U.S. Department of Transportation
Federal Highway Administration

Brent Spence Bridge
Replacement/Rehabilitation Project

Bridge Type Selection Report

ODOT PID No. 75119
HAM-71/75-0.00/0.22
KYTC Project Item No. 6-17

March 2011








Prepared by:
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In Association with:
Rosales + Partners
HDR Engineering, Inc.
H.C. Nutting, a Terracon Company
Rowan Williams Davies & Irwin, Inc.

U.S. Department of Transportation
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Brent Spence Bridge
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KENTUCKY TRANSPORTATION CABINET

Bridge Type Selection Report Executive Summary

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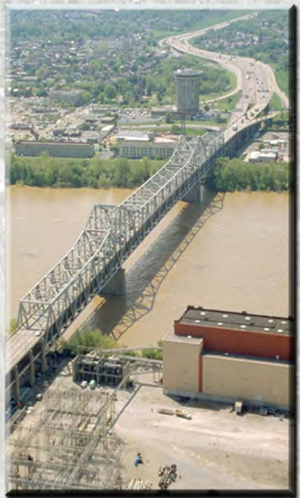

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Executive Summary

A Bridge Type Selection Process (BTSP) was completed as part of the Brent Spence Bridge Replacement/ Rehabilitation Project to assist the Kentucky Transportation Cabinet (KYTC) and the Ohio Department of Transportation (ODOT) in selecting one bridge alternative to be constructed across the Ohio River. The recommended Final 3 Bridge Alternatives presented in this document are the result of the project's functional and budgetary requirements, as well as the public feedback received during the course of the BTSP.

The proposed bridge will span the Ohio River downstream (west) of the current Brent Spence Bridge which facilitates interstate and local travel by providing access to Covington, Kenton County, Kentucky and downtown Cincinnati, Hamilton County, Ohio. The Brent Spence Bridge, which opened to traffic in 1963, was designed to carry 80,000 vehicles per day. Currently, approximately 160,000 vehicles per day use the Brent Spence Bridge and traffic volumes are projected to increase to approximately 233,000 vehicles per day in 2035. Safety, congestion and geometric problems exist on the structure and its approaches.

Within this context, the new bridge must meet several requirements:

- Minimize its impact on local historic structures and local infrastructure;
- Work in conjunction with the existing Brent Spence Bridge;
- Fit into the construction schedule and budget of the larger project to increase capacity on I-75;
- Require minimal maintenance and maximum durability;
- Have no permanent effect on river navigation;
- Integrate itself in the landscape of the riverfront;
- Provide an improved crossing experience for drivers; and
- Conform to current design standards.

About The Project

The I-75 corridor within the Greater Cincinnati/Northern Kentucky region is experiencing problems which threaten the overall efficiency and flexibility of this vital trade corridor. Areas of concern include, but are not limited to, growing demand and congestion, land use pressures, environmental concerns, adequate safety margins and maintaining linkage in key mobility, trade, and national defense highways.

The Brent Spence Bridge Replacement/Rehabilitation Project is intended to improve the operational characteristics within the corridor for both local and through traffic. In the Greater Cincinnati/Northern Kentucky region, the interstate corridor suffers from congestion and safety–related issues as a result of inadequate capacity to accommodate current traffic demand. The objectives of this project are to:

- Improve traffic flow and level of service;
- Improve safety;
- Correct geometric deficiencies; and
- Maintain connections to key regional and national transportation corridors.

The Brent Spence Project/Bridge Design Team

The Brent Spence Replacement/Rehabilitation Project is directed by the KYTC and ODOT, along with the Federal Highway Administration (FHWA). Led by Parsons Brinckerhoff, the project design team includes a number of technical specialists required to provide all of the necessary professional services for the Brent Spence Replacement/Rehabilitation Project. Within the project design team, a bridge design team including KYTC, ODOT, and FHWA, was utilized for the BTSP.

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Advisory and Aesthetic Committees

At the outset of the project, KYTC and ODOT instituted two committees to provide input and guidance to the project design team. The Advisory Committee (AC) provides input from local community and political leaders on community issues and concerns. This provides an opportunity for important issues brought up to the AC to be communicated to the project design team, and how these issues were subsequently addressed reported back to the organizations represented by the members of the AC.

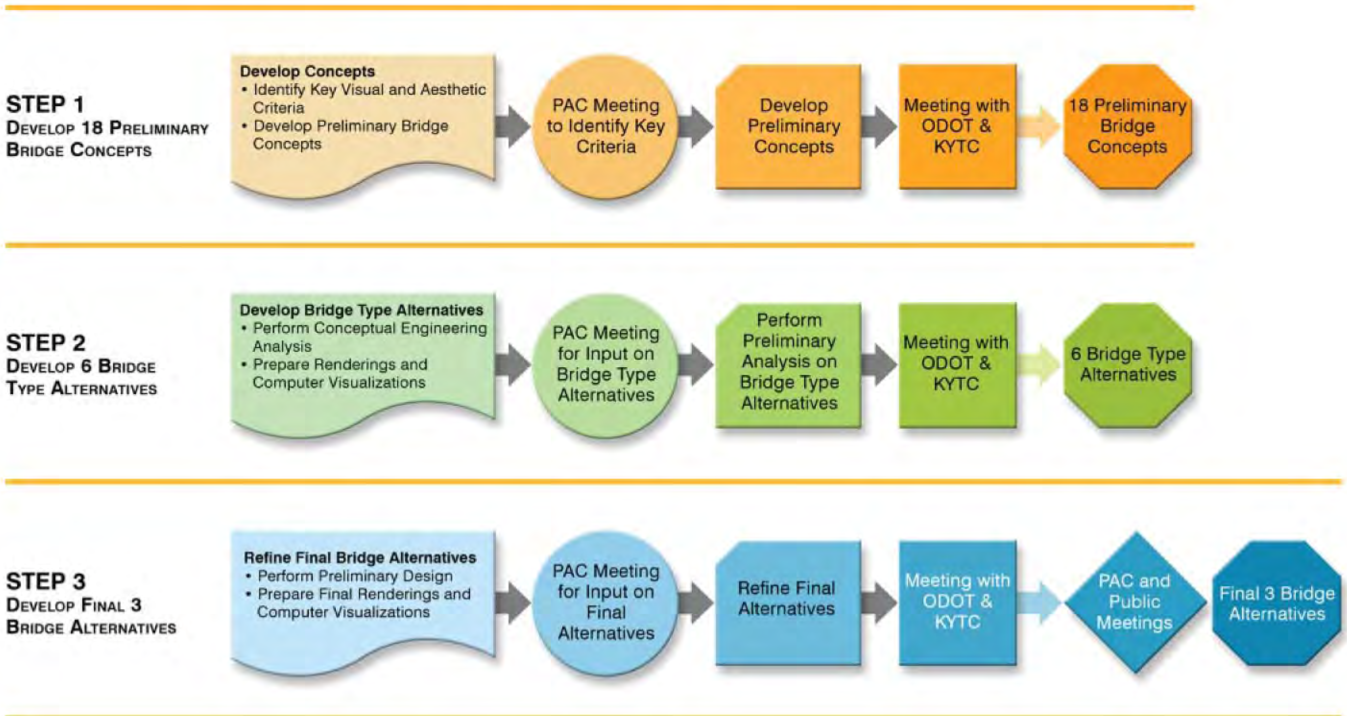
The Project Aesthetics Committee (PAC) is a sub-committee of the AC, and provides local input on the design and aesthetic appearance of the corridor, the main span of the new Ohio River Bridge, and the rehabilitated Brent Spence Bridge structure. The PAC is comprised of citizen and agency representatives from Kentucky and Ohio. This committee works with the project design team to develop context sensitive design solutions for the project.

Bridge Type Selection Process

As part of the Brent Spence Bridge Replacement/Rehabilitation Project, the BTSP was initiated in 2009. The BTSP is a three step process, which involves developing and analyzing numerous bridge concepts leading to a recommendation of the Final 3 Bridge Alternatives. This process will culminate in the selection of a new bridge that will be designed and built across the Ohio River just downstream (west) of the existing Brent Spence Bridge.

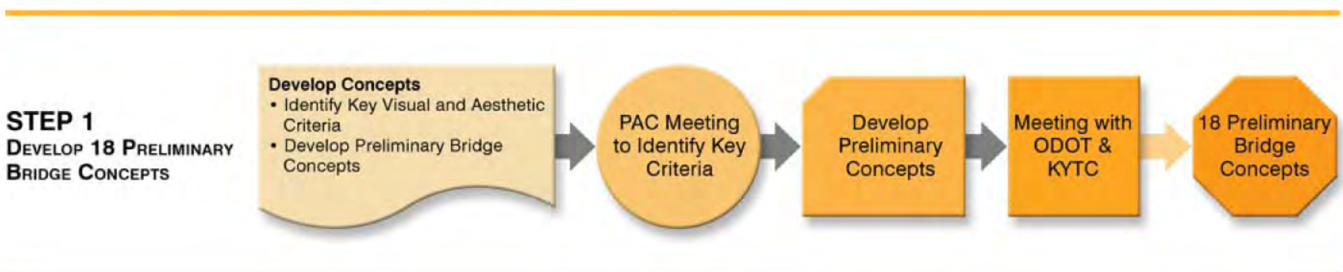
As described below, the BTSP is collaborative in nature and based on public input and engineering requirements. Public involvement was used throughout the three steps of the BTSP. The role of public involvement was to help create and provide avenues for local citizens, stakeholders, and officials to ask questions and offer their comments and suggestions. This feedback will ultimately be used in determining a final bridge type that would reflect, as much as possible, the needs and desires of the community.

The following BTSP flowchart presents the elements of the three steps:



STEP 1:

Develop 18 Preliminary Bridge Concepts



The Objective

The objective of Step 1 was to:

- Identify key visual and aesthetic criteria to be used as part of the BTSP;
- Obtain US Coast Guard design requirements for the new bridge; and
- Develop approximately 18 Preliminary Bridge Concepts.

The Process

Prior to meeting with the PAC, coordination with the US Coast Guard was conducted to determine their design requirements for the new bridge. Following verification by the US Coast Guard of the bridge clearance and pier locations, the bridge design team met with the PAC on September 25, 2009 to identify the key visual and aesthetic criteria. These visual and aesthetic criteria were then used to develop and refine the Preliminary Bridge Concepts, reflecting feasible bridge types and using engineering solutions that best reflected the characteristics identified by the PAC.

In coordination with the Federal Highway Administration (FHWA), KYTC, and ODOT, key design criteria and guidelines were developed as evaluation methodology to be used to evaluate the preliminary bridge concepts. The key design criteria developed to be used during each step were:

- Construction Cost;
- Constructability;
- Maintenance and Durability; and
- Major Rehabilitation Feasibility.

Evaluation guidelines were also developed as part of the overall project. Some of the guidelines reflected navigational, structural and highway limitations, and physical restrictions that exist at the bridge site. Other guidelines represented environmental commitments and financial constraints necessary to meet budgetary goals. The key design criteria, key visual and aesthetic criteria, and evaluation guidelines were used to select and to develop the Preliminary Bridge Concepts.

Public involvement in Step 1 included input provided at the PAC Meeting on September 25, 2009. The purpose of this meeting was to provide a project status report, present context of aesthetics in the project study area, and develop key visual and aesthetic criteria for the project. Prior to the BTSP, a Brent Spence Bridge Project Aesthetic Committee Charter was developed to define the role of the PAC. The general role of the PAC is to provide aesthetic guidelines and recommendations for the project corridor and to provide input on aesthetic treatments of bridge structure types.

During the PAC meeting, committee members provided their input on key visual and aesthetic criteria for the overall project and the new bridge. The key visual and aesthetic criteria identified by the PAC for assisting with selecting a bridge type included the following:

- The new bridge should be visually attractive;
- The new bridge should be visible looking “through” the existing bridge (from the east);
- As much as possible, crossing the new bridge should allow views of the surrounding context (unlike existing bridge);
- The new bridge should have distinctive characteristics that identify it as a local landmark; and
- The new bridge should have a visual relationship with the existing bridge.

Additional aesthetic criteria identified by the PAC were:

- The new bridge colors/textures/landscaping, etc. should be aesthetically pleasing; and
- The existing bridge should be maintained/repainted to blend in with the new bridge.

As a result of the September 25, 2009 PAC meeting, 24 preliminary bridge concepts were developed and evaluated during Step 1. During the evaluation process, 12 preliminary bridge concepts reflecting feasible bridge types and using engineering solutions that best reflected the characteristics identified by the PAC were reviewed and approved by FHWA, KYTC, and ODOT as best meeting the objectives of Step 1.

The Outcome

The results of Step 1 consisted of:

- Design Parameters (Mandatory Requirements);
- Design Guidelines (Desirable Objectives); and
- 12 Preliminary Bridge Concepts.

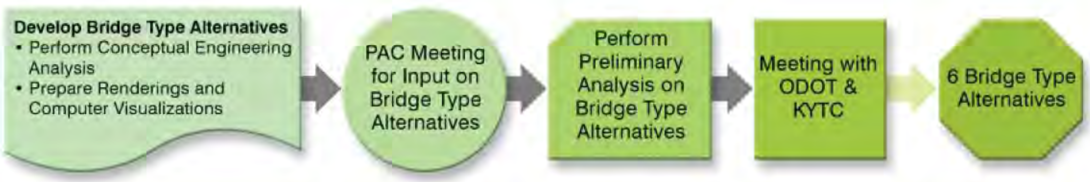
The bridge types that were recommended for further consideration in Step 2 included Through Truss, Through Arch, and Cable Stayed bridges. In all cases, the bridge concepts included a double-decked roadway with the top and bottom deck connected by trusses.

Existing Brent Spence Bridge

STEP 2:

Develop 6 Bridge Type Alternatives

STEP 2
Develop 6 Bridge Type Alternatives



The Objective

The objective of Step 2 was to:

- Present the preliminary bridge concepts approved during Step 1 to the PAC and public to gain feedback to help select the concepts to be recommended as the 6 Bridge Type Alternatives for further development in Step 2;
- Perform conceptual engineering analysis on the 6 Bridge Type Alternatives;
- Prepare renderings and computer visualizations of the 6 Bridge Type Alternatives; and
- Prepare cost estimates for the 6 Bridge Type Alternatives.

The Process

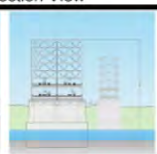
At the beginning of Step 2, the 12 preliminary bridge concepts were presented to a combined meeting of the AC and PAC on January 29, 2010. During this combined AC/PAC meeting, the bridge design team presented the 12 preliminary bridge concepts consisting of two truss bridge, three arch bridges and seven cable-stayed bridges. The bridge design team then solicited feedback from the two committees as to which concepts best met the five key visual and aesthetic criteria. During the meeting, the bridge design team presented various bridge components incorporated into the 12 preliminary bridge concepts and requested additional feedback on them to aid in the Step 2 bridge type selection process. The 12 preliminary bridge concepts were also posted on the project website to solicit public comment as well. Following the AC/PAC meeting, the public was provided a one-week comment period to submit feedback regarding the aesthetic elements of the new Ohio River Bridge. Comments were received through emails, the project website, faxes, and phone calls.

The 12 Preliminary Bridge Concepts presented were:

Truss Concepts

Concept | Elevation View | Section View | View from Upstream | Aerial Perspective

1

Construction Cost		Constructability		Maintenance and Durability	Major Rehabilitation Feasibility	
Truss Support Type		Truss Inclination		Top Bracing		
Cantilever	Simply Supported	Inclined Trussess	**Vertical Trussess**	Strut	**K-Brace**	Cross Braced

2




Construction Cost		Constructability		Maintenance and Durability	Major Rehabilitation Feasibility	
Truss Support Type		Truss Inclination		Top Bracing		
Cantilever	**Simply Supported**	Inclined Trussess	**Vertical Trussess**	Strut	**K-Brace**	

Arch Concepts

Concept | Elevation View | Section View | View from Upstream | Aerial Perspective

3




Construction Cost		Constructability		Maintenance and Durability		Major Rehabilitation Feasibility	
Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	**Vertical Leg**	Strut	K-Brace	Cross Braced	**Lattice**	**Shallower**	Deeper
Hanger Arrangement			Deck Truss Type				
Vertical	Inclined	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

4




Construction Cost		Constructability		Maintenance and Durability		Major Rehabilitation Feasibility	
Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	Vertical Leg	**Strut**	K-Brace	Cross Braced	Lattice	Shallower	**Deeper**
Hanger Arrangement			Deck Truss Type				
Vertical	Inclined	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

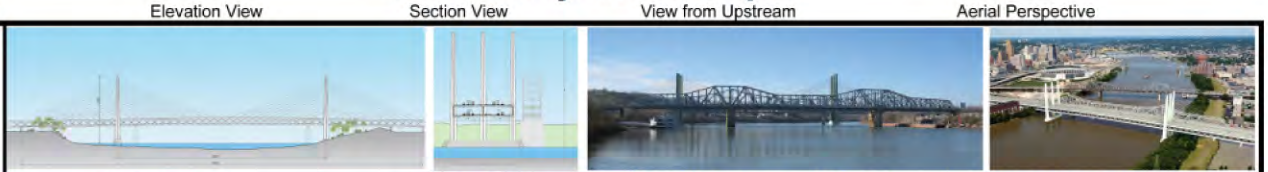
5




Construction Cost		Constructability		Maintenance and Durability		Major Rehabilitation Feasibility	
Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	Vertical Leg	Strut	**K-Brace**	Cross Braced	Lattice	Shallower	**Deeper**
Hanger Arrangement			Deck Truss Type				
Vertical	**Inclined**	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

Legend		
Feasible	Feasible with Constraints	Not Feasible

Cable Stayed Concepts

Concept	Elevation View	Section View	View from Upstream	Aerial Perspective
6				
7				
8				
9				
10				

Concept 6

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan

Deck Truss Type			
Warren	Warren	**Lattice**	Pratt

Concept 7

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan

Deck Truss Type			
Warren	Warren	Lattice	**Pratt**

Concept 8

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	**Vertical Leg 2 Leg Option**	Vertical Leg 3 Leg Option	Harp	Fan	**Semi Fan**

Deck Truss Type			
Warren	Warren	Lattice	Pratt

Concept 9

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	Harp	Fan	**Semi Fan**

Deck Truss Type			
Warren	**Warren**	Lattice	Pratt

Concept 10

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan

Deck Truss Type			
Warren	Warren	Lattice	**Pratt**

Legend		
Feasible	Feasible with Constraints	Not Feasible

Cable Stayed Concepts, Continued

Concept	Elevation View	Section View	View from Upstream	Aerial Perspective
11				
	Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility
	Tower Shape & Number of Legs			
	Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	Vertical Leg 3 Leg Option	
	Stay Cable Arrangement			
	Harp	Fan	**Semi Fan**	
	Deck Truss Type			
	Warren	**Warren**	Lattice	Pratt
12				
	Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility
	Tower Shape & Number of Legs			
	Inclined Leg/ Arch Tower	**Vertical Leg 2 Leg Option**	Vertical Leg 3 Leg Option	
	Stay Cable Arrangement			
	Harp	Fan	Semi Fan	
	Deck Truss Type			
	Warren	**Warren**	Lattice	Pratt

Legend		
Feasible	Feasible with Constraints	Not Feasible

Based on the results of the January 29th AC/PAC meeting and the public comments received, 6 preliminary bridge concepts were selected and approved by FHWA, KYTC, and ODOT to move forward during Step 2. The recommended 6 Bridge Type Alternatives for further development during Step 2 were:

1. **Arch Bridge**
   - Simply supported arch
   - Inclined arch ribs
   (Concept 4)

2. **Arch Bridge**
   (New Concept)
   - Continuous arch
   - Vertical arch ribs

3. **Cable Stayed Bridge**
   two towers, three vertical legs/tower
   - Various stay cable arrangements (developed from Concepts 6 and 7)

4. **Cable Stayed Bridge**
   two towers, three inclined legs/tower
   - Harp stay cable arrangement (Concept 10)

5. **Cable Stayed Bridge**
   two towers, two inclined legs/tower
   - Various stay cable arrangements (developed from Concept 9)

6. **Cable Stayed Bridge**
   one tower, two vertical legs/tower
   - Harp stay cable arrangement (Concept 12)

The truss bridges were consistently unpopular and were eliminated from further consideration.

Through a series of design meetings with the FHWA, KYTC, and ODOT during Step 2, the 6 Bridge Type Alternatives were further refined for conformance to the design parameters and best meeting the design guidelines of the project. As a result of the conceptual engineering analysis, each of the 6 Bridge Type Alternatives were evaluated based on construction cost, constructability/construction time, maintenance and durability, major rehabilitation feasibility, and maintenance of traffic. Renderings and computer visualizations showing different views and details were developed for each of the 6 Bridge Type Alternatives. At the end of Step 2, the 6 Bridge Type Alternatives were reviewed and approved by FHWA, KYTC, and ODOT as best meeting the objectives of Step 2.

The Outcome

The primary results from Step 2 included the analysis of the 6 Bridge Type Alternatives and an updated evaluation matrix.

STEP 3:

Develop Final 3 Bridge Alternatives

STEP 3
Develop Final 3 Bridge Alternatives

The Objective

The objective of Step 3 was to:

- Present the 6 Bridge Type Alternatives approved during Step 2 to the PAC and public to gain feedback to support selection of the bridge type alternatives recommended as the Final 3 Bridge Alternatives for preliminary design in Step 3;
- Perform significant preliminary design on the Final 3 Bridge Alternatives;
- Revise and develop additional renderings and computer visualizations of the Final 3 Bridge Alternatives, including animations;
- Prepare cost estimates for the Final 3 Bridge Alternatives;
- Present the Final 3 Bridge Alternatives at two public meetings; and
- Complete the Bridge Type Selection Report.

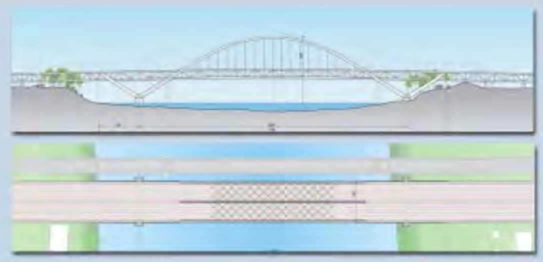
The Process

Step 3 involved one combined meeting of the AC and PAC on April 15, 2010 and an AC meeting on December 17, 2010. The 6 Bridge Type Alternatives were presented to a combined meeting of the AC and PAC on April 15, 2010. The purpose of the meeting was to receive feedback on the 6 Bridge Type Alternatives to aid the project design team in selecting the Final 3 Bridge Alternatives. Key visual and aesthetic criteria previously established were used by the AC and PAC to evaluate the 6 Bridge Type Alternatives.

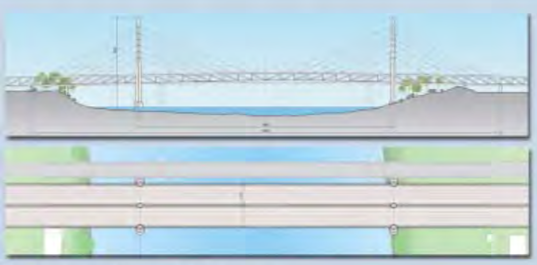
The 6 Bridge Type Alternatives were also posted on the project website to solicit public comment. Following the AC/PAC meeting, the public was provided a one-week comment period to submit feedback. Comments received indicated that the public is in favor of both the arch type bridges as well as the cable stayed bridge types, with no clear preference for either.

The 6 Bridge Type Alternatives presented were:

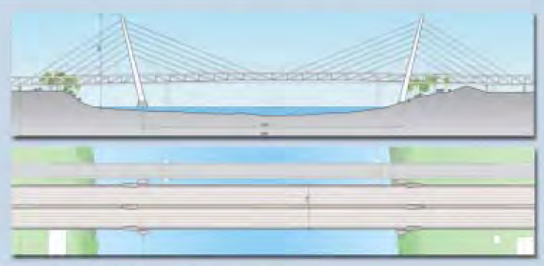
1 **Arch Bridge**
Simply supported arch with inclined arch ribs



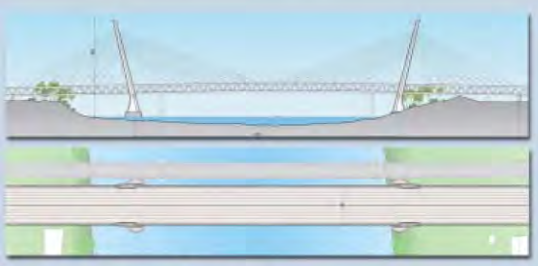
2 **Arch Bridge**
Continuous arch with vertical arch ribs



3 **Cable Stayed Bridge**
Two towers, three vertical legs/tower



4 **Cable Stayed Bridge**
Two towers, three inclined legs/tower



5 **Cable Stayed Bridge**
Two towers, two inclined legs/tower

6 **Cable Stayed Bridge**
One tower, two vertical legs/tower

Based upon the results of the AC/PAC meeting and public outreach efforts, the following Final 3 Bridge Alternatives were selected and approved by FHWA, KYTC and ODOT for further study. Additional technical analysis for the Final 3 Bridge Alternatives was also presented to the AC on December 17, 2010. To date, the bridge design team has not received additional comments from the AC.

During this step, the project bridge design team assessed the suitability of the Final 3 Bridge Alternatives based on more detailed examination of the structural requirements, cost, constructability, environmental impacts, aesthetics, and other key criteria. This task included performing significant preliminary design and preparing additional renderings for the Final 3 Bridge Alternatives.

While each of the Final 3 Bridge Alternatives has distinct characteristics, there are some elements common to all. The following is a list of these common elements:

- A bridge alignment adjacent to and downstream (west) of the existing Brent Spence Bridge;
- A double-decked truss superstructure carrying two roadways on each deck, with each roadway composed of two or three 12-foot-wide lanes and two 14 foot-wide shoulders;
- An approximately 1,000-foot main span with piers outside of the main span piers of the existing Brent Spence Bridge;
- A river to superstructure clearance no lower than that of the existing Brent Spence Bridge, and
- A bridge to work in conjunction with the existing Brent Spence Bridge, to carry the Design Year 2035 traffic projection of approximately 233,000 vehicles per day.

Alternative 1: Tied Arch Bridge

Alternative 3: Two Tower Cable Stayed Bridge

Alternative 6: One Tower Cable Stayed Bridge

The estimated construction cost for the Final 3 Alternatives are:

Alternative		Main Bridge ($M)	Approaches ($M)	Total ($M)
Alternative 1 Tied Arch		$358.3	$212.4	$570.7
Alternative 3 Two Tower Cable Stayed		$632.3	$36.3	$668.6
Alternative 6 Single Tower Cable Stayed		$561.0	$85.3	$646.3

The Outcome

The technical analyses for the Final 3 Bridge Alternatives were presented to the AC on December 17, 2010. To date, the bridge design team has not received additional comments from the AC.

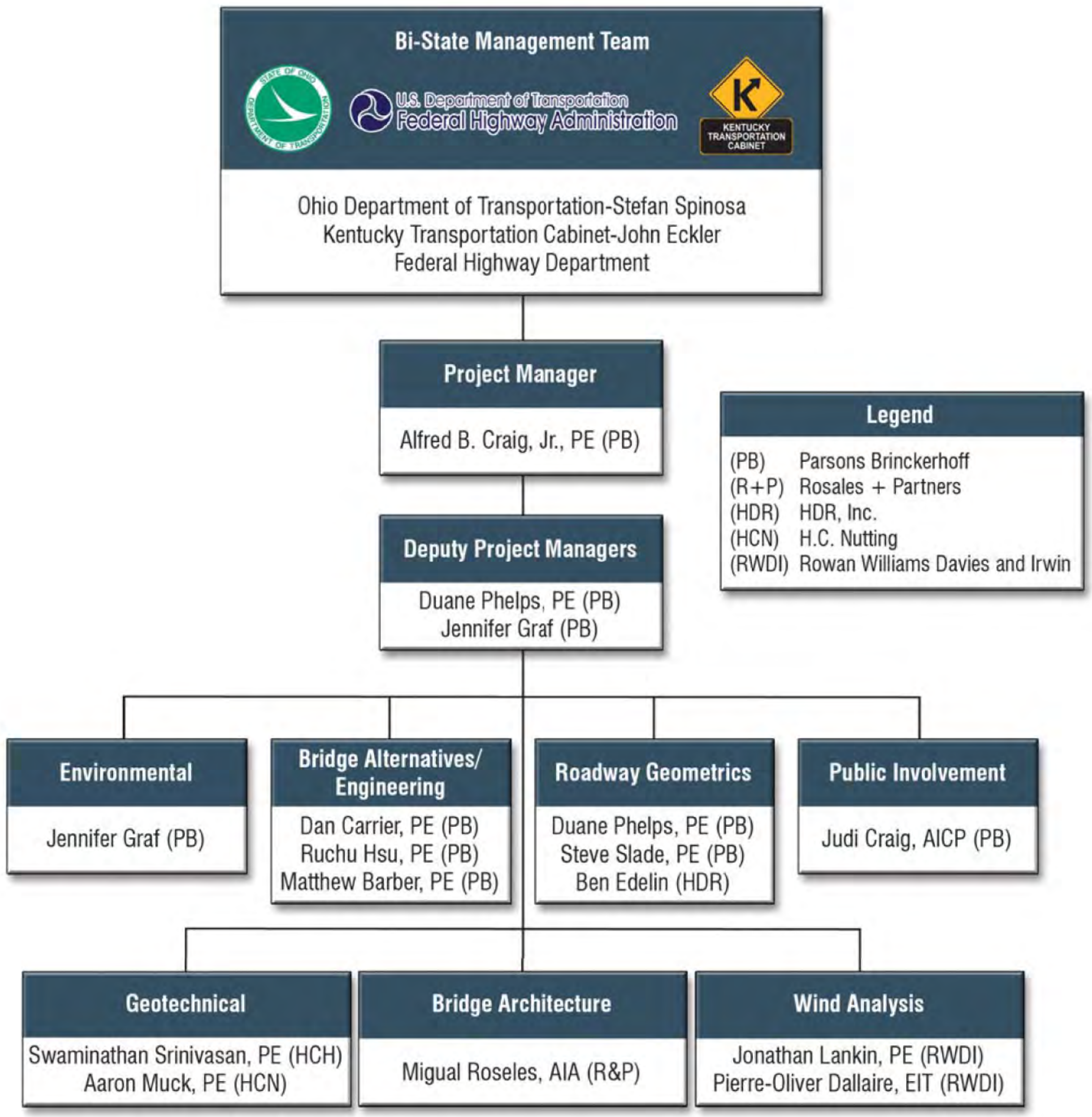
Bridge Type Study – Next Steps

As part of the National Environmental Policy Act (NEPA), public hearings for the Brent Spence Bridge Replacement/Rehabilitation Project will be held in 2011. The focus of the hearings will be the selection of the recommended Preferred Alternative for the highway and the new bridge type crossing the Ohio River. The purpose of the hearings is to provide the public the opportunity to comment on the project, its impacts, and proposed mitigation strategies.

During public hearings, the public will have the opportunity to vote on components of the three bridge alternatives using a hand-held audience response polling system.

In addition, a comment period of at least 14 days will follow the public hearings. Following the public comment period, the selection of a new Ohio River Bridge type will be determined by KYTC and ODOT in consultation with FHWA. The selection of the preferred bridge type will be based upon consideration of several factors including the technical analyses completed for the project and public input.

The Brent Spence Bridge Replacement/Rehabilitation Project Bridge Design Team



NOTES

2.0 Project Overview
The recommended bridge alternatives were developed using the Bridge Type Selection Process (BTSP) described in Section 2.3. This report documents the BTSP and is organized into the following chapters.

- Chapter 1 - Executive Summary
- Chapter 2 - Project Overview
- Chapter 3 - Recommended Bridge Alternatives
- Chapter 4 - Public Involvement
- Chapter 5 - Environmental Commitments
- Chapter 6 - Development of Bridge Alternatives

This Chapter presents an overview of the Brent Spence Bridge Replacement/Rehabilitation Project and the BTSP.

2.1 Introduction
Interstate 75 (I-75) within the Greater Cincinnati/Northern Kentucky region is a major thoroughfare for local and regional mobility. Locally, it connects to I-71, I-74 and US Route 50. The Brent Spence Bridge provides an interstate connection over the Ohio River and carries both I-71 and I-75 traffic (Exhibit 2-1). The bridge also facilitates local travel by providing access to downtown Cincinnati, Ohio and Covington, Kentucky. Safety, congestion and geometric problems exist on the structure and its approaches. The Brent Spence Bridge, which opened to traffic in 1963, was designed to carry 80,000 vehicles per day. Currently, approximately 160,000 vehicles per day use the Brent Spence Bridge and traffic volumes are projected to increase to approximately 233,000 vehicles per day in 2035.

The I-75 corridor within the Greater Cincinnati/Northern Kentucky region is experiencing problems which threaten the overall efficiency and flexibility of this vital trade corridor. Areas of concern include, but are not limited to growing demand and congestion, land use pressures, environmental concerns, adequate safety margins, and maintaining linkage in key mobility, trade, and national defense highways.

To address these critical transportation needs, the purpose of the Brent Spence Bridge Replacement/Rehabilitation Project is to:

- Improve traffic flow and level of service;
- Improve safety;
- Correct geometric deficiencies; and
- Maintain connections to key regional and national transportation corridors.

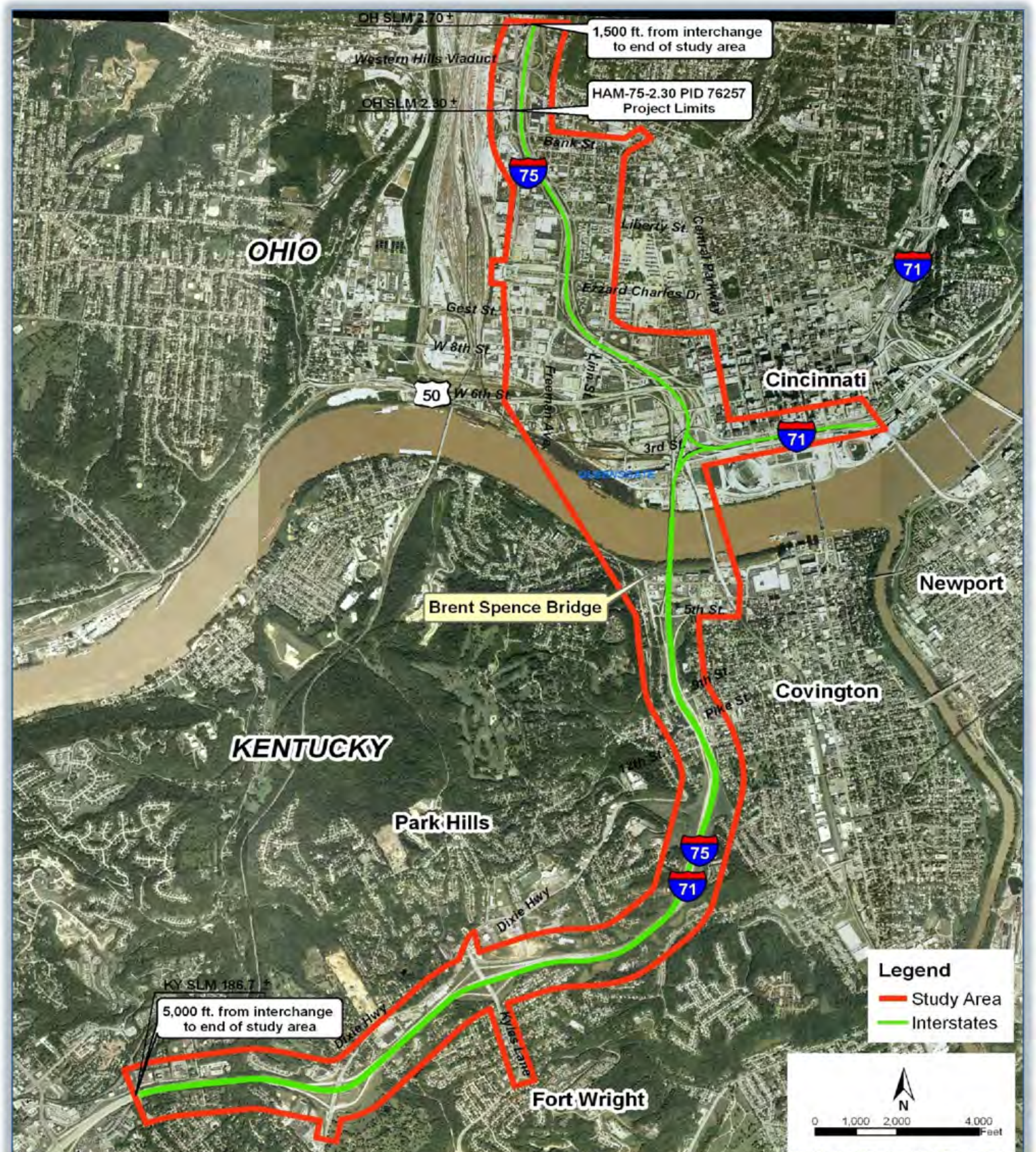
Existing Brent Spence Bridge in Forefront

2.2 Site Context
The project corridor includes portions of Covington, Kentucky, the Ohio River, and Cincinnati, Ohio. The corridor context varies from suburban in the southern portion of the study area to urban near the Ohio River and northward into Ohio. Land uses in Kentucky include single-family residential, multi-family residential, commercial development, maintained grass areas, and institutional uses. In Ohio, land uses include commercial, single-family residential, multi-family residential, industrial, commercial-residential, commercial-industrial, and undeveloped areas along the Ohio River.

The Ohio River is the most prominent natural feature of the project corridor. Other notable features within the study area include the following attractions and landmarks:

- Downtown Cincinnati – Central Business District;
- Downtown Covington – Central Business District;
- Paul Brown Stadium – Home of the Cincinnati Bengals;
- The Banks – Cincinnati Riverfront Redevelopment area;
- National Underground Railroad Freedom Center – Museum;
- Great American Ball Park – Home of the Cincinnati Reds;
- Duke Energy Station – Electrical Substation;
- Longworth Hall – A National Register of Historic Places listed building; and
- Cincinnati Museum Center at Union Terminal – Museum.

Exhibit 2-1. Project Study Area Map



The topography in the study area ranges from steep hillsides to nearly level and is characterized by a severely to moderately undulating terrain. Near the Ohio River, the terrain has a gentle topography in Kentucky and then transitions into a steep hillside to the west of the I-71/I-75 corridor.

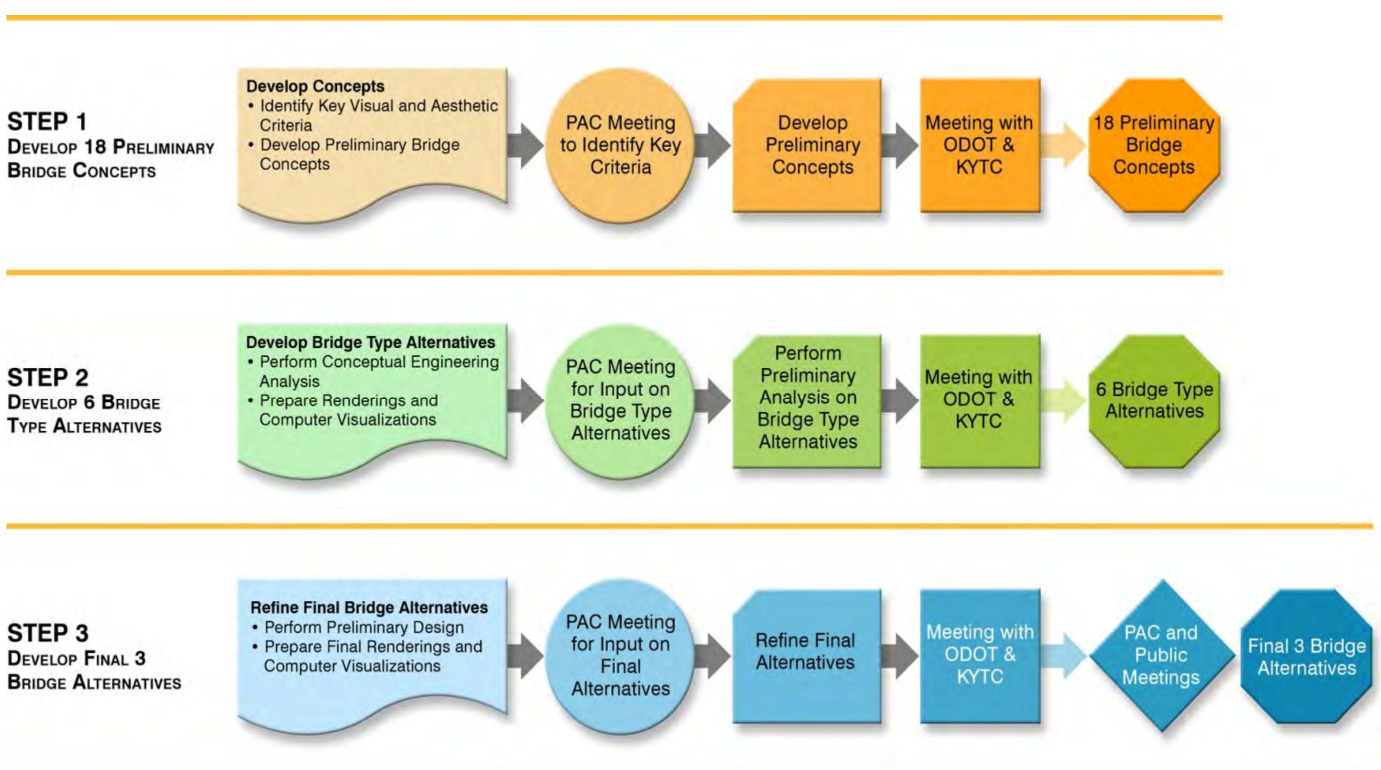
Due to the changing topography, the Brent Spence Bridge is visible from a distance, and is one of eight bridges that cross the Ohio River in this area (Exhibit 2-2). The various bridge types serve pedestrians, vehicles, and railroad traffic. The Cincinnati Southern Bridge, located west of the existing Brent Spence Bridge, carries railroad traffic. Directly to the east of the existing Brent Spence Bridge are the C&O Railroad Bridge, which carries railroad traffic, and the Clay Wade Bailey Bridge, which carries local traffic. Further to the east is the John A. Roebling Suspension Bridge which provides a local connection between Covington, Kentucky and Cincinnati, Ohio. Beyond the Roebling Suspension Bridge are the Taylor Southgate Bridge, the Newport Southbank “Purple People Bridge” (pedestrian bridge), and the Daniel Carter Beard “Big Mac” Bridge that carries I-471 traffic. The Clay Wade Bailey Bridge, John A. Roebling Suspension Bridge, and the Taylor Southgate Bridge all carry both local and pedestrian traffic.

Exhibit 2-2. River Zone Site Context

2.3 The Bridge Type Selection Process

The BTSP is collaborative in nature and based on public input and engineering details. The process began in 2009 and includes three steps:

- Step 1 - Develop 18 Preliminary Bridge Concepts;
- Step 2 - Develop 6 Bridge Type Alternatives; and
- Step 3 - Develop Final 3 Bridge Alternatives.



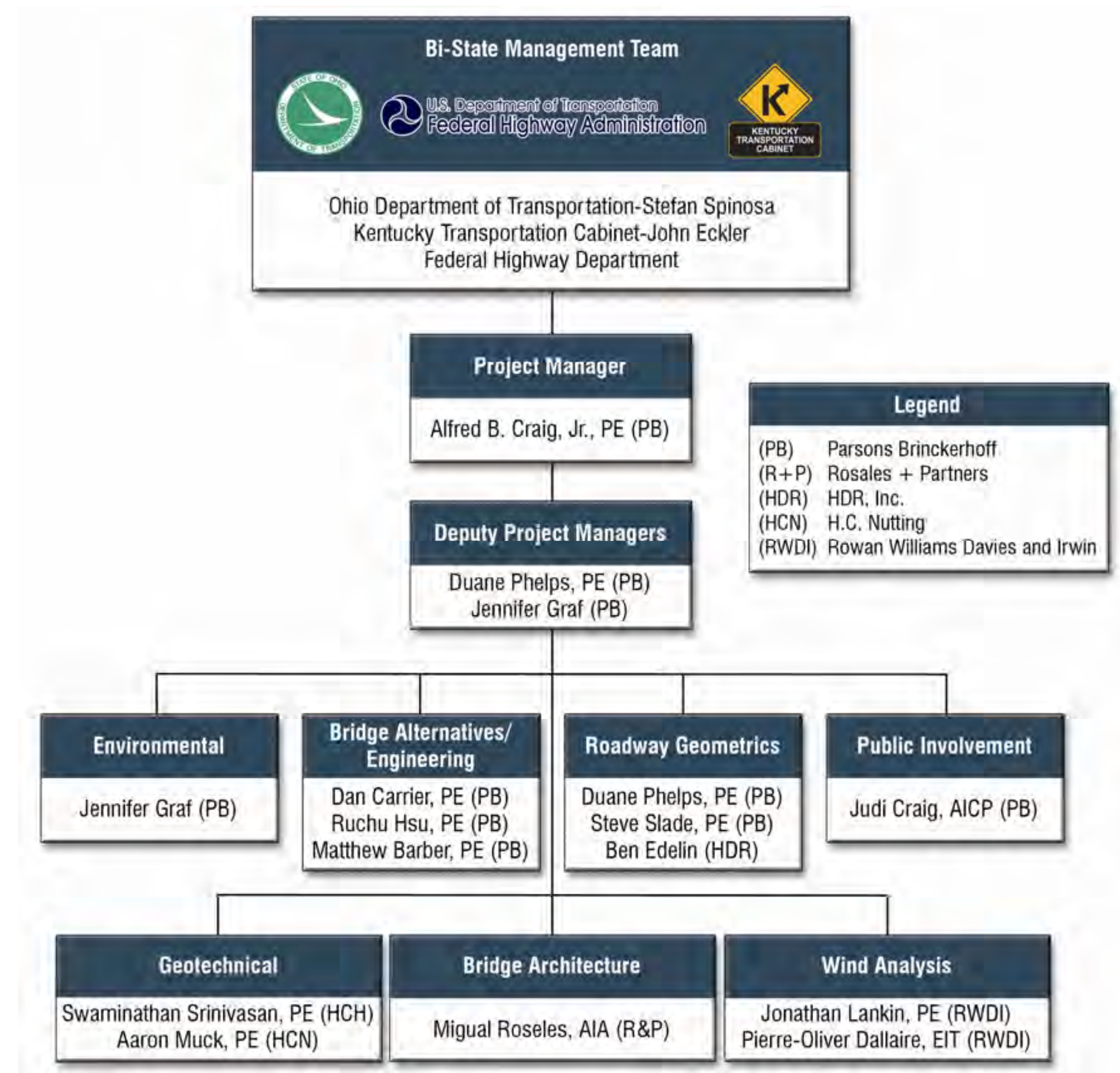
2.4 Advisory and Aesthetic Committees

At the outset of the project, KYTC and ODOT instituted two committees to help provide guidance to the project design team. The Advisory Committee (AC) provides input from local community and political leaders on community issues and concerns. This provides an opportunity for important issues brought up to the AC to be communicated to the project design team, and how these issues were subsequently addressed reported back to the organizations represented by the members of the AC.

The Project Aesthetics Committee (PAC) is a sub-committee of the AC, and provides local input on the design and aesthetic appearance of the corridor, the main span of the new Ohio River Bridge, and the rehabilitated Brent Spence Bridge structure. The PAC is comprised of citizen and agency representatives from Kentucky and Ohio to collaborate with the project design team to develop context sensitive design solutions for the project.

2.5 The Brent Spence Project/Bridge Design Team

The Brent Spence Replacement/Rehabilitation Project is directed by the Kentucky Transportation Cabinet (KYTC) and the Ohio Department of Transportation (ODOT), along with the Federal Highway Administration (FHWA). Led by Parsons Brinckerhoff, the project design team includes a number of technical specialists required to provide all of the necessary professional services for the Brent Spence Replacement/Rehabilitation Project. Within the project design team, a bridge design team including KYTC, ODOT, and FHWA, was utilized for the BTSP. The following is an organizational chart of the bridge design team.



2.6 Contacts

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3.0 Recommended Bridge Alternatives

This chapter compares and contrasts the recommended Final 3 Bridge Alternatives and includes visual renderings and a discussion of structural analysis, construction cost, constructability, and maintenance for each.

3.1 Introduction

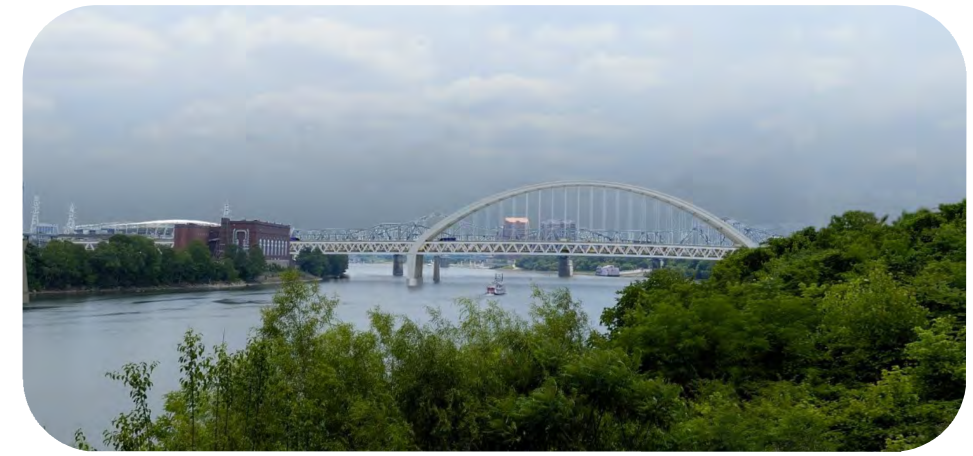
The Bridge Type Selection Process concludes with the following recommended Final 3 Bridge Alternatives for the Brent Spence Bridge Replacement/Rehabilitation Project:

- Alternative 1 – Tied Arch;
- Alternative 3 – Two Tower Cable Stayed (3-Needle-Tower); and
- Alternative 6 – Single Tower Cable Stayed (2-Needle-Tower).

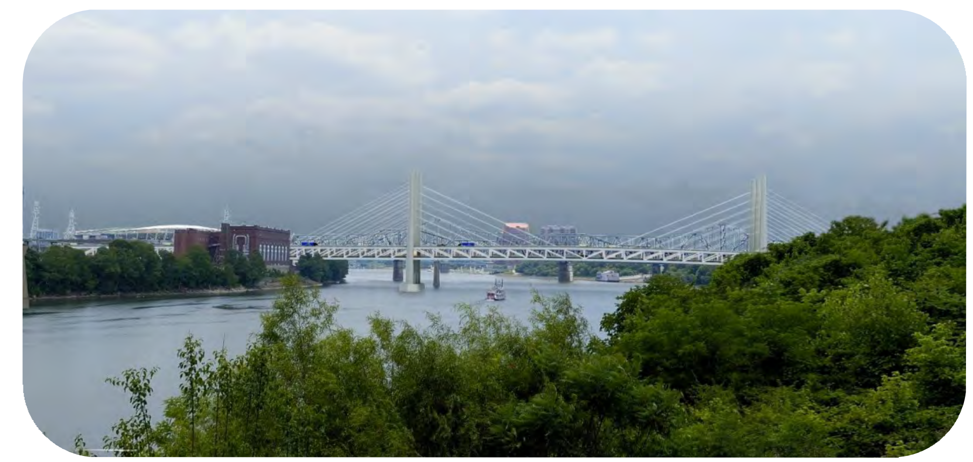
These Final 3 Bridge Alternatives were developed from over 18 Preliminary Bridge Concepts, of which 12 were presented to the Project Advisory Committee (PAC) and the public. In turn, these 12 concepts were narrowed to 6 Bridge Type Alternatives. Additional structural designs and cost estimates were done for each alternative, and these Final 3 Bridge Alternatives were selected for further development. At each step of the process, various bridge elements were examined to assist in the development of the subsequent alternatives.

While each of the Final 3 Bridge Alternatives has distinct characteristics, there are some elements common to all. The following is a list of these common elements:

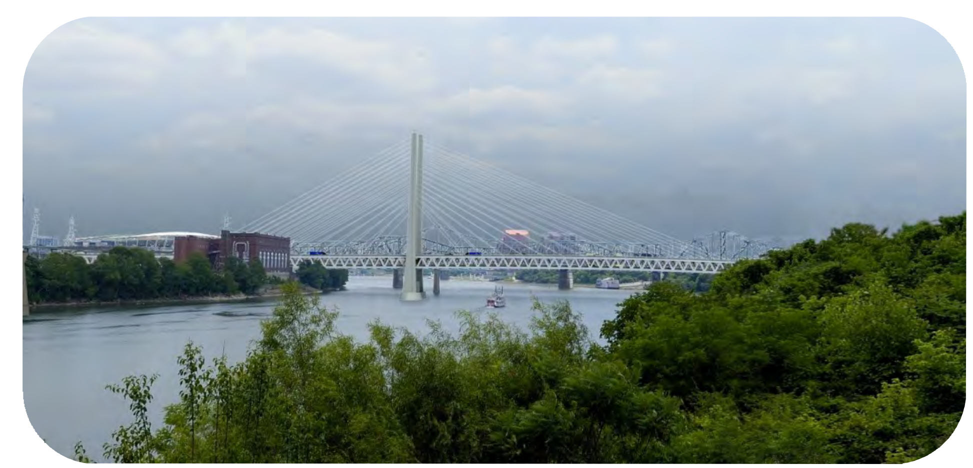
- A bridge alignment adjacent to and downstream (west) of the existing Brent Spence Bridge;
- A double-decked truss superstructure carrying two roadways on each deck, with each roadway composed of two or three 12-foot-wide lanes and two 14 foot-wide shoulders;
- An approximately 1,000-foot main span with piers outside of the main span piers of the existing Brent Spence Bridge;
- A river to superstructure clearance no lower than that of the existing Brent Spence Bridge, and
- A bridge to work in conjunction with the existing Brent Spence Bridge, to carry the Design Year 2035 traffic projection of approximately 233,000 vehicles per day.



Alternative 1 – Tied Arch



Alternative 3 – Two Tower Cable Stayed (3-Needle-Tower)



Alternative 6 – Single Tower Cable Stayed (2-Needle-Tower)

3.2 Final Bridge Alternative 1 – Tied Arch

Alternative 1 consists of a 1,000-foot span three rib tied arch with a crown height of approximately 200 feet and a double deck truss system with a top and bottom deck width of approximately 155 feet and 180 feet, respectively. The crown height allows for more slender, aesthetically pleasing arch ribs. The arch ties consist of three 38-foot deep trusses each located at the base of the arch ribs. Both the top and bottom truss chords carry approximately equal tension forces and provide some redundancy to the tie system. The tied arch hangers are connected to the arch ribs at the top and anchored into the truss top chords at the bottom.

The deck trusses serving as the arch ties are made continuous over the main span river piers in order to eliminate a deck joint at the spring points of the arch. To balance the horizontal forces created by the arch ribs, the top and bottom truss chords are large. In contrast, the truss diagonals are relatively small, allowing optimal visibility of the surrounding area to those driving along the bottom deck. The outer truss planes are inclined to match the slope of the outer arch ribs, providing a visually pleasing effect to those driving over the bridge, as well as to those observing from shore. The arch ribs and the deck truss chords feature architectural reveals which evoke a slender appearance to the structure and are intended to reference some of the region's prominent art deco landmarks such as Union Terminal and Carew Tower.

3.3 Final Bridge Alternative 3 – Two Tower Cable Stayed (3-Needle-Tower)

Alternative 3 consists of a two towered cable-stayed bridge, with each tower composed of three 335-foot tall needles and a 1,000-foot main span. Each tower needle carries a plane of stay cables which in turn support a truss at the top deck level. The deck system consists of an approximately 172-foot wide double-decked, triple-trussed superstructure. The cables and truss diagonals are inclined at the same angle, which provides a smooth visual transition from the light cables to the relatively bulkier truss. In addition, the diagonals help distribute the horizontal force of the cables into the top and bottom chords of the trusses, where that load can then be carried in part by the concrete deck. This feature maximizes the efficiency of the superstructure.

At the towers, the trusses are integrally connected to the concrete needles. This connection has two main advantages. First, it minimizes the overall width of the bridge, an important consideration with historic structures to both the east and west of the span. Second, the integral truss/tower connection eliminates the requirement for costly tower bearings, which would require periodic replacement.

This alternative's clean geometry is defined by crisp, simple lines. The harp-strung cables afford drivers unfettered views of the region's other Ohio River bridges and downtown Cincinnati, Ohio and Covington, Kentucky. This bridge's austere design also serves as a counterpoint to the complicated geometry of the other bridges along the riverfront without overcomplicating the downtown skyline.

3.4 Final Bridge Alternative 6 – Single Tower Cable Stayed (2-Needle-Tower)

Alternative 6 consists of a single tower cable stayed bridge with an approximately 1,023-foot main span. The single tower is composed of two 500-foot tall needles supporting an approximately 155-foot wide double-decked truss superstructure via two planes of doubled cables, which connect to the top chord of the edge trusses. The trusses distribute the horizontal cable load evenly to the top and bottom deck of the superstructure, a structurally efficient means of carrying these forces.

As on Alternative 3, the trusses of Alternative 6 are designed to be integral with the towers, which eliminates the necessity for a truss bearing at the tower, while also minimizing the width of the bridge.

The tower of the bridge will be one of the tallest structures on the riverfront, and will be visible from vantages on both sides of the river, despite the adjacent truss bridges upstream (east) between the new bridge location and the downtowns of Cincinnati and Covington. As such, this bridge alternative will serve as a landmark, updating the skyline of both Cincinnati and Covington, with its simple geometry producing a monumental structure.

3.5 Comparison of the Final 3 Bridge Alternatives

Table 3-1 presents a comparison of the Final 3 Bridge Alternatives.

Table 3-1. Bridge Type Alternatives

Bridge Type Alternatives		Criteria			
		Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility
1		KY: $484.6 M OH: $86.1 M Total: $570.7 M	Construction will be complicated by the inclined arch and slowed by the requirement to maintain river traffic.	Items included in M&D will be: 1. Standard Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Hanger Replacement
3		KY: $538.0 M OH: $130.6 M Total: $668.6 M	Cantilever construction of the superstructure will minimize interference to river traffic.	Items included in M&D will be: 1. High-Tech Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Stay-Cable Replacement
6		KY: $478.6 M OH: $167.8 M Total: $646.4 M	Cantilever construction of the superstructure will minimize interference to river traffic.	Items included in M&D will be: 1. High-Tech Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Stay-Cable Replacement

3.5.1 Design Considerations

Structural analyses of dead load, live load, wind load and seismic load were performed on each of the Final 3 Bridge Alternatives. The strength of major structural members was also verified. All of these analyses were based on simplified models that confirmed the major member sizes of the structures.

Current highway design requirements state the bridge should carry 11 to 12 traffic lanes over a maximum span of approximately 1,000 feet. However, the American Association of State Highway Transportation Officials (AASHTO) code requires that the bridge be designed to carry 20 traffic lanes. This will accommodate future widening of the roadway into the provided shoulders. This combination of span and load is particularly demanding and requires very special design considerations. One example of this special detailing is the angle change in the arch rib of Alternative 1 (tied arch bridge). This angle change causes the top and bottom chord of the truss to share the arch tie force, rather than using the diagonals to transfer the differential tensile tie loads.

On Alternative 3 (two tower cable stayed bridge) and Alternative 6 (single tower cable stayed bridge), the superstructure flanking either side of each tower is designed to take advantage of the approximate 30-foot truss depth in order to be self-supporting. This detail reduces the demand on the cables closest to each tower, while maintaining the openness of the architectural concept. Additionally, the trusses are directly fixed to the needle towers of the cable stayed spans rather than being supported on bearings at the tower, as is traditionally done. As a result, the overall width of the structure is narrower and reduces interferences with existing structures. This also eliminates bearings, which require regular inspection and occasional replacement.

For Alternative 1, (tied arch bridge) bearings will be used to support the vertical load of the tied arch, which is in the order of magnitude of 28,000 kips. This will require dual disk bearings to keep the bearing diameter under five feet. A disk bearing manufacturer was consulted and indicated that the dual disk bearing detail was feasible.

On each of the Final 3 Bridge Alternatives, a barge impact protection wall is provided to protect the river piers up to the 100-year flood level. The wall is designed to be hollow in order to reduce the load on the foundation.

The bridge approaches are designed to be double deck trusses consisting of multiple 200-foot spans. To date, the approach span design has not been optimized in this study. However, a preliminary design was performed for cost estimate purposes.

Drilled shaft foundations were selected for the bridges because of their high load carrying capacity in axial load and in bending. Eight-foot diameter drilled shafts were used for all foundations in order to simplify the cost comparisons. While drilled shafts are the most likely choice for the main span pier foundations, during final design, all foundations will be based on site-specific conditions, including the potential use of displacement piles in the Duke Energy property to minimize environmental impacts. Final foundation recommendations will be made during detail design.

Due to the heavy weight of the double deck superstructure, Alternatives 3 and 6 require approximately 200 strands at each cable support point. This exceeds the industry standard of a maximum of 127 strands/cable for a typical cable stayed bridge. Therefore, each cable support point will be carried by a pair of cables. This double cable detail brings the cable sizes to within common industry practice and simplifies the design of the cable to truss and cable to tower connections.

On Alternative 1, the tied arch unit extends one truss span past the arch span. This detail eliminates the deck joint at the spring points of the arch, which is a heavily congested area of the bridge, and simplifies the arrangement of the bearings. On Alternative 3, the two tower cable stayed bridge unit extends one span past each cable-supported back span to reduce the counterweight demand and move the deck joints away from the anchor pier.

On Alternative 6 (single tower cable stayed bridge), an orthotropic steel deck was selected for the main span in order to reduce the self weight. When it is combined with a concrete deck on the back span, this arrangement will balance the dead load moment at the base of the tower.

3.5.2 Construction Cost

For each of the Final 3 Bridge Alternatives, a preliminary summary of quantities was developed to include items expected to contribute significantly to the cost. Contingencies were included for items not estimated or currently anticipated. These material quantities and an assumed construction method were the basis for the estimated construction cost.

Because the cable stayed and arch main spans of the Final 3 Bridge Alternatives are of different length, the bridge cost estimates are based on the cost of the main bridge unit plus the approaches required to cover the same 2,200 feet between two fixed points. The approach span costs per square foot are based on an assumed 200-foot approach span.

Construction costs are based on 2010 costs inflated to the median construction date for each bridge alternative with an anticipated start of construction date of January 2016. An inflation rate of 37.6 percent was used for Alternative 1 based on a three year estimated construction schedule with a median construction date of June 2017. An inflation rate of 41.0 percent was used for Alternatives 3 and 6 based on a four year estimated construction schedule with a median construction date of January 2018. The ODOT FY 2010-2011 Business Plan Inflation Calculator was used to calculate the inflation rates.

The estimated construction cost for the alternatives are presented in Tables 3-2 and 3-3.

Table 3-2. Main Bridge/Approach Span Cost Breakdown

Alternative	Main Bridge ($M)	Approaches ($M)	Total ($M)
1 - Tied Arch	$358.3	$212.4	$570.7
3 - Two Tower Cable Stayed	$632.3	$36.3	$668.6
6 - Single Tower Cable Stayed	$561.1	$85.3	$646.4

Table 3-3. Main Bridge/Approach Span Cost Breakdown by State

Alternative	Main Bridge		Approaches		Total	
	KY Cost ($M)	OH Cost ($M)	KY Cost ($M)	OH Cost ($M)	KY Cost ($M)	OH Cost ($M)
1 - Tied Arch	$358.3	$0.0	$126.3	$86.1	$484.6	$86.1
3 - Two Tower Cable Stayed	$532.8	$99.5	$5.2	$31.1	$538.0	$130.6
6 - Single Tower Cable Stayed	$393.3	$167.8	$85.3	$0.0	$478.6	$167.8

3.5.3 Constructability

Construction of the Final 3 Bridge Alternatives will be very difficult due to the double deck configuration and large size of the structure. Geometry control, especially cambering of the deck trusses, will be especially difficult. However, despite the inherent difficulties, construction of these alternatives is feasible.

One method of erecting Alternative 1 in place would require a temporary cable stayed system before and after the arch rib closure. This would be expensive and risky. Another possible option is to erect the complete tied arch on land, place it on barges and then lower it on to the bearings and piers. This erection method would cost less and requires less time to complete than the other method. However, it would require the complete closure of the Ohio River for several hours. If Alternative 1 is selected for construction, the construction method to be used would be determined through coordination with the US Coast Guard.

For Alternatives 3 and 6, cable stayed bridge superstructure construction would be traditional. The truss members, floorbeam, stringers and precast deck panels will be erected by balanced cantilever method. The members and materials would be delivered under the bridge by barge. Floating cranes or deck gantries would lift the structural members to their final position. It is expected that the construction barge would be narrow enough to allow normal river traffic operations.

For the Final 3 Bridge Alternatives, the footing of the river foundations will be constructed on drilled shafts inside cofferdams.

Construction schedule was considered in the analysis of the Final 3 Bridge Alternatives with regards to constructability and the construction cost estimates. The construction schedule for Alternative 1 was based on the offsite construction/float-in method and is expected to take approximately 2.5 to 3 years. The construction schedule for Alternatives 3 and 6 was based on the cantilever construction method. Alternatives 3 and 6 are expected to take approximately 3.5 to 4 years to construct, with Alternative 6 at the higher end of that range due to the possibility of unpredictable construction delays related to the single tower construction. Any such delays could be minimized by initiating the erection of the back and main span trusses before the completion of the tower construction.

3.5.4 Maintenance

Accessibility and maintenance were considered in the design of the alternatives. The box shaped truss members were sized and arranged to allow people to work inside the member, and every corner designed to be accessible for inspection, painting, and other maintenance work. The foundations of all Final 3 Bridge Alternatives are concrete footings supported by drilled shafts into rock, which are extremely durable items.

For Alternatives 3 and 6, neither cable stayed alternative has any bearing at the tower. This eliminates the requirement of inspection, maintenance and replacement. For Alternative 1, the bearings supporting the tied arch ribs are very large. A disk bearing system is recommended for its reliability and low maintenance requirements.

Painting is required for all steel truss members, floorbeams, and stringers. In addition, because the main span of Alternative 6 utilizes a steel orthotropic deck, the underside of that steel deck will require painting.

Stay cables and arch hangers require regular bi-annual inspection but generally do not require much maintenance. However, if deterioration of the stays or hangers does occur, replacement is feasible.

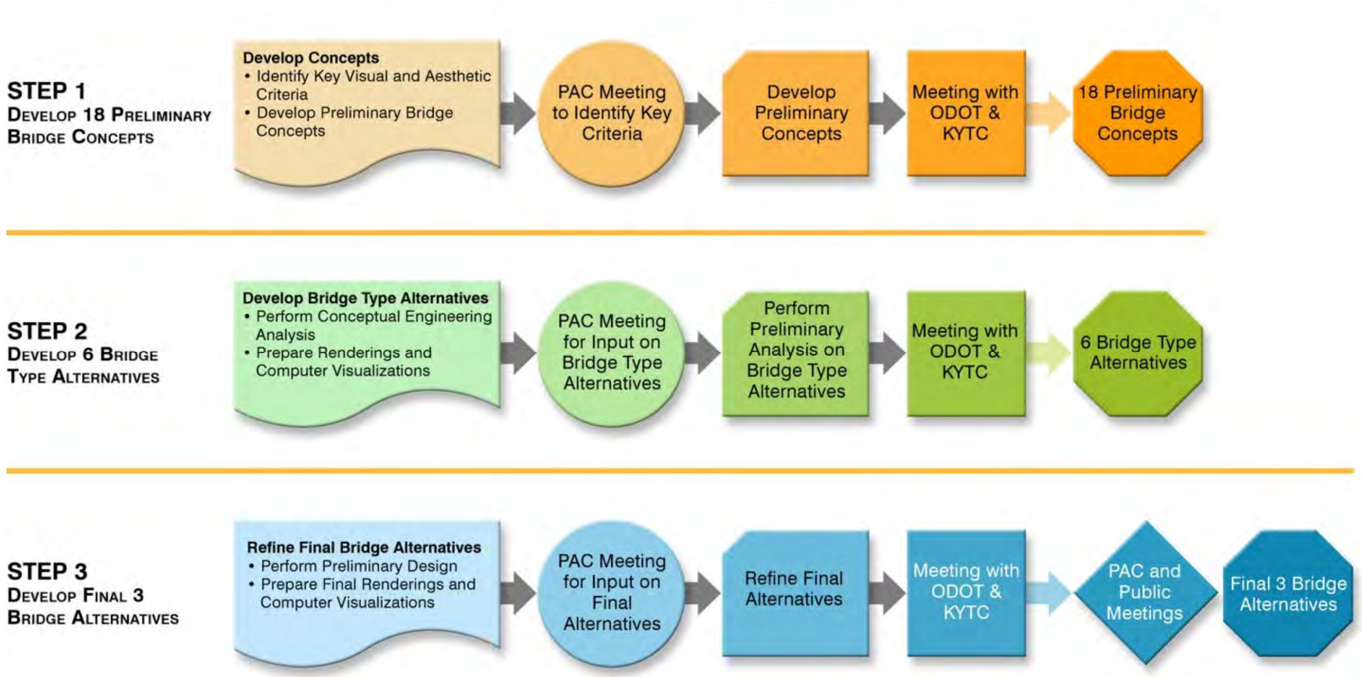
The cable stayed bridge deck shares load with the truss chords to resist the horizontal component of the cable force. A very low permeability overlay is required to protect the concrete reinforcement or orthotropic deck plate from chloride attack. Good maintenance of the deck overlay will determine the life span of the deck and bridge.

4.0 Public Involvement

This Chapter presents the Public Involvement portion of the Bridge Type Selection Process (BTSP).

4.1 Introduction

At the outset of the Brent Spence Bridge Replacement/Rehabilitation Project, the Kentucky Transportation Cabinet (KYTC) and the Ohio Department of Transportation (ODOT) instituted a Project Aesthetics Committee (PAC), a subcommittee of the project's Advisory Committee (AC), to provide local input on the design and aesthetic appearance of the corridor and the main span of the new Ohio River Bridge. Public involvement was used throughout the three steps of the BTSP. The role of public involvement was to help create and provide avenues for local citizens, stakeholders, and officials to offer their comments, suggestions and ask questions. This feedback would be used in determining a final bridge type that would reflect, as much as possible, the needs and desires of the community. This chapter describes the stakeholder and public involvement activities that occurred throughout the Bridge Type Selection Process.

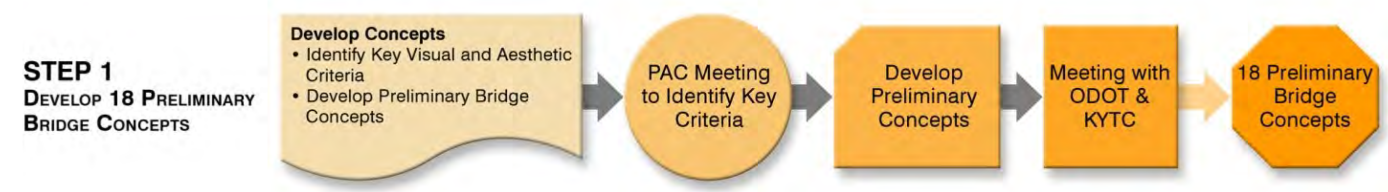


In Step 1, the PAC identified key visual and aesthetic criteria to develop preliminary bridge concepts. The project team used these criteria to develop the preliminary bridge concepts.

In Step 2, the bridge team presented these preliminary bridge concepts to the PAC for input in determining the 6 Bridge Type Alternatives. The public was presented 12 preliminary bridge concepts for review and comment through a press release and on the project website. Public comments received on these bridge concepts were used to refine the bridge types.

In Step 3, the bridge team presented the 6 Bridge Type Alternatives to the PAC and received input comparing the alternatives with key criteria from the group. This input was then used to develop the Final 3 Bridge Alternatives.

4.2 Step 1 – Develop 18 Preliminary Bridge Concepts



The objective of Step 1 of the Bridge Type Selection Process was to identify approximately six key visual and aesthetic criteria, and to use those criteria to determine approximately 18 Preliminary Bridge Concepts to be developed during this step of the process.

Following verification by the US Coast Guard of the bridge clearance and pier locations, the project team met with the PAC to identify the key visual and aesthetic criteria. These visual and aesthetic criteria were then used to develop and refine the Preliminary Bridge Concepts, reflecting all feasible bridge types and using engineering solutions that best reflected the characteristics identified by the PAC.

Public involvement in Step 1 included:

<u>PAC Meeting</u>

- September 25, 2009 - Cincinnati City Hall, Ohio

The purpose of the PAC meeting held on September 25, 2009 was to present an update of the project, present context of aesthetics in the project study area, and develop key visual and aesthetic criteria for the project. The role of the PAC was to provide input on aesthetic treatments of bridge structure types.

At the PAC meeting, the possible feasible bridge types were presented, which included cable stayed, arch, and truss bridges. A suspension bridge is not feasible at this location due to the proposed roadway geometry and excessive costs.

During the meeting, committee members provided their input on key aesthetic criteria for the project. The key visual and aesthetic criteria identified by the PAC for selecting a bridge type included the following:

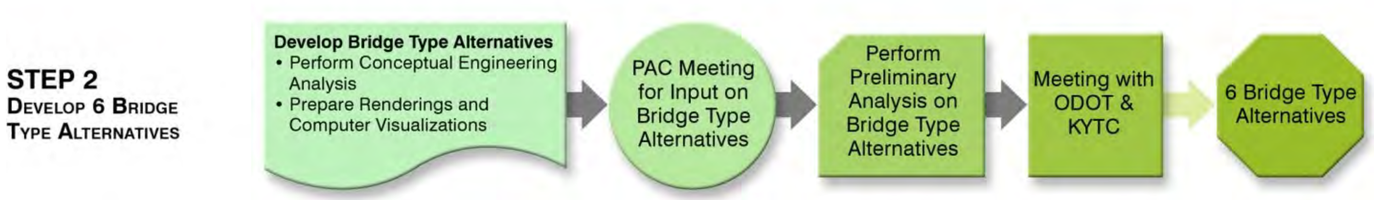
- The new bridge should be visually attractive;
- The new bridge should be visible looking "through" the existing bridge (from the east);
- As much as possible, crossing the new bridge should allow views of the surrounding context (unlike existing bridge);
- The new bridge should have distinctive characteristics that identify it as a local landmark; and
- The new bridge should have a visual relationship with the existing bridge.

Additional aesthetic criteria identified by the PAC were:

- The new bridge colors/textures/landscaping, etc. should be aesthetically pleasing; and
- The existing bridge should be maintained/repainted to blend in with the new bridge.

<u>Results</u>
As a result of the September 25, 2009 PAC meeting, 24 preliminary bridge concepts were developed and evaluated during Step 1. During the evaluation process, 12 preliminary bridge concepts were reviewed and approved by the Federal Highway Administration (FHWA), KYTC, and ODOT as meeting the objectives of Step 1.

4.3 Step 2 – Develop 6 Bridge Type Alternatives



Public involvement activities in Step 2 included a combined meeting of the Advisory Committee (AC) and PAC, and a press release soliciting public input.

<u>PAC Meeting</u>

- January 29, 2010 - Northern Kentucky Convention Center, Covington, Kentucky

Twelve preliminary bridge concepts were developed and refined during Step 1 and presented to the AC/PAC on January 29, 2010. These preliminary bridge concepts consisted of two truss bridges, three arch bridges, and seven cable stayed bridges. Various bridge components were also presented that could be incorporated into the 12 preliminary bridge concepts. Feedback was requested on these components to aid the design process. The PAC members also completed a criteria matrix for the 12 preliminary bridge concepts. Additional comments were received from committee members following the PAC meeting.

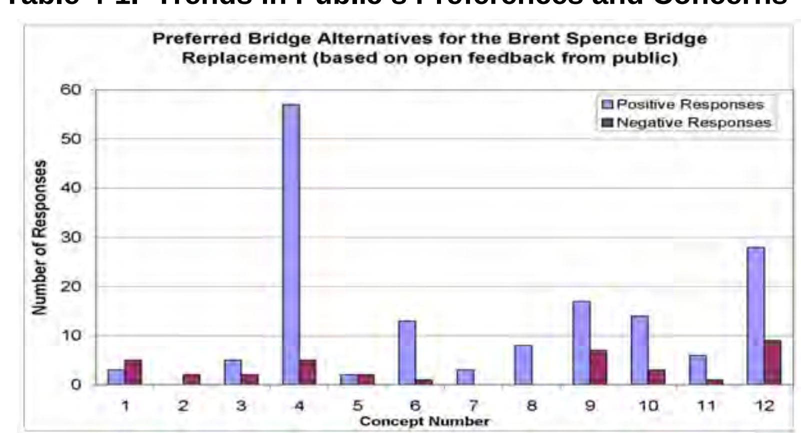
Some general preferences were noted from the January 29, 2010 meeting on each of the bridge types. The preference for cable stayed bridges was a harp arrangement paired with a Pratt truss with stays in line with the truss diagonals. A double-deck truss style was not preferred. Two-legged cable stayed towers were preferred over the three-legged tower options.

<u>Press Release/Public Input</u>
Following the January 29, 2010 PAC meeting, the public was provided a one-week comment period to submit comments and provide feedback regarding their thoughts on the aesthetic elements of the Brent Spence Bridge Replacement/Rehabilitation Project. Comments were received through emails, the project website, faxes, and phone calls.

The 12 preliminary bridge concepts were made available to the public, with variations on each bridge type. The three types included truss type bridges (Concepts 1 and 2) which were similar in design to the current Brent Spence Bridge, arch type bridges (Concepts 3 through 5) which were similar in design to the I-471 Daniel Carter Beard or "Big Mac" bridge, and cable stayed-type bridges (Concepts 6 through 12).

Public comments were analyzed and used to quantify the trends in the public's preferences and concerns regarding the overall project and the various bridge concepts. Table 4-1 is a visual representation of those trends. In order to generate the bar chart in Table 4-1, those comments which liked all or none of the bridge concepts, or which showed no preference (neutral) are not included in the table. In general, up to three positive or three negative comments from each commenter were included in the analysis. Showing a preference for one concept over another was not considered a negative comment for the less preferred concept, unless a specifically negative comment was made about that concept.

Table 4-1. Trends in Public's Preferences and Concerns



Overall, one arch bridge and the cable stayed type bridges received the most positive feedback and were singled out as being distinctive and providing a gateway experience when entering Cincinnati from Kentucky. The cable stayed bridges with angled piers, Concept 9 and Concept 10 received the most negative comments for their designs.

The two concepts which received both favorable and unfavorable comments were Concept 4 and Concept 12. The public felt very strongly about these concepts. They had the most “votes” cast in favor of the designs and received the majority of negative comments.

Concept 4 received more favorable votes than any of the 12 concepts presented. The favorable comments were based upon its resemblance to the I-471 Daniel Carter Beard Bridge. Concept 4 would provide a “bookend” to the west with the Daniel Carter Beard Bridge being the “bookend” to the east. Comments were also made that the arched design looked strong and sturdy.

The negative comments for Concept 4 focused on a desire for a new type of bridge design rather than duplicating an existing bridge design in the area. These comments requested a distinctive bridge that would be a landmark for Cincinnati and would provide a gateway experience upon entering Cincinnati from Kentucky. Some comments stated that the Concept 4 design would show that the greater Cincinnati area is not creative or distinctive.

Concept 12, similar to Concept 4 had a very large discrepancy in public comments. The comments in favor of Concept 12 described it as distinctive, visually stunning, and impressive as well as an excellent gateway into Cincinnati. The negative comments disliked the asymmetrical layout of the one large pier and a few comments noted it was “ugly.”

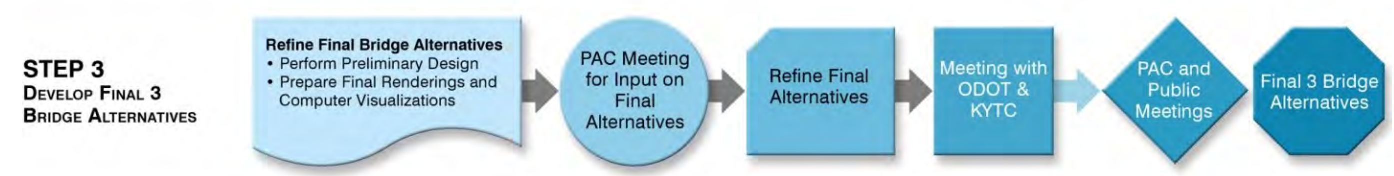
Other generalized comments included painting the bridge a bright color and painting the new bridge something other than white, which would blend into Cincinnati’s gray winter sky. There were several comments suggesting that the current Brent Spence Bridge be torn down, because it is unattractive and would look even more so next to a new bridge. The incorporation of symbols of both Kentucky and Ohio onto the bridge was suggested numerous times. Examples included the addition of smoke stacks on the tops of the piers, particularly for Concept 12 because this bridge resembles a riverboat.

