Driller J. SPOON
Engineer L. SPEARS



Water

VERY LOOSE brown fine SAND (SP)

VERY LOOSE brown slightly shelly fine SAND (SP)

Gray shelly sandy LIMESTONE

Cream-colored clayey LIMESTONE

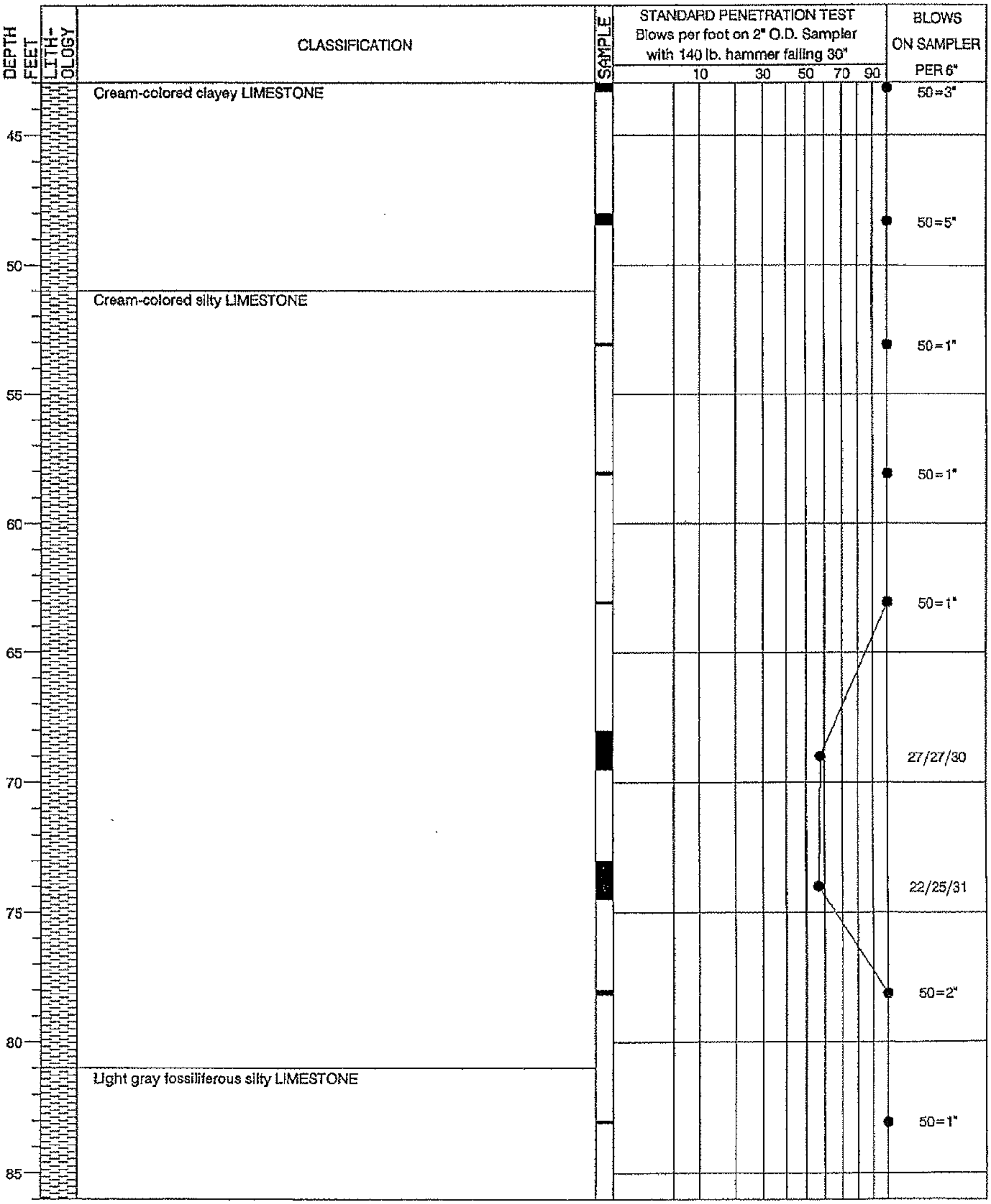
1=12"/1
1/1/2
1/2/1
50=5"
50=1"
50=1"
50=4"

TEST BORING LOG

WILLIAMS
EARTH SCIENCES, INC.

Project BECKETT BRIDGE REPLACEMENT

Boring No. B-2
Sheet 2 of 3
Job No. C394348

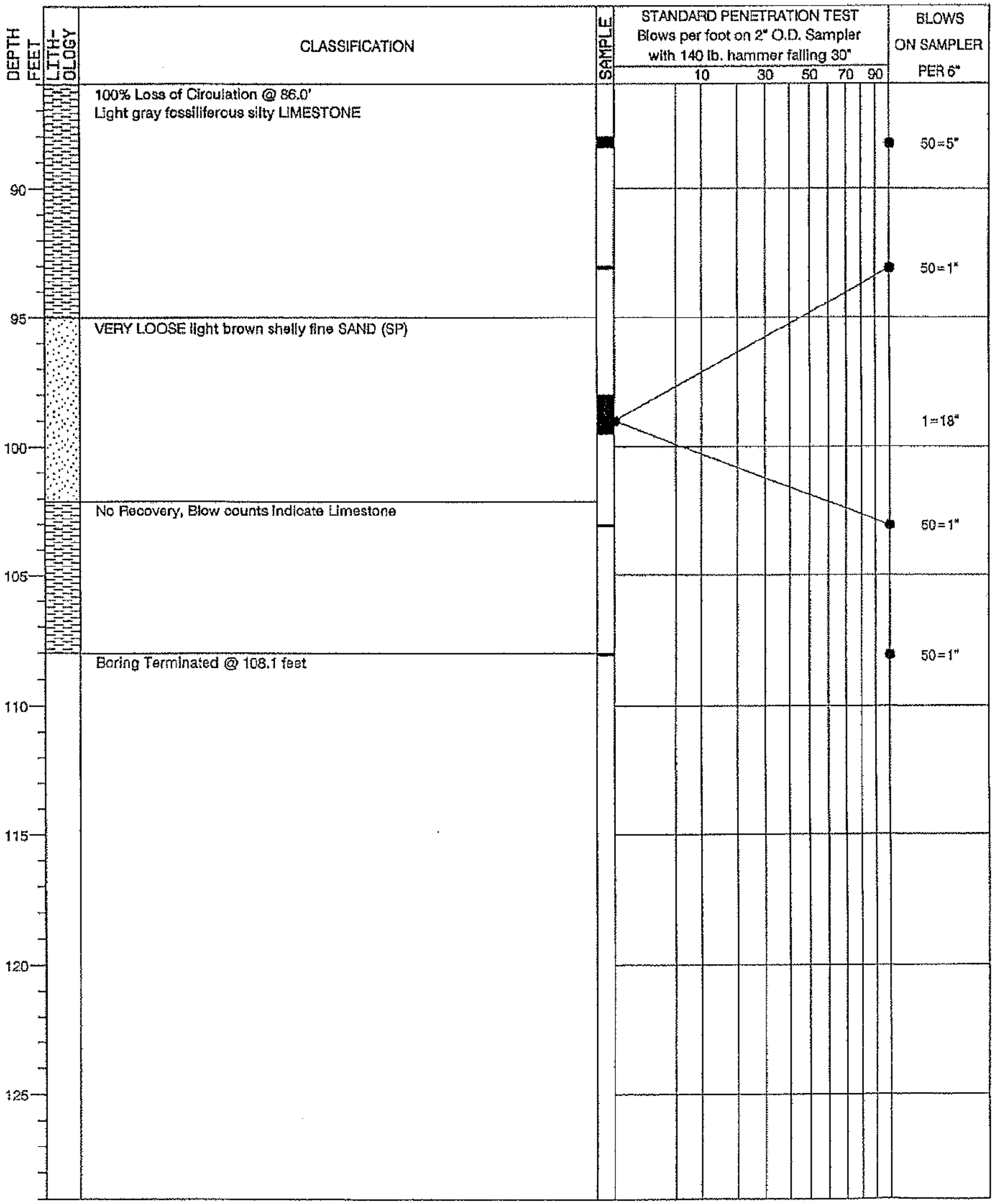


TEST BORING LOG

WILLIAMS
EARTH SCIENCES, INC.

Project BECKETT BRIDGE REPLACEMENT

Boring No. B-2
Sheet 3 of 3
Job No. C394348



100% Loss of Circulation @ 86.0'
Light gray fossiliferous silty LIMESTONE

VERY LOOSE light brown shelly fine SAND (SP)

No Recovery, Blow counts Indicate Limestone

Boring Terminated @ 108.1 feet

TEST BORING LOG

WILLIAMS
EARTH SCIENCES, INC.

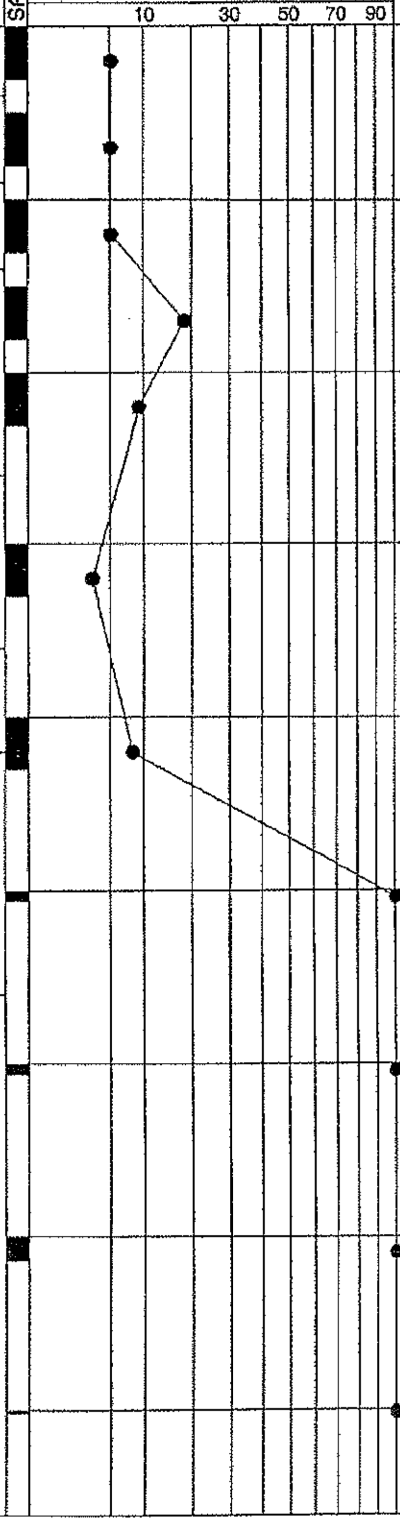
Project BECKETT BRIDGE REPLACEMENT
Boring Location 21' N OF CL
Ground Elevation N/A
Groundwater Depth ⊻ 3.5'
Length of Casing Set 25'

Boring No. B-3
Sheet 1 of 2
Job No. C394348
Boring Completed 10/20/94
Driller J. SPOON
Engineer L. SPEARS

DEPTH FEET	CLASSIFICATION	BLOWS ON SAMPLER PER 6"
	LOOSE dark brown fine SAND with asphalt fragments (SP)	6/3/2
	LOOSE light brown fine SAND (SP)	3/2/3
5	LOOSE light gray fine SAND (SP)	3/2/3
	MEDIUM DENSE TO LOOSE reddish brown fine SAND (SP)	6/6/12
10		3/3/6
	VERY LOOSE greenish gray clayey fine SAND (SC)	
15		3/1/2
	FIRM green CLAY with limestone fragments (CH)	
20	100% Loss of Circulation @ 21.0' Cream-colored silty LIMESTONE with seams of green partly indurated clay	2/5/3
25		50=3"
	Cream-colored silty LIMESTONE	
30		50=3"
35		28/50=2"
40		50=1"

SAMPLE

STANDARD PENETRATION TEST
Blows per foot on 2" O.D. Sampler
with 140 lb. hammer falling 30"

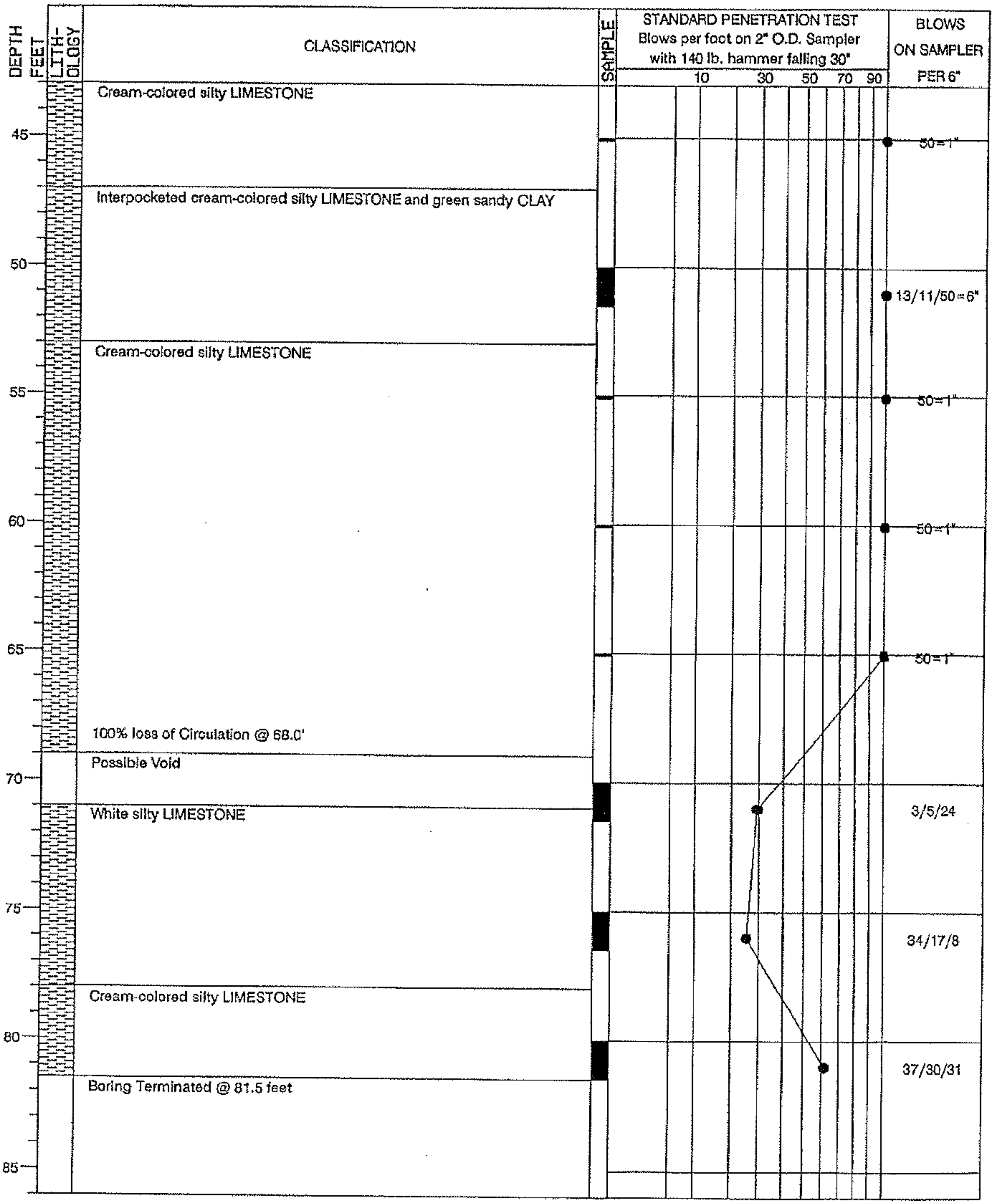


TEST BORING LOG

WILLIAMS
EARTH SCIENCES, INC.