<u>Results</u>

Using this guidance from the PAC and other public input on these Preliminary Bridge Concepts, 6 Bridge Type Alternatives were identified for further study during Step 2.

Through a series of design meetings with the FHWA, KYTC, and ODOT during Step 2, the 6 Bridge Type Alternatives were further refined for conformance to the purpose and needs of the project. As a result of the conceptual engineering analysis, each of the 6 Bridge Type Alternatives were evaluated based on construction cost, constructability/construction time, maintenance and durability, major rehabilitation feasibility, and maintenance of traffic. Renderings and computer visualizations showing different view and details were developed for each of the 6 Bridge Type Alternatives. At the end of Step 2, the 6 Bridge Type Alternatives were reviewed and approved by FHWA, KYTC, and ODOT as meeting the objectives of Step 2.

4.4 Step 3 – Develop Final 3 Bridge Alternatives



Public involvement activities in Step 3 included a combined meeting of the AC/PAC, a press release soliciting public input, and public meetings.

<u>PAC Meeting</u>

- April 15, 2010 - Duke Energy Convention Center, Cincinnati, Ohio

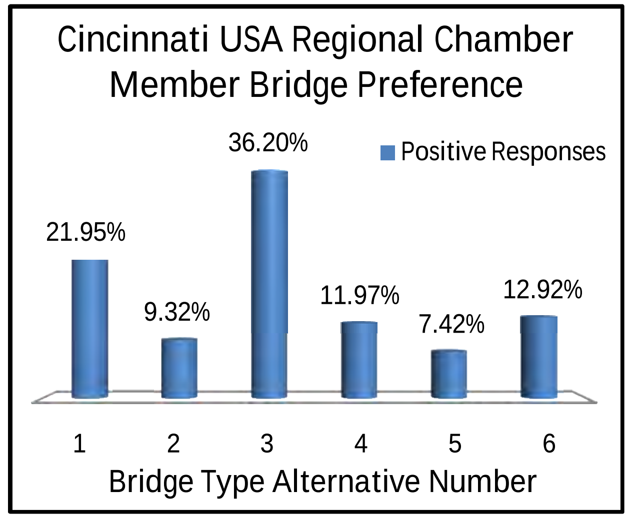
The 6 Bridge Type Alternatives approved by FHWA, KYTC, and ODOT were presented in greater detail to the AC/PAC on April 15, 2010. The purpose of the meeting was to receive feedback on the 6 Bridge Type Alternatives to aid the project team in selecting 3 Final Bridge Alternatives. Key visual and aesthetic criteria previously established were used by the PAC to evaluate the 6 Bridge Type Alternatives. Two steel arch bridge alternatives and four cable stayed bridge alternatives were presented at the meeting.

The cable stayed bridges were received more favorably than the arch bridges. The PAC was interested in additional components such as colors, shapes, and views. These bridge components will be presented in a later meeting. Costs of the various bridges were also noted as a concern in addition to views of the bridge alternatives. Additional comments were received from committee members following the PAC meeting.

Press Release/Public Input

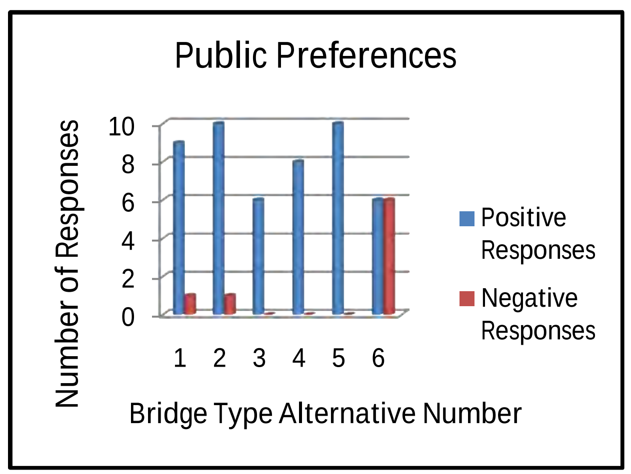
The 6 Bridge Type Alternatives presented at the April 15, 2010 PAC meeting were also made available for public comments during Step 3 of the Bridge Type Selection Process. The public was provided a one-week comment period to submit comments and provide feedback regarding their thoughts on the aesthetic elements of the Brent Spence Bridge Replacement/Rehabilitation Project. Comments received by the public varied. Each of the alternatives was given both positive and negative comments. Comments showed that the public is in favor of both the cable stayed bridge types as well as the arch type bridges, with no clear preference.

Table 4-2



As a member of the AC, the Cincinnati USA Regional Chamber conducted a member survey of the 6 Bridge Type Alternatives. The Chamber received 1,362 responses from their members over a two-day period. The member's bridge preference results are shown in Table 4-2.

Table 4-3



The public comments received were analyzed and used to quantify trends for the public's preferences and concerns regarding the overall project and for the various bridge concepts. Table 4-3 is a visual representation of those trends. Those comments which liked all or none of the bridge concepts, or which did not indicate a preference (neutral) are not included in Table 4-3. In general, up to three positive or three negative comments from each commenter were included in the analysis. Showing a preference for one concept over another was not considered a negative comment for the less preferred concepts, unless a specifically negative comment was made about that concept. The bar chart in Table 4-3 provides a summary of public opinions on the bridge concepts, and was used as one source of input for the recommendation of the Final 3 Bridge Alternatives.

Cable stayed bridges were noted as something new and liked as opposed to the arch bridge type, which is already present in the area. Some comments stated that the designs needed to be more impressive. Comments also referred to whether or not the alternatives would fit into the context of the existing landscape.

Other comments recommended that the appearance of the new bridge should not be a concern because safety is more important and the project needs to progress faster. Cost was also noted as an important factor in selecting a bridge design. Safety concerns were expressed about the alternatives with three legs and the possibility of vehicles running into the posts.

Alternative 1 was well regarded by the public via the input received from the project website and the Cincinnati USA Regional Chamber poll. Comments related to Alternative 1 noted its similarities to the Daniel Carter Beard Bridge. This alternative works best with the existing bridges on the Ohio River since it serves as a bookend to the Daniel Carter Beard Bridge. In contrast, this alternative was also described as being too similar to the Daniel Carter Beard Bridge.

Alternative 3 was well regarded by the public via the input received from the AC and PAC, the project website, and, especially, the Cincinnati USA Regional Chamber poll. From the driver's point of view, the three needle towers are well proportioned and the vertical towers are more traditional and straightforward than the inclined tower Bridge Type Alternatives. Alternative 3 was noted as adding variety to the other existing bridges. Being a cable stayed bridge, it provides a modern style to the landscape but still resembles the Roebling Suspension Bridge.

Comments for Alternative 6 were both favorable and unfavorable. Alternative 6 is the most visible of the Bridge Type Alternatives, especially from Cincinnati and Covington. This alternative was highly regarded by the public via the input received from the Cincinnati USA Regional Chamber poll and, especially, the AC and PAC. Comments stated that the single structure is too dramatic and appears lopsided. Other comments noted that since there is only one structure, the towers would not obstruct views of the Ohio River. The single structure was also described as unique and simple and a gateway into both Ohio and Kentucky.

AC Meeting

- December 17, 2010 – Ohio Kentucky Indiana Regional Council of Governments, Cincinnati, Ohio

The technical analyses for the Final 3 Bridge Alternatives were presented to the AC on December 17, 2010. To date, the bridge design team has not received additional comments from the AC.

Public Hearings

As part of the National Environmental Policy Act (NEPA), public hearings for the Brent Spence Bridge Replacement/Rehabilitation Project will be held in 2011. The focus of the hearings will be the selection of the recommended Preferred Alternative for the highway and the new bridge crossing over the Ohio River. The pulbic will be encouraged to provide written and/or verbal comments. During public hearings, the public will have the opportunity to vote on components of the three bridge alternatives using a hand-held audience response polling system. A two week comment period will follow the public hearings.

Results

The objective of this final step of the Bridge Type Selection Process was for the project team to further develop and refine the Final 3 Bridge Alternatives and to document the entire three step process. During this step, the project team assessed the suitability of the Final 3 Bridge Alternatives based on more detailed examination of the structural requirements, cost, constructability, environmental impacts, aesthetics, and other key criteria. This task included performing significant preliminary design and preparing additional renderings for the Final 3 Bridge Alternatives. During this step significant feedback on these alternatives were received from AC and PAC.

Following the public hearings' two-week public comment period, the selection of a new Ohio River Bridge will be determined by KYTC and ODOT in consultation with FHWA. The selection of the preferred bridge type will based upon consideration of several factors including the technical analyses completed for the project and public input.

4.5 Appendix

The following documents are provided on the CD enclosed with this report:

Bridge Type Selection Process – Step 1

4A 9-25-09 PAC Meeting Presentation
4B 9-25-09 PAC Meeting Handouts
4C 9-25-09 PAC Meeting Minutes

Bridge Type Selection Process – Step 2

4D 1-29-10 PAC Meeting Presentation
4E 1-29-10 PAC Meeting Handouts
4F 1-29-10 PAC Meeting Minutes
4G Main River Bridge Structure Type Study - Step 1 Recommendation Memo

Bridge Type Selection Process – Step 3

4H 4-15-10 PAC Meeting Presentation
4I 4-15-10 PAC Meeting Handouts
4J 4-15-10 PAC Meeting Minutes
4K Main River Bridge Structure Type Study - Step 2 Recommendation Memo
4L 12-17-10 AC Meeting Presentation
4M 12-17-10 AC Meeting Handouts
4N 12-17-10 AC Meeting Minutes

5.0 Environmental Commitments

The commitments that will be included in the Environmental Assessment (EA) and Finding of No Significant Impact (FONSI) will be legally binding and are necessary for completion of the Brent Spence Bridge Replacement/Rehabilitation Project. This Chapter describes the various commitments that are related to the new Ohio River Bridge.

5.1 Introduction

Interstate 75 (I-75) within the Greater Cincinnati/Northern Kentucky region is a major thoroughfare for local and regional mobility. Locally, it connects to I-71, I-74 and US Route 50. The Brent Spence Bridge provides an interstate connection over the Ohio River and carries both I-71 and I-75 traffic. The bridge also facilitates local travel by providing access to downtown Cincinnati, Ohio and Covington, Kentucky. Safety, congestion and geometric problems exist on the structure and its approaches. The Brent Spence Bridge, which opened to traffic in 1963, was designed to carry 80,000 vehicles per day. Currently, approximately 160,000 vehicles per day use the Brent Spence Bridge and traffic volumes are projected to increase to approximately 233,000 vehicles per day in 2035.

The I-75 corridor within the Greater Cincinnati/Northern Kentucky region is experiencing problems which threaten the overall efficiency and flexibility of this vital trade corridor. Areas of concern include, but are not limited to, growing demand and congestion, land use pressures, environmental concerns, adequate safety margins, and maintaining linkage in key mobility, trade, and national defense highways.

5.2 Environmental Context and Application

The project area for the Brent Spence Bridge Replacement/Rehabilitation Project and the overall I-71/I-75 corridor varies from suburban in the southern portion of the study area to urban near the Ohio River and northward into Ohio. Section 5.3 describes the considerations and commitments that are expected to minimize and mitigate impacts to the human and natural environment as a result of the project.

The Final 3 Bridge Alternatives took into consideration the location and appearance of the existing Brent Spence Bridge in order to achieve a visually compatible pair of new and old structures along the Ohio River. The key aesthetic criteria for selecting a bridge type are further discussed in Chapter 4 of this document. The key aesthetic criteria address views of the bridge, views from the bridge, views of the surrounding context, local characteristics, and the relationship with the existing Brent Spence Bridge. The aesthetic criteria were developed by the Project Aesthetic Committee (PAC) members. Selection of the Final 3 Bridge Alternatives was made with the key aesthetic criteria and surrounding context in mind.

5.3 Considerations

Throughout the Brent Spence Bridge Replacement/Rehabilitation Project development process, specific measures were implemented to avoid, minimize or mitigate environmental impacts associated with the new Ohio River Bridge. These measures are identified in the EA, and will be implemented during detailed design and construction. These environmental commitments are listed in Table 5-1. Further commitments may be identified before the EA is completed and those will be noted in the EA.

Table 5-1. Environmental Commitments Developed During The Project Development Process For The New Ohio River Bridge

Issue	Commitment	Implementation Time Frame
Ohio River	No instream work will occur between March 15 and June 30. Best management practices will be used to ensure minimization of silt entering streams impacted by construction.	Construction
Floodplain Resources	A floodplain analysis will be completed to identify impacts to the Ohio River Floodplain.	Final design
Threatened and Endangered Species	Mussels - A mussel survey will be conducted on the preferred alternative and an effects determination will be completed and coordinated with the USFWS, the ODNR, and the Kentucky Department of Fish and Wildlife Resources. Peregrine Falcon - Coordination with the non-game board of Kentucky Department of Fish and Wildlife Resources would occur in the spring prior to demolition of the bridge approaches to address nesting of Peregrine Falcons.	At least one year prior to instream work. Spring prior to demolition of the bridge and approaches.
Residential and Business Displacements	Acquisition of property for right-of-way will be in accordance with the Uniform Relocation Assistance and Real Property Acquisition Policies Act of 1970 (P.L. 91-646).	Right-of-Way acquisition
Section 4(f) Resources	Impacts to Longworth Hall will result from the construction of the new bridge. Mitigation of impacts will be coordinated with the Ohio Historic Preservation Office during detailed design of the bridge.	Any pertinent commitments made will be developed in the construction plan notes.
Noise	Noise Walls are proposed for the overall project and will be developed in accordance with KYTC and ODOT procedures.	Final design
Utility Issues	Utility coordination will occur through final design and into construction.	Final design/Construction

5.3.1 River Crossing and Pier Locations

The Ohio River is the most prominent natural feature of the project corridor, and was given great consideration throughout the Bridge Type Selection Process. Through coordination with the US Coast Guard, the requirements for the new bridge included:

- Maintaining the Ohio River channel width at this location;
- Providing the minimum vertical clearance for Ohio River; and
- Locating piers outside of the existing Brent Spence Bridge piers locations.

Ohio River looking downstream (west)

5.3.2 Permits

A number of federal and state permits will be required for the project. A final permit determination will be made after the selection of the new Ohio River Bridge. The anticipated permits include:

- Section 9 Bridge Permit;
- Section 404 Individual Permit;
- Section 401 Water Quality Certification; and
- Section 10 Navigable Waterways.

5.3.3 Ecological

Potential stream mitigation measures may include payment into the Kentucky Department of Fish and Wildlife Resources (KDFWR) In-lieu Fee Program, or a stream restoration project within the watershed using natural channel design.

The US Army Corps of Engineers requires mitigation for impacts greater than 0.1 acres of jurisdictional wetland. Potential wetland mitigation measures for small impacts could be accomplished through purchase of wetland mitigation bank credits (if applicable) or creation of wetland within similar dry detention basins along the proposed corridor.

Since a new bridge will be constructed adjacent to the existing bridge, best management practices will be used during placement of bridge piers to minimize impacts to aquatic life. In addition, in-stream work within the Ohio River will be restricted between March 15 and June 30.

During construction, best management practices will also be used to ensure minimization of silt entering nearby headwater streams. Best management practices may include use of silt fences, staked straw bales, brush barriers, sediment basins, diversion ditches, and timing of construction to dry periods of the year.

A detailed mussel survey will be completed after the new Ohio River Bridge has been selected. An effects determination on these mussel species will be based on the results of the survey and the proposed level of disturbance, and coordinated with state and federal resources agencies prior to construction.

5.3.4 Historic Resources

The proposed bridge passes through 198 feet of the eastern end of the Longworth Hall building, requiring that three 15-foot, two 13-foot, and six 12-foot bays of the building be demolished. This affected section of the building is that portion which was previously altered by reducing its length and adding a five-story brick addition. In order to mitigate these effects the following mitigation options are being considered:

Longworth Hall looking east along Pete Rose Way

- Preparation of Historic American Building Survey (HABS) Documentation of Longworth Hall;
- Reconstruction of the portion of the building that was demolished by fire, which would allow the building to regain historic integrity and floor space that will otherwise be lost during the construction of the bridge;
- Installation of appropriate storm windows throughout the building to reduce traffic and ambient noise, reduce dust and debris from the roadway, and to protect the historic windows;
- Rehabilitation of the associated scale house, located on the property north of Longworth Hall, for interpretative use; and
- Completion of a contextual study of extant large scale railroad freight houses in Ohio.

Additional coordination with the Ohio Historic Preservation Office (OHPO) and Section 106 consulting parties will be undertaken to develop appropriate mitigation measures to address the adverse effects resulting from the project. Such efforts will be documented in detail under separate cover in a Memorandum of Agreement prepared for impacts and mitigation to historic resources.

5.4 Appendix

The following documents are provided on the CD enclosed with this report:

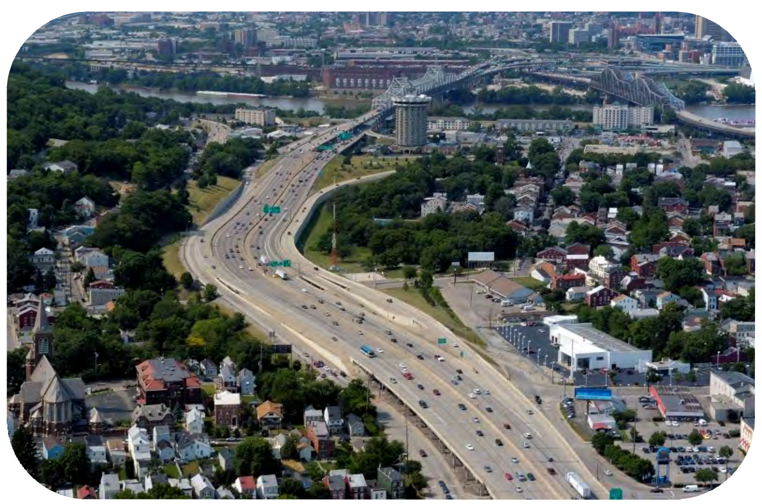
5A Agency Correspondence

6.0 Development of Bridge Alternatives

This chapter provides information about the development of the bridge alternatives for the Brent Spence Bridge Replacement/Rehabilitation Project, explaining in detail the 3 Step Bridge Type Selection Process.

6.1 Introduction

A Bridge Type Selection Process (BTSP) was completed as part of the Brent Spence Bridge Replacement/Rehabilitation Project to assist the Kentucky Transportation Cabinet (KYTC) and the Ohio Department of Transportation (ODOT) in selecting one bridge alternative to be constructed across the Ohio River. The recommended Final 3 Bridge Alternatives presented in this document are the result of the project's functional and budgetary requirements, as well as the public feedback received during the course of the BTSP.



Kentucky approach to Brent Spence Bridge

The proposed bridge will span the Ohio River just west of the current Brent Spence Bridge which facilitates interstate and local travel by providing access to Covington, Kenton County, Kentucky and downtown Cincinnati, Hamilton County, Ohio. The Brent Spence Bridge, which opened to traffic in 1963, was designed to carry 80,000 vehicles per day. Currently, approximately 160,000 vehicles per day use the Brent Spence Bridge and traffic volumes are projected to increase to approximately 233,000 vehicles per day in 2035. Safety, congestion and geometric problems exist on the structure and its approaches.

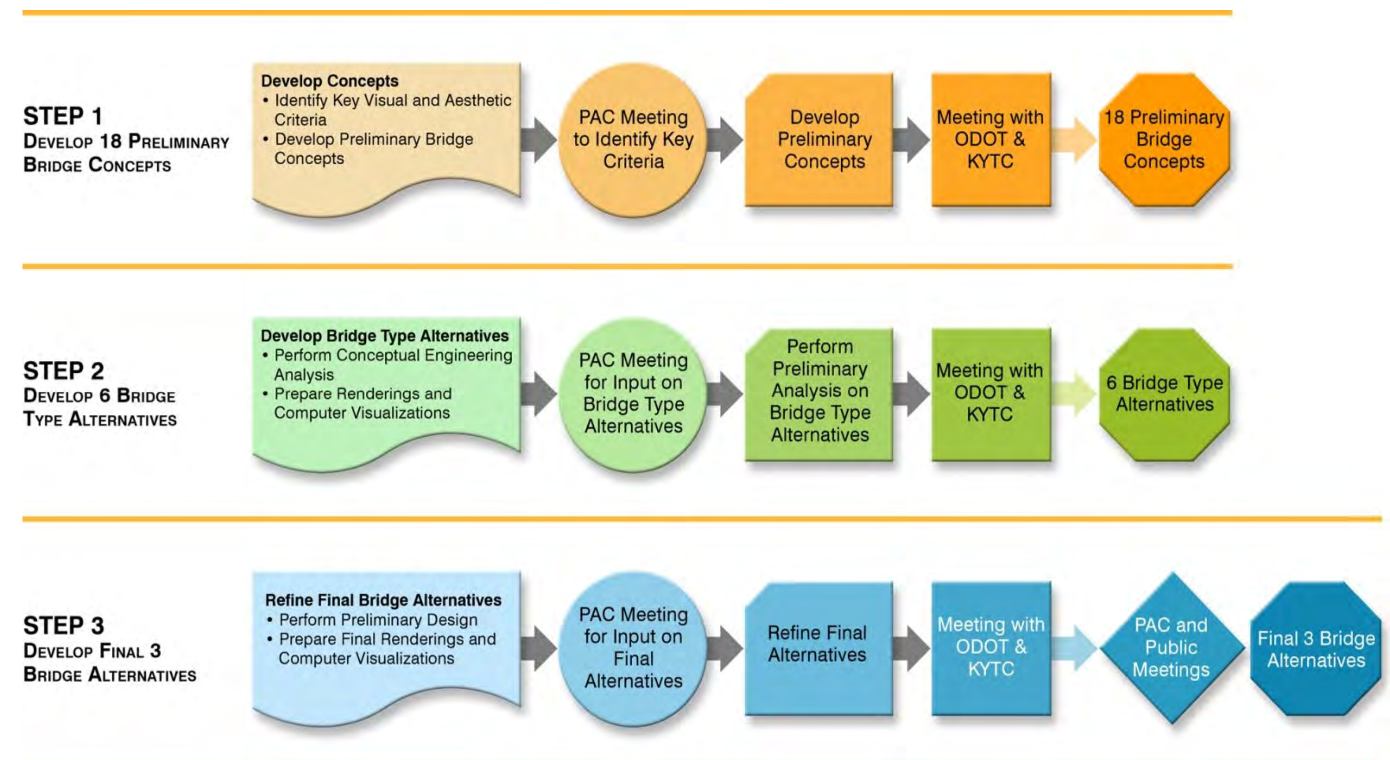
Within this context, the new bridge must meet several requirements:

- Minimize its impact on local historic structures and local infrastructure;
- Work in conjunction with the existing Brent Spence Bridge;
- Fit into the construction schedule and budget of the larger project to increase capacity on I-75;
- Require minimal maintenance and maximum durability;
- Have no permanent effect on river navigation;
- Integrate itself in the landscape of the riverfront;
- Provide an improved crossing experience for drivers; and
- Conform to current design standards.

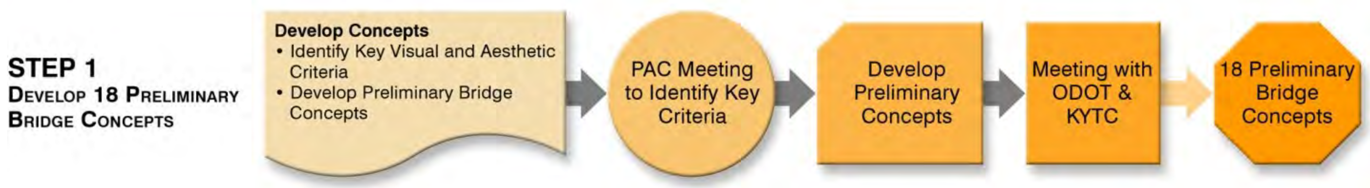
6.2 The Bridge Type Selection Process

As depicted below, the Bridge Type Selection Process (BTSP) is collaborative in nature and based on public input and engineering details. The process began in 2009 and includes the following three steps:

- Step 1 - Develop 18 Preliminary Bridge Concepts;
- Step 2 - Develop 6 Bridge Type Alternatives; and
- Step 3 - Develop Final 3 Bridge Alternatives.



6.2.1 Step 1 – Develop 18 Preliminary Bridge Concepts



The objective of Step 1 of the BTSP was to:

- Identify key visual and aesthetic criteria to be used as part of the BTSP;
- Obtain US Coast Guard design requirements for the new bridge; and
- Develop approximately 18 Preliminary Bridge Concepts.

Prior to meeting with the Project Aesthetics Committee (PAC), coordination with the US Coast Guard was conducted to determine their design requirements for the new bridge. Following verification by the US Coast Guard of the bridge clearance and pier locations, the bridge design team met with the PAC on September 25, 2009 to identify the key visual and aesthetic criteria. These visual and aesthetic criteria were then used to develop and refine the Preliminary Bridge Concepts, reflecting feasible bridge types and using engineering solutions that best addressed the characteristics identified by the PAC.

In coordination with the Federal Highway Administration (FHWA), KYTC, and ODOT, key design criteria and guidelines were developed as evaluation methodology to be used to evaluate the preliminary bridge concepts. The key design criteria developed to be used during each step were:

- Construction Cost;
- Constructability;
- Maintenance and Durability; and
- Major Rehabilitation Feasibility.

Evaluation guidelines were also developed as part of the overall project. Some of the guidelines reflected navigational, structural and highway limitations, and physical restrictions that exist at the bridge site. Other guidelines represented environmental commitments and financial constraints necessary to meet budgetary goals. The key design criteria, key visual and aesthetic criteria, and evaluation guidelines were used to select and develop the Preliminary Bridge Concepts.

As a result of the September 25, 2009 PAC meeting, 24 preliminary bridge concepts were developed and evaluated during Step 1. The evaluation process recommended 12 preliminary bridge concepts to be carried forward for further study. The 12 preliminary bridge concepts represented all feasible bridge types and engineering solutions that addressed the PAC's criteria. FHWA, KYTC, and ODOT concurred that the 12 concepts best met the Step 1 objectives.

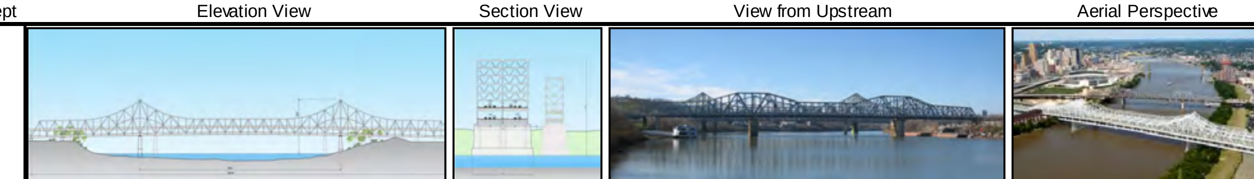
The 12 Preliminary Bridge Concepts selected as meeting the objectives of Step 1 were:

Legend		
Feasible	Feasible with Constraints	Not Feasible

Truss Concepts

Concept | Elevation View | Section View | View from Upstream | Aerial Perspective

1



Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Truss Support Type		Truss Inclination		Top Bracing		
Cantilever	Simply Supported	Inclined Trussess	**Vertical Trussess**	Strut	**K-Brace**	Cross Braced

2

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Truss Support Type		Truss Inclination		Top Bracing		
Cantilever	**Simply Supported**	Inclined Trussess	**Vertical Trussess**	Strut	**K-Brace**	

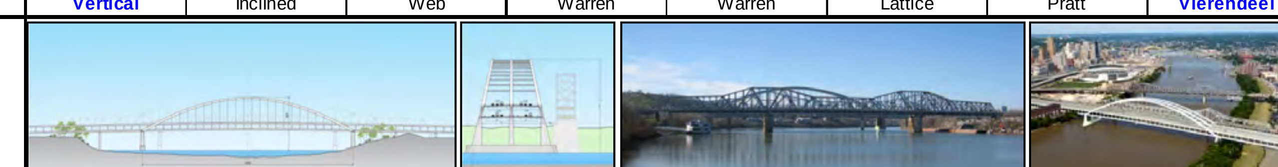
Arch Concepts

3

Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	**Vertical Leg**	Strut	K-Brace	Cross Braced	**Lattice**	**Shallower**	Deeper
Hanger Arrangement			**Deck Truss Type**				
Vertical	Inclined	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

4



Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility

Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	Vertical Leg	**Strut**	K-Brace	Cross Braced	Lattice	Shallower	**Deeper**
Hanger Arrangement			**Deck Truss Type**				
Vertical	Inclined	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

Arch Concepts, Continued

5

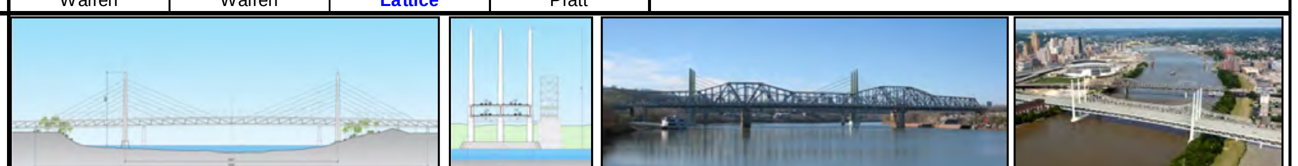
Construction Cost		Constructability		Maintenance and Durability		Major Rehabilitation Feasibility	
Leg Inclination		Top Bracing				Depth of Arch	
Inclined Leg	Vertical Leg	Strut	**K-Brace**	Cross Braced	Lattice	Shallower	**Deeper**
Hanger Arrangement			Deck Truss Type				
Vertical	**Inclined**	Web	Warren	Warren	Lattice	Pratt	**Vierendeel**

Cable Stayed Concepts

6

Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan
Deck Truss Type					
Warren	Warren	**Lattice**	Pratt		

7



Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan
Deck Truss Type					
Warren	Warren	Lattice	**Pratt**		

8

Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	**Vertical Leg 2 Leg Option**	Vertical Leg 3 Leg Option	Harp	Fan	**Semi Fan**
Deck Truss Type					
Warren	Warren	Lattice	Pratt		

Cable Stayed Concepts, Continued

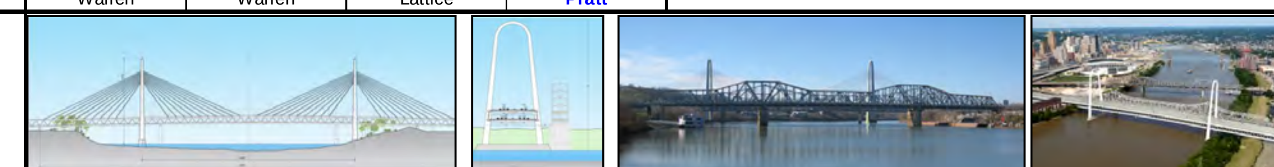
9

Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	Harp	Fan	**Semi Fan**
Deck Truss Type					
Warren	**Warren**	Lattice	Pratt		

10

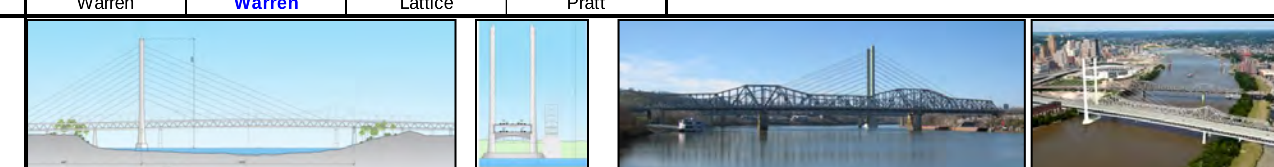
Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	**Vertical Leg 3 Leg Option**	**Harp**	Fan	Semi Fan
Deck Truss Type					
Warren	Warren	Lattice	**Pratt**		

11



Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	Vertical Leg 2 Leg Option	Vertical Leg 3 Leg Option	Harp	Fan	**Semi Fan**
Deck Truss Type					
Warren	**Warren**	Lattice	Pratt		

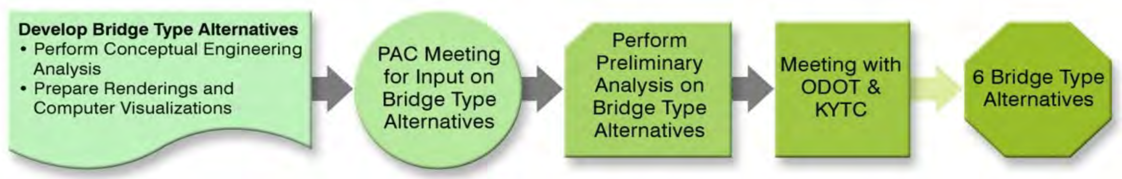
12



Construction Cost	Constructability		Maintenance and Durability		Major Rehabilitation Feasibility
Tower Shape & Number of Legs			Stay Cable Arrangement		
Inclined Leg/ Arch Tower	**Vertical Leg 2 Leg Option**	Vertical Leg 3 Leg Option	**Harp**	Fan	Semi Fan
Deck Truss Type					
Warren	**Warren**	Lattice	Pratt		

6.2.2 Step 2 - Develop 6 Bridge Type Alternatives

STEP 2
Develop 6 Bridge Type Alternatives



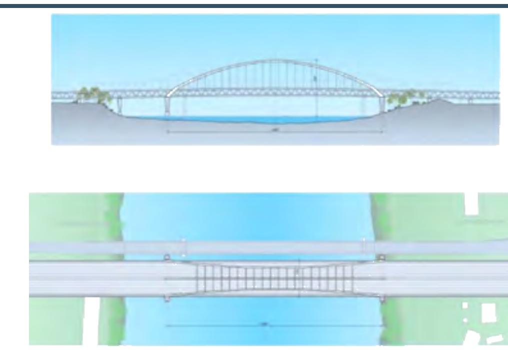
The objective of Step 2 was to:

- Present the preliminary bridge concepts approved during Step 1 to the PAC and public to gain feedback to help select the concepts to be recommended as the 6 Bridge Type Alternatives for further development in Step 2;
- Perform conceptual engineering analysis on the 6 Bridge Type Alternatives;
- Prepare renderings and computer visualizations of the 6 Bridge Type Alternatives; and
- Prepare cost estimates for the 6 Bridge Type Alternatives.

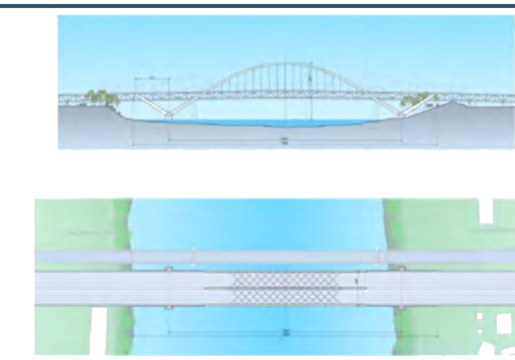
At the beginning of Step 2, the 12 preliminary bridge concepts were presented to a combined meeting of the Advisory Committee (AC) and PAC on January 29, 2010. During this meeting, the bridge design team presented the 12 preliminary bridge concepts consisting of two truss bridge, three arch bridges and seven cable-stayed bridges. The bridge design team then solicited feedback from the two committees as to which concepts best met the five key visual and aesthetic criteria. During the meeting, the bridge design team presented various bridge components incorporated into the 12 preliminary bridge concepts and requested additional feedback to aid in the Step 2 bridge type selection process. The 12 preliminary bridge concepts were also posted on the project website to solicit public comment as well. Following the AC/PAC meeting, the public was provided a one-week comment period to submit feedback regarding the aesthetic elements of the new Ohio River Bridge. Comments were received through emails, the project website, faxes, and phone calls.

Based on the results of the January 29th AC/PAC meeting and the public comments received, 6 preliminary bridge concepts were selected and approved by FHWA, KYTC and ODOT to be evaluated in more detail during Step 2.

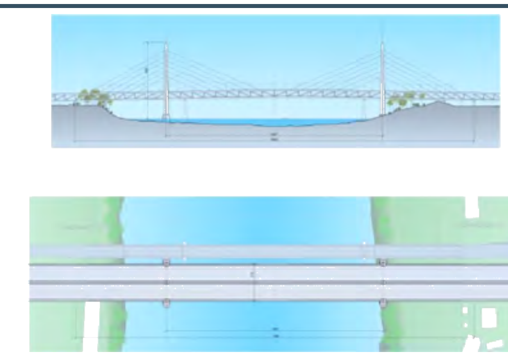
Through a series of design meetings with the FHWA, KYTC, and ODOT, the 6 Bridge Type Alternatives were further refined for conformance to the design parameters and to determine which best met the design guidelines of the project. During this process, each of the 6 Bridge Type Alternatives was evaluated for construction cost, constructability/construction time, maintenance and durability, major rehabilitation feasibility, and maintenance of traffic. Renderings and computer visualizations showing different views and details were developed for each of the 6 Bridge Type Alternatives. At the end of Step 2, the 6 Bridge Type Alternatives were reviewed and approved by FHWA, KYTC, and ODOT. The agencies concurred that the following 6 Bridge Type Alternatives best met the objective of Step 2:



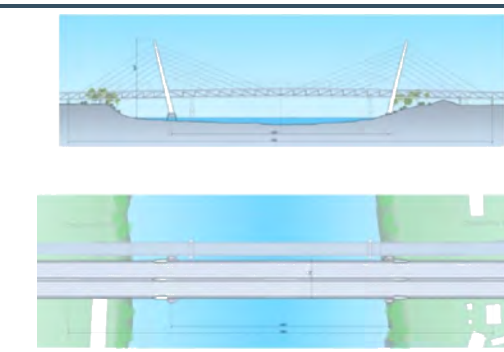
Alternative 1 Arch Bridge:
Simply supported arch with inclined arch ribs



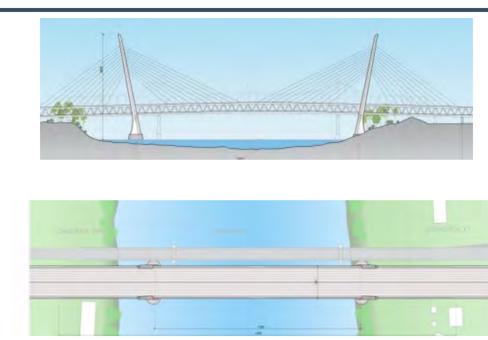
Alternative 2 Arch Bridge:
Continuous arch with vertical arch ribs



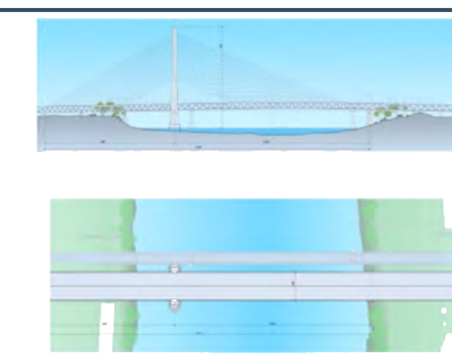
Alternative 3 Cable Stayed Bridge:
Two towers, three vertical legs/tower



Alternative 4 Cable Stayed Bridge:
Two towers, three inclined legs/tower

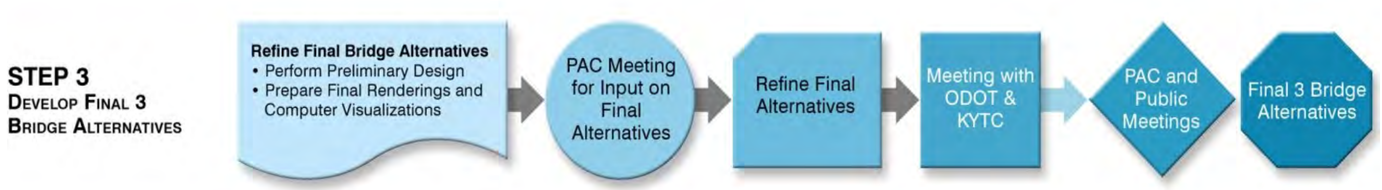


Alternative 5 Cable Stayed Bridge:
Two towers, two inclined legs/tower



Alternative 6 Cable Stayed Bridge:
One tower, two vertical legs/tower

6.2.3 Step 3 - Develop Final 3 Bridge Alternatives



The objective of Step 3 was to:

- Present the 6 Bridge Type Alternatives approved during Step 2 to the PAC and public to gain feedback to support selection of the bridge type alternatives recommended as the Final 3 Bridge Alternatives for preliminary design in Step 3;
- Perform significant preliminary design on the Final 3 Bridge Alternatives;
- Revise and develop additional renderings and computer visualizations of the Final 3 Bridge Alternatives, including animations;
- Prepare cost estimates for the Final 3 Bridge Alternatives;
- Present the Final 3 Bridge Alternatives at two public meetings; and
- Complete the Bridge Type Selection Report.

Step 3 began with the presentation of the 6 Bridge Type Alternatives to a combined AC and PAC meeting on April 15, 2010. The purpose of the meeting was to receive feedback on the 6 Bridge Type Alternatives to aid the bridge design team in selecting the Final 3 Bridge Alternatives. Key visual and aesthetic criteria previously established were used by the PAC to evaluate the 6 Bridge Type Alternatives.

The 6 Bridge Type Alternatives were also posted on the project website to solicit public comment as well. Following the PAC meeting, the public was provided a one-week comment period to submit feedback. Comments received indicated that the public is in favor of both the arch type bridges as well as the cable stayed bridge types with no clear preference for either.

Based upon the results of the PAC meeting and public outreach efforts, the following Final 3 Bridge Alternatives were selected and approved by FHWA, KYTC and ODOT for further study during Step 3:

- Alternative 1: Tied Arch Bridge;
- Alternative 3: Two Tower Cable Stayed Bridge; and
- Alternative 6: One Tower Cable Stayed Bridge.

Additional technical analyses for the Final 3 Bridge Alternatives was also presented to the AC on December 17, 2010. To date, the bridge design team has not received additional comments from the AC. The Final 3 Bridge Alternatives are depicted in the following figures:

Alternative 1: Tied Arch Bridge

Alternative 3: Two Tower Cable Stayed Bridge

Alternative 6: One Tower Cable Stayed Bridge

In Step 3, the bridge design team assessed the suitability of the Final 3 Bridge Alternatives based on more detailed examination of the structural requirements, cost, constructability, environmental impacts, aesthetics, and other key criteria. This assessment included performing significant preliminary design and preparing additional renderings for the Final 3 Bridge Alternatives.

While each of the Final 3 Bridge Alternatives has distinct characteristics, there are some elements common to all. The following is a list of these common elements:

- A bridge alignment adjacent to and downstream (west) of the existing Brent Spence Bridge;
- A double-decked truss superstructure carrying two roadways on each deck, with each roadway composed of two or three 12-foot-wide lanes and two 14 foot-wide shoulders;
- An approximately 1,000-foot main span with piers outside of the main span piers of the existing Brent Spence Bridge;
- A river to superstructure clearance no lower than that of the existing Brent Spence Bridge, and
- A bridge to work in conjunction with the existing Brent Spence Bridge, to carry the Design Year 2035 traffic projection of approximately 233,000 vehicles per day.

Table 6-1 presents an evaluation matrix which compares several features considered in Step 3.

Table 6-1. Evaluation Matrix

Bridge Type Alternatives		Criteria			
		Construction Cost	Constructability	Maintenance and Durability	Major Rehabilitation Feasibility
1		KY: $484.6 M OH: $86.1 M Total: $570.7 M	Construction will be complicated by the inclined arch and slowed by the requirement to maintain river traffic.	Items included in M&D will be: 1. Standard Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Hanger Replacement
3		KY: $538.0 M OH: $130.6 M Total: $668.6 M	Cantilever construction of the superstructure will minimize interference to river traffic.	Items included in M&D will be: 1. High-Tech Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Stay-Cable Replacement
6		KY: $478.6 M OH: $167.8 M Total: $646.4 M	Cantilever construction of the superstructure will minimize interference to river traffic.	Items included in M&D will be: 1. High-Tech Inspections 2. Overlay Replacement 3. Painting of Steel	Items included in rehab will be: 1. Deck replacement 2. Future Widening 3. Stay-Cable Replacement

6.3 Engineering Analysis

In order to minimize impacts to Longworth Hall, a National Register of Historic Places listed resource, the proposed bridge regardless of its type (i.e., Tied-Arch or Cable Stayed) will be a double deck configuration. Long span double decked bridges are not common, and require detailed structural analysis and evaluation.

Due to right-of-way constraints, the new bridge must be located about 50 feet west of the existing Brent Spence Bridge. The main span length of the Brent Spence Bridge is 830-feet 6-inches. The proposed bridge's main span will be approximately 1,000 feet to avoid interference between the new tower or main pier foundations and those of the existing structure.

This section provides information about the engineering analysis that was performed during Step 3.

6.3.1 Structural Analysis

A series of analyses were conducted on structural models developed for the Final 3 Bridge Alternatives. First, dead load and live load analyses were performed to justify the sizes of all major structural members. Secondly, wind load and seismic load analyses were performed to ensure that the structure will be adequate to satisfy the American Association of State Highway Transportation Officials (AASHTO) code requirements. Span uplift on the cable stayed bridges was studied in detail. Counterweights were used to eliminate the uplift to ensure the safety and stability of the structure.

Barge impact was a major consideration in the structural design of the Final 3 Bridge Alternatives. The towers of the cable stayed alternatives and the pier of the tied arch alternative are made of relatively slender columns, which are vulnerable to barge impact. As a protective measure, a wall will be built to link the three columns together to resist the barge impact force. Pointed ends of the wall will minimize both backwater rise and barge impact force. The top of the wall will be at the 100-year flood water elevation to eliminate the risk of structural failure due to runaway barge impact.

Due to the high expected loads, drilled shaft foundations were selected for their high vertical and horizontal load capacities. Eight-foot diameter shafts were assumed for each alternative to simplify the study process. Larger or smaller shafts may be used in the final design of the selected alternative, after more detailed structural analysis and more geotechnical data become available.

Wind loads and buckling are critical to the cable stayed alternatives needle towers. Both load conditions were analyzed using non-linear analysis methods. During the analysis, the sizes of the needle towers were confirmed to be adequate.

For each Final 3 Bridge Alternative, several major or unique structural details were designed to a greater degree in order to insure their feasibility. Those details were:

- Alternative 1, Tied Arch
  - Angle change in arch rib at top chord of deck truss (arch knuckle). This detail equalizes the tie force between the top and bottom chords.
  - Approach spans flanking the arch span designed to be integral with arch. Keeping these spans integral will move an expansion joint out of a congested and sensitive area.

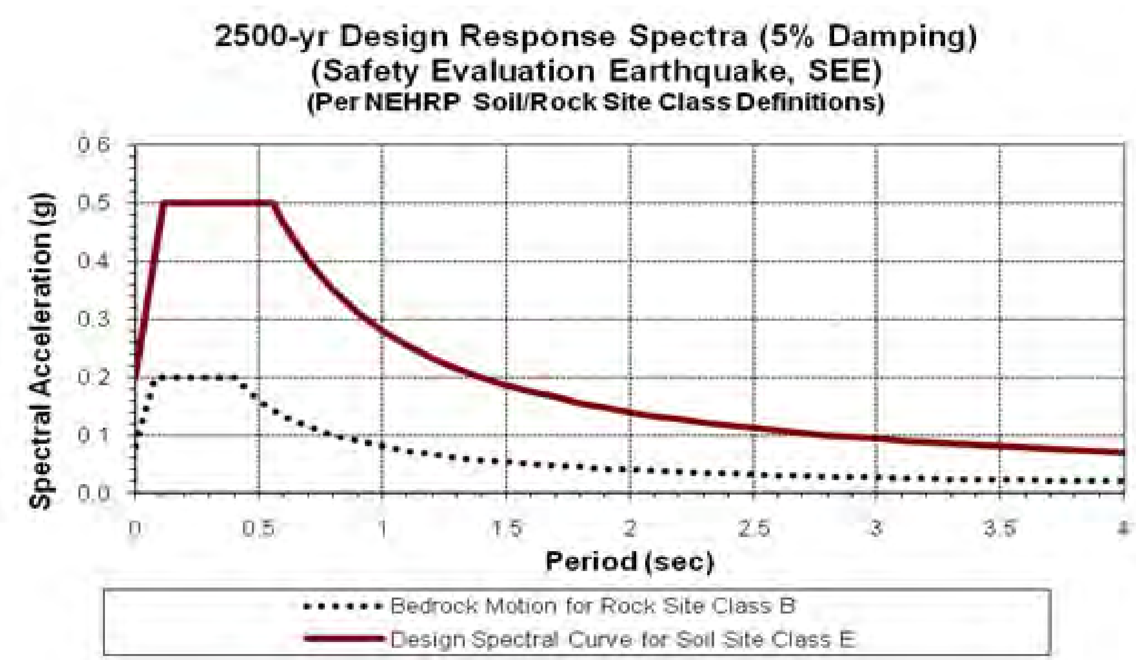
- Alternative 3, Two Tower, Three Needle Cable-Stay
  - Deck trusses are bolted directly to needle towers; though this adds thermal stresses to the lower leg of the tower and its foundations, it will eliminate several high-load bearings.
  - Truss diagonals and stay cables are aligned. Aligning the diagonals and stay cables is not only aesthetically pleasing, it also helps split the horizontal component of the stay force between the top and bottom chords of the truss.

- Alternative 6, Single Tower, Two Needle Cable-Stay
  - Trusses are bolted directly to needle towers to eliminate several high-load bearings. In contrast with Alternative 3, this will not add thermal stresses to the lower leg of the tower and its foundations.
  - Tower cross section designed to minimize eccentricity of cable loads. Because the tower to cable connections is offset from the centerline of the needle towers, the tower cross section is designed to shift the neutral axis of the tower to coincide with the center of the cable connections as much as possible.

6.3.2 Seismic Analysis

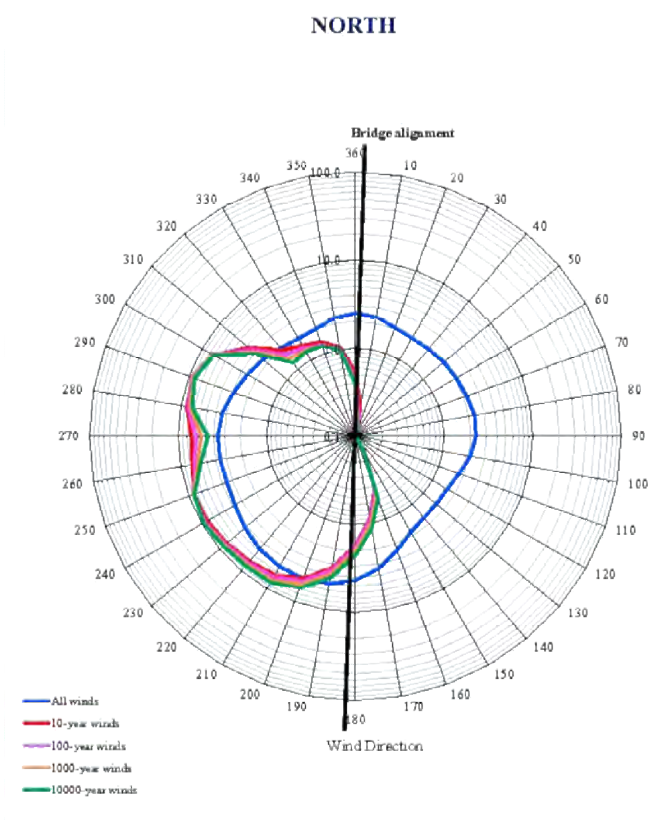
To verify the safety of the new Ohio River Bridge in the event of an earthquake, a seismic analysis was performed for the Final 3 Bridge Alternatives. The results confirmed that the structures of all three can be detailed to satisfy the AASHTO requirements for the proposed bridge location. At this stage of the design, without all structure details complete, only an approximate analysis could be performed.

Site specific rock and soil response spectra were developed for the Final 3 Bridge Alternatives. The bridge site was classified as "Site Class E" using the National Cooperative Highway Research Program (NCHRP) site class definitions and the soil data presented in available boring logs. Using Site Class E, the design response spectral curve for a 2,500-year return period earthquake is presented in the following figure.



6.3.3 Wind Analysis

A site specific wind climate analysis was performed to determine the expected wind velocity at the bridge site. The long-term wind records collected at the Cincinnati-Northern Kentucky International Airport in Covington, Kentucky were the primary source of data used for this analysis. The data were modified to account for the open water and the terrain in the area of the bridge location. The modified wind values were then used to develop site-specific recommended wind speeds and turbulence properties for each alternative. In addition, the impact of the hills southwest of the bridge location was also investigated and determined to have a negligible effect on the wind flow.



A static wind analysis was performed to confirm that the structures of the Final 3 Bridge Alternatives are adequate to resist the design wind load. During final design, dynamic wind loads analysis, including wind tunnel tests, will be performed.

6.3.4 Construction Cost Estimates

The sizes and details of the major structural members were developed based on more detailed analyses than those performed in Steps 1 and 2. More accurate and detailed quantities were computed in Step 3. In addition, an anticipated construction method was developed for each alternative. These quantities and construction methods were used as the basis of the updated construction cost estimates presented in the Appendix.

The cable stayed and arch main spans of the Final 3 Bridge Alternatives are of different lengths. To make a true cost comparison, the bridge cost estimates of the three alternatives were based on the cost of the main bridge unit plus the approaches covering 2,200 feet between the same two fixed points.

To facilitate the cost comparison, the approach spans were designed based on the assumptions that the approach span superstructure will consist of spans of 200 feet. Using a generic 200-foot approach span as a basis, the per-foot unit cost of the approach spans was computed. The total approach span construction cost of each of the Final 3 Bridge Alternatives was computed based on this per-foot unit cost and the total approach length of each alternative.

Construction costs are based on 2010 costs inflated to the median construction date for each bridge alternative with an anticipated start of construction date of January 2016. An inflation rate of 37.6 percent was used for Alternative 1 based on a three year estimated construction schedule with a median construction date of June 2017. An inflation rate of 41.0 percent was used for Alternatives 3 and 6 based on a four year estimated construction schedule with a median construction date of January 2018. The ODOT Fiscal Year 2010-2011 Business Plan Inflation Calculator was used to calculate the inflation rates.

A contingency was added to the construction cost of all Final 3 Bridge Alternatives to cover construction risk. For Alternative 1, an additional contingency was added to cover the arch erection works on barge. For Alternative 6, an additional contingency was added to cover the tall needle tower.

6.3.5 Constructability

Constructability was considered in the general layout and detail development of the Final 3 Bridge Alternatives. Drilled shaft construction in rivers is traditional and special constructability issues are not expected. The footing construction will occur within sheet pile cofferdams, which is also a traditional construction method. The concrete towers and pier columns are anticipated to be cast-in-place using self-climbing forms.

The steel tied arch would be erected on a temporary pile-supported platform. The north river bank on the west side of the bridge site is ideal for the temporary platform. When the steel arch and its ties and floorbeams are assembled, it could be placed on one or two barges. The barge(s) would be towed to the bridge site. Water would be pumped into the barge(s) to lower the arch to six bearings. Concrete deck construction and approach truss erection would continue after the steel portion of the superstructure has been placed on the piers.

The deck truss of the cable stayed alternatives would be bolted directly to the tower. Erection of the deck trusses, floorbeams and stringers would use floating cranes and/or deck gantries. Due to the weight limits on the lifting equipment, the superstructure would likely be erected piece by piece. The superstructure erection would require more time than for the tied arch, but the construction operations would not block river traffic.

Architectural design suggests box-shaped truss members. The connection of the diagonal, wind bracing and others to the chords were studied and properly sized to ensure access for welding and bolting. Special attention was paid to the tied arch knuckle, where the arch ribs and the deck truss come together to ensure that the fabrication of the stiffeners and deck truss chord in the knuckle area is feasible.

Construction schedule was considered in the analysis of the Final 3 Bridge Alternatives with regards to constructability and the construction cost estimates. The construction schedule for Alternative 1 was based on the offsite construction/float-in method of construction of the main span. The float-in construction method is assumed and is expected to take approximately 2.5 to 3 years. The construction schedule for Alternatives 3 and 6 was based on cantilever construction methodology. Once the needle tower construction reaches the elevation of the first or second cable, the construction of the deck trusses/decks can proceed simultaneously with the construction of the needle towers, which is assumed in the construction schedule for these alternatives. The total estimated time in the schedule is less than the sum of the individual items schedules, which indicates the time savings due to the overlap of the construction of the pylons and superstructure. Alternatives 3 and 6 are expected to take approximately 3.5 to 4 years to construct. Alternative 6 with the single taller tower may have additional schedule implications associated with it due to the superstructure erection being dependent on the single tower on the critical path for construction.

Table 6-2.: Approximate Construction Schedule Durations

Construction Element	Alternative 1	Alternative 3	Alternative 6
Foundation	11 – 12 Months	11 – 12 Months	11 – 12 Months
Pier/Tower	6 Months	14 Months	19 Months
Superstructure	10 – 15 Months	20 – 25 Months	15 – 20 Months
Finishing	3 Months	3 Months	3 Months
Total	2.5 – 3 Years	3.5 – 4 Years	3.5 – 4 Years

6.3.6 Maintenance/Rehabilitation

The design approach focused on providing easy inspection access and the simplification or elimination of elements which would require more maintenance. The box-shaped chord and diagonal members were sized to provide manholes and hand holes for inspection and repairs. Bolted connections were used whenever a welded connection could be avoided.

Cable stayed bridges usually have tie-down devices to overcome span uplift. Any type of tie-down requires high intensity inspection and maintenance and future replacement will be virtually impossible. Our cable stayed alternatives use concrete counterweights permanently bonded to the steel superstructure in order to completely eliminate tie-downs and their maintenance requirements.

Pot bearings are usually used for heavy loads. However, the pot bearing has moving parts, which are subjected to wear and tear. Bearing failure and replacement are thus expected. Disc bearings, a newer bearing technology, use high compressive load capacity plastic disks as load carrying members, and are recommended to be used in this bridge. The disc bearings used to support the tied arch span have no moving parts and are expected to last much longer.

The 14-foot shoulders on both sides of each roadway on all alternatives will provide space for maintenance of traffic during inspection and maintenance operations. Alternative 6 provides a greater flexibility for maintenance of traffic than Alternatives 1 and 3 because there is not a center deck truss component, which, if present, prevents switching of traffic from one lane to another across the median of the bridge.

6.4 Aesthetics

During the PAC meetings, a series of important considerations in relation to the aesthetics of the Brent Spence Bridge were defined. These include the following:

- The new bridge should be visually attractive;
- The new bridge should be visible looking "through" the existing bridge (from the east);
- As much as possible, crossing the new bridge should allow views of the surrounding context (unlike existing bridge);
- The new bridge should have distinctive characteristics that identify it as a local landmark; and
- The new bridge should have a visual relationship with the existing bridge.

6.4.1 Bridge Type Selection Process Step 1

In this initial step, 12 preliminary bridge concepts were developed including cable stayed, arch and truss bridges. All of the initial 12 concepts complied with the fundamental criteria for a main span over the Ohio River of a minimum of 1,000 feet and the required navigational vertical clearance over the water of approximately 70 feet. The initial 12 concepts included seven cable stayed bridges, three arch bridges and two truss bridges. The main differences among the seven cable stayed bridge concepts were the tower shapes, the cable arrangements and the geometry of the trusses between the upper and lower decks. Some of the tower shapes explored included "A" shape, needle and inclined tower configurations. Harp and semi-fan cable arrangements were also explored. The three arches were all tied arches with both vertical and tilted arches above the deck.

Of the two truss bridges, one resembled the existing Brent Spence Bridge in terms of structural layout and the other was a parallel chord truss.

6.4.2 Bridge Type Selection Process Step 2

After a comprehensive review of the initial 12 preliminary bridge concepts and analysis of feedback received from the PAC and the general public, 6 Bridge Type Alternatives were developed for further review and study. The 6 alternatives included two arches and four cable stayed bridges. The trusses were eliminated from further consideration due to both maintenance and aesthetic related issues. One of the arches was a tied arch configuration and the other one was a through arch. The tied arch had inclined arch ribs and the through arch had vertical arch ribs.

Of the four cable stayed bridges, two had vertical needle towers and two had inclined needle towers. Harp and semi-fan cable arrangements were represented in the four options. A system of either two or three towers across the bridge cross section was studied. The bridges with only two towers at the edges of the bridge deck require deep girders to span the multiple lanes. The bridges with three towers, locating one tower at the median between opposing direction lanes require shallower transversal beams.

6.4.3 Bridge Type Selection Process Step 3

The 6 Bridge Type Alternatives presented to the PAC were also made available for public comments during Step 3 of the Bridge Type Selection Process. Comments received from the public were varied. In general, comments showed that the public is in favor of both the cable stayed bridge types as well as the arch-type bridges, with no clear preference. Cable stayed bridges were noted as something new and liked as opposed to the arch bridge type, which is already present in the area. Some comments stated that the designs needed to be more impressive. Comments also referred to whether or not the alternatives would fit into the context of the existing landscape. From the public comments received, Alternative 2 was provided the most favorable comments in terms of number of votes. Alternative 4 and Alternative 6 were noted the least favored among the six alternatives.

Additional technical analyses for the Final 3 Bridge Alternatives were presented to the AC on December 17, 2010. To date, the design team has not received additional comments from the AC.

As part of the National Environmental Policy Act (NEPA), public hearings for the Brent Spence Bridge Replacement/Rehabilitation Project will be held in 2011. The focus of the hearings will be the selection of the recommended Preferred Alternative for the highway and the new bridge type crossing the Ohio River. The purpose of the hearings is to provide the public the opportunity to comment on the project, its impacts, and proposed mitigation strategies.

During public hearings, the public will have the opportunity to vote on components of the three bridge alternatives using a hand-held audience response polling system.

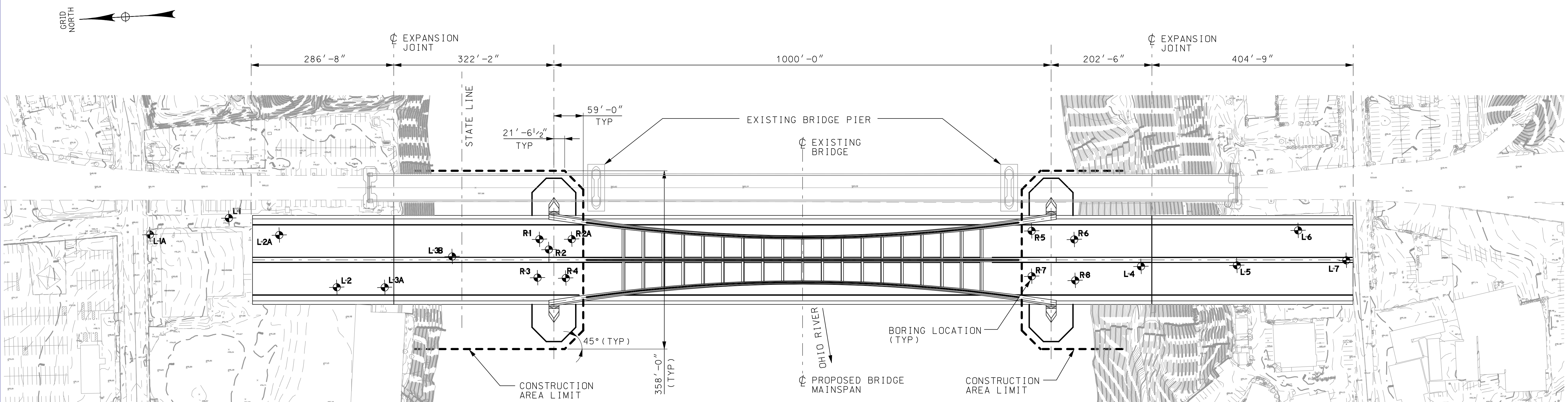
In addition, a comment period of at least 14 days will follow the public hearings. Following the public comment period, the selection of a new Ohio River Bridge type will be determined by KYTC and ODOT in consultation with FHWA. The selection of the preferred bridge type will be based upon consideration of several factors including the technical analyses completed for the project and public input.

6.5 Appendix

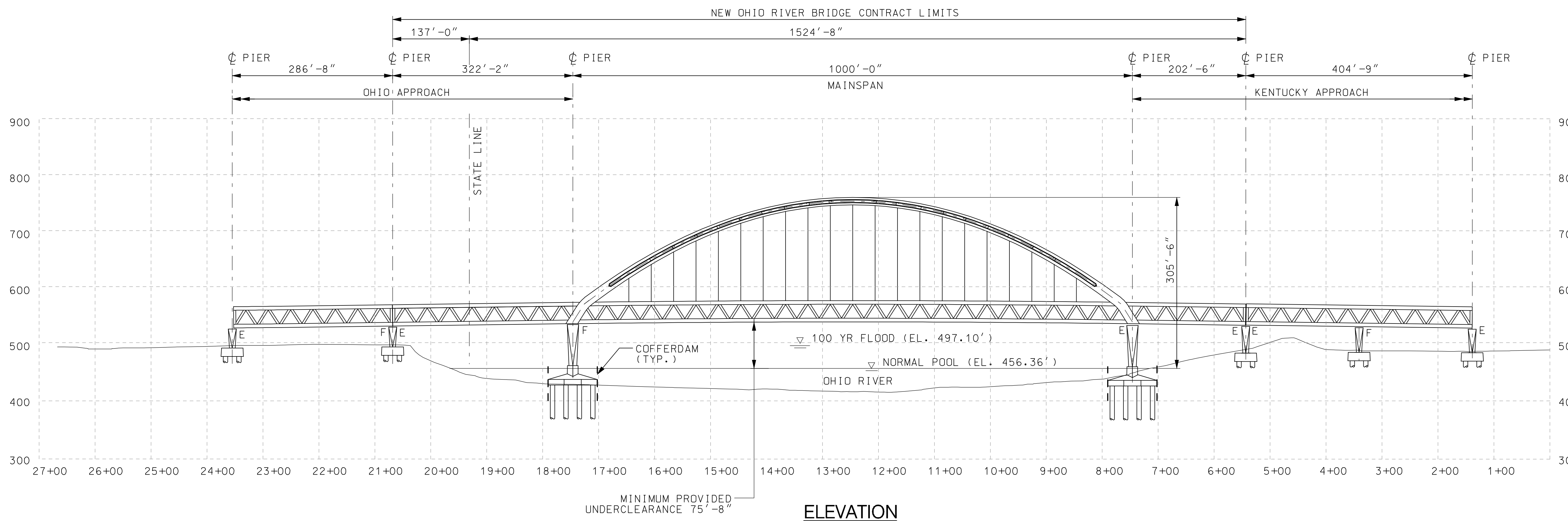
The following documents are provided in hard copy and on the CD enclosed with this report:

6A Engineering Drawings of Final 3 Bridge Alternatives
6B Renderings of Final 3 Bridge Alternatives
6C Geotechnical Report
6D Preliminary Construction Cost Estimates
6E Wind Analysis Reports
6F Photo Simulations

Appendix 6A
Engineering Drawings of Final 3 Bridge Alternatives



PLAN



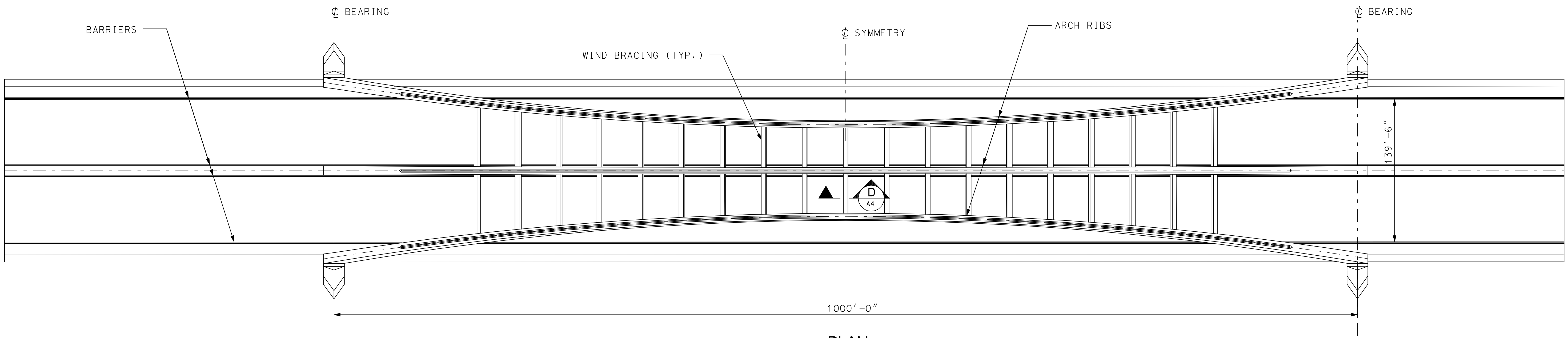
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

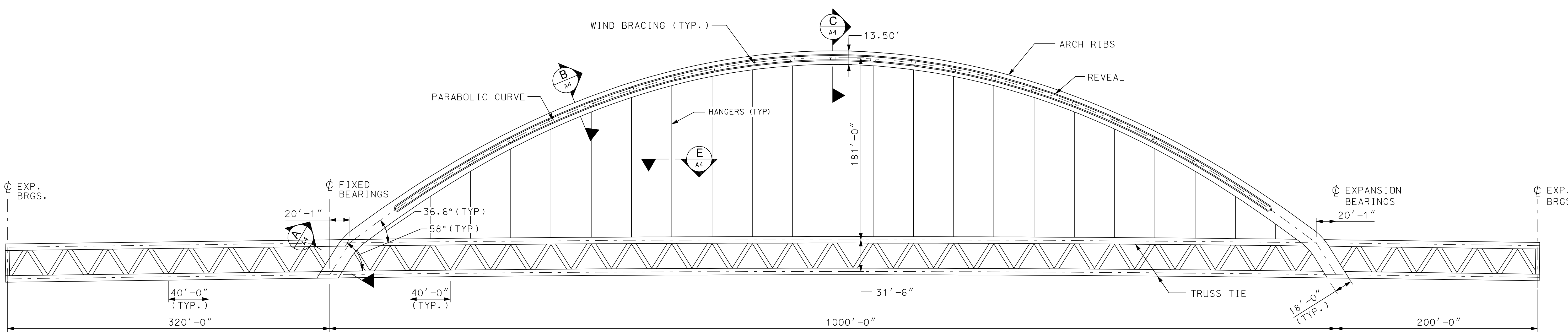
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
GENERAL PLAN AND ELEVATION

EXHIBIT
A1

Prepared by:
PARSONS BRINCKERHOFF



PLAN



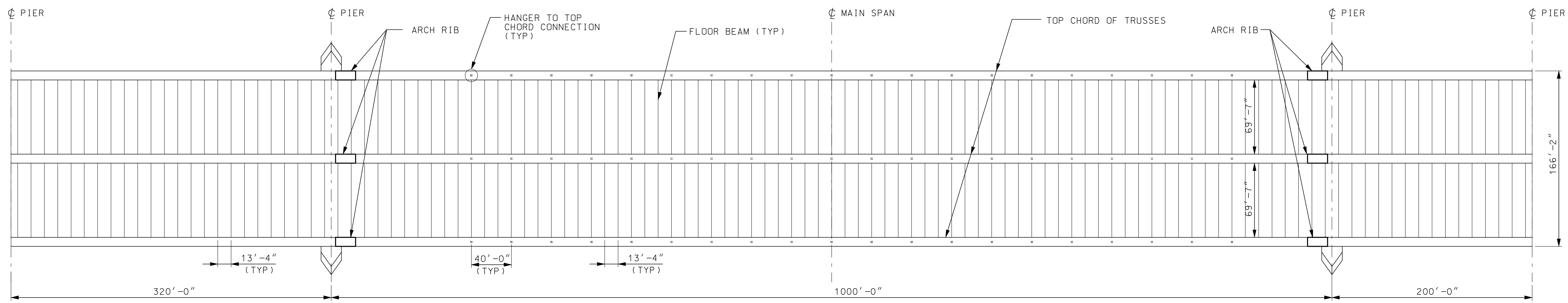
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

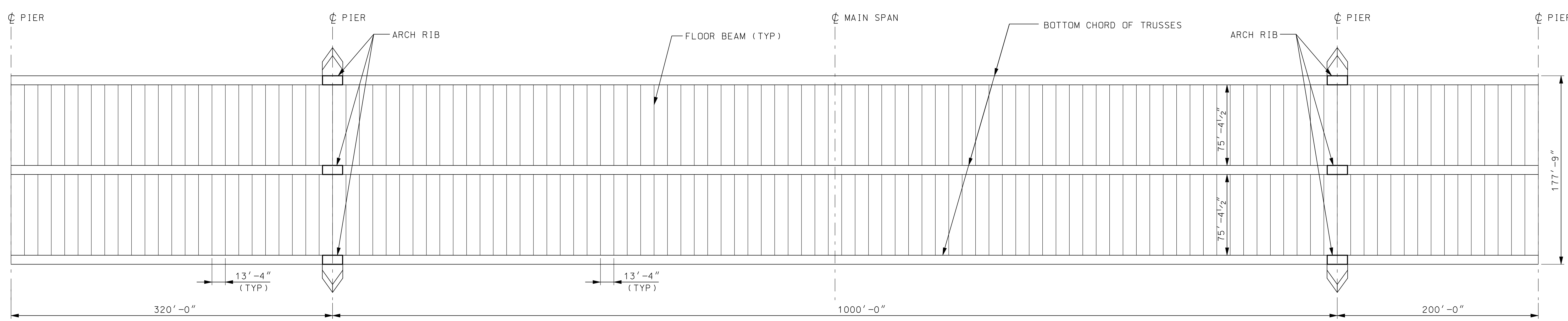
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
SUPERSTRUCTURE PLAN & ELEVATION

EXHIBIT
A2

Prepared by:
PARSONS BRINCKERHOFF



UPPER DECK FRAMING PLAN



LOWER DECK FRAMING PLAN

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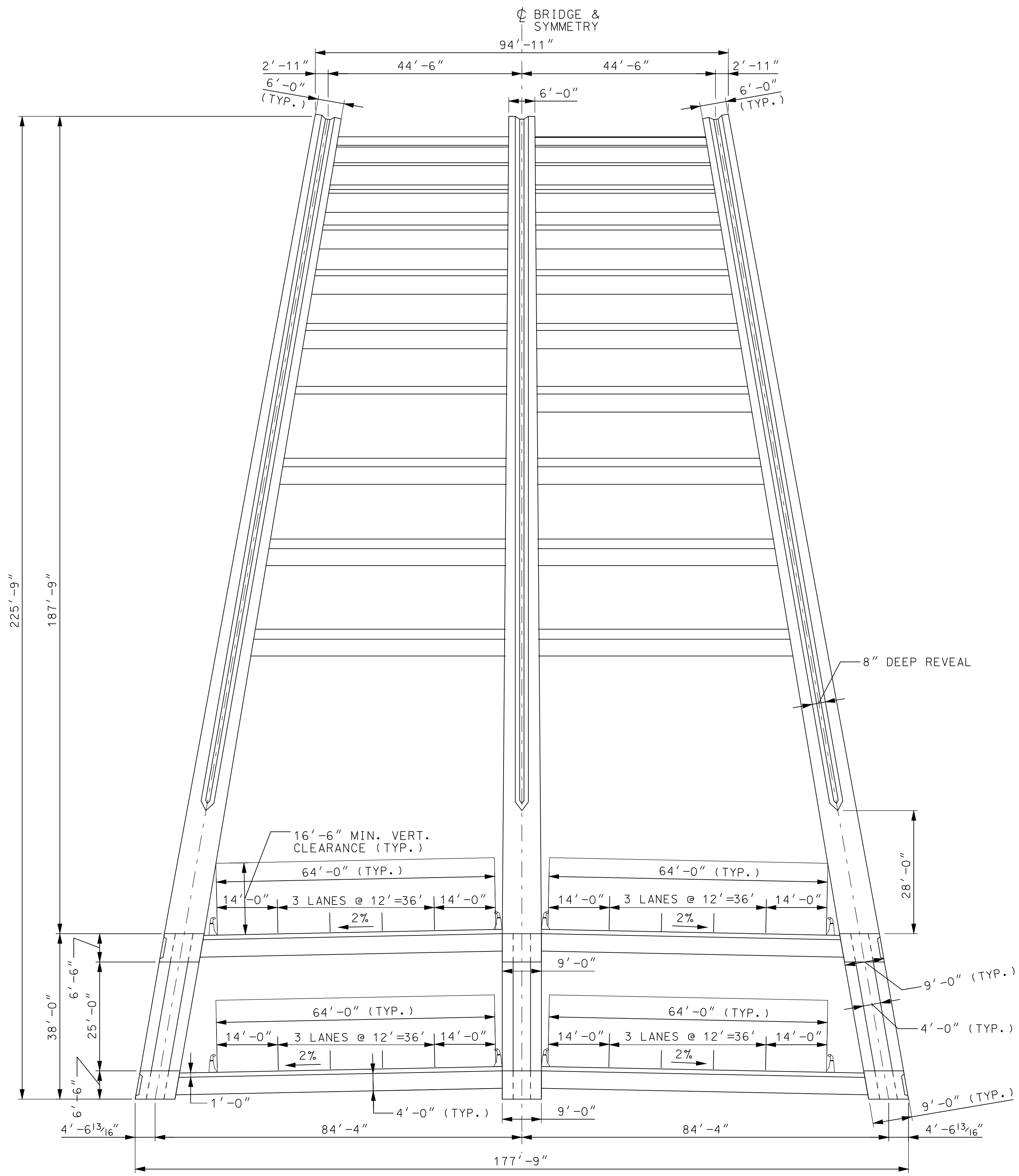
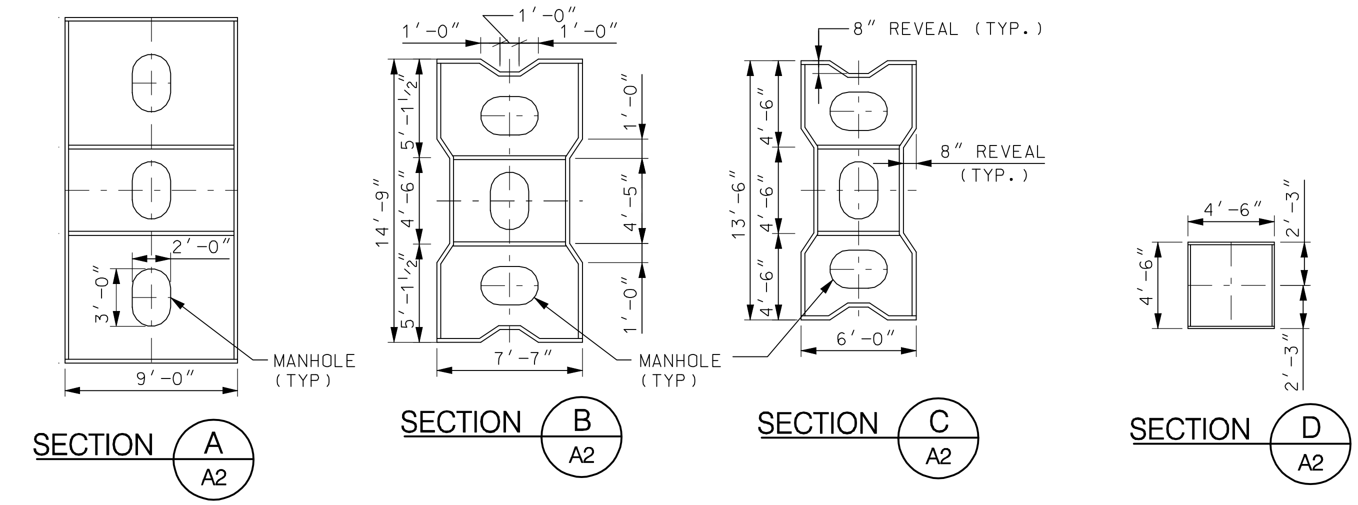
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
FRAMING PLANS

EXHIBIT
A3

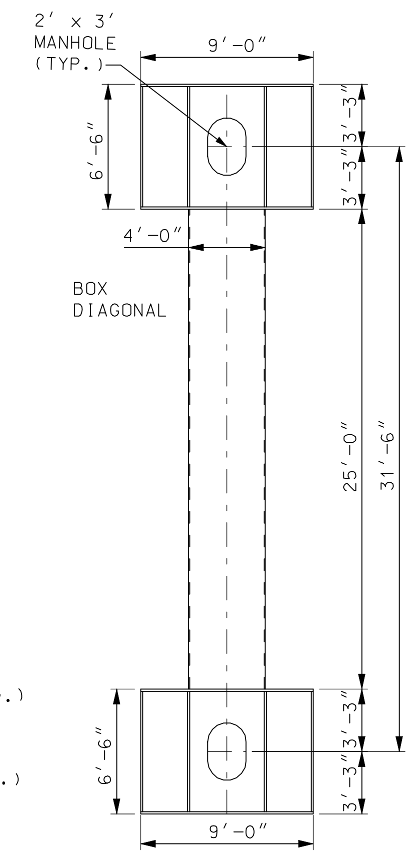
Prepared by:
PARSONS BRINCKERHOFF



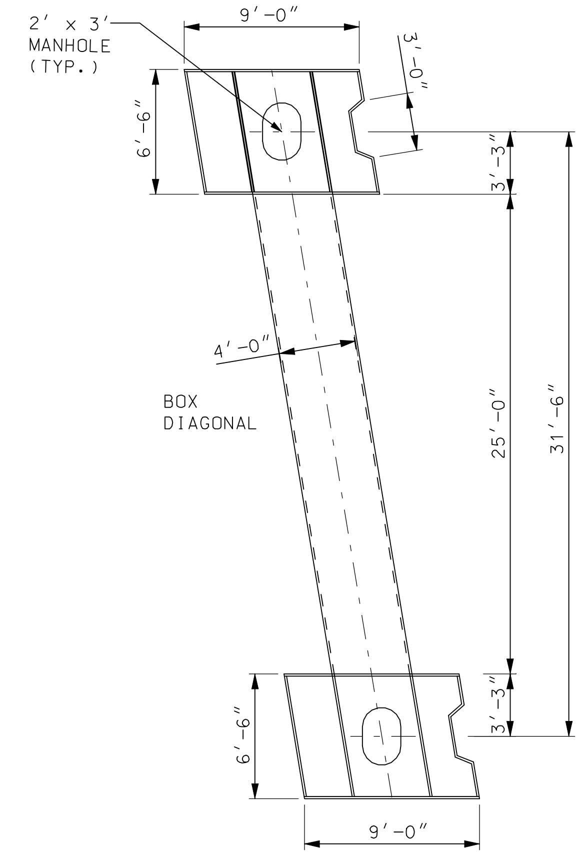
END VIEW



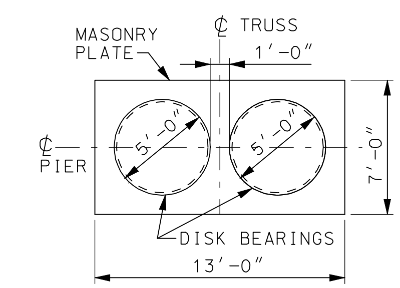
ARCH RIB SECTIONS



CENTER TRUSS CROSS SECTION

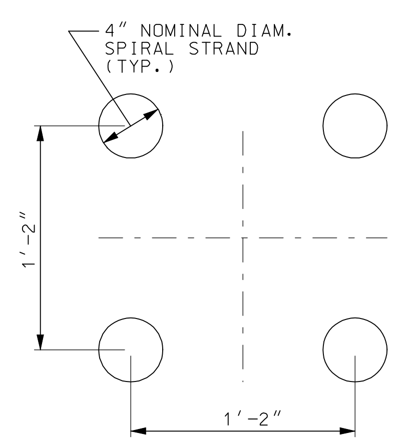


EXTERIOR TRUSS CROSS SECTION



BEARING DETAIL

SCALE: 1″=10′



SECTION E A2

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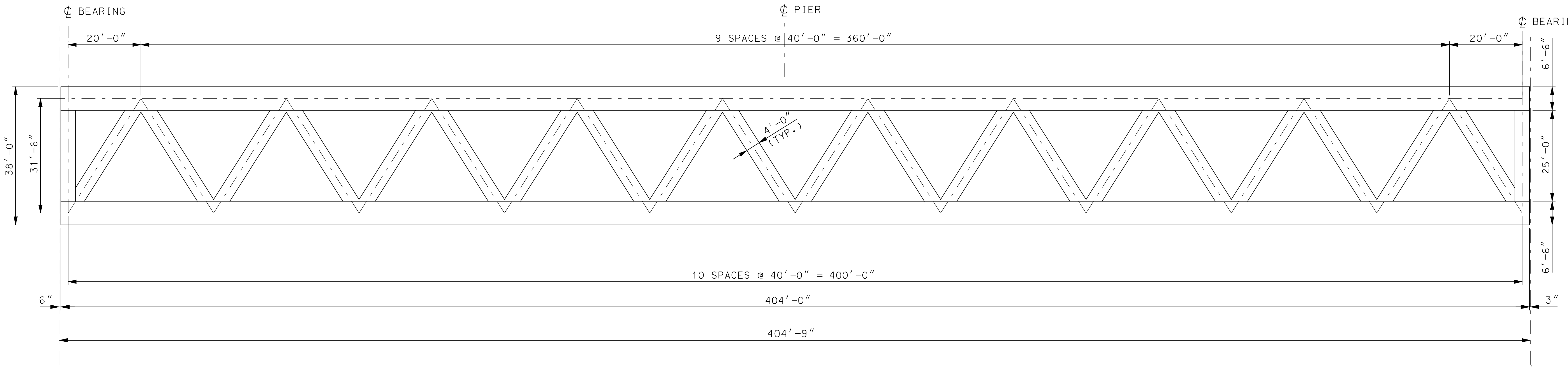
Federal Highway Administration

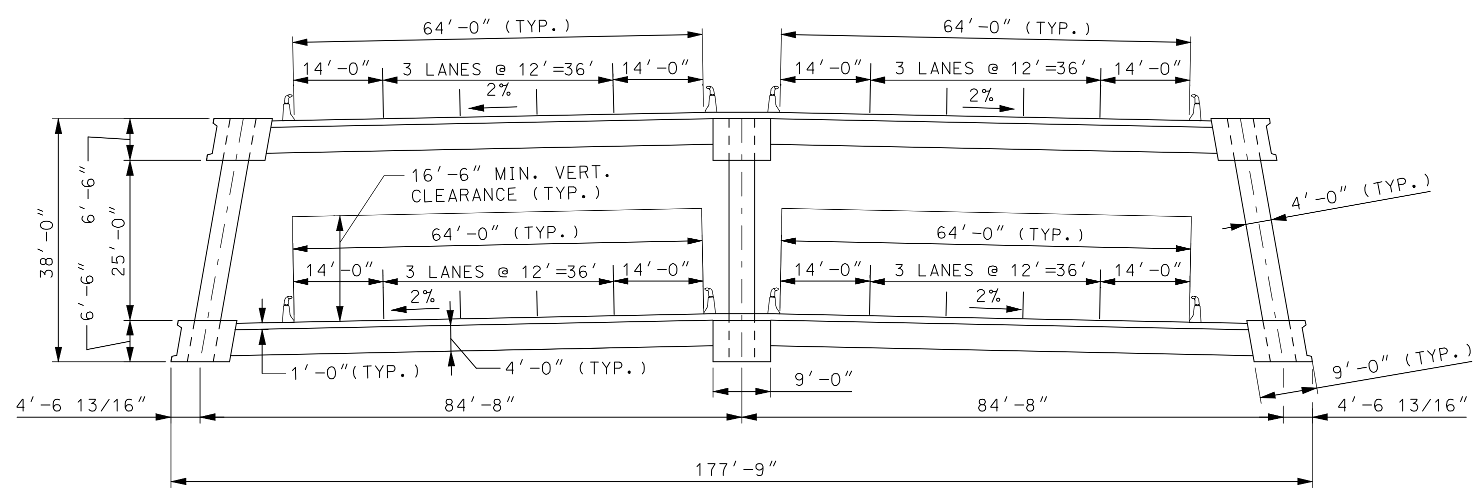
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
TIED ARCH SUPERSTRUCTURE DETAILS

EXHIBIT
A4

Prepared by:
PARSONS BRINCKERHOFF



KENTUCKY APPROACH (OHIO APPROACH SIMILAR)



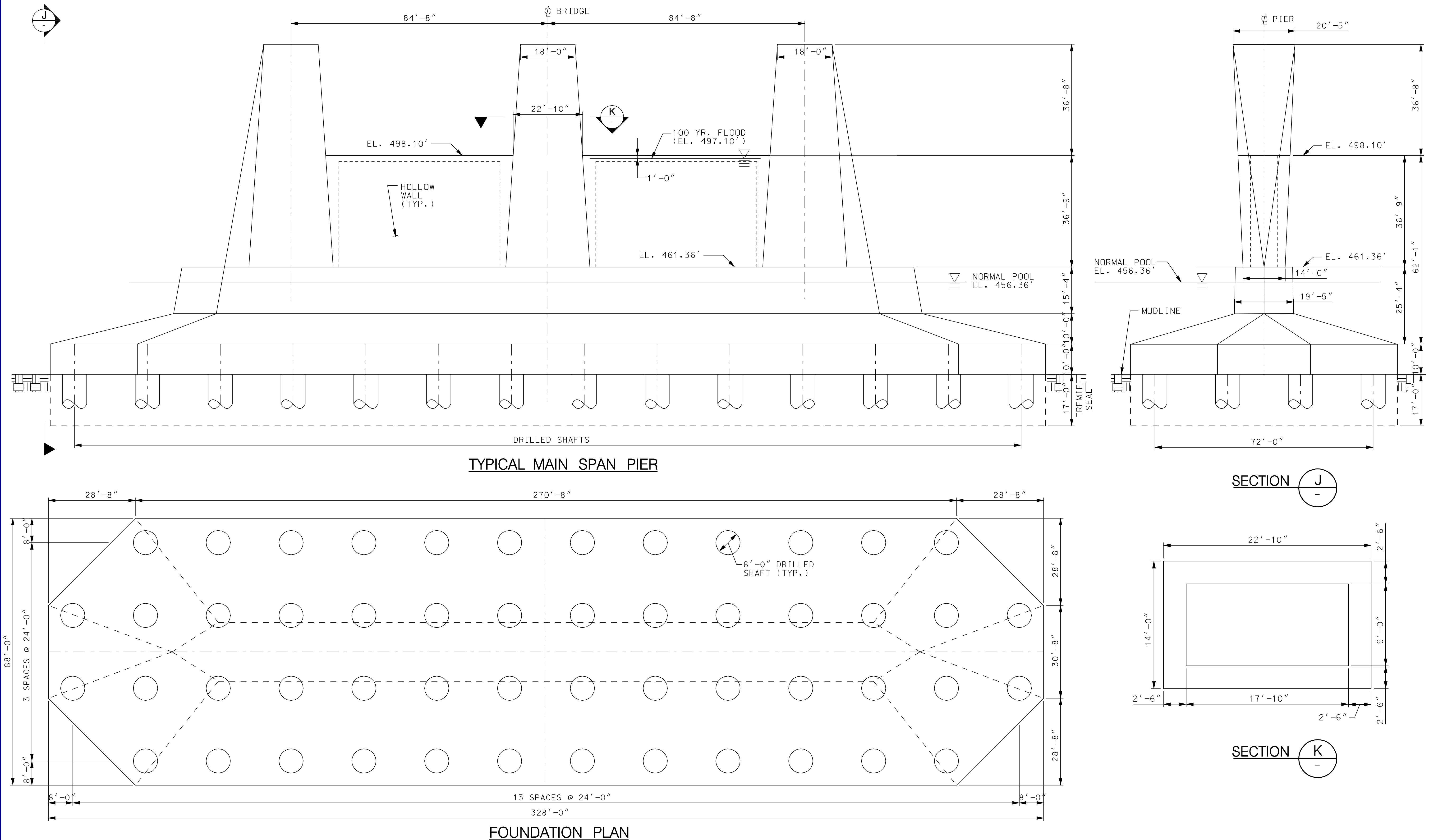
TYPICAL CROSS SECTION

U.S. Department of Transportation
Federal Highway Administration

BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
APPROACH SUPERSTRUCTURE DETAILS

EXHIBIT
A5

Prepared by:
PARSONS BRINCKERHOFF

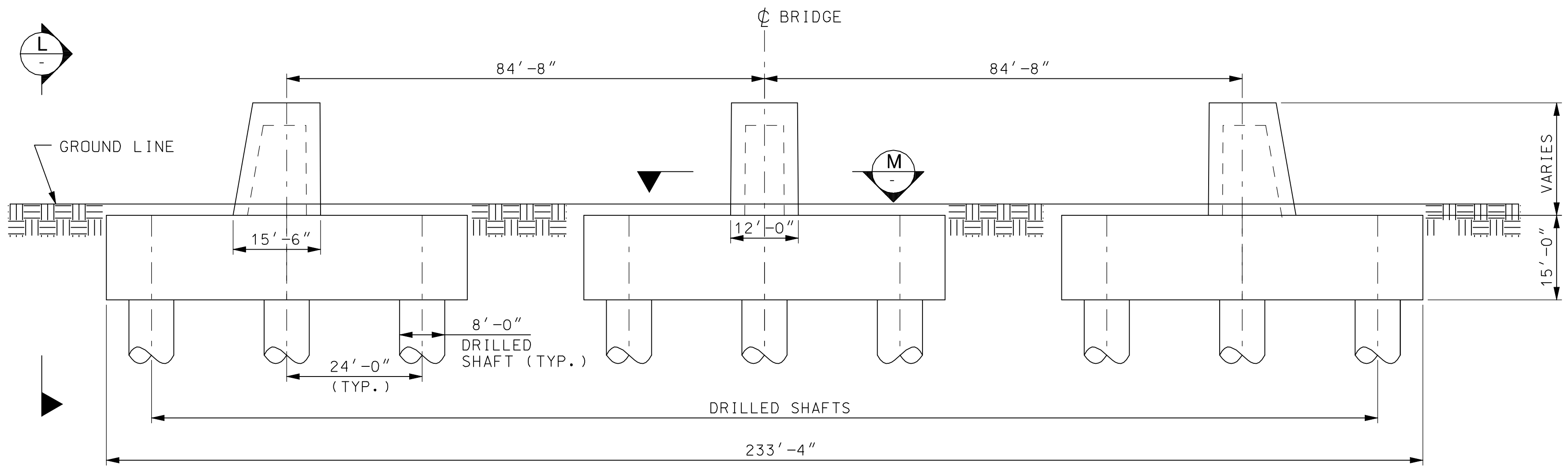


U.S. Department of Transportation
Federal Highway Administration

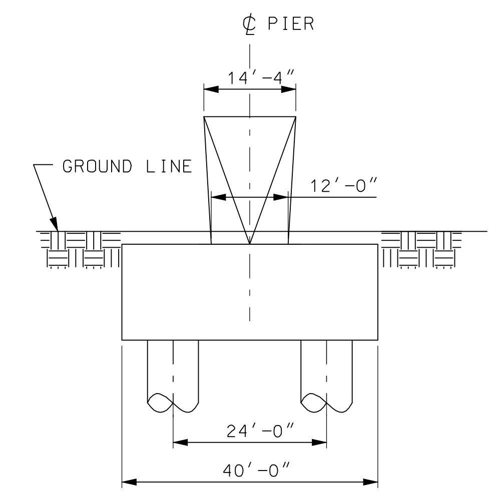
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
TIED ARCH PIER & FOUNDATION DETAILS

EXHIBIT
A6

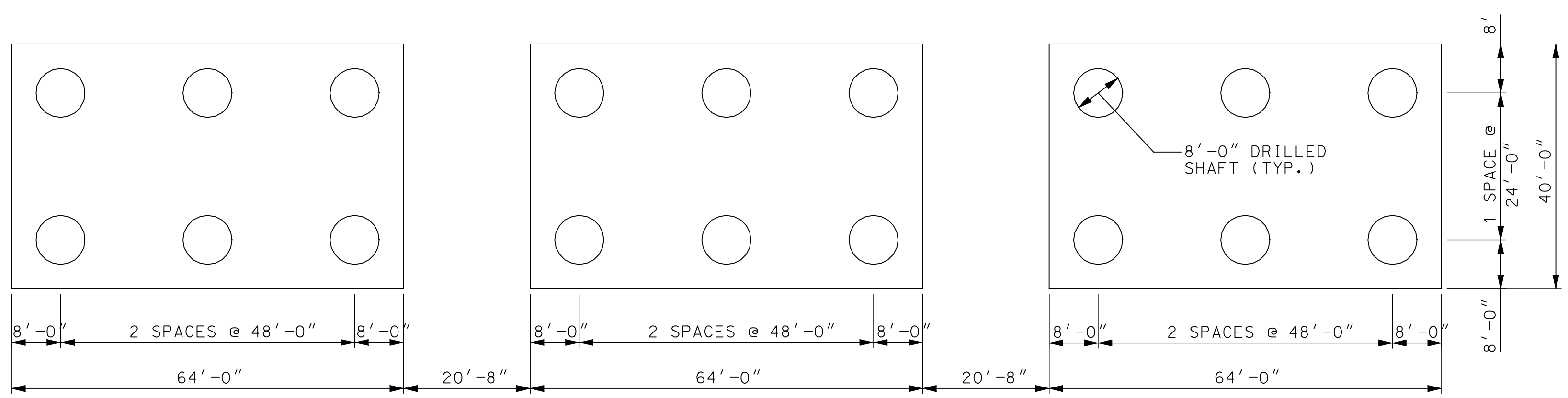
Prepared by:
PARSONS BRINCKERHOFF



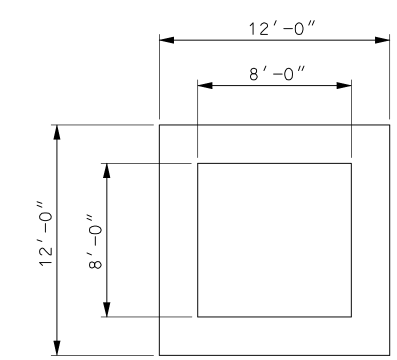
APPROACH LAND PIER



SECTION L



FOUNDATION PLAN



SECTION M

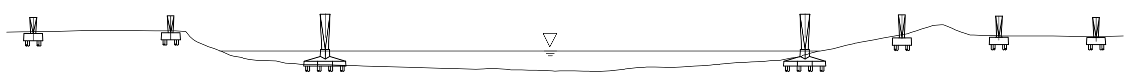
U.S. Department of Transportation
Federal Highway Administration

BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
APPROACH LAND PIER & DETAILS

EXHIBIT
A7

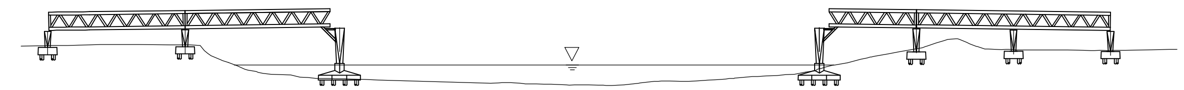
Prepared by:
PARSONS BRINCKERHOFF

STAGE 1

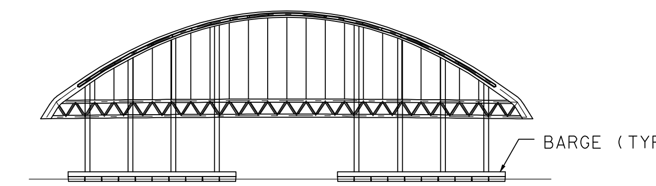


* CONSTRUCT PIERS

STAGE 2

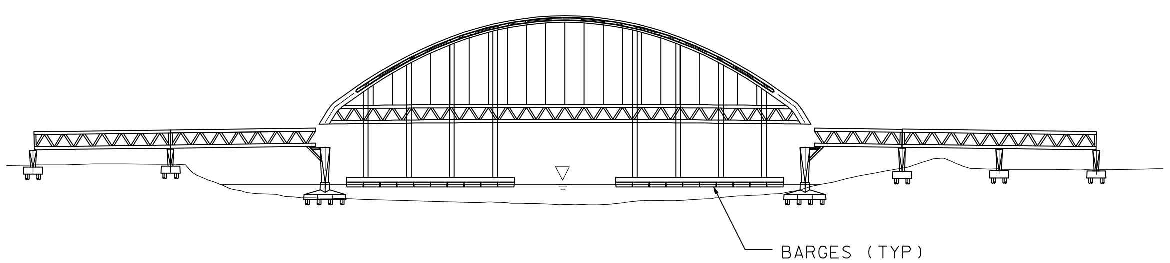


* CONSTRUCT APPROACH TRUSSES USING TEMPORARY SUPPORTS WHERE NECESSARY



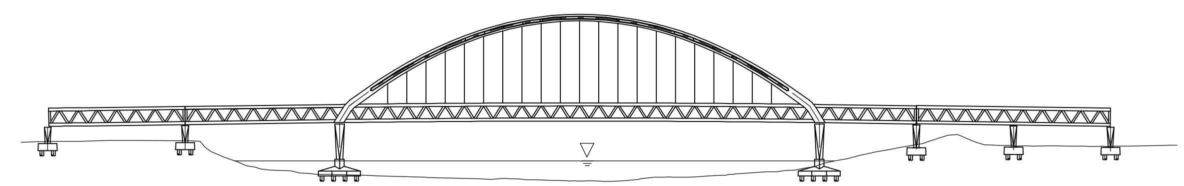
* CONSTRUCT MAIN SPAN ARCH OFF SITE ON BARGES OR TEMPORARY TRESTLE

STAGE 3



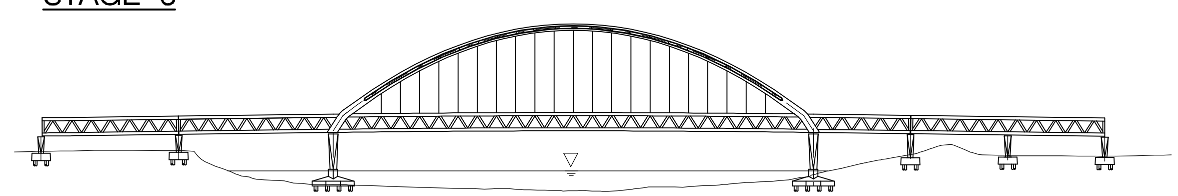
* TRANSPORT MAIN SPAN ARCH TO SITE BY BARGES
* LOWER MAIN SPAN ARCH ON TO MAIN PIERS

STAGE 4



* COMPLETE CONNECTIONS BETWEEN APPROACH TRUSSES AND MAIN SPAN ARCH

STAGE 5



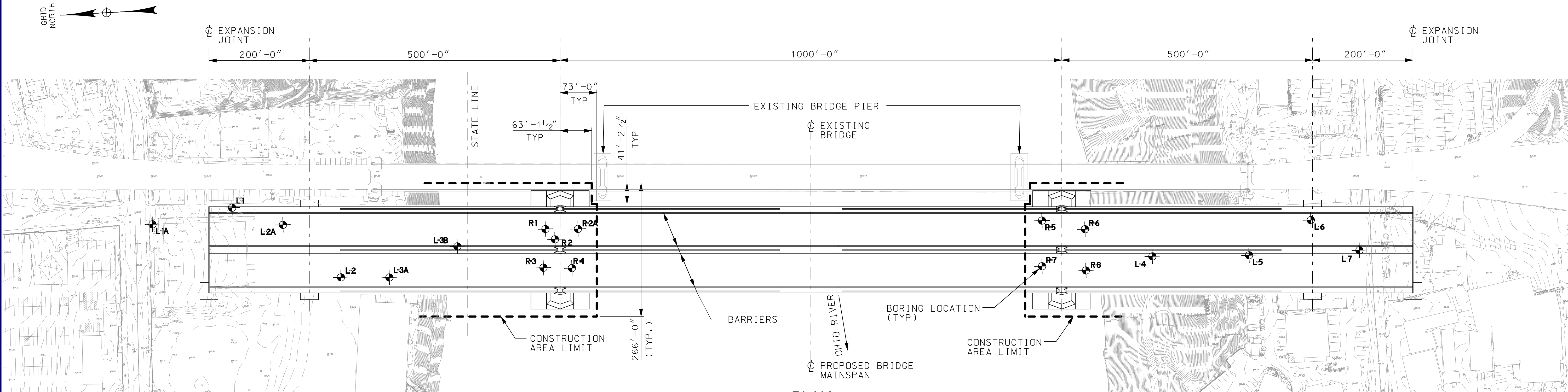
* PLACE CONCRETE DECK FOR MAIN SPAN ARCH AND APPROACH SPANS
* INSTALL BARRIERS AND PLACE DECK OVERLAY

U.S. Department of Transportation
Federal Highway Administration

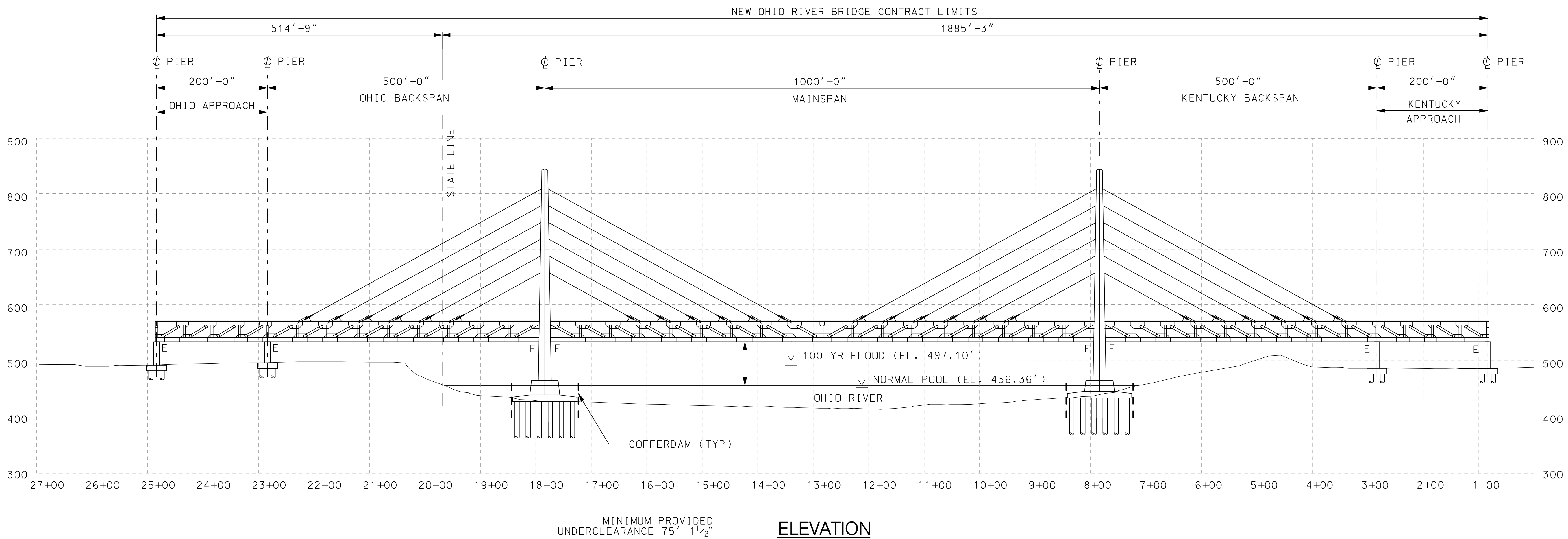
BRENT SPENCE BRIDGE
ALTERNATIVE 1: TIED ARCH
CONSTRUCTION SEQUENCE

EXHIBIT
A8

Prepared by:
PARSONS BRINCKERHOFF



PLAN



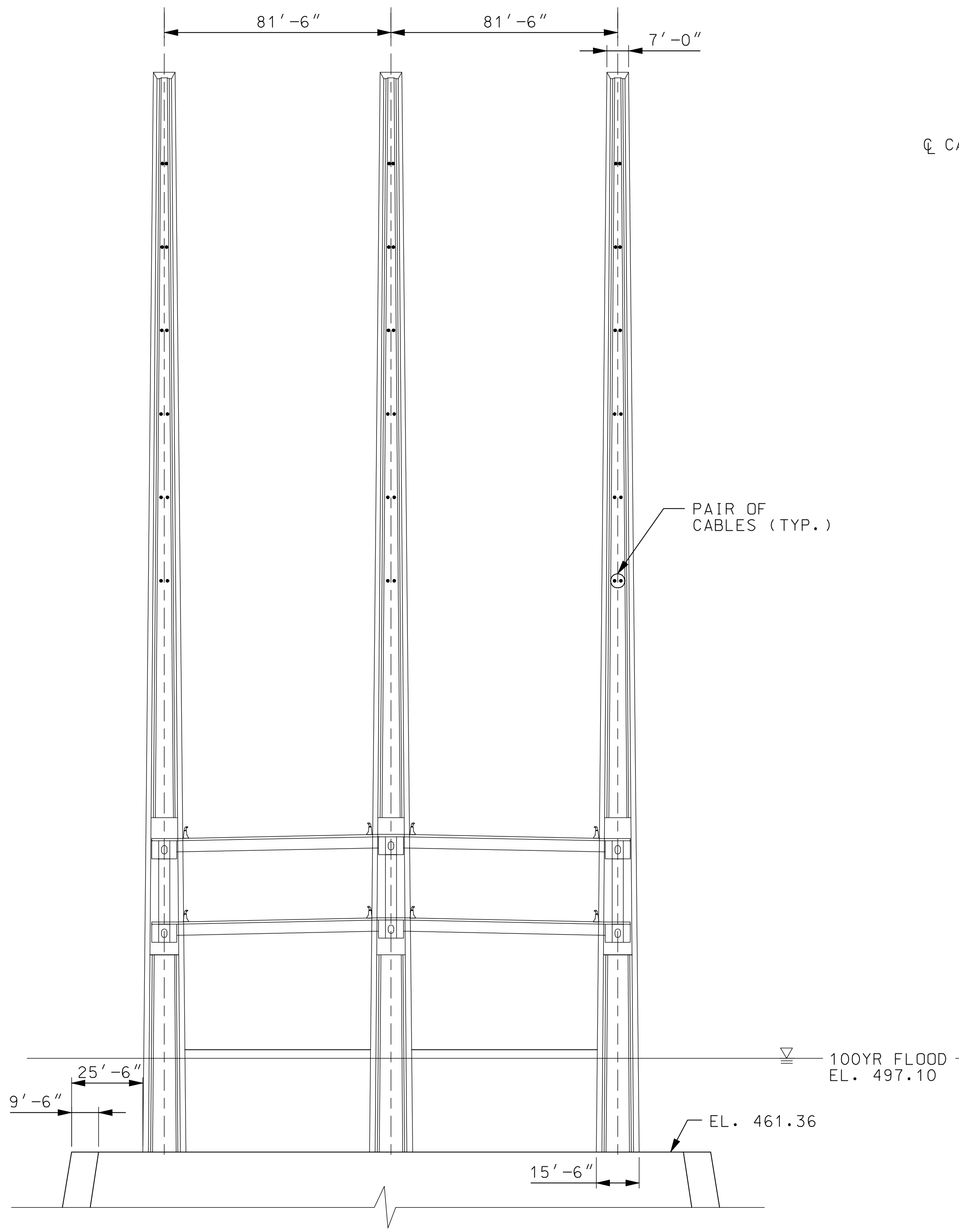
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

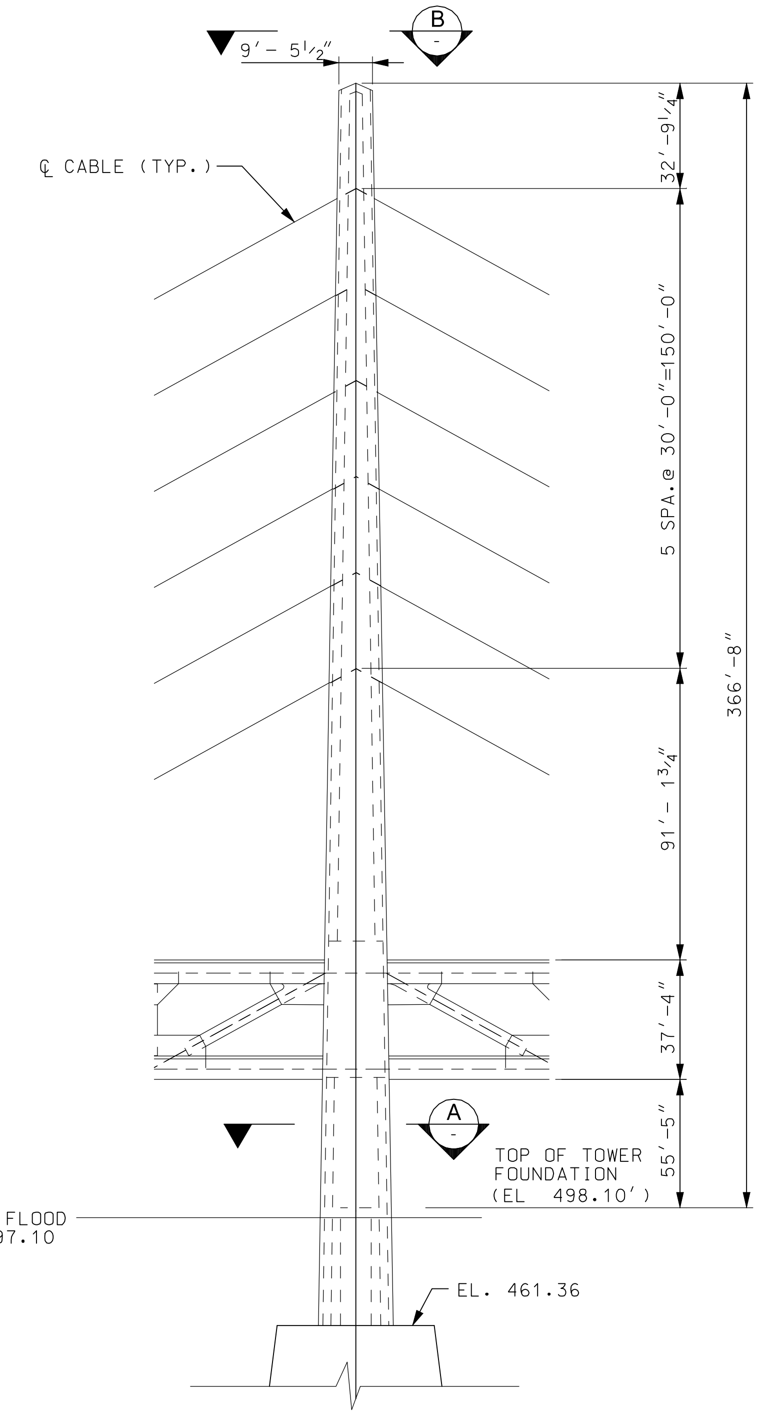
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
GENERAL PLAN AND ELEVATION

EXHIBIT
B1

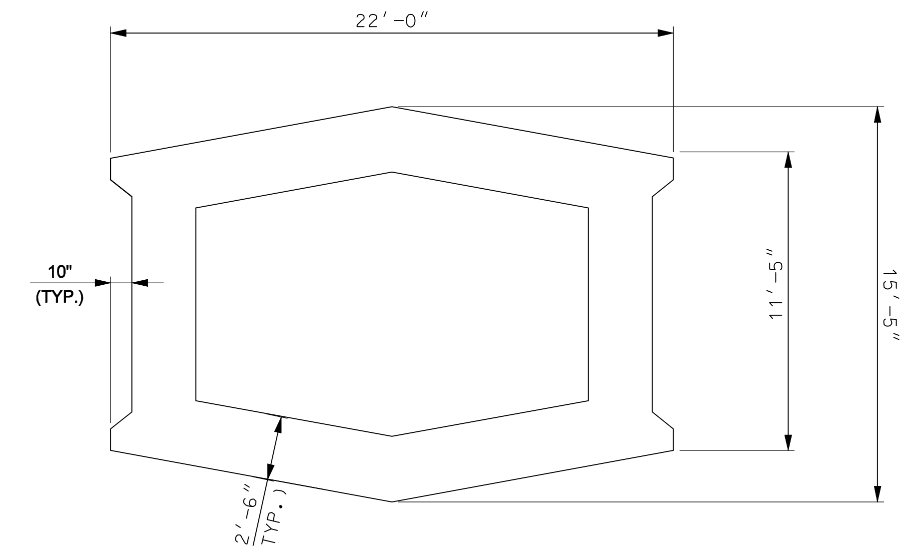
Prepared by:
PARSONS BRINCKERHOFF



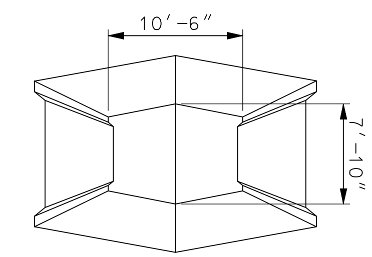
TOWER ELEVATION



TOWER SIDE VIEW



SECTION A



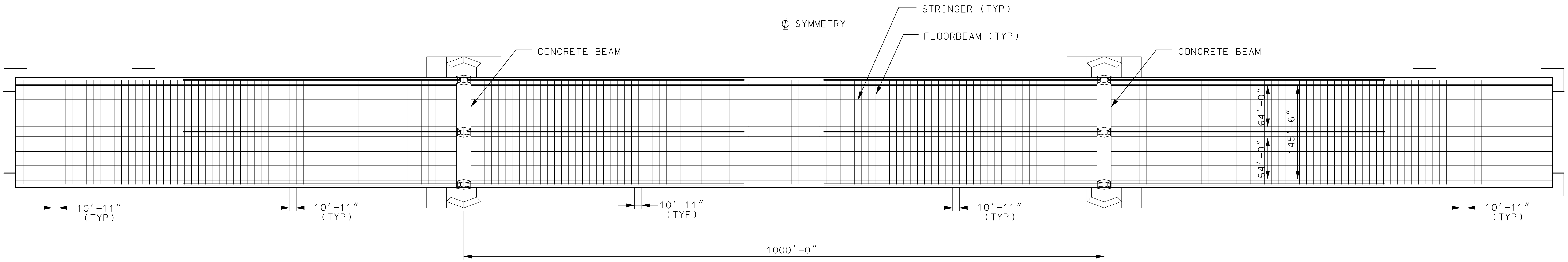
VIEW B

U.S. Department of Transportation
Federal Highway Administration

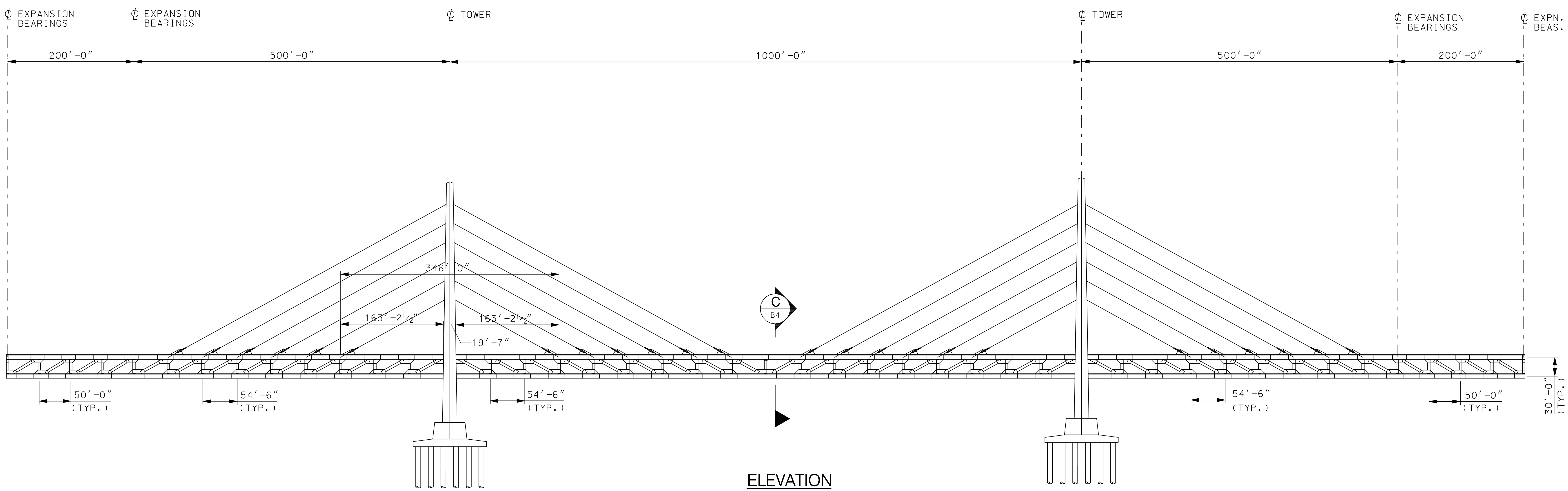
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
TOWER ELEVATION AND SECTIONS

EXHIBIT
B2

Prepared by:
PARSONS BRINCKERHOFF



FRAMING PLAN



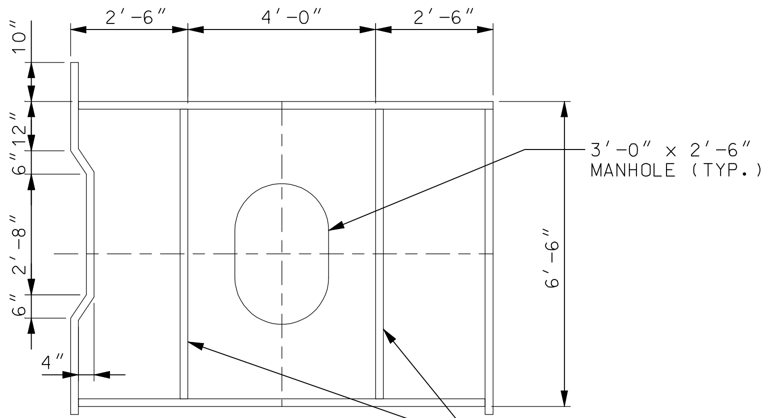
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

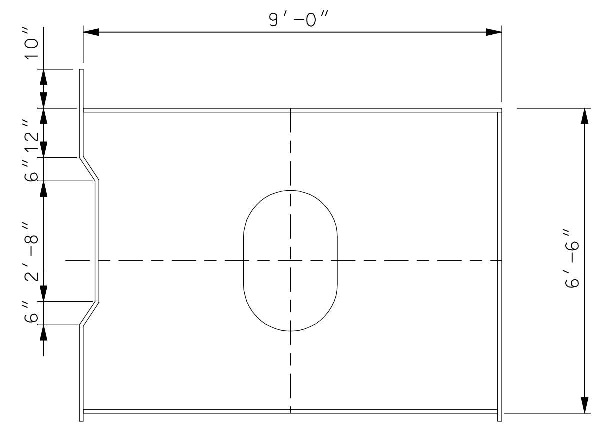
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
SUPERSTRUCTURE FRAMING PLAN & ELEVATION

EXHIBIT
B3

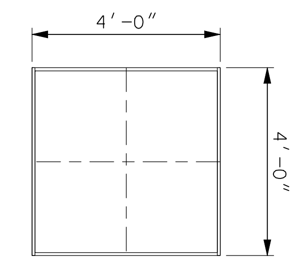
Prepared by:
PARSONS BRINCKERHOFF



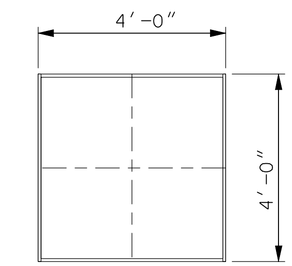
CHORD TYPICAL SECTION
(TOWER LOCATION)



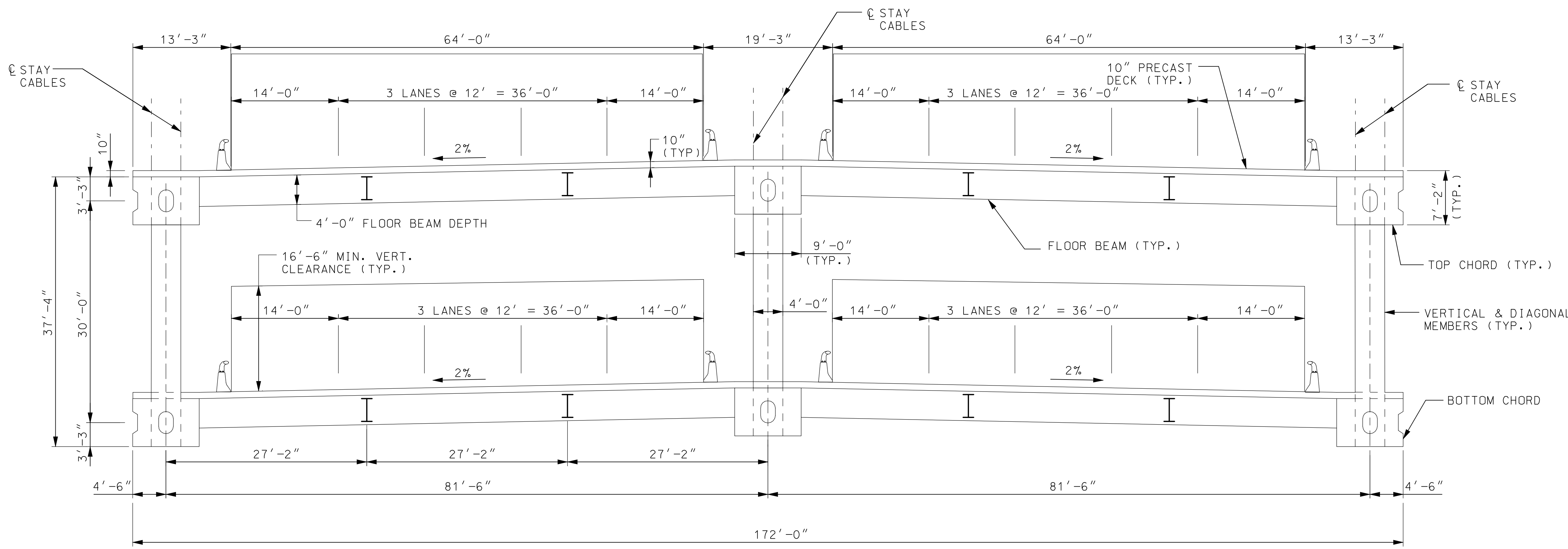
CHORD TYPICAL SECTION
(GENERAL LOCATION)



VERTICAL TYPICAL SECTION



DIAGONAL TYPICAL SECTION



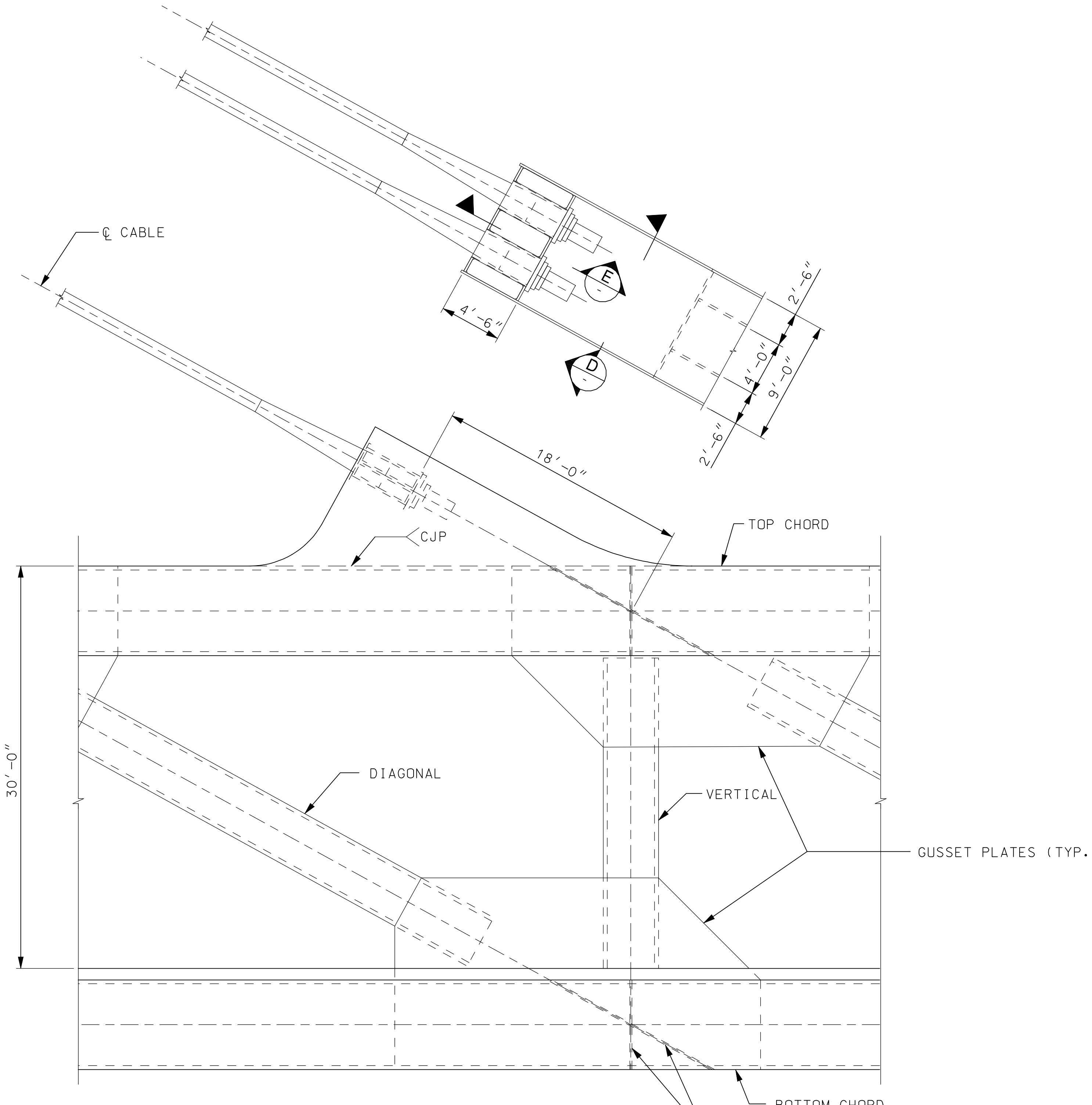
SECTION C / B3

U.S. Department of Transportation
Federal Highway Administration

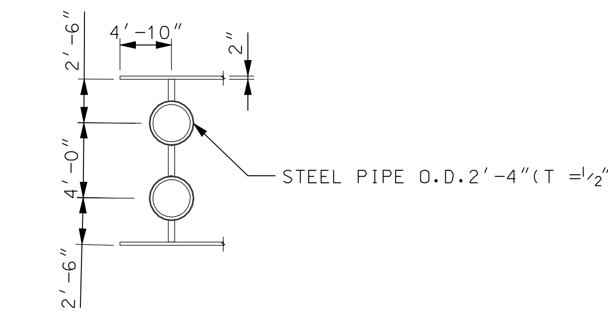
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
SUPERSTRUCTURE TYPICAL SECTION

EXHIBIT
B4

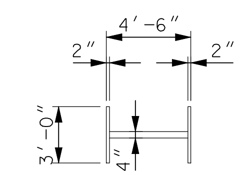
Prepared by:
PARSONS BRINCKERHOFF



CABLE ANCHORAGE DETAIL



VIEW D



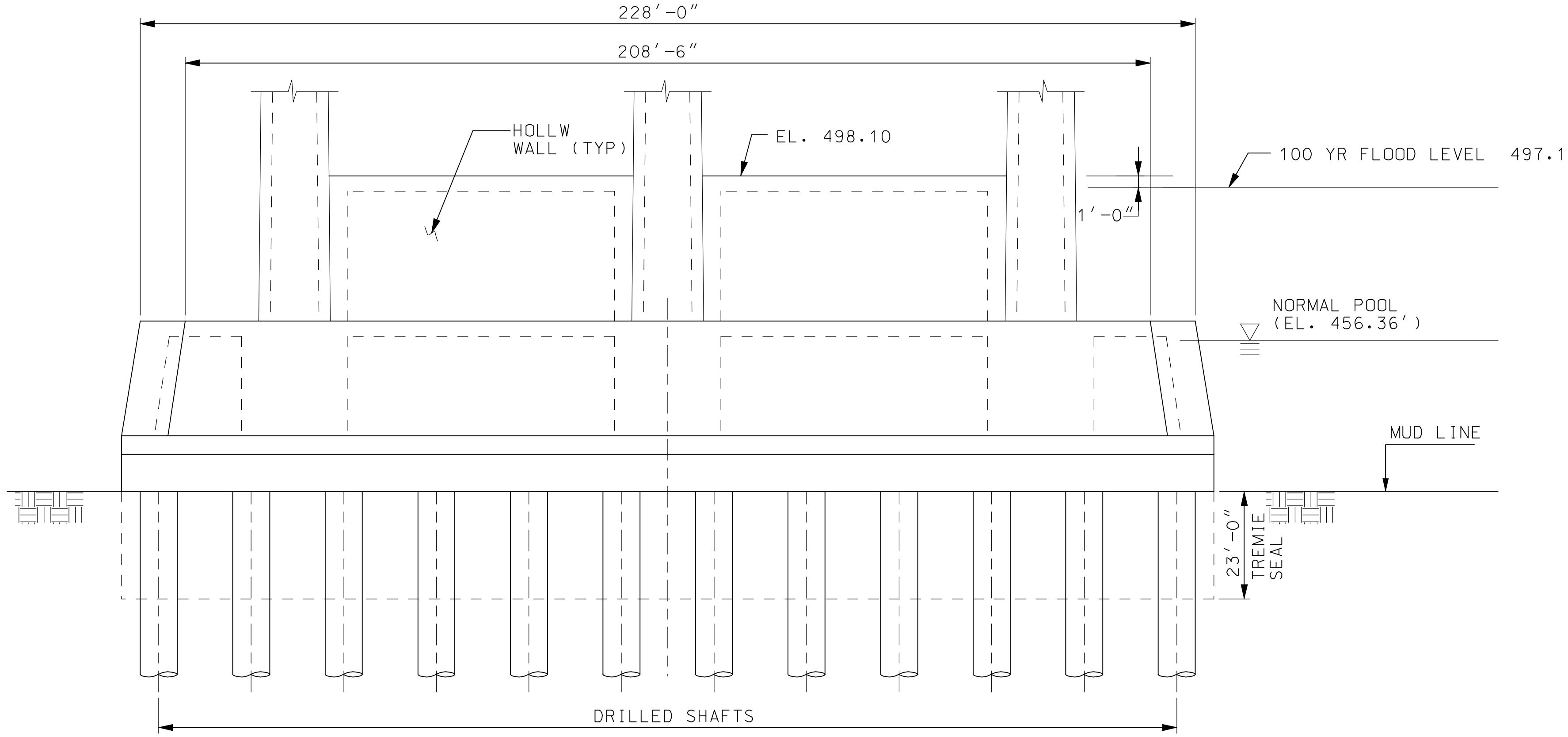
SECTION E

U.S. Department of Transportation
Federal Highway Administration

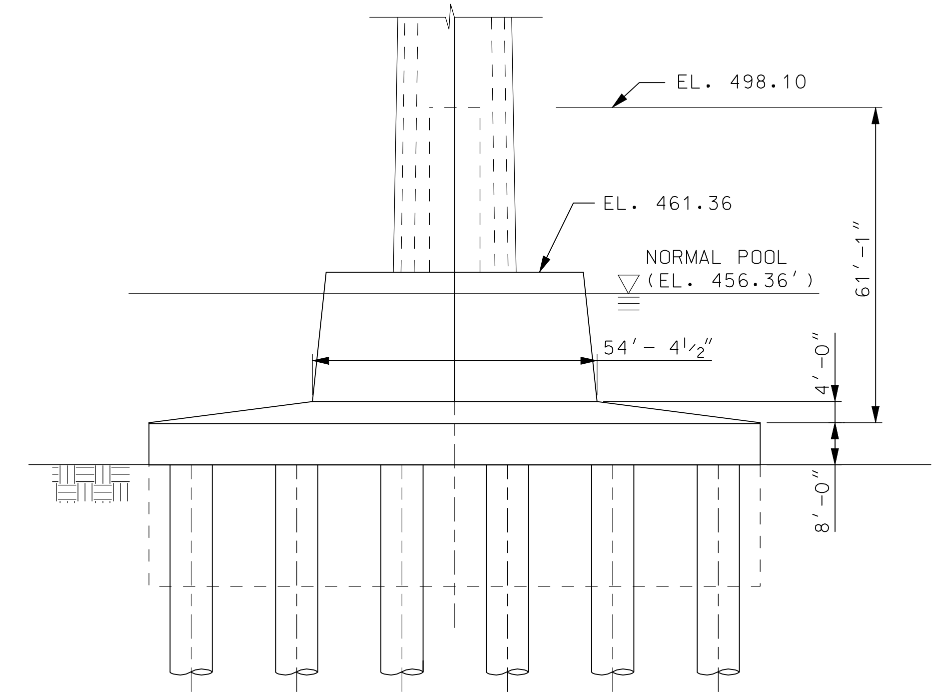
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
TRUSS AND CABLE CONNECTION DETAILS

EXHIBIT
B5

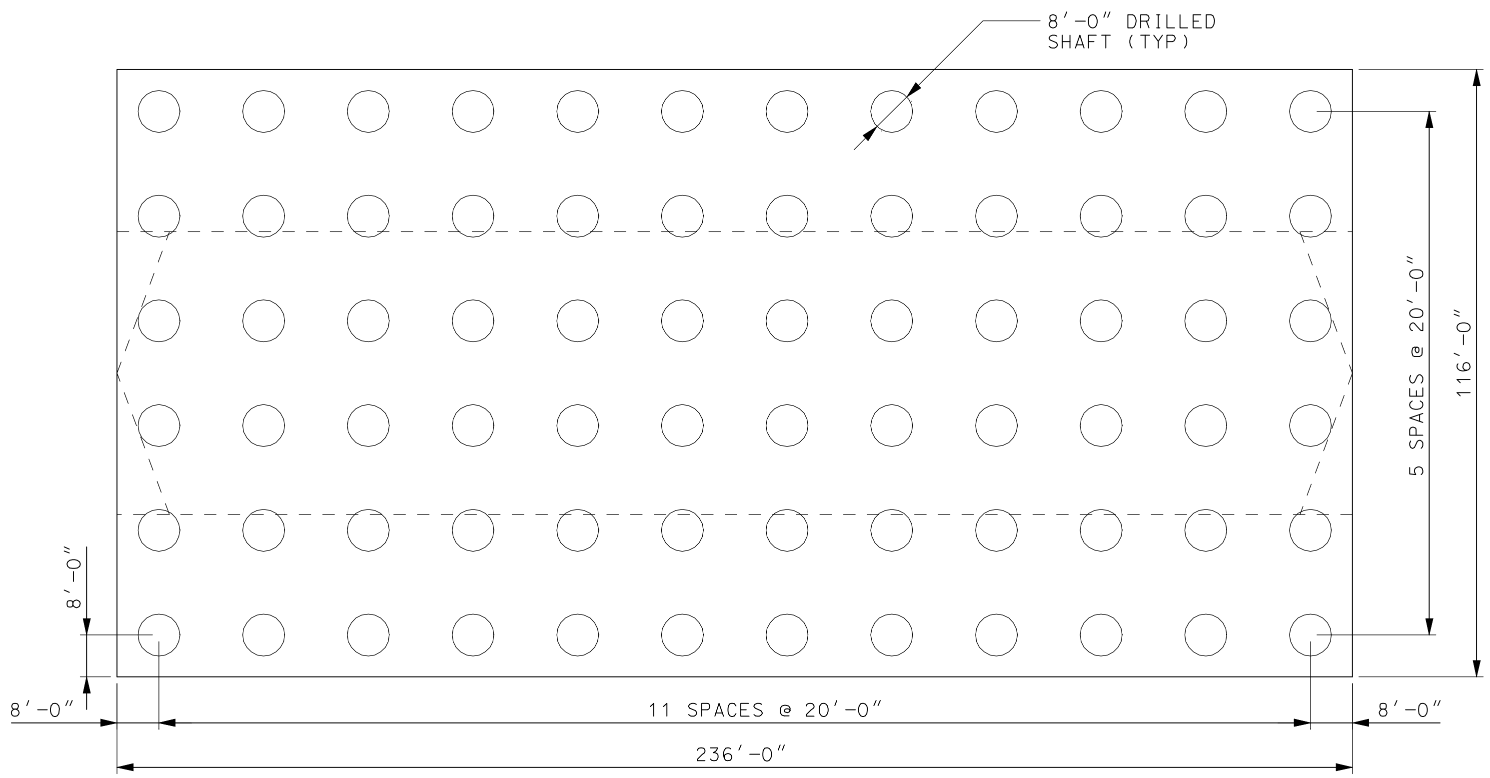
Prepared by:
PARSONS BRINCKERHOFF



TOWER FOUNDATION ELEVATION



TOWER FOUNDATION END VIEW



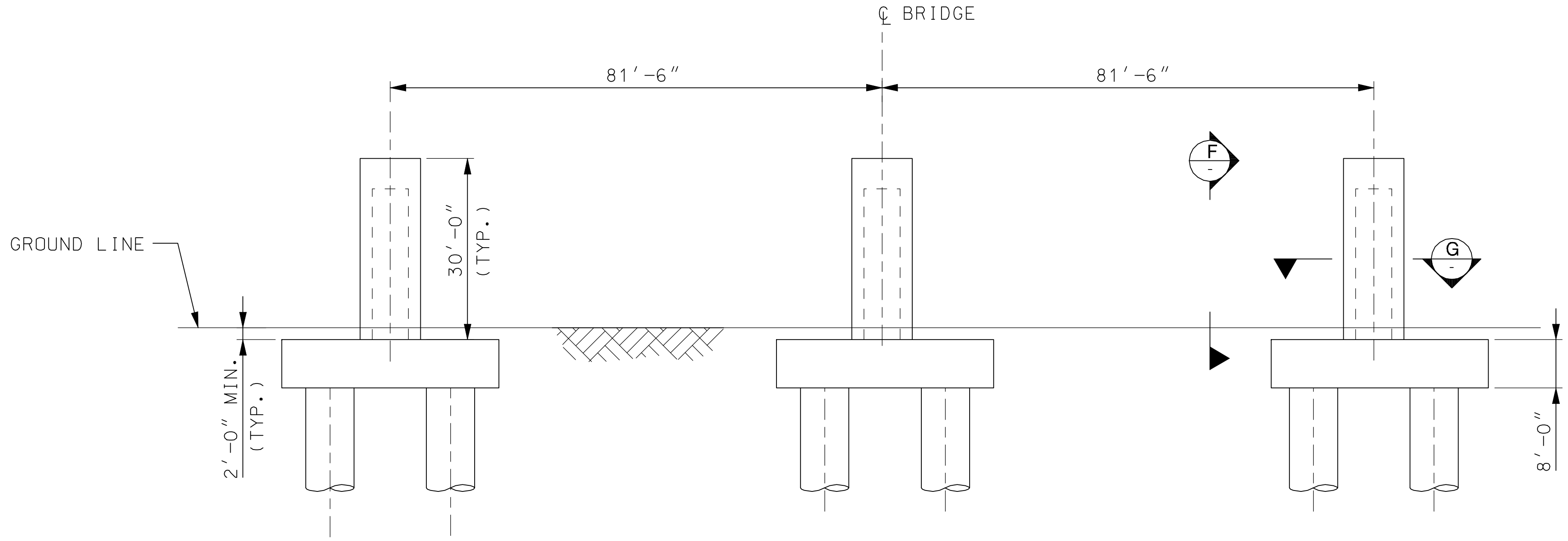
FOUNDATION PLAN

U.S. Department of Transportation
Federal Highway Administration

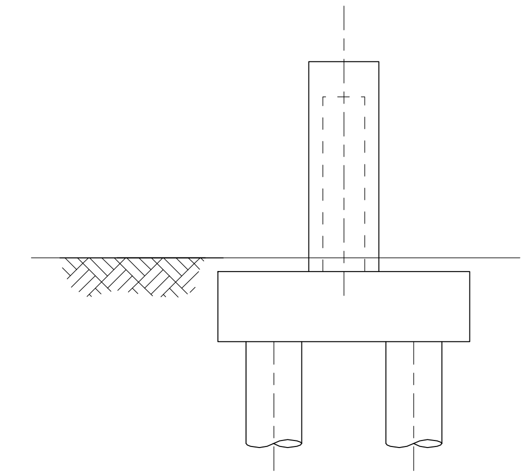
BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
TOWER FOUNDATION DETAILS

EXHIBIT
B6

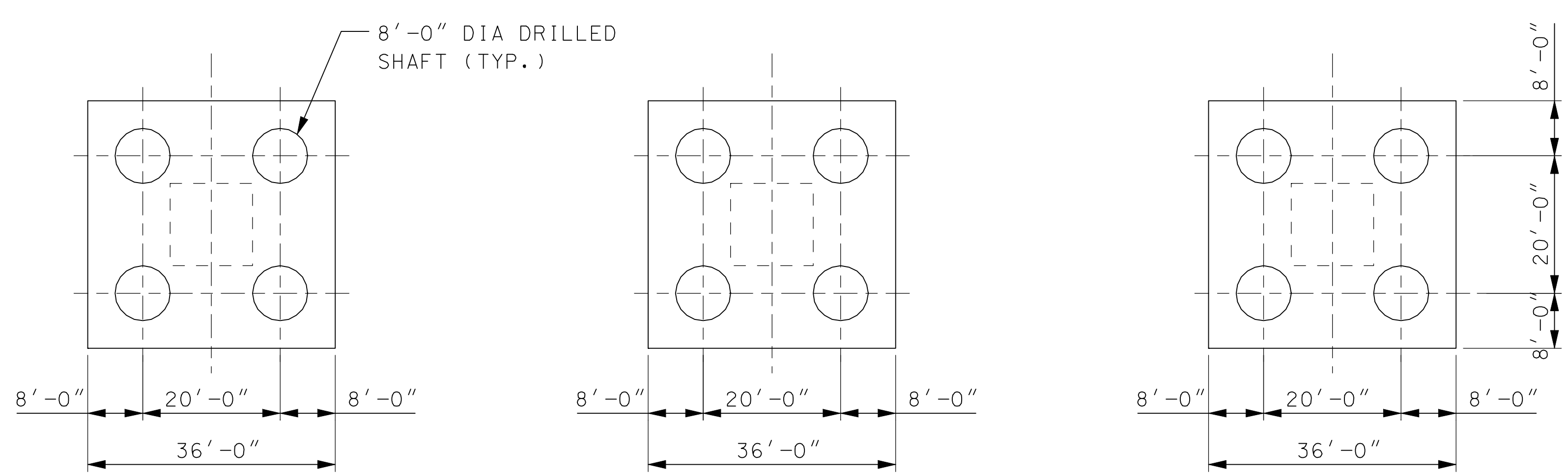
Prepared by:
PARSONS BRINCKERHOFF



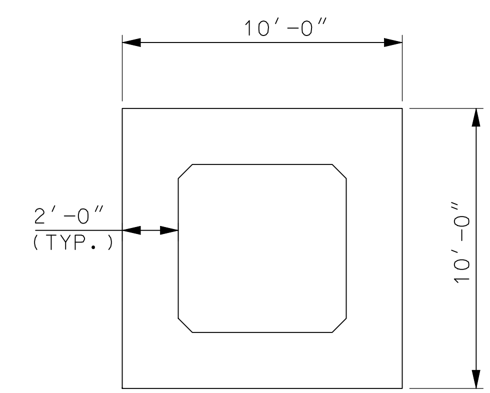
APPROACH LAND PIER



VIEW F



FOUNDATION PLAN



SECTION G

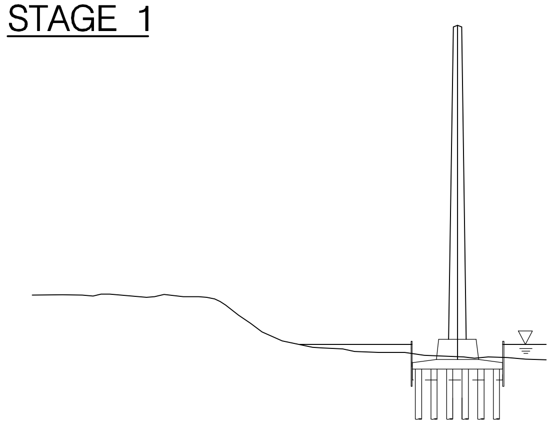
U.S. Department of Transportation
Federal Highway Administration

BRENT SPENCE BRIDGE
ALTERNATIVE 3: TWO TOWER CABLE-STAYED
APPROACH PIER AND FOUNDATION DETAILS

EXHIBIT
B7

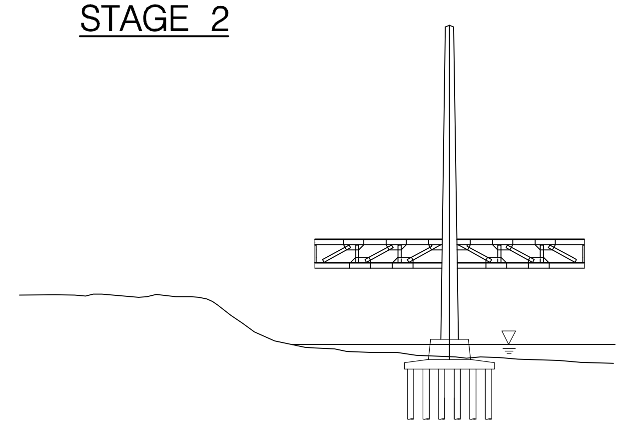
Prepared by:
PARSONS BRINCKERHOFF

STAGE 1



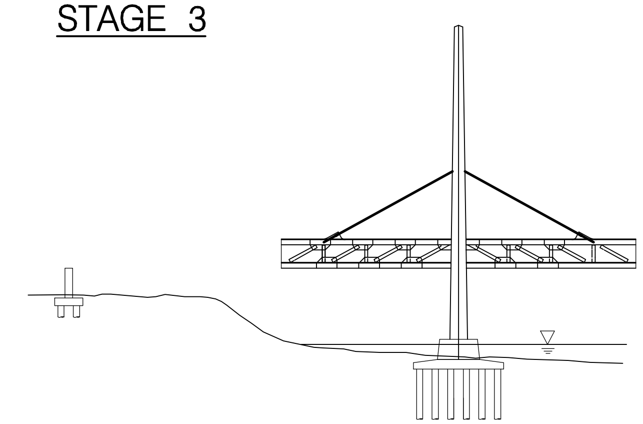
* CONSTRUCT TOWERS AND FOUNDATION.

STAGE 2



* ERECT SUPERSTRUCTURE SEGMENTS BY BALANCED CANTILEVER METHOD.

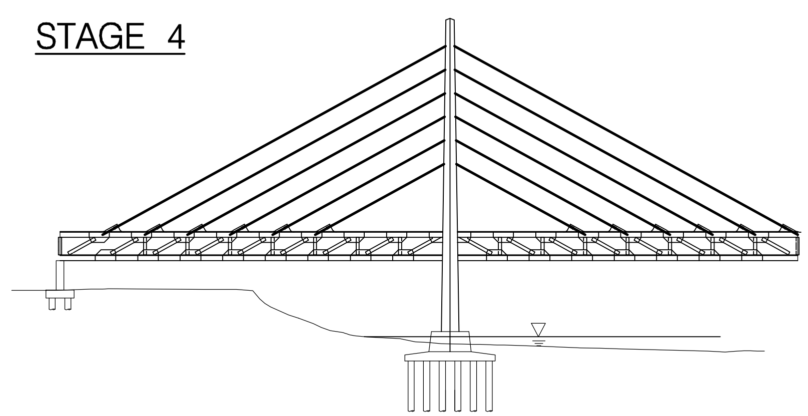
STAGE 3



* ERECT SUPERSTRUCTURE SEGMENTS AND CABLES BY BALANCED CANTILEVER METHOD.

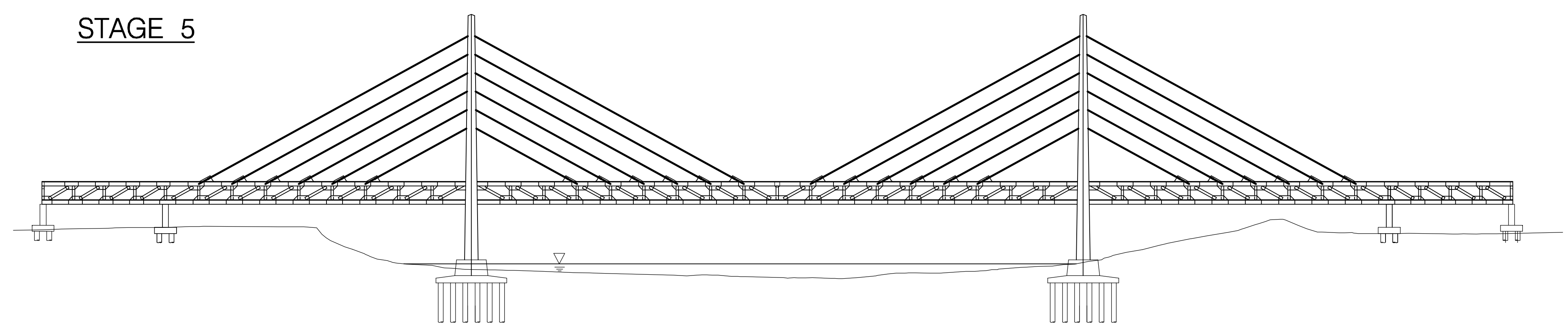
* CONSTRUCT ANCHOR PIER AND FOUNDATION.

STAGE 4



* CONTINUE ERECTING SUPERSTRUCTURE SEGMENTS AND MAKE CLOSURE AT ANCHOR PIER.

STAGE 5

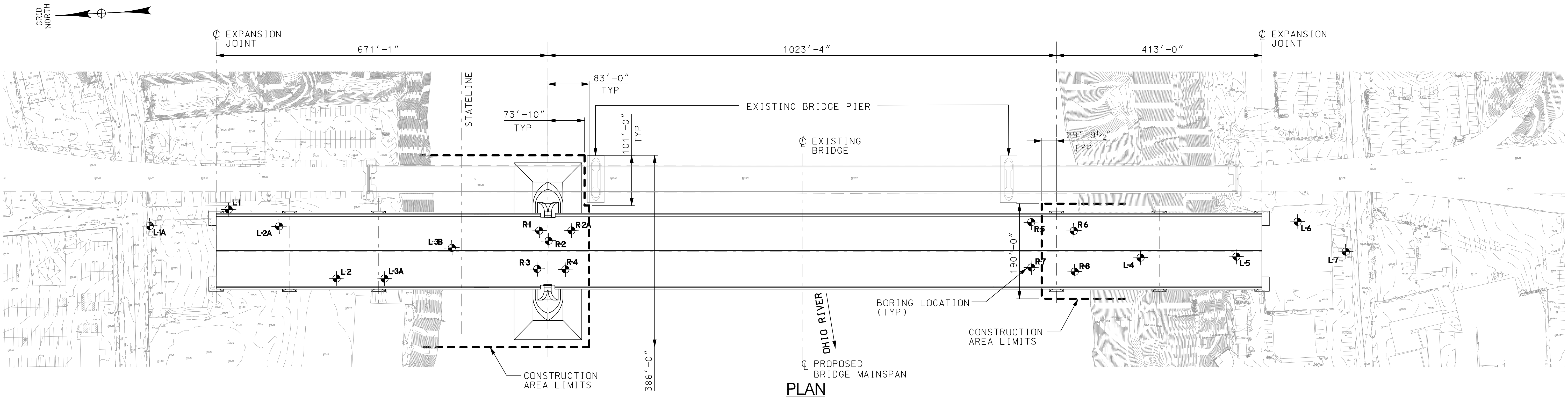


* COMPLETE ERECTING SUPERSTRUCTURE SEGMENTS IN MAIN SPAN AND MAKE FINAL CLOSURE.

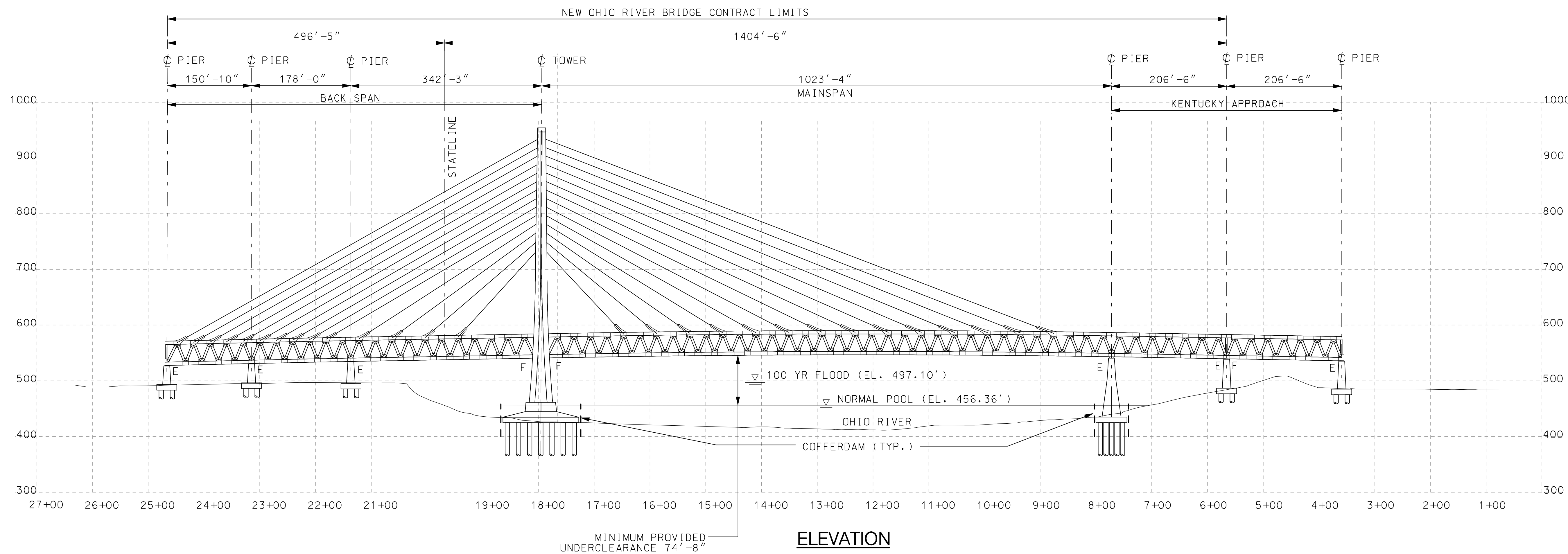
* ERECT THE APPROACH SUPERSTRUCTURE SPANS.

* INSTALL BARRIERS AND PLACE DECK OVERLAY.

NOTE: IN STAGES 1-4 BOTH TOWERS AND THEIR ASSOCIATED SUPERSTRUCTURE WILL BE CONSTRUCTED SIMULTANEOUSLY.