Project BECKETT BRIDGE REPLACEMENT

Boring No. **B-3**
Sheet 2 of 2
Job No. C394348



Cream-colored silty LIMESTONE

50=1"

Interpocketed cream-colored silty LIMESTONE and green sandy CLAY

13/11/50=6"

Cream-colored silty LIMESTONE

50=1"

50=1"

50=1"

100% loss of Circulation @ 68.0'

Possible Void

White silty LIMESTONE

3/5/24

34/17/8

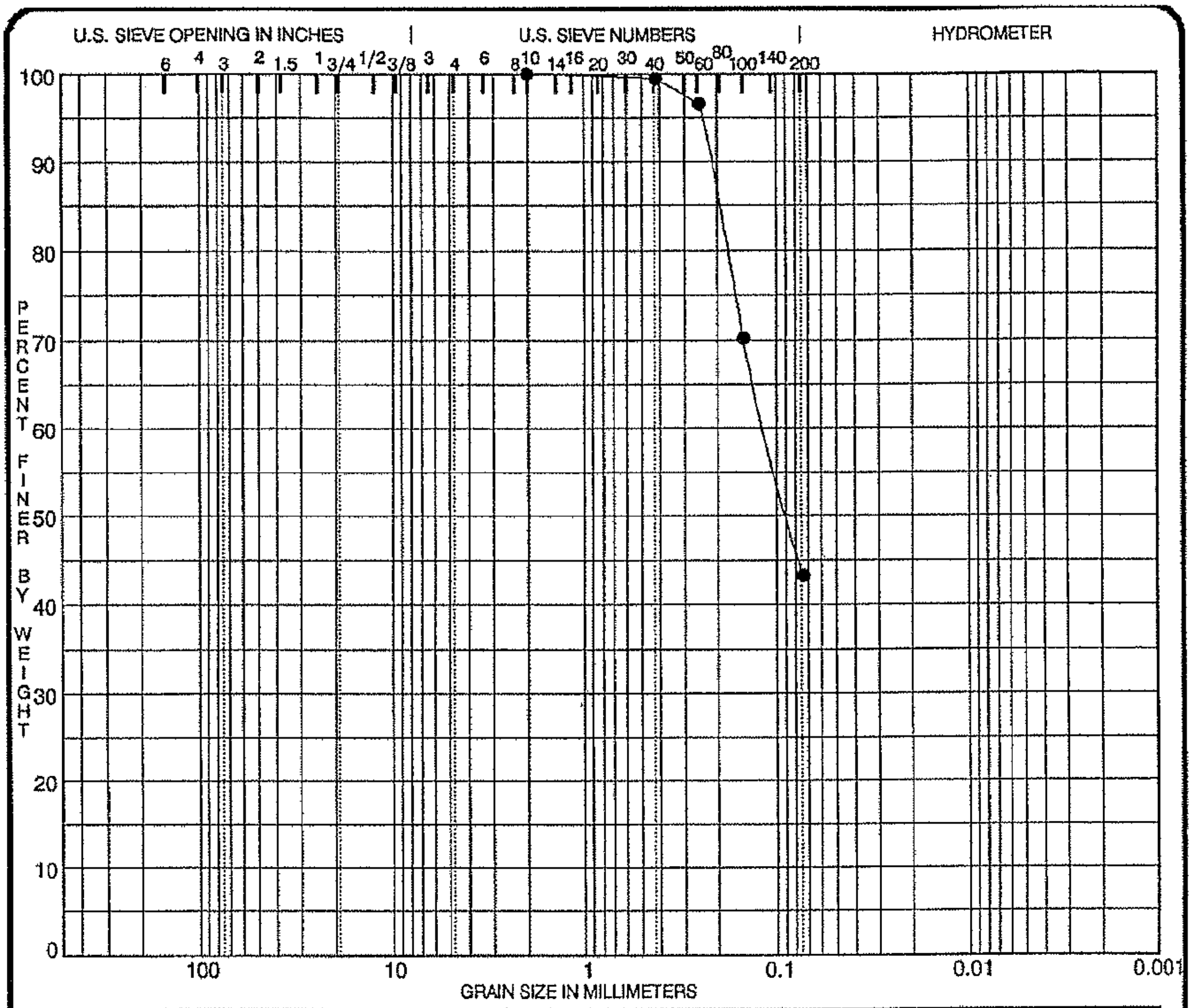
Cream-colored silty LIMESTONE

37/30/31

Boring Terminated @ 81.5 feet

APPENDIX B

Gradation Curves
Corrosion Test Results



COBBLES	GRAVEL		SAND			SILT	CLAY
	coarse	fine	coarse	medium	fine		

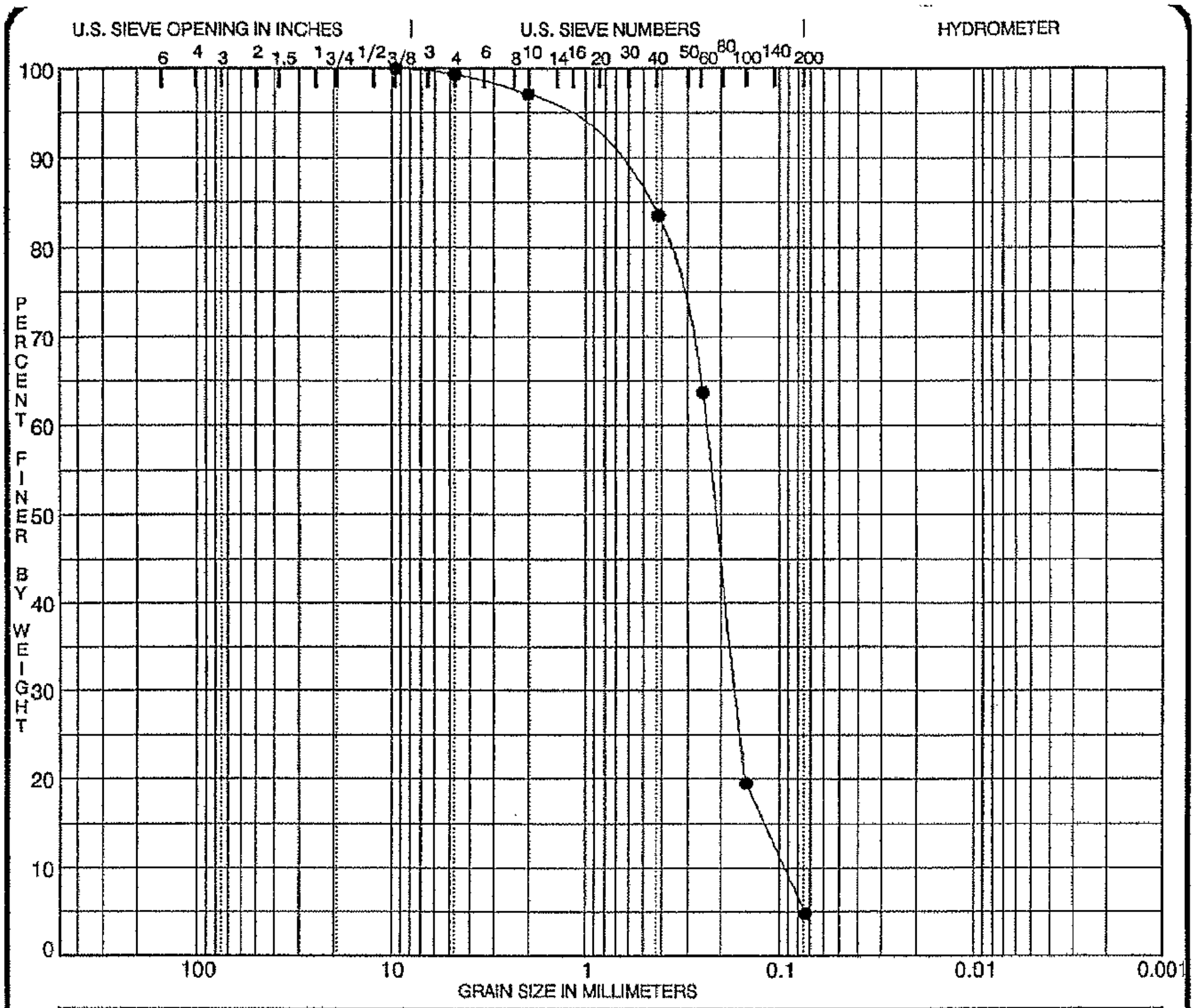
	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-1**	**GREEN VERY CLAYEY FINE SAND**	**29**					
	S-6	**(SC)**						
	15.0' - 16.5'							

Organics	D100	D50	D30	D10	%Gravel	%Sand	%Silt	%Clay
	2.00	**0.088**			**0**	**56**	**44**	

PROJECT **BECKETT BRIDGE REPLACEMENT** JOB NO. C394348

GRADATION CURVES

Williams Earth Sciences, Inc.
Largo, Florida



COBBLES	GRAVEL		SAND			SILT	CLAY
	coarse	fine	coarse	medium	fine		

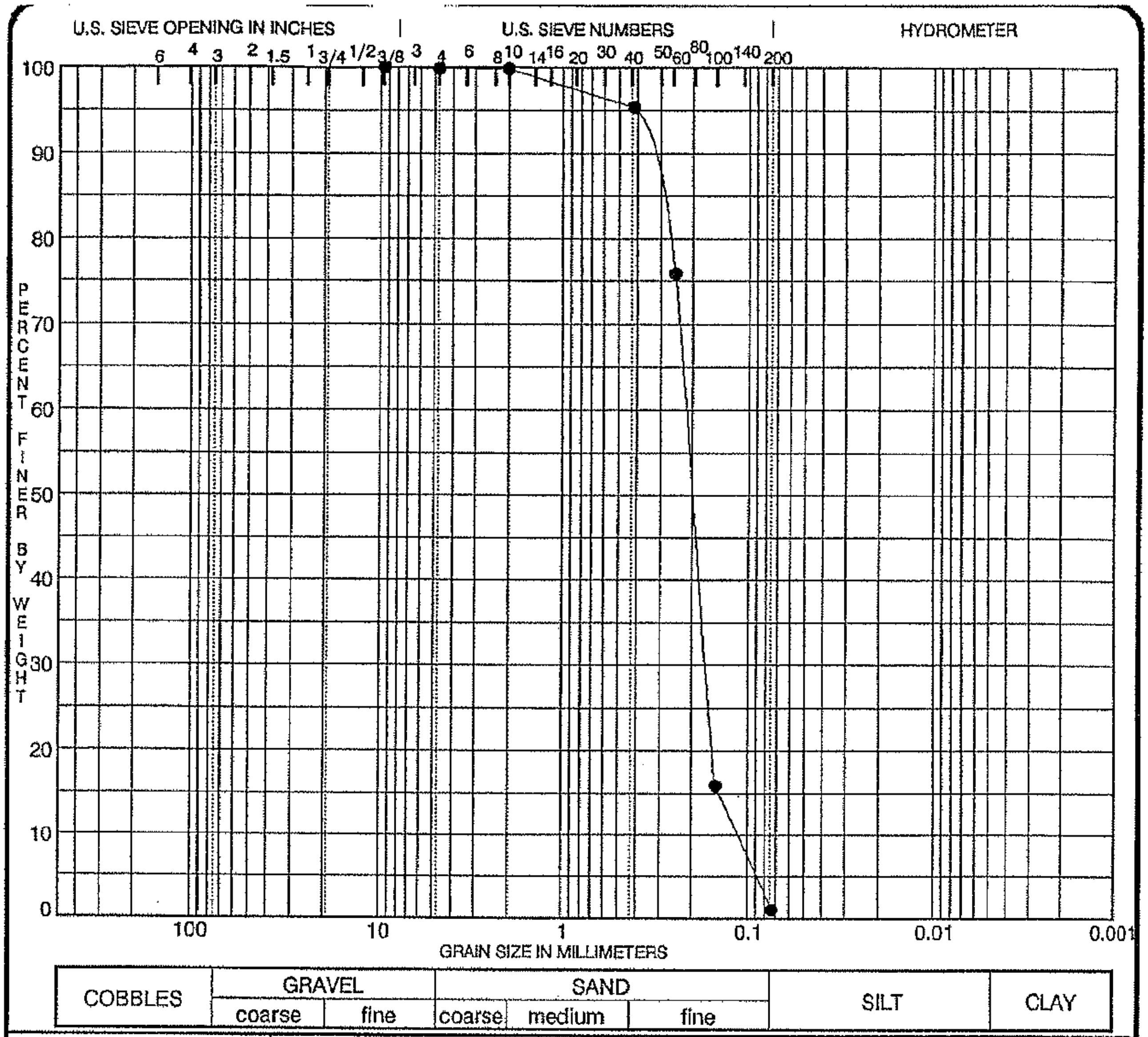
	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	**B-2**	**LIGHT BROWN SHELLY FINE SAND**	**24**					
	S-19	**(SP)**						
	98.0' - 99.5'							

Organics	D100	D50	D30	D10	%Gravel	%Sand	%Silt	%Clay
	9.51	**0.213**	**0.169**	**0.095**	**1**	**94**	**5**	

PROJECT **BECKETT BRIDGE REPLACEMENT** JOB NO. C394348

GRADATION CURVES

Williams Earth Sciences, Inc.
Largo, Florida



	Specimen Identification	Classification	MC%	LL	PL	PI	Cc	Cu
●	B-3	REDDISH BROWN FINE SAND	23					
	S-4	(SP)						
	7.5' - 9.0'							

Organics	D100	D50	D30	D10	%Gravel	%Sand	%Silt	%Clay
	9.51	0.200	0.168	0.113	0	98	1	

PROJECT BECKETT BRIDGE REPLACEMENT JOB NO. C394348

GRADATION CURVES

Williams Earth Sciences, Inc.
Largo, Florida

WILLIAMS
EARTH SCIENCES, Inc.

Corporate Office:
10600 Endeavour Way, Largo, Florida 34647
(813) 541-3444 FAX (813) 541-1510

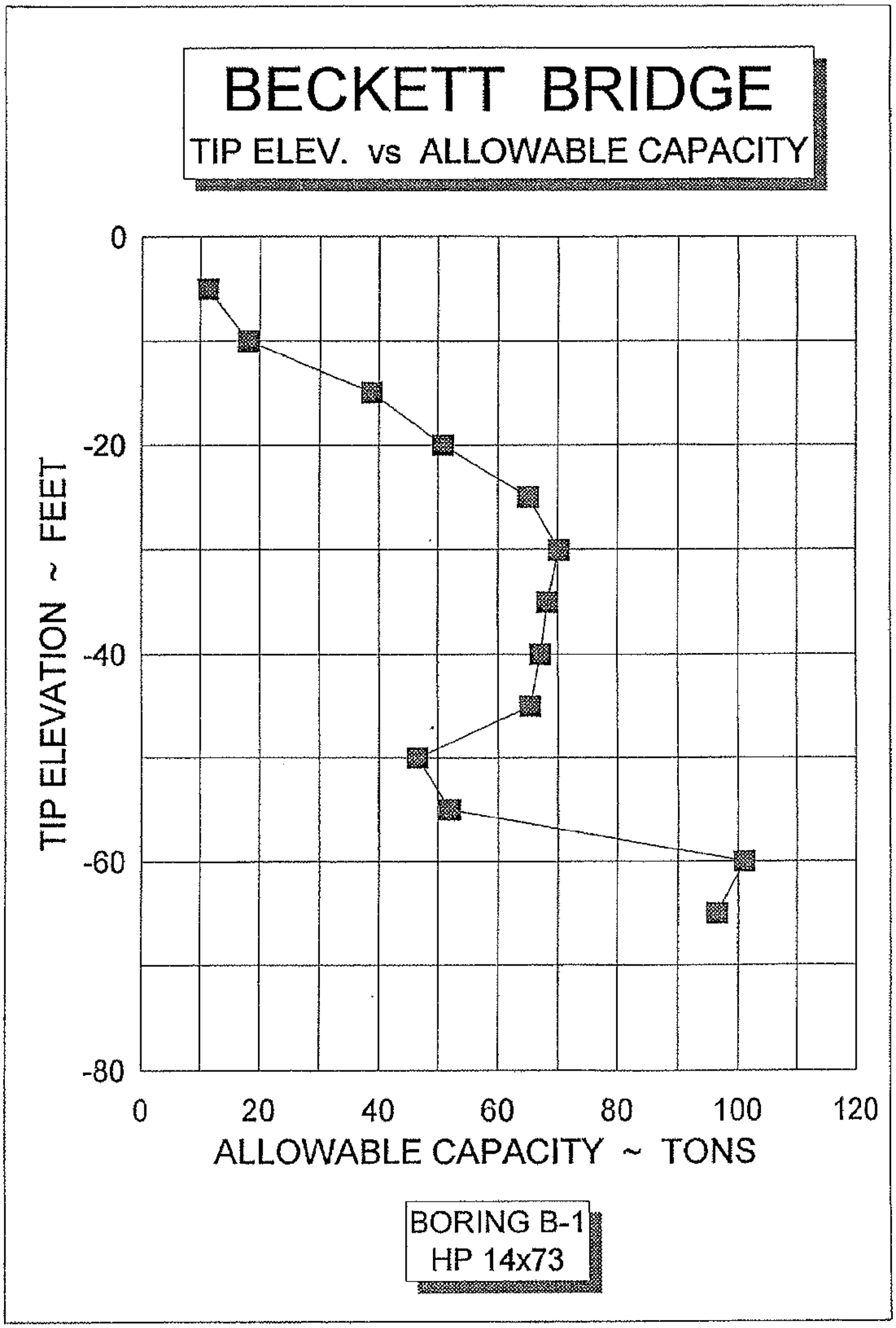
CORROSION TEST RESULTS

Job Name: Beckett Bridge Repairs
Job №: C394348
Tested by: M. Fowler

Sample ID	Sample Date	Sample Location	Sample Type	Sample Depth	pH	Chlorides ppm	Sulfates ppm	Resistivity ohm-cm
S-1	10/20/94	West approach, north side	Soil	1.0	8.8	300	<2	1440
W-1	10/20/94	Middle of channel	Water	1.0	7.9	14,000	7,920	41