PLAN



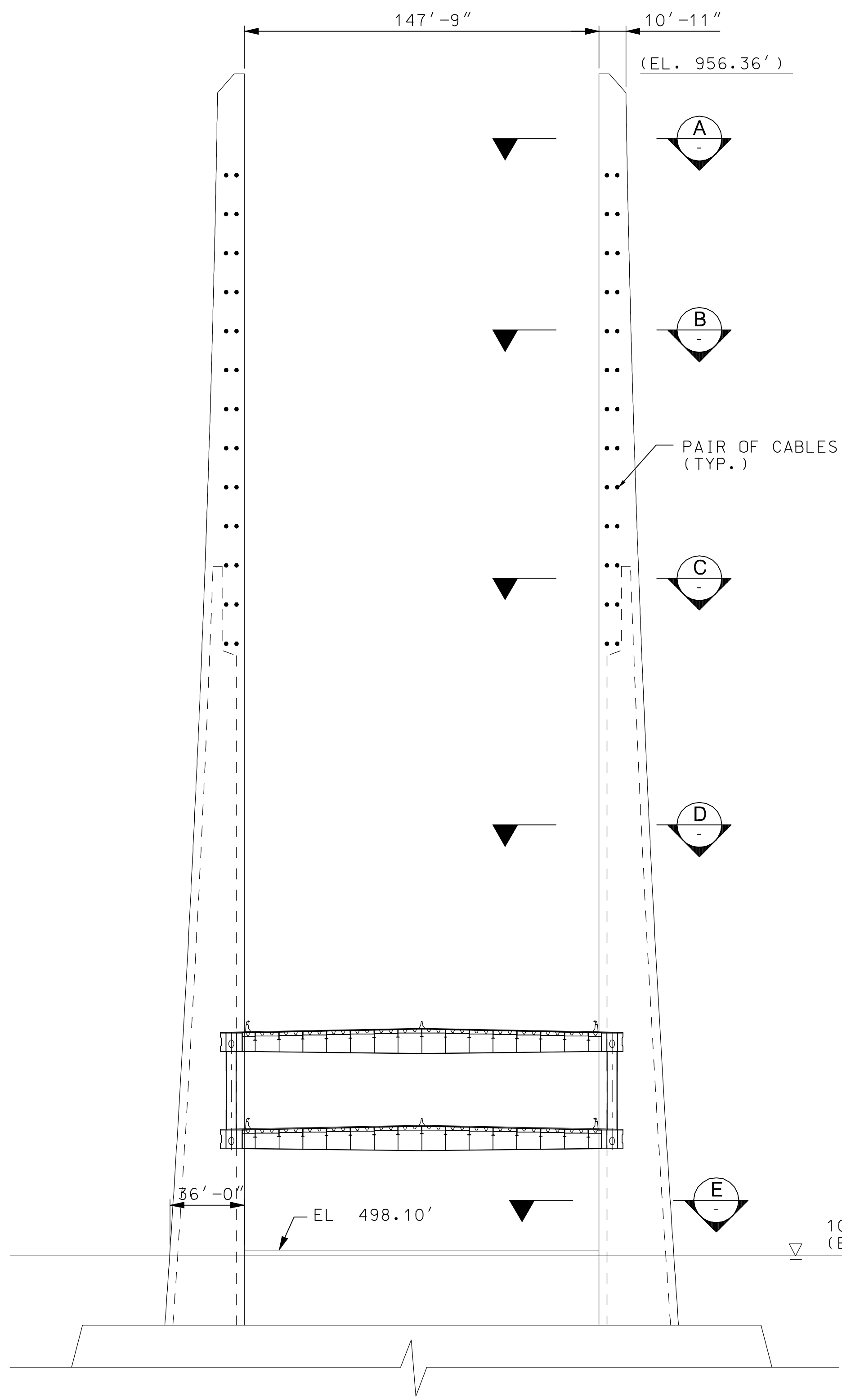
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

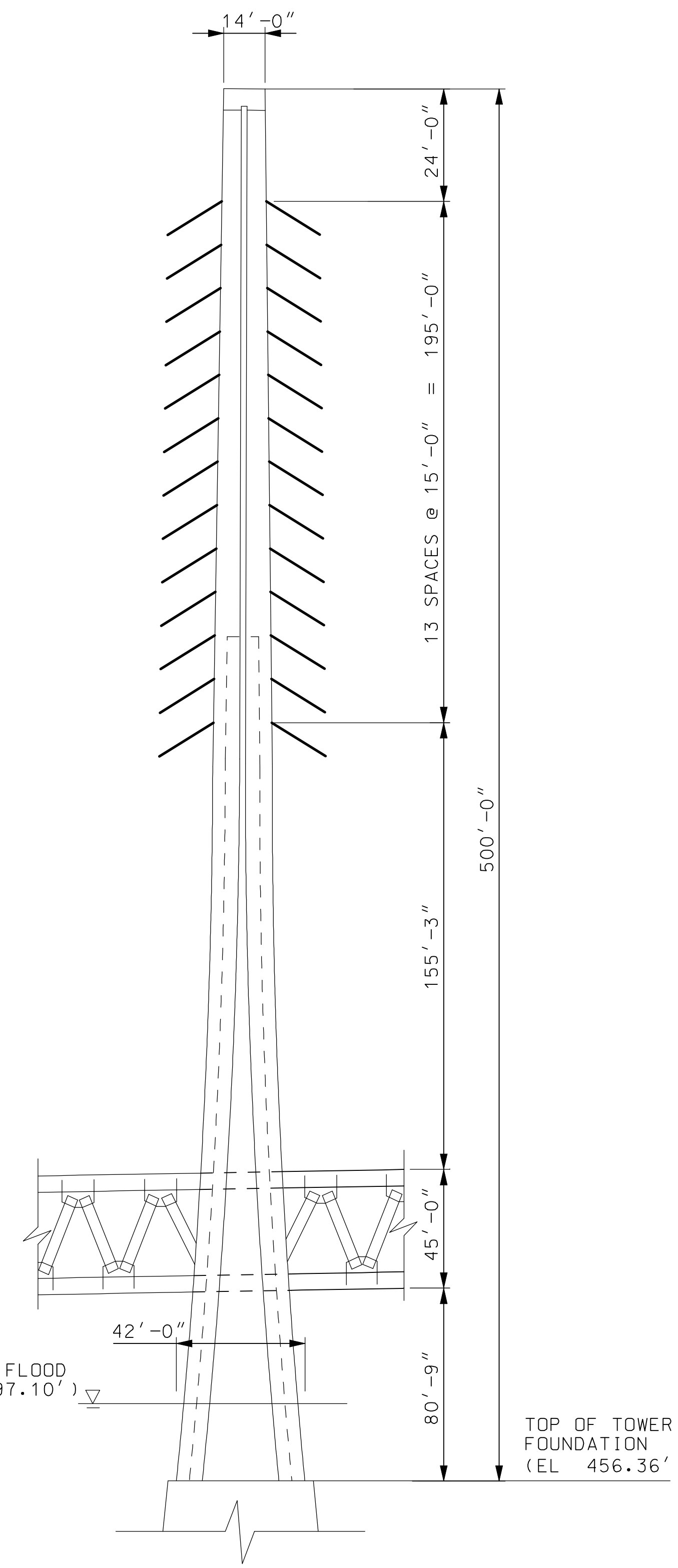
BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
GENERAL PLAN AND ELEVATION

EXHIBIT
C1

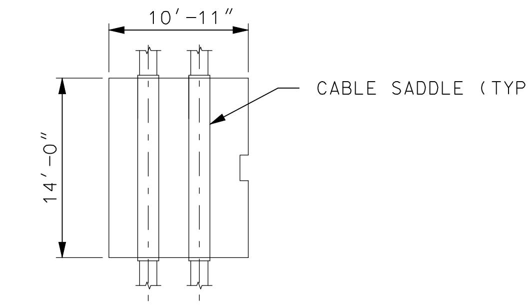
Prepared by:
PARSONS BRINCKERHOFF



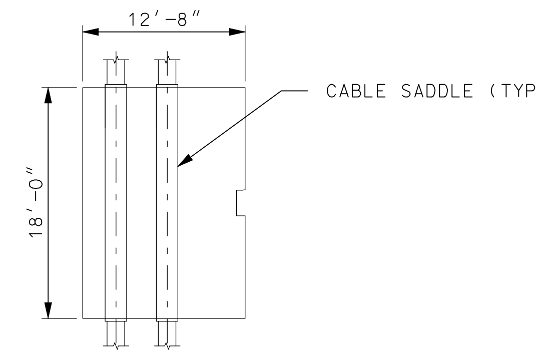
TOWER ELEVATION



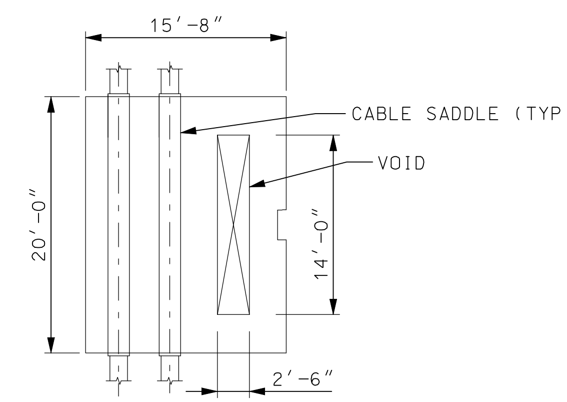
TOWER END VIEW



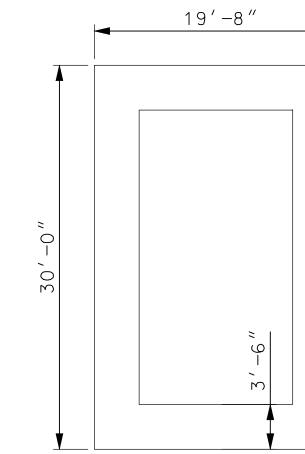
SECTION A



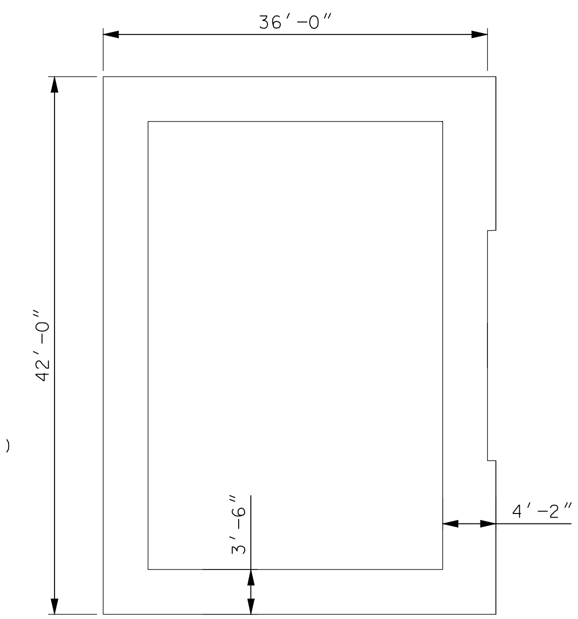
SECTION B



SECTION C



SECTION D



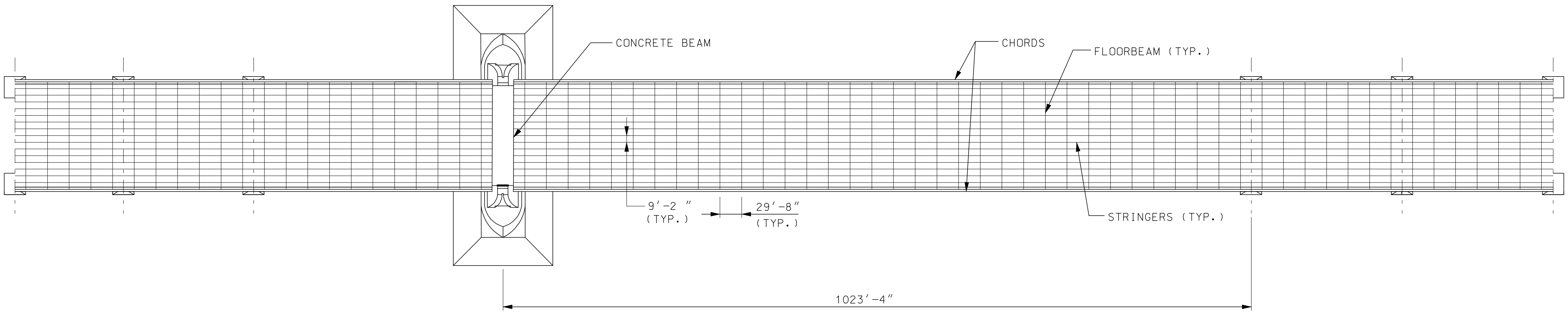
SECTION E

U.S. Department of Transportation
Federal Highway Administration

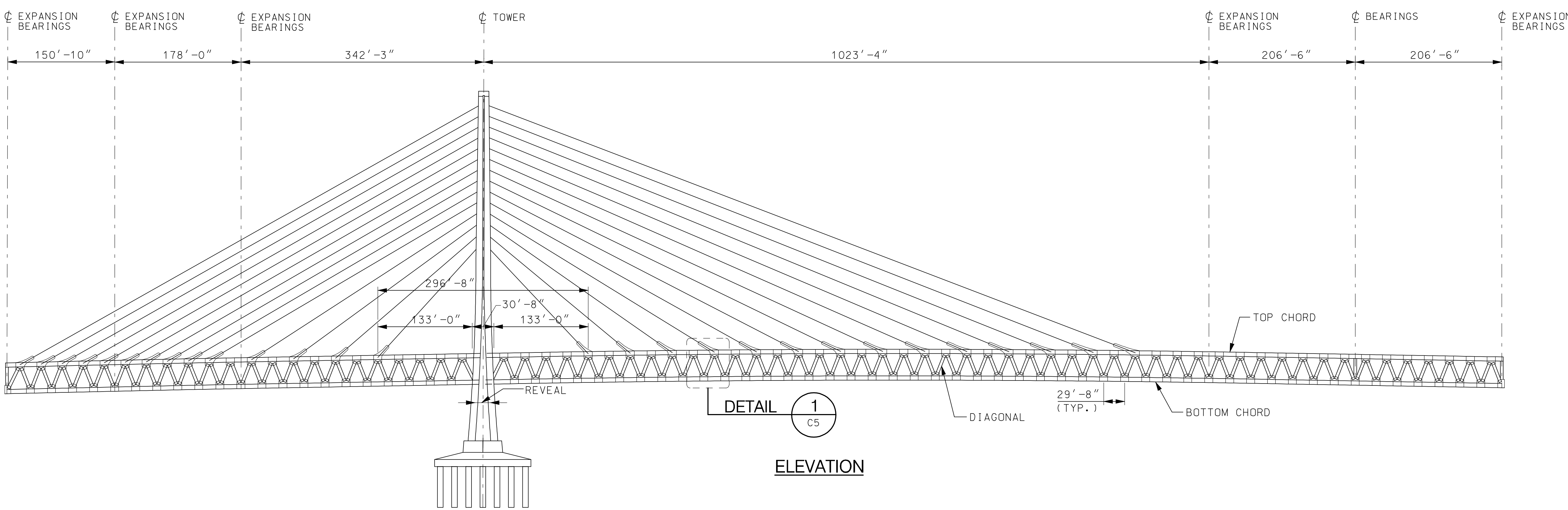
BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
TOWER ELEVATION AND SECTIONS

EXHIBIT
C2

Prepared by:
PARSONS BRINCKERHOFF



FRAMING PLAN



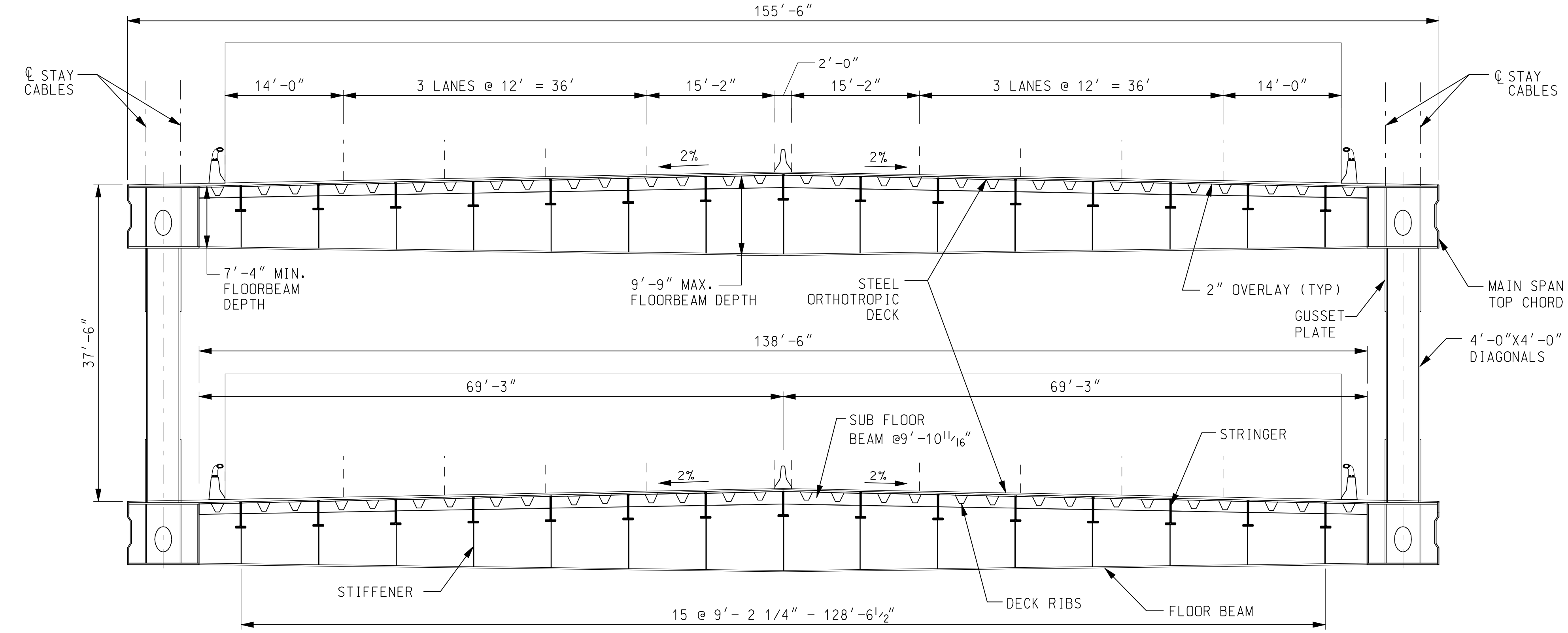
ELEVATION

U.S. Department of Transportation
Federal Highway Administration

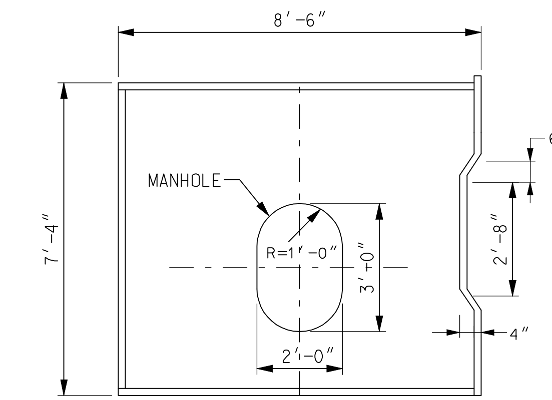
BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
SUPERSTRUCTURE FRAMING PLAN & ELEVATION

EXHIBIT
C3

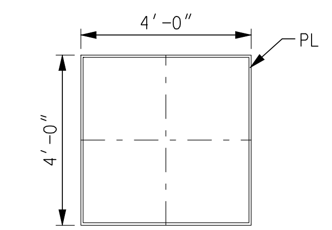
Prepared by:
PARSONS BRINCKERHOFF



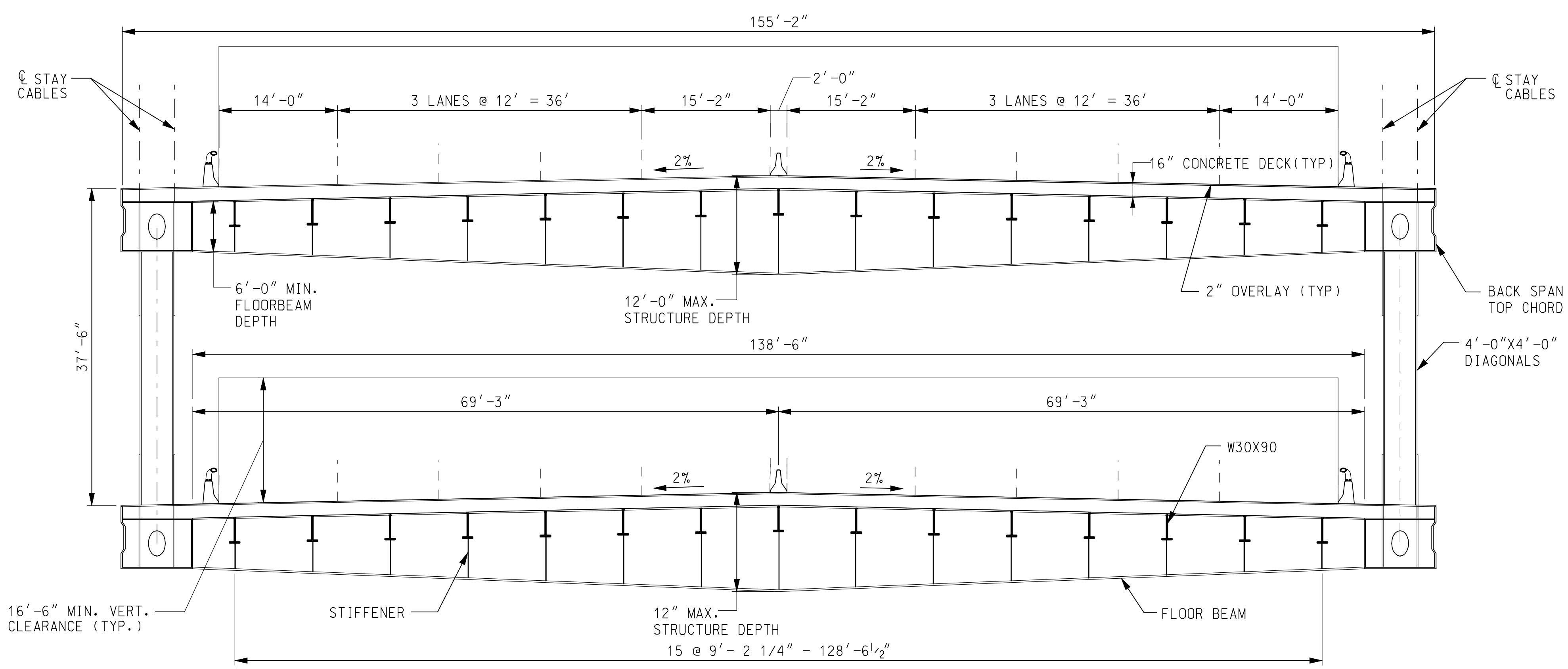
MAIN SPAN TYPICAL SECTION ORTHOTROPIC DECK



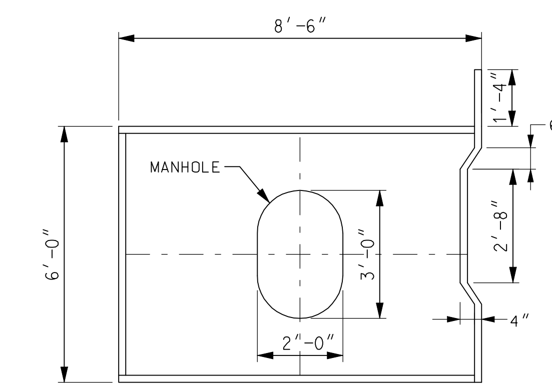
TYPICAL MAIN SPAN TRUSS CHORD SECTION



TYPICAL TRUSS DIAGONAL



BACK SPAN TYPICAL SECTION 16" CONCRETE DECK



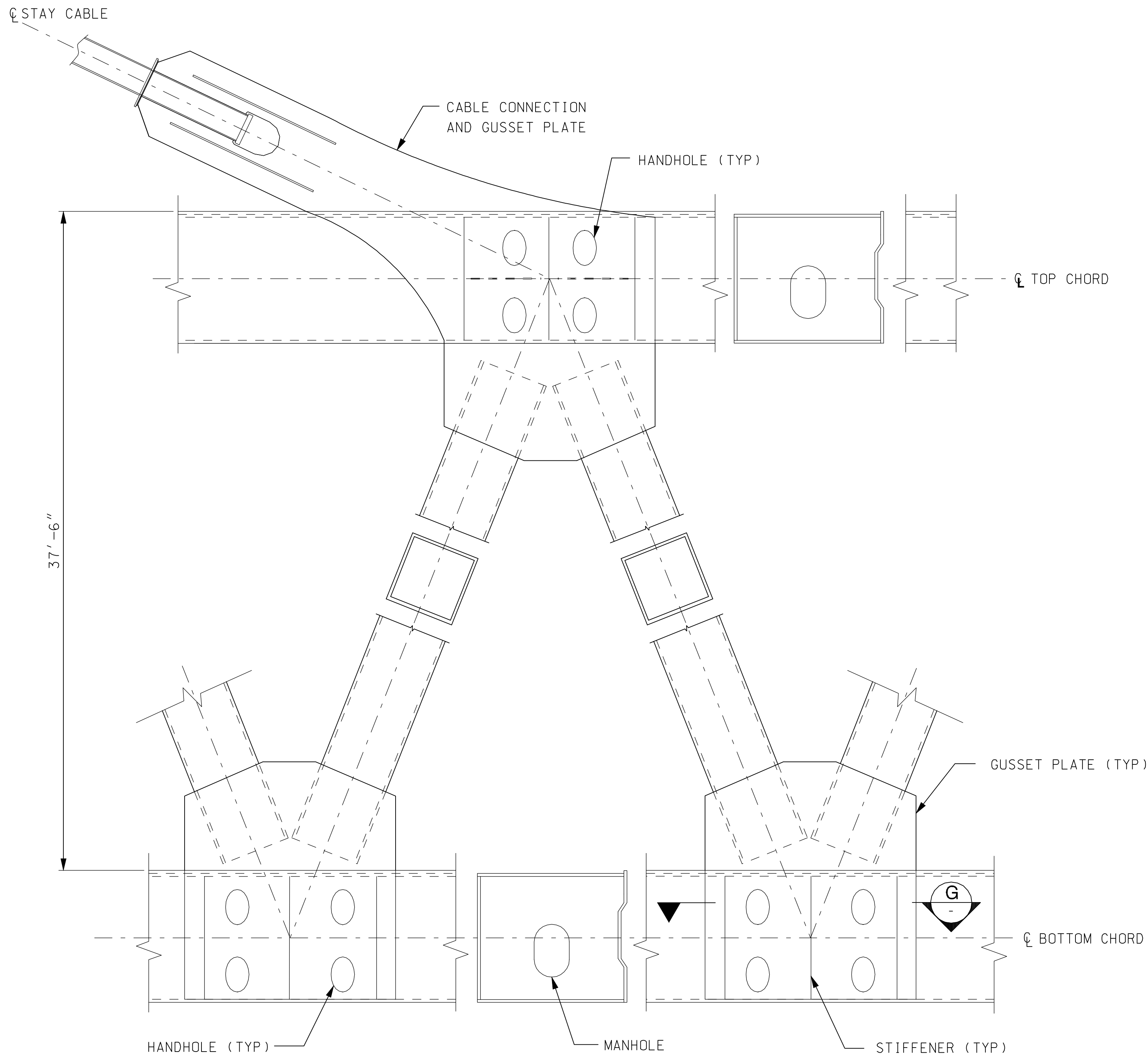
TYPICAL BACK SPAN TRUSS CHORD SECTION

U.S. Department of Transportation
Federal Highway Administration

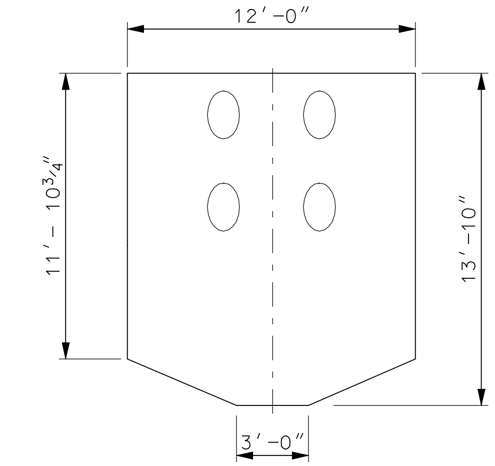
BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
MAIN SPAN AND BACK SPAN SECTIONS

EXHIBIT
C4

Prepared by:
PARSONS BRINCKERHOFF

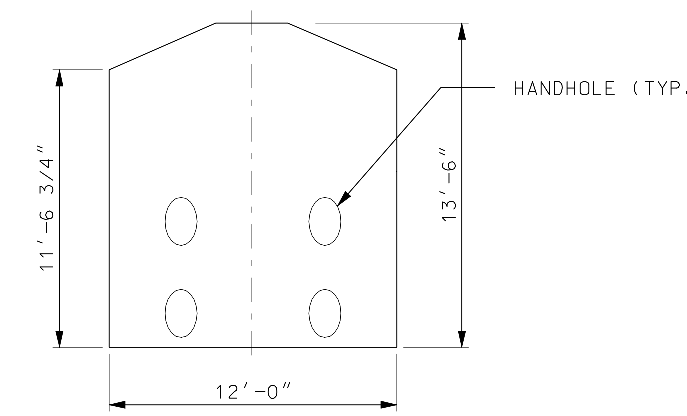


DETAIL 1 C3

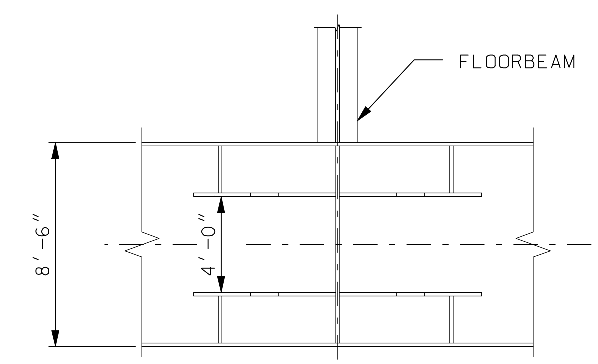


TOP DECK TYPICAL MAIN SPAN GUSSET PLATE

NOTE: GUSSET NOT SHOWN AT CABLE CONNECTION



BOTTOM DECK MAIN SPAN GUSSET PLATE



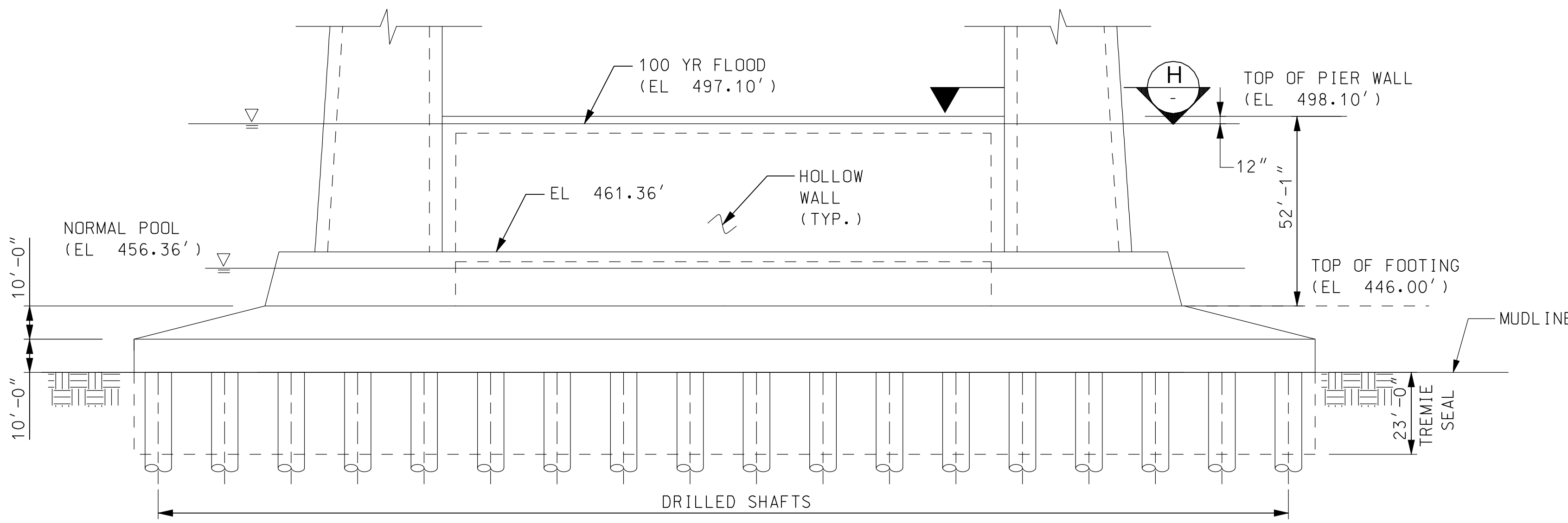
SECTION G –

U.S. Department of Transportation
Federal Highway Administration

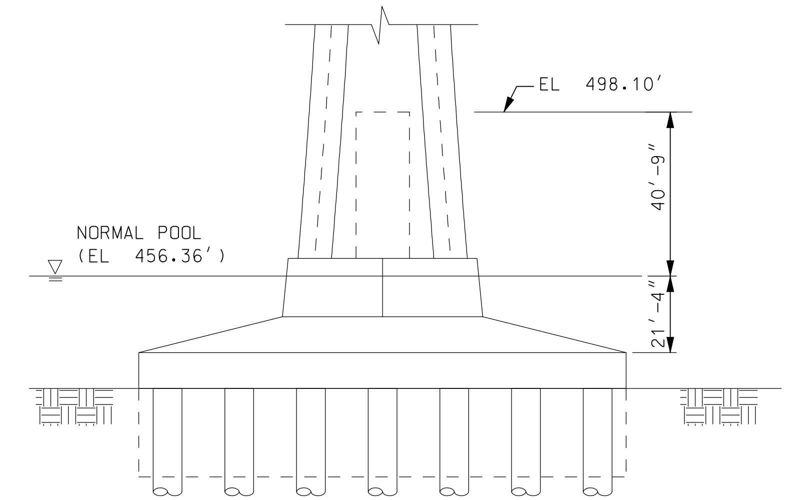
BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
MAIN SPAN TRUSS DETAILS

EXHIBIT
C5

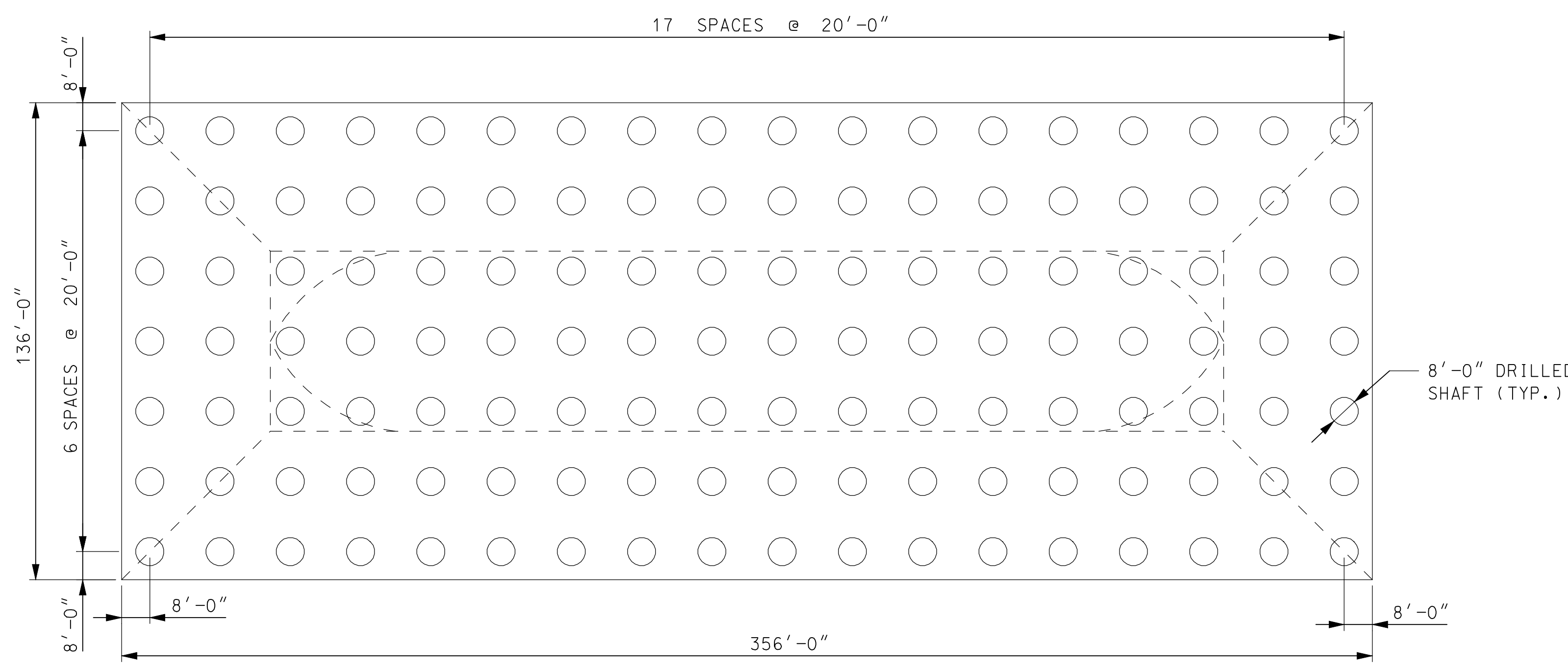
Prepared by:
PARSONS BRINCKERHOFF



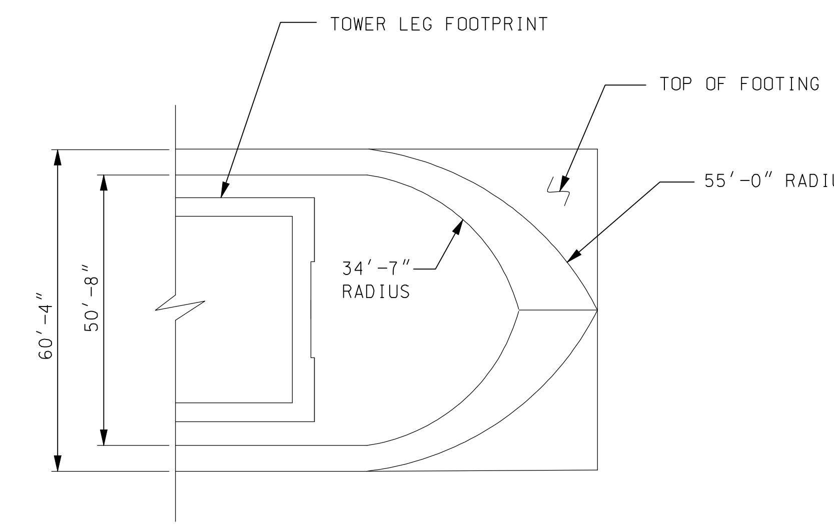
TOWER FOUNDATION ELEVATION



TOWER FOUNDATION END VIEW



FOUNDATION PLAN



SECTION H

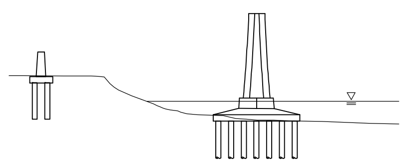
U.S. Department of Transportation
Federal Highway Administration

BRENT SPENCE BRIDGE
ALTERNATIVE 6: ONE TOWER CABLE-STAYED
TOWER FOUNDATION DETAILS

EXHIBIT
C6

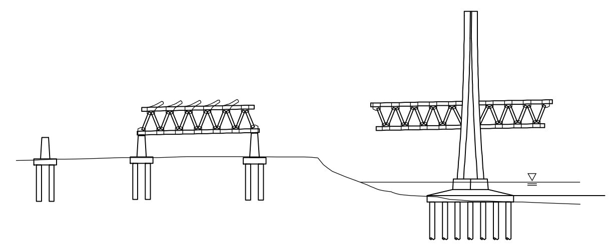
Prepared by:
PARSONS BRINCKERHOFF

STAGE 1



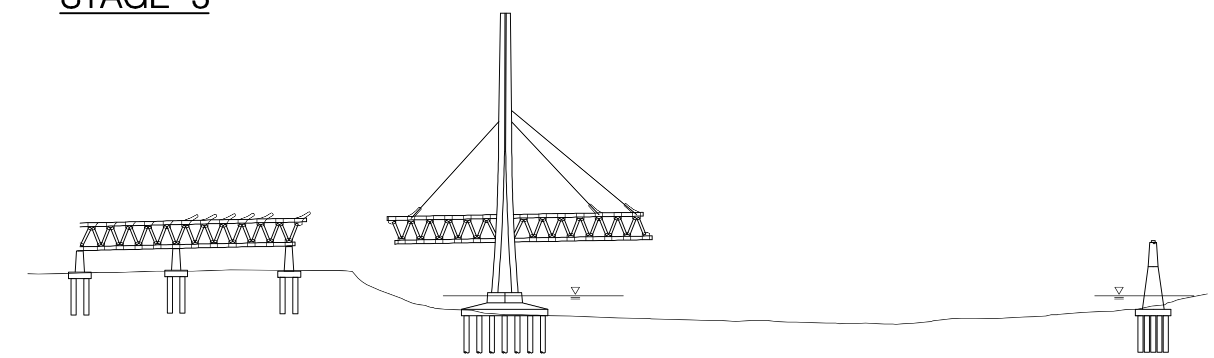
* CONSTRUCT TOWER FOUNDATIONS AND BEGIN TOWER CONSTRUCTION
* CONSTRUCT BACK SPAN ANCHOR PIERS AND FOUNDATIONS

STAGE 2



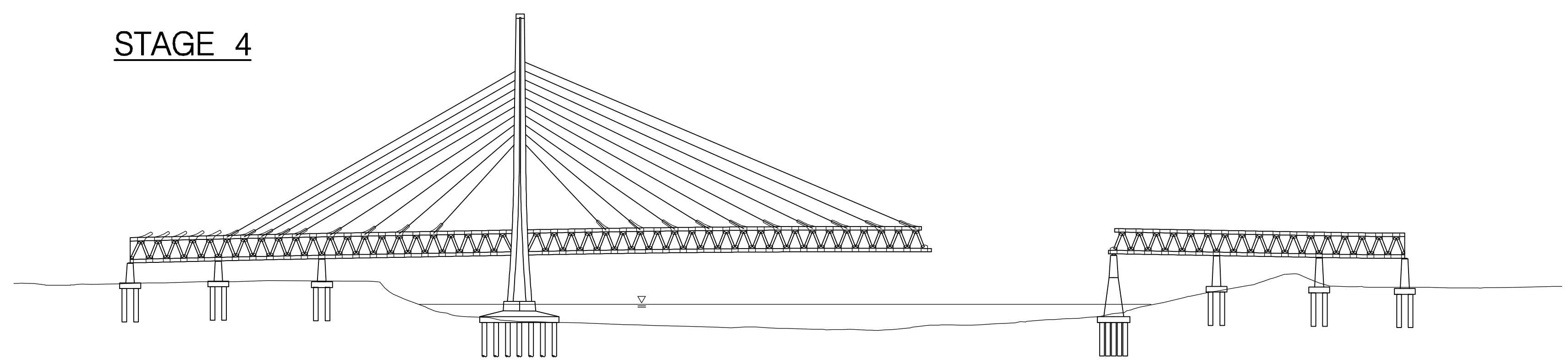
* ERECT SUPERSTRUCTURE SEGMENTS BY BALANCED CANTILEVER METHOD
* CONSTRUCT BACK SPAN SUPERSTRUCTURE
* CONTINUE TOWER CONSTRUCTION

STAGE 3

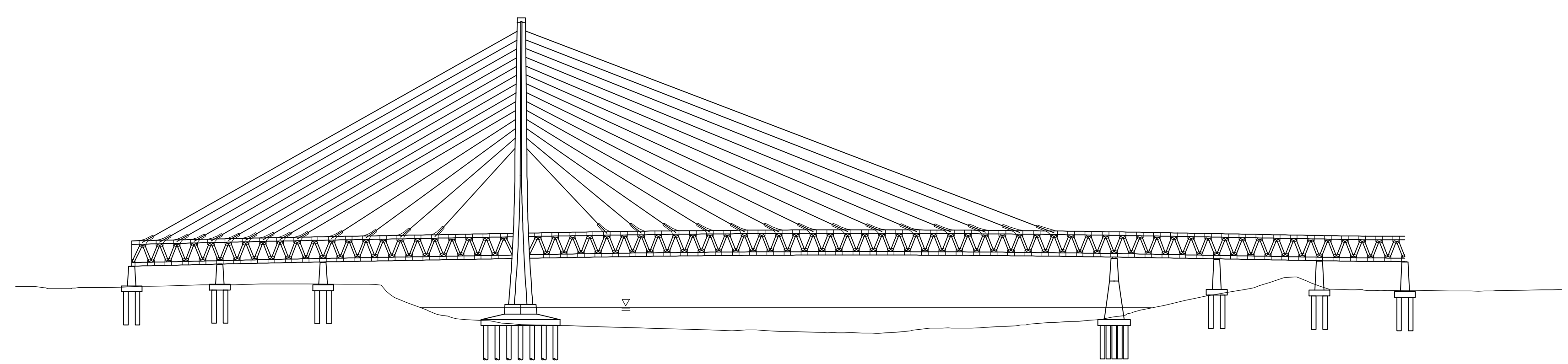


* ERECT SUPERSTRUCTURE SEGMENTS BY BALANCED CANTILEVER METHOD
* CONSTRUCT BACK SPAN SUPERSTRUCTURE
* CONSTRUCT APPROACH SPAN PIERS AND FOUNDATIONS
* CONTINUE TOWER CONSTRUCTION

STAGE 4



* COMPLETE TOWER CONSTRUCTION
* CONTINUE ERECTING SUPERSTRUCTURE SEGMENTS AND MAKE CLOSURE AT ANCHOR PIERS
* CONSTRUCT APPROACH SPAN SUPERSTRUCTURE



* COMPLETE ERECTING SUPERSTRUCTURE SEGMENTS IN MAIN SPAN AND MAKE FINAL CLOSURE
* INSTALL BARRIERS AND PLACE DECK OVERLAY

Appendix 6B
Renderings of
Final 3 Bridge Alternatives

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 1 – TIED ARCH

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

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ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 3 – TWO TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

ALTERNATIVE 6 – ONE TOWER CABLE STAYED

Appendix 6C
Geotechnical
Report

Structure Foundation Exploration

Brent Spence Bridge Replacement
Interstate 71/Interstate 75
Cincinnati, Ohio/Covington, Kentucky
Project No. N1105070
March 11, 2011



Prepared for:
Parsons Brinckerhoff, Inc.
Cincinnati, Ohio

Prepared by:
HCN
A Terracon COMPANY
Cincinnati, Ohio

March 11, 2011

A Terracon COMPANY

Parsons Brinckerhoff
312 Elm Street, Suite 2500
Cincinnati, Ohio 45202

Attn: Mr. Duane F. Phelps, P.E.
P: 513-639-2138
F: 513-421-1040
E:phelpsd@pbworld.com

Re: Geotechnical Engineering Report
Proposed Brent Spence Bridge Replacement
Interstate 71/Interstate 75 Corridor
Cincinnati, Ohio- Covington, Kentucky
HCN/Terracon Project No.: N1105070

Dear Mr. Phelps:

H.C. Nutting, a Terracon Company (HCN) has completed the geotechnical engineering services for the above referenced project. This report presents the findings of the subsurface exploration and provides geotechnical and foundation recommendations regarding the proposed Brent Spence Bridge Replacement project.

We appreciate the opportunity to be of service to you on this exciting project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
H.C. Nutting, a Terracon Company

David W. Westendorf, P.E.
Senior Staff Geotechnical Engineer

Aaron J. Muck, P.E.
Senior Associate/Department Manager

Swaminathan Srinivasan, P.E.
Senior Vice President/Division Manager

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The following provides a brief summary of our exploration, findings, and recommendations.

- This report is intended for use in the bridge foundations only. Grading and earthwork related issues, roadway and embankment design/construction have not been finalized and are not discussed in this report.

- Nineteen (19) test borings were performed for the project; nine (9) test borings were performed in the river with the remaining borings performed on land in Ohio and Kentucky. All borings extended to bedrock with approximately 40 to 80 feet of rock coring performed at each location. The test borings encountered primarily granular overburden soils (both fill and natural) overlying limestone and shale bedrock.

- Geophysical testing consisting of PS Suspension Logging was performed in three (3) of the test borings (L-1, L-4, R-2A) by GeoVision Geophysical Services. The purpose of the geophysical testing was to acquire site specific shear wave velocities and compressional wave velocities as a function of depth to aid in seismic design.

- Given the subsurface conditions and the preliminary design plans provided by Parsons Brinckerhoff, drilled shafts are indicated for the bridge pier foundations. Driven (CIP) piles, H-piles, and drilled shafts could be considered for the approach spans and abutments located on land. Design parameters for drilled shafts and driven piles are provided.

- Several types of cofferdams (if needed) could be considered for the proposed construction; braced, cellular, or double-walled sheet piles. The designer should consider hydrostatic, soil, current, waves, and ice load as well as construction loading. Accidental loading, such as due to a ship strike, and seismic loading may also need to be considered.

- Quality control is critical to the success of the deep foundation system performance. Quality control of drilled shafts can be divided into three categories; diligent inspection, integrity testing and load testing. Besides installation quality control, we recommend both integrity and load testing be included in the specifications for the proposed bridge foundations.

This summary should be used in conjunction with the entire report for design purposes. It should be recognized that details were not included or fully developed in this section, and the report must be read in its entirety for a comprehensive understanding of the items contained herein. The section titled **GENERAL COMMENTS** should be read for an understanding of the report limitations.

ACRONYMS

AASHTO – American Association of State Highway Transportation Officials
ATH – Ambient Temperature Headspace
BSB – Brent Spence Bridge
CEUS – Central and Eastern U.S. Seismic Zone
CIP - Cast-in-place concrete piles
CSL – Crosshole Sonic Logging
CT – Crosshole tomography
DHC – Downhole Camera
FHWA – Federal Highway Administration
FID – Flame ionization detector
GGL - Gamma-Gamma logging
KYTC – Kentucky Transportation Cabinet
LRFD – Load and Resistance Factor Design
NHI – National Highway Institute
ODOT – Ohio Department of Transportation
PB- Parsons Brinckerhoff
PSI – Photo Science, Inc.
REC - Recovery
RMR – Rock mass rating
RQD - Rock Quality Designation
RTK – Real time kinematics
SDI – Slake Durability Index
SID – Shaft Inspection Device
SPT – Standard Penetration Test
USGS – U.S. Geological Survey
VOC – Volatile organic compounds
WOH – Weight of hammer

GEOTECHNICAL ENGINEERING REPORT
BRENT SPENCE BRIDGE REPLACMENT
INTERSTATE 71 / INTERSTATE 75
CINCINNATI, OHIO-COVINGTON, KENTUCKY

Terracon Project No. N1105070
March 11, 2011

1.0 INTRODUCTION

A geotechnical study has been performed for the proposed Brent Spence Bridge (BSB) replacement project by H.C. Nutting, a Terracon Company (HCN) in support of the ongoing design efforts by Parsons Brinckerhoff (PB), the Kentucky Transportation Cabinet (KYTC), and the Ohio Department of Transportation (ODOT). This report includes a description of the field activities, a summary of the encountered subsurface conditions, laboratory test results, and foundation recommendations, along with construction considerations and recommended quality control testing during the project construction phase. Exhibit A-1 in the Appendix provides a general overview map of the project location.

Nineteen (19) test borings were performed for the project; nine (9) test borings were performed in the river with the remaining borings performed on land in Ohio and Kentucky. Each of the test borings was extended to bedrock with approximately 40 to 80 feet of rock coring being performed at each location. Limited environmental screening was performed on soil samples during drilling activities. Shear wave velocity testing was completed at three (3) locations, one (1) within the Ohio River and the remaining two (2) on land. The Suspension P-S velocity logging method was used to measure the seismic wave velocity profiles.

Two (2) previous geotechnical reports were performed by HCN for the BSB replacement project; the 2005 Red Flag study and the 2007 Queensgate alignment study. Six (6) borings were performed as part of the 2007 study. In addition, HCN performed 12 borings for the existing bridge in 1958. These borings have been reviewed as part of this study and have been included in the Appendix.

The existing Brent Spence Bridge links Cincinnati, Ohio and Covington, Kentucky and carries Interstate 75 and Interstate 71 traffic over the Ohio River. The proposed replacement bridge will be located immediately west (downriver) of the existing bridge. At the time this report is being published, three (3) alternatives are being considered for the proposed replacement bridge. The alternatives consist of a tied arch bridge (alternative 1), a two tower cable-stayed bridge (alternative 3), and a single tower cable-stayed bridge (alternative 6). All three (3) alternatives have a main span length of 1,000 to 1,023 feet with the main span piers located near each shore. The roadway will consist of a double-deck truss with six (6) lanes of traffic in each direction as well as shoulders.

Drilled shaft foundations with pier caps extending from the mud line to the waterline are proposed for the bridge replacement. The preliminary drawings provided by PB indicate that minimum 8-foot diameter drilled shafts are anticipated. The following table summarizes the proposed bridge foundations.

Table 1, Summary of Proposed Bridge Types

Bridge Alternative	River Pier Cap Length (feet)	River Pier Cap Width (feet)	Drilled Shaft Spacing (feet)	Number of Drilled Shafts
Tied Arch (Alt. 1)	328	88	24	52
Two Tower Cable-Stayed (Alt. 3)	236	116	20	72
Single Tower Cable-Stayed (Alt. 6)	356	136	20	126

This report focuses on bridge foundations only. Final grading schemes and alignments have not been finalized during this phase of study. Therefore, grading and earthwork related recommendations, along with roadway and embankment design and construction considerations are not discussed further in this report

The following sections include a description of the geology, field activities, encountered subsurface conditions, laboratory test results, and recommendations for drilled pier and driven pile capacities/construction/quality control for the proposed bridge.

2.0 GEOLOGY AND OBSERVATIONS

Currently, the proposed bridge alignment is occupied by a Duke Energy Facility and Substation on the Ohio side of the river. The riverbank is brush and tree covered and relatively steep (approximately 1.5 to 2H:1V). The Kentucky riverbank, also brush and tree covered, extends gradually up from the river's edge to the toe of the levee protecting Covington, Kentucky. On the southern side of the levee, the area is occupied by several small businesses and parking lots.

An overview of the geology in the project area is briefly described below. The subject area lies near the southern extent of the historic glacial progression/regression, which has resulted in a notably variable geology across the region. The general overburden geology is discussed, followed by the bedrock geology in the region. An overview of the seismic geology of the region is also provided. The geology of the region is based on various published and on-line resources and maps, in conjunction with our experience in the general project area.

2.1 General Overburden Geology of Southwest Ohio/Northern Kentucky

An estimated two million years before present time, the first major ice sheet arrived in Southwest Ohio and Northern Kentucky. At the time, the northwesterly-flowing Teays River flowed across West Virginia and entered Ohio near Portsmouth. This ancestral river occurred along with several tributaries, including the north-flowing Licking River. The valleys at that time were only about 150 feet deep, compared with 400 feet deep today.

Between an estimated 1.2 and 2 million years ago, the Kansan and Nebraskan glaciers advanced into Cincinnati and the Northern Kentucky area. At that time, the north-flowing Teays Age Licking River was dammed by the snout of the glacial ice, resulting in deposition of lake clays within the valleys. The base elevation of the lake-filled valley was about elevation 650 feet.

In time, the lake waters rose and eventually overflowed a divide near Madison, Indiana. The glacial meltwaters caused elevated water flow through the new drainage system westward, near Hamilton, Ohio and southwesterly toward Ross and Harrison, Ohio, Lawrenceburg, Indiana, and on to Louisville, Kentucky. The water flow eroded a deep and wide channel, termed the Deep Stage Ohio. The valley bottom was deepened well below today's Ohio, Little Miami, and Great Miami Rivers to about elevation 380 feet.

The Teays Age Licking River abandoned its former course and shifted somewhat westerly, cutting its Deep Stage valley where the present day Licking River occurs. However, in Deep Stage time, the Licking River did not terminate at its present day mouth location. Instead, it continued northerly across the basin of present day downtown Cincinnati, west of Great American Ball Park and northward to what is now called the Mill Creek Valley to join the Deep Stage Ohio River near Norwood, Ohio.

The Illinoian Age glacier then advanced into southwest Ohio about 400,000 years ago. This glacier did not reach Northern Kentucky. The ice dammed the north flowing Deep Stage Ohio River, forming a lake, which extended towards Portsmouth and well into the Deep Stage Licking valley to the south. The resulting deposition above the valley bottom consisted of Deep Stage gravels topped by Illinoian lakebed silts and clays. The lake filled and eventually spilled over directly west from Cincinnati. A new valley was now cut through Anderson Ferry, Saylor Park, and on to North Bend, Ohio. This process created the present day course of the Ohio River. Also occurring at this time, the Illinoian glacier continued to creep southwesterly and deposited till on top of the lake clays.

Over the next 300,000 years, well after the Illinoian glacier retreated, extensive weathering and erosion took place. New valleys were carved by streams, within the partially filled former valleys. The last glacial advance began about 70,000 years ago. This glacier, called the

Wisconsin glacier, retreated slightly and then re-advanced into Northern Hamilton County, Ohio about 18,000 years ago. This glacier left till and then granular outwash from its meltwaters. Subsequent stream erosion has cut terraces into this outwash along many of the valleys.

2.2 General Bedrock Geology of Southwest Ohio/Northern Kentucky

During the Ordovician Period (444 to 448 million years ago), Southwest Ohio, Northern Kentucky and Eastern Indiana was largely covered with a shallow saltwater sea. This environment encouraged the growth of organisms and the precipitation of calcium carbonate that became the dominant source of the calcareous material in the deposits along the sea floor. In the Late Ordovician Period, collisions of eastern North America with ancestral Europe, Africa, and South America caused an upward bulge of the area and formed what is known as the Cincinnati Arch. The Cincinnati Arch is a gentle, wide structure with bedrock inclinations typically less than 1 degree.

Primarily two formations of bedrock are located within the limits of the project and the maximum depth explored; the lower portions of the Point Pleasant Formation and the upper portions of the Lexington Limestone Formation. The Point Pleasant Formation, deposited during the Middle Ordovician, is approximately 90 to 110 feet thick with an upper elevation of approximately 420 feet in Southwestern Ohio. The Point Pleasant Formation consists of interbedded dark argillaceous limestone, brown to black calcareous shales and fossiliferous layers. The amount of limestone increases with depth in this formation. The Point Pleasant Formation is typically more thickly bedded than the underlying Lexington Limestone Formation and contains appreciably more shale.

The Point Pleasant is underlain by the Lexington Limestone, deposited in the Middle Ordovician, which is approximately 100 feet thick in the Tri-State region. The Lexington Limestone is generally light- to medium-gray limestone. Fossiliferous and argillaceous seams are encountered throughout the formation. Interbeds of shale are encountered in this formation, primarily in the upper portions of the formation.

2.3 General Seismic Geology of Project Area

Plate tectonic theories do not adequately explain the mechanisms associated with intra-plate earthquakes such as those which occur in this area. To our knowledge, there are no mapped faults within the project site area. Further, there are no mapped faults which have experienced surface displacement due to seismic activity during the Holocene Epoch (past 11,000 years) within 100 miles of the project site. The closest mapped fault with such movement is the New Madrid Seismic Zone, which is about 200 miles southwest of the site.

A preliminary seismic hazard analysis was performed for the proposed bridge corridor. The steps for the analysis generally include the identification of the seismic sources capable of strong motions at the project site, evaluation of the seismic potential for each capable source, and evaluation of the intensity of the design ground motions at the project site.

For this preliminary analysis, the evaluation of the intensity of ground motions was accomplished using U.S. Geological Survey (USGS) published information regarding the seismic hazard for the Central and Eastern United States. This information for the project site is strongly influenced by the New Madrid Seismic Zone in southeastern Missouri. To a lesser degree, historical local seismicity of Ohio, Kentucky and Indiana contribute to the seismic hazard as well. The USGS Internet website seismic hazard mapping tools were used to estimate the potential ground motions for the project site corridor. For the purposes of this analysis, the design event evaluated was an earthquake whose ground motions have a 2 percent probability of exceedance in 50 years (equivalent to a 10 percent probability of exceedance in 250 years, or a recurrence interval of 2475 years). This is consistent with the classification of the Brent Spence Bridge as a "critical structure."

The USGS mapping evaluation uses a database that considers the contribution of all recorded earthquakes that may influence the project site area. The coordinates at the Ohio River were entered to obtain peak ground accelerations and spectral accelerations at the soil-bedrock interface. The following table summarizes the information obtained for each of the locations for the design event:

Table 2, Preliminary Seismic Hazard Data – Ohio River

		Site-Source Mean Event		**Site-Source Modal Event**		**Relative Contribution**		**CEUS Source Mean Event**	
Criteria	Accel (g)	M	D (km)	M	D (km)	NMSZ (%)	CEUS (%)	M	D (km)
PGA	0.080	6.21	150	7.7	455	14	86	5.94	100
0.2 sec SA	0.179	6.42	183	7.7	455	18	82	6.13	125
0.3 sec SA	0.156	6.73	237	7.7	455	28	72	6.33	150
1.0 sec SA	0.076	7.25	357	7.7	455	51	48	6.74	240

Notes: Accel.=acceleration value, M=earthquake magnitude, D=distance, NMSZ= New Madrid Seismic Zone, CEUS=Central and Eastern US Seismic Zone, PGA = peak ground accelerations, SA = spectral accelerations

The primary conclusions that may be derived from the information presented above are:

- The relative contribution of the New Madrid Seismic Zone is limited except for the spectral accelerations predicted at a period of 1.0 second.
- The relative contribution of the random seismicity of the Central and Eastern U.S. Seismic Zone (CEUS) appear to be higher for spectral accelerations at the other selected periods and for the peak ground acceleration.

These observations suggest that seismic site response analyses should be performed using a series of several time histories that represent the smaller magnitude earthquakes of the CEUS and at least one time history that represents the New Madrid Zone event.

3.0 EXPLORATION

The exploration performed for this study consisted of a geotechnical test boring program. The test borings were supplemented by environmental screening during our drilling activities at each of the test boring locations and geophysical testing at three (3) selected test boring locations. In addition, the collected soil and rock core samples were subjected to an extensive laboratory testing program, which is further discussed below.

3.1 Test Borings

Nineteen (19) soil borings were performed for this project. Nine (9) of the borings were performed within the Ohio River (R-1 to R-8), six (6) were performed on land in Ohio (L-1 to L-3A), and the remaining four (4) on land in Kentucky (L-4 to L-7). See Exhibit A-2 in the Appendix for a boring location plan.

The boring locations were laid out on-site by PhotoScience Geospatial Solutions. Based on a summary report (Exhibit A-12) provided by PhotoScience Geospatial Solutions, a two-person RTK (real time kinematics) GPS crew was mobilized to the site. The crew was equipped with dual-frequency Trimble 5700 Base, Trimble R8 Rover GPS units, and Trimble TRIMMARK 3 Radio, to establish horizontal and vertical control values for the boring locations. The crew used BSB/PSI's control monuments 11 and 12 as base known positions. Both RTK and traditional surveying techniques were used in locating the borings. Each of the river borings were located with a TOPCON GTS223 Total Station by making use of two control points set by RTK near the river's edge. The elevation of the top of the barge was recorded for each of the river borings. When allowable, boring locations on land were located by direct RTK occupation. If the boring location wasn't suitable for direct occupation, a pair of control points were established nearby and then located with the total station.

The borings were drilled with truck and ATV-mounted rotary drill rigs using continuous flight hollow-stem augers to advance the boreholes. The drill rig was placed on a barge to drill the borings located within the Ohio River. The barge was anchored at the boring locations using spuds located at the barge corners. Barge coordination and permitting was performed by HCN. Samples of the soil encountered in the borings were obtained using the split-barrel sampling procedures. Relatively undisturbed samples were obtained by pushing Shelby Tubes into primarily cohesive soils.

In the split barrel sampling procedure, the number of blows required to advance a standard 2 inch O.D. split barrel sampler the last 12 inches of the typical total 18 inch penetration by means of a rope and cathead manual safety hammer with a free fall of 30 inches, is the standard penetration resistance value (SPT-N). For this project, a calibrated automatic SPT hammer was used to advance the split-barrel sampler in the borings performed on this site. A greater efficiency is typically achieved with the automatic hammer compared to the conventional safety hammer operated with a cathead and rope. This SPT N-value is used to estimate the in-situ relative density of cohesionless soils and consistency of cohesive soils.

Published correlations between the SPT values and soil properties are based on the lower efficiency cathead and rope method. This higher efficiency affects the standard penetration resistance blow count (N) value by generally increasing the penetration per hammer blow over what would be obtained by using the cathead and rope method. The effect of the automatic hammer's efficiency has been considered for the test boring performed. N_{60} values have been provided on the boring logs.

Rock coring was performed using wireline NQ and HQ size, double-tube core barrels per ASTM D2113. Water was added during coring to cool the bit and clear the cuttings. The rock coring was performed to explore the characteristics and quality of the bedrock. Recovery (REC) and rock quality designation (RQD) values were measured in the field and confirmed in the laboratory for each core run. Recovery is the length of core recovered as a percentage of the core run. RQD is the sum of pieces of solid core that are 4 inches or longer in length measured along the centerline of the core, divided by the length of the core run. Rock core fractures and breaks due to rock coring and retrieval methods were not included in the determination of RQD. Following the measurement of the recovery and RQD, the samples were placed in wooden boxes and wrapped with plastic and aluminum foil to help maintain the integrity and natural moisture content. Photographs were taken of the rock core in the laboratory and have been included in the Appendix following each boring log.

The soil samples were tagged for identification, sealed to reduce moisture loss, and taken to our laboratory for further examination, testing, and classification. Information provided on the boring logs attached to this report includes soil descriptions, consistency evaluations, boring depths, sampling intervals, and groundwater conditions. The borings were backfilled with cement-bentonite grout prior to the drill crew leaving the site.

A field log of each boring was prepared by the drill crew. These logs included visual classifications of the materials encountered during drilling as well as the driller’s interpretation of the subsurface conditions between samples. Final boring logs included with this report represent the engineer's review of obtained soil samples, driller’s field logs, and include modifications based on laboratory tests of the samples.

All borings were backfilled after their completion and patched at surface (if located within the existing paved areas). Excess auger cuttings were disposed of on the site. The borings were backfilled with a bentonite-cement grout. Details of the backfill materials are included on each boring log.

3.2 Geophysical Testing

Geophysical testing consisting of PS Suspension Logging was performed in three (3) of the test borings (L-1, L-4, R-2A) by GeoVision Geophysical Services. The borings were cored a minimum of an additional 15 feet and cased with 3-inch-diameter PVC pipe for the geophysical testing. Installation of the casing at R-2A encountered an obstruction along the sidewall at 139 feet, which was approximately 50 feet above the total boring depth. The casing was abandoned and grouted in place. The river location was grouted below the mudline and then broken off.

The purpose of the geophysical testing was to acquire compressional (P) wave velocities and shear wave (S_H) velocities as a function of depth. In geophysical testing, a dynamic or vibratory force applied to soil or rock results in wave propagation outward from the source in all directions through that soil and/or rock. In general, three wave types are generated in soil and rock (compressional, shear and Rayleigh waves). A P-wave is a dilational wave that displaces soil or rock particles parallel to the direction of the wavefront and has the highest velocity of the three wave types. An S_H-wave is a distortional wave that displaces soil or rock particles perpendicular to the direction of the wavefront and has a relatively lower wave velocity as compared to the P-wave. For the purposes of this study, the Rayleigh wave is not relevant.

Suspension soil velocity measurements were performed in all borings using the PS suspension logging system, manufactured by OYO Corporation. This system directly determines the average velocity of a 3.3-foot-high segment of the soil column surrounding the boring of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the boring producing relatively constant amplitude signals at all depths.

The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the borehole walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling in the boring and surrounding the source. This pressure wave is converted to P- and S_H-waves in the surrounding soil and rock as it impinges upon the wall of the boring. These waves propagate through the soil and rock surrounding the boring, causing pressure waves to be generated in the fluid surrounding the receivers as the soil waves pass their location.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and recorded on disk before moving to the next depth.

Following data collection, the data was analyzed by GeoVision using the program PSLOG and the results were plotted in Excel. The results of the analysis and further details on the testing and analysis are included in the GeoVision report provided in the Appendix (Exhibit A-11).

3.3 Environmental Screening

All samples were screened for volatile organic vapors associated with petroleum products using the Ambient Temperature Headspace (ATH) method. Screening of soil samples was performed with a Foxboro Toxic Vapor Analyzer (TVA) 1000B flame ionization detector (FID). Vapors are measured as present in the soil sample jar head space, which may be a different concentration than the concentration measured in the soil. The FID yields readings of ionizable vapors in parts per million vapor by volume (ppm v/v) present in the soil relative to ambient air and the calibration gas (methane in air). The FID was factory calibrated to methane in air.

The ATH screening method consists of splitting a soil sample and placing it into new, clean jars with lids. One of the split soil samples from each sampling interval is vigorously shaken to aid in releasing organic compounds and allowed to stabilize. The lid of the sample jar is slightly opened, and the organic vapors in the headspace of the sample jar are then screened with the FID. In the event that FID readings above detection limits were observed, an activated charcoal filter tip fitted to the FID was used to screen the soil samples a second time to identify and quantify the presence of ionizable methane and ethane. Methane and ethane are naturally occurring soil gases typically resulting from the decay of organic matter within the soils. FID readings obtained with the charcoal filter tip represent readings of ionizable methane and/or ethane in ppm v/v. A summary of the screening results is provided in Exhibit A-10 in the Appendix.

In summary, elevated readings, particularly in the river borings, generally appeared to be attributable to the presence of methane and organics. However, elevated readings occurring in fill materials are likely partially attributable to something else. The significance of any of the elevated readings is not really determinable at this point since chemical analysis of the collected samples was not performed and is beyond our scope. Based on the overall field screening readings, visual observations, and general lack of odors, it appears unlikely that the samples at the test boring locations are significantly environmentally impacted. It should be noted however, that the presence of heavy metals cannot be determined unless further environmentally-specific exploration and analysis is performed.

3.4 Laboratory Testing

Selected samples were tested in the laboratory to evaluate the engineering properties of the soil and rock. Laboratory testing included:

- Soil Samples
  - Natural Moisture Content (T265/D2216)
  - Atterberg (Liquid/Plastic) Limits (T89/T90/D4318)
  - Organic Content/Loss-on-Ignition (T267/D2974)
  - Sieve/Hydrometer Analysis (T88/D422)
  - Consolidation Testing (T216/D2435)
  - Triaxial Testing (T296/D2850)
- Bedrock Samples
  - Unconfined Compressive Strength (D7012 Method C)
  - Elastic Modulus (D7012 Method D)
  - Point Load Strength (D5731)
  - Slake Durability Index (D4644)

A factory-calibrated hand penetrometer was used to estimate the approximate unconfined compressive strength of cohesive soil samples. The calibrated hand penetrometer has been correlated with unconfined compression tests and provides a better estimate of soil consistency than visual examination alone. The elastic modulus testing was performed by the Earth Mechanics Institute and the Colorado School of Mines. The remaining soil and rock testing was performed by HCN.

Descriptive classifications of the soil and rock are indicated on the boring logs and are in accordance with ODOT Specifications for Geotechnical Explorations (SGE). Classification was performed by both visual examination and laboratory test results. The test results are provided on the boring logs and included in summary tables in Appendix B of this report.

3.5 Previous Geotechnical Studies

Two (2) previous geotechnical reports were performed by H.C. Nutting for the Brent Spence Bridge replacement project; the 2005 Red Flag Study and the 2007 Queensgate alignment study. No borings were performed for the Red Flag Study. Six (6) borings were performed as part of the 2007 study along the proposed Queensgate alignment located approximately 800 to 1200 feet west of the existing bridge. Borings for this study were performed in Kentucky, Ohio, and the Ohio River. The borings ranged in depth from 75 to 121 feet with a minimum of 45 feet or rock core performed at each location. Limited environmental screening was performed at the time of drilling. The boring location plan and boring logs from these boring have been included in Appendix A of this report.

In addition to the 2007 borings, HCN performed 12 borings for the existing Brent Spence Bridge in 1958. Two (2) borings were performed at each abutment in Ohio and Kentucky. Four (4) borings were performed at each pier location in the river. The borings ranged in depth from 79 to 116.5 feet. Rock coring was performed at each of the eight (8) borings located in the river. The boring logs and location plan have been included in Appendix A of this report.

4.0 FINDINGS

In general the test borings encountered primarily granular soils (both fill and natural) overlying limestone and shale bedrock. The proposed bridge project has been separated into four (4) segments for this report: 1) Ohio-Land, 2) Ohio-River, 3) Kentucky-River, and 4) Kentucky-Land. Due to the generally similar materials encountered in the river borings, the Ohio and Kentucky sides have combined in this section. Detailed borings logs and photographs of the rock core, as well as geotechnical summary sheets (Exhibits A-3 to A-6), are included in the Appendix. The following sections provide generalized descriptions for each area of the project.

4.1 Ohio- Land Borings (L-1, L-1A, L-2, L-2A, L-3, and L-3A)

Existing Fill

Vacuum extraction was performed in the upper 4 to 10 feet of each boring in this area to expose possible existing utility conflicts with the test borings. Below the vacuum excavation, existing fill was encountered in the test borings. The fill material consisted of silt, sandy silt, sand, sand and gravel, and rock fragments (A-4b, A-4a, A-3, A-3a, A-1-b, and A-1-a). Variable amounts of brick fragments, concrete, cinders, and occasional organics (topsoil, wood/fibrous material, and/or decayed matter) were observed in the fill materials. Boring L-2 also includes A-6(b) and A-7-6 fill soils. The thickness of fill ranged from approximately 5 feet in L-1 to 60 feet in L-2 near the bank of the Ohio River. It is our understanding that the existing fill in the Duke Energy property is known to be environmentally impacted and will be remediated in-place or excavated and replaced. The areal extent and depth of the removal is unknown at this time.

The consistency of the existing fill ranged from very soft to soft for the cohesive fill and very loose to loose for the granular fill. Blow counts (N-values) in the existing fill ranged from weight-of-hammer (WOH) to as high as 53 blows per foot (bpf). The average N-value was 11 bpf in the fill. N-values in the fill may not be representative of the actual density/stiffness due to obstructions and its non-uniform composition. Moisture contents of both predominantly cohesive and granular materials varied greatly in the fill, ranging from 9% to 65%.

Natural Overburden Soils

Cohesive soils were encountered in the upper portions of the overburden in this area. The cohesive soils consisted primarily of silt with occasional clay and silt layers. A large percentage of sand and gravel was also present in these soils. The cohesive overburden soils were generally medium stiff to stiff in consistency. Underlying the natural cohesive soils, the overburden soils consisted primarily of gravel and stone fragments with sand, sandy silt, silt, and fine sand (A-1-a, A-1-b, A-4a, A-4b, and A-3). These soils were deposited as alluvial soils by the Ohio River, and as glacial outwash in the deeper profile. The consistency of the overburden soils was generally loose to dense with occasional very loose or very dense zones. Typically, the overburden soils became increasingly dense with depth. Blow counts in the overburden soils varied from WOH to over 100 bpf, with an average N-value of 36. The higher blow counts were encountered just above the bedrock surface or in zones with higher percentages of gravel and rock fragments. Cobbles, boulders, and large rock fragments are likely to be encountered erratically throughout the natural soils and in particular just above the bedrock surface.

Bedrock

Bedrock was encountered in this area at an average elevation of 371 feet. A summary of the depth to bedrock in the test borings is provided in the following table.

Table 3, Summary of Encountered Bedrock – Ohio Land

Test Boring	Surface Elevation (ft.)	Approximate River Depth (ft.)	Approximate Depth to Bedrock (ft.)(1)	Approximate Bedrock Elevation (ft.)(2)
L-1	493.5	-	127.0	366.5
L-1A	491.5	-	121.0	370.5
L-2	496.3	-	115.0	381.3
L-2A	494.5	-	128.5	366.0
L-3	458.7	15.0	88.2	370.5
L-3A	496.1	-	125.0	371.1

(1) The depth to rock indicated in this table is for estimation purposes. Actual depth to rock may vary, as determined by construction conditions and as approved by the engineer based on the encountered field conditions.
(2) Up to 15 feet of variation in the bedrock elevation was observed. We recommend additional test borings be performed during the project design and construction phases to better define the rock surface.

The bedrock consisted primarily of limestone and shale. Interbedded limestone and shale was also encountered primarily in the upper portions of the bedrock. The percentage of limestone in the interbedded zones ranged from approximately 70% to 80%. The percentage of limestone typically increased with depth which is consistent with the gradual transition from the Point Pleasant Formation, which has as much as 50% shale, to the Lexington Limestone Formation, which is primarily limestone. Fossiliferous and argillaceous limestone seams were noted in the bedrock. The thickness of shale seams/layers in the interbedded limestone and shale ranged from approximately 8 inches to less than ¼ inch. Limestone layers ranged from thin partings to 3 feet in thickness, with a typical thickness of approximately 3 to 6 inches.

Rock Quality Designation (RQD) values for the Ohio-Land borings averaged 46% with values generally increasing with depth (see Figure 1). The RQD values ranged from 0% to 100%. The Recovery (REC) values ranged from 40% to 100%, with an average of 97%. The measured RQD and recovery values are summarized in the figure below.

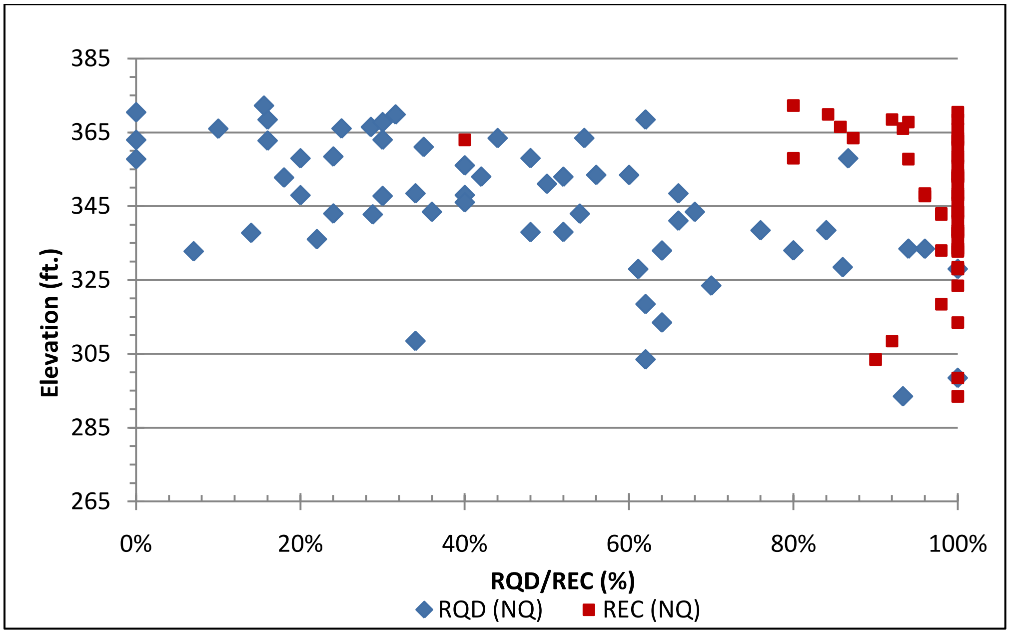


Figure 1, Bedrock RQD/REC Summary – Ohio Land

The overall average unconfined compressive strength (q_u) was 10,938 pounds per square inch (psi) for the Ohio-Land portion. Lower values were seen in samples with shale and argillaceous limestone seams while the higher values were measured in predominately limestone samples. Additionally, lower strength values were observed in the shale samples with generally high moisture contents (See Figure 3). Elastic modulus testing was also performed on select limestone samples. An average elastic modulus of 8,608 kips per square inch (ksi) was observed in this testing (see Exhibit B-6). See Figure 2 below for a summary of the unconfined strengths versus elevation.