APPENDIX C

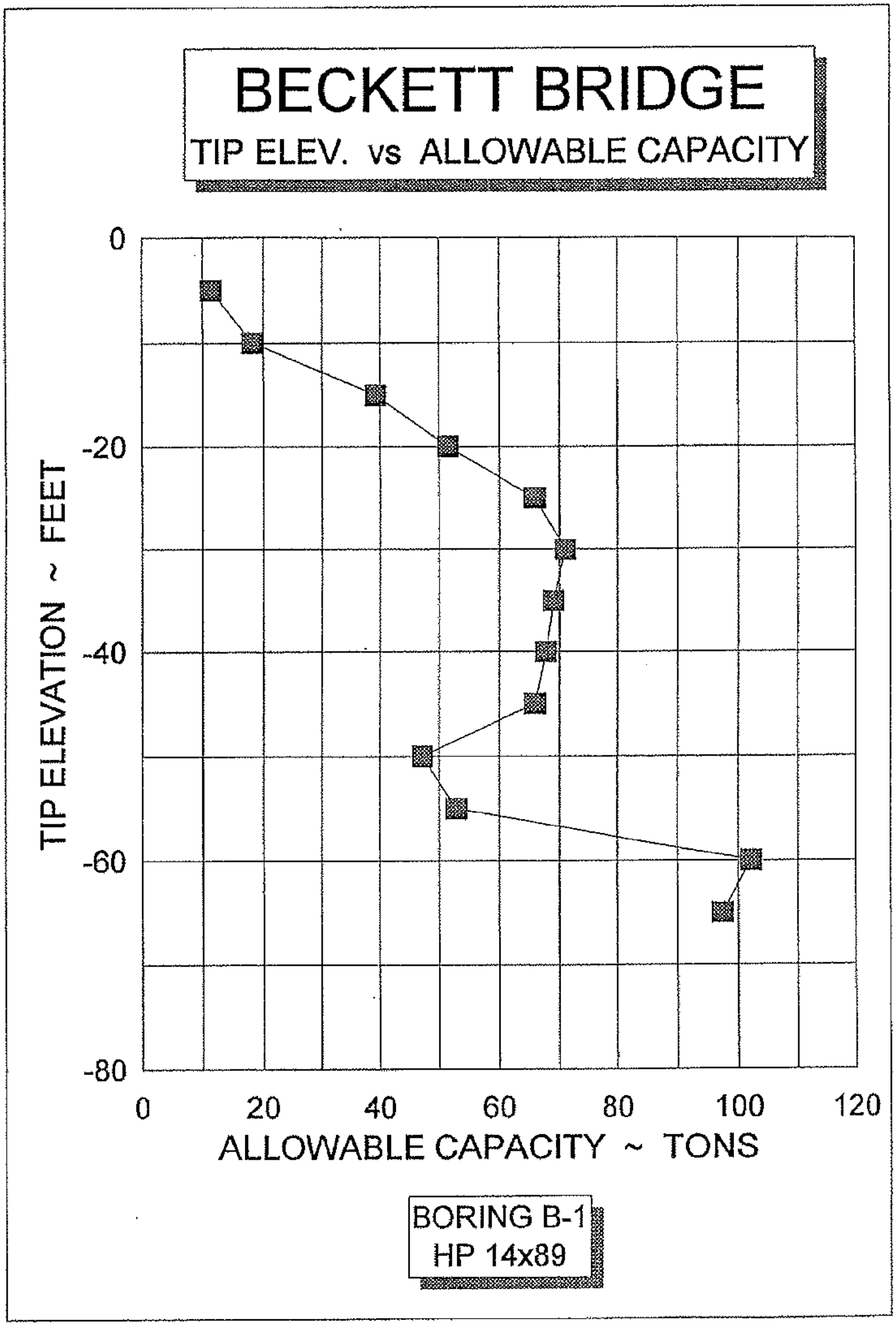
Pile Capacity Curves
"SPT94" Computer Output



BECKETT BRIDGE

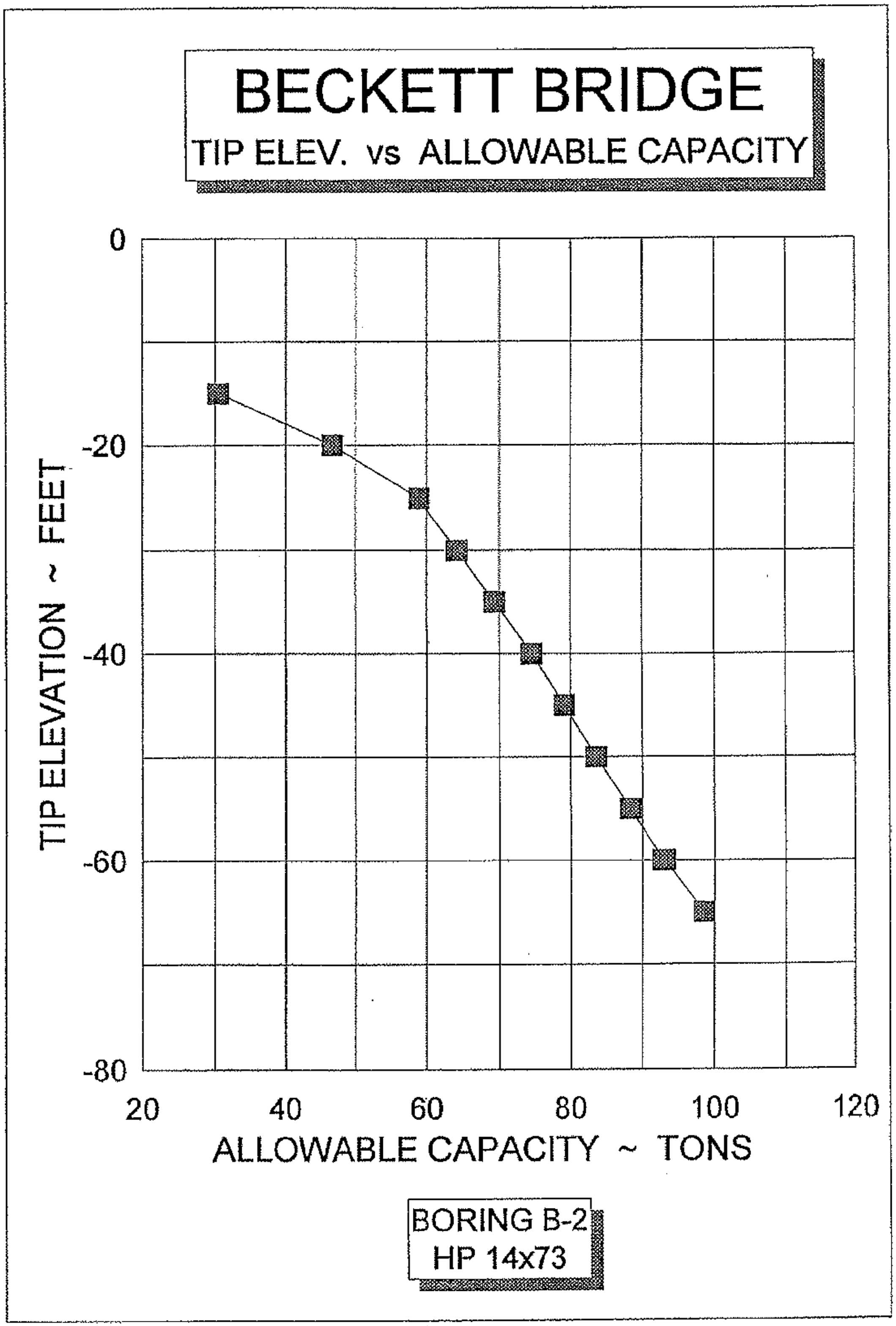
TIP ELEV. vs ALLOWABLE CAPACITY

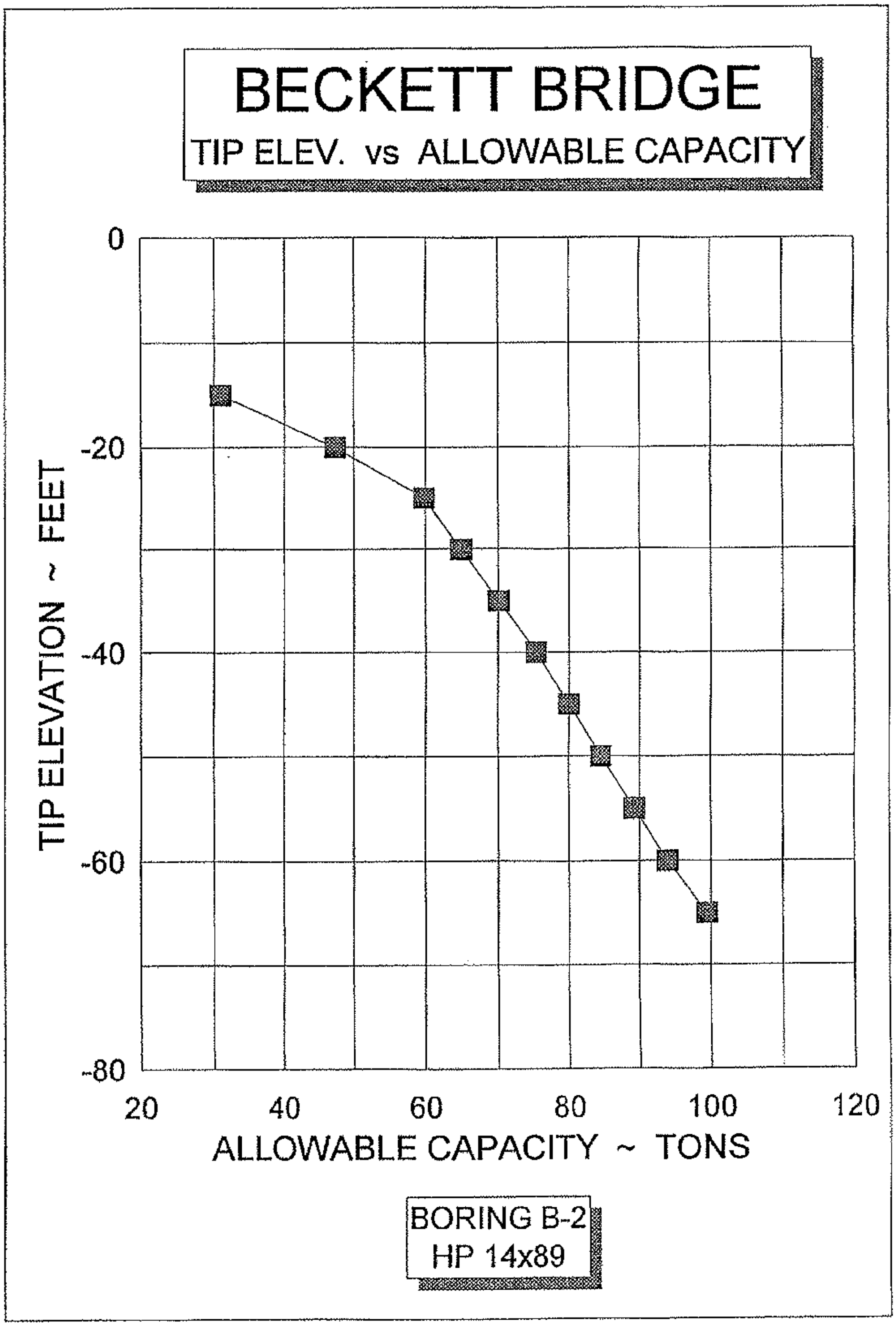
BORING B-1
HP 14x73



BECKETT BRIDGE

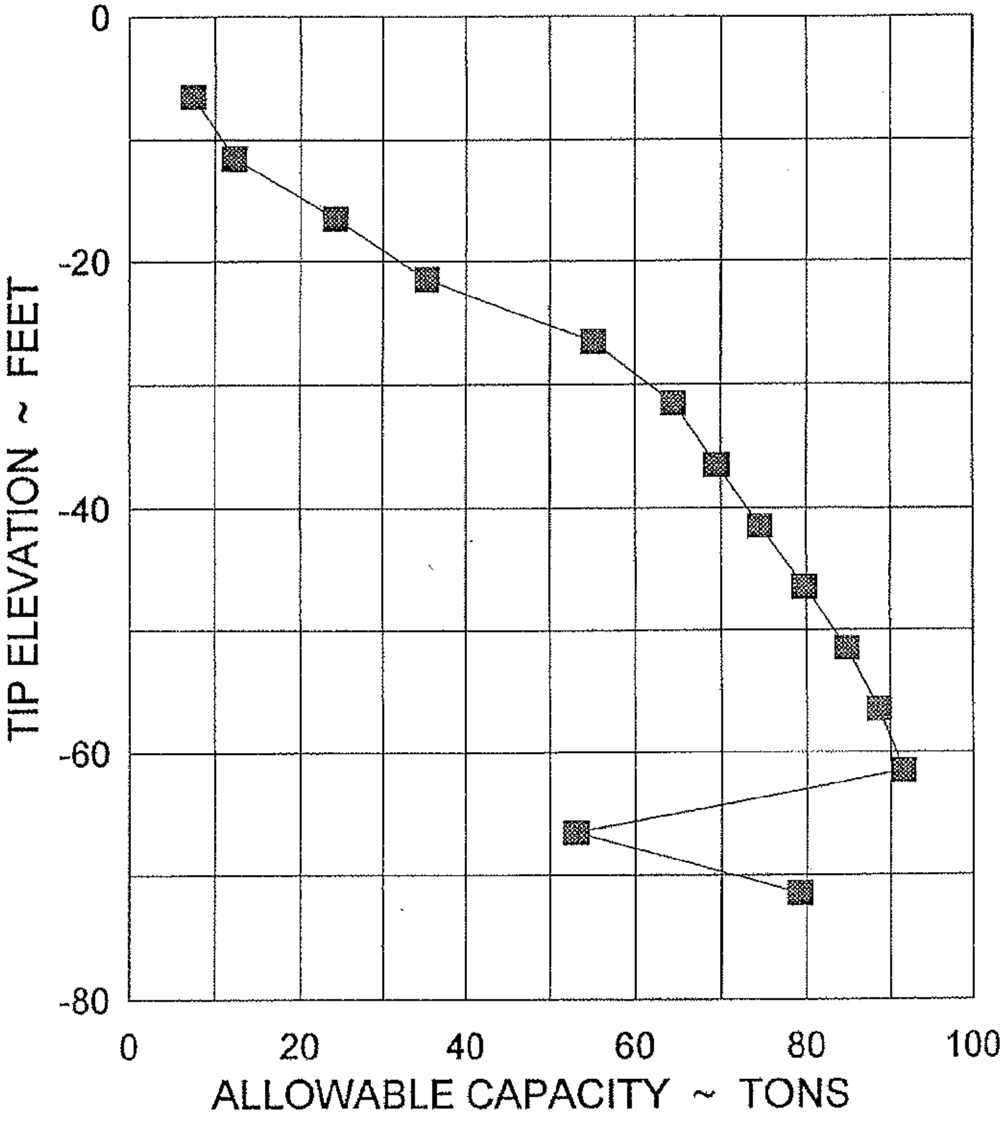
TIP ELEV. vs ALLOWABLE CAPACITY


BECKETT BRIDGE
TIP ELEV. vs ALLOWABLE CAPACITY
0
-20
-40
-60
-80
TIP ELEVATION ~ FEET
20
40
60
80
100
120
ALLOWABLE CAPACITY ~ TONS
BORING B-2
HP 14x73


BECKETT BRIDGE
TIP ELEV. vs ALLOWABLE CAPACITY
0
-20
-40
-60
-80
TIP ELEVATION ~ FEET
20
40
60
80
100
120
ALLOWABLE CAPACITY ~ TONS
BORING B-2
HP 14x89

BECKETT BRIDGE

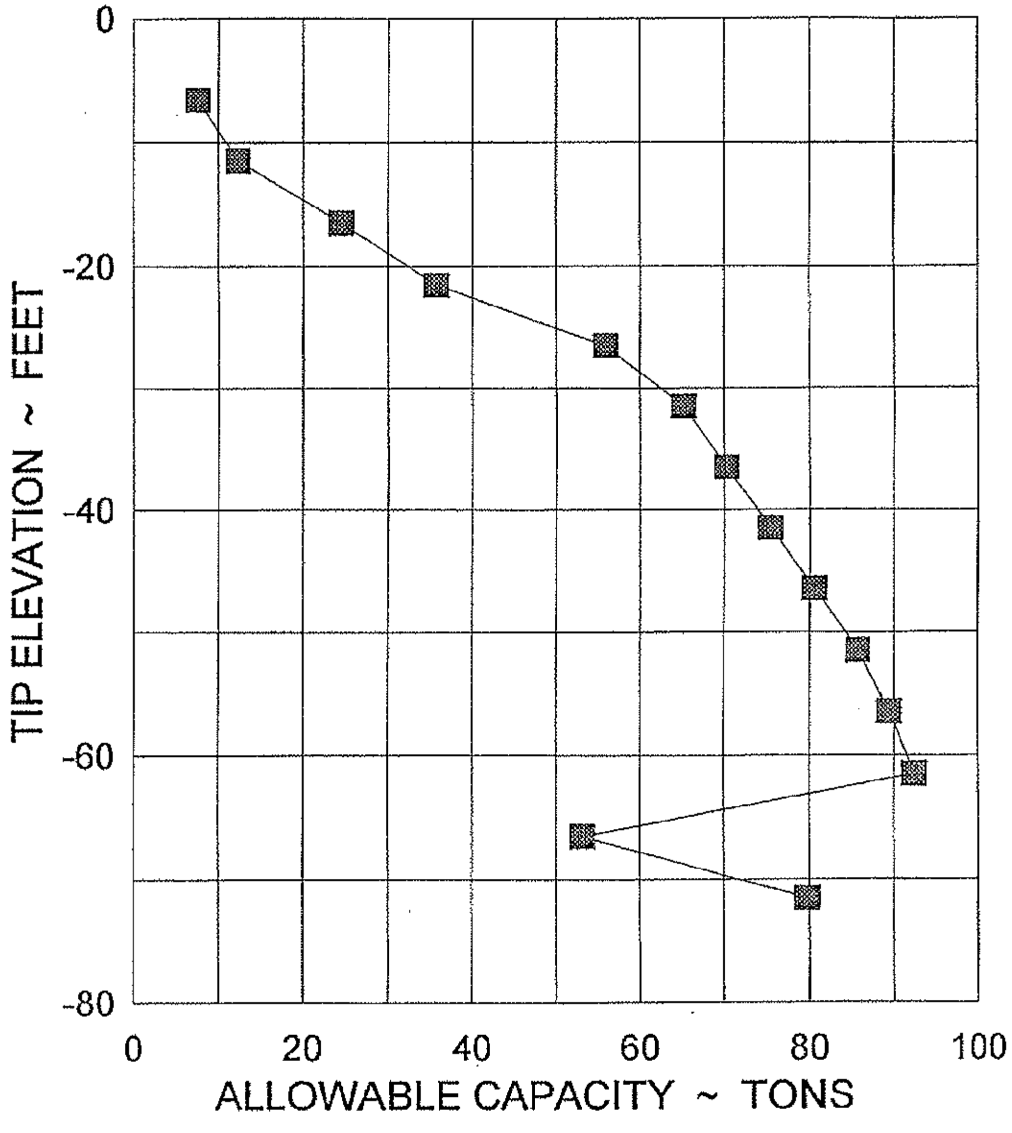
TIP ELEV. vs ALLOWABLE CAPACITY



BORING B-3
HP 14x73

BECKETT BRIDGE

TIP ELEV. vs ALLOWABLE CAPACITY



BORING B-3
HP 14x89

```
                    FLORIDA DEPARTMENT OF TRANSPORTATION
                          STRUCTURES DESIGN OFFICE
                 STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                        SPT94 - VERSION 1.0 JUNE, 1994
                        BASED ON RESEARCH BULLETIN RB-121
                "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                         AND DESIGN OF FOUNDATIONS FOR
                   BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

            NOTE    -    THIS PROGRAM IS EXPANDED FROM SPT91
                         TO INCLUDE STEEL H AND PIPE PILES


  A. GENERAL INFORMATION
  =======================

       INPUT FILE NAME                C:\SPT94\BECKETT\B173.DAT
       RUN DATE                       11/09/94
       RUN TIME                       18:15:06

       PROJECT NUMBER                 C394348
       JOB NAME                       BECKETT BRIDGE REPAIRS
       SUBMITTING ENGINEER            LDS
       BORING NO.                     B-1  HP 14x73
       DRILLING DATE                  10/27/94
       STATION NO.                    N/A
       GROUND SURFACE ELEVATION         5.00 FEET
       TYPE OF ANALYSIS                2 - DETERMINATION OF STATIC
                                           PILE BEARING CAPACITIES
                                           FOR A RANGE OF PILE LENGTHS
                                           (CAPACITY VS. TIP ELEVATION)
```

```
 oject No: C394348                 BECKETT BRIDGE REPAIRS
 ring No:  B-1  HP 14x73
```

```
B. BORING LOG
==============

                    DEPTH (FT)     ELEVATION    SPT BLOWS/FT    SOIL TYPE
   ENTRY NO.          D(I)           (FT)          N(I)           ST(I)
   ---------       ----------     ---------    ------------    ---------

       1               1.5            3.5          19.0             3
       2               4.0            1.0          10.0             3
       3               6.5           -1.5           3.0             3
       4               9.0           -4.0           6.0             3
       5              11.5           -6.5          12.0             3
       6              16.5          -11.5           8.0             2
       7              20.0          -15.0          99.0             4
       8              25.0          -20.0          99.0             4
       9              30.0          -25.0          99.0             4
      10              35.0          -30.0          99.0             4
      11              40.0          -35.0          99.0             4
      12              45.0          -40.0          99.0             4
      13              51.5          -46.5          37.0             2
      14              56.5          -51.5          12.0             2
      15              60.5          -55.5          99.0             4
      16              65.0          -60.0          99.0             4
      17              70.0          -65.0          99.0             4
      18              75.0          -70.0          99.0             4
      19              85.0          -80.0            .0             0

                           SOIL TYPE LEGEND
                           ----------------
           0  -  BOTTOM OF BORING.
           1  -  PLASTIC CLAYS
           2  -  CLAY/SILT SAND MIXTURES, SILTS & MARLS
           3  -  CLEAN SAND
           4  -  SOFT LIMESTONE, VERY SHELLY SANDS
           5  -  VOID (NO CAPACITY)
```

```
  C. PILE INFORMATION
 ====================

      TEST PILE SECTION                         ISECT =     4
                                                   {steel H-pile}
      WIDTH OF FLANGE                           WIDTH =     14.00 INCHES
      DEPTH OF SECTION                          DEPTH =     13.61 INCHES
      TRUE X-SECTIONAL AREA                     TAREA =       21.4INCH^2

  D. PILE CAPACITY VS. PENETRATION
 =================================

    TEST      PILE     WT.      ULT.   MOBILIZED  ESTIMATED  ALLOWABLE  ULTIMATE
    PILE      TIP      OF       SIDE      END      FAILURE     PILE       PILE
   LENGTH     ELEV    PILE   FRICTION BEARING    CAPACITY   CAPACITY   CAPACITY
    (FT)      (FT)   (TONS)   (TONS)    (TONS)     (TONS)     (TONS)     (TONS)
   ------     ----   ------   ----------------   ---------  ---------  --------

    10.0      -5.0     .36     4.04     18.82      22.50      11.25      41.33
    15.0     -10.0     .55     9.14     27.68      36.28      18.14      63.96
    20.0     -15.0     .73    17.33     60.81      77.41      38.71     138.22
    25.0     -20.0     .91    26.02     76.60     101.71      50.85     178.30
    30.0     -25.0    1.09    35.84     95.27     130.02      65.01     225.29
    35.0     -30.0    1.27    46.34     95.27     140.33      70.17     235.60
    40.0     -35.0    1.46    56.83     81.15     136.52      68.26     217.67
    45.0     -40.0    1.64    69.96     65.95     134.27      67.13     200.21
    50.0     -45.0    1.82    77.20     55.38     130.76      65.38     186.14
    55.0     -50.0    2.00    90.20      4.75      92.95      46.47     102.44
    60.0     -55.0    2.18    99.02      7.08     103.91      51.96     118.08
    65.0     -60.0    2.37   109.37     95.27     202.28     101.14     297.55
    70.0     -65.0    2.55   119.87     75.82     193.14      96.57     268.95

      *** ERROR *** PILE TIP EXCEEDS BORING LOG FOR LENGTH =  75.00 FT

      NOTES
     -------

      1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
         CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
         FOR SOIL TYPE 1 AND 2.

      2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
         AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.

      3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.
```

```
 4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
                        FOR TIP IN SOIL TYPE 3 OR 4,
                  = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                        FOR TIP IN SOIL TYPE 1 OR 2.

 5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
    NEGATIVE.

 PROBLEM COMPLETED                        ANALYSIS NO.  1
```

```
                 FLORIDA DEPARTMENT OF TRANSPORTATION
                       STRUCTURES DESIGN OFFICE
              STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                    SPT94 - VERSION 1.0 JUNE, 1994
                   BASED ON RESEARCH BULLETIN RB-121
             "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                     AND DESIGN OF FOUNDATIONS FOR
               BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

          NOTE   -    THIS PROGRAM IS EXPANDED FROM SPT91
                      TO INCLUDE STEEL H AND PIPE PILES


  A. GENERAL INFORMATION
  ======================

       INPUT FILE NAME                C:\SPT94\BECKETT\B189.DAT
       RUN DATE                       11/09/94
       RUN TIME                       18:16:16

       PROJECT NUMBER                 C394348
       JOB NAME                       BECKETT BRIDGE REPAIRS
       SUBMITTING ENGINEER            LDS
       BORING NO.                     B-1  HP 14x89
       DRILLING DATE                  10/27/94
       STATION NO.                    N/A
       GROUND SURFACE ELEVATION         5.00 FEET
       TYPE OF ANALYSIS                2 - DETERMINATION OF STATIC
                                           PILE BEARING CAPACITIES
                                           FOR A RANGE OF PILE LENGTHS
                                           (CAPACITY VS. TIP ELEVATION)
```

```
B. BORING LOG
==============

                   DEPTH (FT)   ELEVATION   SPT BLOWS/FT   SOIL TYPE
   ENTRY NO.         D(I)         (FT)         N(I)          ST(I)
   ---------      ----------   ---------   ------------   ---------

       1              1.5          3.5          19.0           3
       2              4.0          1.0          10.0           3
       3              6.5         -1.5           3.0           3
       4              9.0         -4.0           6.0           3
       5             11.5         -6.5          12.0           3
       6             16.5        -11.5           8.0           2
       7             20.0        -15.0          99.0           4
       8             25.0        -20.0          99.0           4
       9             30.0        -25.0          99.0           4
      10             35.0        -30.0          99.0           4
      11             40.0        -35.0          99.0           4
      12             45.0        -40.0          99.0           4
      13             51.5        -46.5          37.0           2
      14             56.5        -51.5          12.0           2
      15             60.5        -55.5          99.0           4
      16             65.0        -60.0          99.0           4
      17             70.0        -65.0          99.0           4
      18             75.0        -70.0          99.0           4
      19             85.0        -80.0            .0           0

                          SOIL TYPE LEGEND
                         -----------------
          0  -  BOTTOM OF BORING
          1  -  PLASTIC CLAYS
          2  -  CLAY/SILT SAND MIXTURES, SILTS & MARLS
          3  -  CLEAN SAND
          4  -  SOFT LIMESTONE, VERY SHELLY SANDS
          5  -  VOID (NO CAPACITY)
```

```
+-----------------------------------------------------------------------------+
| STATIC PILE BEARING CAPACITY ANALYSIS     - SPT94                  Page    3 |
 -----------------------------------------------------------------------------+
  oject No: C394348                 BECKETT BRIDGE REPAIRS                    |
|  ring No:  B-1  HP 14x89                                                    |
+-----------------------------------------------------------------------------+


  C. PILE INFORMATION
 =====================

      TEST PILE SECTION                              ISECT =     4
                                                         {steel H-pile}
      WIDTH OF FLANGE                                WIDTH =     14.00 INCHES
      DEPTH OF SECTION                               DEPTH =     13.83 INCHES
      TRUE X-SECTIONAL AREA                          TAREA =       26.1INCH^2


  D. PILE CAPACITY VS. PENETRATION
 ==================================

     TEST      PILE     WT.      ULT.   MOBILIZED   ESTIMATED   ALLOWABLE   ULTIMATE
     PILE      TIP      OF       SIDE      END      FAILURE       PILE        PILE
    LENGTH     ELEV    PILE   FRICTION  BEARING    CAPACITY    CAPACITY    CAPACITY
     (FT)      (FT)   (TONS)    (TONS)    (TONS)     (TONS)      (TONS)      (TONS)
    ------     ----   ------  ----------------   ---------   ---------   --------

     10.0      -5.0     .44      4.07     19.13       22.76       11.38       41.89
     15.0     -10.0     .67      9.21     28.13       36.68       18.34       64.80
     20.0     -15.0     .89     17.46     61.80       78.37       39.19      140.17
     25.0     -20.0    1.11     26.23     77.83      102.95       51.48      180.78
     30.0     -25.0    1.33     36.13     96.81      131.61       65.80      228.42
     35.0     -30.0    1.55     46.71     96.81      141.96       70.98      238.77
     40.0     -35.0    1.78     57.28     82.46      137.97       68.98      220.43
     45.0     -40.0    2.00     70.52     67.01      135.53       67.77      202.54
     50.0     -45.0    2.22     77.81     56.27      131.87       65.93      188.14
     55.0     -50.0    2.44     90.92      5.79       94.27       47.13      105.85
     60.0     -55.0    2.66     99.80      8.64      105.78       52.89      123.06
     65.0     -60.0    2.89    110.24     96.81      204.17      102.08      300.98
     70.0     -65.0    3.11    120.82     77.04      194.76       97.38      271.80

      *** ERROR *** PILE TIP EXCEEDS BORING LOG FOR LENGTH =   75.00 FT

      NOTES
     -------

      1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
         CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
         FOR SOIL TYPE 1 AND 2.

      2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
         AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.

      3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.
```

```
  4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
                          FOR TIP IN SOIL TYPE 3 OR 4,
                   = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                          FOR TIP IN SOIL TYPE 1 OR 2.

  5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
     NEGATIVE.

  PROBLEM COMPLETED                         ANALYSIS NO.   1
```

```
                    FLORIDA DEPARTMENT OF TRANSPORTATION
                          STRUCTURES DESIGN OFFICE
               STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                      SPT94 - VERSION 1.0 JUNE, 1994
                      BASED ON RESEARCH BULLETIN RB-121
              "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                       AND DESIGN OF FOUNDATIONS FOR
                 BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

          NOTE    -    THIS PROGRAM IS EXPANDED FROM SPT91
                       TO INCLUDE STEEL H AND PIPE PILES


  A. GENERAL INFORMATION
  ========================

      INPUT FILE NAME                C:\SPT94\BECKETT\B273.DAT
      RUN DATE                       11/09/94
      RUN TIME                       18:16:54

      PROJECT NUMBER                 C394348
      JOB NAME                       BECKETT BRIDGE REPAIRS
      SUBMITTING ENGINEER            LDS
      BORING NO.                     B-2  HP 14x73
      DRILLING DATE                  10/22/94
      STATION NO.                    N/A
      GROUND SURFACE ELEVATION        -5.00 FEET
      TYPE OF ANALYSIS                2 - DETERMINATION OF STATIC
                                          PILE BEARING CAPACITIES
                                          FOR A RANGE OF PILE LENGTHS
                                          (CAPACITY VS. TIP ELEVATION)
```

```
 B. BORING LOG
 ==============

                   DEPTH (FT)    ELEVATION    SPT BLOWS/FT    SOIL TYPE
     ENTRY NO.        D(I)          (FT)          N(I)          ST(I)
     ---------     ---------    ---------    ------------    ---------

         1            1.5          -6.5          1.0             3
         2            4.0          -9.0          3.0             3
         3            6.5         -11.5          3.0             3
         4            9.0         -14.0         99.0             4
         5           11.5         -16.5         99.0             4
         6           16.5         -21.5         99.0             4
         7           20.0         -25.0         99.0             4
         8           25.0         -30.0         99.0             4
         9           30.0         -35.0         99.0             4
        10           35.0         -40.0         99.0             4
        11           40.0         -45.0         99.0             4
        12           45.0         -50.0         99.0             4
        13           51.5         -56.5         57.0             4
        14           56.5         -61.5         56.0             4
        15           60.5         -65.5         99.0             4
        16           65.0         -70.0         99.0             4
        17           70.0         -75.0         99.0             4
        18           75.0         -80.0         99.0             4
        19           80.0         -85.0           .0             3
        20           85.0         -90.0         99.0             4
        21           90.0         -95.0         99.0             4
        22          100.0        -105.0           .0             0

                              SOIL TYPE LEGEND
                             ------------------
            0  -  BOTTOM OF BORING
            1  -  PLASTIC CLAYS
            2  -  CLAY/SILT SAND MIXTURES, SILTS & MARLS
            3  -  CLEAN SAND
            4  -  SOFT LIMESTONE, VERY SHELLY SANDS
            5  -  VOID (NO CAPACITY)
```

```
C. PILE INFORMATION
===================

     TEST PILE SECTION                         ISECT =     4
                                                   {steel H-pile}
     WIDTH OF FLANGE                           WIDTH =     14.00 INCHES
     DEPTH OF SECTION                          DEPTH =     13.61 INCHES
     TRUE X-SECTIONAL AREA                     TAREA =       21.4INCH^2

D. PILE CAPACITY VS. PENETRATION
================================

   TEST     PILE    WT.      ULT.   MOBILIZED  ESTIMATED  ALLOWABLE  ULTIMATE
   PILE     TIP     OF       SIDE      END     FAILURE     PILE       PILE
  LENGTH    ELEV   PILE   FRICTION BEARING    CAPACITY   CAPACITY   CAPACITY
   (FT)     (FT)  (TONS)   (TONS)   (TONS)     (TONS)     (TONS)     (TONS)
  ------    ----  ------   ---------------   ---------  ---------  --------

   10.0    -15.0    .36      5.09    56.24      60.97      30.49     117.21
   15.0    -20.0    .55     13.27    80.49      93.21      46.60     173.69
   20.0    -25.0    .73     23.43    95.27     117.98      58.99     213.25
   25.0    -30.0    .91     33.93    95.27     128.29      64.14     223.56
   30.0    -35.0   1.09     44.42    95.27     138.60      69.30     233.87
   35.0    -40.0   1.27     54.91    95.27     148.91      74.45     244.18
   40.0    -45.0   1.46     65.40    94.30     158.24      79.12     252.54
   45.0    -50.0   1.64     75.64    93.19     167.19      83.60     260.39
   50.0    -55.0   1.82     86.07    92.46     176.70      88.35     269.16
   55.0    -60.0   2.00     96.30    91.70     185.99      93.00     277.69
   60.0    -65.0   2.18    106.20    92.96     196.98      98.49     289.94

     *** THE MAXIMUM PILE LENGTH HAS BEEN REACHED

     NOTES
    -------

     1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
        CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
        FOR SOIL TYPE 1 AND 2.

     2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
        AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.

     3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.

     4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
```

```
                                FOR TIP IN SOIL TYPE 3 OR 4,
                         = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                                FOR TIP IN SOIL TYPE 1 OR 2.

  5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
     NEGATIVE.

  PROBLEM COMPLETED                       ANALYSIS NO.  1
```

```
                    FLORIDA DEPARTMENT OF TRANSPORTATION
                          STRUCTURES DESIGN OFFICE
                 STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                       SPT94 - VERSION 1.0 JUNE, 1994
                        BASED ON RESEARCH BULLETIN RB-121
                "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                          AND DESIGN OF FOUNDATIONS FOR
                  BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

              NOTE    -    THIS PROGRAM IS EXPANDED FROM SPT91
                           TO INCLUDE STEEL H AND PIPE PILES


   A. GENERAL INFORMATION
  ========================

        INPUT FILE NAME                C:\SPT94\BECKETT\B289.DAT
        RUN DATE                       11/09/94
        RUN TIME                       18:17:28

        PROJECT NUMBER                 C394348
        JOB NAME                       BECKETT BRIDGE REPAIRS
        SUBMITTING ENGINEER            LDS
        BORING NO.                     B-2  HP 14x89
        DRILLING DATE                  10/22/94
        STATION NO.                    N/A
        GROUND SURFACE ELEVATION        -5.00 FEET
        TYPE OF ANALYSIS                2 - DETERMINATION OF STATIC
                                            PILE BEARING CAPACITIES
                                            FOR A RANGE OF PILE LENGTHS
                                            (CAPACITY VS. TIP ELEVATION)
```

```
  B. BORING LOG
 ===============

                    DEPTH (FT)    ELEVATION    SPT BLOWS/FT    SOIL TYPE
      ENTRY NO.       D(I)          (FT)          N(I)          ST(I)
      ---------     ---------    ---------    ------------    ---------

          1            1.5          -6.5           1.0             3
          2            4.0          -9.0           3.0             3
          3            6.5         -11.5           3.0             3
          4            9.0         -14.0          99.0             4
          5           11.5         -16.5          99.0             4
          6           16.5         -21.5          99.0             4
          7           20.0         -25.0          99.0             4
          8           25.0         -30.0          99.0             4
          9           30.0         -35.0          99.0             4
         10           35.0         -40.0          99.0             4
         11           40.0         -45.0          99.0             4
         12           45.0         -50.0          99.0             4
         13           51.5         -56.5          57.0             4
         14           56.5         -61.5          56.0             4
         15           60.5         -65.5          99.0             4
         16           65.0         -70.0          99.0             4
         17           70.0         -75.0          99.0             4
         18           75.0         -80.0          99.0             4
         19           80.0         -85.0            .0             3
         20           85.0         -90.0          99.0             4
         21           90.0         -95.0          99.0             4
         22          100.0        -105.0            .0             0

                             SOIL TYPE LEGEND
                            ------------------
            0  -  BOTTOM OF BORING
            1  -  PLASTIC CLAYS
            2  -  CLAY/SILT SAND MIXTURES, SILTS & MARLS
            3  -  CLEAN SAND
            4  -  SOFT LIMESTONE, VERY SHELLY SANDS
            5  -  VOID (NO CAPACITY)
```

```
 C. PILE INFORMATION
 =====================

      TEST PILE SECTION                     ISECT =     4
                                               {steel H-pile}
      WIDTH OF FLANGE                       WIDTH =     14.00 INCHES
      DEPTH OF SECTION                      DEPTH =     13.83 INCHES
      TRUE X-SECTIONAL AREA                 TAREA =       26.1INCH^2

 D. PILE CAPACITY VS. PENETRATION
 =================================

    TEST      PILE     WT.      ULT.   MOBILIZED  ESTIMATED  ALLOWABLE  ULTIMATE
    PILE      TIP      OF       SIDE      END     FAILURE      PILE       PILE
   LENGTH     ELEV    PILE   FRICTION BEARING    CAPACITY   CAPACITY   CAPACITY
    (FT)      (FT)   (TONS)   (TONS)   (TONS)     (TONS)     (TONS)     (TONS)
   ------     ----   ------   ---------------    --------   --------   --------

    10.0     -15.0     .44     5.14     57.15      61.84      30.92     118.99
    15.0     -20.0     .67    13.37     81.79      94.49      47.25     176.28
    20.0     -25.0     .89    23.62     96.81     119.54      59.77     216.35
    25.0     -30.0    1.11    34.20     96.81     129.90      64.95     226.71
    30.0     -35.0    1.33    44.77     96.81     140.25      70.13     237.06
    35.0     -40.0    1.55    55.35     96.81     150.60      75.30     247.41
    40.0     -45.0    1.78    65.92     95.82     159.97      79.98     255.79
    45.0     -50.0    2.00    76.24     94.70     168.94      84.47     263.64
    50.0     -55.0    2.22    86.75     93.95     178.48      89.24     272.43
    55.0     -60.0    2.44    97.06     93.18     187.80      93.90     280.98
    60.0     -65.0    2.66   107.05     94.47     198.85      99.42     293.32

      *** THE MAXIMUM PILE LENGTH HAS BEEN REACHED

      NOTES
      -----

      1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
         CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
         FOR SOIL TYPE 1 AND 2.

      2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
         AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.

      3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.

      4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
```

```
                           FOR TIP IN SOIL TYPE 3 OR 4,
                     = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                           FOR TIP IN SOIL TYPE 1 OR 2.

   5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
      NEGATIVE.

   PROBLEM COMPLETED                         ANALYSIS NO.  1
```

```
                 FLORIDA DEPARTMENT OF TRANSPORTATION
                      STRUCTURES DESIGN OFFICE
             STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                   SPT94 - VERSION 1.0 JUNE, 1994
                     BASED ON RESEARCH BULLETIN RB-121
            "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                      AND DESIGN OF FOUNDATIONS FOR
               BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

          NOTE   -    THIS PROGRAM IS EXPANDED FROM SPT91
                      TO INCLUDE STEEL H AND PIPE PILES


  A. GENERAL INFORMATION
 ========================

      INPUT FILE NAME               C:\SPT94\BECKETT\B373.DAT
      RUN DATE                      11/09/94
      RUN TIME                      18:17:55

      PROJECT NUMBER                C394348
      JOB NAME                      BECKETT BRIDGE REPAIRS
      SUBMITTING ENGINEER           LDS
      BORING NO.                    B-3  HP 14x73
      DRILLING DATE                 10/20/94
      STATION NO.                   N/A
      GROUND SURFACE ELEVATION        3.50 FEET
      TYPE OF ANALYSIS               2 - DETERMINATION OF STATIC
                                         PILE BEARING CAPACITIES
                                         FOR A RANGE OF PILE LENGTHS
                                         (CAPACITY VS. TIP ELEVATION)
```

```
B. BORING LOG
===============

                 DEPTH (FT)    ELEVATION    SPT BLOWS/FT    SOIL TYPE
   ENTRY NO.        D(I)          (FT)          N(I)           ST(I)
   ---------     ----------    ---------    ------------    ---------

       1             1.5          2.0           5.0             3
       2             4.0          -.5           5.0             3
       3             6.5         -3.0           5.0             3
       4             9.0         -5.5          19.0             3
       5            11.5         -8.0           9.0             3
       6            16.5        -13.0           3.0             2
       7            20.0        -16.5           8.0             4
       8            25.0        -21.5          99.0             4
       9            30.0        -26.5          99.0             4
      10            35.0        -31.5          99.0             4
      11            40.0        -36.5          99.0             4
      12            45.0        -41.5          99.0             4
      13            51.5        -48.0          99.0             4
      14            56.5        -53.0          99.0             4
      15            60.5        -57.0          99.0             4
      16            65.0        -61.5          99.0             4
      17            69.0        -65.5          50.0             4
      18            69.1        -65.6            .0             5
      19            71.0        -67.5            .0             5
      20            71.1        -67.6          29.0             4
      21            75.0        -71.5          25.0             4
      22            80.0        -76.5          61.0             4
      23            90.0        -86.5            .0             0

                         SOIL TYPE LEGEND
                        -----------------
          0  -  BOTTOM OF BORING
          1  -  PLASTIC CLAYS
          2  -  CLAY/SILT SAND MIXTURES, SILTS & MARLS
          3  -  CLEAN SAND
          4  -  SOFT LIMESTONE, VERY SHELLY SANDS
          5  -  VOID (NO CAPACITY)
```

```
 C. PILE INFORMATION
 ====================

      TEST PILE SECTION                        ISECT =     4
                                                   {steel H-pile}
      WIDTH OF FLANGE                          WIDTH =     14.00 INCHES
      DEPTH OF SECTION                         DEPTH =     13.61 INCHES
      TRUE X-SECTIONAL AREA                    TAREA =       21.4INCH^2

 D. PILE CAPACITY VS. PENETRATION
 ================================

    TEST      PILE     WT.      ULT.   MOBILIZED  ESTIMATED  ALLOWABLE  ULTIMATE
    PILE      TIP      OF       SIDE      END      FAILURE     PILE       PILE
   LENGTH     ELEV    PILE   FRICTION  BEARING    CAPACITY   CAPACITY   CAPACITY
    (FT)      (FT)   (TONS)   (TONS)    (TONS)     (TONS)     (TONS)     (TONS)
   ------     ----   ------   ---------------     ---------  ---------  --------

    10.0      -6.5     .36      3.47     11.74       14.85       7.42      26.58
    15.0     -11.5     .55      5.80     19.25       24.51      12.25      43.76
    20.0     -16.5     .73     10.82     38.30       48.39      24.19      86.69
    25.0     -21.5     .91     15.25     56.18       70.52      35.26     126.71
    30.0     -26.5    1.09     24.51     86.96      110.38      55.19     197.34
    35.0     -31.5    1.27     34.75     95.27      128.74      64.37     224.01
    40.0     -36.5    1.46     45.11     95.27      138.92      69.46     234.19
    45.0     -41.5    1.64     55.53     95.27      149.16      74.58     244.43
    50.0     -46.5    1.82     65.97     95.27      159.42      79.71     254.69
    55.0     -51.5    2.00     76.42     95.27      169.69      84.85     264.96
    60.0     -56.5    2.18     86.89     92.50      177.20      88.60     269.69
    65.0     -61.5    2.37     96.60     88.90      183.13      91.57     272.03
    70.0     -66.5    2.55    108.70       .00      106.15      53.07     106.15
    75.0     -71.5    2.73    114.09     47.09      158.44      79.22     205.53

      *** ERROR *** PILE TIP EXCEEDS BORING LOG FOR LENGTH =   80.00 FT

      NOTES
     -------

      1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
         CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
         FOR SOIL TYPE 1 AND 2.

      2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
         AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.
```

```
  3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.

  4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
                         FOR TIP IN SOIL TYPE 3 OR 4,
                   = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                         FOR TIP IN SOIL TYPE 1 OR 2.

  5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
     NEGATIVE.

  PROBLEM COMPLETED                         ANALYSIS NO.  1
```

```
               FLORIDA DEPARTMENT OF TRANSPORTATION
                     STRUCTURES DESIGN OFFICE
           STATIC PILE BEARING CAPACITY ANALYSIS PROGRAM
                  SPT94 - VERSION 1.0 JUNE, 1994
                  BASED ON RESEARCH BULLETIN RB-121
          "GUIDELINES FOR USE IN THE SOILS INVESTIGATION
                   AND DESIGN OF FOUNDATIONS FOR
            BRIDGE STRUCTURES IN THE STATE OF FLORIDA"

        NOTE    -    THIS PROGRAM IS EXPANDED FROM SPT91
                     TO INCLUDE STEEL H AND PIPE PILES


  A. GENERAL INFORMATION
 =======================

       INPUT FILE NAME                  C:\SPT94\BECKETT\B389.DAT
       RUN DATE                         11/09/94
       RUN TIME                         18:18:21

       PROJECT NUMBER                   C394348
       JOB NAME                         BECKETT BRIDGE REPAIRS
       SUBMITTING ENGINEER              LDS
       BORING NO.                       B-3  HP 14x89
       DRILLING DATE                    10/20/94
       STATION NO.                      N/A
       GROUND SURFACE ELEVATION           3.50 FEET
       TYPE OF ANALYSIS                  2 - DETERMINATION OF STATIC
                                             PILE BEARING CAPACITIES
                                             FOR A RANGE OF PILE LENGTHS
                                             (CAPACITY VS. TIP ELEVATION)
```