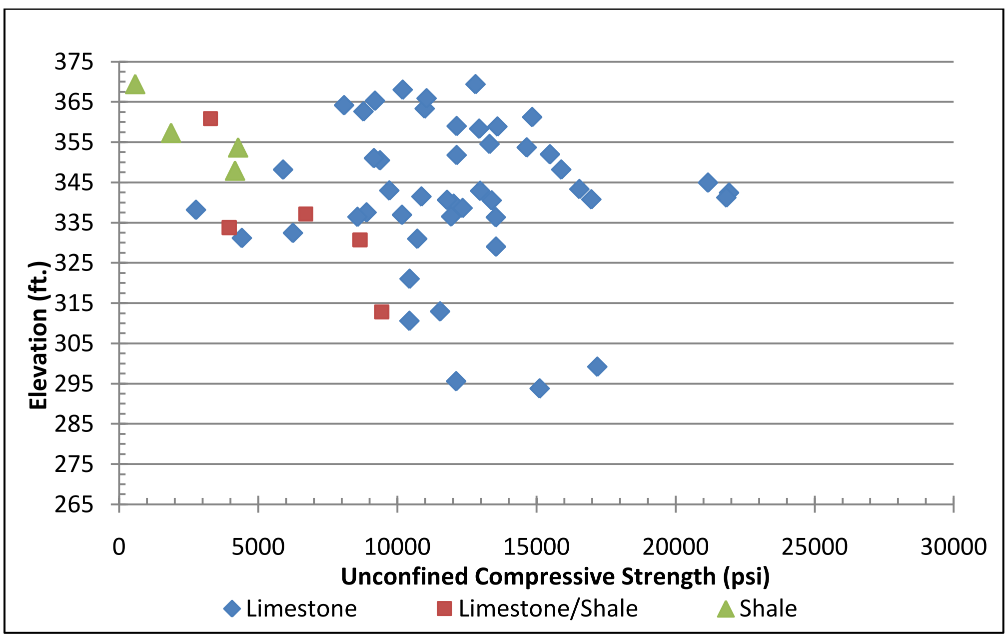


Figure 2, Bedrock Unconfined Strength Summary – Ohio Land

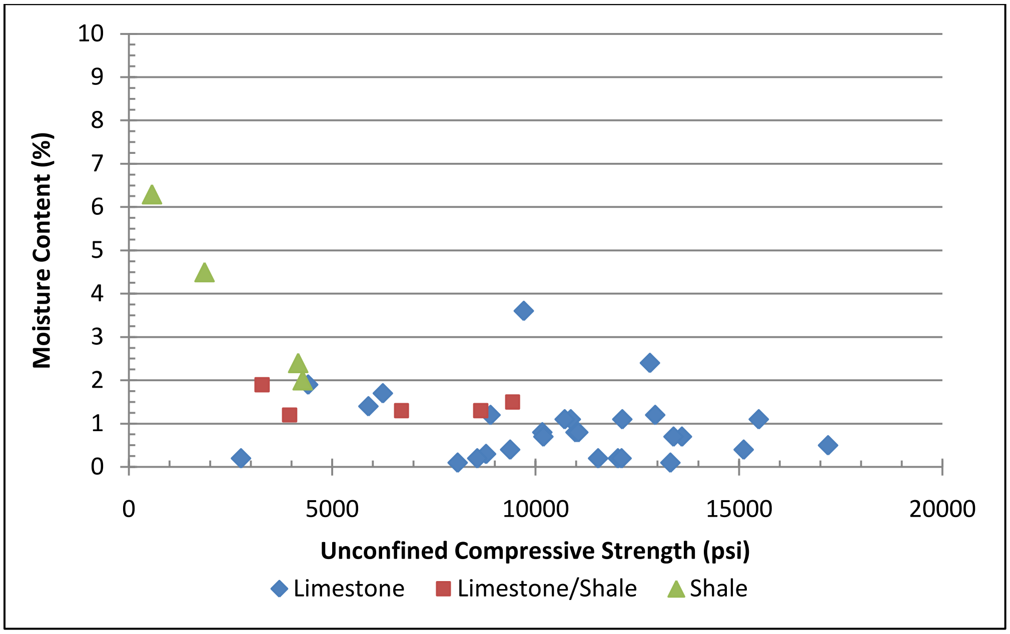


Figure 3, Unconfined Compressive Strength vs. Moisture Content – Ohio Land

Slake durability testing was performed on shale samples to evaluate potential deterioration in the presence of water. Values less than 60% are generally considered susceptible to degradation. The average value was about 77% for this portion of the project. One sample, located at an elevation of 347.2 feet in boring L-2A, had a value less than 60%. The slake durability tests are summarized in Figure 4.

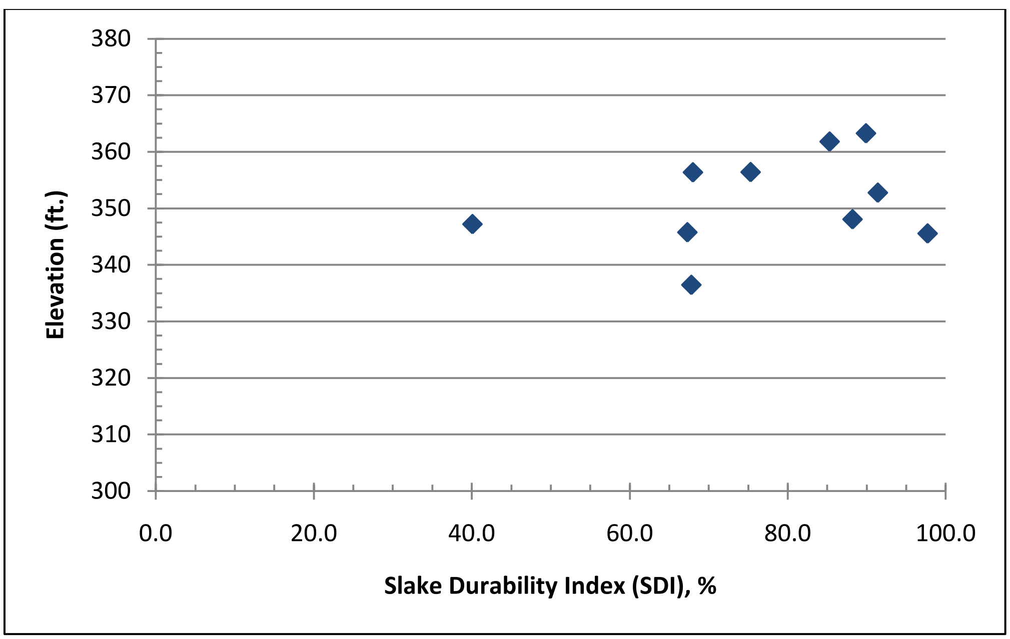


Figure 4, Bedrock Slake Durability Index Summary – Ohio Land

4.2 River Borings (R-1, R-2, R-2A, R-3, R-4, R-5, R-6, R-7, R-8)

Natural Overburden Soils

The borings located within the Ohio River near both the Ohio side (R-1, R-2, R-2A, R-3, and R-4) and Kentucky side (R-5, R-6, R-7, and R-8) encountered predominately granular soils overlying the shale and limestone bedrock. On the Kentucky side, approximately 12 to 30 feet of predominately cohesive soils were encountered above the granular soils. The total thickness of overburden soils ranged from about 51 to 84 feet.

The granular soils encountered in the river borings consisted primarily of sand and gravel (A-1- and A-1-b) as well as occasional fine sand (A-3). These granular soils were mostly medium dense in the upper zones grading with depth, to dense and very dense. The cohesive soils encountered in the upper portions of the Kentucky borings consisted of a mixture of silt and clay soils (A-4, A-6, and A-7-6). The consistency of these soils ranged from soft to medium stiff.

Blow counts in the overburden soils ranged from WOH to over 100 bpf. The higher blow counts were encountered just above the bedrock surface or in zones with higher percentages of gravel, cobbles, and rock fragments. Cobbles, boulders, and large rock fragments are likely to be encountered erratically throughout the soil profile particularly just above the bedrock surface. The average blow count was 24 bpf in the river borings. Natural moisture contents in the overburden soils ranged from about 6% to 49%.

Bedrock

Bedrock was encountered on average at elevation 372 feet in the river. This corresponds to a depth of about 84 feet below the normal pool level (456.36 feet) of the Ohio River. The bedrock consisted of primarily limestone, with interbedded limestone and shale being encountered in the upper portions of the borings. A summary of the encountered depth to bedrock is provided in the following table.

Table 4, Summary of Encountered Bedrock – Ohio River

Test Boring	Surface Elevation (ft.)	Approximate River Depth (ft.)	Approximate Depth to Bedrock (ft.)[(1)]	Approximate Bedrock Elevation (ft.)
R-1	458.0	32.0	87.0	371.0
R-2	458.1	29.0	87.0	371.1
R-2A	457.6	29.0	88.0	369.6
R-3	458.0	28.0	86.5	371.5
R-4	458.0	30.5	86.5	371.5
R-5	458.6	16.0	85.0	373.6
R-6	457.0	-	84.0	373.0
R-7	458.5	21.0	82.5	376.0
R-8	455.7	-	80.0	375.7

(1) The depth to rock indicated in this table is for estimation purposes. Actual depth to rock may vary, as determined by construction conditions and as approved by the engineer based on the encountered field conditions.

Fossiliferous and argillaceous seams were noted in the bedrock. The percentage of limestone in the interbedded layers ranged from approximately 70% to 80%. The percentage of limestone typically increased with depth. Shale seams and layers within the interbedded limestone and shale typically ranged in thickness from thin partings to 6 inches. Limestone layers ranged from thin partings to 3 feet or more in thickness, with a typical thickness of approximately 4 to 8 inches.

Rock Quality Designation values for the Ohio River borings averaged about 76% on the Ohio side of the river and 77% on the Kentucky side of the river. In both areas the RQD generally increased with depth. The RQD values in the river ranged from 0% to 100% while the rock core recovery values ranged from about 50% to 100%, with an average of about 97%. The figure below summarizes the RQD and Rock Core Recovery of samples obtained within the Ohio River.

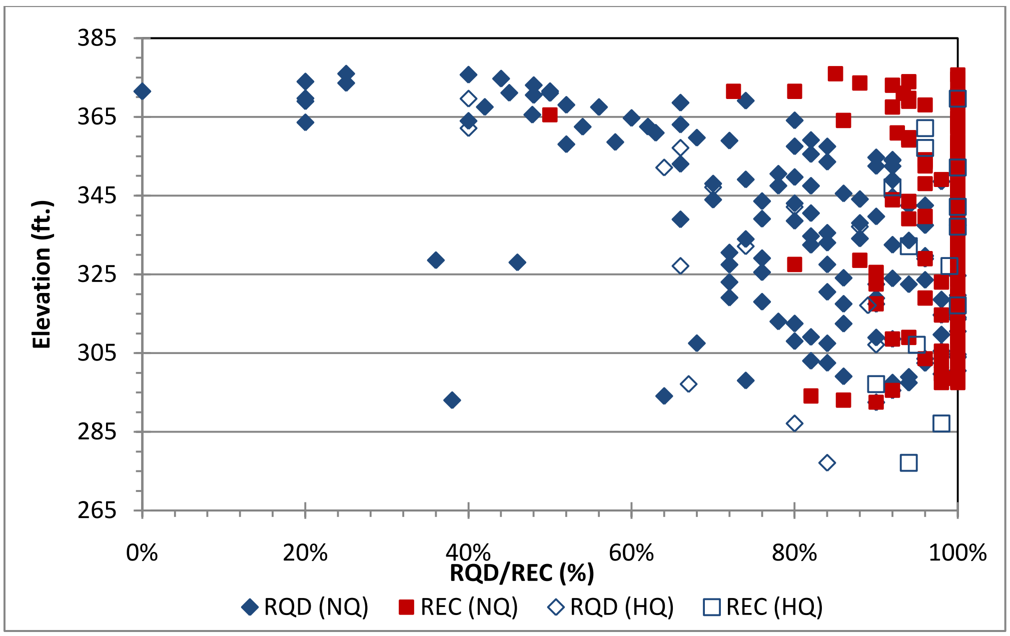


Figure 5, Bedrock RQD/REC Summary – Ohio River

Unconfined compressive strength (q_u) testing resulted in an overall average strength of 11,268 psi on the Ohio side of the river and 11,044 psi on the Kentucky side. Higher strengths were seen in the samples that were primarily limestone while the lower strengths were seen in primarily shale samples. Also, lower strengths were correlated with shale samples with a higher natural moisture content (see Figure 7). Elastic modulus testing was also performed on select limestone samples. Elastic modulus testing yielded an average elastic modulus of 7,787 and 7,794 ksi for the Ohio and Kentucky sides, respectively (see Exhibit B-6). A summary of the unconfined compressive strength on tested rock core samples is shown in the figure below.

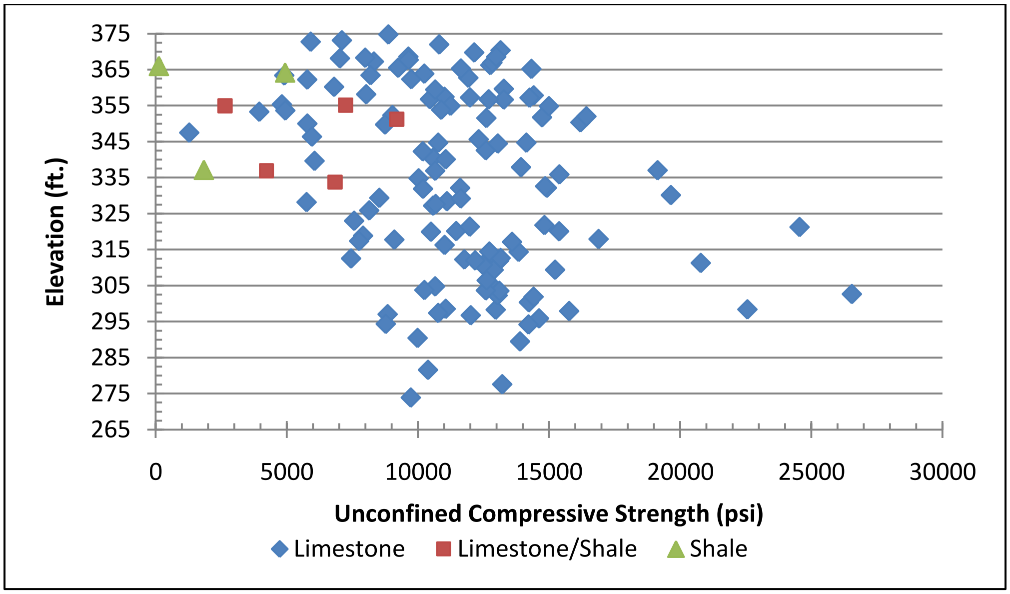


Figure 6, Bedrock Unconfined Strength Summary – Ohio River

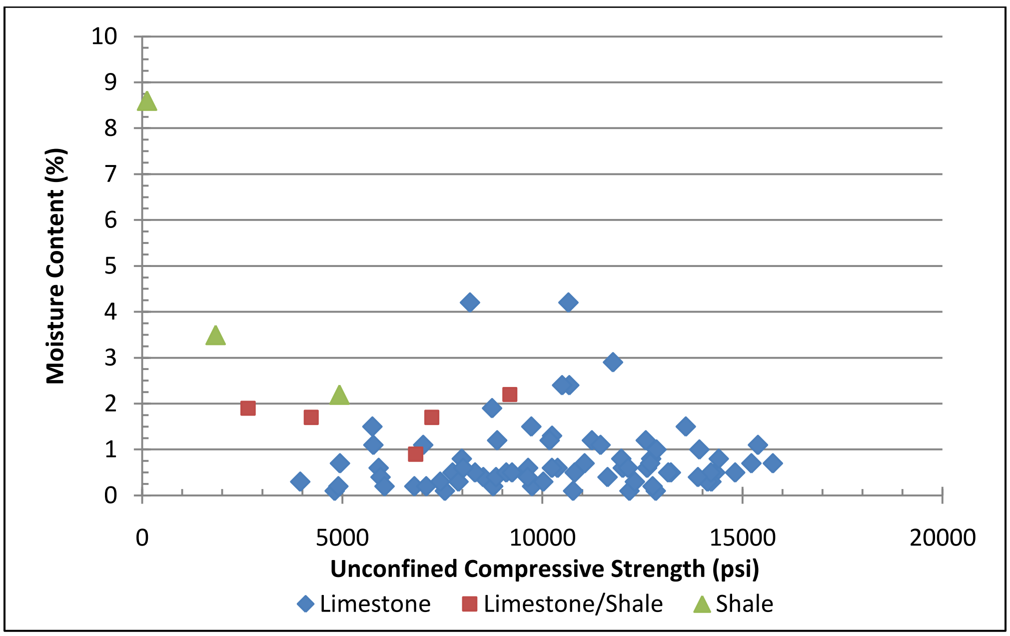


Figure 7, Unconfined Compressive Strength vs. Moisture Content – Ohio River

Slake durability testing was performed on shale samples to evaluate potential deterioration in the presence of water. Values less than 60% are generally considered susceptible to degradation. The average value was 76% for this portion of the project. A total of four (4) shale samples located in borings L-5 and L-6 had a value less than 60%. These values ranged from 36% to 59% and all were in samples above elevation 360 ft., which was within about 10 feet of the bedrock surface.

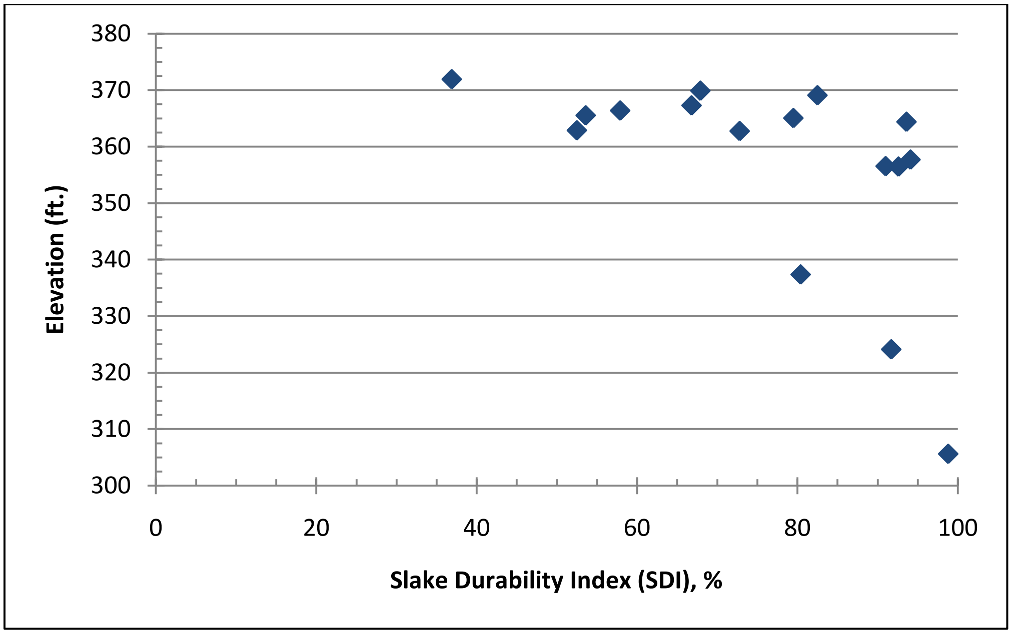


Figure 8, Slake Durability Index Summary – Ohio River

4.3 Kentucky- Land Borings (L-4, L-5, L-6, L-7)

Existing Fill

Vacuum extraction was performed at L-5, L-6, and L-7 to expose possible underground utility conflicts; vacuum extraction was not performed at L-4 located between the levee and the riverfront. Fill material was encountered to depths of about 10 to 25 feet below existing grade. Fill was not encountered in boring L-6; however, it is likely some fill is present within the depth that was vacuum excavated. The fill consisted of silt, sandy silt, and silt and clay (A-4a, A-4b, A-6a, and A-6b) as well as sand, sand and gravel, and rock fragments (A-1-b). Evidence of fill included slag, wood, organics (topsoil, wood/fibrous material, and/or decayed matter), and concrete fragments.

The consistency of the existing fill was generally very loose to loose in the granular fill and medium stiff to stiff in the cohesive fill. Blow counts ranged from 1 to 18 bpf, with an average of 9 bpf. Natural moisture contents in both the granular and cohesive portions of the fill ranged from 17% to 38%.

Natural Overburden Soils

Overlying the thick granular layers at borings L-5, L-6, and L-7, the natural overburden was typically stiff silty clay (A-6a) and medium dense to dense silt or sandy silt (A-4a and A-4b). These layers were approximately 20 feet thick at borings L-5 and L-7, but were about 72.5 feet thick at boring L-6 where no fill was encountered. At boring L-4, located between the levee and the riverfront, the zone consisted of about 20 feet of soft gray clay (A-7-6).

Underlying these soils was mostly sand (A-3) underlain by varying amounts of gravel and gravel with sand (A-1-a and A-1-b). These layers were medium dense in the top layers grading with depth to dense and very dense. Cobbles, boulders, and large rock fragments are likely to be encountered erratically throughout the soil profile particularly just above the bedrock surface. Blow counts in the natural overburden soils ranged from 3 to over 100 bpf. The average value was 46.

Bedrock

Bedrock was encountered on average at about elevation 372 feet in this area. The bedrock consisted of primarily limestone as well as interbedded limestone and shale in the upper portions of the bedrock. Occasional fossiliferous and argillaceous seams were present in the limestone. The percentage of limestone in the interbedded layers ranged from approximately 70% to 80%. The percentage of limestone typically increased with depth. The thickness of shale seams/layers in the interbedded limestone and shale ranged from approximately 8 inches to less than ¼ inch. Limestone layers ranged from thin partings to 3 feet in thickness with a typical thickness of approximately 3 to 6 inches. A summary of the depth to bedrock is provided in the following table.

Table 5, Summary of Encountered Bedrock – Kentucky Land

Test Boring	Surface Elevation (ft.)	Approximate Depth to Bedrock (ft.) (1)	Approximate Bedrock Elevation (ft.)
L-4	480.0	104.0	376.0
L-5	486.3	107.0	379.3
L-6	485.7	108.5	377.2
L-7	484.4	100.0	384.4

(1) The depth to rock indicated in this table is for estimation purposes. Actual depth to rock may vary, as determined by construction conditions and as approved by the engineer based on the encountered field conditions.

Rock Quality Designation values for the Kentucky-Land borings averaged about 53% and ranged from about 0% to 92%. Rock core recovery values ranged from about 17% to 100%, with an average of about 53%. The figure below summarizes the RQD and Rock Core Recovery for samples obtained in the land borings in Kentucky.

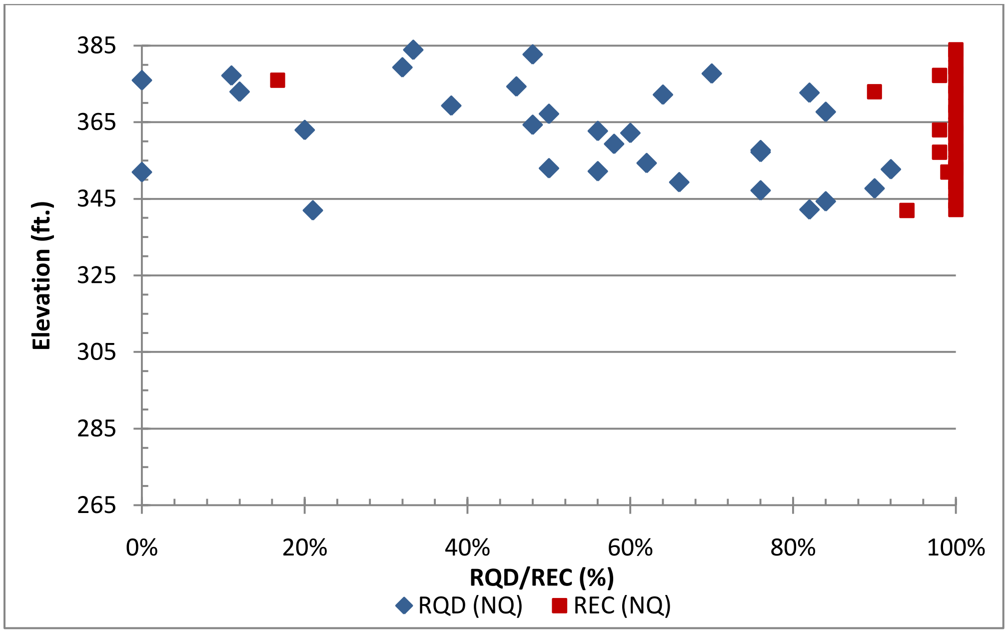


Figure 9, Bedrock RQD/REC Summary – Kentucky Land

The overall average unconfined compressive strength (q_u) was 11,989 psi for the Kentucky portion of the project. Figure 6 shows a summary of the unconfined compressive strength test results. Compressive strengths were generally greater in shale samples with lower moisture contents (figure 11) and those samples consisting primarily of limestone. In addition to the strength testing, elastic modulus testing was performed on select limestone samples. The average elastic modulus in this area was 9,104 ksi (see Exhibit B-6).

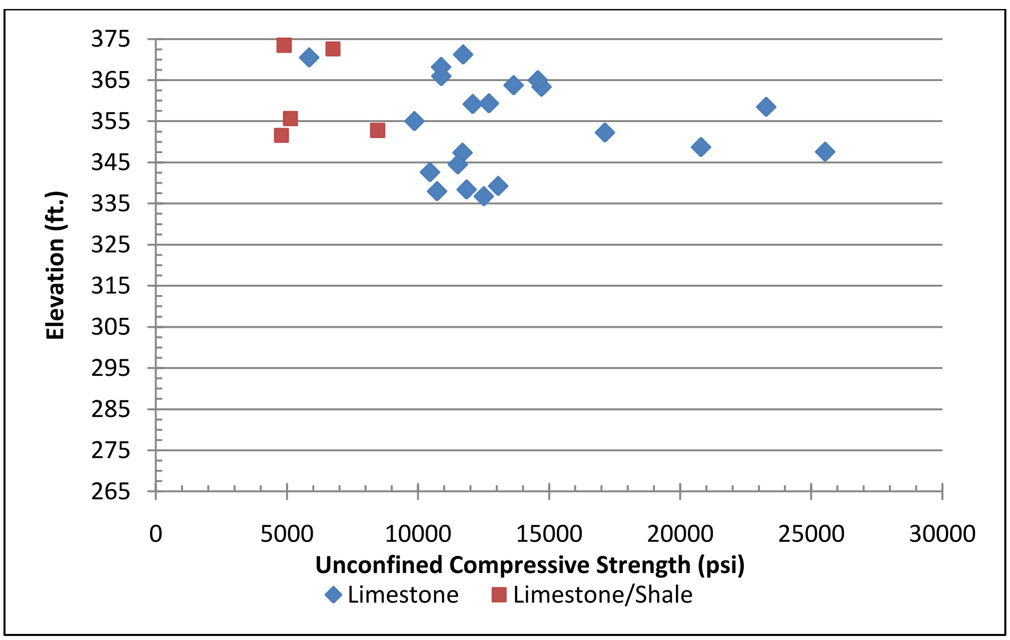


Figure 10, Bedrock Unconfined Strength Summary – Kentucky Land

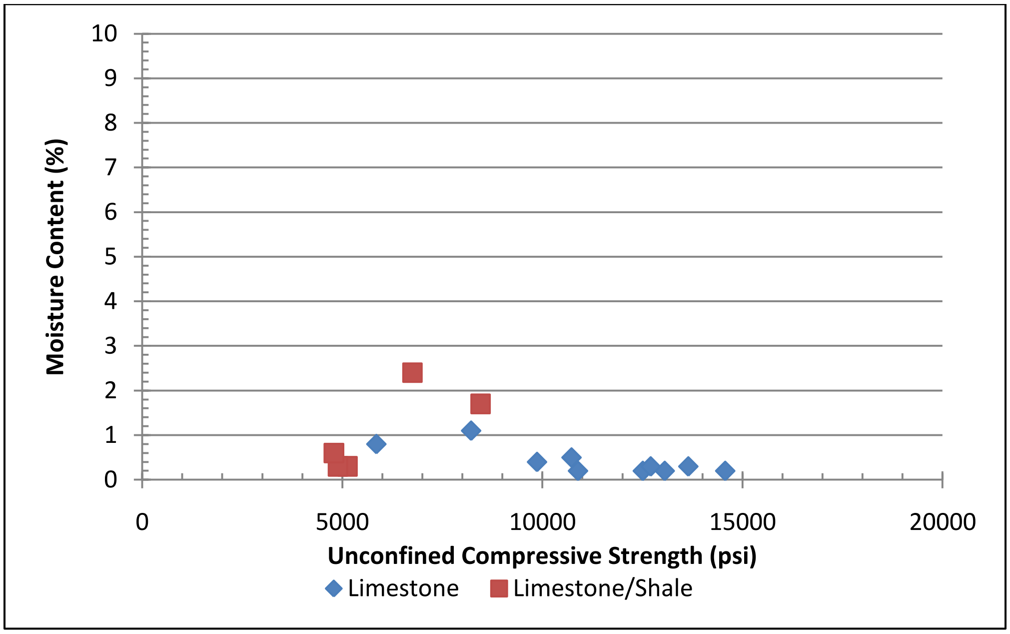


Figure 11, Unconfined Compressive Strength vs. Moisture Content– Kentucky Land

Slake durability testing was performed on several samples in the Kentucky-Land portion of the project. Values less than 60% are generally considered susceptible to degradation. The average value for this area was 63.3%. Five (5) of the eight (8) samples in this area have slake durability indexes of less than 60%.

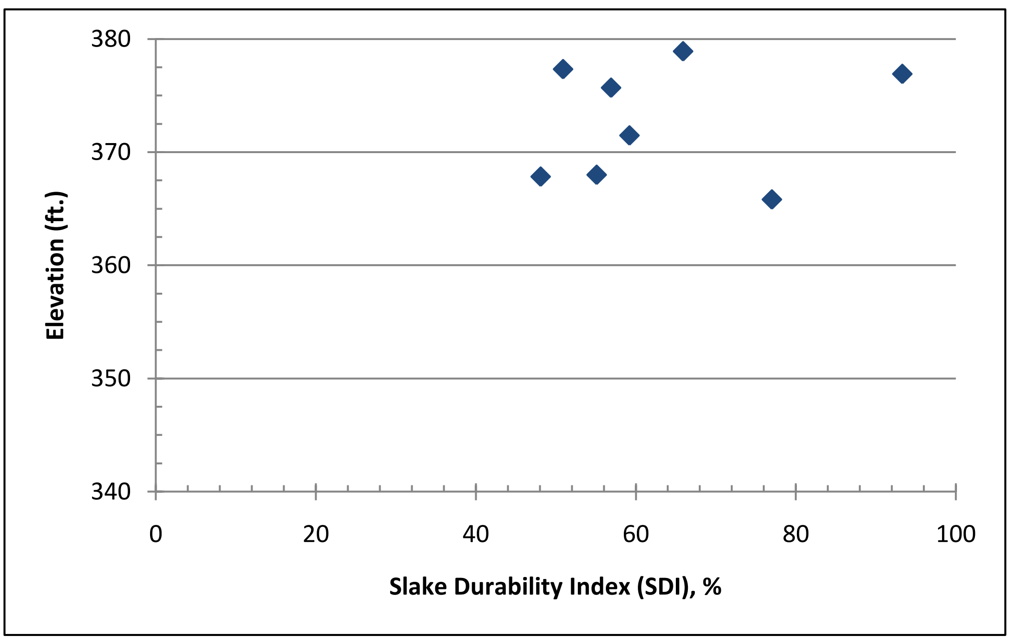


Figure 12, Slake Durability Index Summary– Kentucky Land

4.4 Groundwater

Groundwater observations were made during drilling. Water level readings are not considered reliable since water was introduced to the borehole during rock coring operations and in granular soils to prevent heave into the augers. Long-term (24-hour) water level observations were not made since the test borings were backfilled immediately upon completion for safety reasons. The groundwater levels measured during drilling may not accurately represent the prevailing groundwater levels at the test boring locations. The groundwater in the boreholes requires sufficient time to stabilize and reach the static groundwater level. To obtain long-term groundwater measurements, it is necessary to install water level observation wells or piezometers.

Perched water may be encountered at higher elevations within the existing fill and at the fill/natural interface. The long-term groundwater levels are influenced by amount of precipitation, degree of surface runoff, and primarily the water level in the Ohio River.

The Ohio River, forming the border between Ohio and Kentucky, is about 1,300 feet wide at the existing Brent Spence Bridge location. The normal pool elevation of the Ohio River in the area of the bridge is about 456 feet. On the Kentucky side of the Ohio River, the nearest tributary is the Licking River, which is located about 1 mile to the east of the existing I-71/I-75 roadway. In Ohio, the nearest tributary is the Mill Creek, which is located about ½ to ¾ of a mile to the west of the existing roadway. The USGS map indicates several smaller water features, including lakes, ponds, and manmade ponds/reservoirs.

Water drainage in the corridor study area is generally achieved by diverting water towards the Ohio River and/or adjacent connecting streams. Due to the relatively large watershed that the Ohio River covers upstream to the north and east, periodic flooding is generally common in low-lying areas along the Ohio River in the Cincinnati/Covington area. The following flood information was obtained from the Louisville District U.S. Army Corp of Engineers for the project location:

- Normal pool – Elevation 456.36 feet
- Ordinary High Water Mark – Elevation 468.5 feet
- 100 Year Flood – Elevation 497.10 feet
- 500 Year Flood – Elevation 512 feet

The river level ranged in elevation from a low of 455.1 feet to a high of 465.9 feet during drilling (5/17/2010 to 9/4/2010). At the time the borings located in the river were drilled (6/29/2010 to 9/4/2010), the river elevation ranged between about 455 and 456 feet.

4.5 Shear Wave Velocity Profiles

The results of the PS Suspension Logging at test borings (L-1, L-4, and R-2A) were evaluated for the AASHTO seismic Site Class in accordance with AASHTO LRFD 2010 Section 3.10.3.1. The shear wave velocity results for each boring are included in Appendix A as Exhibit A-11. The interval shear wave velocity values were used to calculate the average shear wave velocity of the upper 100 feet. The approach described in Method A of Table C3.10.3.1-1 was used to obtain the following results:

Location	V_s (feet/second)	Site Class
L-1	754	D
L-4	940	D
R-2A	2565	B (C[1])

Note: 1. Defaults to C since rock is more than 10 feet below bottom of pile cap.

4.6 Previous Geotechnical Studies

Soil borings were performed by H.C. Nutting for both the existing Brent Spence Bridge (1958 study) and the Queensgate alignment (2007 study). The results of these test boring programs were generally consistent with the borings performed for this study. The major differences are the lack of overburden soils in the river and the depth to bedrock is shallower by approximately 50 feet along the Queensgate alignment.

The overburden soils encountered in the 1958 borings consisted of existing fill overlying primarily granular soils. The existing fill consisted of sandy clay, silty clay, sand, gravel, and cinders. Various amounts of brick fragments and organic material were also encountered throughout the fill. Underlying the fill the natural soils were primarily granular consisting of sand and gravel. Silty and sandy clay was also encountered, mostly in the upper 10 to 20 feet of the natural overburden soils. Bedrock was encountered in these borings at elevations ranging from 371 to 375.2 at the river pier locations, 379 to 381 feet at the Ohio abutment, and 382 to 387 feet at the Kentucky abutment. The bedrock encountered consisted of interbedded limestone and shale.

Six (6) borings were performed in 2007 to investigate the subsurface conditions for the proposed Queensgate alignment located approximately 800 to 1200 feet west of the existing bridge. The overburden soils encountered in the land borings were generally consistent with the borings performed for this study. Existing fill consisting of both cohesive and granular soils as well as cinders, brick fragments, and organics was encountered in the borings located on land in Ohio and Kentucky. The natural soils underlying the fill were primarily cohesive in the Kentucky borings and granular in Ohio. The major difference between the 2007 borings and the borings performed in 1958 and 2010 is the lack of overburden soils in the river and the shallower depth to bedrock. At the two (2) borings performed in the river along the Queensgate alignment only 0.5 feet of overburden soils (sand and gravel) were encountered overlying the bedrock. The bedrock at this location was approximately 50 feet higher than at the existing bridge location. This difference in bedrock elevations is consistent with the geology of the area with the existing bridge located within the ancient Deep Stage Licking River.

5.0 ANALYSIS AND RECOMMENDATIONS

The following text provides foundation recommendations for the proposed Brent Spence Bridge project. Details regarding construction considerations and field testing of the foundations are also provided. The provided foundation recommendations and construction considerations are each critical to bridge foundation design and should not be viewed independently. Grading and earthwork plans, along with roadway and embankment alignments have not been finalized at this time. Therefore, details beyond the proposed bridge foundations are not discussed in this report.

Based on review of various foundation types, construction practices, and major river crossing projects, it is our opinion that drilled shafts are an effective and cost-practical foundation for bridge support at both the interior (river) pier and abutment (land) locations. In consideration of the structure type, loads and constructability, it appears that drilled shafts are the preferred foundation choice for this project.

Driven pile types have been considered as a feasible foundation alternative. Both H-piles and CIP piles have been evaluated for the bridge abutments and approach spans. H-piles driven to bedrock have been considered for the river foundations and additional discussion is provided in section 5.3.

The following sections further develop these foundation recommendations. Following the foundation recommendations, detailed discussions regarding quality control during construction and field testing are provided. A well-conceived field testing program and strict quality control during construction are considered part of the foundation design process and are essential to the long-term performance of the foundation system.

5.1 Foundation Discussion

Tower foundations like those expected for the proposed bridge require large compressive, uplift, lateral, and overturning moment capacities. A general subsurface profile of the bridge alignment consists of overburden soils, primarily granular, overlying unweathered shale and limestone bedrock. Based on the limited number of borings, the bedrock surface on the Ohio land side varied by up to 15 feet. Bedrock elevation variation within the Ohio River was typically less than 3 feet. On the Kentucky land side, bedrock elevation varied nearly 10 feet between the test boring locations. We recommend additional test borings be performed during the project design and construction phases to better define the bedrock surface. The general profile at the project site is considered suitable for consideration of both driven pile and drilled shaft foundation types.

Driven piles could consist of steel pipe piles (CIP) or H-piles. Steel piles can provide high-strength, they are easy to handle, and are capable of carrying large loads to deep loading bearing strata. For depths greater than about 60 feet, splicing of the piles is usually required to achieve the design length. Driven steel piles do not produce excavation spoils requiring disposal. A common problem with driven steel piles is deviation from vertical (lack of plumbness) and loss of load capacity when driving through soils with cobbles, rock fragments, or into an uneven bedrock surface. In addition, battered piles may be required to provide the lateral capacities required for the tower foundations. Driven steel piles could be considered for the bridge particularly on the portions over land. Preliminary design recommendations have been provided for driven piles at the abutment locations on land. If driven piles are deemed viable, further analysis could be performed and detailed recommendations developed.

Drilled shafts consist of cast-in-place, reinforced concrete piers socketed into the bedrock. Drilled shafts are a common type of construction in the area and are familiar to contractors. Drilled shafts allow for a reduction in the pile cap size and the overall number of foundation elements compared to driven piles. The construction of drilled shafts would require steel permanent casing, possible use of slurry, as well as the disposal of the excavated spoils. Drilled shafts are the recommended option for the proposed bridge foundations in the river and can also be used for the land foundations.

5.2 Drilled Shafts

The bridge structure can be supported on a cast-in-place drilled shaft foundation that is sufficiently embedded into shale and limestone bedrock. Drilled shaft performance is strongly related to the effectiveness of the construction technique in preserving the integrity of the bearing materials and ensuring the structural integrity of the reinforced concrete shaft element. The typical construction sequence is anticipated to consist of the following components:

- Install a temporary casing through water and upper overburden soils,
- Using polymer slurry, drill through the overburden soils,
- Place permanent casing into the upper shale bedrock,
- Excavate the bedrock socket under polymer fluid to the design tip elevation,
- Roughen the sidewall bedrock surface to remove any slick or decomposed material,
- Thoroughly clean out the shaft base,
- Place steel reinforcement and concrete

The following sections discuss design recommendations along with certain aspects of the construction sequence for drilled shafts, as they relate to design.

5.2.1 Design Parameters

Given the subsurface conditions and the provided preliminary concept design, drilled shafts are recommended for the bridge foundations. Design parameters for both axial end bearing and side resistance for rock socketed drilled shafts are provided. Shafts will also need to be evaluated for lateral resistance which may control rock socket embedment depths. Strain compatibility when using side and end bearing would need to be evaluated as well as group settlement, as part of final design when the drill shaft geometry and layout are finalized.

5.2.1.1 Axial Loading

The drilled shaft design parameters for axial loading were developed based on the test borings, detailed review of rock cores, laboratory testing, and review of published literature. Design of the drilled shafts can include both base resistance and side resistance in the bedrock. An estimate of the total scour should be performed to determine what side resistance is available from the overburden soils. The load-displacement relationship (strain compatibility) between base and side resistance should be considered in the design since the maximum side resistance typically occurs at a lower displacement than the maximum base resistance.

Reasons cited in published literature for neglecting side resistance of rock sockets include; (1) possibility of strain-softening behavior of the sidewall interface (2) possibility of degradation of material in the borehole wall in argillaceous rock, (3) uncertainty regarding the roughness of the sidewall. Site specific laboratory testing has not been performed to determine load-deformation behavior on the rock/concrete interface. Based on published literature on similar bedrock material as those encountered for this project, strain softening is not commonly observed and therefore strain compatibility should not be a factor in combining side resistance and base resistance. This tendency is likely related to the dilatency of the shaft/rock interface. Field load testing along with careful quality control during construction to confirm sidewall conditions should be performed to confirm and justify our assumption that side resistance can be used in combination with base resistance. Laboratory testing can also be performed in addition to field testing if strain softening is a concern.

Based on the subsurface data collected during field exploration, drilled shafts would be socketed within the Point Pleasant formation or the much deeper Lexington Limestone formation. A detailed discussion of the bedrock geology, bedrock characteristics and strength properties has been presented before. The Point Pleasant formation consists of interbedded limestone and shale. The amount of limestone increases with depth in this formation. The unconfined compressive strengths obtained from intact rock core samples yielded average values of 8,000 to 10,000 psi. However, significant variability was observed with the standard deviation being about 3200 psi. The rock core in the upper 30 ft. exhibited RQD values being less than 50% in many locations. The shale samples were brittle and at many locations could not be tested as they were easily broken and a sufficient length of sample was not available for testing. Considering the low RQD values, rock core recovery, careful review of the rock core, presence of thin soft zones of shale (which could not be tested) and the variability across the site, the unconfined compressive strength ($q_{u\ design}$) suggested for use in design has been selected to be lower than average tested values. The selected $q_{u\ design}$ value also considers the disturbance and constructability considerations which has a significant impact on design performance of drilled shafts.

The Lexington Limestone is more competent than the Point Pleasant formation. However, this typically occurs 50 to 60 feet below the top of encountered bedrock. Review of the rock core and laboratory testing data indicates that higher base and side resistance is likely available within this formation. However, considering the depth of rock socket needed to bear within the formation, we do not think it will be cost effective to design shafts bearing within this formation.

Using AASHTO LRFD design procedures, the ultimate capacities of the drilled shafts were determined based on unconfined compressive strength of the intact rock and the rock mass rating (RMR). The elastic modulus used in design has been reduced to two-thirds of the average measured value to account for the shale in the rock mass. Most of the elastic modulus tests were performed on limestone specimens. The bridge project was broken into four areas (Ohio-Land, Ohio-River, Kentucky-River, and Kentucky-Land) and recommended values are given for each area. A summary of the recommended values is provided in the following table. Calculations for these values are provided in Exhibit C-2 in the appendix.

Table 6, Drilled Shaft Design Input Values

Location	Avg. RQD (%) (upper 30 ft.)	Avg. Unconfined Compressive Strength Used In Design (q_u, psi)	Design Elastic Modulus (E_I, ksi) (upper 30 ft.)
Ohio-Land	38%	4,000	6,043
Ohio-River	67%	4,800	5,311
Kentucky-River	59%	4,800	4,757
Kentucky-Land	49%	4,000	6,073

Table 7, Drilled Shaft Design Parameters

Location	Rock Mass Rating (RMR)		Rock Mass Modulus (E_M, ksi)	Nominal Shaft Resistance (q_S, ksf)			Nominal Base Resistance* (q_P, ksf)
				Rock	<	Concrete	
Ohio-Land	42	III (Fair Rock)	1,220	14.3		22.7	350
Ohio-River	57	III (Fair Rock)	1,627	17.7		22.7	350
Kentucky-River	57	III (Fair Rock)	1,627	17.9		22.7	350
Kentucky-Land	42	III (Fair Rock)	1,220	14.3		22.7	350

*Values reported are limiting values (see discussion)

The nominal base resistance is computed using the empirical relationship (FHWA-NH1-10-016):

$$q_{bN} = N^{*}_{CR} \cdot q_u$$

$N^{*}_{CR} =$ empirical bearing capacity factor for rock

$q_u =$ unconfined compressive strength of rock

$q_{bN} =$ nominal base resistance

Based on various research studies, N^{*}_{CR} =2.5 is recommended for design when the following conditions are met:

- The shaft is bearing on rock that is either massive or tightly jointed
- No solution cavities or voids exist beneath the base
- A clean base can be achieved and verified using conventional clean-out equipment

The empirical factor N^{*}_{CR} can vary and be as low as 0.4 if there are joints and discontinuities in the rock mass. O-cell testing data (1995) of the Maysville New US 62/68 Ohio River Bridge on the Point Pleasant Formation bedrock indicated that ultimate end bearing was 160 tsf at 1.0 inch of base movement. A description of O-cell testing is provided in section 5.2.5.2. Displacements required to mobilize the base resistance are related to shaft diameter. The design guidelines for geotechnical strength are based on limiting the displacement at nominal resistance to 2.5% of diameter, considering that larger diameter shafts will be used. We have limited the nominal base resistance to 350 ksf to satisfy the above discussed criterion. Also, for loads greater than 350 ksf, large creep movements are likely. The limiting of base resistance also appears reasonable considering the need to ensure strain compatibility between side resistance and end bearing and limiting the overall foundation movement to less than 1.0 inch. The bridge structure may be able to tolerate settlements greater than 1.0 inch and the tolerable settlement (total and differential) will need to be determined jointly by the geotechnical and structural engineer. Project specific load testing will be performed to help determine load displacement data and modify design values, as needed.

Additional axial design considerations include:

- Minimum rock socket the greater of 1.5B or 10 feet.
- Per AASHTO section 10.8.3.5.6 and table 10.5.5.2.4-1, resistance factors for axial compression and uplift (socket resistance), considering static load testing is performed, are 0.7 and 0.6, respectively. If applied to a single shaft supporting a bridge pier, then the resistance factors should be reduced by 20 percent (per AASHTO section 10.5.5.2.4).
- Overburden should not be considered to contribute axial capacity due to strain compatibility considerations.
- The base capacity may be limited by allowable shaft movement.

The drilled shafts are expected to be subjected to lateral loads and should be designed accordingly. The shaft lengths should be designed such that the lateral deflections are acceptable due to the anticipated lateral loads. Non-linear p-y analyses can be used to estimate the shear and moment along the length of the shaft. The following table provides recommended LPILE parameters to be used for static lateral analysis of the drilled shafts.

Table 8, Recommended Soil Parameters for Single Lateral Pier (LPILE) Static Analysis

Soil Type	Moist Unit Weight of Soil – γ (pcf)	Buoyant Unit Weight - γ (pcf)	LPILE P-y Modulus – k (pci)	Internal Angle of Friction – ϕ(°)	Undrained Shear Strength – Su (psf)	Uniaxial Compressive Strength – q_u (psi)	Strain Parameter – ε_{50} or k_rm
Cohesive Existing Fill (stiff to very stiff)[1]	120	57.6	500	--	2,000	--	0.007
Granular Existing Fill (medium dense to dense) [2]	120	57.6	50	32	--	--	--
Granular Natural Soil (loose to medium dense) [2]	125[2]	62.6	80	33	--	--	--
Granular Natural Soil (dense to very dense) [2]	130[2]	67.6	100	36	--	--	--
Cohesive Natural Soils (medium stiff to stiff) [1]	125	62.6	300	--	750	--	0.01
Cohesive Natural Soils (very stiff) [1]	125	62.6	750	--	3,000	--	0.006
Limestone Bedrock [3]	165	102.6	--	--	--	10,000	0.0005

[1] - Anticipated to be modeled as "stiff clay without free water
[2] - Anticipated to be modeled as "sand (Reese)"
[3]- Use a modulus of elasticity value of $8x10^6$ psi for limestone bedrock

The parameters provided in the above table are considered to be "initial" parameters under static loading. The basis of the lateral analyses is soil-structure interaction, and the behavior of the soil is non-linear depending on the loading conditions and the stiffness of the structural element. The reaction/resistance of the soil is dependent on the movement of the structure and hence the input soil properties are not fundamental properties of the soil. Therefore, lateral analysis is an iterative process based on an initial set of soil parameters that may need to be adjusted depending on the initial results and engineering judgment. HCN/Terracon requests the opportunity to review and comment, as necessary, on the lateral analysis results.

5.2.1.2 Group Effects – Axial Loading

Considering that all the drilled shafts will be socketed a sufficient distance in competent bedrock and because the strength of the bedrock is anticipated to be greater than the strength of the shaft/rock interface, group effects are generally not expected to control design. Superposition of stresses from adjacent drilled shafts may result in increased deformations of group of shafts relative to that of single shafts, however, settlement of drilled shafts founded on bedrock are anticipated to be small and group effects should be minimal. A more detailed analysis of shaft groups will be needed once the shaft diameter, spacing, loading and bedrock embedments have been finalized.

Drilled shafts which develop their capacity from a combination of side resistance and end bearing should be installed with a minimum center-to-center spacing of 2.5 times the shaft diameter. No reduction in individual axial shaft capacity is needed for this spacing. Adjacent shafts should not be constructed on the same day. If the drilled shafts are spaced closer than 2.5D, then further evaluation to determine group effects will be needed.

5.2.1.3 Group Effects – Lateral Loading

The lateral resistance in the scour zone (computed by the design team) should be neglected. When laterally loaded drilled shafts are used in closely spaced groups, a given shaft will deflect further under a given system of loads that if loaded when the neighboring shafts are not present, and bending stresses will be greater. It is therefore recommended to consider group effects due to loading when shaft spacing is less than about six diameters in any direction. A “p-multiplier” to accommodate the group effects can be considered. For group effects, then “Pm” factor provided in this table can be used.

Table 9, Recommended P-Multiplier, P_M, Values for Design by Row Position

	Design P-multiplier, P_M			
Pile Spacing (c-c)	3D	4D	5D	≥6D
Lead row	0.7	0.85	1.0	1.0
2nd Row	0.5	0.65	0.85	1.0
3rd and Higher Rows	0.35	0.5	0.7	1.0

FBPIER, a computer program capable of considering coupled effects of the drilled shafts and pier cap in addition to much more complex three-dimensional group configurations, three-dimensional loading conditions, and GROUP in 2-D and 3-D should be utilized for analyses of pier groups.

5.2.1.4 Uplift Design

The drilled shafts can be subject to uplift loads. The uplift nominal unit side resistance are the same for uplift and compression. However, a lower resistance factor is recommended for uplift than axial compression. The recommended resistance factors for uplift are typically 0.10 less than those for compression.

5.2.1.5 Downdrag

The effects of downdrag should be evaluated as part of the final drilled shaft design. The relative settlement of the soil to the shaft as a function of time and depth must be known in order to determine the magnitude of downdrag. For preliminary considerations, downdrag is not expected to be a significant factor for the river foundations. However, once the grading and bridge foundation details, including installation procedures have been determined, evaluation of downdrag should be performed. The effects of the change in river levels under normal pool and flood conditions will also need to be considered during final design.

5.2.2 Scour Considerations

Bridge scour is the loss of soil by erosion due to flowing water around bridge supports. Scour analysis is being performed by the design team. We would anticipate that the majority of the overburden soils are susceptible to scour. Axial capacity within the overburden soils have been neglected to account for scour, strain compatibility, and other constructability considerations. Effects of scour that must be taken into account for drilled shaft design (FHWA-NHI-10-016) include (1) changes in subsurface stress, (2) reduced embedment and therefore changes in axial and lateral resistances, and (3) possible changes in the structural response and resulting foundation force effects. AASHTO Specifications also require evaluation of bridge foundations for two scour conditions (1) design flood scour condition for foundation strength and service limit state and (2) check flood scour condition for extreme limit state.

Scour should include the general scour and channel construction scour plus local scour immediately around the bridge piers. The effects of the existing Brent Spence Bridge piers relative to scour development should also be considered in the analysis.

The minimum rock sockets for drilled shafts should be designed below the maximum (predicted design) scour elevation in bedrock. Generally, we would anticipate that the limestone and shale bedrock is not erodible. A final determination of the erodibility of shale bedrock would need to be made after detailed scour analyses by the design team. In addition, the estimated scour depths should be considered in the lateral load analysis.

5.2.3 Drilled Shaft- Cofferdams

Construction of the drilled shafts located in the river can be performed in cofferdams. A cofferdam is a temporary structure designed to keep water and/or soil out of the excavation in which a bridge pier or other structure is built. Sheet piling is driven around the work site, seal concrete is placed into the bottom, and the water is pumped out. The concrete seal course is used to seal off the water, resist its pressure, and also can be used to act as a slab to brace against the inward movement of the sheet piles.

Several types of cofferdams could be considered for the proposed construction; braced, cellular, or double-walled sheet piles. The proposed cofferdam will experience several loading conditions. The designer should consider hydrostatic, soil, current, waves, and ice load as well as construction loading. Accidental loading, such as due to a ship strike, and seismic loading may also need to be considered.

As an alternative to a traditional cofferdam the shafts could be installed from a temporary trestle. Then the footing forms would be assembled above the water level and lowered around the shafts to the required level. A tremie seal is then placed, the form dewatered, the shafts cut off at the desired level, and the footing placed. It is our understanding that this option was used successfully on the Audubon Bridge over the Mississippi River in Louisiana. This method can accommodate a wide fluctuation in river levels and may be less costly than cofferdams.

5.2.4 Drilled Shaft – Construction Considerations

5.2.4.1 General Discussion

Drilled shaft construction generally falls into three (3) categories based on the method of construction. These include the dry method, the casing method, and the wet method. Selection of the appropriate method is dependent on the subsurface conditions at a site and is typically the contractor's responsibility to select the appropriate method. Based on the drilled shaft construction extending into bedrock to achieve the desired capacities at locations within the river or in close proximity to the river, we do not anticipate dry construction methods will be feasible. Wet construction methods, including utilization of casing, in combination with drilling slurry, is anticipated at the river and land abutment locations. The following sections further develop feasible construction methods, provide criteria for drilled shaft construction, and address other relevant construction considerations.

Random miscellaneous fill, both manmade and river debris, are anticipated along the river banks. Such deposits may consist of, but not be limited to, abandoned utilities, boulders, foundations, tree trunks, wood, concrete slabs, etc. Dense sands and gravel were encountered in lower portions of the overburden soils. Cobbles and boulders may be encountered in the outwash deposits, which may cause difficulties during drilled shaft construction. Based on discussions with the project team, we understand that the existing fill on the Ohio landside within the existing West End Duke Energy Substation will be environmentally remediated. If the remediation effort includes removal and replacement of the existing fill soils, then the majority of obstructions are anticipated to be removed; however, in-place remediation efforts will not alleviate the presence of the possible obstructions and variable fill. At the time of this report, such environmental remediation evaluation and efforts have not been completed. The presence of the variable fill and associated environmental concerns at all locations should be further evaluated during the final study.

5.2.4.2 Drilled Shaft Installation

Construction of a drilled shaft requires boring a hole of a specified diameter and depth and then backfilling the hole with reinforced concrete. The selection of equipment and procedures for constructing drilled shafts is a function of the shaft dimensions, the subsoil conditions, and the groundwater characteristics. Consequently, the design and performance of drilled shafts can be significantly influenced by the equipment and construction procedures used for construction and also by method of placement and properties of concrete. Construction procedures and methods are of paramount importance to the success of the drilled shaft installation at this project site.

Drilled shaft contractors who participate on this project should be required to demonstrate that they have suitable equipment for this project, and adequate experience in the construction of drilled shafts of the required size and depth, and with similar subsurface conditions. A detailed installation plan along with equipment and methods should be submitted by the contractor for review and approval by the design team.

The installation of the drilled shaft is critical to the successful performance of the shaft. Extending the drilled shaft to the proper depth and careful preparation of the borehole are critical during the drilled shaft construction process. Although construction techniques and methodologies may vary between contractors, the following criteria are considered minimal in the design and construction of the drilled shaft foundations. Project specifications must be developed that present all requirements for drilled shaft construction and address the specific requirements for the project.

1) It is recommended that the approximate top of rock and design bottom elevation be shown for each drilled shaft on the plans, with these elevations being determined using the test borings and lateral and axial load analyses. The “minimum lengths” should be based on lateral load requirements, while “estimated lengths” would reflect axial resistance requirements and will be verified by load tests. Minimum lengths should be based on lateral load requirements, while estimated lengths would reflect axial resistance requirements and will be verified by load tests. The final bearing elevation should be determined by inspection of each shaft hole in the field by qualified geotechnical personnel. We recommend additional test borings be performed during the design phase of the project to better define the rock surface due to variations encountered in the borings performed for this study.

2) The specifications should be clear that the design bottom of the drilled shaft elevations shown on the plans is for estimation purposes only. Actual determination of the top of rock and bottom elevation will be made from examination of materials brought to the surface on the drilling tools by the project geotechnical engineer. As an additional quality control measure, pre-coring at drilled shaft locations could be performed to assess bedrock quality and conditions.

3) The specifications should require that no concrete be placed until the dimensions, bottom elevation, bearing socket depth, and excavation for each shaft has been observed and is to the satisfaction of the geotechnical engineer. A Shaft Inspection Device (SID), mini-SID, or Downhole Camera System (DHC) could be employed for inspection of the drilled shafts prior to concrete placement. This will allow for visual

inspection of the bottom conditions. The mini-SID is a camera, lights, and feelers gauges housed in a steel bell. The bell is pressurized with dry nitrogen as it is lowered in the slurry to keep camera free of slurry. Once at the bottom, water jets are used to clear the lens to expose the shaft bottom for camera inspection.

4) Sonic caliper testing should be performed after the shaft base has been cleaned to determine and confirm as-built dimensions and compare them to the planned design dimensions. At a minimum, sonic caliper testing should be performed on technique and test shafts, and some selected production shafts.

5) Due to the random nature of the fill at the abutments, and the presence of outwash sand and groundwater, full length temporary steel casing should be used and be available on-site to prevent shaft collapse during drilling and concrete placement. The specifications should state that casings be required to stabilize loose or caving materials, or to seal off any water-bearing zones. A concrete core barrel or other suitable tool should also be available on site, if an obstruction within the fill or in the cobble/boulder zone immediately above the bedrock cannot be penetrated with the drilled shaft equipment.

6) The permanent casing should be strong enough to withstand handling stresses, withstand the pressures of concrete and of the surrounding earth and groundwater, and to prevent water seepage.

7) A permanent steel casing seated within the upper shale bedrock is recommended for the river drilled shafts. The permanent steel casings provide additional strength, abrasion protection, ductility, and confinement for the bending stresses in the drilled shafts and facilitate construction by providing a stable environment in which to construct rock sockets. If the permanent casing is used for structural support, consideration must be given to corrosion of the steel. Also, the full structural capacity cannot be assumed within a certain development length at the top and bottom of the casing. The casing will provide confinement, and may allow a reduction in the spiral or hoop reinforcement, particularly if large shear reinforcement is found to be necessary. They can also assist in avoiding any significant issues with bottom cleanout or entrapped debris.

   If the permanent casing is used for structural support, consideration must be given to corrosion potential of the steel. The structural design should evaluate the effectiveness of the casing to resist bending moment as the full structural capacity cannot be assumed within a certain development length at the top and bottom of the casing. The casing will provide confinement and may allow a reduction in the shear reinforcement.

8) If water exists in amounts greater than three inches in depth or enters at a rate of more than twelve inches per hour then the shaft excavation should be filled with slurry. A positive head of slurry or concrete, relative to water trapped outside the casing, must always be maintained within the casing to reduce the risk of water and/or soil from

infiltrating into the shaft and contaminating the concrete. An improper head balance could potentially cause water and/or soil to flow into the shaft and compromise the concrete integrity.

9) It is recommended that the contractor have appropriate equipment on site to facilitate excavation through variable fill and cobble/boulder zones. The contractor should prepare attachments for the drill rig, such as but not limited to, a rock auger and/or core barrel, attachments to break up the hard loam with rock fragments, and a muck/cleanout bucket to clean the bottom of the shaft effectively. The drill rig should have adequate torque and downpressure to facilitate drilling or coring through the variable materials and very dense/hard zone.

10) Concrete placement should be continuous and the discharge end should allow the discharged concrete to flow freely in all directions. If concrete placement is interrupted, the water on top of the concrete and all surficial concrete that has become contaminated with water must be completely removed to fresh concrete prior to final concrete placement to complete the drilled shaft. Shaft excavations should not be left open for an extended period of time.

11) Crosshole Sonic Logging (CSL) testing should be performed on every production drilled shaft as well as the technique and test shafts. The use of CSL testing will confirm adequate structural integrity of the shafts. A minimum of six (6) inspection tubes measuring 2 inches in diameter should be installed to facilitate CSL testing; however, the actual number of inspection tubes is dependent on shaft diameter. More detail is provided in section 5.2.5.1.

12) Due to the urban nature of the surrounding site, and close proximity of the existing bridge and other structures, a preconstruction survey should be performed prior to construction. We recommend that vibration monitoring be performed along the existing bridge during casing installation using vibratory methods. Vibration monitoring should also be considered during construction near sensitive structures and/or underground features.

Due to the potential risk of variable groundwater conditions within the granular zones, full length permanent steel casing will be required to seal off water bearing and saturated granular zones during drilling. We recommend polymer slurry or other type of heavier slurry (bentonite is not recommended) be added to the drilled hole throughout the entire drilled shaft excavation to resist hydraulic head and prevent collapse of side walls.

The bridge test borings encountered wet primarily granular soils overlying the bedrock. For the river borings the use of permanent casing and/or drilling slurry will be necessary for prevention of caving-in of these wet and granular soils and to produce a seal along the soil-rock contact to minimize infiltration of groundwater into the socket. In addition, permanent casing provides confinement and will increase the flexural stiffness and capacity.

5.2.4.3 Rock Socket Sidewall Disturbance

The drilled shafts will be socketed into the underlying bedrock and develop their capacity based on a combination of end bearing and side friction. The condition of the sidewalls of the shaft within the rock socket is critical to the capacity of the drilled shaft. Based on the test borings and recovered bedrock at the land and river boring locations, the predominant bedrock profile consists of shale and/or limestone. Therefore, careful consideration should be given to the construction technique and participation of an experienced contractor. It is recommended that artificial roughening of the rock sockets through use of grooving tools or other measures be used during final pass.

A roughened bedrock sidewall at the concrete-bedrock interface is preferred since increased side resistance develops as opposed to a smooth surface. Smearing of the shale/argillaceous zones in the presence of even minor amounts of water seepage can cause the surface of the rock to become softened. Softening of the sidewall or the creation of a smooth sidewall during drilling can reduce side friction by greater than 50 percent. This effect should be considered during assessment of the contractor's proposed drilled shaft construction method.

5.2.4.4 Additional Comments/Considerations

The slake durability test provides an index for rock that will weather and degrade rapidly by measurement of the physical breakdown of a rock sample after a series of wet/dry cycles with mechanical agitation by tumbling in a drum. Rock with slake durability index less than 60% are considered prone to rapid deterioration and formation of "smear zones" when the borehole well is exposed to water.

Slake durability testing was performed on portions of the shale bedrock. The SDI (slake durability index) ranged from about 40 to 98 percent – averaging about 73 percent. The effect of drilling fluid on maintaining the integrity of the shale during construction has been documented in several studies. These studies showed the use of polymer slurry during SDI testing showed a markedly improved value and is preferred for use during drilling of the rock socket. Additional slake durability testing using riverwater and potential slurry mixes should be performed during the final study or prior to construction to further evaluate the impact that the drilling fluid has on the shale.

5.2.5 Drilled Shaft – Quality Control

The performance of a drilled shaft is dependent on the structural strength, geotechnical strength, deformation properties of the soil and rock, pile-soil/rock interaction, and the applied loads. Quality control is critical to the success of the deep foundation system performance. Quality control of drilled shafts can be divided into three categories; diligent inspection, integrity testing and load testing. We recommend both integrity and load testing be included in the specifications for the proposed bridge foundations.

5.2.5.1 Integrity Testing

Integrity Testing should be employed to assess the structural integrity of the drilled shafts. This testing evaluates the concrete quality, method of placement, construction method, and workmanship. Several methods can be employed including cross-hole sonic logging (CSL), crosshole tomography (CT), and gamma-gamma logging (GGL).

Crosshole sonic logging (CSL) is currently the most commonly used method for quality assurance of drilled shaft concrete. This method provides little indication of concrete soundness outside the cage. The method requires steel (preferred) or plastic tubes installed in the drilled shaft and tied to the rebar cage. One CSL tube should be placed for each foot of shaft diameter. After the shaft is drilled the cage is lowered into the hole and the concrete is placed. The tubes are filled with water as an intermediate medium. After curing for several days, a sound source and receiver are lowered, maintaining a consistent elevation between source and sensor. A signal generator generates a sonic pulse from the emitter which is recorded by the sensor. Relative energy, waveform and differential time are recorded, and logged. This procedure is repeated at regular intervals throughout the shaft and then mapped. The graphs from the various combinations of access tubes are compared and a qualitative idea of the soundness of the concrete throughout the shaft can be established.

Gamma-gamma logging (GGL) can also be performed for evaluation of the drilled shafts. Gamma-gamma logging uses the same principles as nuclear density testing commonly employed in construction. GGL is performed within PVC inspection tubes cast into the shaft during construction. The tubes can could also be used for CSL testing. The gamma-gamma probe, which consists of a radioactive source and gamma photon detector separated by a length of shielded material, is lowered and raised within the tubes. During the test, gamma particles are emitted into the concrete surrounding the PVC tube. Some of the gamma particles are scattered back to the detector in the instrument. GGL is performed continuously along the shaft length with gamma count rates collected at set intervals. Multiple inspection tubes, placed around the interior of the steel reinforcing cage, are provided within a pile to obtain a representative sample of the shaft. Typically, one inspection tube per 0.3 meter (1 foot) of shaft diameter is used.

Considering the high loads supported by the drilled shafts, it is recommended that 100% of the shafts be tested using crosshole sonic logging. Crosshole tomography should also be used to develop two and three-dimensional images of signal velocities and assist in quality assurance of drilled shaft concrete. Crosshole tomography testing should be performed when CSL testing indicates significant anomalies are present.

5.2.5.2 Load Testing

As a means to demonstrate the installation plan and to verify the adequacy of the construction methods, tools and quality control/assurance procedures, test shafts should be constructed consistent with the planned procedures for production shafts. The test shafts can be used to provide site-specific measurement of axial performance under the as-built conditions.

Site-specific field loading tests offer the potential to improve accuracy of the predictions of performance and reliability of the constructed foundations. Load testing can provide information on base resistance, side resistance (total and as a function of depth), and load versus displacement relationships. Both axial and lateral resistance can be determined using the appropriate type of load tests. Because site-specific field loading tests reduces some of the variability associated with predicting performance, the use of large resistance factors are justified when loading tests are performed. Per AASHTO section 10.8.3.5.6 and table 10.5.5.2.4-1, resistance factors for axial compression and uplift (socket resistance), considering static load testing is performed, are 0.7 and 0.6, respectively. If applied to a single shaft supporting a bridge pier, then the resistance factors should be reduced by 20 percent (per AASHTO section 10.5.5.2.4). The AASTHTO guidelines in section 10.8.3.5.6 should be adhered to when developing the load testing program.

We recommend axial pier load testing be performed for the proposed bridge foundations. Lateral load testing should also be considered based on the design loads compared to the calculated lateral resistance and pier head movement under the design loads.

Axial pier load testing can be performed using static or dynamic methods. Static load testing generally involves the application of the load through the use of a reaction frame anchored by four or more piers. With the large loads expected for the proposed bridge foundations this method may be costly and difficult to perform, particularly for drilled shafts located in the river. Another method that has been successfully used for large diameter shafts and should be considered here is the Osterberg Cell (O-Cell). The Osterberg Cell consists of a sacrificial hydraulic jack(s) attached to the base of the reinforcing cage and placed in the drilled shaft. After the concrete has cured to a specified strength, the cell is pressurized and load is applied bi-directionally; upward against side friction and downwards against the base friction. Instrumentation including tell-tales and strain gages are used to measure deformation and movement of the shaft. The advantages of employing Osterberg testing versus traditional load testing is no reaction frame is required, higher applied loads can usually be applied, and the side and base resistance components are directly measured.

Considering the size of the project and subsurface variations, a minimum of four (4) load tests is recommended. One (1) test should be performed at each of the river piers and at least one (1) at the Ohio approach structures and at least one (1) at the Kentucky approach structures. The load test locations should be selected based on the loading conditions and evaluation of bedrock conditions. It is recommended that the technique shaft(s) be installed prior to the installation of the load test shafts to allow for an assessment, and if necessary, modifications of the contractor's proposed means and methods of drilled shaft construction before starting work on any of the load test shafts. Once load capacities are finalized and construction means/methods are established, a detailed load testing program can be developed.

Where the design of the foundation is controlled by considerations of lateral loading and significant cost savings are possible with an aggressive design model it may be appropriate to consider lateral load tests to validate or improve the design models. As with the axial testing, both static and dynamic methods can be considered. Static methods typically involve using a hydraulic jack to push two adjacent shafts apart. Load cells and displacement gages are placed between the shafts to measure the applied load and lateral deflection of the shaft head. Dynamic lateral load testing can be performed using the Statnamic system applied horizontally to the shaft head. This method can apply loads 1,000 tons or greater and may be more appropriate for considering impact loading such as vessels or ice. Lateral load testing of single piers or group of piers can be performed.

5.3 Driven Piles

5.3.1 Driven Piles- Design

Driven H-piles to rock were considered for the pier locations in the river. The overburden profile is primarily granular in nature. During drilling some large size gravel and cobbles were also noted in the granular profile. There is an approximate average of a 6 to 10 foot thick cobble/boulder zone above bedrock along the entire bridge alignment. Based on our experience and preliminary driveability analyses, H-piles will not be able to be driven to bedrock. Significant pile damage (even with pile points) is likely. Refusal within the cobble layer is likely at variable depths. We do not recommend that H-piles tip in the cobble zone due to long-term creep/settlement concerns and the reliability of mobilizing end bearing within the highly variable cobble zone. Potential scour, lateral loads, buckling potential of piles in the scour zone, the large number of piles in the pile groups, and the size of the pile cap are some other factors that should also be considered.

Pipe piles filled with concrete (CIP piles) or H-piles could be considered only for support of the approach span piers located on land. We have performed a preliminary analysis to evaluate the load capacity and driveability of 14 and 16 inch diameter CIP piles and HP14x73 piles. The piles develop their capacity through a combination of skin friction and end bearing. Per the 2007 ODOT Bridge Design Manual, 14 inch diameter CIP piles (0.25 inch thickness) can be designed for a Nominal Bearing Value, R_{ndr}, of 390 kips while 16 inch diameter CIP piles (0.375 inch thickness) can be designed for 450 kips. HP14x73 piles can be designed for a Nominal Bearing Value of 440 kips.

Using the laboratory testing results and the test boring data, DRIVEN software was used to evaluate the pile capacities. A representative boring was chosen for both the Ohio (L-2A) and Kentucky (L-5) portion of the project for this preliminary analysis. Final driven pile design should consider borings at each approach pier location due to variations in the subsurface conditions. In addition, factors such as settlement/fill placement and pre-drilling through debris in the existing fill would need to be considered in the final design.

Based on the DRIVEN analysis, we estimate that the maximum R_{ndr} value of 390 kips (14-inch pile), 450 kips (16-inch pile), and 440 kips (HP14x73) can be achieved on the Ohio and Kentucky land portions of the project at the following elevations. These values do not account for any predrilling, negative skin friction or potential scour effects. Negative skin friction will need to be considered if settlement of the soils may occur. In addition, if debris is encountered within the existing fill, then pre-drilling for the pile may be required. If pre-drilling is performed, then adjustments would be required to the design pile length. Final analyses should also consider remediation efforts within the Duke Energy facility on the Ohio land side. A resistance factor (φ_{dyn}) of 0.7 should be applied for piles installed per ODOT CMS Items 507 and 523. The minimum pile spacing should be 3 pile diameters such that a group efficiency of 1.0 can be used in axial design.

Table 10, Preliminary Driven Pile Recommendations

Location	Pile Dimensions/Type	R_{ndr} (kips)[1]	Estimated Pile Tip Elevation (feet)
Ohio Abutment (L-2A)	14 inch/CIP	390	401
Ohio Abutment (L-2A)	16 inch/CIP	450	407
Ohio Abutment (L-2A)	HP14x73	440	401
Kentucky Abutment (L-5)	14 inch/CIP	390	408
Kentucky Abutment (L-5)	16 inch/CIP	450	411
Kentucky Abutment (L-5)	HP14x73	440	405

[1] Confirm by restriking piles

A driveability analysis of these piles was performed using GRLWEAP. The analysis shows that driving of both the 14 and 16 inch CIP piles and the HP14x73 piles is feasible to the recommended tip elevation. GRLWEAP software performs wave equation analysis to assess the ability of the proposed pile driving system to install the piles to the required capacity and desired depth within the allowable driving stresses prior to driving piles in the field. The preliminary analysis was performed using the ICE 40-S model hammer.

The pile driving contractor should provide data for the proposed pile driving system prior to commencement of production piles. WAVE Equation analyses should be utilized to assess the ability of the proposed pile driving system to install the piles to the required capacity and desired depth within the allowable driving stresses prior to driving piles in the field. Approval of the proposed driving system (by the engineer) should be required prior to any field load testing program.

The preliminary calculations and results performed using DRIVEN and GRLWEAP have been included in Appendix C of this report.

5.3.2 Construction Considerations

Driven piles for bridge support at the abutments should be installed to depths as required to mobilize design capacities. The capacity of each individual pile should be confirmed during driving using established criteria based on pile load testing. The use of dynamic formulas is a helpful guide but becomes increasingly limited in such soil profiles **and is not recommended for use to establish the production pile driving criteria**.

Prior to installing production piles, a load testing program should be undertaken. This program should involve both dynamic testing during test pile driving, and static pile load tests. Specifically, we recommend the following:

1. Using data provided by the pile driving contractor, use the WAVE Equation analyses (such as GRLWEAP) to assess the ability of the proposed driving system to install piles as to the required capacity and desired penetration depth within the allowable driving stresses. Approval of the proposed driving system (by the Engineer) should be required prior to any field load testing program.

2. Dynamic pile testing is recommended on the piles on which static load tests are performed. The indicator (test) pile testing should be performed to monitor hammer and drive system performance, assess pile installation stresses and integrity, as well as to evaluate pile capacity. It is suggested that dynamic testing be performed during both initial and restrike driving. The testing during initial driving is primarily to monitor drive system performance and driving stresses. Dynamic testing during restrike is recommended since it yields a better indication of long-term pile capacity. The dynamic load test data should be analyzed using CAPWAP analyses to determine the actual pile capacity. The final production pile driving criteria and final driving system approval will be based on CAPWAP test results.

3. Static loading testing should also be performed per ODOT guidelines. The load testing program should be reviewed by the geotechnical engineer prior to implementing the load testing program to allow for modifications. It is recommended that at least two static pile load (compression) tests for each design capacity be performed on both the Ohio and Kentucky sides of the river. If significant uplift loads are present, the uplift load test(s) should also be performed. Lateral load test(s) may also be needed if large lateral loads are anticipated and based on computed load deflection response.

4. It is recommended that the piles which are statically load tested be restruck with dynamic testing within 48 hours after completion of the static load test so that a correlation between static and dynamic test results can be obtained for reference across the site. The restrike driving sequence should be performed with a warmed up hammer and shall consist of striking the piles for 50 blows or until the pile

penetrates an additional 3 inches, whichever occurs first. Also, CAPWAP analyses of the dynamic pile testing data should be performed on the data obtained from the end of the initial driving and the beginning of restrike of indicator piles. CAPWAP is an analytical method that combines field measured data with wave equation type procedures to predict the pile's static bearing capacity and resistance distribution.

5. Perform dynamic load testing on the first two production piles and about 5 percent of all piles during installation. The production pile driving criteria may continually need to be modified based on the results of these dynamic tests.

The bridge foundation piles should be spaced at least a distance of 3 times the pile width/diameter dimension. This spacing is to eliminate group effects for axially loaded piles. For laterally loaded vertical piles, detailed analyses (such as LPILE and/or GROUP) will be needed to assess pile spacing effects. Additional details regarding spacing are discussed in the drilled shaft section of this report.

The program GROUP was developed to compute the distribution of loads (vertical, lateral, and bending moment) from the pile cap to piles in a symmetrical group. The program also computes deflection, translation, and settlement of the cap. The program generates internally the nonlinear response of the soil, in the form of t-z curves for axial loading and p-y curves for lateral loading. The equations of equilibrium are satisfied, and compatibility is achieved between pile movement and soil response, and between the movement of the cap and pile head movement. Once the pile configurations, pile head fixity, and lateral loads are known, detailed lateral load analyses for pile groups can be performed.

Settlement of pile groups will need to be evaluated once the pile group geometry and loading has been finalized. Downdrag should also be included (if applicable).

Due to the urban nature of the surrounding site, and close proximity of the existing bridge and other structures, a preconstruction survey should be performed prior to construction. We recommend that vibration monitoring be performed along the existing bridge during pile driving. Vibration monitoring should also be considered during construction near sensitive structures and/or underground features.

5.4 Seismic Considerations

We based our approach for the seismic considerations on the following documents:

- AASHTO LRFD Bridge Design Specifications
- AASHTO Guide Specifications for LRFD Seismic Bridge Design
- Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (MCEER/ATC-49)

The AASHTO documents specify designing for the life safety performance objective considering a seismic hazard corresponding to a seven percent probability of exceedance in 75 years (return period of approximately 1,000 years) for an "essential" structure. Life safety for this design event is taken to imply that the bridge has a low probability of collapse, but may suffer significant damage. "Critical" structures (bridges) must remain open to all traffic after the design earthquake and be usable by emergency vehicles and for security/defense purposes immediately after a large earthquake, e.g., a 2500-yr return period event.

5.4.1 Essential Structure Parameters (AASHTO 7% PE in 75 years – 1,000 return period)

If it is determined by the project stakeholders that this bridge design should be considered an "essential" structure, the following ground motion parameters would be used. Considering the 1.0-second spectral acceleration of 0.048g on bedrock for the AASHTO 7% PE in 75 years, and a seismic Site Class D for the overall bridge alignment based on shear wave velocity measurements, under Article 3.10.6 of AASHTO LRFD Bridge Design Specifications, the bridge should be assigned to Seismic Zone 1. Liquefaction evaluation is not required for structures located in Seismic Zone 1.

Code Used	**Site Classification**
2010 AASHTO LRFD Bridge Design Specifications (AASHTO) [1]	D [2]

1. In general accordance with the 2010 AASHTO LRFD Bridge Design Specifications, Table 3.10.3.1-1 AASHTO Site Class is based on the characteristics of the upper 100 feet of the subsurface profile.
2. The 2010 AASHTO LRFD Bridge Design Specifications (2010 AASHTO) requires a site soil profile determination extending a depth of 100 feet for seismic site classification. Terracon used borehole geophysical logging (Suspension PS Velocity Measurements) as included in Exhibit A-11. The Site Class is based upon the subsurface conditions encountered on the project site and the average shear wave velocity of 847 feet/second derived from our seismic survey data at Locations L-1 and L-4).

Ground Motion Parameter	**Value (g)1**
PGA	0.048
S_s	0.111
S_1	0.047
A_s	0.077
S_{DS}	0.177
S_{D1}	0.113

1. Latitude 39.0888 and Longitude -84.5233 degrees (AASHTO Spectrum 7% PE in 75 years)
2. F_{pga} = 1.60 from Table 3.10.3.2-1
3. F_a = 1.60 from Table 3.10.3.2-2
4. F_v = 2.40 from Table 3.10.3.2-3

5.4.2 Critical Structure Parameters (2% PE in 50 years - 2,475-year return period)

If it is determined by the project stakeholders that this bridge design should be considered an "critical" structure, the following seismic guidelines will apply. Considering the 1.0-second spectral acceleration of 0.076g on bedrock identified in Section 2.3 above, and a seismic Site Class D for the overall bridge alignment based on shear wave velocity measurements, under Article 3.10.6 of AASHTO LRFD Bridge Design Specifications, the bridge should be assigned to Seismic Zone 2. Under Article 10.5.4.1, "where loose to very loose saturated sands are within the subsurface soil profile such that liquefaction of these soils could impact the stability of the structure, the potential for liquefaction in Seismic Zone 2 should be considered." The AASHTO Commentary (p.10-32) indicates that for Seismic Zone 2, this is only required if A_s is 0.15g or greater. Under these specifications, a liquefaction evaluation is not required. The following ground motion parameters would be applied if it determined that this is a "critical" structure.

Code Used	Site Classification
2010 AASHTO LRFD Bridge Design Specifications (AASHTO) [1]	D [2]

1. In general accordance with the 2010 AASHTO LRFD Bridge Design Specifications, Table 3.10.3.1-1 AASHTO Site Class is based on the characteristics of the upper 100 feet of the subsurface profile.
2. The 2010 AASHTO LRFD Bridge Design Specifications (2010 AASHTO) requires a site soil profile determination extending a depth of 100 feet for seismic site classification. Terracon used borehole geophysical logging (Suspension PS Velocity Measurements) as included in Exhibit A-11. The Site Class is based upon the subsurface conditions encountered on the project site and the average shear wave velocity of 847 feet/second derived from our seismic survey data at Locations L-1 and L-2.