B. BORING LOG

ENTRY NO.	DEPTH (FT) D(I)	ELEVATION (FT)	SPT BLOWS/FT N(I)	SOIL TYPE ST(I)
1	1.5	2.0	5.0	3
2	4.0	-.5	5.0	3
3	6.5	-3.0	5.0	3
4	9.0	-5.5	19.0	3
5	11.5	-8.0	9.0	3
6	16.5	-13.0	3.0	2
7	20.0	-16.5	8.0	4
8	25.0	-21.5	99.0	4
9	30.0	-26.5	99.0	4
10	35.0	-31.5	99.0	4
11	40.0	-36.5	99.0	4
12	45.0	-41.5	99.0	4
13	51.5	-48.0	99.0	4
14	56.5	-53.0	99.0	4
15	60.5	-57.0	99.0	4
16	65.0	-61.5	99.0	4
17	69.0	-65.5	50.0	4
18	69.1	-65.6	.0	5
19	71.0	-67.5	.0	5
20	71.1	-67.6	29.0	4
21	75.0	-71.5	25.0	4
22	80.0	-76.5	61.0	4
23	90.0	-86.5	.0	0

SOIL TYPE LEGEND

0 - BOTTOM OF BORING
1 - PLASTIC CLAYS
2 - CLAY/SILT SAND MIXTURES, SILTS & MARLS
3 - CLEAN SAND
4 - SOFT LIMESTONE, VERY SHELLY SANDS
5 - VOID (NO CAPACITY)

```
 C. PILE INFORMATION
=====================

     TEST PILE SECTION                              ISECT =     4
                                                        {steel H-pile}
     WIDTH OF FLANGE                                WIDTH =    14.00 INCHES
     DEPTH OF SECTION                               DEPTH =    13.83 INCHES
     TRUE X-SECTIONAL AREA                          TAREA =      26.1INCH^2

 D. PILE CAPACITY VS. PENETRATION
==================================

   TEST      PILE     WT.      ULT.   MOBILIZED   ESTIMATED   ALLOWABLE   ULTIMATE
   PILE      TIP      OF       SIDE      END      FAILURE      PILE        PILE
  LENGTH     ELEV     PILE   FRICTION BEARING     CAPACITY    CAPACITY    CAPACITY
   (FT)      (FT)    (TONS)   (TONS)   (TONS)      (TONS)      (TONS)      (TONS)
  ------     ----    ------  ---------------     ---------   ---------   --------

   10.0      -6.5      .44     3.50     11.93       14.98        7.49       26.91
   15.0     -11.5      .67     5.85     19.56       24.75       12.37       44.31
   20.0     -16.5      .89    10.91     38.92       48.93       24.47       87.85
   25.0     -21.5     1.11    15.38     57.09       71.35       35.68      128.44
   30.0     -26.5     1.33    24.70     88.37      111.74       55.87      200.11
   35.0     -31.5     1.55    35.02     96.81      130.28       65.14      227.09
   40.0     -36.5     1.78    45.47     96.81      140.50       70.25      237.31
   45.0     -41.5     2.00    55.97     96.81      150.78       75.39      247.59
   50.0     -46.5     2.22    66.49     96.81      161.08       80.54      257.89
   55.0     -51.5     2.44    77.03     96.81      171.40       85.70      268.21
   60.0     -56.5     2.66    87.58     93.99      178.91       89.45      272.90
   65.0     -61.5     2.89    97.37     90.34      184.82       92.41      275.16
   70.0     -66.5     3.11   109.56       .00      106.45       53.23      106.45
   75.0     -71.5     3.33   115.00     47.85      159.51       79.76      207.36

      *** ERROR *** PILE TIP EXCEEDS BORING LOG FOR LENGTH =   80.00 FT

      NOTES
     -------

      1. FOR PILE TIP EMBEDDED IN SOIL TYPE 3 AND 4, END BEARING IS
         CALCULATED BASED ON BLOCK AREA WHILE TRUE X-SECTIONAL AREA IS USED
         FOR SOIL TYPE 1 AND 2.

      2. DAVISSON PILE CAPACITY IS AN ESTIMATE BASED ON FAILURE CRITERIA,
         AND EQUALS ULTIMATE SIDE FRICTION PLUS MOBILIZED END BEARING.
```

```
   3. ALLOWABLE PILE CAPACITY IS 1/2 THE DAVISSON PILE CAPACITY.

   4. ULT. CAPACITY = ULT. SKIN FRICTION + 2*MOBILIZED END BEARING,
                          FOR TIP IN SOIL TYPE 3 OR 4,
                    = ULT. SKIN FRICTION + 3*MOBILIZED END BEARING,
                          FOR TIP IN SOIL TYPE 1 OR 2.

   5. PILE CAPACITIES ARE SET TO ZERO IF THEIR COMPUTED VALUES ARE
      NEGATIVE.

   PROBLEM COMPLETED                      ANALYSIS NO.  1
```

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

More construction problems are caused by site subsurface conditions than any other factor. As troublesome as subsurface problems can be, their frequency and extent have been lessened considerably in recent years, thanks to the Association of Soil and Foundation Engineers (ASFE).

When ASFE was founded in 1969, subsurface problems were frequently being resolved through lawsuits. In fact, the situation had grown to such alarming proportions that consulting geotechnical engineers had the worst professional liability record of all design professionals. By 1980, ASFE-*member consulting soil and foundation engineers had the best professional liability record*. This dramatic turn-about can be attributed directly to client acceptance of problem-solving programs and materials developed by ASFE for its members' application. *This acceptance was gained because clients perceived the* ASFE *approach to be in their own best interests.* Disputes benefit only those who earn their living from others' disagreements.

The following suggestions and observations are offered to help you reduce the geotechnical-related delays, cost-overruns and other costly headaches that can occur during a construction project.

A GEOTECHNICAL ENGINEERING REPORT IS BASED ON A UNIQUE SET OF PROJECT-SPECIFIC FACTORS

A geotechnical engineering report is based on a subsurface exploration plan designed to incorporate a unique set of project-specific factors. These typically include: the general nature of the structure involved, its size and configuration; the location of the structure on the site and its orientation; physical concomitants such as access roads, parking lots, and underground utilities, and the level of additional risk which the client assumed by virtue of limitations imposed upon the exploratory program. To help avoid costly problems, consult the geotechnical engineer to determine how any factors which change subsequent to the date of his report may affect his recommendations.

Unless your consulting geotechnical engineer indicates otherwise, *your geotechnical engineering report should not be used:*

- When the nature of the proposed structure is changed, for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one;
- when the size or configuration of the proposed structure is altered;
- when the location or orientation of the proposed structure is modified;
- when there is a change of ownership, or
- for application to an adjacent site.

A geotechnical engineer cannot accept responsibility for problems which may develop if he is not consulted after factors considered in his report's development have changed.

MOST GEOTECHNICAL "FINDINGS" ARE PROFESSIONAL ESTIMATES

Site exploration identifies actual subsurface conditions only at those points where samples are taken, when they are taken. Data derived through sampling and subsequent laboratory testing are extrapolated by the geotechnical engineer who then renders an opinion about overall subsurface conditions, their likely reaction to proposed construction activity, and appropriate foundation design. Even under optimal circumstances actual conditions may differ from those opined to exist, because no geotechnical engineer, no matter how qualified, and no subsurface exploration program, no matter how comprehensive, can reveal what is hidden by earth, rock and time. For example, the actual interface between materials may be far more gradual or abrupt than the report indicates, and actual conditions in areas not sampled may differ from predictions. *Nothing can be done to prevent the unanticipated, but steps can be taken to help minimize their impact.* For this reason, *most experienced owners retain their geotechnical consultant through the construction stage,* to identify variances, conduct additional tests which may be needed, and to recommend solutions to problems encountered on site.

SUBSURFACE CONDITIONS CAN CHANGE

Subsurface conditions may be modified by constantly-changing natural forces. Because a geotechnical engineering report is based on conditions which existed at the time of subsurface exploration, *construction decisions should not be based on a geotechnical engineering report whose adequacy may have been affected by time.* Speak with the geotechnical consultant to learn if additional tests are advisable before construction starts.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical report. The geotechnical engineer should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

A GEOTECHNICAL ENGINEERING REPORT IS SUBJECT TO MISINTERPRETATION

Costly problems can occur when other design professionals develop their plans based on misinterpretations of a geotechnical engineering report. To help avoid these problems, the geotechnical engineer should be retained to work with other appropriate design professionals to explain relevant geotechnical findings and to review the adequacy

APPENDIX B

Williams Earth Science Phase 1 Geotechnical Report, Dated May 18, 2009

WILLIAMS
EARTH SCIENCES, INC.
Choose Integrity.

10600 Endeavour Way
Largo, Florida 33777
Office: (727) 541-3444
Fax: (727) 541-1510
www.williamsearthsciences.com

May 18, 2009

Murray McDonough, P.E.
URS Corporation
7650 W. Courtney Campbell Causeway
Tampa, FL 33607-1462

Subject: Phase 1 Geotechnical Report
Beckett Bridge
Pinellas County
Williams' Project No. 1309-004-01

Gentlemen:

Williams Earth Sciences, Inc. (Williams) has completed the Phase 1 Geotechnical work for the referenced project. This work was performed in accordance with our agreement with URS, dated April 17, 2009.

This report contains the results and discussion of the Electrical Resistivity Imaging (ERI) conducted during this Phase 1 Geotechnical study. In addition, recommendations for additional subsurface exploration, settlement and rotation monitoring are provided.

Williams Earth Sciences, Inc. appreciates this opportunity to provide this report and looks forward to continuing working with you on this project. If you have any questions concerning this report, please contact the undersigned.

Sincerely,

WILLIAMS EARTH SCIENCES, INC.

Larry D. Spears, P.E.
Senior Engineer
Florida Registration No. 52105

Brian Jory, P.E.
Senior Geotechnical Engineer
Florida Registration N. 46634

Distribution: (3) Addressee
(1) File

I:\Projects\LARGO\13\1309\1309-004-00 Beckett Bridge - URS Corp\Report\Phase I Report 5-18-09.DOC

Table of Contents

Appendices

1. Project Information

This Phase 1 study was performed to identify karst features in the area of the footprint of the Beckett Bridge foundation. Our original proposal included soil borings spread across the Beckett Bridge footprint to 1) identify the subsurface conditions and 2) to assist in the repair of the existing bridge or design of a replacement bridge. However, due to cost constraints, the scope of work was reduced to simply conducting the ERI study, and performing the soil borings later based on the results of the ERI study. The Beckett Bridge is located in Tarpon Springs, Florida, along Riverside Drive at the Anclote River, as shown on Figure 1, Site Location Map, in Appendix A.

The bridge is multi-spanned and has been experiencing lateral movement and subsidence. The bridge is a two-lane bascule bridge about 20 feet across and 360 feet in length with two-foot wide sidewalks on both sides. The approach span structures are constructed of 14-inch square prestressed concrete piles. There are four spans on the east approach and five spans on the west approach. The bascule is approximately 40 feet long and is supported on a concrete pier. The bridge was originally constructed in 1924 using timber piling and timber bents. The bridge approach spans were reconstructed in 1956 using reinforced concrete, however, the original bascule span remained. Structural repairs were performed in 1979 and crutch bents installed in 1995.

2. Previous Geotechnical Study

Williams provided a report dated November 10, 1994, which provided recommendations for the installation of crutch bents using H-Piles. During the 1994 study, Williams performed three Standard penetration Test (SPT) borings; one was performed at the west abutment, one at the east abutment, and one was performed in the vicinity of the Bent 5, adjacent to the bascule. The two abutment borings were performed from land and the Bent 5 boring was performed from the bridge (as opposed to a barge over water). The results of the borings are included in Appendix A. Two SPT borings were also performed by others (PSI). These two borings were performed at Bent 6 from the bridge. One was performed in the westbound lane and the other was performed in the eastbound lane.

3. Phase 1 Study

For this Phase 1 study, Electrical Resistivity Imaging (ERI) was conducted. The purpose of the ERI testing was to determine the vertical extent and lateral continuity of soil layers and to identify possible karst hazards within the river along the sides of the bridge. The ERI testing was performed by “Subsurface Evaluations, Inc.” (SEI) and their report, dated April 28, 2009, is included in Appendix B.

The results of the ERI testing indicated several interesting features and anomalies within the vicinity of the bridge footprint. First, there appears to be an anomaly near Bent 6, with the center approximated just north of the bridge, as depicted on Figure 1 of the SEI report. In addition, there appears to be a shelf at about 20 to 40 feet in depth indicating a change in soil material and/or density, as indicated on Figure 1.

Boring B-1 was performed very close to the ERI anomaly indicated at Bent 6. The boring indicates that there is dense grading to medium dense dark brown to brown fine sand with trace of silt from the mud-line to about 10 feet below the mud-line, followed by a nine foot thick layer of stiff dark gray sandy silt layer, from 10 to 19 feet below the mud-line. The silt layer was underlain by a relatively thin layer of hard limestone, from 19 to 24 feet below the mud-line. From 24 to 40 feet below the mud-line, a medium dense grading to very loose layer of brown fine sand with trace of silt (SP-SM) was encountered. A second layer of hard limestone was present from 40 to 45 feet below the mud-line, followed by medium dense brown find sand with trace of silt (SP-SM) to the termination depth of the boring at about 57 feet below the mud-line.

Boring B-1 (PSI) and the ERI results correlate at Bent 6. In addition, this anomaly is indicative of a relic sinkhole, albeit in the Anclote River. Boring B-2 was also performed at Bent 6, on the opposite side of the bridge (eastbound lane). This boring indicated somewhat similar soils to Boring B-1, however, there was no evidence of the stiff silt layer at 10 to 19 feet below the mud-line.

The borings conducted by Williams in the 1994 study indicated a soil Stratigraphy that was quite dissimilar to the borings conducted at Bent 6 by PSI. These borings generally indicate a surficial layer of sands to silty sands or clayey soils, followed by very hard limestone to the full depth of the borings. There were a few minor variations in the subsurface soils, such as a thin layer of clay (CH) material in boring B-1 at a depth of 47 to 58 feet below the ground surface; a very loose shelly fine sand layer from 77 to 84 feet below the mud-line at boring B-2; and a possible void from 69 to 71 feet below the ground surface at boring B-3. Nonetheless, the medium dense fine sand with trace of silty soils was not encountered in the SPT borings conducted by Williams.

The nature of encountering highly dissimilar soils in a relatively short distance indicates that this area has localized karst features. Anclote River in known for its erratic karst features. The subsurface is characterized by a sand layer overlying a shallow limestone. There is a lack of clay layering in this area and therefore there is a high degree of localized subsidence and raveling of the surficial soils into the karst limestone. Review of the ERI results indicates that the surficial karst solution features, or surficial relic sinkhole features, may be more prevalent near the center of the bridge. There also appears to be an apparent shelf, as indicated on ERI transects T3 and T4. Review of ERI transects T3, T4 and T5 indicate the possibility of a solution zone near to below the bridge footprint that may be located in a southwest orientation. However, it may be possible that the bascule bridge footing and the piles may be providing interference of the ERI data.

It has been reported that there has been settlement and rotation of the bents and/or bascule pier. There are a number of potential causes for this, both structurally and geotechnically, however, from a geotechnical standpoint, the causes may be due to subsidence of the piles due to 1) active sinkhole conditions, or 2) insufficient pile bearing both axially and laterally, or some combination of all. Since the settlement and rotation is occurring slowly, it is difficult to ascertain if it is continuing or if the settlement has ceased. Another consideration is the age of the timber piles supporting the bascule pier, which are about 85 years old, and are likely in poor condition due to fatigue, rot, or some other form of deterioration.

As previously mentioned, there was HP 14 X 73 crutch bent piles installed in 1996. The 1996 Plans indicate crutch bents at Bent 6 and Bent 7, and pier stabilizers for the bascule. The lengths of the crutch bent piles varied dramatically from tip elevations of about -30 to -200 feet. These lengths were taken from old facsimile correspondence between Williams and DSA. Interestingly, there was a minimum tip elevation of -35 feet indicated on the plans; therefore, one of the piles did not achieve the minimum tip elevation in accordance with the plans. The piles were also supposedly preformed to an elevation of -27 feet, and the preformed hole was supposed to be grouted. The HP crutch bent piles were also planned to be jacketed using an epoxy mix from elevation -4 to +4 feet, at the splash zone of the piles. Based on the 2007 Bridge Inspection Report, performed by Volkert & Associates, Inc., the "jackets are in good condition with no washouts or exposed base pile".