Ground Motion Parameter	Value (g)1
PGA	0.080
S_s	0.178
S_1	0.076
A_s	0.128
S_{DS}	0.285
S_{D1}	0.182

1. Latitude 39.0888 and Longitude -84.5233 degrees (NEHRP Spectrum 2% PE in 50 years)
2. F_{pga} = 1.60 from Table 3.10.3.2-1
3. F_a = 1.60 from Table 3.10.3.2-2
4. F_v = 2.40 from Table 3.10.3.2-3

As noted in Section 4.5, the river pier locations have a distinctly different stratigraphic section than the river banks that will result in different behavior under seismic loads than the abutments. The AASHTO site class and response spectrum approach does not consider such differences explicitly. Site response analyses should be considered to evaluate the seismic demand on the bridges structural elements and possibly develop time histories for input at each of the abutment and pier locations.

6.0 GENERAL COMMENTS

HCN/Terracon should be retained to review the final design plans and specifications so comments can be made regarding interpretation and implementation of our geotechnical recommendations in the design and specifications. HCN/Terracon also should be retained to provide observation and testing services during grading, excavation, foundation construction and other earth-related construction phases of the project.

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental or biological (e.g., mold, fungi, bacteria) assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either expressed or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

APPENDIX A
FIELD EXPLORATION

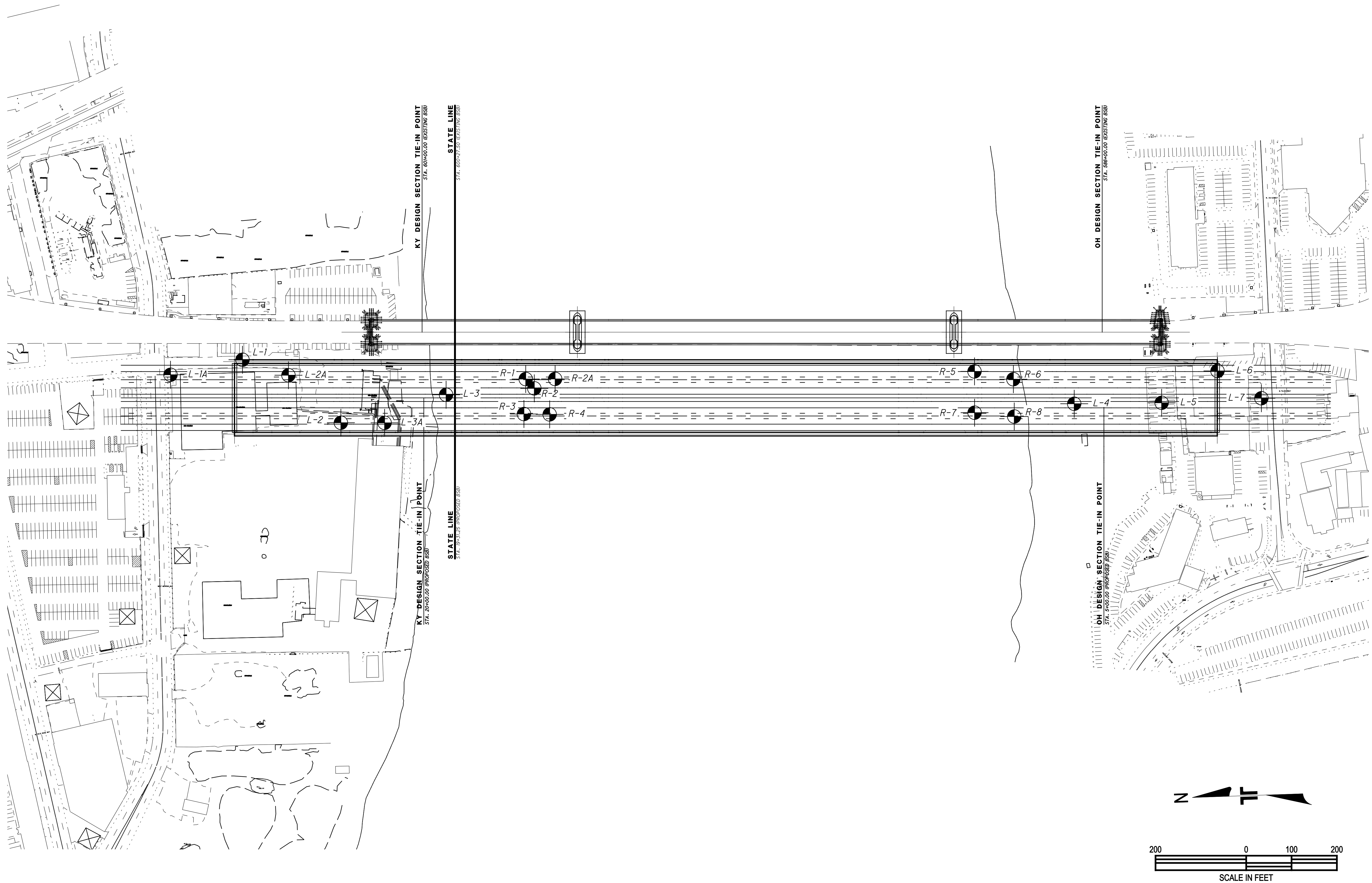
FIGURES

N

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES

Project Manager: AJM	Project No. N1105070	HCN A Terracon COMPANY 611 Lunken Park Drive Cincinnati, Ohio 45226 PH. (513) 321-5816 FAX. (513) 321-0294	SITE VICINITY MAP PROPOSED BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO – COVINGTON , KENTUCKY
Drawn by: DWW	Scale: As Shown		
Checked by: AJM	File Name: A1		
Approved by: AJM	Date: 11/30/2010		

Exhibit A-1



REV.	DATE	BY	DESCRIPTION

TEST BORING LOCATION PLAN

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

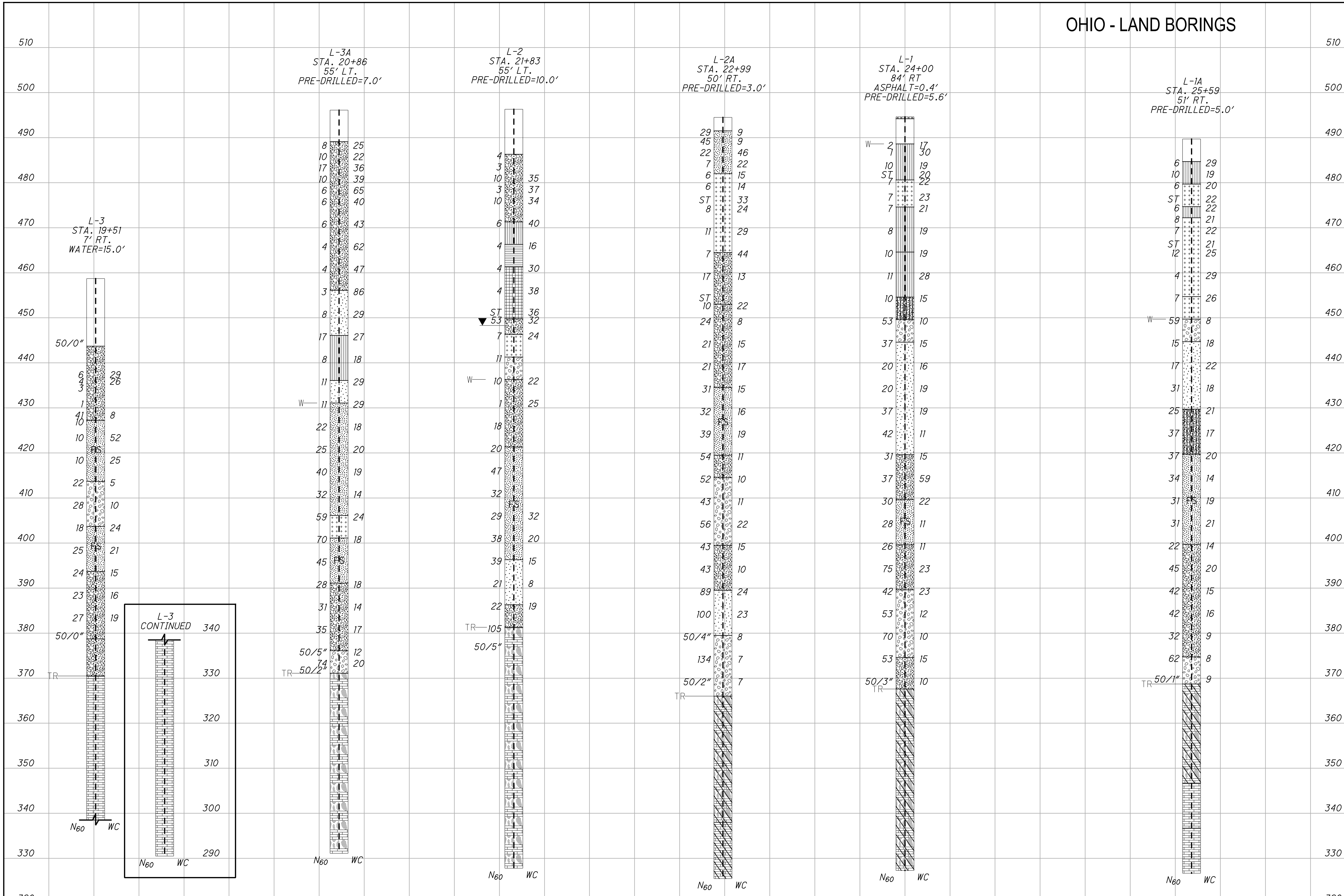
PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

A Terracon COMPANY
611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
PH. (513) 321-5816
FAX. (513) 321-4540

EXHIBIT A-2

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=200'
DATE:	12/01/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-2.DGN
SHEET NO.:	A-2



REV.	DATE	BY	DESCRIPTION

SUMMARY OF GEOTECHNICAL DATA

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

A Terracon COMPANY

611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
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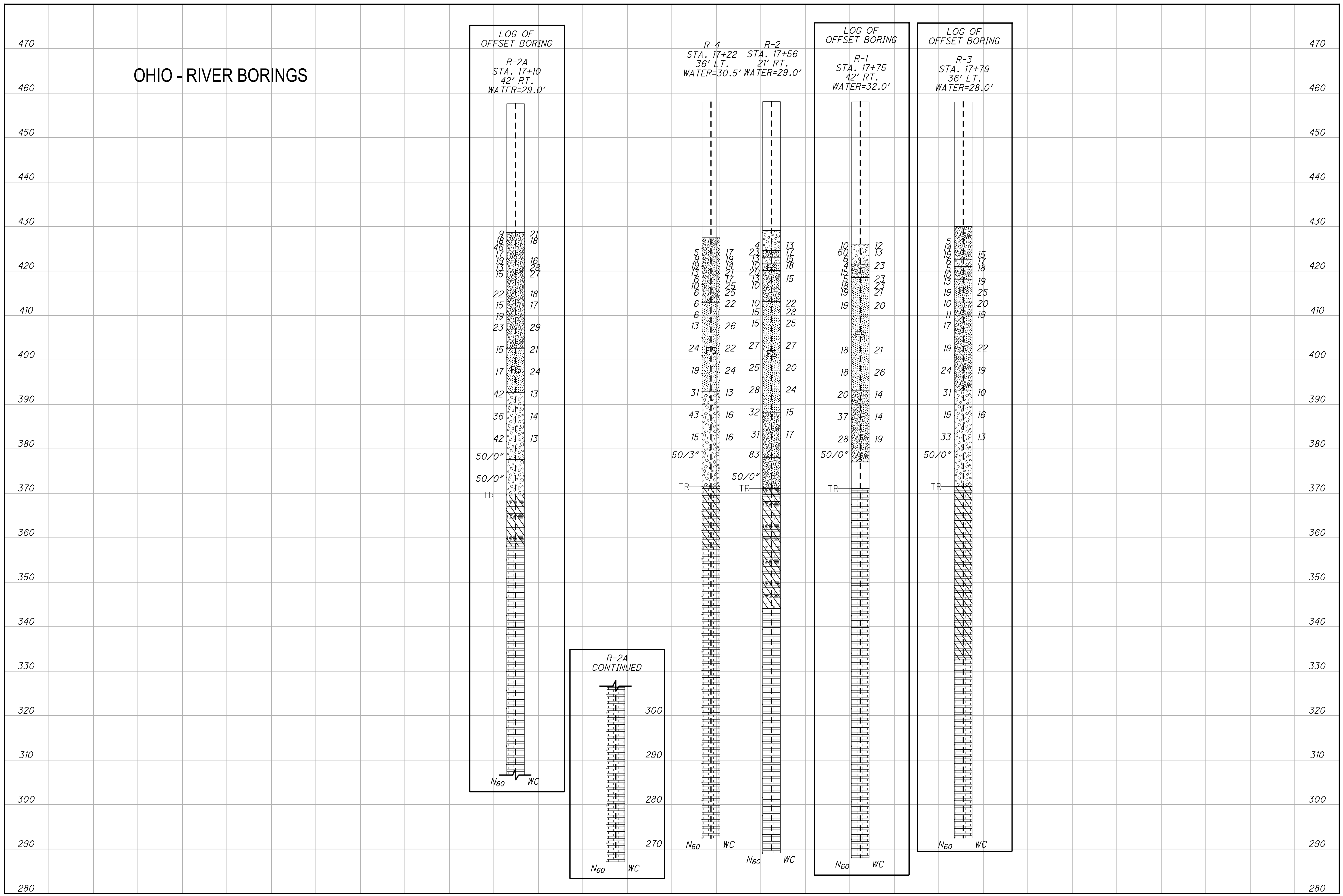
EXHIBIT A-3

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=50' H
DATE:	12/01/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-3.DGN
SHEET NO.:	A-3



SUMMARY OF GEOTECHNICAL DATA

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

A Terracon COMPANY
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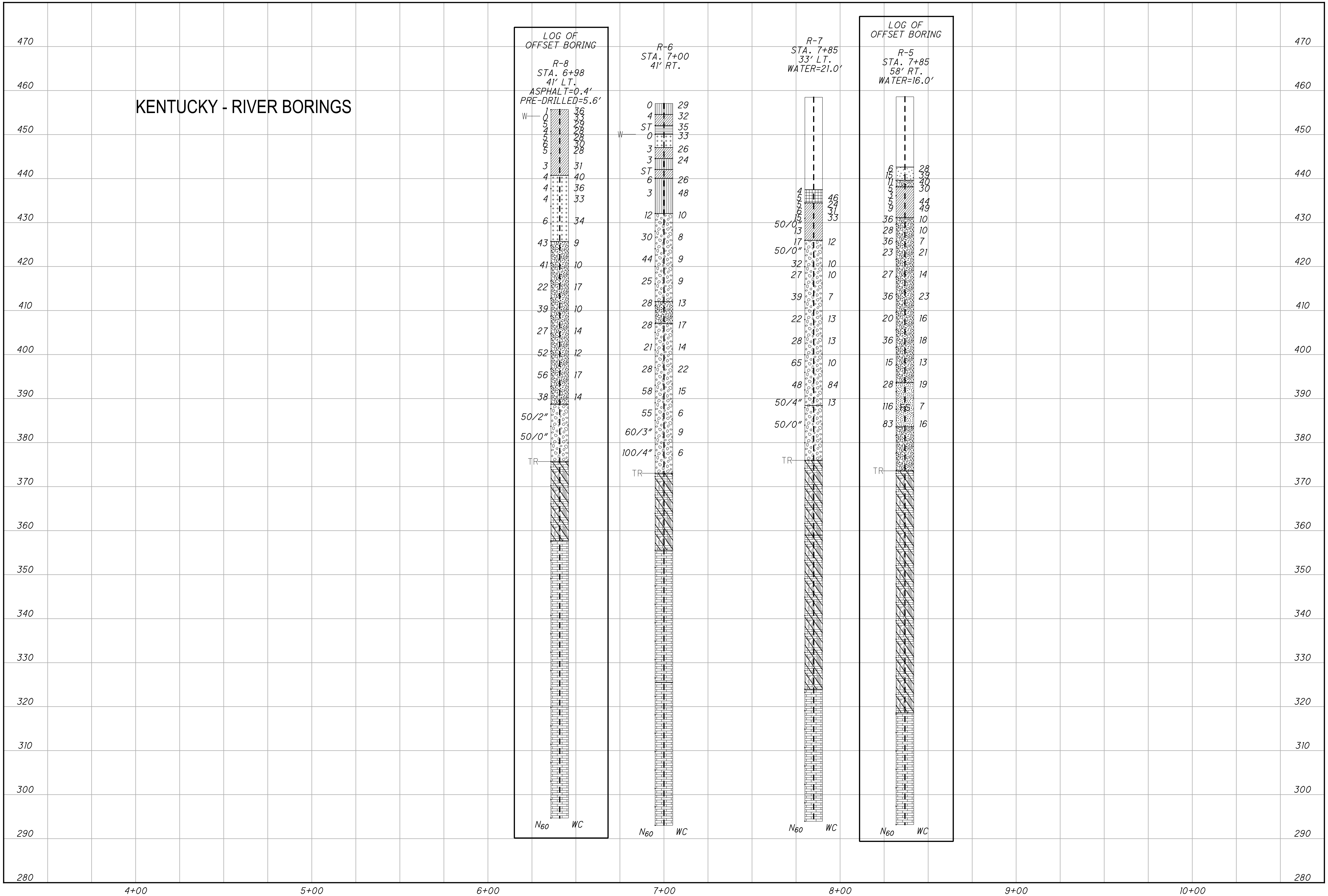
REV.	DATE	BY	DESCRIPTION

EXHIBIT A-4

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=50' H
DATE:	12/01/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-4.DGN
SHEET NO.:	A-4



REV.	DATE	BY	DESCRIPTION

SUMMARY OF GEOTECHNICAL DATA

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

A Terracon COMPANY

611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226

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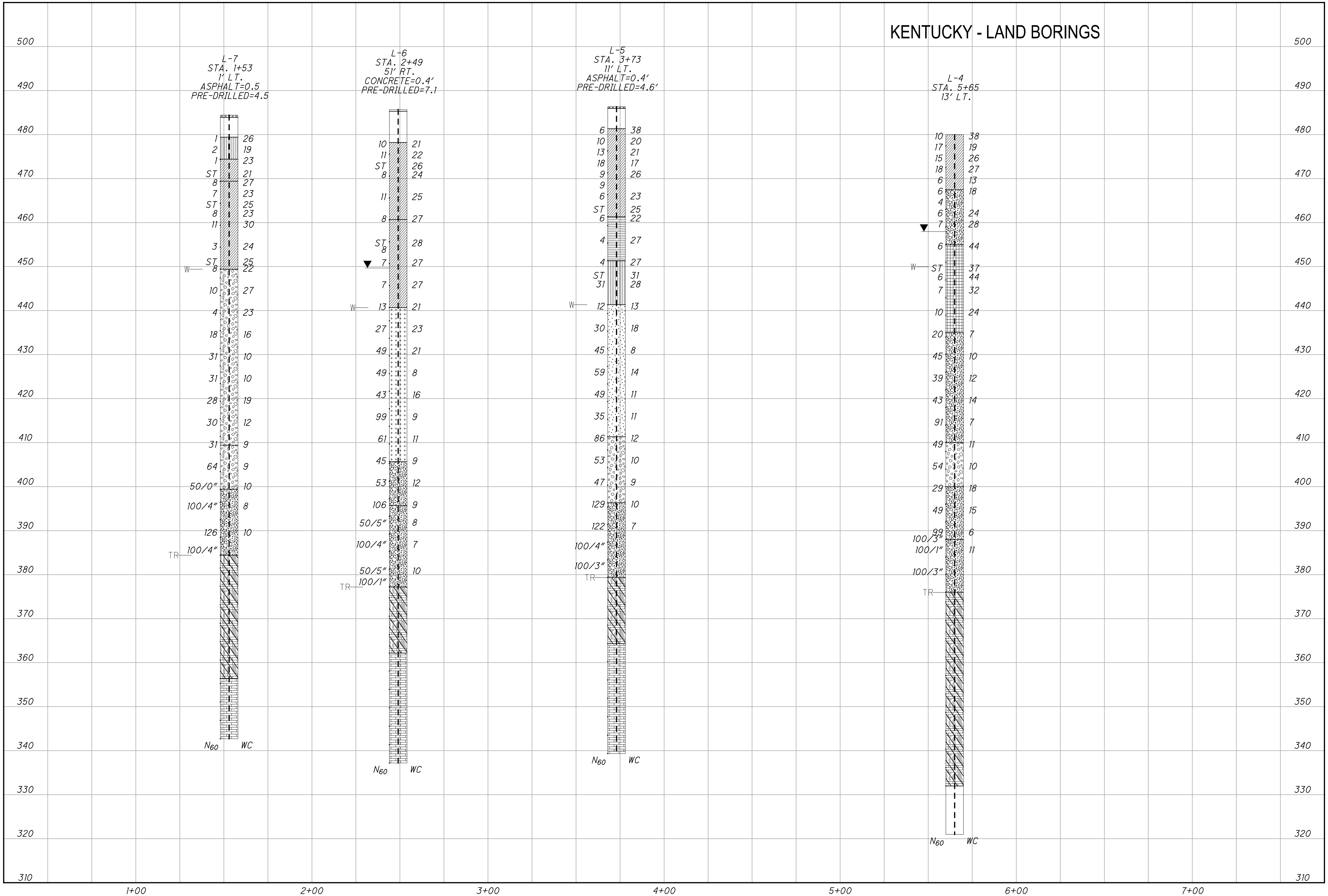
EXHIBIT A-5

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=50' H
DATE:	12/01/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-5.DGN
SHEET NO.:	A-5



REV.	DATE	BY	DESCRIPTION

SUMMARY OF GEOTECHNICAL DATA

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

A Terracon COMPANY
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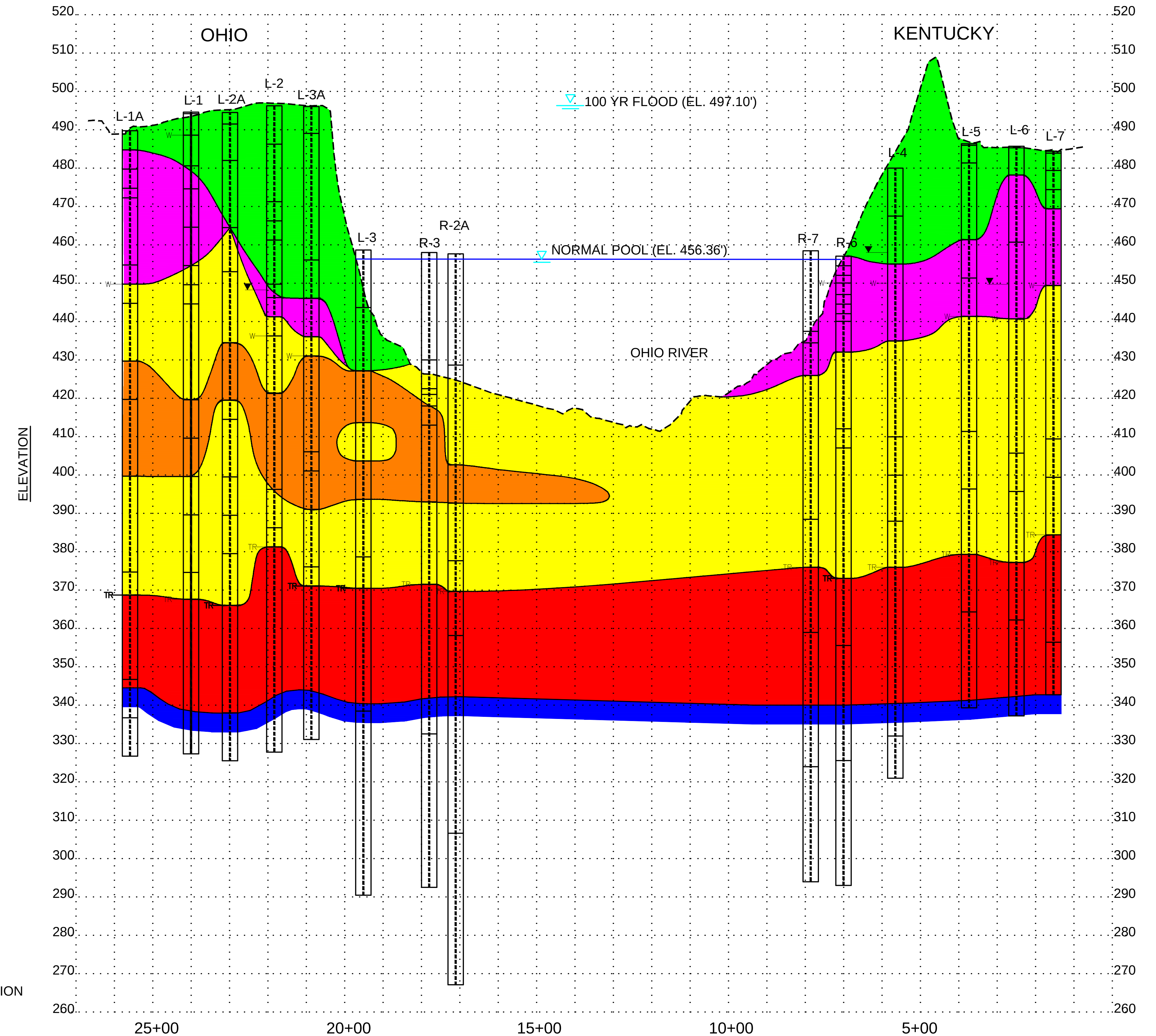
EXHIBIT A-6

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=50' H
DATE:	12/01/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-6.DGN
SHEET NO.:	A-6



GENERALIZED SUBSURFACE PROFILE

LEGEND

- EXISTING FILL
- FINE SAND
- SILT AND CLAY
- GRAVEL AND SAND
- POINT PLEASANT FORMATION
- LEXINGTON LIMESTONE FORMATION

NOTE:
TRANSITION FROM POINT PLEASANT TO LEXINGTON LIMESTONE FORMATION IS GRADUAL AND NOT SUDDEN AS DEPICTED HERE.

NOTE:
DIAGRAM IS FOR GENERAL ILLUSTRATION PURPOSES ONLY AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES. THE STRATIFICATION HAS BEEN INTERPOLATED AND VARIATIONS SHOULD BE EXPECTED.

REV.	DATE	BY	DESCRIPTION

SUBSURFACE PROFILE

PROPOSED BRENT SPENCE BRIDGE REPLACEMENT

PARSONS BRINCKERHOFF

CINCINNATI, OHIO - COVINGTON, KENTUCKY

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611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
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EXHIBIT A-7

DESIGNED BY:	DW
DRAWN BY:	KM
APPVD. BY:	DW
SCALE:	1"=300'H
DATE:	12/03/2010
JOB NO.	N1105070
ACAD NO.	EXHIBIT A-7.DWG
SHEET NO.:	A-7

TEST BORING LOGS &
ROCK CORE PHOTOGRAPHS

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JJ	DRILL RIG: CME 550X ATV- 9333	STATION / OFFSET: 23+99.9, 84.1 RT	EXPLORATION ID L-1
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 494.6 (MSL) EOB: 182.5 ft.	PAGE 1 OF 3
START: 7/16/10 END: 7/20/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 67.1	COORD: 39.093833610, -84.522929480	

MATERIAL DESCRIPTION AND NOTES	ELEV. 494.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
ASPHALT	494.2																	
PRE-DRILLED (VACUUM EXCAVATED)	488.6	W																
VERY STIFF, BROWN, **SANDY SILT**, TRACE TO LITTLE GRAVEL, LITTLE CLAY, TRACE CONCRETE AND BRICK (FILL), WET			2 1 1	2	33	SS-1	-	-	-	-	-	-	-	-	-	17	A-4a (V)	
			1		33	SS-2	-	17	5	27	36	15	27	17	10	30	A-4a (3)	
			3 4 5	10	100	SS-3	2.50	0	0	36	47	17	24	16	8	19	A-4a (6)	
	480.6				100	ST-4	-	-	-	-	-	-	-	-	-	20	A-4a (V)	
MEDIUM STIFF TO STIFF, BROWN, **SILT**, LITTLE CLAY, SOME SAND, TRACE ORGANICS, LOI=1.6% (17.5'), MOIST			2 3 3	7	100	SS-5	1.50	0	0	24	56	20	27	17	10	22	A-4b (8)	
	474.6		2 3 3	7	100	SS-6	0.50	-	-	-	-	-	-	-	-	23	A-4b (V)	
MEDIUM STIFF, BROWN, **SANDY SILT**, LITTLE CLAY, SOME SAND SEAMS, MOIST			2 3 3	7	100	SS-7	0.75	0	0	43	41	16	20	18	2	21	A-4a (4)	
	464.6		2 3 4	8	100	SS-8	1.00	-	-	-	-	-	-	-	-	19	A-4a (V)	
LOOSE TO MEDIUM DENSE, BROWN, **SANDY SILT**, LITTLE CLAY, WET			2 4 5	10	100	SS-9	1.00	0	1	36	45	18	23	17	6	19	A-4a (6)	
	454.6		3 5 5	11	100	SS-10	0.50	-	-	-	-	-	-	-	-	28	A-4a (V)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, WET	449.6		4 4 5	10	100	SS-11	-	38	19	17	19	7	24	16	8	15	A-2-4 (0)	
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET	444.6		25 28 19	53	100	SS-12	-	61	20	11	6	2	NP	NP	NP	10	A-1-a (0)	
MEDIUM DENSE TO DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE GRAVEL, LITTLE SILT, TRACE CLAY, WET			20 18 15	37	33	SS-13	-	-	-	-	-	-	-	-	-	15	A-3a (V)	
			5 6 12	20	67	SS-14	-	11	32	39	14	4	NP	NP	NP	16	A-3a (0)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 23+99.9, 84.1 RT	START: 7/16/10	END: 7/20/10	PG 2 OF 3	L-1

MATERIAL DESCRIPTION AND NOTES	ELEV. 434.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE GRAVEL, LITTLE SILT, TRACE CLAY, WET *(continued)*		61	9 9 9	20	78	SS-15	-	-	-	-	-	-	-	-	-	19	A-3a (V)	
		66	20 18 15	37	67	SS-16	-	6	27	54	9	4	NP	NP	NP	19	A-3a (0)	
		71	16 17 21	42	67	SS-17	-	-	-	-	-	-	-	-	-	11	A-3a (V)	
DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	419.6	75																
		76	15 17 11	31	100	SS-18	-	32	20	38	7	3	NP	NP	NP	15	A-1-b (0)	
		81	24 17 16	37	100	SS-19	-	-	-	-	-	-	-	-	-	59	A-1-b (V)	
MEDIUM DENSE, BROWN, **FINE SAND**, SOME COARSE SAND, TRACE SILT, TRACE CLAY, WET	409.6	85																
		86	14 13 14	30	67	SS-20	-	0	21	71	4	4	NP	NP	NP	22	A-3 (0)	
		91	13 12 13	28	67	SS-21	-	-	-	-	-	-	-	-	-	11	A-3 (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY DENSE BELOW 100', WET	399.6	95																
		96	11 11 12	26	83	SS-22	-	47	26	20	5	2	NP	NP	NP	11	A-1-b (0)	
		101	17 33 34	75	100	SS-23	-	-	-	-	-	-	-	-	-	23	A-1-b (V)	
DENSE TO VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET	389.6	105																
		106	15 18 20	42	67	SS-24	-	-	-	-	-	-	-	-	-	23	A-1-a (V)	
		111	19 23 24	53	56	SS-25	-	58	21	13	7	1	NP	NP	NP	12	A-1-a (0)	
		116	20 23 40	70	33	SS-26	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	374.6	120																
		121	10 16 31	53	44	SS-27	-	11	52	29	7	1	NP	NP	NP	15	A-1-b (0)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 23+99.9, 84.1 RT	START: 7/16/10	END: 7/20/10	PG 3 OF 3	L-1

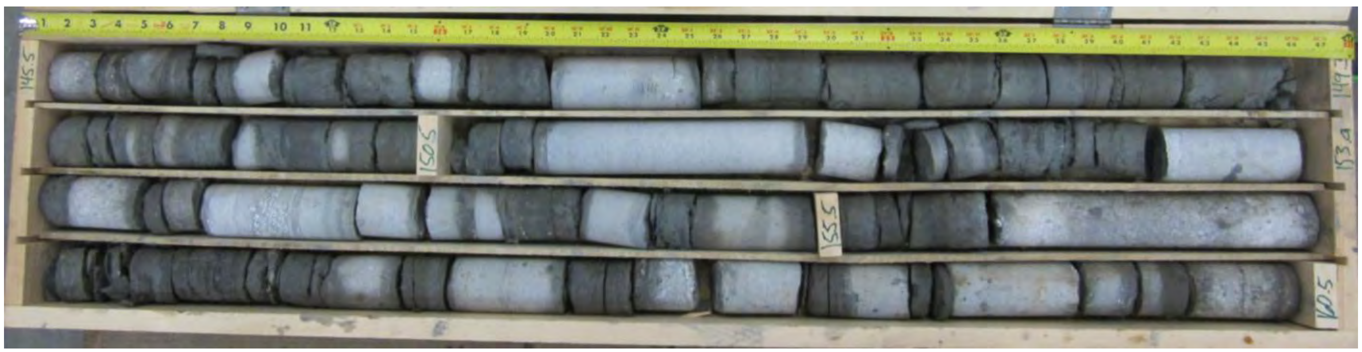
MATERIAL DESCRIPTION AND NOTES	ELEV. 372.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET *(continued)*		123, 124, 125, 126	30 37 50/3"	-	80	SS-28	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
INTERBEDDED LIMESTONE (80%) AND SHALE (20%); **LIMESTONE**, GRAY, SLIGHTLY WEATHERED, STRONG, THIN BEDDED, CRYSTALLINE, FOSSILIFEROUS SEAMS, FRACTURED, LOSS 10%, RQD 39%; **SHALE**, GRAY, SLIGHTLY WEATHERED, WEAK, LAMINATED, FISSILE, FRACTURED LS @129.9'-130.3' QU=10982 PSI SH @ 137.1' SDI = 68.0 LS @ 139.5' POINT LOAD = 14651 PSI LS @142.7'-143.2' QU=9375 PSI LS @ 145.0' POINT LOAD = 15893 PSI SH @ 147.7' SDI = 67.3 LS @150.7'-151.3' QU=21926 PSI LS @153.5'-154' QU=12023 PSI LS @156'-157' QU=10166 PSI SH @ 157.0' SDI = 67.8 LS/SH @162.5'-163' QU=8652 PSI.	367.6	TR 127, 128, 129, 130	29		86	NQ-1											CORE	
		131, 132, 133, 134, 135	0		40	NQ-2											CORE	
		136, 137, 138, 139, 140	20		80	NQ-3											CORE	
		141, 142, 143, 144, 145	52		100	NQ-4											CORE	
		146, 147, 148, 149, 150	20		100	NQ-5											CORE	
		151, 152, 153, 154, 155	54		100	NQ-6											CORE	
		156, 157, 158, 159, 160	48		100	NQ-7											CORE	
		161, 162, 163, 164, 165	64		100	NQ-8											CORE	
	327.3	166, 167	60		98	NQ-9											CORE	
BLANK DRILLED FOR SEISMIC TESTING	312.1	168, 169, 170, 171, 172, 173, 174, 175, 176, 177, 178, 179, 180, 181, 182 EOB																

NOTES: WATER USED BELOW 6 FT. FOR DRILLING/ROCK CORING PURPOSES. 3 INCH PVC CASING PLACED FROM SURFACE TO 182.5 FEET. CASING BROKE DURING ATTEMPTED REMOVAL AND WA

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (13 BAGS CEMENT/2 BAGS BENTONITE)

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 3/9/11 10:06 - N:\PROJECTS\2010\N1105070\GINT\ODOT LOGS.GPJ

BORING NO.: L-1
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 127.0-145.5
ELEVATION (ft.): 366.46
1/NQ: 127.0’ – 130.5’; REC. 86%, RQD 29%
2/NQ: 130.5’ – 135.5’; REC. 40%, RQD 0%
3/NQ: 135.5’ – 140.5’; REC. 80%, RQD 20%
4/NQ: 140.5’ – 145.5’; REC. 100%, RQD 52%



BORING NO.: L-1
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 145.5-160.5
ELEVATION (ft.): 347.96
5/NQ: 145.5’ – 150.5’; REC. 100%, RQD 20%
6/NQ: 150.5’ – 155.5‘; REC. 100%, RQD 54%
7/NQ: 155.5’ – 160.5‘; REC. 100%, RQD 48%

BORING NO.: L-1
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 160.5-167.3
ELEVATION (ft.): 332.96
8/NQ: 160.5’ 165.5‘; REC. 100%, RQD 64%
9/NQ: 165.5’-167.3‘; REC. 100%, RQD 61%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core A Date: 9-3-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-1

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JJ	DRILL RIG: CME 550X ATV- 9333	STATION / OFFSET: 25+58.6, 50.9 RT	EXPLORATION ID L-1A
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 489.7 (MSL) EOB: 163.0 ft.	PAGE 1 OF 3
START: 7/29/10 END: 8/1/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 67.1	COORD: 39.094153260, -84.522842640	

MATERIAL DESCRIPTION AND NOTES	ELEV. 489.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
PRE-DRILLED (VACUUM EXCAVATED)	484.7	1–5																
MEDIUM STIFF TO STIFF, BROWN, **SANDY SILT**, LITTLE CLAY, MOIST		6	2 2 3	6	100	SS-1	0.50	-	-	-	-	-	-	-	-	29	A-4a (V)	
	479.7	8	3 4 5	10	100	SS-2	1.50	0	0	32	49	19	26	16	10	19	A-4a (7)	
LOOSE, BROWN, **SILT**, SOME SAND, LITTLE CLAY, MOIST		11	2 2 3	6	100	SS-3	1.50	0	2	28	52	18	24	16	8	20	A-4b (7)	
	474.7	14			75	ST-4	-	-	-	-	-	-	-	-	-	22	A-4b (V)	
LOOSE, GRAY AND BROWN, **SANDY SILT**, LITTLE CLAY, MOIST	472.2	16	3 2 3	6	100	SS-5	-	0	2	48	39	11	NP	NP	NP	22	A-4a (3)	
STIFF, BROWN, **SILT**, SOME FINE SAND, LITTLE CLAY, MOIST		18	3 3 4	8	100	SS-6	1.75	-	-	-	-	-	-	-	-	21	A-4b (V)	
		21	3 3 3	7	100	SS-7	1.50	0	0	30	51	19	25	16	9	22	A-4b (7)	
		24			100	ST-8	1.25	-	-	-	-	-	-	-	-	21	A-4b (V)	
		26	2 4 7	12	100	SS-9	1.00	-	-	-	-	-	-	-	-	25	A-4b (V)	
	454.7	31	2 2 2	4	100	SS-10	-	0	0	15	63	22	28	18	10	29	A-4b (8)	
LOOSE, GRAY, **SILT**, SOME FINE SAND, LITTLE CLAY, MOIST	449.7	36	3 3 3	7	100	SS-11	-	0	0	30	54	16	27	20	7	26	A-4b (7)	
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET	444.7	W 40–41	17 21 32	59	100	SS-12	-	53	25	9	9	4	NP	NP	NP	8	A-1-a (0)	
MEDIUM DENSE TO DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE SILT, TRACE TO LITTLE GRAVEL, TRACE CLAY, WET		46	6 8 5	15	100	SS-13	-	19	27	37	13	4	NP	NP	NP	18	A-3a (0)	
		51	6 7 8	17	100	SS-14	-	1	22	59	13	5	NP	NP	NP	22	A-3a (0)	
	429.7	56	14 14 14	31	100	SS-15	-	-	-	-	-	-	-	-	-	18	A-3a (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 25+58.6, 50.9 RT	START: 7/29/10	END: 8/1/10	PG 2 OF 3	L-1A

MATERIAL DESCRIPTION AND NOTES	ELEV. 429.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND AND SILT**, TRACE CLAY, WET		61	8-10-12	25	100	SS-16	-	22	13	34	21	10	NP	NP	NP	21	A-2-4 (0)	
		66	12-16-17	37	100	SS-17	-	-	-	-	-	-	-	-	-	17	A-2-4 (V)	
	419.7	70																
DENSE, BROWN, **FINE SAND**, TRACE SILT AND CLAY, WET		71	13-14-19	37	100	SS-18	-	3	32	57	4	4	NP	NP	NP	20	A-3 (0)	
		76	11-15-15	34	100	SS-19	-	-	-	-	-	-	-	-	-	14	A-3 (V)	
		81	8-13-15	31	100	SS-20	-	7	28	56	5	4	NP	NP	NP	19	A-3 (0)	
		86	14-13-15	31	100	SS-21	-	-	-	-	-	-	-	-	-	21	A-3 (V)	
	399.7	90																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		91	16-10-10	22	56	SS-22	-	38	21	34	4	3	NP	NP	NP	14	A-1-b (0)	
		96	10-22-18	45	100	SS-23	-	-	-	-	-	-	-	-	-	20	A-1-b (V)	
		101	15-23-15	42	100	SS-24	-	32	31	28	6	3	NP	NP	NP	15	A-1-b (0)	
		106	16-17-21	42	100	SS-25	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
		111	10-11-18	32	67	SS-26	-	43	35	16	4	2	NP	NP	NP	9	A-1-b (0)	
	374.7	115																
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, LITTLE SAND, TRACE SILT, TRACE CLAY, WET		116	43-30-25	62	100	SS-27	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
		121	20-48-50/1"	-	92	SS-28	-	74	7	12	5	2	NP	NP	NP	9	A-1-a (0)	
	368.7	TR																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 25+58.6, 50.9 RT	START: 7/29/10	END: 8/1/10	PG 3 OF 3	L-1A

MATERIAL DESCRIPTION AND NOTES	ELEV. 367.8	DEPTHS
INTERBEDDED LIMESTONE (80%) AND SHALE (20%); **LIMESTONE**, GRAY, SLIGHTLY WEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, MODERATELY FRACTURED, LOSS 0%, RQD 36%; **SHALE**, GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, LAMINATED, LS @123.1'-123.7' QU=10192 PSI LS @132.3'-132.8' QU=13597 PSI LS @ 140.1' POINT LOAD = 9157 PSI LS @ 152.6' POINT LOAD = 12346 PSI LS @ 154.5' POINT LOAD = 11932 PSI. *(continued)*	346.7	122 – 143
LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, ARGILLACEOUS, SHALE PARTINGS, LOSS 0%, RQD 67% LS @143'-143.5' QU=5891 PSI LS @150.7'-151.1' QU=13391 PSI.	336.7	143 – 153
LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG, THIN BEDDED, FOSSILIFEROUS, ARGILLACEOUS SEAMS, LOSS 0%, RQD 86% LS @160'-160.5' QU=4409 PSI.	326.7	153 – 163 (EOB)

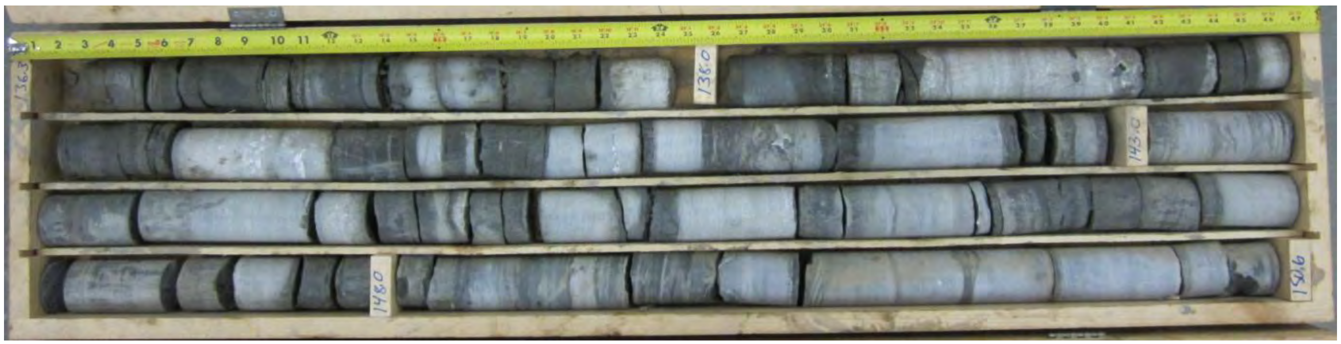
DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
123–128	16		100	NQ-1											CORE	
128–133	44		100	NQ-2											CORE	
133–138	24		100	NQ-3											CORE	
138–143	60		100	NQ-4											CORE	
143–148	66		100	NQ-5											CORE	
148–153	68		100	NQ-6											CORE	
153–158	76		100	NQ-7											CORE	
158–163	96		100	NQ-8											CORE	

EOB — 163

NOTES: WATER USED BELOW 40 FT. FOR DRILLING/ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (15 BAGS CEMENT/2 BAGS BENTONITE)

BORING NO.: L- 1A
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 123.0-138.0
ELEVATION (ft.): 368.45
1/NQ: 123.0' – 128.0'; REC. 100%, RQD 16%
2/NQ: 128.0' – 133.0'; REC. 100%, RQD 44%
3/NQ: 133.0 – 138.0'; REC. 100%, RQD 24%



BORING NO.: L- 1A
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 138.0 – 153.0
ELEVATION (ft.): 353.45
4/NQ: 138.0' – 143.0'; REC. 100%, RQD 60%
5/NQ: 143.0'-148.0'; REC. 100%, RQD 66%
6/NQ: 148.0' – 153.0'; REC. 100%, RQD 68%

BORING NO.: L- 1A
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 153.0 – 163.0
ELEVATION (ft.): 333.45
7/NQ: 153.0' – 158.0'; REC. 100%, RQD 76%
8/NQ: 158.0' – 163.0'; REC. 100%, RQD 96%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core A Date: 9-3-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-1A

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JM	DRILL RIG: DIEDRICH D-50	STATION / OFFSET: 21+82.9, 54.9 LT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 9/9/10	ELEVATION: 496.3 (MSL) EOB: 168.5 ft.
START: 5/28/10 END: 6/1/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 83.7	COORD: 39.093247060, -84.523175560

EXPLORATION ID L-2

MATERIAL DESCRIPTION AND NOTES	ELEV. 496.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
PRE-DRILLED (VACUUM EXCAVATION)		0–10																
VERY LOOSE TO LOOSE, BLACK, **GRAVEL AND STONE FRAGMENTS WITH SAND**, SOME CINDERS, TRACE BRICK FRAGMENTS (FILL), MOIST TO WET	486.3	10–11.5	2 1 2	4	67	SS-1	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		13–14.5	1 1 1	3	67	SS-2	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		15–16.5	3 4 3	10	100	SS-3	-	-	-	-	-	-	-	-	-	35	A-1-b (V)	
		18–19.5	2 1 1	3	67	SS-4	-	-	-	-	-	-	-	-	-	37	A-1-b (V)	
		20–21.5	2 3 4	10	67	SS-5	-	-	-	-	-	-	-	-	-	34	A-1-b (V)	
VERY LOOSE, BROWN, **SANDY SILT**, TRACE GRAVEL (FILL), WET	471.3	25–26.5	WOR 2 2	6	100	SS-6	-	-	-	-	-	-	-	-	-	40	A-4a (V)	
MEDIUM STIFF, BROWN, **SILTY CLAY**, SOME SAND, SOME GRAVEL AND BRICK FRAGMENTS, (FILL), MOIST	466.3	30–31.5	3 2 1	4	100	SS-7	1.00	26	23	10	21	20	35	17	18	16	A-6b (3)	
SOFT, GRAY AND BROWN, **CLAY**, AND SILT, TRACE FINE SAND, (FILL), MOIST	461.3	35–36.5	2 1	-	67	SS-8	-	-	-	-	-	-	-	-	-	30	A-7-6 (V)	
		40–41.5	2 1 2	4	100	SS-9	-	0	0	8	60	32	48	29	19	38	A-7-6 (13)	
		45–47			78	ST-10	-	-	-	-	-	-	-	-	-	36	A-7-6 (V)	
VERY DENSE, BLACK, **GRAVEL AND STONE FRAGMENTS WITH SAND**, SOME CINDERS, LITTLE BRICK FRAGMENTS (FILL), WET	449.8	46.5–48	16 24 14	53	67	SS-11	-	-	-	-	-	-	-	-	-	32	A-1-b (V)	
LOOSE, GRAY AND BROWN, **SILT**, TRACE FINE SAND, TRACE ORGANICS (FILL), WET	446.3	50–51.5	3 2 3	7	67	SS-12	-	-	-	-	-	-	-	-	-	24	A-4b (V)	
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, (FILL), WET	441.3	55–56.5	4 4 4	11	11	SS-13	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
	436.3	60																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 21+82.9, 54.9 LT	START: 5/28/10	END: 6/1/10	PG 2 OF 3	L-2

MATERIAL DESCRIPTION AND NOTES	ELEV. 436.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		61	2 3 4	10	67	SS-14	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	
		66	WOH 1 WOH	1	67	SS-15	-	8	45	38	6	3	NP	NP	NP	25	A-1-b (0)	
		71	14 8 5	18	11	SS-16	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	421.3	75																
MEDIUM DENSE TO DENSE, BROWN, **FINE SAND**, SOME COARSE SAND, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET		76	6 8 6	20	28	SS-17	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
		81	8 16 18	47	0	SS-18	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
		86	6 10 13	32	11	SS-19	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
		91	8 10 11	29	28	SS-20	-	-	-	-	-	-	-	-	-	32	A-3 (V)	
		96	12 13 14	38	67	SS-21	-	3	34	54	6	3	NP	NP	NP	20	A-3 (0)	
	396.3	100																
MEDIUM DENSE TO DENSE, BROWN, **COARSE AND FINE SAND**, TRACE SILT, TRACE GRAVEL, TRACE CLAY, WET		101	13 14 14	39	56	SS-22	-	-	-	-	-	-	-	-	-	15	A-3a (V)	
		106	8 8 7	21	11	SS-23	-	-	-	-	-	-	-	-	-	8	A-3a (V)	
	386.3	110																
MEDIUM DENSE, BROWN AND GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, WET		111	13 8 8	22	67	SS-24	-	-	-	-	-	-	-	-	-	19	A-1-b (V)	
	381.3	TR 115																
		116	23 33 42	105	67	SS-25	-	-	-	-	-	-	-	-	-	-	Rock (V)	
		120	50/5"	-	100	SS-26	-	-	-	-	-	-	-	-	-	-	Rock (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 21+82.9, 54.9 LT	START: 5/28/10	END: 6/1/10	PG 3 OF 3	L-2

MATERIAL DESCRIPTION AND NOTES — ELEV. 374.4

INTERBEDDED LIMESTONE (75%) AND SHALE (25%);
LIMESTONE, LIGHT GRAY, UNWEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, ARGILLACEOUS, MODERATELY FRACTURED, LOSS=7%, RQD=18%;
SHALE, GRAY, UNWEATHERED TO HIGHLY WEATHERED, MODERATELY STRONG, LAMINATED, MODERATELY FRACTURED

LS @126.7'-127' QU=12810 PSI

LS @130'-130.7' QU=11050 PSI

SH @ 133' SDI = 89.9

LS @137'-137.5' QU=12131 PSI

SH @ 143.5' SDI = 91.4

LS @144'-144.5' QU=15486 PSI

SH @ 148.2' SDI = 88.2

SH @148.2'-148.5' QU=4162 PSI

LS @153'-153.5' QU=9710 PSI

LS @154.5'-155' QU=10865 PSI

LS @158.5'-158.9' QU=8892 PSI

LS @163.6'-164' QU=6246 PSI

LS @165.1'-165.4' QU=10715 PSI. *(continued)*

327.8 — EOB (168)

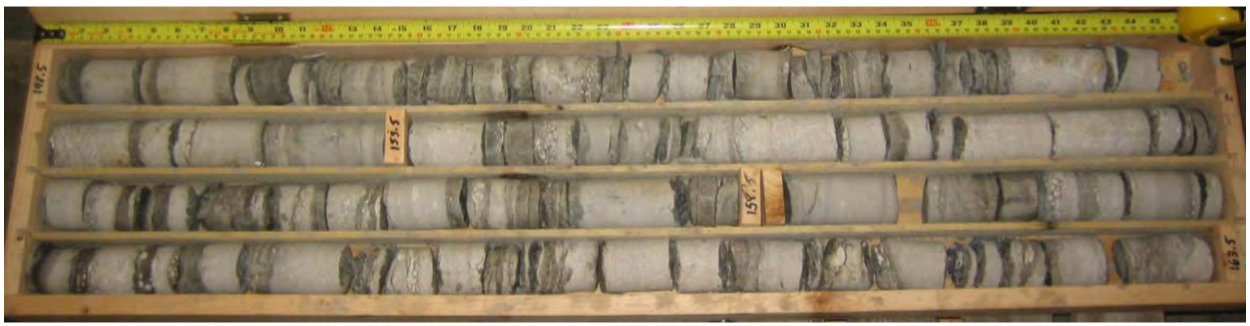
DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
123–124																
124–128	16		80	NQ-1											CORE	
128–133	30		93	NQ-2											CORE	
133–138	16		100	NQ-3											CORE	
138–143	0		93	NQ-4											CORE	
143–148	18		100	NQ-5											CORE	
148–153	30		97	NQ-6											CORE	
153–158	29		98	NQ-7											CORE	
158–163	14		100	NQ-8											CORE	
163–168	7		100	NQ-9											CORE	

NOTES: WATER USED BELOW THE SURFACE FOR DRILLING/ROCK CORING PURPOSES. WATER NOTED AT 52 FT. AFTER 24 HRS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (13 BAGS CEMENT/2 BAGS BENTONITE)

BORING NO.: L- 2
CORE BOX NO.: 1 OF 4
DEPTH (ft.): 124.0-143.5
ELEVATION (ft.): 372.26
1/NQ: 124.0’ – 128.5‘; REC. 80%, RQD 16%
2/NQ: 128.5’ – 133.5‘; REC. 94%, RQD 30%
3/NQ: 133.5’ – 138.5‘; REC. 100%, RQD 16%
4/NQ: 138.5’ – 143.5‘; REC. 94%, RQD 0%

BORING NO.: L- 2
CORE BOX NO.: 2 OF 4
DEPTH (ft.): 143.5-148.5
ELEVATION (ft.): 352.76
5/NQ: 143.5’ – 148.5‘; REC. 100%, RQD 18%



BORING NO.: L- 2
CORE BOX NO.: 3 OF 4
DEPTH (ft.): 148.5-163.5
ELEVATION (ft.): 347.76
6/NQ: 148.5’ – 153.3‘; REC. 96%, RQD 30%
7/NQ: 153.5’ – 158.5‘; REC. 98%, RQD 29%
8/NQ: 158.5’ – 163.5‘; REC. 100%, RQD 14%

BORING NO.: L- 2
CORE BOX NO.: 4 OF 4
DEPTH (ft.): 163.5-168.5
ELEVATION (ft.): 332.76
9/NQ: 163.5’ – 168.5‘; REC. 100%, RQD 7%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	L-2
Chkd By: DWW	File No. Core A	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226		
Approved By: AJM	Date: 9-3-10			

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JM	DRILL RIG: DIEDRICH D-50	STATION / OFFSET: 22+98.5, 50.0 RT	EXPLORATION ID L-2A
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 9/9/10	ELEVATION: 494.5 (MSL) EOB: 169.0 ft.	PAGE
START: 7/12/10 END: 7/15/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 83.7	COORD: 39.093551680, -84.522788620	1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 494.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
PREDRILLED (VACUUM EXCAVATED)		1 2																
LOOSE TO MEDIUM DENSE, GRAY AND BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT AND ORGANICS, LITTLE BRICK FRAGMENTS, TRACE CLAY, (FILL), MOIST	491.5	3 4	7 11 10	29	56	SS-1	-	1	35	54	6	4	NP	NP	NP	9	A-3 (0)	
		5 6	13 16 16	45	56	SS-2	-	-	-	-	-	-	-	-	-	9	A-3 (V)	
		7 8 9	11 9 7	22	100	SS-3	-	-	-	-	-	-	-	-	-	46	A-3 (V)	
		10 11	3 3 2	7	100	SS-4	-	-	-	-	-	-	-	-	-	22	A-3 (V)	
LOOSE TO MEDIUM DENSE, GRAY, **SILT**, AND SAND, LITTLE CLAY, TRACE BRICK FRAGMENTS, TRACE ORGANICS, (FILL), LOI=4.9% (18')	482.0	12 13 14	4 2 2	6	100	SS-5	-	-	-	-	-	-	-	-	-	15	A-4b (V)	
		15 16	3 2 2	6	100	SS-6	-	-	-	-	-	-	-	-	-	14	A-4b (V)	
		17 18 19			79	ST-7	-	0	0	38	50	12	NP	NP	NP	33	A-4b (5)	
		20 21	4 3 3	8	100	SS-8	-	-	-	-	-	-	-	-	-	24	A-4b (V)	
		22 23 24 25 26	2 3 5	11	100	SS-9	-	-	-	-	-	-	-	-	-	29	A-4b (V)	
		27 28 29																
LOOSE TO MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, MOIST	464.5	30 31	2 2 3	7	100	SS-10	-	-	-	-	-	-	-	-	-	44	A-1-b (V)	
		32 33 34 35 36	3 6 6	17	67	SS-11	-	48	24	16	9	3	NP	NP	NP	13	A-1-b (0)	
		37 38 39 40 41			0	ST-12	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	453.0	42 43	2 3 4	10	33	SS-13	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	
		44 45 46	5 7 10	24	56	SS-14	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		47 48 49 50 51	5 7 8	21	89	SS-15	-	31	30	26	9	4	NP	NP	NP	15	A-1-b (0)	
		52 53 54 55 56	6 6 9	21	100	SS-16	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
	434.5	57 58 59																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 22+98.5, 50.0 RT	START: 7/12/10	END: 7/15/10	PG 2 OF 3	L-2A

MATERIAL DESCRIPTION AND NOTES	ELEV. 434.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET		61	7 12 10	31	100	SS-17	-	-	-	-	-	-	-	-	-	15	A-3 (V)	
		66	7 13 10	32	100	SS-18	-	-	-	-	-	-	-	-	-	16	A-3 (V)	
		71	10 13 15	39	100	SS-19	-	1	35	54	6	4	NP	NP	NP	19	A-3 (0)	
	419.5	75																
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY		76	13 19 20	54	100	SS-20	-	37	33	21	6	3	NP	NP	NP	11	A-1-b (0)	
	414.5	80																
DENSE TO VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, WET		81	11 16 21	52	67	SS-21	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		86	9 14 17	43	67	SS-22	-	49	34	10	4	3	NP	NP	NP	11	A-1-a (0)	
		91	12 14 26	56	100	SS-23	-	-	-	-	-	-	-	-	-	22	A-1-a (V)	
	399.5	95																
DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY		96	17 9 22	43	67	SS-24	-	21	40	29	8	2	NP	NP	NP	15	A-1-b (0)	
		101	11 11 20	43	67	SS-25	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	389.5	105																
VERY DENSE, BROWN, **COARSE AND FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET		106	18 29 35	89	100	SS-26	-	1	3	80	12	4	NP	NP	NP	24	A-3a (0)	
		111	20 30 42	100	100	SS-27	-	-	-	-	-	-	-	-	-	23	A-3a (V)	
	379.5	115																
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, WET		116	52 54 50/4"	-	94	SS-28	-	54	22	14	7	3	NP	NP	NP	8	A-1-a (0)	
		121	19 56 40	134	100	SS-29	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	

MATERIAL DESCRIPTION AND NOTES	ELEV. 372.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, WET *(continued)*		123–128	20 20 50/2"	-	100	SS-30	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
	366.0	TR																
INTERBEDDED LIMESTONE (70%) AND SHALE (30%); **LIMESTONE**, LIGHT GRAY, MODERATELY TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, MODERATELY WEATHERED, WEAK, VERY THIN TO THIN BEDDED, LOSS 1%, RQD 34% LS @130.1'-130.5' QU=8084 PSI LS @131.5'-132.2' QU=8782 PSI SH @137'-137.4' QU=1861 PSI SH @ 138.1' SDI = 75.3 LS @ 142.5' POINT LOAD = 12131 PSI SH @ 147.3' SDI = 40.1 LS @150.9'-151.4' QU=16544 PSI.		129–131	10		93	NQ-1											CORE	
		132–136	30		100	NQ-2											CORE	
		137–141	48		100	NQ-3											CORE	
		142–146	42		100	NQ-4											CORE	
		147–151	40		100	NQ-5											CORE	
		152–156	24		98	NQ-6											CORE	
	338.0																	
LIMESTONE, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, TRACE SHALE PARTINGS TO SEAMS, LOSS 1%, RQD 73% LS @157.8'-158.3' QU=8566 PSI LS @ 165.2' POINT LOAD = 13547 PSI.		157–161	52		100	NQ-7											CORE	
		162–166	80		98	NQ-8											CORE	
		167–169	100		100	NQ-9											CORE	
	325.5	EOB 169																

NOTES: WATER USED BELOW 40 FT. FOR DRILLING/ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (14 BAGS CEMENT/2.5 BAGS BENTONITE)

BORING NO.: L- 2A
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 128.5-146.5
ELEVATION (ft.): 366
1/NQ: 128.5' – 131.5'; REC. 93%, RQD 10%
2/NQ: 131.5' – 136.5'; REC. 100%, RQD 30%
3/NQ: 136.5' – 141.5'; REC. 100%, RQD 48%
4/NQ: 141.5' – 146.5'; REC. 100%, RQD 42%

BORING NO.: L- 2A
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 146.5-161.5
ELEVATION (ft.): 348
5/NQ: 146.5' – 151.5'; REC. 100%, RQD 42%
6/NQ: 151.5' – 156.5'; REC. 98%, RQD 24%
7/NQ: 156.5' – 161.5'; REC. 100%, RQD 52%

BORING NO.: L- 2A
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 161.5-169.0
ELEVATION (ft.): 328
8/NQ: 161.5' – 166.5'; REC. 98%, RQD 80%
9/NQ: 166.5' – 169.0'; REC. 100%, RQD 100%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core A Date: 9-3-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-2A

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 19+50.8, 6.9 RT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 458.7 (MSL) EOB: 168.2 ft.
START: 7/14/10 END: 7/16/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 76.3	COORD: 39.092982590, -84.523191040

EXPLORATION ID: L-3

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.7	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WATER (OHIO RIVER)																	
VERY LOOSE TO LOOSE, DARK BROWN, **GRAVEL AND/OR STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, CONCRETE FRAGMENTS FROM 15'-21', (FILL), WET	443.7	50/0"	-		SS-1	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		1 2 3	6	22	SS-2	-	-	-	-	-	-	-	-	-	29	A-1-b (V)	
		1 2 1	4	67	SS-3	-	29	36	24	6	5	NP	NP	NP	26	A-1-b (0)	
		2 1 1	3	0	SS-4	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		1 0 1	1	0	SS-5	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		8 16 16	41	28	SS-6	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
LOOSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET	427.2	4 4 4	10	100	SS-7	-	6	26	63	3	2	NP	NP	NP	-	A-3 (0)	
		4 4 4	10	22	SS-8	-	-	-	-	-	-	-	-	-	52	A-3 (V)	
		3 4 4	10	56	SS-9	-	1	39	56	2	2	NP	NP	NP	25	A-3 (0)	
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET	413.7	10 8 9	22	11	SS-10	-	-	-	-	-	-	-	-	-	5	A-1-a (V)	
		10 11 11	28	67	SS-11	-	61	14	16	7	2	NP	NP	NP	10	A-1-a (0)	
MEDIUM DENSE, BROWN, **FINE SAND**, TRACE TO LITTLE GRAVEL, TRACE SILT, TRACE CLAY, WET	403.7	8 6 8	18	67	SS-12	-	4	32	54	6	4	NP	NP	NP	24	A-3 (0)	

PID: 75119 | BR ID: | PROJECT: BRENT SPENCE BRIDGE | STATION / OFFSET: 19+50.8, 6.9 RT | START: 7/14/10 | END: 7/16/10 | | L-3

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE, BROWN, **FINE SAND**, TRACE TO LITTLE GRAVEL, TRACE SILT, TRACE CLAY, WET *(continued)*		61	5 9 11	25	67	SS-13	-	11	28	55	4	2	NP	NP	NP	21	A-3 (0)	
	393.7	65																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		66	16 9 10	24	67	SS-14	-	27	36	29	5	3	NP	NP	NP	15	A-1-b (0)	
		71	11 9 9	23	67	SS-15	-	21	58	15	3	3	NP	NP	NP	16	A-1-b (0)	
		76	8 9 12	27	67	SS-16	-	9	52	33	4	2	NP	NP	NP	19	A-1-b (0)	
	378.7	80																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, AUGERED FROM 80' TO 88.2', AUGER REFUSAL AT 88.2', WET		80	50/0"	-		SS-17	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	370.5	88 TR																
LIMESTONE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, THIN SHALE SEAMS, LOSS 3%, RQD 51%		89	0		100	NQ-1											CORE	
LS @ 93.1' POINT LOAD = 9195 PSI LS/SH @ 97.6'-98' QU=3277 PSI LS @100.2'-100.4' QU=12940 PSI LS @103.8'-104.4' QU=13314 PSI LS @113.2'-114.2' QU=21169 PSI		93	62		92	NQ-2											CORE	
LS @ 117.6' POINT LOAD = 11786 PSI.		98	60		96	NQ-3											CORE	
		103	78		98	NQ-4											CORE	
		108	56		100	NQ-5											CORE	
		113	34		96	NQ-6											CORE	
		118	36		100	NQ-7											CORE	
	338.5	120																
		121																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 19+50.8, 6.9 RT	START: 7/14/10	END: 7/16/10	PG 3 OF 3	L-3

MATERIAL DESCRIPTION AND NOTES	ELEV. 336.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, THIN SHALE PARTINGS, FOSSILIFEROUS SEAMS, LOSS 2%, RQD 74% LS/SH @121.2'-121.8' QU=6704 PSI LS @121.8'-122.8' QU=13540 PSI LS/SH @124.6'-125.2' QU= 3954 PSI LS @ 137.2' POINT LOAD = 10439 PSI LS @145.2'-146.2' QU=11537 PSI LS/SH @145.6'-146.1' QU=9434 PSI LS @ 147.8' POINT LOAD = 10434 PSI LS @158.7'-160.2' QU=17189 PSI LS @162.8'-163.3' QU=12114 PSI LS @164.5'-165.2' QU=15115 PSI. *(continued)*		123–125	84		100	NQ-8											CORE	
		126–130	94		100	NQ-9											CORE	
		131–135	86		100	NQ-10											CORE	
		136–140	70		100	NQ-11											CORE	
		141–145	62		98	NQ-12											CORE	
		146–150	64		100	NQ-13											CORE	
		151–155	34		92	NQ-14											CORE	
		156–160	62		90	NQ-15											CORE	
		161–165	100		100	NQ-16											CORE	
	290.5	166–168 EOB	93		100	NQ-17											CORE	

NOTES: WATER USED BELOW 88 FT. FOR ROCK CORING PURPOSES.

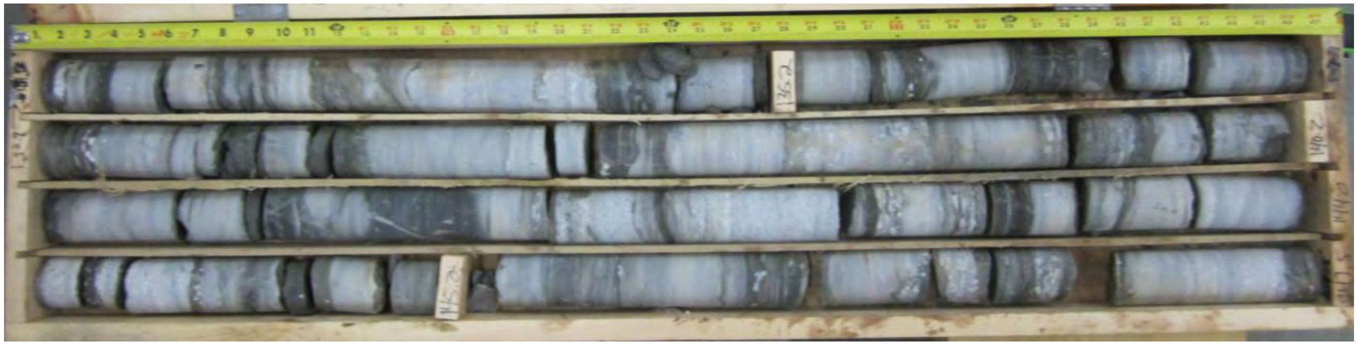
ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (12 BAGS CEMENT/1.5 BAGS BENTONITE)

BORING NO.: L- 3
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 88.2-105.2
ELEVATION (ft.): 370.46
1/NQ: 88.2' – 90.2'; REC. 100%, RQD 0%
2/NQ: 90.2' – 95.2'; REC. 92%, RQD 62%
3/NQ: 95.2' – 100.7'; REC. 87%, RQD 55%
4/NQ: 100.7' – 105.2'; REC. 100%, RQD 87%

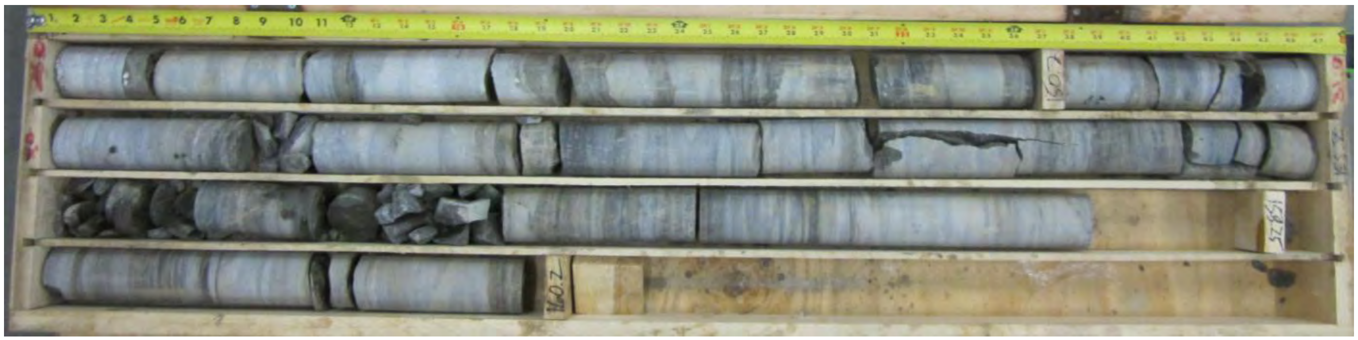
BORING NO.: L- 3
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 105.2-120.2
ELEVATION (ft.): 353.46
5/NQ: 105.2' – 110.2'; REC. 100%, RQD 56%
6/NQ: 110.2' – 115.2'; REC. 96%, RQD 34%
7/NQ: 115.2' – 120.2'; REC. 100%, RQD 36%

BORING NO.: L- 3
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 120.2-135.2
ELEVATION (ft.): 338.46
8/NQ: 120.2' – 125.2'; REC. 100%, RQD 84%
9/NQ: 125.2' – 130.2'; REC. 100%, RQD 94%
10/NQ: 130.2' – 135.2'; REC. 100%, RQD 86%

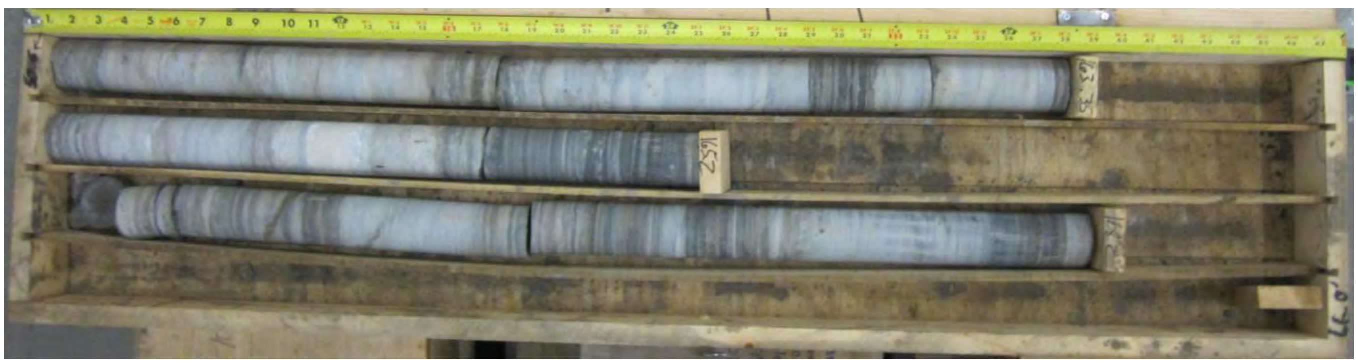
Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	L-3
Chkd By: DWW	File No. Core A	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-3-10		CINCINNATI, OHIO	



BORING NO.: L- 3
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 135.2-150.2
ELEVATION (ft.): 323.46
11/NQ: 135.2' – 140.2'; REC. 100%, RQD 70%
12/NQ: 140.2' – 145.2'; REC. 98%, RQD 62%
13/NQ: 145.2' – 150.2'; REC. 100%, RQD 64%



BORING NO.: L- 3
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 150.2-160.2
ELEVATION (ft.): 308.46
14/NQ: 150.2' – 155.2'; REC. 92%, RQD 34%
15/NQ: 155.2' – 160.2'; REC. 90%, RQD 62%
16/NQ: 160.2' – 165.2'; REC. 100%, RQD 100%



BORING NO.: L- 3
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 160.2-168.2
ELEVATION (ft.): 298.46
17/NQ: 165.2' – 165.2''; REC. 100%, RQD 93%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	L-3
Chkd By: DWW	File No. Core A	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226		
Approved By: AJM	Date: 9-3-10			

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JM	DRILL RIG: DIEDRICH D-50	STATION / OFFSET: 20+86.5, 55.1 LT	EXPLORATION ID L-3A
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 9/9/10	ELEVATION: 496.1 (MSL) EOB: 165.0 ft.	PAGE
START: 5/17/10 END: 5/20/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 83.7	COORD: 39.092603170, -84.522993590	1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 496.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
PRE-DRILLED (VACUUM EXCAVATION)		0 - 7																
LOOSE TO MEDIUM DENSE, BLACK, **GRAVEL AND/OR STONE FRAGMENTS WITH SAND**, LITTLE CINDERS, TRACE BRICK FRAGMENTS, TRACE SILT, TRACE CLAY, (FILL), VERY LOOSE FROM 30' TO 40', MOIST TO WET	489.1	7																
		8 - 9	3 4 2	8	67	SS-1	-	-	-	-	-	-	-	-	-	25	A-1-b (V)	
		10 - 11	1 3 4	10	100	SS-2	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	
		13 - 14	2 5 7	17	67	SS-3	-	-	-	-	-	-	-	-	-	36	A-1-b (V)	
		15 - 16	2 3 4	10	100	SS-4	-	-	-	-	-	-	-	-	-	39	A-1-b (V)	
		18 - 19	2 2 2	6	100	SS-5	-	49	24	16	7	4	NP	NP	NP	65	A-1-b (0)	
		20 - 21	1 2 2	6	100	SS-6	-	-	-	-	-	-	-	-	-	40	A-1-b (V)	
		25 - 26	1 1 3	6	44	SS-7	-	-	-	-	-	-	-	-	-	43	A-1-b (V)	
		30 - 31	2 1 2	4	100	SS-8	-	25	32	25	9	9	NP	NP	NP	62	A-1-b (0)	
		35 - 36	1 1 2	4	100	SS-9	-	-	-	-	-	-	-	-	-	47	A-1-b (V)	
VERY LOOSE TO LOOSE, GRAY, **COARSE AND FINE SAND**, TRACE SILT AND WOOD FRAGMENTS, (FILL), WET	456.1	40 - 41	1 1 1	3	44	SS-10	2.00	-	-	-	-	-	-	-	-	86	A-3a (V)	
		45 - 46	3 3 3	8	100	SS-11	-	-	-	-	-	-	-	-	-	29	A-3a (V)	
STIFF, LIGHT BROWN, **SANDY SILT**, SOME CLAY, TRACE TO SOME GRAVEL, MOIST	446.1	50 - 51	9 7 5	17	44	SS-12	1.50	17	17	22	24	20	26	17	9	27	A-4a (2)	
		55 - 56	2 4	8	0	SS-13	-	-	-	-	-	-	-	-	-	18	A-4a (V)	
	436.1	60																

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 3/9/11 10:06 - N:\PROJECTS\2010\N1105070\GINT\ODOT LOGS.GPJ

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 20+86.5, 55.1 LT	START: 5/17/10	END: 5/20/10	PG 2 OF 3	L-3A

MATERIAL DESCRIPTION AND NOTES	ELEV. 436.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY STIFF, BLACK AND BROWN, **COARSE AND FINE SAND**, SOME WOOD FRAGMENTS, WET		61	5 4 4	11	100	SS-14	2.25	-	-	-	-	-	-	-	-	29	A-3a (V)	
	431.1	65 W																
MEDIUM DENSE TO DENSE, BROWN AND GRAY, **FINE SAND**, AND COARSE SAND, TRACE SILT, TRACE CLAY, TRACE GRAVEL, WET		66	6 4 4	11	100	SS-15	-	-	-	-	-	-	-	-	-	29	A-3 (V)	
		71	5 8 8	22	100	SS-16	-	-	-	-	-	-	-	-	-	18	A-3 (V)	
		76	5 7 11	25	100	SS-17	-	7	38	45	5	5	NP	NP	NP	20	A-3 (0)	
		81	11 15 14	40	100	SS-18	-	-	-	-	-	-	-	-	-	19	A-3 (V)	
		86	10 12 11	32	100	SS-19	-	-	-	-	-	-	-	-	-	14	A-3 (V)	
	406.1	90																
VERY DENSE, BROWN, **SILT**, TRACE SAND, WET		91	10 18 24	59	100	SS-20	-	-	-	-	-	-	-	-	-	24	A-4b (V)	
	401.1	95																
DENSE TO VERY DENSE, BROWN, **FINE SAND**, TRACE SILT, TRACE CLAY, WET		96	22 25 25	70	100	SS-21	-	-	-	-	-	-	-	-	-	18	A-3 (V)	
		101	8 16 16	45	0	SS-22	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
	391.1	105																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL WITH SAND**, TRACE SILT, TRACE CLAY, WET		106	19 9 11	28	100	SS-23	-	39	33	23	2	3	NP	NP	NP	18	A-1-b (0)	
		111	18 12 10	31	67	SS-24	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		116	13 13 12	35	100	SS-25	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
	376.1	120																
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET		121	49 49 50/5"	-	106	SS-26	-	59	17	17	4	3	NP	NP	NP	12	A-1-a (0)	

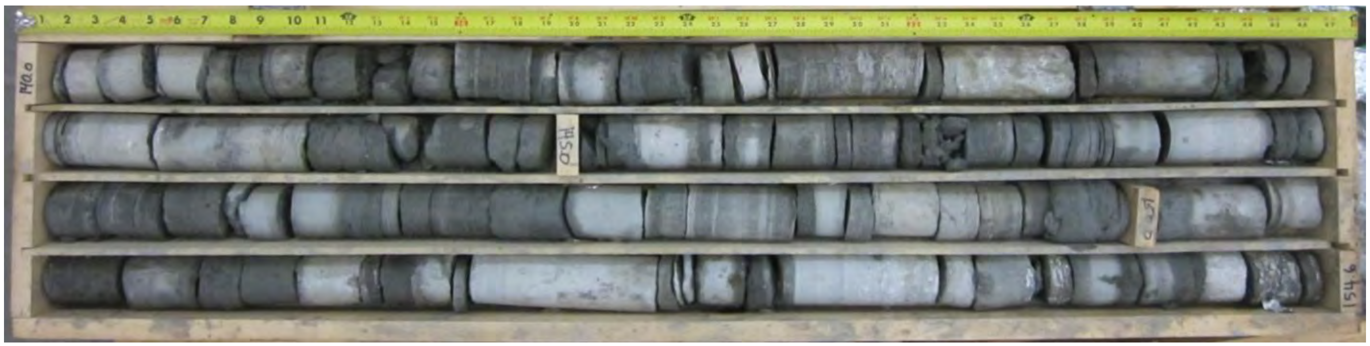
PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 20+86.5, 55.1 LT	START: 5/17/10	END: 5/20/10	PG 3 OF 3	L-3A

MATERIAL DESCRIPTION AND NOTES	ELEV. 374.2	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET *(continued)*		123–124	37 42 11	74	100	SS-27	-	-	-	-	-	-	-	-	-	20	A-1-a (V)	
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED, VERY STRONG, THIN BEDDED, FOSSILIFEROUS, ARGILLACEOUS, LOSS 2%, RQD 39%, MODERATELY FRACTURED; **SHALE**, GRAY, UNWEATHERED TO HIGHLY WEATHERED, WEAK TO SLIGHTLY STRONG, LAMINATED,	371.1	TR 125	50/2"	-	100	SS-28	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SH @126.5'-126.75' QU=570 PSI		125–130	31		83	NQ-1											CORE	
SH @ 134.25' SDI = 85.3		130–135	25		100	NQ-2											CORE	
LS @ 134.6' POINT LOAD = 14846 PSI		135–140	35		100	NQ-3											CORE	
SH @142.3'-142.5' QU=4272 PSI		140–145	40		100	NQ-4											CORE	
SH @ 150.5' SDI = 97.7		145–150	50		100	NQ-5											CORE	
LS @ 152.75' POINT LOAD = 12976 PSI		150–155	40		100	NQ-6											CORE	
LS @155'-155.5' QU=16975 PSI		155–160	66		100	NQ-7											CORE	
SH @157.7'-158' QU=2759 PSI.		160–165	22		100	NQ-8											CORE	
	331.1	EOB 165																

NOTES: WATER USED AT 126 FT. FOR ROCK CORING PURPOSES. WATER NOTED AT 38 FT. AFTER 24 HRS.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (13 BAGS CEMENT/2 BAGS BENTONITE)

BORING NO.: L-3A
STA. OFFSET
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 126.2-140.0
ELEVATION (ft.): 369.85
1/NQ: 126.2' – 130.0'; REC. 84%, RQD 32%
2/NQ: 130.0 – 135.0'; REC. 100%, RQD 25%
3/NQ: 135.0 – 140.0'; REC. 100%, RQD 35%



BORING NO.: L-3A
STA. OFFSET
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 140.0-155.0
ELEVATION (ft.): 356.05
4/NQ: 140.0' – 145.0'; REC. 100%, RQD 50%
5/NQ: 145.0' – 150.0'; REC. 100%, RQD 50%
6/NQ: 150.0' – 155.0'; REC. 100%, RQD 40%

BORING NO.: L-3A
STA. OFFSET
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 155.5-165.0
ELEVATION (ft.): 341.05
7/NQ: 155.0' – 160.0'; REC. 100%, RQD 66%
8/NQ: 160.0'-165.0'; REC. 100%, RQD 22%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core B Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-3A

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JM	DRILL RIG: DIEDRICH D-50	STATION / OFFSET: 5+65.5, 12.9 LT	EXPLORATION ID L-4
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 9/9/10	ELEVATION: 480.0 (MSL) EOB: 159.0 ft.	PAGE
START: 6/30/10 END: 7/7/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 83.7	COORD: 39.088805640, -84.523275430	1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 480.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM STIFF, BROWN AND GRAY, **SILT AND CLAY**, AND SAND, TRACE SHALE FRAGMENTS, WOOD, ORGANICS, AND GRAVEL (FILL), MOIST		1	1 3 4	10	78	SS-1	1.50	-	-	-	-	-	-	-	-	38	A-6a (V)	
		3	4 6 6	17	89	SS-2	2.00	-	-	-	-	-	-	-	-	19	A-6a (V)	
		6	3 5 6	15	100	SS-3	2.25	-	-	-	-	-	-	-	-	26	A-6a (V)	
		8	3 4 9	18	67	SS-4	2.00	10	24	18	31	17	35	24	11	27	A-6a (3)	
		11	2 3 1	6	6	SS-5	2.50	-	-	-	-	-	-	-	-	13	A-6a (V)	
VERY LOOSE TO LOOSE, DARK BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, SLAG, AND ORGANICS, (FILL), WET	467.5	13	12 2 2	6	33	SS-6	-	-	-	-	-	-	-	-	-	18	A-1-b (V)	
		16	3 1 2	4	0	SS-7	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		18	2 2 2	6	22	SS-8	-	-	-	-	-	-	-	-	-	24	A-1-b (V)	
		21	3 2 3	7	39	SS-9	-	-	-	-	-	-	-	-	-	28	A-1-b (V)	
		22 ▼																
SOFT, GRAY, **CLAY**, AND SILT, TRACE ORGANICS AND SAND, LOI=5.4% (25'), MOIST	455.0	26	2 2 2	6	67	SS-10	1.25	0	0	2	62	36	50	29	21	44	A-7-6 (14)	
		30 W																
		31			83	ST-11	1.50	0	0	1	62	37	46	25	21	37	A-7-6 (14)	
		33	2 1 3	6	6	SS-12	1.00	-	-	-	-	-	-	-	-	44	A-7-6 (V)	
		36	1 2 3	7	100	SS-13	1.50	-	-	-	-	-	-	-	-	32	A-7-6 (V)	
		41	3 4 3	10	72	SS-14	-	-	-	-	-	-	-	-	-	24	A-7-6 (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY DENSE AT 65', WET	435.0	46	7 8 6	20	28	SS-15	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		51	11 15 17	45	56	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		56	17 14 14	39	56	SS-17	-	54	9	27	7	3	NP	NP	NP	12	A-1-b (0)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 5+65.5, 12.9 LT	START: 6/30/10	END: 7/7/10	PG 2 OF 3	L-4

MATERIAL DESCRIPTION AND NOTES	ELEV. 420.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY DENSE AT 65', WET *(continued)*		61	18 15 16	43	33	SS-18	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		66	40 31 34	91	33	SS-19	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
	410.0	70																
DENSE TO VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, WET		71	9 17 18	49	67	SS-20	-	56	25	13	3	3	NP	NP	NP	11	A-1-a (0)	
		76	11 19 20	54	56	SS-21	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
	400.0	80																
DENSE TO VERY DENSE, BROWN TRACE GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, MEDIUM DENSE AT 80', WET		81	9 11 10	29	67	SS-22	-	20	63	11	3	3	NP	NP	NP	18	A-1-b (0)	
		86	9 18 17	49	6	SS-23	-	-	-	-	-	-	-	-	-	15	A-1-b (V)	
		91	31 26 45	99	67	SS-24	-	-	-	-	-	-	-	-	-	6	A-1-b (V)	
	388.0	92																
VERY DENSE, GRAY, **STONE FRAGMENTS WITH SAND**, LIMESTONE FRAGMENTS, WET		92	34 100/3"	-	67	SS-25	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		95	100/1"	-	0	SS-26	-	-	-	-	-	-	-	-	-	11	A-1-b (V)	
		100	100/3"	-	0	SS-27	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	376.0	TR 104																
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, FRACTURED, LOSS 11%, RQD 12%; **SHALE**, GRAY, SLIGHTLY WEATHERED, WEAK, LAMINATED, FISSILE, SH @ 108.5' SDI = 59.2		104–107	0		17	NQ-1											CORE	
LS @116'-116.5' QU=13646 PSI LS @120.4'-120.9' QU=12705 PSI LS @127.5'-128' QU=17130 PSI LS @ 132.4' POINT LOAD = 11696 PSI LS @140.5'-141' QU=13056 PSI LS @143'-143.5' QU=12509 PSI LS @ 141.4' POINT LOAD = 11853 PSI.		107–117	12		90	NQ-2											CORE	
		117–121																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 5+65.5, 12.9 LT	START: 6/30/10	END: 7/7/10	PG 3 OF 3	L-4

MATERIAL DESCRIPTION AND NOTES	ELEV. 358.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, FRACTURED, LOSS 11%, RQD 12%; **SHALE**, GRAY, SLIGHTLY WEATHERED, WEAK, LAMINATED, FISSILE, SH @ 108.5' SDI = 59.2 LS @116'-116.5' QU=13646 PSI LS @120.4'-120.9' QU=12705 PSI LS @127.5'-128' QU=17130 PSI LS @ 132.4' POINT LOAD = 11696 PSI LS @140.5'-141' QU=13056 PSI LS @143'-143.5' QU=12509 PSI LS @ 141.4' POINT LOAD = 11853 PSI. *(continued)*		122–127	20		98	NQ-3											CORE	
		127–128	50		100	NQ-4											CORE	
		128–138	0		94	NQ-5											CORE	
	332.0	138–148	21		94	NQ-6											CORE	
BLANK DRILLED FOR SEISMIC TESTING	321.0	148–159 EOB																

NOTES: WATER USED IN DRILLING AT THE SURFACE. WATER NOTED AT 32 FT. AFTER 24 HRS. 3 INCH PVC CASING INSTALLED FROM SURFACE TO 159 FEET. CASING BROKE DURING REMOVAL AN

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (12 BAGS CEMENT/1.5 BAGS BENTONITE)

BORING NO.: L-4
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 104.0-127.0
ELEVATION (ft.): 375.97
1/NQ: 104.0’-107.0‘; REC. 17%, RQD 0%
2/NQ: 107.0’-117.0’; REC. 90%, RQD 12%
3/NQ: 117.0’-127.0’; REC. 98%, RQD 20%

BORING NO.: L- 4
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 127.0-138.0
ELEVATION (ft.): 352.97
4/NQ: 127.’-128.0’; REC. 100%, RQD 50%
5/NQ: 128.0’-138.0’; REC. 99%, RQD 0%

BORING NO.: L- 4
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 138.0-148.0
ELEVATION (ft.): 341.97
6/NQ: 138.0’-148.0‘; REC. 94%, RQD 21%

Project Mngr.: AJM	PN. N1105070	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	L-4
Chkd By: DWW	File No. Core B			
Approved By: AJM	Date: 9-8-10			

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JJ	DRILL RIG: CME 550X ATV- 9333	STATION / OFFSET: 3+72.7, 10.9 LT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 486.3 (MSL) EOB: 147.0 ft.
START: 6/30/10 END: 7/2/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 67.1	COORD: 39.088276550, -84.523297540

EXPLORATION ID L-5

MATERIAL DESCRIPTION AND NOTES	ELEV. 486.3
ASPHALT PAVEMENT	485.9
PRE-DRILLED (VACUUM EXCAVATED)	481.3
MEDIUM STIFF TO STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE ORGANICS, TRACE ROCK FRAGMENTS, (FILL), MOIST	461.3
STIFF, BROWN, TRACE GRAY, **SILTY CLAY**, TRACE FINE SAND, MOIST	451.3
MEDIUM DENSE, GRAY, **SANDY SILT**, TRACE GRAVEL, VERY LOOSE AT 35', MOIST TO WET	441.3 (W)
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, CLAY SEAM AT 50', VERY DENSE AT 60', WET	

SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
5		100	SS-1	1.00	-	-	-	-	-	-	-	-	38	A-6a (V)	
2 3 6	10	100	SS-2	1.00	-	-	-	-	-	-	-	-	20	A-6a (V)	
3 5 7	13	67	SS-3	1.75	-	-	-	-	-	-	-	-	21	A-6a (V)	
4 8 8	18	100	SS-4	2.00	-	-	-	-	-	-	-	-	17	A-6a (V)	
4 4 4	9	100	SS-5	2.00	-	-	-	-	-	-	-	-	26	A-6a (V)	
2 3 5	9	100	SS-6	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
2 2 3	6	100	SS-7	1.50	-	-	-	-	-	-	-	-	23	A-6a (V)	
		83	ST-8	-	0	0	16	46	38	29	17	12	25	A-6a (9)	
2 2 3	6	100	SS-9	2.00	-	-	-	-	-	-	-	-	22	A-6b (V)	
2 2 2	4	100	SS-10	1.75	-	-	-	-	-	-	-	-	27	A-6b (V)	
2 2 2	4	100	SS-11	-	-	-	-	-	-	-	-	-	27	A-4a (V)	
		100	ST-12	2.50	0	0	21	48	31	29	19	10	31	A-4a (8)	
10 13 15	31	100	SS-13	-	-	-	-	-	-	-	-	-	28	A-4a (V)	
4 5 6	12	56	SS-14	-	-	-	-	-	-	-	-	-	13	A-3a (V)	
16 7 20	30	100	SS-15	-	-	-	-	-	-	-	-	-	18	A-3a (V)	
22 20 20	45	100	SS-16	-	-	-	-	-	-	-	-	-	8	A-3a (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 3+72.7, 10.9 LT	START: 6/30/10	END: 7/2/10	PG 2 OF 3	L-5

MATERIAL DESCRIPTION AND NOTES	ELEV. 426.3	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE, BROWN, **COARSE AND FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, CLAY SEAM AT 50', VERY DENSE AT 60', WET *(continued)*		61	12 26 27	59	100	SS-17	-	-	-	-	-	-	-	-	-	14	A-3a (V)	
		62–64																
		65–66	16 23 21	49	67	SS-18	-	-	-	-	-	-	-	-	-	11	A-3a (V)	
		67–69																
		70–71	13 14 17	35	67	SS-19	-	-	-	-	-	-	-	-	-	11	A-3a (V)	
	411.3	72–74																
DENSE TO VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET		75–76	23 37 40	86	100	SS-20	-	56	17	17	7	3	NP	NP	NP	12	A-1-a (0)	
		77–79																
		80–81	18 24 23	53	67	SS-21	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		82–84																
		85–86	16 20 22	47	100	SS-22	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	396.3	87–89																
VERY DENSE, GRAY AND BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE COBBLES, TRACE CLAY, WET		90–91	57 65 50	129	100	SS-23	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		92–94																
		95–96	76 52 57	122	100	SS-24	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		97–99																
		100	39 100/4"	-	100	SS-25	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		101–104																
		105	100/3"	-	133	SS-26	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	379.3	106																
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, LOSS 0%, RQD 39%; **SHALE**, GRAY, MODERATELY TO SLIGHTLY WEATHERED, VERY WEAK TO WEAK, LAMINATED, FISSILE, SH @ 109' SDI = 50.9 LS/SH @113.5'-114' QU=6755 PSI SH @ 118.5' SDI = 48.1 LS @120.2'-120.6' QU=10888 PSI.		TR 107–111	32		100	NQ-1											CORE	
		112–116	46		100	NQ-2											CORE	
		117–121	38		100	NQ-3											CORE	

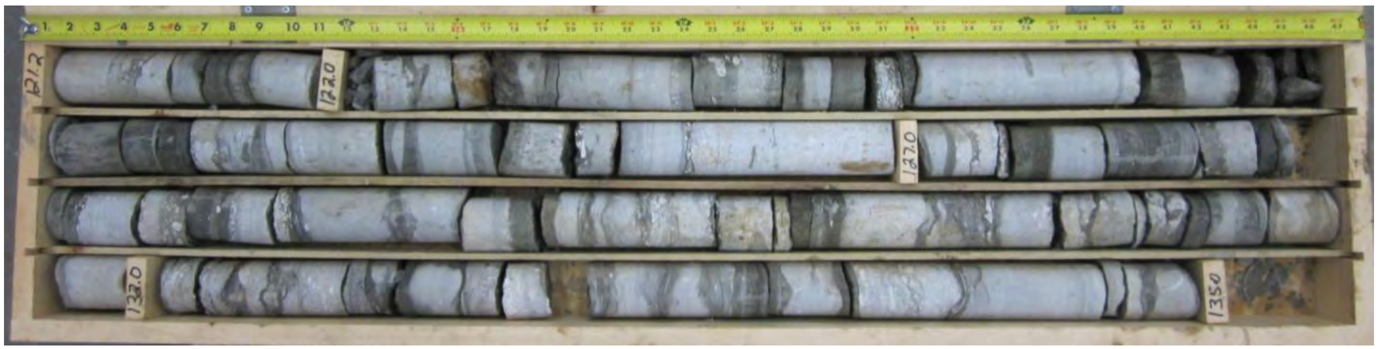
PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 3+72.7, 10.9 LT	START: 6/30/10	END: 7/2/10	PG 3 OF 3	L-5

MATERIAL DESCRIPTION AND NOTES	ELEV. 364.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, ARGILLACEOUS, TRACE SHALE PARTINGS TO SEAMS, NOTED CALCITE FILLED VUGS; LOSS 0%, RQD=64% LS @ 122.7' POINT LOAD = 14712 PSI LS/SH @130.3'-131' QU=6755 PSI LS/SH @133.3'-133.8' QU=8455 PSI LS @137.3'-138' QU=20794 PSI LS @ 143.5' POINT LOAD = 144 PSI.	364.3	123–127	48		100	NQ-4											CORE	
		127–132	58		100	NQ-5											CORE	
		132–137	62		100	NQ-6											CORE	
		137–142	66		100	NQ-7											CORE	
	339.3	142–147 EOB	84		100	NQ-8											CORE	

NOTES: WATER USED BELOW 45 FT. FOR DRILLING/ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (11 BAGS CEMENT/1.5 BAGS BENTONITE)

BORING NO.: L- 5
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 107.0-122.0
ELEVATION (ft.): 379.33
1/NQ: 107.0'-112.0'; REC. 100%, RQD 32%
2/NQ: 112.0'-117.0'; REC. 100%, RQD 46%
3/NQ: 117.0'-122.0'; REC. 100%, RQD 38%



BORING NO.: L- 5
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 122.0-135.0
ELEVATION (ft.): 364.33
4/NQ: 122.0'-127.0'; REC. 100%, RQD 48%
5/NQ: 127.0'-132.0'; REC. 100%, RQD 58%
6/NQ: 132.0'-137.0'; REC.100 %, RQD 62%

BORING NO.: L- 5
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 135.0-147.0
ELEVATION (ft.): 351.33
7/NQ: 137.0'-142.0'; REC. 100%, RQD 66%
8/NQ: 142.0'-147.0'; REC. 100%, RQD 84%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core B Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-5

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / JM	DRILL RIG: DIEDRICH D-50	STATION / OFFSET: 2+49.4, 51.4 RT	EXPLORATION ID L-6
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 9/9/10	ELEVATION: 485.7 (MSL) EOB: 148.5 ft.	PAGE
START: 6/28/10 END: 6/30/10	SAMPLING METHOD: SPT / ST / NQ	ENERGY RATIO (%): 83.7	COORD: 39.087930220, -84.523068980	1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 485.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
CONCRETE	485.3																	
PRE-DRILLED (VACUUM EXCAVATED)	478.2																	
STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, MOIST		8 – 9	2 3 4	10	67	SS-1	2.00	-	-	-	-	-	-	-	-	21	A-6a (V)	
		10 – 11	2 3 5	11	83	SS-2	2.00	0	0	10	53	37	33	19	14	22	A-6a (10)	
		13 – 14			83	ST-3	1.50	-	-	-	-	-	-	-	-	26	A-6a (V)	
		15 – 16	3 3 3	8	67	SS-4	2.00	0	0	9	54	37	33	19	14	24	A-6a (10)	
		20 – 21	2 3 5	11	100	SS-5	2.00	-	-	-	-	-	-	-	-	25	A-6a (V)	
	460.7																	
STIFF, GRAY, **SILT AND CLAY**, TRACE SILT SEAMS, TRACE SAND, MOIST		25 – 26	2 2 4	8	100	SS-6	1.50	0	0	5	62	33	32	20	12	27	A-6a (9)	
		30 – 31			100	ST-7	1.25	0	0	10	56	34	30	19	11	28	A-6a (8)	
		32 – 33	1 2 4	8	100	SS-8	1.50	-	-	-	-	-	-	-	-	-	A-6a (V)	
		35 – 36	1 2 3	7	100	SS-9	2.00	-	-	-	-	-	-	-	-	27	A-6a (V)	
		40 – 41	1 2 3	7	100	SS-10	1.75	-	-	-	-	-	-	-	-	27	A-6a (V)	
	440.7																	
MEDIUM DENSE TO DENSE, BROWN, **SILT**, LITTLE SAND, TRACE CLAY, LITTLE GRAVEL, VERY DENSE AT 70', WET		45 – 46	3 4 5	13	78	SS-11	-	-	-	-	-	-	-	-	-	21	A-4b (V)	
		50 – 51	9 10 9	27	67	SS-12	-	14	6	9	57	14	NP	NP	NP	23	A-4b (7)	
		55 – 56	5 17 18	49	67	SS-13	-	-	-	-	-	-	-	-	-	21	A-4b (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 2+49.4, 51.4 RT	START: 6/28/10	END: 6/30/10	PG 2 OF 3	L-6

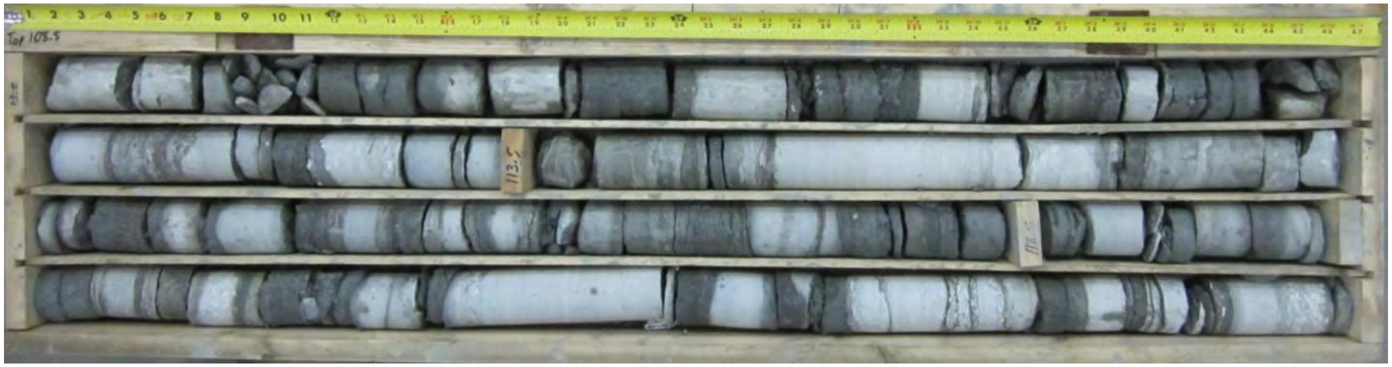
MATERIAL DESCRIPTION AND NOTES	ELEV. 425.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, BROWN, **SILT**, LITTLE SAND, TRACE CLAY, LITTLE GRAVEL, VERY DENSE AT 70', WET *(continued)*		61	15 18 17	49	33	SS-14	-	-	-	-	-	-	-	-	-	8	A-4b (V)	
		62 – 65																
		66	11 13 18	43	67	SS-15	-	-	-	-	-	-	-	-	-	16	A-4b (V)	
		67 – 70																
		71	19 37 34	99	78	SS-16	-	-	-	-	-	-	-	-	-	9	A-4b (V)	
		72 – 75																
		76	10 21 23	61	67	SS-17	-	-	-	-	-	-	-	-	-	11	A-4b (V)	
	405.7	77 – 80																
DENSE TO VERY DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		81	19 19 13	45	78	SS-18	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		82 – 85																
		86	17 17 21	53	67	SS-19	-	-	-	-	-	-	-	-	-	12	A-1-b (V)	
	395.7	87 – 90																
VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		91	26 40 36	106	67	SS-20	-	31	28	27	10	4	NP	NP	NP	9	A-1-b (0)	
		92 – 94																
		95	50/5"	-	80	SS-21	-	-	-	-	-	-	-	-	-	8	A-1-b (V)	
		96 – 99																
		100	100/4"	-	75	SS-22	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
		101 – 105																
		106	44 39 50/5"	-	47	SS-23	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	377.2	107 – 108 (TR)																
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, FOSSILIFEROUS, LOSS 1%, RQD 42%; **SHALE**, GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, LAMINATED, FISSILE, SH @ 110' SDI = 56.9		109	100/1"	-	0	SS-24	-	-	-	-	-	-	-	-	-	-		
LS/SH @112'-112.4' QU=4889 PSI		111	11		100	NQ-1											CORE	
LS @ 114' POINT LOAD = 11720 PSI																		
SH @ 117.7' SDI = 55.1																		
LS @120.5'-121' QU=14568 PSI		116	64		100	NQ-2											CORE	
LS @ 126.3' POINT LOAD = 12087 PSI.																		
		121	50		100	NQ-3											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 2+49.4, 51.4 RT	START: 6/28/10	END: 6/30/10	PG 3 OF 3	L-6

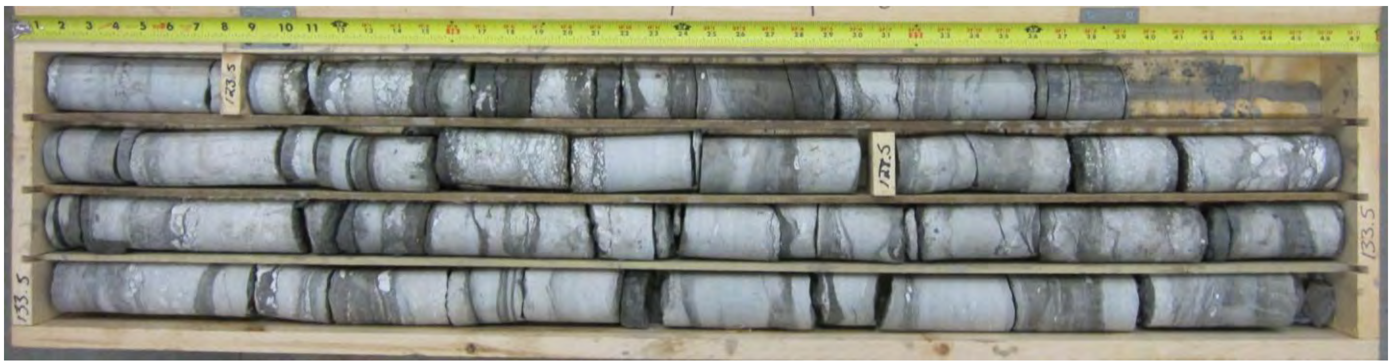
MATERIAL DESCRIPTION AND NOTES	ELEV. 363.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
	362.2	123																
LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, ARGILLACEOUS, ARGILLACEOUS SEAMS, FOSSILIFEROUS SEAMS, TRACE SHALE PARTINGS; LOSS 1%, RQD=68% LS @130.5'-130.9' QU=9864 PSI LS @138'-138.3' QU=25530 PSI LS @147.5'-148' QU=10726 PSI.		124–128	60		100	NQ-4											CORE	
		129–133	76		98	NQ-5											CORE	
		134–138	56		100	NQ-6											CORE	
		139–143	76		100	NQ-8											CORE	
		144–148	82		100	NQ-7											CORE	
	337.2	EOB																

NOTES: WATER USED BELOW 45 FT. FOR DRILLING/ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (12 BAGS CEMENT/1.5 BAGS BENTONITE)



BORING NO.: L- 6
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 108.5-123.5
ELEVATION (ft.): 377.19
1/NQ: 108.5'-113.5'; REC. 98%, RQD 11%
2/NQ: 113.5'-118.5'; REC. 100%, RQD 64%
3/NQ: 118.5'-123.5'; REC. 100%, RQD 50%



BORING NO.: L- 6
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 123.5-138.5
ELEVATION (ft.): 362.19
4/NQ: 123.5'-128.5'; REC. 100%, RQD 60%
5/NQ: 128.5'-133.5'; REC. 98%, RQD 76%
6/NQ: 133.5'-138.5'; REC. 100%, RQD 56%

BORING NO.: L- 6
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 138.5-143.5
ELEVATION (ft.): 347.19
7/NQ: 138.5'-143.5'; REC. 100%, RQD 76%
8/NQ: 143.5'-148.5'; REC. 100%, RQD 82%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core B Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING L-6

PROJECT: BRENT SPENCE BRIDGE TYPE: BRIDGE REPLACEMENT PID: 75119 BR ID: START: 6/28/10 END: 6/30/10	DRILLING FIRM / OPERATOR: HCN / JJ SAMPLING FIRM / LOGGER: HCN / DRK/DWW DRILLING METHOD: 3.25" HSA / NQ SAMPLING METHOD: SPT / ST / NQ	DRILL RIG: CME 550X ATV- 9333 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/4/10 ENERGY RATIO (%): 67.1	STATION / OFFSET: 1+52.6, 0.8 LT ALIGNMENT: PROPOSED BSB ELEVATION: 484.4 (MSL) EOB: 141.7 ft. COORD: 39.087671860, -84.523295650

EXPLORATION ID: L-7

MATERIAL DESCRIPTION AND NOTES	ELEV. 484.4
ASPHALT PAVEMENT	483.9
PRE-DRILLED (VACUUM EXCAVATED)	479.4
VERY LOOSE, GRAY AND BLACK, **SANDY SILT**, LITTLE GRAVEL, LITTLE CLAY, TRACE CONCRETE (FILL), MOIST	474.4
MEDIUM STIFF, GRAY, **SILT AND CLAY**, SOME GRAVEL, TRACE ORGANICS, TRACE SAND (FILL), MOIST	469.4
STIFF TO VERY STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, TRACE GRAVEL, MOIST	449.4 (W)
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, VERY LOOSE AT 45', WET	

DEPTHS: 1 – 59

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WOH WOH 1	1	61	SS-1	-	-	-	-	-	-	-	-	-	26	A-4a (V)	
1 1 1	2	100	SS-2	-	23	17	22	20	18	NP	NP	NP	19	A-4a (1)	
WOH WOH 1	1	100	SS-3	1.00	-	-	-	-	-	-	-	-	23	A-6a (V)	
		63	ST-4	-	31	2	8	30	29	29	17	12	21	A-6a (6)	
2 3 4	8	100	SS-5	1.00	-	-	-	-	-	-	-	-	27	A-6a (V)	
2 2 4	7	100	SS-6	-	-	-	-	-	-	-	-	-	23	A-6a (V)	
		100	ST-7	1.50	0	0	18	44	38	31	17	14	25	A-6a (10)	
2 3 4	8	33	SS-8	2.50	0	0	17	48	35	32	18	14	23	A-6a (10)	
3 4 6	11	100	SS-9	1.00	-	-	-	-	-	-	-	-	30	A-6a (V)	
1 1 2	3	100	SS-10	-	-	-	-	-	-	-	-	-	24	A-6a (V)	
		75	ST-11	-	33	4	14	25	24	31	17	14	25	A-6a (4)	
3 3 4	8	100	SS-12	-	-	-	-	-	-	-	-	-	22	A-1-a (V)	
3 4 5	10	44	SS-13	-	-	-	-	-	-	-	-	-	27	A-1-a (V)	
2 2 2	4	100	SS-14	-	-	-	-	-	-	-	-	-	33	A-1-a (V)	
53 9 7	18	67	SS-15	-	-	-	-	-	-	-	-	-	16	A-1-a (V)	
9 11 17	31	100	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 1+52.6, 0.8 LT	START: 6/28/10	END: 6/30/10	PG 2 OF 3	L-7

MATERIAL DESCRIPTION AND NOTES	ELEV. 424.4	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, AND SAND, TRACE SILT, TRACE CLAY, VERY LOOSE AT 45', WET *(continued)*		61	13 14 14	31	67	SS-17	-	55	28	10	5	2	NP	NP	NP	10	A-1-a (0)	
		66	7 12 13	28	100	SS-18	-	-	-	-	-	-	-	-	-	19	A-1-a (V)	
		71	10 13 14	30	67	SS-19	-	-	-	-	-	-	-	-	-	12	A-1-a (V)	
	409.4	75																
DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE COBBLES, TRACE SILT, TRACE CLAY, WET		76	14 13 15	31	83	SS-20	-	61	18	13	5	3	NP	NP	NP	9	A-1-a (0)	
		81	20 30 27	64	67	SS-21	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
	399.4	85																
VERY DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE COBBLES, LITTLE SILT, TRACE CLAY, WET		85	100 50/0"	-	100	SS-22	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		90	100/4"	-	100	SS-23	-	52	14	17	11	6	NP	NP	NP	8	A-1-b (0)	
		96	37 53 60	126	67	SS-24	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
	384.4	100 (TR)																
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG, THIN BEDDED, FOSSILIFEROUS, LOSS 0%, RQD 66%; **SHALE**, GRAY, SLIGHTLY WEATHERED, VERY WEAK TO WEAK, LAMINATED, FISSILE, LS @ 101'-101.5' QU=8217 PSI		100	100/4"	-	100	SS-25	-	-	-	-	-	-	-	-	-	-	Rock (V)	
SH @ 105.5' SDI = 65.9		101	34		100	NQ-1											CORE	
SH @ 107.5' SDI = 93.3		104	48		100	NQ-2											CORE	
LS @ 113.7'-114.2' QU=5847 PSI		109	70		100	NQ-3											CORE	
LS @ 116' POINT LOAD = 10879 PSI		114	82		100	NQ-4											CORE	
LS @125.7'-126.2' QU=23281 PSI		119	84		100	NQ-5											CORE	
SH @ 118.6' SDI = 77.0.																		

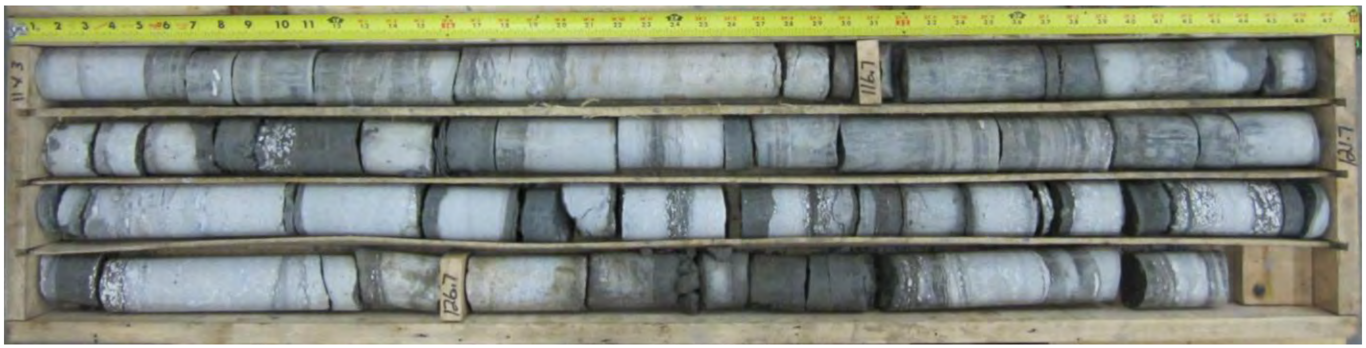
PID: 75119 | BR ID: ______________ | PROJECT: BRENT SPENCE BRIDGE | STATION / OFFSET: 1+52.6, 0.8 LT | START: 6/28/10 | END: 6/30/10 | PG 3 OF 3 | L-7

MATERIAL DESCRIPTION AND NOTES	ELEV. 362.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
		123 – 127	56		100	NQ-6											CORE	
LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG, ARGILLACEOUS, FOSSILIFEROUS SEAMS, TRACE SHALE PARTINGS; LOSS 0%, RQD=86% LS/SH @132.5'-133.2' QU=4790 PSI LS @ 139.7' POINT LOAD = 11517 PSI.	356.4	128 – 131	76		100	NQ-7											CORE	
		132 – 136	92		100	NQ-8											CORE	
	342.7	137 – 141, EOB	90		100	NQ-9											CORE	

NOTES: WATER USED BELOW 35 FT. FOR DRILLING/ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (12 BAGS CEMENT/1 BAGS BENTONITE)

BORING NO.: L- 7
CORE BOX NO.: 1 OF 3
DEPTH (ft.): 100.5-116.7
ELEVATION (ft.): 383.91
1/NQ: 100.5'-101.7'; REC. 100%, RQD 33%
2/NQ: 101.7'-106.7'; REC. 100%, RQD 48%
3/NQ: 106.7'-111.7'; REC. 100%, RQD 70%
4/NQ: 111.7'-116.7'; REC. 100%, RQD 82%



BORING NO.: L- 7
CORE BOX NO.: 2 OF 3
DEPTH (ft.): 116.7-131.7
ELEVATION (ft.): 367.71
5/NQ: 116.7'-121.7'; REC. 100%, RQD 84%
6/NQ: 121.7'-126.7'; REC. 100%, RQD 56%
7/NQ: 126.7'-131.7'; REC. 100%, RQD 76%

BORING NO.: L- 7
CORE BOX NO.: 3 OF 3
DEPTH (ft.): 131.7-141.7
ELEVATION (ft.): 352.71
8/NQ: 131.7'-136.7'; REC. 100%, RQD 92%
9/NQ: 136.7'-141.7'; REC. 100%, RQD 90%

Project Mngr.: AJM	PN. N1105070	A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	L-7
Chkd By: DWW	File No. Core B	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-8-10		CINCINNATI, OHIO	

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 17+75.2, 41.52 RT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 458.0 (MSL) EOB: 170.0 ft.
START: 7/9/10 END: 7/11/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 76.3	COORD: 39.092117290, -84.522898570

EXPLORATION ID R-1

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WATER (OHIO RIVER)		0–32																
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, LITTLE SAND, TRACE SILT, TRACE CLAY, VERY DENSE AT 33.5', WET	426.0	32–33.5	5 4 4	10	22	SS-1	-	87	11	2	0	0	NP	NP	NP	12	A-1-a (0)	
		33.5–35	9 24 23	60	33	SS-2	-	70	21	7	1	1	NP	NP	NP	13	A-1-a (0)	
		35–36.5	13 2 3	6	0	SS-3	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY LOOSE AT 36.5', WET	421.5	36.5–38	3 1 2	4	67	SS-4	-	2	64	29	2	3	NP	NP	NP	23	A-1-b (0)	
		38–39.5	5 6 6	15	0	SS-5	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, SAMPLE NOT OBTAINED AT 50' DUE TO SAND IN CASING, WET	418.5	39.5–41	3 2 2	5	56	SS-6	-	2	24	69	2	3	NP	NP	NP	23	A-3 (0)	
		41–42.5	2 4 10	18	33	SS-7	-	-	-	-	-	-	-	-	-	23	A-3 (V)	
		42.5–44	6 7 8	19	22	SS-8	-	-	-	-	-	-	-	-	-	21	A-3 (V)	
		45–46.5	14 7 8	19	100	SS-9	-	-	-	-	-	-	-	-	-	20	A-3 (V)	
		50–51.5			0	SS-10	-	-	-	-	-	-	-	-	-	-	A-3 (V)	
		55–56.5	6 7 7	18	100	SS-11	-	2	32	61	3	2	NP	NP	NP	21	A-3 (0)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+75.2, 41.52 RT	START: 7/9/10	END: 7/11/10	PG 2 OF 3	R-1

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, SAMPLE NOT OBTAINED AT 50' DUE TO SAND IN CASING, WET *(continued)*		61	5-7-7	18	67	SS-12	-	-	-	-	-	-	-	-	-	26	A-3 (V)	
	393.0	62–65																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET		66	15-8-8	20	67	SS-13	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		71	22-17-12	37	22	SS-14	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		76	5-7-15	28	100	SS-15	-	-	-	-	-	-	-	-	-	19	A-1-b (V)	
ROCK (AUGERED TO 87')	377.0	80	50/0"	-		SS-16	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
	371.0	81–87 (TR at 87)																
LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, MODERATELY FRACTURED, LOSS 2%, RQD 68%		87–90	50		93	NQ-1											CORE	
LS @91.5'-92.1' QU=12758 PSI LS @94.3'-95' QU=4903 PSI LS @ 101' POINT LOAD = 10455 PSI LS @104.5'-105' QU=3951 PSI LS @ 110.2' POINT LOAD = 1282 PSI		90–95	52		96	NQ-2											CORE	
LS @115'-115.9' 12584 PSI LS @123'-123.5' QU=10024 PSI LS @ 129.4' POINT LOAD = 11103 PSI LS @136'-136.5' QU=14820 PSI LS @137.7'-138.2' QU=15380 PSI		95–100	66		100	NQ-3											CORE	
LS @145.3'-145.7' QU=7449 PSI LS @ 145.7' POINT LOAD = 13095 PSI LS @146.5'-147' QU=20779 PSI LS @153'-153.6' QU=12853 PSI LS @159.1'-159.9' QU=11057 PSI		100–105	52		100	NQ-4											CORE	
LS @ 161.8' POINT LOAD = 14614 PSI LS @163.5'-164.2' QU=14214 PSI LS @168.2'-168.9' QU=13890 PSI.		105–110	66		100	NQ-5											CORE	
		110–115	70		96	NQ-6											CORE	
		115–120	80		100	NQ-7											CORE	
		120–121																

STANDARD ODOT SOIL BORING LOG (11 X 17) - OH DOT.GDT - 3/9/11 10:07 - N:\PROJECTS\2010\N1105070\GINT\ODOT LOGS.GPJ

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+75.2, 41.52 RT	START: 7/9/10	END: 7/11/10	PG 3 OF 3	R-1

MATERIAL DESCRIPTION AND NOTES — ELEV. 336.2

LIMESTONE, GRAY, UNWEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, MODERATELY FRACTURED, LOSS 2%, RQD 68%

LS @91.5'-92.1' QU=12758 PSI

LS @94.3'-95' QU=4903 PSI

LS @ 101' POINT LOAD = 10455 PSI

LS @104.5'-105' QU=3951 PSI

LS @ 110.2' POINT LOAD = 1282 PSI

LS @115'-115.9' 12584 PSI

LS @123'-123.5' QU=10024 PSI

LS @ 129.4' POINT LOAD = 11103 PSI

LS @136'-136.5' QU=14820 PSI

LS @137.7'-138.2' QU=15380 PSI

LS @145.3'-145.7' QU=7449 PSI

LS @ 145.7' POINT LOAD = 13095 PSI

LS @146.5'-147' QU=20779 PSI

LS @153'-153.6' QU=12853 PSI

LS @159.1'-159.9' QU=11057 PSI

LS @ 161.8' POINT LOAD = 14614 PSI

LS @163.5'-164.2' QU=14214 PSI

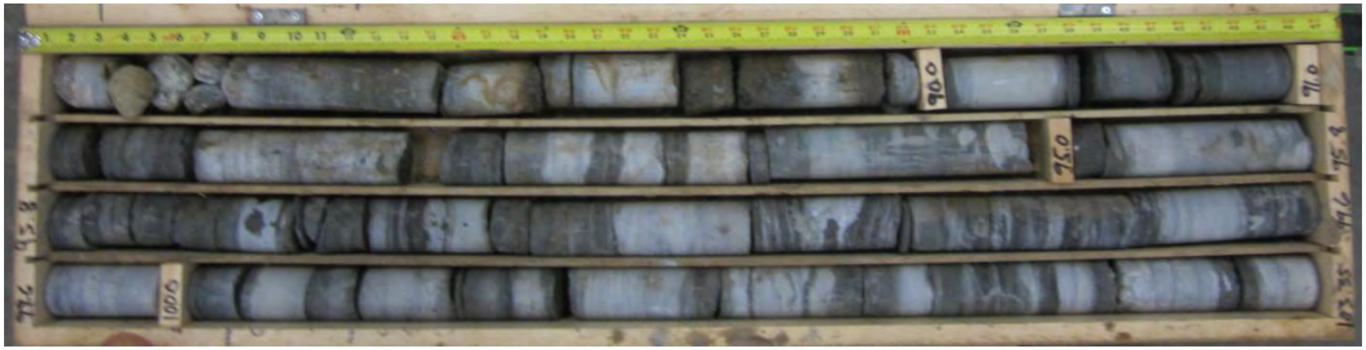
LS @168.2'-168.9' QU=13890 PSI. *(continued)*

288.0 — EOB — 170

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
122–125	88		100	NQ-8											CORE	
125–130	84		100	NQ-9											CORE	
130–135	46		100	NQ-10											CORE	
135–140	72		98	NQ-11											CORE	
140–145	76		100	NQ-12											CORE	
145–150	78		100	NQ-13											CORE	
150–155	80		100	NQ-14											CORE	
155–160	82		98	NQ-15											CORE	
160–165	74		100	NQ-16											CORE	
165–170	38		86	NQ-17											CORE	

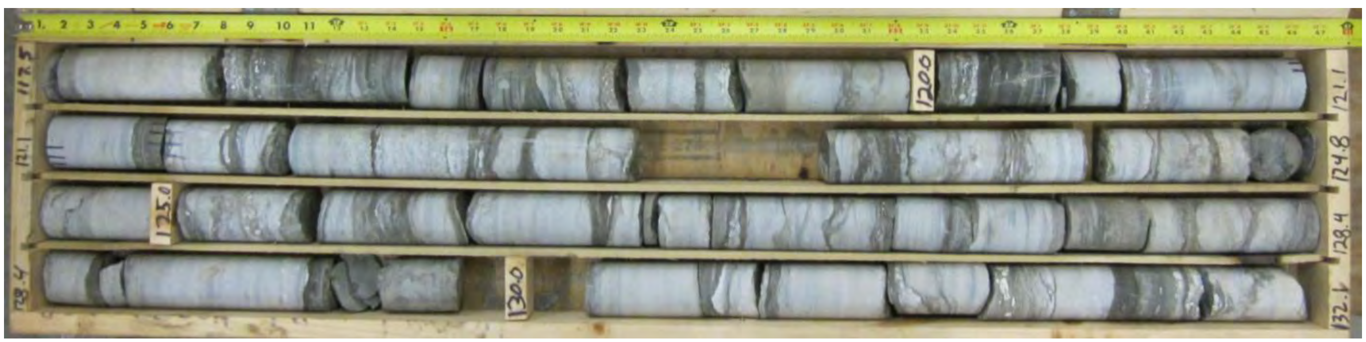
NOTES: WATER USED BELOW 87 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (9 BAGS CEMENT/1 BAG BENTONITE)



BORING NO.: R-1
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 87.0-105.0
ELEVATION (ft.): 371.04
1/NQ: 87.0'-90.0'; REC. 93%, RQD 50%
2/NQ: 90.0'-95.0'; REC. 96%, RQD 52%
3/NQ: 95.0-100.0'; REC. 100%, RQD 66%
4/NQ: 100.0'-105.0'; REC. 100%, RQD 52%

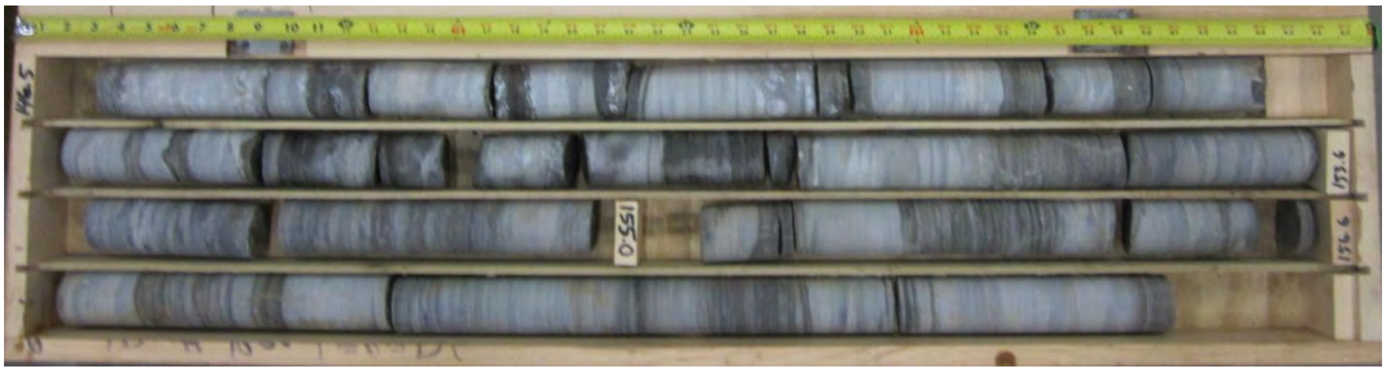
BORING NO.: R-1
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 105.0-120.0
ELEVATION (ft.): 353.04
5/NQ: 105.0'-110.0'; REC. 100%, RQD 66%
6/NQ: 110.0' – 115.0'; REC. 96%, RQD 70%
7/NQ: 115.0' – 120.0'; REC. 100%, RQD 80%



BORING NO.: R-1
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 120.0-135.0
ELEVATION (ft.): 338.04
8/NQ: 120.0'-125.0'; REC. 100%, RQD 88%
9/NQ: 125.0'-130.0'; REC. 100%, RQD 84%
10/NQ: 130.0'-135.0'; REC. 100%, RQD 46%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core C Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-1

BORING NO.: R-1
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 135.0-150.0
ELEVATION (ft.): 323.04
11/NQ: 135.0'-140.0'; REC. 98%, RQD 72%
12/NQ: 140.0'-145.0'; REC. 100%, RQD 76%
13/NQ: 145.0'-150.0'; REC. 100%, RQD 78%



BORING NO.: R-1
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 150.0-160.0
ELEVATION (ft.): 308.04
14/NQ: 150.0'-155.0'; REC. 100%, RQD 80%
15/NQ: 155.0'-160.0'; REC. 98%, RQD 82%

BORING NO.: R-1
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 160.0-170.0
ELEVATION (ft.): 298.04
16/NQ: 160.0'-165.0'; REC. 100%, RQD 74%
17/NQ: 165.0'-170.0'; REC. 86%, RQD 38%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core C Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-1

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 175+56.3, 20.9 RT	EXPLORATION ID R-2
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 458.1 (MSL) EOB: 169.0 ft.	PAGE 1 OF 3
START: 7/4/10 END: 7/5/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 76.3	COORD: 39.092067840, -84.522973760	

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.1
WATER (OHIO RIVER)	429.1
VERY LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, LITTLE SAND, WET	424.6
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	423.1
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, LITTLE SAND, TRACE SILT, TRACE CLAY, WET	421.6
LOOSE, BROWN, **FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, WET	420.1
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	413.1
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE SILT, TRACE GRAVEL, TRACE CLAY, WET	

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
1 1 2	4	33	SS-1	-	87	11	2	0	0	NP	NP	NP	13	A-1-a (0)	
3 6 12	23	67	SS-2	-	22	58	16	2	2	NP	NP	NP	17	A-1-b (0)	
17 6 4	13	33	SS-3	-	62	20	14	2	2	NP	NP	NP	15	A-1-a (0)	
4 4 4	10	100	SS-4	-	23	21	51	2	3	NP	NP	NP	18	A-3 (0)	
11 9 7	20	0	SS-5	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
4 5 5	13	100	SS-6	-	55	13	26	4	2	NP	NP	NP	15	A-1-b (0)	
3 3 5	10	0	SS-7	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
2 3 5	10	56	SS-8	-	9	18	69	1	3	NP	NP	NP	22	A-3 (0)	
11 6 6	15	33	SS-9	-	-	-	-	-	-	-	-	-	28	A-3 (V)	
6 6 6	15	44	SS-10	-	0	6	87	4	3	NP	NP	NP	25	A-3 (0)	
15 10 11	27	44	SS-11	-	-	-	-	-	-	-	-	-	27	A-3 (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 175+56.3, 20.9 RT	START: 7/4/10	END: 7/5/10	PG 2 OF 3	R-2

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE SILT, TRACE GRAVEL, TRACE CLAY, WET *(continued)*		61	6 10 10	25	33	SS-12	-	1	12	80	3	4	NP	NP	NP	20	A-3 (0)	
		66	11 11 11	28	33	SS-13	-	-	-	-	-	-	-	-	-	24	A-3 (V)	
MEDIUM DENSE, BROWN, **GRAVEL AND/OR STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	388.1	70	6 9 16	32	67	SS-14	-	25	46	23	3	3	NP	NP	NP	15	A-1-b (0)	
		76	12 12 12	31	67	SS-15	-	17	40	37	3	3	NP	NP	NP	17	A-1-b (0)	
VERY DENSE, GRAY, **STONE FRAGMENTS WITH SAND**, LIMESTONE FLOATERS/COBBLES, WET	378.1	80	65	-	0	SS-16	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		85	50/0"	-		SS-17	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, SLIGHTLY WEATHERED, VERY WEAK, VERY THIN TO THIN BEDDED, LOSS 5%, RQD 78% LS @87.5'-88' QU=13147 PSI SH @ 88.2' SDI = 67.9 SH @ 89' SDI = 82.5 LS @89.3'-89.7' QU=9634 PSI LS @90.7'-91.6' QU=12836 PSI SH @ 93.7' SDI = 93.6 SH @93.7'-94' QU= 429 PSI LS @99.8'-100.1' QU=8025 PSI SH @ 100.4' SDI = 94.1 LS @ 107.7' POINT LOAD = 5783 PSI LS @112.9'-113.9' QU=14131 PSI.	371.1	TR 87	45		100	NQ-1											CORE	
		89–94	74		100	NQ-2											CORE	
		94–99	80		86	NQ-3											CORE	
		99–104	82		94	NQ-4											CORE	
		104–109	92		96	NQ-5											CORE	
		109–114	74		98	NQ-6											CORE	
	344.1	114–119	88		100	NQ-7											CORE	
		119–	76		94	NQ-8											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 175+56.3, 20.9 RT	START: 7/4/10	END: 7/5/10	PG 3 OF 3	R-2

MATERIAL DESCRIPTION AND NOTES	ELEV. 336.2	DEPTHS	SPT/RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, ARGILLACEOUS, INTERMEDIATE SHALE SEAMS TO PARTINGS, LOSS 1%, RQD 84% LS @119.8'-120.6' QU=13926 PSI LS @ 130.7' POINT LOAD = 10575 PSI SH @ 134' SDI = 91.7 LS @139'-139.5' QU=7906 PSI LS @143.5'-144' QU=13836 PSI LS @ 148.5' POINT LOAD = 12884 PSI. *(continued)*		123–124																
		124–129	88		100	NQ-9											CORE	
		129–134	76		100	NQ-10											CORE	
		134–139	86		100	NQ-11											CORE	
		139–144	72		100	NQ-12											CORE	
	309.1	144–149	100		100	NQ-13											CORE	
LIMESTONE, GRAY, UNWEATHERED, VERY STRONG, THIN BEDDED, ARGILLACEOUS, LOSS 4%, RQD 83% LS @155.3'-155.6' QU=26538 PSI LS @ 159.5' POINT LOAD = 12962 PSI.		149–154	82		100	NQ-14											CORE	
		154–159	100		100	NQ-15											CORE	
		159–164	86		100	NQ-16											CORE	
	289.1	164–169 EOB	64		82	NQ-17											CORE	

NOTES: WATER USED BELOW 87 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (10 BAGS CEMENT/1 BAG BENTONITE)

BORING NO.: R-2
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 87.0-104.0
ELEVATION (ft.): 371.1
1/NQ: 87.0'-89.0''; REC. 100%, RQD 45%
2/NQ: 89.0'-94.0'; REC. 100%, RQD 74%
3/NQ: 94.0'-99.0'; REC. 86%, RQD 80%
4/NQ: 99.0'-104.0'; REC. 94%, RQD 82%

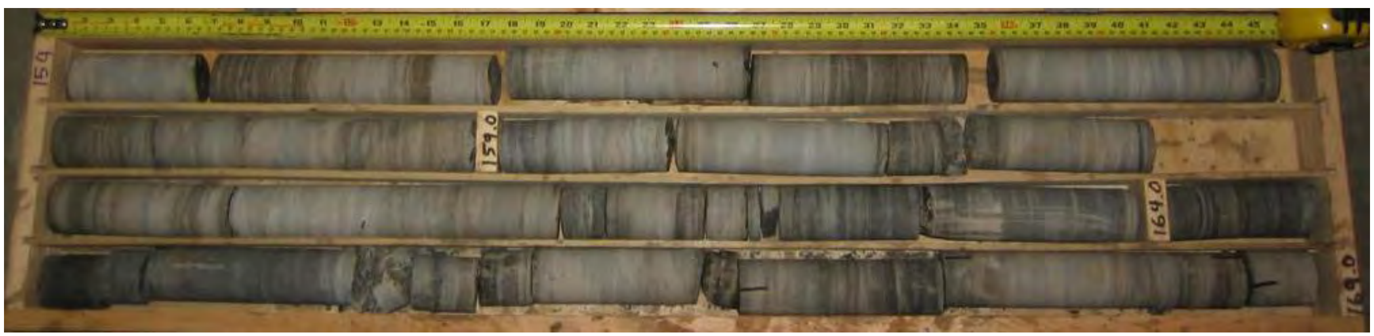
BORING NO.: R-2
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 104.0-114.0
ELEVATION (ft.): 354.1
5/NQ: 104.0'-109.0'; REC. 96%, RQD 92%
6/NQ: 109.0'-114.0'; REC. 98%, RQD 74%

BORING NO.: R-2
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 114.0-134.0
ELEVATION (ft.): 344.1
7/NQ: 114.0'-119.0'; REC. 100%, RQD 88%
8/NQ: 119.0'-124.0'; REC. 94%, RQD 76%
9/NQ: 124.0'-129.0'; REC. 94%, RQD 76%
10/NQ: 129.0'-134.0'; REC. 100%, RQD 76%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT	R-2
Chkd By: DWW	File No. Core C	611 LUNKEN PARK DRIVE	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-8-10	CINCINNATI, OHIO 45226	CINCINNATI, OHIO	

BORING NO.: R-2
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 134.0-144.0
ELEVATION (ft.): 324.1
11/NQ: 134.0'-139.0'; REC. 100%, RQD 86%
12/NQ: 139.0'-144.0'; REC. 100%, RQD 72%

BORING NO.: R-2
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 144.0-154.0
ELEVATION (ft.): 314.1
13/NQ: 144.0'-149.0'; REC. 100%, RQD 100%
14/NQ: 149.0-154.0'; REC. 100%, RQD 82%



BORING NO.: R-2
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 154.0-169.0
ELEVATION (ft.): 304.1
15/NQ: 154.0'-159.0'; REC. 100%, RQD 100%
16/NQ: 159.0'-164.0'; REC. 100%, RQD 86%
17/NQ: 164.0'-169.0'; REC. 100%, RQD 64%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core C Date: 9-8-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-2

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 17+10.4, 41.8 RT	EXPLORATION ID R-2A
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / HQ	CALIBRATION DATE: 2/4/10	ELEVATION: 457.6 (MSL) EOB: 190.5 ft.	PAGE 1 OF 4
START: 8/27/10 END: 9/2/10	SAMPLING METHOD: SPT / HQ	ENERGY RATIO (%): 76.3	COORD: 39.091939445, -84.522976190	

MATERIAL DESCRIPTION AND NOTES	ELEV. 457.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WATER (OHIO RIVER)		0 – 29																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, WET	428.6	29 – 30.5	3 4 3	9	0	SS-1	-	-	-	-	-	-	-	-	-	21	A-1-b (V)	
		30.5 – 32	4 5 9	18	33	SS-2	-	46	37	6	10	1	NP	NP	NP	18	A-1-b (0)	
		32 – 33.5	5 28 8	46	44	SS-3	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		33.5 – 35	8 6 7	17	0	SS-4	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		35 – 36.5	5 7 8	19	22	SS-5	-	-	-	-	-	-	-	-	-	16	A-1-b (V)	
		36.5 – 38	12 6 4	13	33	SS-6	-	-	-	-	-	-	-	-	-	28	A-1-b (V)	
		38 – 39.5	5 6 6	15	22	SS-7	-	-	-	-	-	-	-	-	-	27	A-1-b (V)	
		42.5 – 44	13 10 7	22	22	SS-8	-	-	-	-	-	-	-	-	-	18	A-1-b (V)	
		45 – 46.5	5 6 6	15	56	SS-9	-	32	36	27	3	2	NP	NP	NP	17	A-1-b (0)	
		47.5 – 49	15 6 9	19	0	SS-10	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		50 – 51.5	15 9 9	23	33	SS-11	-	-	-	-	-	-	-	-	-	29	A-1-b (V)	
MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY,, WET	402.6	55 – 56.5	4 5 7	15	67	SS-12	-	3	30	62	3	2	NP	NP	NP	21	A-3 (0)	

MATERIAL DESCRIPTION AND NOTES	ELEV. 397.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY,, WET *(continued)*		61	3 6 7	17	100	SS-13	-	-	-	-	-	-	-	-	-	24	A-3 (V)	
	392.6	62 – 65																
DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET		66	17 18 15	42	100	SS-14	-	60	15	17	6	2	NP	NP	NP	13	A-1-a (0)	
		71	14 15 13	36	67	SS-15	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	
		76	12 18 15	42	33	SS-16	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
	377.6	77 – 80																
VERY DENSE, BROWN AND GRAY, **GRAVEL AND STONE FRAGMENTS**, SOME COBBLES, LITTLE SAND, WET		80	50/0"		-	SS-17	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
		85	50/0"		-	SS-18	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
	369.6	TR 88																
INTERBEDDED LIMESTONE (50%) AND SHALE (50%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS; **SHALE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, VERY WEAK TO WEAK, LAMINATED TO THIN BEDDED, LOSS 2%, RQD 40%.		92	40		100	HQ-1											CORE	
		98	40		96	HQ-2											CORE	
	358.1	99																
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, SHALE PARTINGS, FOSSILIFEROUS SEAMS, LOSS 2%, RQD 75% LS @99.5'-100.1' QU=14410 PSI LS @ 105.1' POINT LOAD = 9027 PSI LS @111.8'-112.2' QU=12314 PSI LS @117.8'-118.2' QU=6058 PSI LS/SH @120.5'-121' QU= 4222 PSI LS @ 131.5' POINT LOAD = 8142 PSI LS @134.4'-134.9' QU=7566 PSI LS @140'-140.5' QU=7757 PSI LS @ 141.2' POINT LOAD = 11014 PSI LS @148'-148.5' QU=15226 PSI.		103	66		96	HQ-3											CORE	
		108	64		100	HQ-4											CORE	
		113	70		92	HQ-5											CORE	
		118	80		100	HQ-6											CORE	
		119 – 121																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+10.4, 41.8 RT	START: 8/27/10	END: 9/2/10	PG 3 OF 4	R-2A

MATERIAL DESCRIPTION AND NOTES	ELEV. 335.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, SHALE PARTINGS, FOSSILIFEROUS SEAMS, LOSS 2%, RQD 75% LS @99.5'-100.1' QU=14410 PSI LS @ 105.1' POINT LOAD = 9027 PSI LS @111.8'-112.2' QU=12314 PSI		123, 124, 125	88		100	HQ-7											CORE	
LS @117.8'-118.2' QU=6058 PSI LS/SH @120.5'-121' QU= 4222 PSI LS @ 131.5' POINT LOAD = 8142 PSI LS @134.4'-134.9' QU=7566 PSI		126, 127, 128, 129, 130	74		94	HQ-8											CORE	
LS @140'-140.5' QU=7757 PSI LS @ 141.2' POINT LOAD = 11014 PSI LS @148'-148.5' QU=15226 PSI. *(continued)*		131, 132, 133, 134, 135, 136, 137, 138, 139, 140	66		99	HQ-9											CORE	
		141, 142, 143, 144, 145, 146, 147, 148, 149, 150	89		100	HQ-10											CORE	
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN TO MEDIUM BEDDED, CRYSTALLINE, LOSS 6%, RQD 80% LS @160'-160.5' QU=10770 PSI LS @ 166.9' POINT LOAD = 9985 PSI LS @175.9'-176.3' QU=10382 PSI LS @179.8'-180.3' QU=13212 PSI LS @183.5'-184' QU=9726 PSI.	306.6	151, 152, 153, 154, 155, 156, 157, 158, 159, 160	90		95	HQ-11											CORE	
		161, 162, 163, 164, 165, 166, 167, 168, 169, 170	67		90	HQ-12											CORE	
		171, 172, 173, 174, 175, 176, 177, 178, 179, 180	80		98	HQ-13											CORE	
		181, 182, 183																

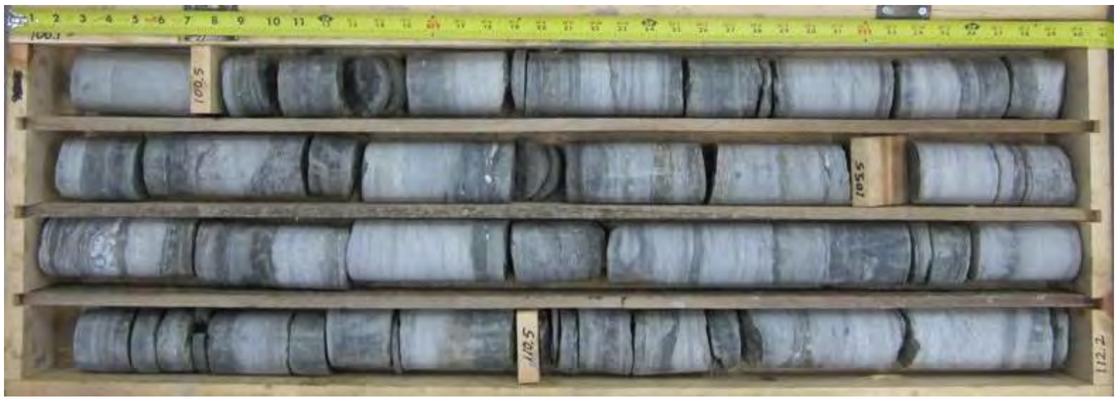
PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+10.4, 41.8 RT	START: 8/27/10	END: 9/2/10	PG 4 OF 4	R-2A

MATERIAL DESCRIPTION AND NOTES	ELEV. 273.9	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN TO MEDIUM BEDDED, CRYSTALLINE, LOSS 6%, RQD 80% LS @160'-160.5' QU=10770 PSI LS @ 166.9' POINT LOAD = 9985 PSI LS @175.9'-176.3' QU=10382 PSI LS @179.8'-180.3' QU=13212 PSI LS @183.5'-184' QU=9726 PSI. *(continued)*	267.1	184–190 EOB	84		94	HQ-14											CORE	

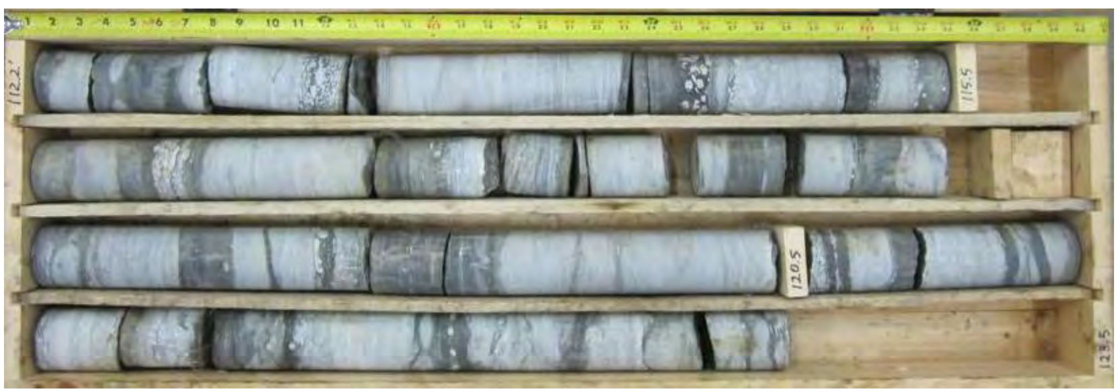
NOTES: WATER USED BELOW 88 FT. FOR ROCK CORING PURPOSES. 3 INCH PVC CASING INSTALLED FROM SURFACE TO 139 FEET. UNABLE TO INSTALL CASING TO FULL DEPTH DUE TO OBSTRUCT

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (15 BAGS CEMENT/2.5 BAGS BENTONITE)

BORING NO.: R-2A
CORE BOX NO.: 1 OF 9
DEPTH (ft.): 88.0-100.1
ELEVATION (ft.): 369.64
1/NQ: 88.0'-95.5'; REC. 100%, RQD 40%
2/NQ: 95.5'-100.5'; REC. 96%, RQD 40%
3/NQ: 100.5'-105.5'; REC. 96%, RQD 66%

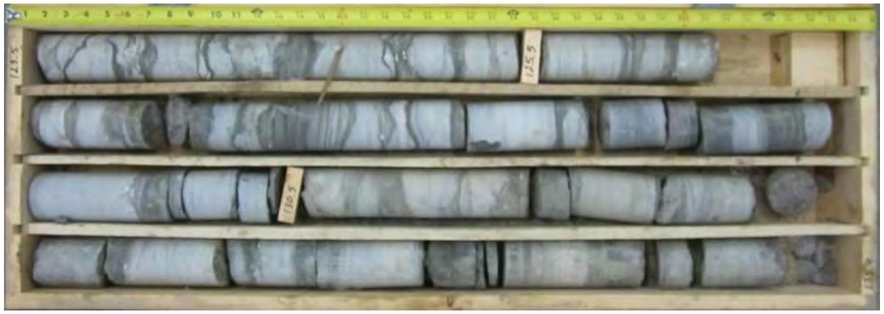


BORING NO.: R-2A
CORE BOX NO.: 2 OF 9
DEPTH (ft.): 100.1-112.2
ELEVATION (ft.): 357.54
4/NQ: 105.5'110.5'; REC. 100%, RQD 64%
5/NQ: 110.5'-115.5'; REC. 92%, RQD 70%

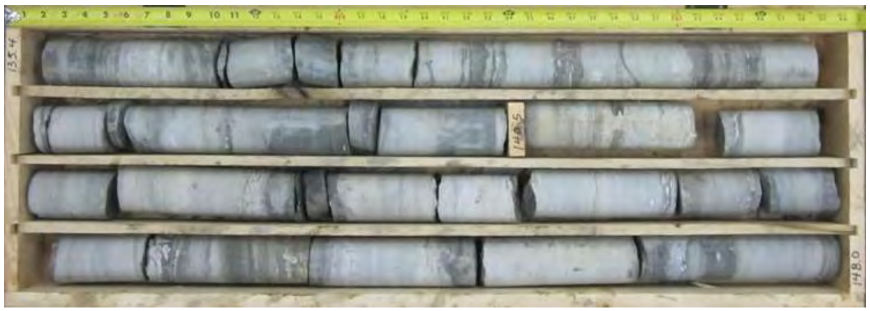


BORING NO.: R-2A
CORE BOX NO.: 3 OF 9
DEPTH (ft.): 112.2-123.5
ELEVATION (ft.): 345.44
6/NQ: 115.5'-120.5'; REC. 100%, RQD 80%
7/NQ: 120.5'-125.5'; REC. 100%, RQD 88%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-2A
Chkd By: DWW	File No. Core C	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-8-10		CINCINNATI, OHIO	



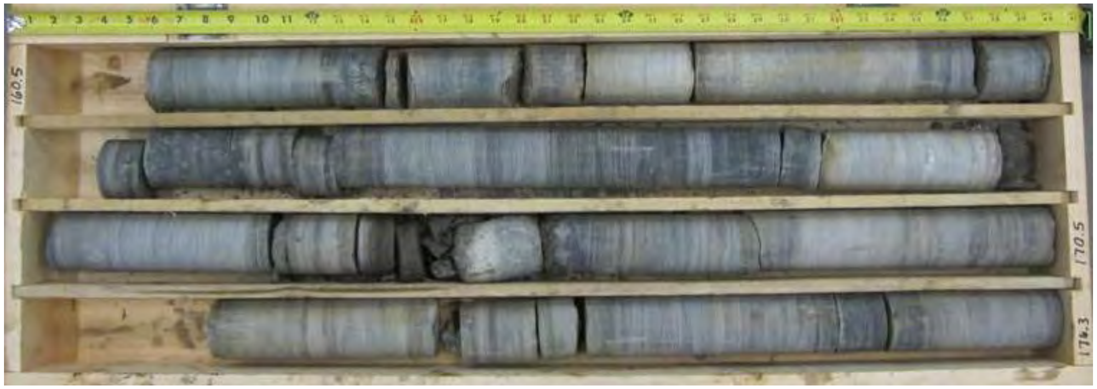
BORING NO.: R-2A
CORE BOX NO.: 4 OF 9
DEPTH (ft.): 123.5-135.4
ELEVATION (ft.): 334.14
8/NQ: 125.5'-130.5'; REC. 94%, RQD 74%
9/NQ: 130.5'-140.5'; REC. 99%, RQD 66%



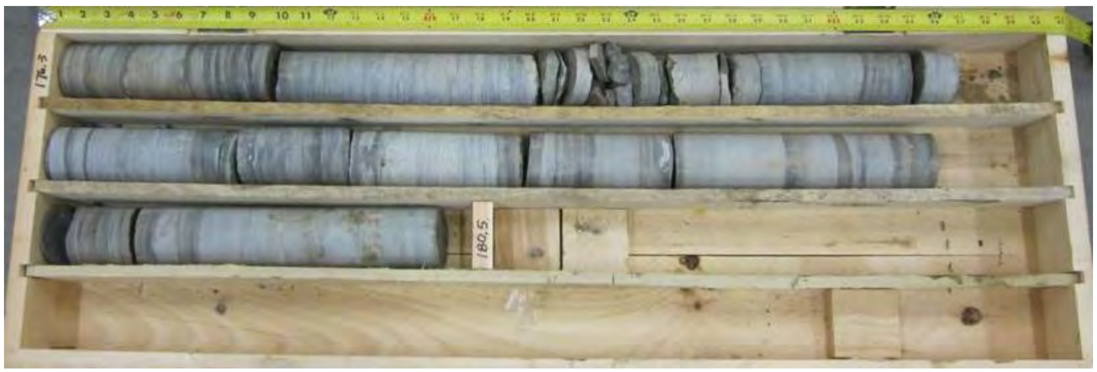
BORING NO.: R-2A
CORE BOX NO.: 5 OF 9
DEPTH (ft.): 135.4-148.0
ELEVATION (ft.): 321.24
10/NQ: 140.5'-150.5'; REC. 100%, RQD 89%

BORING NO.: R-2A
CORE BOX NO.: 6 OF 9
DEPTH (ft.): 148.0-160.5
ELEVATION (ft.): 308.64
11/NQ: 10.5'-160.5'; REC. 95%, RQD 90%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core C Date: 9-8-10	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-2A



BORING NO.: R-2A
CORE BOX NO.: 7 OF 9
DEPTH (ft.): 160.5-176.3
ELEVATION (ft.): 296.14
12/NQ: 160.5'-170.5'; REC. 90%, RQD 67%



BORING NO.: R-2A
CORE BOX NO.: 8 OF 9
DEPTH (ft.): 176.3-180.5
ELEVATION (ft.): 280.34
13/NQ: 170.5'-180.5'; REC. 98%, RQD 80%

BORING NO.: R-2A
CORE BOX NO.: 9 OF 9
DEPTH (ft.): 180.5-190.5
ELEVATION (ft.): 276.14
14/NQ: 180.5'-190.5'; REC. 94%, RQD 84%

Project Mngr.: AJM	PN. N1105070	A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-2A
Chkd By: DWW	File No. Core C	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-8-10		CINCINNATI, OHIO	

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 17+79.1, 35.6 LT	EXPLORATION ID R-3
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB	
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 458.0 (MSL) EOB: 165.5 ft.	PAGE 1 OF 3
START: 7/12/10 END: 7/13/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 76.3	COORD: 39.092137310, -84.523169520	

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WATER (OHIO RIVER)																		
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	430.0	28																
		32	1 2 2	5	0	SS-1	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		33	6 5 6	14	0	SS-2	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		35	3 6 9	19	67	SS-3	-	39	37	20	3	1	NP	NP	NP	15	A-1-b (0)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, TRACE SILT, TRACE CLAY, WET	422.5	36	5 3 2	6	22	SS-4	-	57	34	6	1	2	NP	NP	NP	17	A-1-a (0)	
LOOSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	421.0	38	2 2 2	5	67	SS-5	-	7	69	17	3	4	NP	NP	NP	18	A-1-b (0)	
		39	7 4 4	10	0	SS-6	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET	418.0	41	3 5 5	13	33	SS-7	-	1	39	55	2	3	NP	NP	NP	19	A-3 (0)	
		43	5 7 8	19	33	SS-8	-	-	-	-	-	-	-	-	-	25	A-3 (V)	
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	413.0	46	4 4 4	10	78	SS-9	-	9	48	39	2	2	NP	NP	NP	20	A-1-b (0)	
		48	4 5 4	11	67	SS-10	-	25	36	34	2	3	NP	NP	NP	19	A-1-b (0)	
		51	10 6 7	17	0	SS-11	-	-	-	-	-	-	-	-	-	-	A-1-b (V)	
		56	6 7 8	19	67	SS-12	-	-	-	-	-	-	-	-	-	22	A-1-b (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+79.1, 35.6 LT	START: 7/12/10	END: 7/13/10	PG 2 OF 3	R-3

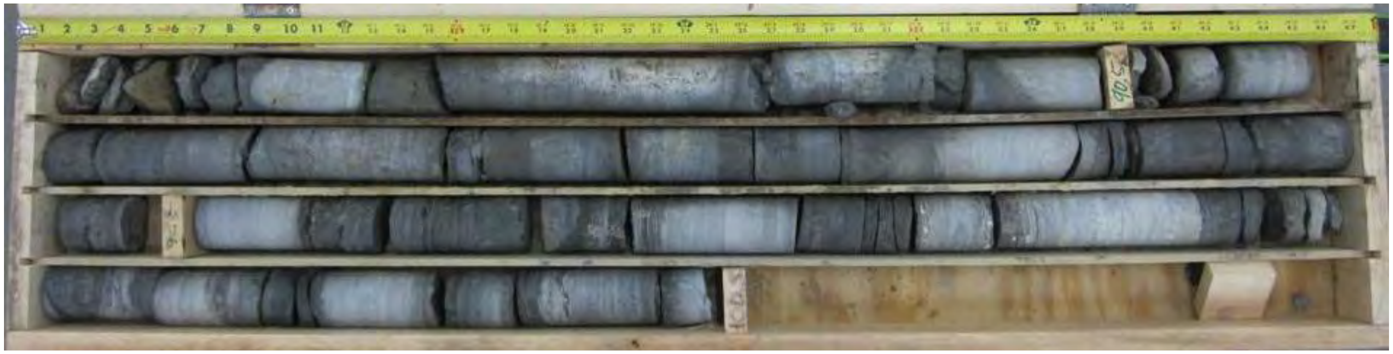
MATERIAL DESCRIPTION AND NOTES	ELEV. 398.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET *(continued)*		61	8 / 9 / 10	24	89	SS-13	-	31	35	29	2	3	NP	NP	NP	19	A-1-b (0)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET	393.0	65–66	20 / 12 / 12	31	100	SS-14	-	70	13	13	3	1	NP	NP	NP	10	A-1-a (0)	
		70–71	15 / 9 / 6	19	67	SS-15	-	57	29	9	3	2	NP	NP	NP	16	A-1-a (0)	
		75–76	22 / 14 / 12	33	56	SS-16	-	53	29	12	3	3	NP	NP	NP	13	A-1-a (0)	
		80	50/0"	-		SS-17	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
LIMESTONE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, MODERATELY FRACTURED, THIN SHALE SEAMS, LOSS 4%, RQD 75% LS @92.3'-92.7' QU=9244 PSI LS @93.8'-94.5' QU=10241 PSI LS @ 98' POINT LOAD = 13271 PSI SH @102.7'-103.1' QU=7236 PSI LS @106.5'-107.1' QU=9187 PSI LS @ 113.3' POINT LOAD = 13042 PSI LS @ 117.2' POINT LOAD = 10568 PSI LS @123.8'-124.7' QU=6833 PSI.	371.5	86 (TR) – 90	50		80	NQ-1											CORE	
		90–95	42		92	NQ-2											CORE	
		95–100	62		100	NQ-3											CORE	
		100–105	80		100	NQ-4											CORE	
		105–110	90		96	NQ-5											CORE	
		110–115	78		100	NQ-6											CORE	
		115–120	96		100	NQ-7											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+79.1, 35.6 LT	START: 7/12/10	END: 7/13/10	PG 3 OF 3	R-3

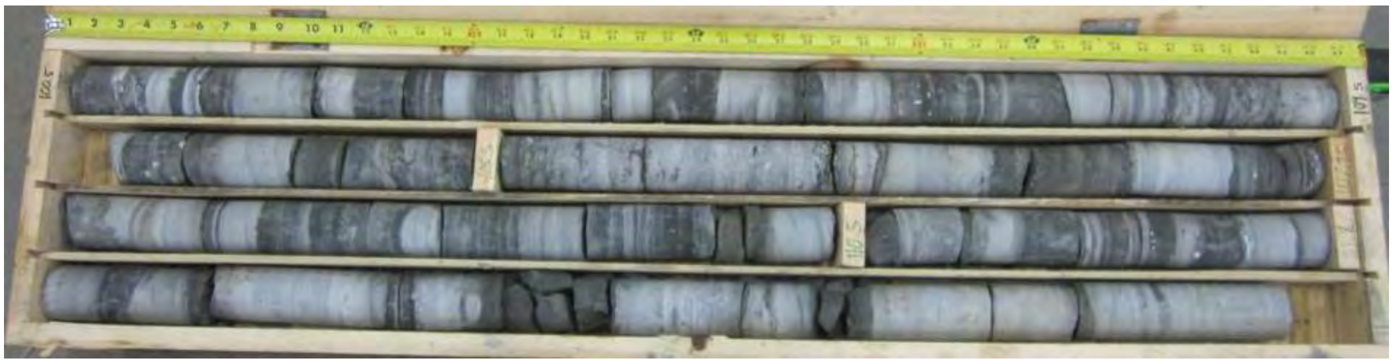
MATERIAL DESCRIPTION AND NOTES	ELEV. 336.1	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
		123	100		100	NQ-8											CORE	
	332.5	125																
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, VERY THIN SHALE PARTINGS, LOSS 2%, RQD 83% LS @106'-106.5' QU=14729 PSI LS @136.5'-137' QU=24544 PSI LS @140'-140.5' QU=9100 PSI LS @145.5'-146' QU=11767 PSI LS @157.3'-158' QU=14226 PSI.		128	92		100	NQ-9											CORE	
		133	84		80	NQ-10											CORE	
		138	90		90	NQ-11											CORE	
		143	86		90	NQ-12											CORE	
		148	80		100	NQ-13											CORE	
		153	68		100	NQ-14											CORE	
		158	84		100	NQ-15											CORE	
		163	92		100	NQ-16											CORE	
	292.5	EOB 165																

NOTES: WATER USED BELOW 86.5 FT. FOR ROCK CORING PURPOSES.