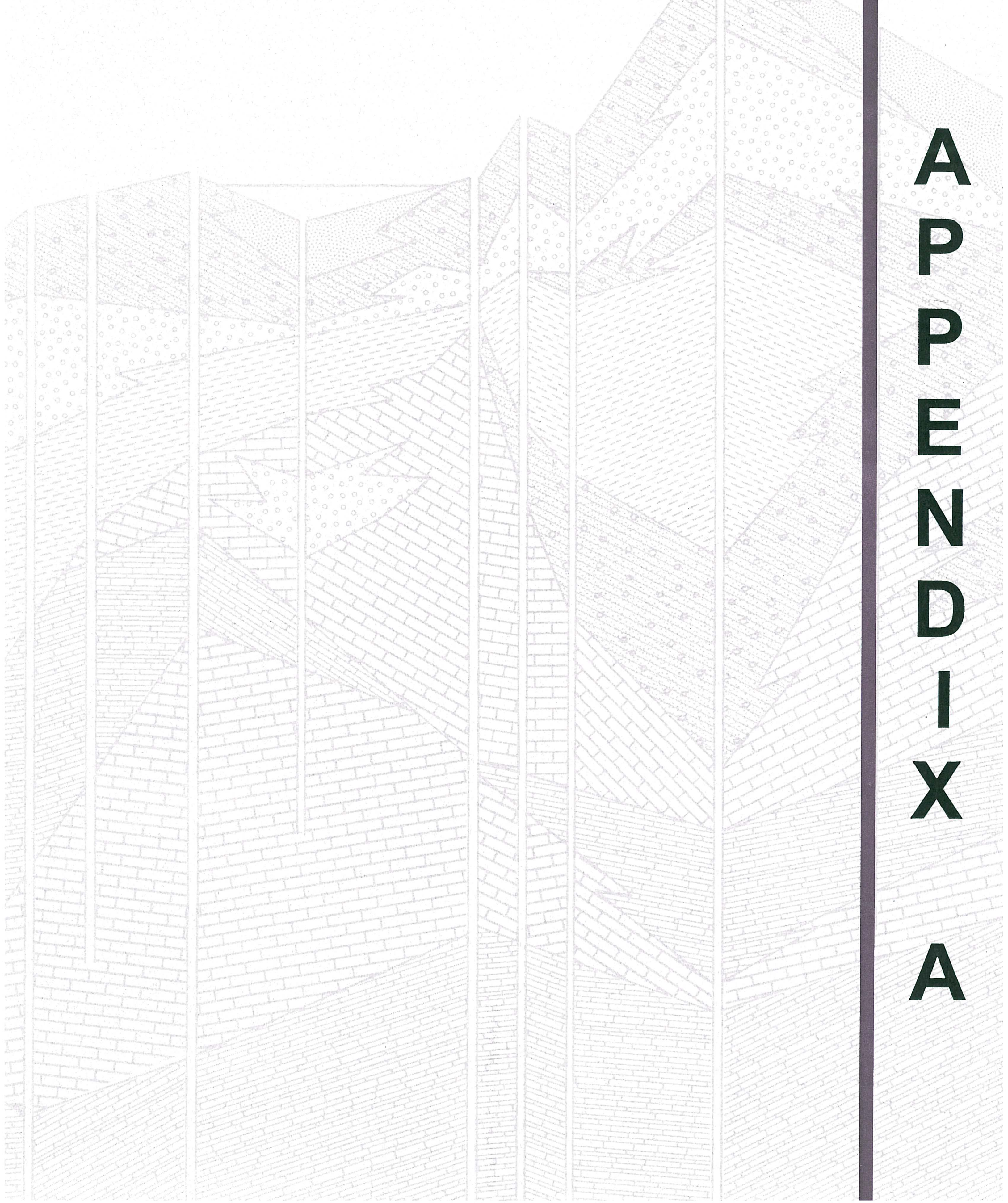
4. Recommendations

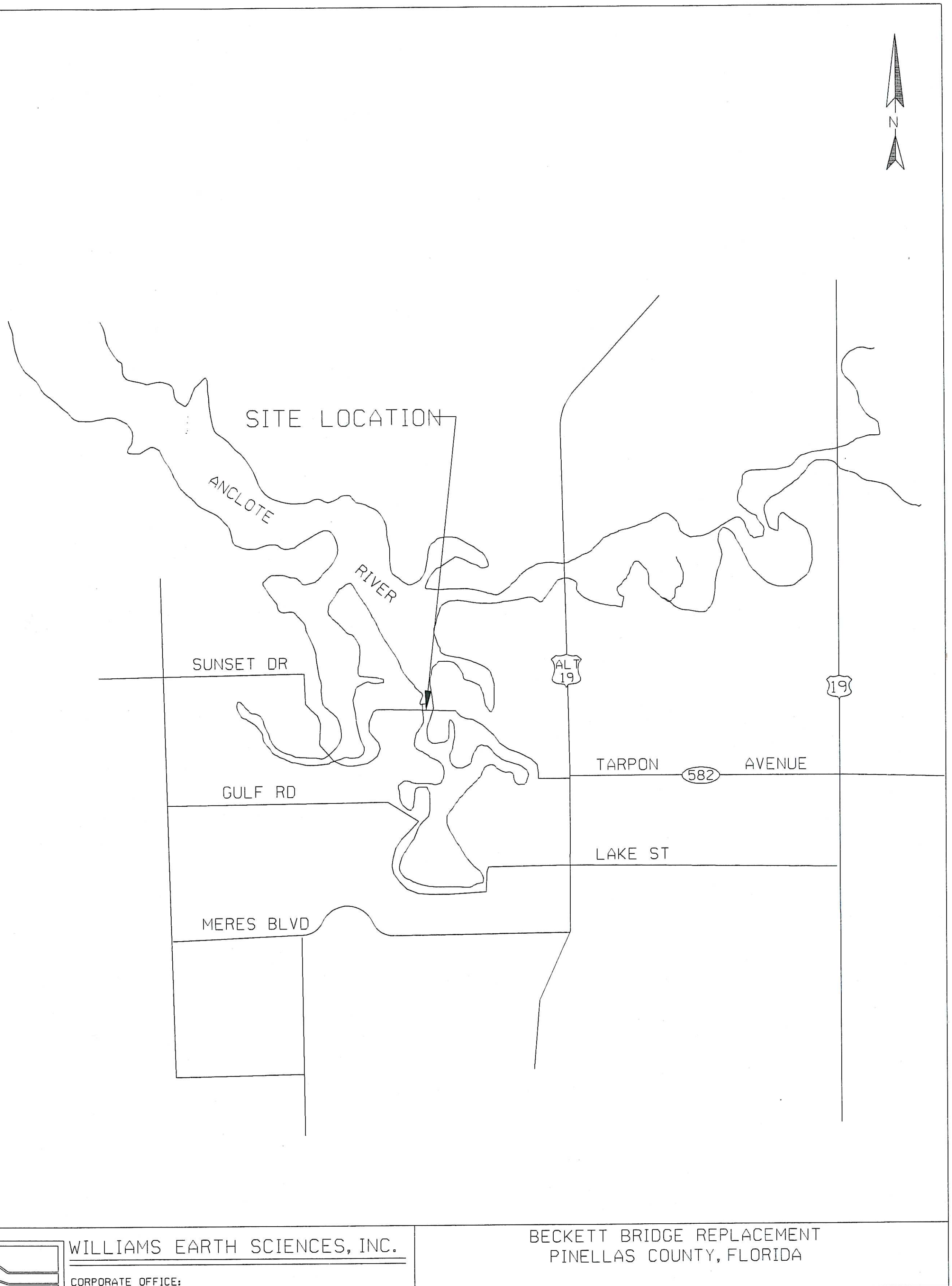
Williams understands that this bridge is under evaluation for repair or replacement. If repair is feasible, then settlement and rotation monitoring of the bents and piers is recommended to determine how, where and the amount that it is occurring so that the bents and/or piers can be shored to stabilize the settlement and rotation. Evaluation of how to shore the bents and/or piers can then be made, however, it will likely require additional crutch bents and stabilizers at the bascule pier if it is determined that the settlement and rotation can be stabilized by reinforcing the substructure.

Additional borings may be required if the settlement and rotation is occurring at locations where there is no soils information to assist in the design and construction of the crutch bents or pier stabilizers.

If it is determined that the bridge should be replaced, then additional soil borings will be required to assist in the design and construction. Williams would coordinate with URS on the number of borings, location and depth that best suites the needs of the design and construction, basing it on the subsurface conditions known to be suspect to subsidence for substructure units. Recommendations for foundation design and selection of foundation support, and recommendations for foundation installation would subsequently be provided in a substructures geotechnical report.

APPENDIX A





WILLIAMS EARTH SCIENCES, INC.

CORPORATE OFFICE:
10600 Endeavour Way, Largo, FL 34647

Largo: (813) 541-3444 FAX: (813) 541-1510
Jacksonville: (904) 262-8852 FAX: (904) 262-8864
Panama City: (904) 747-9419 FAX: (904) 763-2454

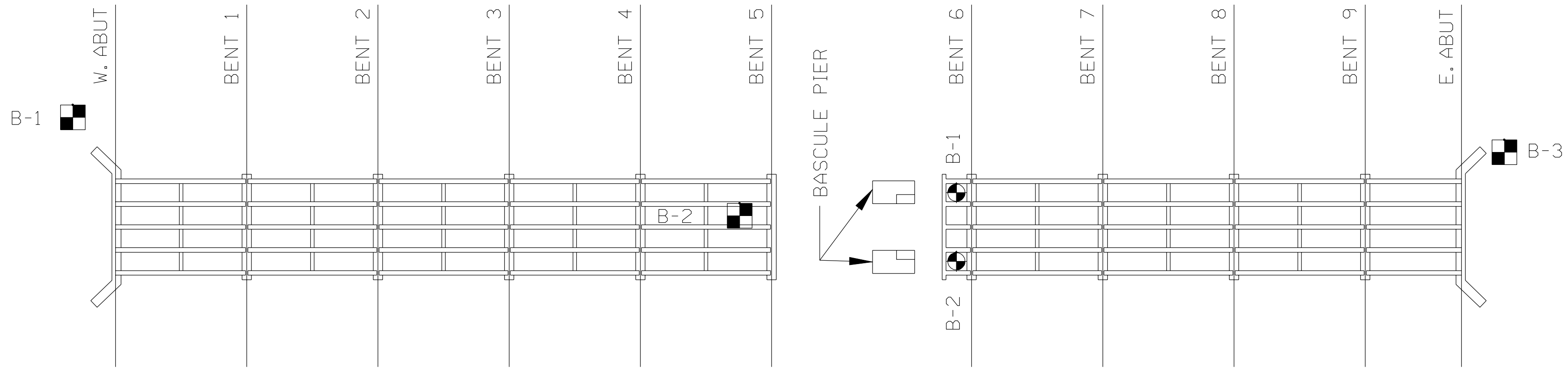
BECKETT BRIDGE REPLACEMENT
PINELLAS COUNTY, FLORIDA

SITE LOCATION MAP

Drawn By: TEJ
Checked By: LDS
Date: 9/11/94
Report No.C394348
Scale: N.T.S.
Figure No. 1

site

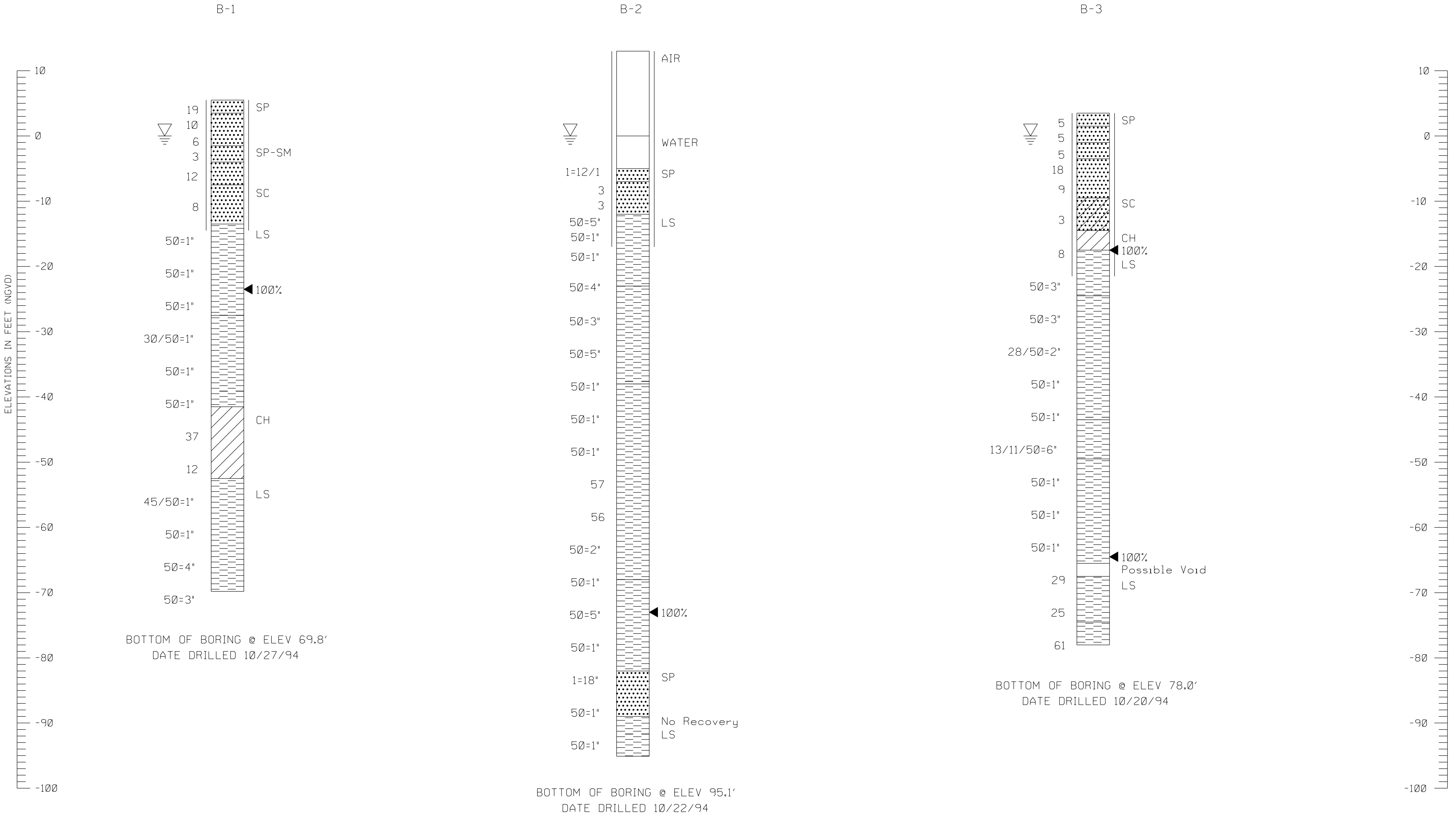
FED. ROAD DIV. NO.	STATE	PROJECT NO.	FISCAL YEAR	SHEET NO.
	FLA			



PLAN

LEGEND

- ▨ = SP, SP-SM and SP-SC, Sands and slightly clayey sands
- ▨ = CH, Inorganic clays of low plasticity
- ▨ = SC, Clayey sands and very sandy clays
- ▤ = LS, Limestone



BOTTOM OF BORING @ ELEV 69.8'
DATE DRILLED 10/27/94

BOTTOM OF BORING @ ELEV 95.1'
DATE DRILLED 10/22/94

BOTTOM OF BORING @ ELEV 78.0'
DATE DRILLED 10/20/94

GENERAL NOTES

DRILL AND PENETRATION TESTING WERE PERFORMED IN ACCORDANCE WITH ASTM D 1586. NUMBER TO LEFT OF BORING INDICATES BLOWS OF 1 3/8" I.D., 2" O.D. SPLIT-SPOON FOR 12" OF PENETRATION (UNLESS OTHERWISE NOTED) WITH A 140 LB. HAMMER DROPPED 30 INCHES.

THE BORING LOGS SHOWN REPRESENT SUBSURFACE CONDITIONS WITHIN THE BOREHOLE AT THE TIME OF DRILLING. NO WARRANTY AS TO THE SUBSURFACE CONDITION, STRATA DEPTH OR SOIL CONSISTENCY BETWEEN OR OUTSIDE BORING LOCATIONS IS EXPRESSED OR IMPLIED BY THIS DRAWING.

ELEVATIONS SHOWN ARE APPROXIMATED BY WATER LEVEL AND WATER TABLE MEASURED AT TIME AND DATE BORINGS WERE COMPLETED.

REFER TO FINAL REPORT FOR ADDITIONAL BORING INFORMATION.

CREW CHIEF: SPOON
DRILLER: PATTERSON
DRILL RIG TYPE:FAILING 250

LEGEND

- ▽ = Water Table @ end of drilling
- ▯ = Casing used
- ▮ = Shelby Tube
- ◀100% = Percent Loss of Circulation

ENVIRONMENTAL CLASSIFICATION

SUBSTRUCTURE: CORROSIVE (EXTREMELY AGGRESSIVE)
SUBSTRUCTURE: CORROSIVE (EXTREMELY AGGRESSIVE)

Granular Materials- Relative Density	SPT (Blows/Ft)
Very Loose	Less than 4
Loose	4 - 10
Medium Dense	11 - 30
Dense	31 - 50
Very Dense	Greater than 50

Silts and Clays- Consistency	SPT (Blows/Ft)
Very Soft	Less than 2
Soft	2 - 4
Firm	5 - 8
Stiff	9 - 15
Very Stiff	16 - 30
Hard	Greater than 30

C394348 BRDG1

REVISIONS					
Date	By	Description	Date	By	Description

	Names	Dates
Drawn by	TEJ	11/9/94
Checked by	LDS	11/9/94
Designed by	LDS	11/9/94
Checked by	KDB	11/9/94
Approved by	K. D. BENNETT	

ENGINEER OF RECORD:
WILLIAMS EARTH SCIENCES, INC.
10600 ENDEAVOUR WAY
LARGO, FLORIDA 34647

LOGO:
WILLIAMS EARTH SCIENCES, INC.
CORPORATE OFFICE:
10600 Endeavour Way, Largo, FL 34647
Largo: (813) 541-3444 FAX: (813) 541-1510
Jacksonville: (904) 262-8852 FAX: (904) 262-8864
Panama City: (904) 747-9419 FAX: (904) 763-2454

SEAL:

FLORIDA DEPARTMENT OF TRANSPORTATION
STRUCTURES DESIGN OFFICE

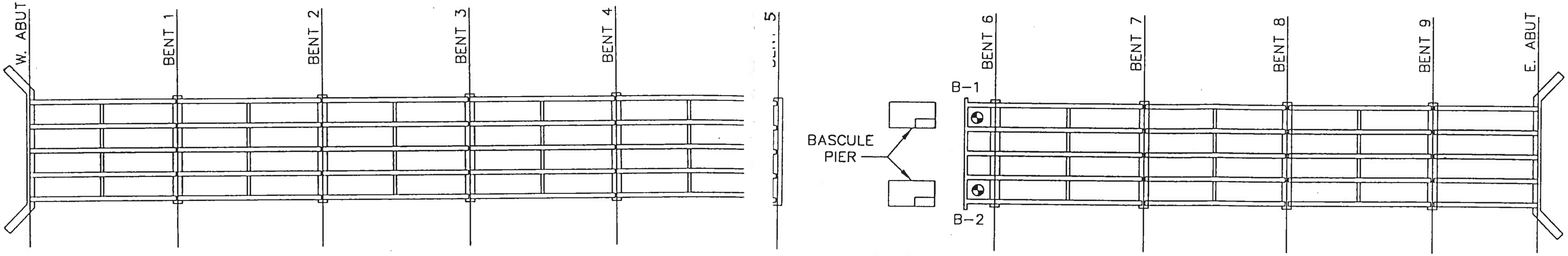
ROAD NO.	COUNTY	PROJECT NO.
SR936	PINELLAS	

SHEET TITLE:
REPORT OF CORE BORINGS

Drawing No.

PROJECT NAME:
BECKETT BRIDGE REPLACEMENT

Index No.



BORING L CATION PLAN

SC E: N.T.S.

L GEND

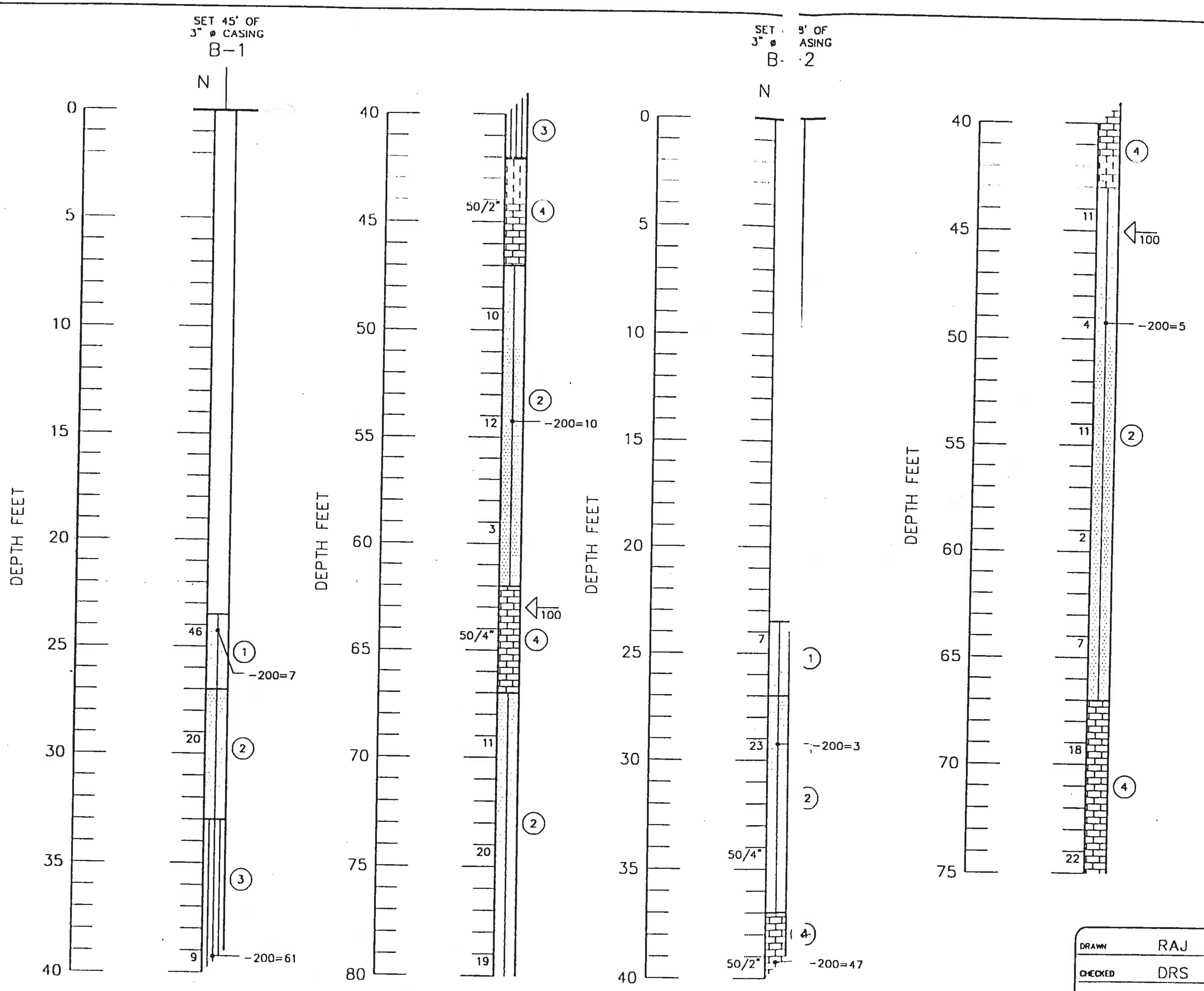
● Approxim e SPT boring location

DRAWN RAJ
CHECKED DRS
APPROVED HVJ
SCALE NOTED

GEOTECHNICAL SERVICES
BECKETT BASCULE
BRIDGE REPLACEMENT
PINELLAS COUNTY, FLORIDA

psi **Jammal & Associates, Inc.**
A Division of Professional Service Industries, Inc.

DATE DEC 93
PROJ. NO. 775-35264
SHEET 1



SOIL PROFILES

VERTICAL SCALE: 1"=5'

LEGEND

① ▯ Dark brown fine SAND with trace silt (SP–SM)

② ▤ Brown fine SAND with trace silt (SP–SM)

③ ▥ Dark gray sandy SILT (ML)

④ ▦ Calcareous silts and weathered LIMESTONE (rock)

SP Unified Soil Classification group symbol as determined by visual review

N SPT "N" value in blows/foot

50/3" Fifty blows for three inches

◁ Loss of circulation (%)

-200 Fines passing No. 200 sieve (%)

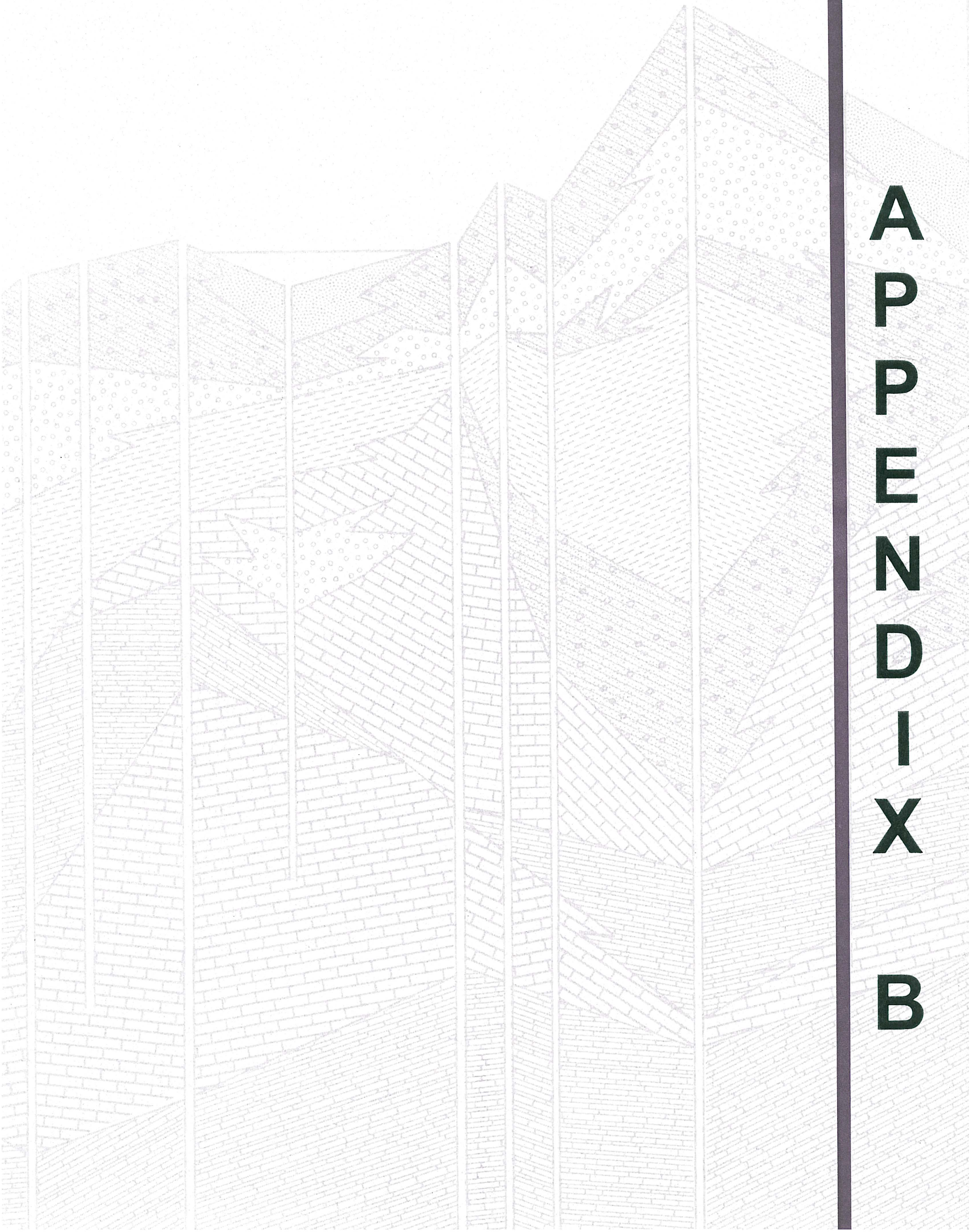
DRAWN	RAJ
CHECKED	DRS
APPROVED	HVJ
SCALE	NOTED

GEOTECHNICAL SERVICES
BECKETT BASCULE
BRIDGE REPLACEMENT
PINELLAS COUNTY, FLORIDA

psi Jammal & Associates, Inc.
A Division of Professional Service Industries, Inc.

DATE DEC 93	PROJ. NO. 775-35264	SHEET 2

APPENDIX B



SUBSURFACE EVALUATIONS, INC.
Engineering Geology & Geophysics

13617 North Florida Avenue
Tampa, FL 33613 USA
Voice: (813) 353-9083
Fax (813) 353-9653
www.sei-tampa.com

April 28, 2009

Mr. Larry Spears, P.E., Geotechnical Engineer
Williams Earth Sciences, Inc. (Client)
10600 Endeavour Way
Largo, Florida 33777

Subject: Electrical Resistivity Imaging Geophysical Survey Report
Beckett Bridge Project
Riverside Drive at the Anclote River
Tarpon Springs, Florida

Dear Mr. Spears:

In accordance with your authorization, Subsurface Evaluations, Inc. (SEI) has conducted an Electrical Resistivity Imaging (ERI) survey at the above-referenced subject site. The ERI survey was performed on April 21st and 22nd, 2009. This report is subject to the limitations shown on Attachment A.

Background and Purpose

The subject site is the existing Beckett Bridge located along Riverside Drive crossing the entrance to Minetta and Whitcomb Bayous in Tarpon Springs, Florida. The bridge is a Bascule bridge reconstructed in 1956. Through our discussions it was indicated that the supports for the bridge have undergone apparent subsidence and lateral displacement resulting in the misalignment of the bridge. The bridge was reported to have been repaired for similar subsidence problems approximately 15 years ago at which time additional supports (H-piles) were installed at the bridge.

The general soil conditions present along the bridge based upon soil borings were indicated to consist of approximately seven (7) feet of sand underlain by hard limestone. However, during the installation of the H-piles, apparent solution features were encountered resulting in some driven piling depths of as much as 120 feet.

The purpose of the geophysical survey is to document the vertical extent and lateral continuity of soil layers and to identify possible karst hazards within the river along the sides of the bridge. The objective of the survey is to characterize the geology directly underlying the river to assist in evaluating ground stability to promote effective geotechnical engineering design and testing.

Electrical Resistivity Imaging (ERI) Survey

ERI Methods and Equipment

Andrew Glasbrenner, P.G., Senior Geologist and Scott Purcell, SEI Project Manager, performed the survey assisted by additional SEI staff. Mr. Glasbrenner and SEI staff prepared the figures and text of the report.

ERI is a geophysical method of obtaining a virtual cross-section of subsurface soil and rock layers. It consists of two separate steps: 1) measuring the apparent (weighted average) electrical resistivity of the ground over numerous stations and 2) computerized processing of apparent resistivity data to obtain a virtual cross-section of estimated true resistivity values.

In the field, an electric current is passed into the ground or water by a pair of electrodes and the potential is measured at a second pair of electrodes. Multiple electrodes and a computerized switching system are used to speed data acquisition. A SuperSting/Swift R8® Memory Earth Resistivity Meter, a 28 takeout passive marine cable set, and stainless steel electrodes were used to perform the survey. Advanced Geosciences, Inc., (AGI), of Austin, Texas, manufactured the equipment, which is designed for shallow geotechnical and geological applications and engineered to have a high signal to noise ratio.

For quality assurance/quality control, SEI performs resistivity surveys in compliance with the ASTM Standard Guide for Using the Direct Current Resistivity Method for Subsurface Investigation, designation D 6431-99.

Array Type

Resistivity data were collected using a dipole-dipole array configuration with the extended data coverage option. This array type maximizes lateral resolution and the total number of data points collected on each transect. A dipole-dipole array places two current (transmitting) electrodes together as a pair and two potential (sensing) electrodes together as a pair. For each successive measurement, the potential electrode pair is moved farther away from the current electrode pair by a distance that is a multiple of the distance between the electrodes.

ERI Transects

Resistivity measurements were made along five (5) transects at the site. All transects consisted of a 28 electrode array on a spacing of 20 feet. Transects T1 and T2 were oriented west to east along the south and north sides of the bridge, respectively. These were placed so that a portion of each end of the transects were located above the waterline on dry ground, and passing approximately ten feet north or south of the edges of the bridge deck where submerged.

Transects T3, T4, and T5 were oriented south to north, crossing beneath the middle three sections of the bridge. These transects were completely submerged, and were deployed from a pontoon boat.

The pontoon boat also held the instrumentation for the duration of data collection for each of these transects.

The approximate location of the ERI transects are shown on the attached Figure 1: Site Location Map. Transect locations were measured and placed using a Trimble™ Differential Global Positioning System (DGPS).

Modeling

After the survey was performed, ERI field data was transferred to a computer and converted into data files for modeling. Two-dimensional inverse resistivity modeling was performed using the RES2DINV version 3.57.37 software package. Special modeling routines included for processing of submarine and mixed data sets were utilized in the processing of this data. The modeling method consists of estimating the true resistivity of the subsurface at points arranged in a grid on a vertical plane. The estimated true resistivity values are used to calculate apparent resistivity values, which are compared to the actual measured resistivity values. Adjustments are made in the model to make the calculated resistivity values more closely match the measured values. The modeling progresses toward better estimates of the true resistivity by iteration using the least-squares method. Up to five iterations were performed.

The iteration process was carried out until the convergence between iterations approached 5%. RMS errors less than 10% are considered ideal, but this cannot be obtained in all cases and is dependent upon local soil conditions. Highly resistive surficial soils, or shallow subsurface lithified materials reduce signal propagation and signal strength at depth, contributing to higher RMS error calculations in the model. Significant deviations from a horizontally layered, laterally homogenous model will also significantly increase the apparent RMS error. SEI reduced the error in the model by trimming data points that have high RMS error values using an editing feature of the RES2DINV software. The estimated true resistivity values were contoured to produce a two-dimensional pseudosection for the plane beneath the survey line. A contour interval was chosen to show minor variations in the lower resistivity values while covering the range of typical material values. Topographic corrections were made with respect to observed sea level at the time of the survey, but are not adjusted to match any formal elevation model.

Resistivity values are not necessarily dependent only on the type of soil or rock present, but are strongly influenced by the presence, salinity and pH of pore fluids in the earth materials. Dry clays may have resistivities that are higher than typical and saturated sands may have resistivities that are lower than typical. In particular, saltwater and low pH (acidic) fresh groundwater will greatly reduce the resistivity of non-conductive materials such as sand and limestone. Different materials and conditions may also present similar electrical signatures, such as dense plastic clays and loose saturated granular soils or voids.

Please note that the resistivity-modeling program contours the modeled data points in a manner that may show gradational changes, when in fact, abrupt contacts are present between layers of earth materials. Also, please be aware that actual lithological contacts can be difficult to identify on the ERI pseudosections without test boring data. Interpretations are made in the Results section, by assuming that certain contour intervals represent the contact between different types of materials, as described above.

Prints of the ERI pseudosections are provided on the attached Figure 2, and form the basis for this report. Other details about the survey and modeling are available in SEI's files should you need them in the future.

ERI Survey Results and Discussion

The results of the survey were apparently impacted by the presence of the steel H-piles, resulting in low resistivity anomalies coincident with the location of the submerged steel. However, despite this interference, the two transects that were oriented parallel to the bridge, T1 and T2, indicate low resistivity anomalies of greater extent than likely due to such interference. It is our interpretation that these larger anomalies may represent areas of increased porosity/lower density, or areas where higher resistivity shallow bedrock has been weathered or replaced. This anomaly is delineated on the Site Location Map (Figure 1) and labeled as Feature 1, and should be considered for additional direct investigation by soil boring or similar method.

Additionally, all five pseudosections indicate a transition in resistivities between 20 and 40 feet below sea level, from lower to higher resistivity. This may be indicative of a stratigraphic transition to bedrock, or perhaps from soil and weathered bedrock to competent bedrock.

Recommendations

SEI recommends that the center of the apparent anomaly documented in the ERI survey and identified as Feature 1 be considered for additional direct investigation. Advancing an SPT boring at the deepest part or center of each feature would serve to verify the inferred possible karst conditions. If the results of these test borings indicate anomalous conditions indicative of karst activity, SEI may be able to identify further appropriate locations for additional investigation after correlation of boring log data and ERI survey results. SEI would be pleased to assist you with further correlation and interpretation of this ERI survey and the findings from the drilling conducted as part of the initial soil boring investigation.

-oOo-

Closing Comments

We appreciate the opportunity of providing these geophysical services to you on this project. Should you have any questions or require additional information, please do not hesitate to contact our office at (813) 353.9083.

Sincerely,

SUBSURFACE EVALUATIONS, INC.

NO. PG2374

Andrew Glasbrenner, P.G.
Licensed Professional Geologist, No. 2374 (Florida)
Senior Geologist
April 28th, 2009

Attachments: Attachment A – Limitations,
Figures 1 through 2

File: X:\2009\Williams Earth Sciences\Beckett Bridge\Beckett Bridge ERI Report.doc

Map is shown in State Plane Florida West 902 NAD 1983 Coordinate System (Feet)

Legend

→ ERI Transect

○ Feature

0 100 200 Feet

Figure 1: Site Location Map

Project: Beckett Bridge
Riverside Drive
Tarpon Springs, Florida

Client: Williams Earth Sciences, Inc.

Date: April 21-22, 2009

Created By: JRW

SUBSURFACE EVALUATIONS, INC.
Engineering Geology & Geophysics
8010 Woodland Center Blvd., Suite 100 Tampa, FL 33614
800-508-2509 (813) 353-9083 Fax: (813) 353-9653
www.SubsurfaceEvaluations.com

Fig 2: Electrical Resistivity Imaging Pseudosections T1-T5 Beckett Bridge, Tarpon Springs, Florida