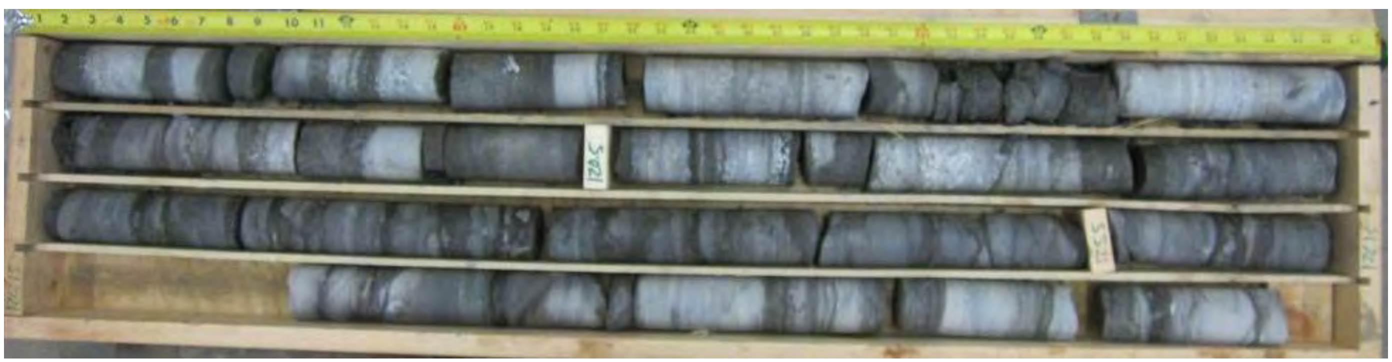
ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (10 BAGS CEMENT/1 BAG BENTONITE)



BORING NO.: R-3
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 86.5-100.5
ELEVATION (ft.): 371.51
1/NQ: 86.5’-90.5‘; REC. 80%, RQD 50%
2/NQ: 90.5-95.5‘; REC. 92%, RQD 42%
3/NQ: 95.5-100.5‘; REC. 100%, RQD 62%



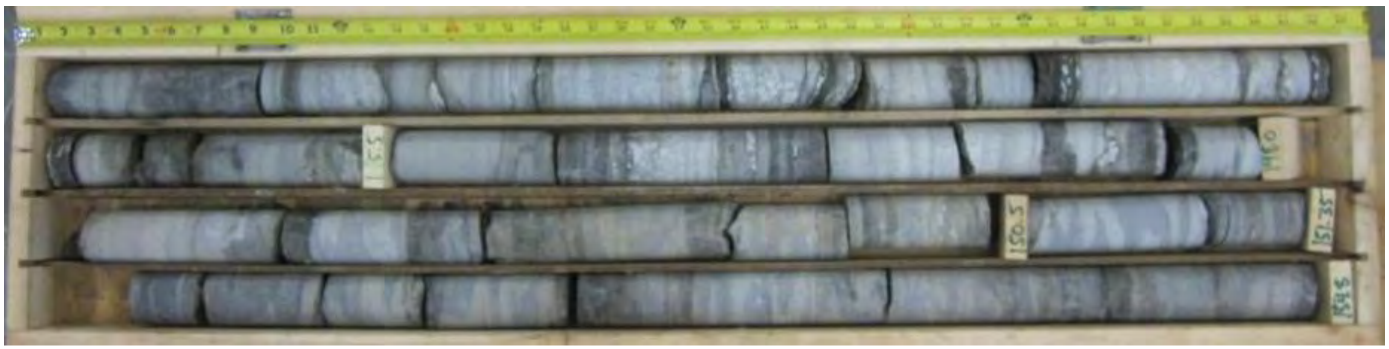
BORING NO.: R-3
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 100.5-115.5
ELEVATION (ft.): 357.51
4/NQ: 100.5’-105.5‘; REC. 100%, RQD 80%
5/NQ: 105.5’-110.5‘; REC. 96%, RQD 90%
6/NQ: 110.5’-115.5‘; REC. 100%, RQD 78%



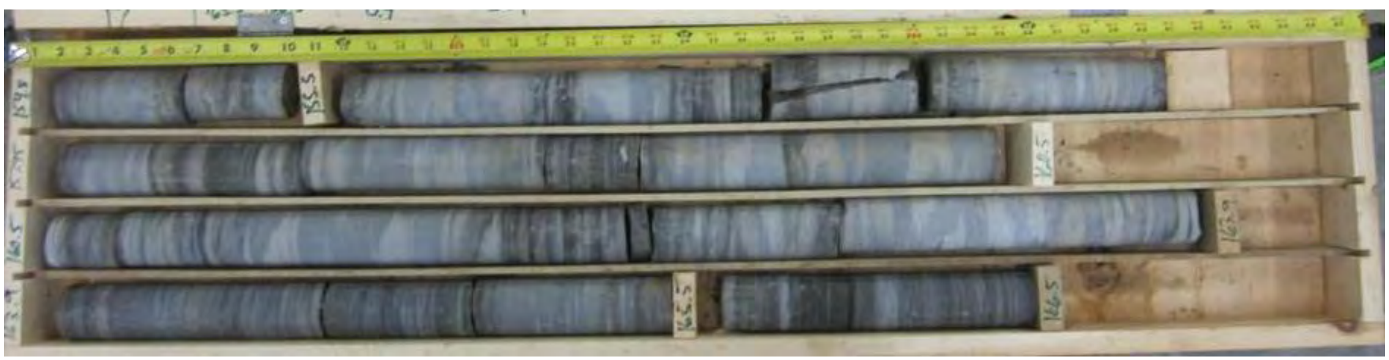
BORING NO.: R-3
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 115.5-130.5
ELEVATION (ft.): 342.51
7/NQ: 115.5’-120.5‘; REC. 100%, RQD 96%
8/NQ: 120.5’-125.5‘; REC. 100%, RQD 100%
9/NQ: 125.5’-130.5‘; REC. 100%, RQD 92%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	R-3
Chkd By: DWW	File No. Core D	611 LUNKEN PARK DRIVE		
Approved By: AJM	Date: 9-23-10	CINCINNATI, OHIO 45226		

BORING NO.: R-3
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 130.5-140.5
ELEVATION (ft.): 327.51
10/NQ: 130.5'-135.5'; REC. 80%, RQD 84%
11/NQ: 135.5'-140.5'; REC. 90%, RQD 90%

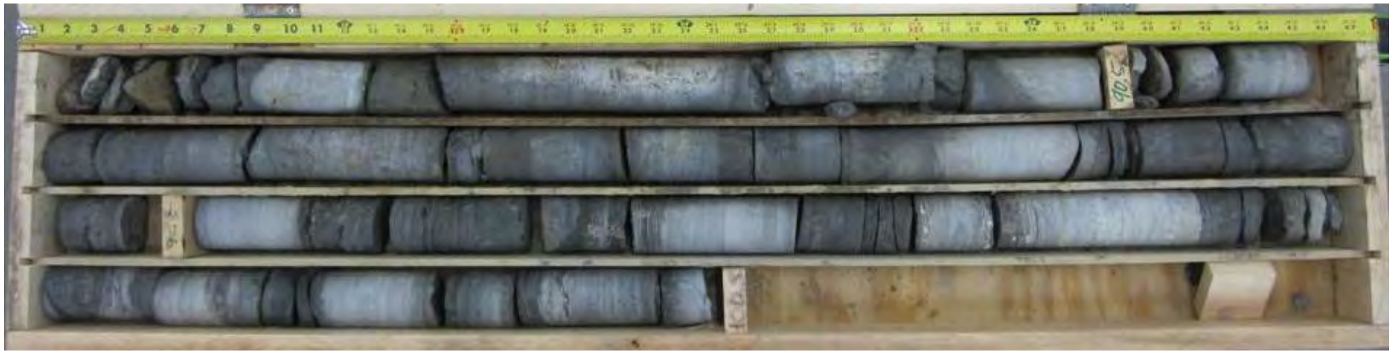


BORING NO.: R-3
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 140.5-154.8
ELEVATION (ft.): 317.51
12/NQ: 140.5'-145.5'; REC. 90%, RQD 86%
13/NQ: 145.5'-150.5'; REC. 100%, RQD 80%
14/NQ: 150.5'-155.5'; REC. 100%, RQD 68%

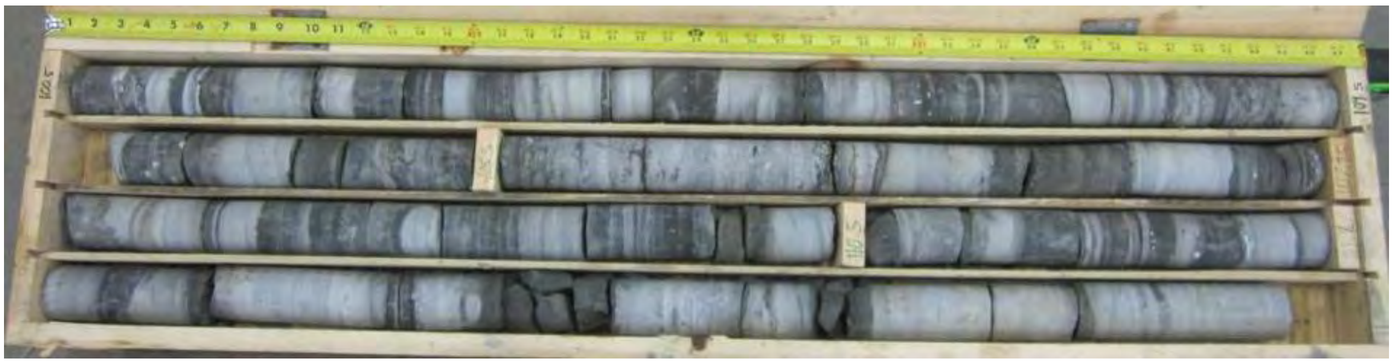


BORING NO.: R-3
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 154.8-166.5
ELEVATION (ft.): 302.51
15/NQ: 155.5'-160.5'; REC. 100%, RQD 84%
16/NQ: 160.5'-165.5'; REC. 100%, RQD 92%
17/NQ: 165.5'-166.5'; REC. 90%, RQD 90%

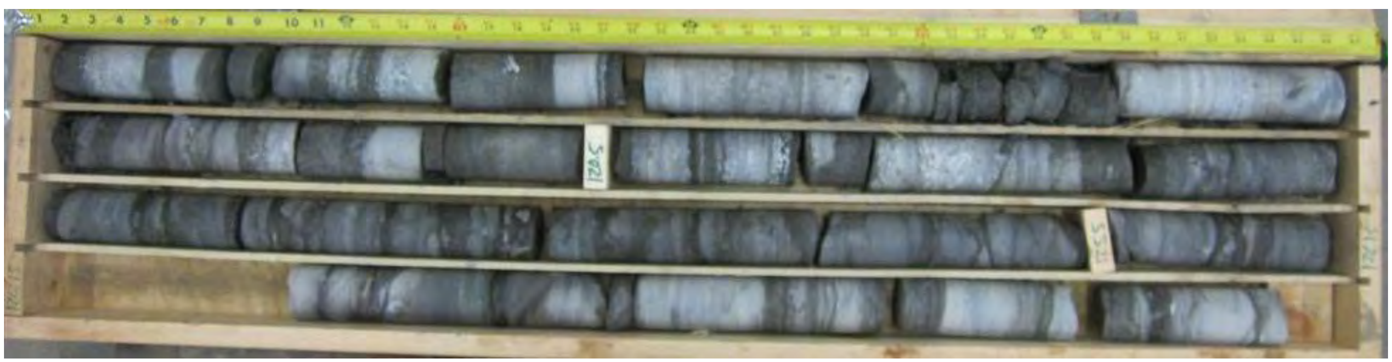
Project Mngr.: AJM	PN. N1105070		ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	HCN A Terracon COMPANY	BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	R-3
Chkd By: DWW	File No. Core D	611 LUNKEN PARK DRIVE		
Approved By: AJM	Date: 9-23-10	CINCINNATI, OHIO 45226		



BORING NO.: R-3
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 86.5-100.5
ELEVATION (ft.): 371.51
1/NQ: 86.5’-90.5‘; REC. 80%, RQD 50%
2/NQ: 90.5-95.5‘; REC. 92%, RQD 42%
3/NQ: 95.5-100.5‘; REC. 100%, RQD 62%



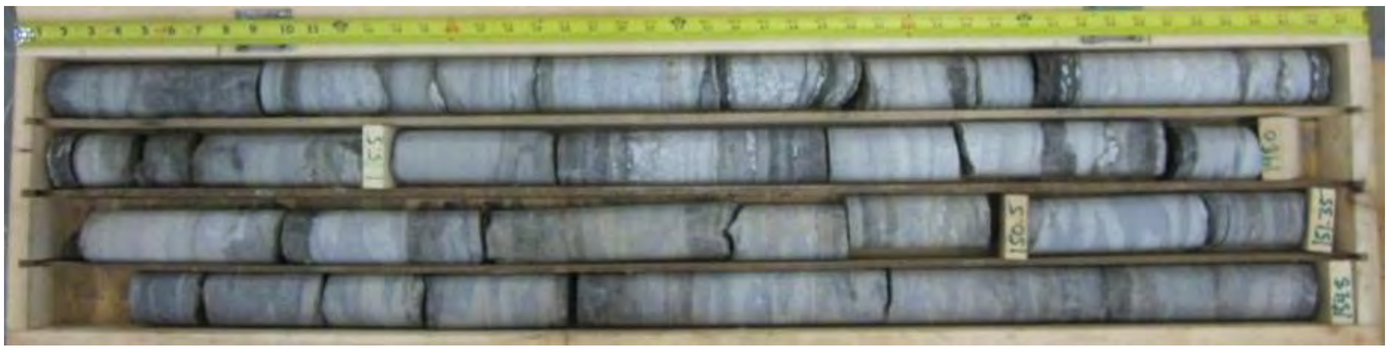
BORING NO.: R-3
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 100.5-115.5
ELEVATION (ft.): 357.51
4/NQ: 100.5’-105.5‘; REC. 100%, RQD 80%
5/NQ: 105.5’-110.5‘; REC. 96%, RQD 90%
6/NQ: 110.5’-115.5‘; REC. 100%, RQD 78%



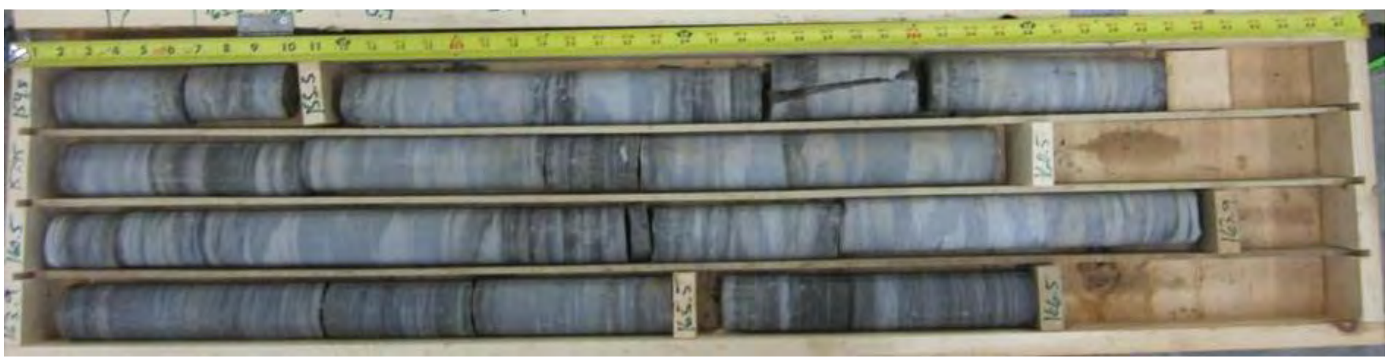
BORING NO.: R-3
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 115.5-130.5
ELEVATION (ft.): 342.51
7/NQ: 115.5’-120.5‘; REC. 100%, RQD 96%
8/NQ: 120.5’-125.5‘; REC. 100%, RQD 100%
9/NQ: 125.5’-130.5‘; REC. 100%, RQD 92%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core D Date: 9-23-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-3

BORING NO.: R-3
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 130.5-140.5
ELEVATION (ft.): 327.51
10/NQ: 130.5'-135.5'; REC. 80%, RQD 84%
11/NQ: 135.5'-140.5'; REC. 90%, RQD 90%



BORING NO.: R-3
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 140.5-154.8
ELEVATION (ft.): 317.51
12/NQ: 140.5'-145.5'; REC. 90%, RQD 86%
13/NQ: 145.5'-150.5'; REC. 100%, RQD 80%
14/NQ: 150.5'-155.5'; REC. 100%, RQD 68%



BORING NO.: R-3
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 154.8-166.5
ELEVATION (ft.): 302.51
15/NQ: 155.5'-160.5'; REC. 100%, RQD 84%
16/NQ: 160.5'-165.5'; REC. 100%, RQD 92%
17/NQ: 165.5'-166.5'; REC. 90%, RQD 90%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-3
Chkd By: DWW	File No. Core D	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-23-10		CINCINNATI, OHIO	

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH	DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 17+22.4, 36.4 LT
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DWW	HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ	CALIBRATION DATE: 2/4/10	ELEVATION: 458.0 (MSL) EOB: 165.5 ft.
START: 7/7/10 END: 7/9/10	SAMPLING METHOD: SPT / NQ	ENERGY RATIO (%): 76.3	COORD: 39.091981720, -84.523181010

EXPLORATION ID R-4

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.0
WATER (OHIO RIVER)	427.5
LOOSE TO MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	413.0
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET	

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)
1 2 2	5	44	SS-1	-	17	68	10	3	2	NP	NP	NP	17	A-1-b (0)
1 2 5	9	56	SS-2	-	37	52	10	0	1	NP	NP	NP	19	A-1-b (0)
7 7 8	19	100	SS-3	-	28	60	9	2	1	NP	NP	NP	14	A-1-b (0)
4 5 5	13	33	SS-4	-	2	85	11	0	2	NP	NP	NP	21	A-1-b (0)
6 2 3	6	67	SS-5	-	39	42	14	3	2	NP	NP	NP	17	A-1-b (0)
2 3 5	10	33	SS-6	-	10	55	28	4	3	NP	NP	NP	25	A-1-b (0)
3 2 3	6	100	SS-7	-	-	-	-	-	-	-	-	-	25	A-1-b (V)
1 2 3	6	33	SS-8	-	5	18	74	1	2	NP	NP	NP	22	A-3 (0)
2 2 3	6	0	SS-9	-	-	-	-	-	-	-	-	-	-	A-3 (V)
4 4 6	13	67	SS-10	-	2	22	72	1	3	NP	NP	NP	26	A-3 (0)
8 9 10	24	67	SS-11	-	3	7	82	4	4	NP	NP	NP	22	A-3 (0)

DEPTHS: 1–59

F.S.

HOLE SEALED

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+22.4, 36.4 LT	START: 7/7/10	END: 7/9/10	PG 2 OF 3	R-4

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.0	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LOOSE TO MEDIUM DENSE, BROWN, **FINE SAND**, TRACE GRAVEL, TRACE SILT, TRACE CLAY, WET *(continued)*		61	5 7 8	19	67	SS-12	-	-	-	-	-	-	-	-	-	24	A-3 (V)	
	393.0	65																
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET		66	8 12 12	31	56	SS-13	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	
		71	14 21 13	43	22	SS-14	-	78	13	6	1	2	NP	NP	NP	16	A-1-a (0)	
		76	14 6 6	15	33	SS-15	-	52	36	7	3	2	NP	NP	NP	16	A-1-a (0)	
		80	50/3"	-	0	SS-16	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
	371.5	86 (TR)																
LIMESTONE, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, MODERATELY STRONG TO STRONG, THIN BEDDED, ARGILLACEOUS, SHALE SEAMS AND PARTINGS, CRYSTALLINE, FRACTURED, LOSS 8%, RQD 40% LS @ 90.5'-91' QU=8320 PSI LS @ 95' POINT LOAD = 11920 PSI LS @ 95.5'-96' QU=5778 PSI.		87-91	0		73	NQ-1											CORE	
		91-96	56		100	NQ-2											CORE	
		96-100	54		100	NQ-3											CORE	
	357.5	100																
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, MODERATELY FRACTURED, LOSS 0%, RQD 88% LS @ 101' POINT LOAD = 13271 PSI LS @102.8'-103.3' QU=2644 PSI LS @111.3'-111.9' QU=5958 PSI LS @120.6'-121.3' QU=19133 PSI LS @121.9'-122.3' QU=15389 PSI LS @129.6'-130' QU=5754 PSI LS @139.6'-140.5' QU=16884 PSI LS @140.6'-141.1' QU=13586 PSI LS @ 147' POINT LOAD = 12473 PSI LS @152.8'-153.6' QU=10653 PSI LS @ 155.5' POINT LOAD = 13035 PSI LS @159.6'-160.5' QU=15762 PSI.		101-106	84		100	NQ-4											CORE	
		106-111	92		100	NQ-5											CORE	
		111-116	82		100	NQ-6											CORE	
		116-121	94		100	NQ-7											CORE	

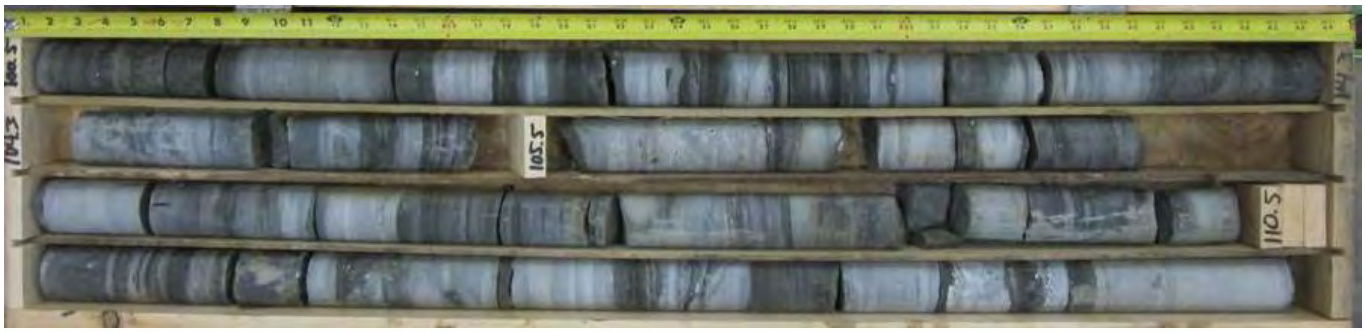
PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 17+22.4, 36.4 LT	START: 7/7/10	END: 7/9/10	PG 3 OF 3	R-4

MATERIAL DESCRIPTION AND NOTES	ELEV.	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, MODERATELY FRACTURED, LOSS 0%, RQD 88% LS @ 101' POINT LOAD = 13271 PSI LS @102.8'-103.3' QU=2644 PSI LS @111.3'-111.9' QU=5958 PSI LS @120.6'-121.3' QU=19133 PSI LS @121.9'-122.3' QU=15389 PSI LS @129.6'-130' QU=5754 PSI LS @139.6'-140.5' QU=16884 PSI LS @140.6'-141.1' QU=13586 PSI LS @ 147' POINT LOAD = 12473 PSI LS @152.8'-153.6' QU=10653 PSI LS @ 155.5' POINT LOAD = 13035 PSI LS @159.6'-160.5' QU=15762 PSI. *(continued)*	336.1																	
		123	96		100	NQ-8											CORE	
		128	82		100	NQ-9											CORE	
		133	72		100	NQ-10											CORE	
		138	94		100	NQ-11											CORE	
		143	90		100	NQ-12											CORE	
		148	86		100	NQ-13											CORE	
		153	84		100	NQ-14											CORE	
		158	96		100	NQ-15											CORE	
		163	94		98	NQ-16											CORE	
	292.5	165 EOB																

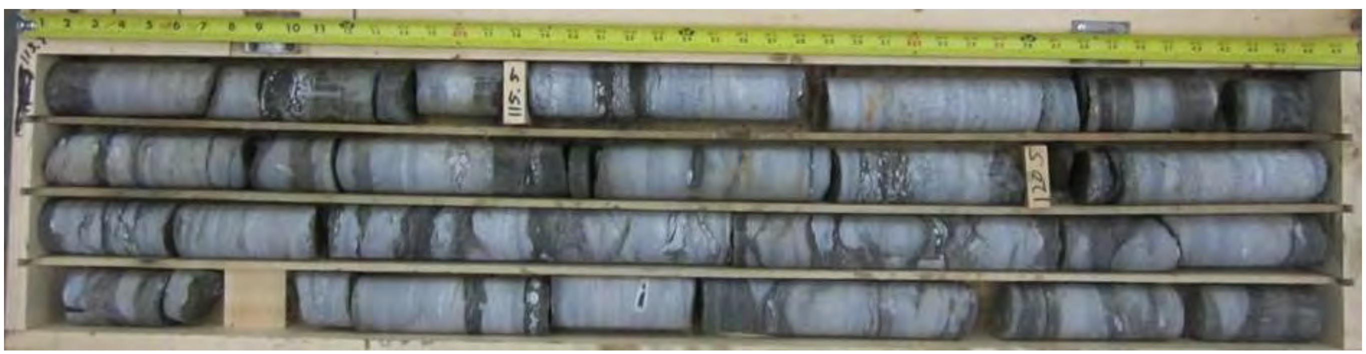
NOTES: WATER USED BELOW 86.5 FT. FOR ROCK CORING PURPOSES. GAS POCKET AT 147.0'

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (9 BAGS CEMENT/1 BAG BENTONITE)

BORING NO.: R-4
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 86.5-100.5
ELEVATION (ft.): 371.48
1/NQ: 86.5'-90.5'; REC. 73%, RQD 0%
2/NQ: 90.5'-95.5'; REC. 100%, RQD 56%
3/NQ: 95.5'-100.5'; REC. 100%, RQD 54%

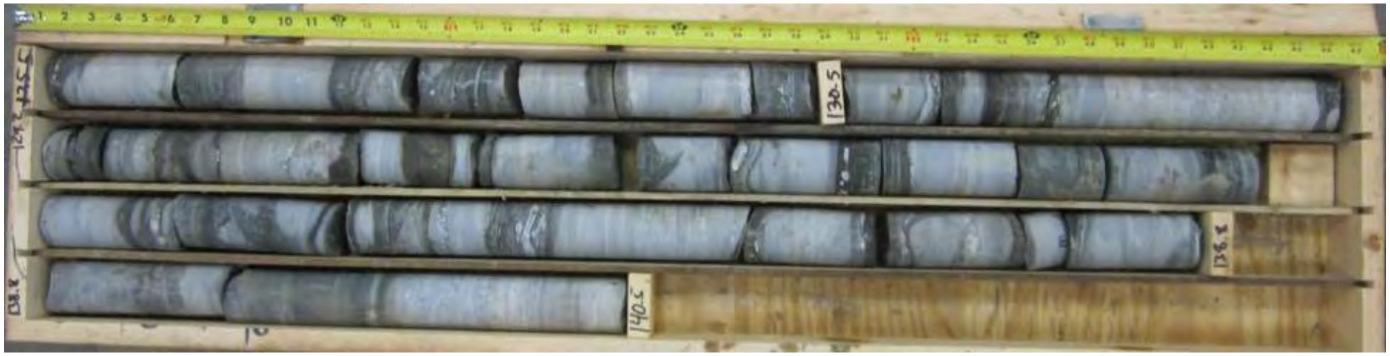


BORING NO.: R-4
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 100.5-115.5
ELEVATION (ft.): 357.48
4/NQ: 100.5-105.5'; REC. 100%, RQD 84%
5/NQ: 105.5'-110.5'; REC. 100%, RQD 92%
6/NQ: 110.5-115.5'; REC. 100%, RQD 82%

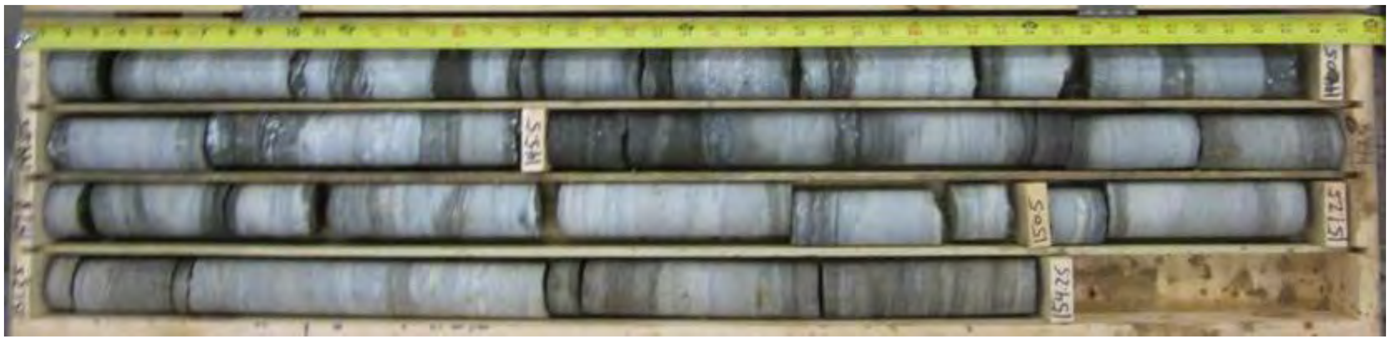


BORING NO.: R-4
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 115.5-130.5
ELEVATION (ft.): 342.48
7/NQ: 115.5'-120.5'; REC. 100%, RQD 94%
8/NQ: 120.5'-125.5'; REC. 100%, RQD 96%
9/NQ: 125.5'-130.5'; REC. 100%, RQD 82%

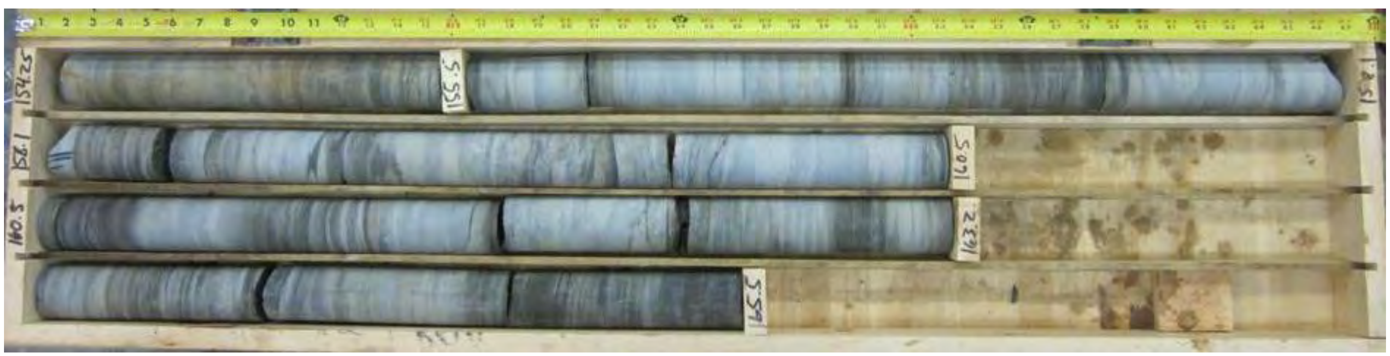
Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core D Date: 9-23-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-4



BORING NO.: R-4
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 130.5-140.5
ELEVATION (ft.): 327.48
10/NQ: 130.5'-135.5'; REC. 100%, RQD 72%
11/NQ: 135.5'140.5'; REC. 100%, RQD 94%



BORING NO.: R-4
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 140.5-155.5
ELEVATION (ft.): 317.48
12/NQ: 140.5'-145.5'; REC. 100%, RQD 90%
13/NQ: 145.5'-150.5'; REC. 100%, RQD 86%
14/NQ: 150.5'-155.5'; REC. 100%, RQD 84%



BORING NO.: R-4
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 155.5-165.5
ELEVATION (ft.): 302.48
15/NQ: 155.5'-160.5'; REC. 100%, RQD 96%
16/NQ:160.5'-165.5 '; REC. 98%, RQD 94%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	R-4
Chkd By: DWW	File No. Core D	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226		
Approved By: AJM	Date: 9-23-10			

PROJECT: BRENT SPENCE BRIDGE TYPE: BRIDGE REPLACEMENT PID: 75119 BR ID: START: 6/29/10 END: 7/1/10	DRILLING FIRM / OPERATOR: HCN / HH SAMPLING FIRM / LOGGER: HCN / DRK/DWW DRILLING METHOD: 3.25" HSA / NQ SAMPLING METHOD: SPT / NQ	DRILL RIG: CME 550X ATV-7253 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/4/10 ENERGY RATIO (%): 76.3	STATION / OFFSET: 7+85.4, 58.4 RT ALIGNMENT: PROPOSED BSB ELEVATION: 458.6 (MSL) EOB: 165.4 ft. COORD: 39.089400310, -84.522990520	EXPLORATION ID R-5 PAGE 1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.6
WATER (OHIO RIVER)	442.6
LOOSE TO MEDIUM DENSE, DARK GRAY, **COARSE AND FINE SAND**, LITTLE TO SOME GRAVEL, TRACE SILT, TRACE CLAY, WET	439.6
MEDIUM DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	438.1
SOFT TO MEDIUM STIFF, GRAY, **SILT AND CLAY**, TRACE TO SOME SAND, MOIST	431.1
MEDIUM DENSE TO DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
1 3 2	6	22	SS-1	-	19	16	49	10	6	NP	NP	NP	28	A-3a (0)	
2 4 8	15	44	SS-2	-	30	18	41	7	4	NP	NP	NP	39	A-3a (0)	
3 4 5	11	33	SS-3	-	35	19	36	6	4	NP	NP	NP	40	A-1-b (0)	
2 2 2	5	67	SS-4	1.00	0	1	24	52	23	31	20	11	30	A-6a (8)	
4 1 1	3	0	SS-5	0.75	-	-	-	-	-	-	-	-	-	A-6a (V)	
3 2 2	5	100	SS-6	1.00	0	0	9	54	37	36	21	15	44	A-6a (10)	
2 4 3	9	33	SS-7	1.00	-	-	-	-	-	-	-	-	49	A-6a (V)	
12 13 15	36	33	SS-8	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
6 11 11	28	67	SS-9	-	45	38	10	4	3	NP	NP	NP	10	A-1-b (0)	
10 14 14	36	100	SS-10	-	-	-	-	-	-	-	-	-	7	A-1-b (V)	
5 9 9	23	11	SS-11	-	-	-	-	-	-	-	-	-	21	A-1-b (V)	
21 14 7	27	33	SS-12	-	47	30	13	7	3	NP	NP	NP	14	A-1-b (0)	
13 14 14	36	100	SS-13	-	-	-	-	-	-	-	-	-	23	A-1-b (V)	
13 8 8	20	44	SS-14	-	44	39	10	5	2	NP	NP	NP	16	A-1-b (0)	
20 16 12	36	44	SS-15	-	-	-	-	-	-	NP	NP	NP	18	A-1-b (V)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 7+85.4, 58.4 RT	START: 6/29/10	END: 7/1/10	PG 2 OF 3	R-5

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GRADATION (%) GR	CS	FS	SI	CL	ATTERBERG LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM DENSE TO DENSE, GRAY, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET *(continued)*		60.5–62	11 6 6	15	67	SS-16	-	-	-	-	-	-	-	-	-	13	A-1-b (V)	
	393.6	62–65																
VERY DENSE, GRAY, **FINE SAND**, LITTLE GRAVEL, TRACE SILT, TRACE CLAY, MEDIUM DENSE AT 65', WET		65–66.5	10 11 11	28	56	SS-17	-	12	21	59	3	5	NP	NP	NP	19	A-3 (0)	
		66.5–70																
		70–71.5	20 40 51	116	67	SS-18	-	-	-	-	-	-	-	-	-	7	A-3 (V)	
	383.6	71.5–75																
VERY DENSE, GRAY, **STONE FRAGMENTS WITH SAND**, LITTLE SILT, TRACE CLAY, LIMESTONE FLOATERS, WET		75	65	-	100	SS-19	-	49	20	11	13	7	NP	NP	NP	16	A-1-b (0)	
	373.6	75–85																
INTERBEDDED LIMESTONE (70%) AND SHALE (30%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, LOSS 3%, RQD 67% LS @85.2'-85.7' QU=7099 PSI LS @86.4'-86.8' QU=10809 PSI		TR 85–90	25		88	NQ-1											CORE	
LS @90.1'-90.8' QU=7024 PSI LS @92.2'-92.8' QU=118 PSI SH @ 92.2' SDI = 57.9 LS @93'-93.8' QU=14324 PSI		90–95	66		100	NQ-2											CORE	
LS @95'-95.3' QU=8193 PSI SH @ 95.7' SDI = 52.5		95–100	20		100	NQ-3											CORE	
LS @ 100.8' POINT LOAD = 11011 PSI LS @103'-103.5' QU=4812 PSI LS @103.5'-104' QU=14991 PSI		100–105	58		100	NQ-4											CORE	
LS @ 108' POINT LOAD = 16192 PSI		105–110	84		96	NQ-5											CORE	
		110–115	98		100	NQ-6											CORE	
LS @ 118.2' POINT LOAD = 11057 PSI		115–120	76		94	NQ-7											CORE	
LS @128.1'-128.8' QU=19640 PSI.		120–122																

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 7+85.4, 58.4 RT	START: 6/29/10	END: 7/1/10	PG 3 OF 3	R-5

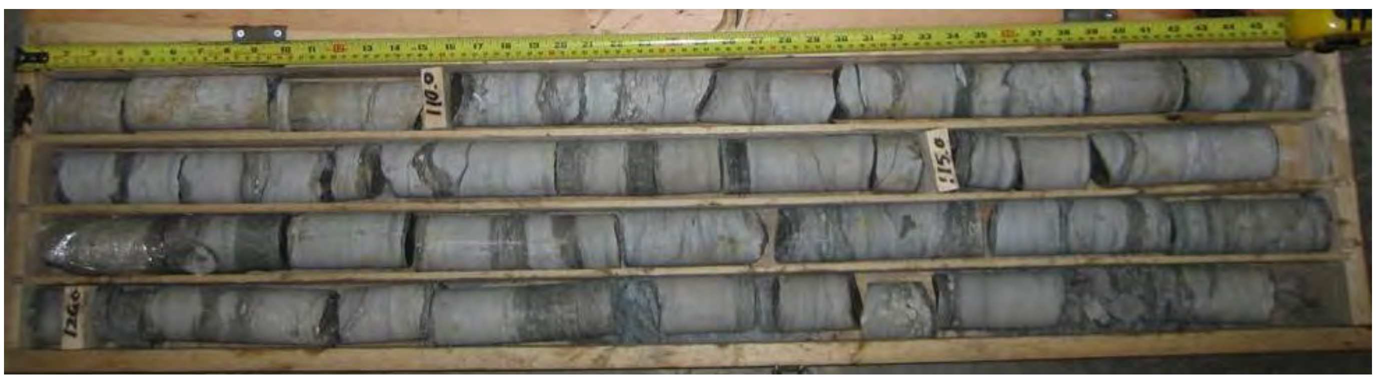
MATERIAL DESCRIPTION AND NOTES	ELEV. 336.7	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
INTERBEDDED LIMESTONE (70%) AND SHALE (30%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, SLIGHTLY TO MODERATELY WEATHERED, MODERATELY STRONG, VERY THIN TO THIN BEDDED, LOSS 3%, RQD 67% LS @85.2'-85.7' QU=7099 PSI LS @86.4'-86.8' QU=10809 PSI LS @90.1'-90.8' QU=7024 PSI LS @92.2'-92.8' QU=118 PSI SH @ 92.2' SDI = 57.9 LS @93'-93.8' QU=14324 PSI LS @95'-95.3' QU=8193 PSI SH @ 95.7' SDI = 52.5 LS @ 100.8' POINT LOAD = 11011 PSI LS @103'-103.5' QU=4812 PSI LS @103.5'-104' QU=14991 PSI LS @ 108' POINT LOAD = 16192 PSI LS @ 118.2' POINT LOAD = 11057 PSI LS @128.1'-128.8' QU=19640 PSI. *(continued)*	318.6	123–140	80		100	NQ-8											CORE	
			94		100	NQ-9											CORE	
			36		88	NQ-10											CORE	
			96		100	NQ-11											CORE	
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, LOSS 1%, RQD 99% LS @146.2'-147' QU=12179 PSI SH @ 153' SDI = 98.8 LS @ 156.4' POINT LOAD = 14406 PSI.	293.2	140–165 EOB	98		100	NQ-12											CORE	
			100		100	NQ-13											CORE	
			92		92	NQ-14											CORE	
			96		96	NQ-15											CORE	
			100		100	NQ-16											CORE	

NOTES: WATER USED BELOW 85 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (11 BAGS CEMENT/1.5 BAGS BENTONITE)

BORING NO.: R-5
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 75.5-95.0
ELEVATION (ft.): 373.59
1/NQ: 85.0'-90.0'; REC. 88%, RQD 25%
2/NQ: 90.0'-95.0'; REC. 100%, RQD 66%
3/NQ: 95.0'-100.0'; REC. 100%, RQD 20%

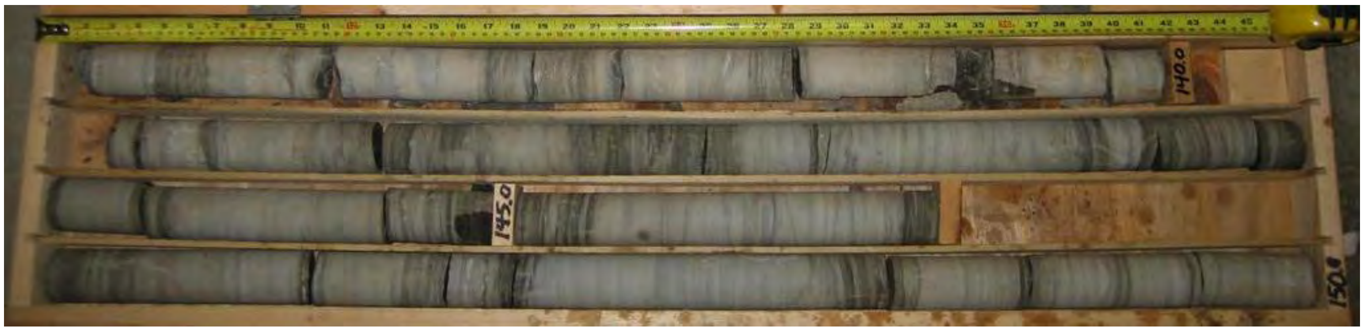
BORING NO.: R-5
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 95.0-110.0
ELEVATION (ft.): 363.59
4/NQ: 100.0'-105.0'; REC. 100%, RQD 58%
5/NQ: 105.0'-110.0'; REC. 96%, RQD 84%



BORING NO.: R-5
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 110.0-125.0
ELEVATION (ft.): 348.59
6/NQ: 110.0'-115.0'; REC. 100%, RQD 98%
7/NQ: 115.0'-120.0'; REC. 94%, RQD 76%
8/NQ: 120.0'-125.0'; REC. 100%, RQD 80%

Project Mngr.: AJM	PN. N1105070	HCN, A Terracon Company	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-5
Chkd By: DWW	File No. Core D	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-23-10		CINCINNATI, OHIO	

BORING NO.: R-5
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 125.0-140.0
ELEVATION (ft.): 333.59
9/NQ: 125.0'-130.0'; REC. 100%, RQD 94%
10/NQ: 130.0'-135.0'; REC. 88%, RQD 36%
11/NQ: 135.0'-140.0'; REC. 100%, RQD 96%



BORING NO.: R-5
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 140.0-150.0
ELEVATION (ft.): 318.59
12/NQ: 140.0'-145.0'; REC. 100%, RQD 98%
13/NQ: 145.0'-150.0'; REC. 100%, RQD 100%

BORING NO.: R-5
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 150.0-165.0
ELEVATION (ft.): 308.59
14/NQ: 150.0'-155.0'; REC. 92%, RQD 92%
15/NQ: 155.0'-160.0'; REC. 96%, RQD 96%
16/NQ: 160.0'-165.0'; REC. 100%, RQD 100%

Project Mngr.: AJM	PN. N1105070		ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	R-5
Chkd By: DWW	File No. Core D	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226		
Approved By: AJM	Date: 9-23-10			

PROJECT: BRENT SPENCE BRIDGE TYPE: BRIDGE REPLACEMENT PID: 75119 BR ID: START: 7/6/10 END: 7/9/10	DRILLING FIRM / OPERATOR: HCN / JJ SAMPLING FIRM / LOGGER: HCN / DRK/DWW DRILLING METHOD: 3.25" HSA / NQ SAMPLING METHOD: SPT / ST / NQ	DRILL RIG: CME 550X ATV- 9333 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/4/10 ENERGY RATIO (%): 67.1	STATION / OFFSET: 6+99.6, 41.1 RT ALIGNMENT: PROPOSED BSB ELEVATION: 457.0 (MSL) EOB: 164.0 ft. COORD: 39.089167170, -84.523064450	EXPLORATION ID R-6 PAGE 1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 457.0	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
MEDIUM STIFF, BROWN AND GRAY, **SANDY SILT**, SOME CLAY, TRACE GRAVEL, MOIST	454.5	WOH WOH WOH	0	100	SS-1	0.50	1	1	25	49	24	28	19	9	29	A-4a (8)	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, TRACE SAND, MOIST	452.0	1 2 2	4	100	SS-2	1.00	0	0	2	67	31	35	22	13	32	A-6a (9)	
MEDIUM STIFF, BROWN, **SILTY CLAY**, TRACE GRAVEL, TRACE SAND, MOIST	450.0			100	ST-3	0.50	1	0	6	60	33	38	22	16	35	A-6b (10)	
LOOSE, BROWN, **SILT**, SOME SAND, SOME CLAY, WET	447.0	WOH WOH WOH	0	100	SS-4	-	0	0	22	52	26	30	20	10	33	A-4b (8)	
MEDIUM STIFF, BROWN, **SILT AND CLAY**, SOME SAND, WET	444.5	1 1 2	3	100	SS-5	0.75	0	0	31	42	27	30	17	13	26	A-6a (8)	
MEDIUM STIFF, BROWN AND GRAY, **SANDY SILT**, SOME CLAY, WET	442.0	1 1 2	3	100	SS-6	0.50	0	0	39	39	22	26	17	9	24	A-4a (5)	
MEDIUM STIFF, BROWN AND GRAY, **SILT AND CLAY**, LITTLE SAND, WET	440.0			92	ST-7	-	0	0	18	55	27	34	23	11	-	A-6a (8)	
MEDIUM STIFF, GRAY, **SANDY SILT**, LITTLE CLAY, TRACE GRAVEL, WET		2 2 3	6	100	SS-8	1.00	6	1	43	31	19	26	17	9	26	A-4a (3)	
	432.0	2 1 2	3	100	SS-9	1.00	0	1	32	40	27	33	23	10	48	A-4a (6)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, TRACE TO SOME SAND, TRACE SILT, TRACE CLAY, WET		4 5 6	12	56	SS-10	-	85	4	2	6	3	NP	NP	NP	10	A-1-a (0)	
		8 11 16	30	67	SS-11	-	-	-	-	-	-	-	-	-	8	A-1-a (V)	
		16 13 26	44	100	SS-12	-	58	20	12	6	4	NP	NP	NP	9	A-1-a (0)	
	412.0	4 7 15	25	67	SS-13	-	-	-	-	-	-	-	-	-	9	A-1-a (V)	
MEDIUM DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, WET	407.0	13 9 16	28	100	SS-14	-	33	39	20	5	3	NP	NP	NP	13	A-1-b (0)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, WET		9 10 15	28	72	SS-15	-	56	21	18	3	2	NP	NP	NP	17	A-1-a (0)	
	397.0	9 8 11	21	100	SS-16	-	-	-	-	-	-	-	-	-	14	A-1-a (V)	

MATERIAL DESCRIPTION AND NOTES	ELEV. 397.0
DENSE TO VERY DENSE, BROWN, **GRAVEL AND/OR STONE FRAGMENTS**, SOME SAND, TRACE COBBLES, TRACE SILT, TRACE CLAY, WET	373.0 (TR at 84)
INTERBEDDED LIMESTONE (60%) AND SHALE (40%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED; **SHALE**, GRAY, MODERATELY WEATHERED, VERY WEAK TO WEAK, LOSS 1%, RQD 53% LS @84.1'-84.5' QU=5911 PSI SH @ 85.1' SDI = 36.9 LS @88.5'-89' QU=7988 PSI LS @ 91.5' POINT LOAD = 11637 PSI SH @ 91.5' SDI = 53.6 LS @94.5'-94.9' QU=9745 PSI LS @99.6'-100.1' QU=14253 PSI LS @100.1'-100.5' QU=12695 PSI SH @ 100.5' SDI = 91.0.	355.5
LIMESTONE, LIGHT GRAY, UNWEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, INTERMEDIATE SHALE SEAMS TO PARTINGS, LOSS 0%, RQD 81% LS @100.1'-100.5' QU=12695 PSI LS @ 105' POINT LOAD = 12607 PSI LS @107.1'-107.5' QU=8745 PSI LS @114.5'-115' QU=10184 PSI LS @ 124.7' POINT LOAD = 11607 PSI.	

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
61	12 9 16	28	100	SS-17	-	-	-	-	-	-	-	-	-	22	A-1-a (V)	
66	25 25 27	58	100	SS-18	-	54	18	20	6	2	NP	NP	NP	15	A-1-a (0)	
71	55 23 26	55	100	SS-19	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
75	100 60/3"	-	100	SS-20	-	58	13	17	8	4	NP	NP	NP	9	A-1-a (0)	
80	100/4"	-	100	SS-21	-	-	-	-	-	-	-	-	-	6	A-1-a (V)	
85	48		92	NQ-1											CORE	
89	48		100	NQ-2											CORE	
94	44		100	NQ-3											CORE	
99	68		100	NQ-4											CORE	
104	82		100	NQ-5											CORE	
109	78		100	NQ-6											CORE	
114	86		100	NQ-7											CORE	
119	82		100	NQ-8											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 6+99.6, 41.1 RT	START: 7/6/10	END: 7/9/10	PG 3 OF 3	R-6

MATERIAL DESCRIPTION AND NOTES	ELEV. 335.2
LIMESTONE, LIGHT GRAY, UNWEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS, INTERMEDIATE SHALE SEAMS TO PARTINGS, LOSS 0%, RQD 81% LS @100.1'-100.5' QU=12695 PSI LS @ 105' POINT LOAD = 12607 PSI LS @107.1'-107.5' QU=8745 PSI LS @114.5'-115' QU=10184 PSI LS @ 124.7' POINT LOAD = 11607 PSI. *(continued)*	325.5
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, LOSS 2%, RQD 93% LS @136.5'-137.3' QU=11456 PSI LS @ 153.1' POINT LOAD = 13102 PSI LS @158.4'-158.9' QU=22557 PSI LS @159.8'-160.2' QU=8843 PSI.	293.0

DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
124	84		100	NQ-9											CORE	
129	72		100	NQ-10											CORE	
134	76		90	NQ-11											CORE	
139	84		100	NQ-12											CORE	
144	100		100	NQ-13											CORE	
149	100		100	NQ-14											CORE	
154	98		98	NQ-15											CORE	
159	100		100	NQ-16											CORE	
163	92		92	NQ-17											CORE	
EOB 164																

NOTES: DRILL FLUID USED BELOW 10 FT. WATER USED BELOW 84 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (12 BAGS CEMENT/1.5 BAGS BENTONITE)

BORING NO.: R-6
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 84.0-96.2
ELEVATION (ft.): 373
1/NQ: 84.0'-86.5'; REC. 92%, RQD 48%
2/NQ: 86.5'-91.5'; REC. 100%, RQD 48%
3/NQ: 91.5-96.1'; REC. 50%, RQD 48%

BORING NO.: R-6
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 96.2-111.5
ELEVATION (ft.): 360.8
4/NQ: 96.1'101.5'; REC. 93%, RQD 63%
5/NQ: 101.5'-106.5'; REC.100 %, RQD 82%
6/NQ: 106.5'-111.5'; REC. 100%, RQD 78%

BORING NO.: R-6
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 111.5-125.7
ELEVATION (ft.): 345.5
7/NQ: 111.5'-116.5'; REC. 100%, RQD 86%
8/NQ: 116.5'-121.5'; REC. 100%, RQD 82%
9/NQ: 121.5'-126.5'; REC. 100%, RQD 84%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core D Date: 9-23-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-6

BORING NO.: R-6
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 125.7-140.7
ELEVATION (ft.): 331.3
10/NQ: 126.5'-131.5'; REC. 100%, RQD 72%
11/NQ: 131.5'-136.5'; REC. 90%, RQD 76%
12/NQ: 136.5'-141.5'; REC. 100%, RQD 84%

BORING NO.: R-6
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 140.7-153.9
ELEVATION (ft.): 316.3
13/NQ:141.5'-146.5 '; REC. 100%, RQD 100%
14/NQ: 146.5'-151.6'; REC. 100%, RQD 100%
15/NQ: 151.5'-156.5'; REC. 100%, RQD 100%

BORING NO.: R-6
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 153.9-164.0
ELEVATION (ft.): 303.1
16/NQ: 156.5'-161.5'; REC. 83%, RQD 83%
17/NQ: 161.5'-164.0'; REC. 92%, RQD 92%

Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core D Date: 9-23-10	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-6

PROJECT: BRENT SPENCE BRIDGE	DRILLING FIRM / OPERATOR: HCN / HH
TYPE: BRIDGE REPLACEMENT	SAMPLING FIRM / LOGGER: HCN / DRK/DWW
PID: 75119 BR ID:	DRILLING METHOD: 3.25" HSA / NQ
START: 7/2/10 END: 7/4/10	SAMPLING METHOD: SPT / NQ
DRILL RIG: CME 550X ATV-7253	STATION / OFFSET: 7+85.2, 32.7 LT
HAMMER: CME AUTOMATIC	ALIGNMENT: PROPOSED BSB
CALIBRATION DATE: 2/4/10	ELEVATION: 458.5 (MSL) EOB: 164.5 ft.
ENERGY RATIO (%): 76.3	COORD: 39.089410800, -84.523311230
EXPLORATION ID: R-7	PAGE 1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 458.5	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
WATER (OHIO RIVER)		0 – 21																
MEDIUM STIFF, GRAY, **CLAY**, AND SILT, TRACE ORGANICS, TRACE GRAVEL, TRACE SAND, WET	437.5	21 – 22.5	1 1 2	4	0	SS-1	1.00	-	-	-	-	-	-	-	-	-	A-7-6 (V)	
		22.5 – 24	2 2 2	5	100	SS-2	0.75	5	2	10	45	38	42	22	20	46	A-7-6 (12)	
MEDIUM STIFF, GRAY, **SILT AND CLAY**, SOME GRAVEL, SOME SAND, WET	434.5	24 – 25.5	1 2 2	5	33	SS-3	1.00	33	3	21	25	18	32	18	14	24	A-6a (3)	
		25.5 – 27	4 3 2	6	22	SS-4	1.00	34	4	24	22	16	-	-	-	31	A-6a (V)	
		27 – 28.5	2 3 9	15	33	SS-5	1.00	35	4	21	25	15	-	-	-	33	A-6a (V)	
		28.5	50/0"	-		SS-6	1.25	-	-	-	-	-	-	-	-	-	A-6a (V)	
		30 – 31.5	9 6 4	13	0	SS-7	-	-	-	-	-	-	-	-	-	-	A-6a (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, VERY DENSE AT 60', WET	426.0	32.5 – 34	9 5 8	17	33	SS-8	-	68	19	5	5	3	-	-	-	12	A-1-a (V)	
		35 – 36	16 18 50/0"	-	100	SS-9	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
		37.5 – 39	7 14 11	32	44	SS-10	-	-	-	-	-	-	-	-	-	10	A-1-a (V)	
		40 – 41.5	9 9 12	27	33	SS-11	-	62	16	11	7	4	NP	NP	NP	10	A-1-a (0)	
		45 – 46.5	14 15 16	39	56	SS-12	-	-	-	-	-	-	-	-	-	7	A-1-a (V)	
		50 – 51.5	16 9 8	22	56	SS-13	-	57	27	8	5	3	NP	NP	NP	13	A-1-a (0)	
		55 – 56.5	18 12 10	28	44	SS-14	-	-	-	-	-	-	-	-	-	13	A-1-a (V)	

PID: 75119 | BR ID: | PROJECT: BRENT SPENCE BRIDGE | STATION / OFFSET: 7+85.2, 32.7 LT | START: 7/2/10 | END: 7/4/10 | | R-7

MATERIAL DESCRIPTION AND NOTES	ELEV. 398.5	DEPTHS
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS**, SOME SAND, TRACE SILT, TRACE CLAY, VERY DENSE AT 60', WET *(continued)*	388.5	70
VERY DENSE, GRAY, **STONE FRAGMENTS**, LITTLE SAND, TRACE SILT, TRACE CLAY, LIMESTONE FLOATERS AND COBBLES, WET	376.0	82 (TR)
INTERBEDDED LIMESTONE (65%) AND SHALE (35%); **LIMESTONE**, LIGHT GRAY, SLIGHTLY TO MODERATELY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, MODERATELY WEATHERED, VERY WEAK, LOSS 5%, RQD 26% LS @83.5'-83.9' QU=8872 PSI SH @ 93.4' SDI = 79.5 SH @ 95.7' SDI = 72.8 LS @98'-98.5' QU=6802 PSI LS @ 89.7' POINT LOAD = 12982 PSI.	359.0	99
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, LOSS 2%, RQD 80% LS @ 100.8' POINT LOAD = 11981 PSI SH @ 102' SDI = 92.6 LS @106.2'-106.7' QU=16419 PSI SH @ 121.1' SDI = 80.4 SH @121.1'-121.4' QU=1833 PSI LS @ 125.9' POINT LOAD = 14914 PSI LS @128.7'-129.5' QU=8525 PSI.		

Depth	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
60–61.5	24 25 26	65	67	SS-15	-	60	16	14	5	5	NP	NP	NP	10	A-1-a (0)	
65–66.5	45 20 18	48	67	SS-16	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
70	50/4"	-	100	SS-17	-	71	12	8	5	4	NP	NP	NP	13	A-1-a (0)	
75	50/0"	-		SS-18	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
82–85	25		85	NQ-1											CORE	
85–90	20		94	NQ-2											CORE	
90–95	20		94	NQ-3											CORE	
95–100	40		100	NQ-4											CORE	
100–105	72		100	NQ-5											CORE	
105–110	92		100	NQ-6											CORE	
110–115	92		100	NQ-7											CORE	
115–120	70		92	NQ-8											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 7+85.2, 32.7 LT	START: 7/2/10	END: 7/4/10	PG 3 OF 3	R-7

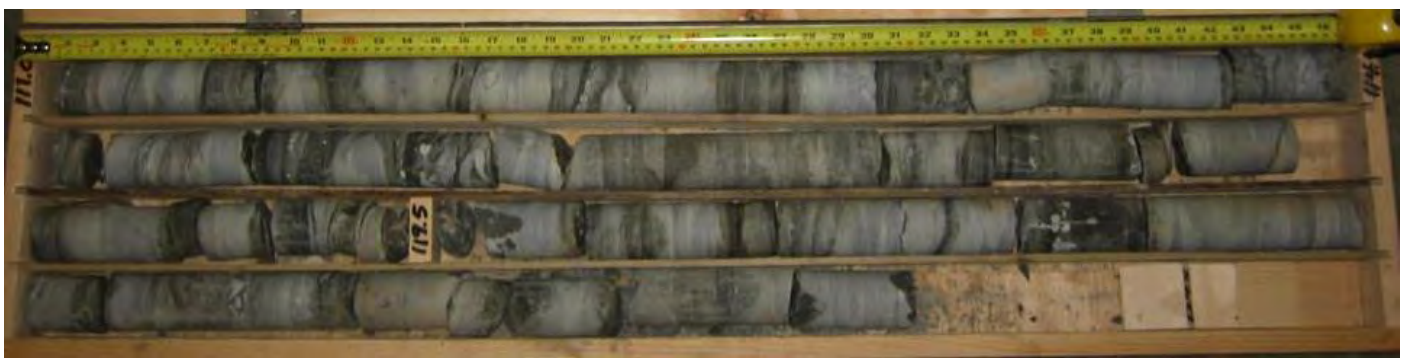
MATERIAL DESCRIPTION AND NOTES	ELEV. 336.6	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
INTERBEDDED LIMESTONE (75%) AND SHALE (25%); **LIMESTONE**, LIGHT GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, FOSSILIFEROUS; **SHALE**, GRAY, SLIGHTLY WEATHERED, MODERATELY STRONG, LOSS 2%, RQD 80% LS @ 100.8' POINT LOAD = 11981 PSI SH @ 102' SDI = 92.6 LS @106.2'-106.7' QU=16419 PSI SH @ 121.1' SDI = 80.4 SH @121.1'-121.4' QU=1833 PSI LS @ 125.9' POINT LOAD = 14914 PSI LS @128.7'-129.5' QU=8525 PSI. *(continued)*	324.0	123–134	66		100	NQ-9											CORE	
			74		100	NQ-10											CORE	
			96		96	NQ-11											CORE	
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN BEDDED, ARGILLACEOUS, LOSS 2%, RQD 94% LS @136.6'-137.6' QU=11974 PSI LS @ 145.5' POINT LOAD = 13149 PSI LS @154.5'-155.1' QU=12586 PSI LS @163.7'-164.5' QU=8772 PSI.	294.0	135–164, EOB	92		100	NQ-12											CORE	
			90		96	NQ-13											CORE	
			100		100	NQ-14											CORE	
			90		94	NQ-15											CORE	
			100		100	NQ-16											CORE	
			94		100	NQ-17											CORE	

NOTES: WATER USED BELOW 82.5 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (10 BAGS CEMENT/1 BAG BENTONITE)

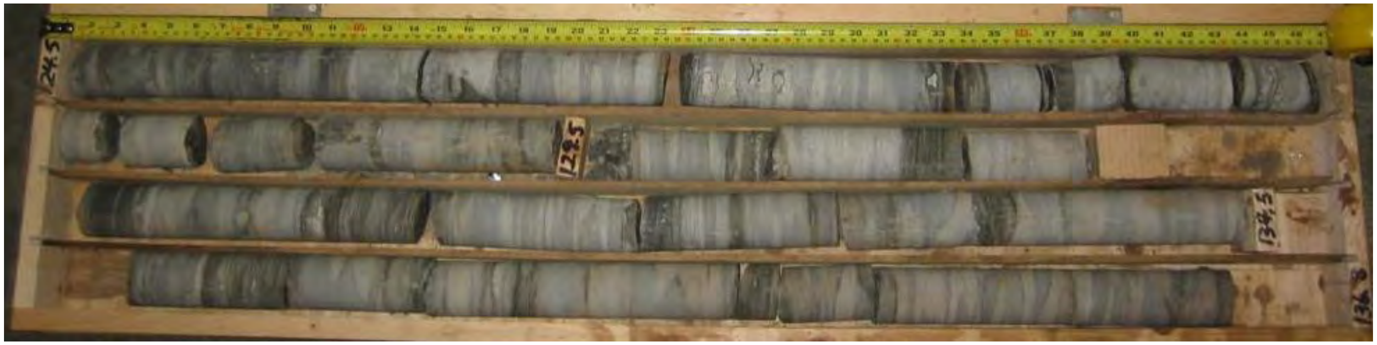
BORING NO.: R-7
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 78.0-96.7
ELEVATION (ft.): 376.0
1/NQ: 82.5'-84.5'; REC. 85%, RQD 25%
2/NQ: 84.5'-89.5'; REC. 94%, RQD 20%
3/NQ: 89.5'-94.5'; REC. 94%, RQD 20%
4/NQ: 94.5'-99.5'; REC. 100%, RQD 40%

BORING NO.: R-7
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 96.7-111.0
ELEVATION (ft.): 357.3
5/NQ: 99.5'104.5'; REC. 100%, RQD 72%
6/NQ: 104.5'-109.5'; REC. 100%, RQD 92%
7/NQ: 109.5-114.5'; REC. 100%, RQD 92%



BORING NO.: R-7
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 111.0-124.5
ELEVATION (ft.): 343
8/NQ: 114.5'-119.5'; REC. 92%, RQD 70%
9/NQ: 119.5'-124.5'; REC. 100%, RQD 66%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-7
Chkd By: DWW	File No. Core D	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-23-10		CINCINNATI, OHIO	



BORING NO.: R-7
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 124.5-136.8
ELEVATION (ft.): 329.5
10/NQ: 124.5'-129.5'; REC. 100%, RQD 74%
11/NQ: 129.5'-134.5'; REC. 96%, RQD 96%
12/NQ: 134.5'-139.5'; REC. 100%, RQD 92%

BORING NO.: R-7
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 136.8-149.5
ELEVATION (ft.): 317.2
13/NQ: 139.5'-144.5'; REC. 96%, RQD 90%
14/NQ: 144.5'-149.5'; REC. 100%, RQD 100%

BORING NO.: R-7
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 149.5-164.5
ELEVATION (ft.): 304.5
15/NQ: 149.5'-154.5'; REC. 94%, RQD 90%
16/NQ: 154.5'-159.5'; REC. 100%, RQD 100%
17/NQ: 159.5'-164.5'; REC. 100%, RQD 94%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown	611 LUNKEN PARK DRIVE	BRENT SPENCE BRIDGE REPLACEMENT	R-7
Chkd By: DWW	File No. Core D	CINCINNATI, OHIO 45226	PARSONS BRINCKERHOFF	
Approved By: AJM	Date: 9-23-10		CINCINNATI, OHIO	

PROJECT: BRENT SPENCE BRIDGE TYPE: BRIDGE REPLACEMENT PID: 75119 BR ID: START: 9/3/10 END: 9/4/10	DRILLING FIRM / OPERATOR: HCN / HH SAMPLING FIRM / LOGGER: HCN / DWW DRILLING METHOD: 3.25" HSA / NQ SAMPLING METHOD: SPT / NQ	DRILL RIG: CME 550X ATV-7253 HAMMER: CME AUTOMATIC CALIBRATION DATE: 2/4/10 ENERGY RATIO (%): 76.3	STATION / OFFSET: 6+97.7, 41.1 LT ALIGNMENT: PROPOSED BSB ELEVATION: 455.7 (MSL) EOB: 161.0 ft. COORD: 39.089171808, -84.523354152	EXPLORATION ID R-8 PAGE 1 OF 3

MATERIAL DESCRIPTION AND NOTES	ELEV. 455.7	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
VERY SOFT TO SOFT, BROWN, **SILT AND CLAY**, TRACE SAND, MOIST TO WET		1 1 0	1	44	SS-1	0.25	-	-	-	-	-	-	-	-	36	A-6a (V)	
		1 0 0	0	33	SS-2	0.25	0	0	6	63	31	36	21	15	33	A-6a (10)	
		2 2 2	5	100	SS-3	0.25	-	-	-	-	-	-	-	-	29	A-6a (V)	
		2 1 2	4	100	SS-4	0.50	-	-	-	-	-	-	-	-	28	A-6a (V)	
		2 2 2	5	67	SS-5	0.25	-	-	-	-	-	-	-	-	28	A-6a (V)	
		1 3 2	6	100	SS-6	0.25	-	-	-	-	-	-	-	-	30	A-6a (V)	
		1 2 2	5	100	SS-7	0.25	-	-	-	-	-	-	-	-	28	A-6a (V)	
	440.7	1 1 1	3	100	SS-8	0.25	-	-	-	-	-	-	-	-	31	A-6a (V)	
SOFT TO MEDIUM STIFF, BROWN, **SILT**, SOME SAND, SOME CLAY, WET		1 2 1	4	100	SS-9	0.50	0	0	24	52	24	30	21	9	40	A-4b (8)	
		1 2 1	4	100	SS-10	0.25	-	-	-	-	-	-	-	-	36	A-4b (V)	
		1 1 2	4	100	SS-11	0.25	-	-	-	-	-	-	-	-	33	A-4b (V)	
	425.7	1 2 3	6	100	SS-12	0.25	-	-	-	-	-	-	-	-	34	A-4b (V)	
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY DENSE AT 55', WET		8 14 20	43	100	SS-13	-	-	-	-	-	-	-	-	-	9	A-1-b (V)	
		7 14 18	41	100	SS-14	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		3 6 11	22	100	SS-15	-	39	35	20	4	2	NP	NP	NP	17	A-1-b (0)	
		10 15 16	39	100	SS-16	-	-	-	-	-	-	-	-	-	10	A-1-b (V)	
		10 10 11	27	100	SS-17	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
		13 20 21	52	100	SS-18	-	27	49	15	6	3	NP	NP	NP	12	A-1-b (0)	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 6+97.7, 41.1 LT	START: 9/3/10	END: 9/4/10	PG 2 OF 3	R-8

MATERIAL DESCRIPTION AND NOTES	ELEV. 395.7	DEPTHS
MEDIUM DENSE TO DENSE, BROWN, **GRAVEL AND STONE FRAGMENTS WITH SAND**, TRACE SILT, TRACE CLAY, VERY DENSE AT 55', WET *(continued)*	388.7	61–67
VERY DENSE, BROWN AND GRAY, **GRAVEL AND STONE FRAGMENTS**, SOME COBBLES, LITTLE SAND, WET	375.7	67–80 (TR at 80)
INTERBEDDED LIMESTONE (50%) AND SHALE (50%); **LIMESTONE**, GRAY, UNWEATHERED TO SLIGHTLY WEATHERED, STRONG, THIN BEDDED, TRACE FOSSILIFEROUS SEAMS; **SHALE**, GRAY, SLIGHTLY WEATHERED, VERY WEAK, LAMINATED, LOSS 2%, RQD 41% LS @87.8'-88.2' QU=9645 PSI LS @ 96' POINT LOAD = 10656 PSI.	357.7	80–98
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN TO MEDIUM BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, LOSS 1%, RQD 92% SH @ 88.4' SDI = 66.8 LS @100.5'-101' QU=11240 PSI LS @101.8'-102.3' QU=4944 PSI LS @ 118.6' POINT LOAD = 10656 PSI LS @126.3'-126.7' QU=11631 PSI LS @127.8'-128.3' QU=10674 PSI LS @135.5'-136' QU=10495 PSI LS @141'-141.5' QU=12721 PSI LS @149'-149.5 QU=12619 PSI LS @151.8'-152.1' QU=10244 PSI LS @158.7'-159.2' QU=12011 PSI.		98–121

SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
7 22 22	56	100	SS-19	-	-	-	-	-	-	-	-	-	17	A-1-b (V)	
12 10 20	38	100	SS-20	-	-	-	-	-	-	-	-	-	14	A-1-b (V)	
12 50/2"	-	0	SS-21	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
50/0"	-		SS-22	-	-	-	-	-	-	-	-	-	-	A-1-a (V)	
42		100	NQ-1											CORE	
43		100	NQ-2											CORE	
20		94	NQ-3											CORE	
60		100	NQ-4											CORE	
68		94	NQ-5											CORE	
90		100	NQ-6											CORE	
80		100	NQ-7											CORE	
92		100	NQ-8											CORE	
90		96	NQ-9											CORE	

PID: 75119	BR ID:	PROJECT: BRENT SPENCE BRIDGE	STATION / OFFSET: 6+97.7, 41.1 LT	START: 9/3/10	END: 9/4/10	PG 3 OF 3	R-8

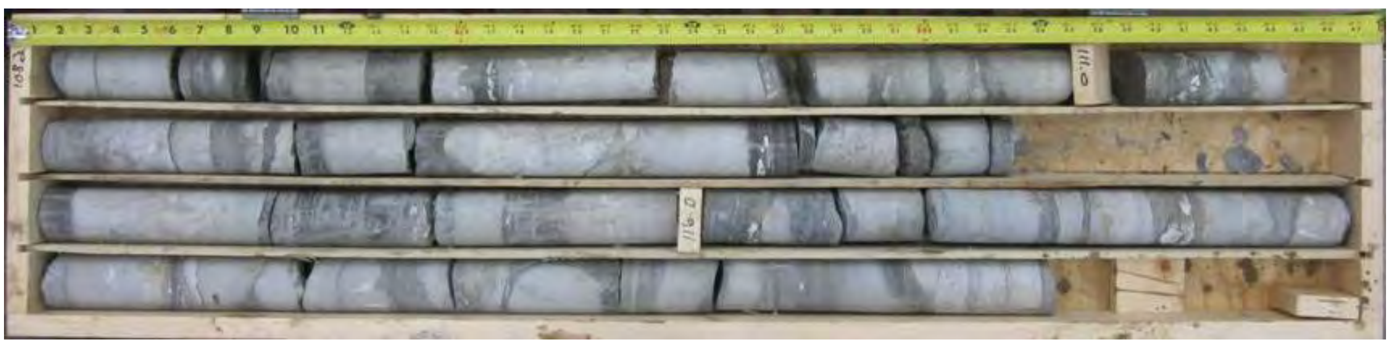
MATERIAL DESCRIPTION AND NOTES	ELEV. 333.8	DEPTHS	SPT/ RQD	N_{60}	REC (%)	SAMPLE ID	HP (tsf)	GR	CS	FS	SI	CL	LL	PL	PI	WC	ODOT CLASS (GI)	HOLE SEALED
LIMESTONE, GRAY, UNWEATHERED, STRONG, THIN TO MEDIUM BEDDED, ARGILLACEOUS, FOSSILIFEROUS SEAMS, LOSS 1%, RQD 92% SH @ 88.4' SDI = 66.8 LS @100.5'-101' QU=11240 PSI LS @101.8'-102.3' QU=4944 PSI LS @ 118.6' POINT LOAD = 10656 PSI LS @126.3'-126.7' QU=11631 PSI LS @127.8'-128.3' QU=10674 PSI LS @135.5'-136' QU=10495 PSI LS @141'-141.5' QU=12721 PSI LS @149'-149.5 QU=12619 PSI LS @151.8'-152.1' QU=10244 PSI LS @158.7'-159.2' QU=12011 PSI. *(continued)*		123–126	82		100	NQ-10											CORE	
		126–131	96		100	NQ-11											CORE	
		131–136	100		100	NQ-12											CORE	
		136–141	100		100	NQ-13											CORE	
		141–146	98		98	NQ-14											CORE	
		146–151	98		100	NQ-15											CORE	
		151–156	100		100	NQ-16											CORE	
		156–161	98		96	NQ-17											CORE	
	294.7	EOB 161																

NOTES: WATER USED BELOW 80 FT. FOR ROCK CORING PURPOSES.

ABANDONMENT METHODS, MATERIALS, QUANTITIES: BACKFILLED WITH BENTONITE GROUT (11 BAGS CEMENT/1 BAGS BENTONITE)

BORING NO.: R-8
CORE BOX NO.: 1 OF 6
DEPTH (ft.): 80.0-94.0
ELEVATION (ft.): 375.70
1/NQ: 80.0'-81.0'; REC. 100%, RQD 40%
2/NQ: 81.0-86.0'; REC. 100%, RQD 44%
3/NQ: 86.0'-91.0'; REC. 94%, RQD 20%
4/NQ: 91.0'-96.0'; REC. 100%, RQD 60%

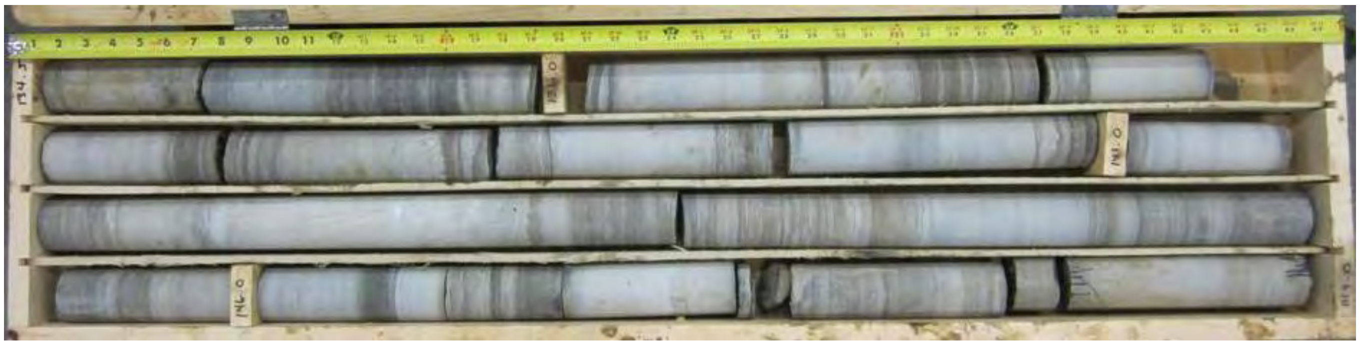
BORING NO.: R-8
CORE BOX NO.: 2 OF 6
DEPTH (ft.): 94.0-108.2
ELEVATION (ft.): 361.7
5/NQ: 96.0'-101.0'; REC. 94%, RQD 68%
6/NQ: 101.0'-106.0'; REC. 100%, RQD 90%
7/NQ: 106.0'-111.0'; REC. 100%, RQD 80%



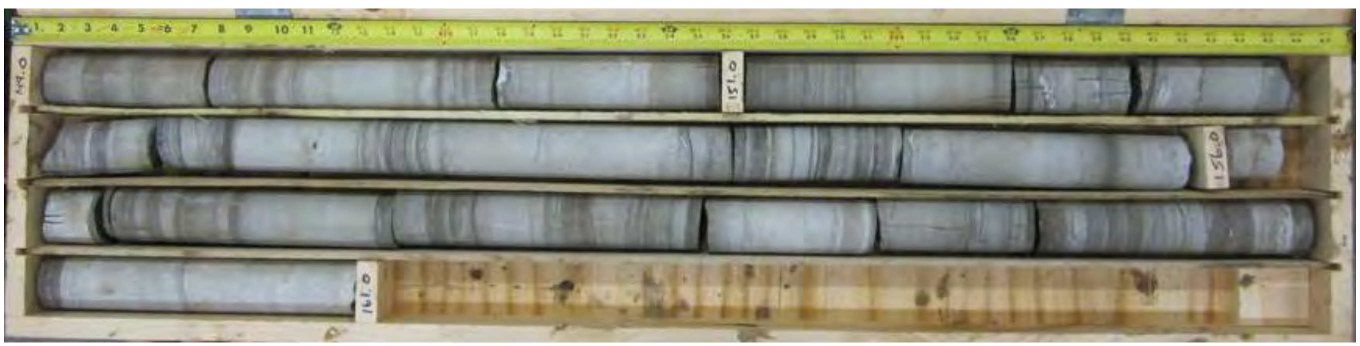
BORING NO.: R-8
CORE BOX NO.: 3 OF 6
DEPTH (ft.): 108.2-121.0
ELEVATION (ft.): 347.5
8/NQ: 111.0'-116.0'; REC. 100%, RQD 92%
9/NQ: 116.0'-121.0'; REC. 96%, RQD 90%

Project Mngr.: AJM	PN. N1105070	HCN A Terracon COMPANY	ROCK CORE PHOTOGRAPHS	BORING
Drawn By: TCF	Scale: As Shown		BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	R-8
Chkd By: DWW	File No. Core D	611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226		
Approved By: AJM	Date: 9-23-10			

BORING NO.: R-8
CORE BOX NO.: 4 OF 6
DEPTH (ft.): 121.0-134.5
ELEVATION (ft.): 334.7
10/NQ: 121.0’-126.0‘; REC. 100%, RQD 82%
11/NQ: 126.0’-131.0‘; REC. 100%, RQD 96%
12/NQ: 131.0’-136.0‘; REC. 100%, RQD 100%



BORING NO.: R-8
CORE BOX NO.: 5 OF 6
DEPTH (ft.): 134.5-149.0
ELEVATION (ft.): 321.2
13/NQ: 136.0’-141.0‘; REC. 100%, RQD 100%
14/NQ: 141.0’-146.0‘; REC. 98%, RQD 98%
15/NQ: 146.0’-151.0‘; REC. 100%, RQD 98%

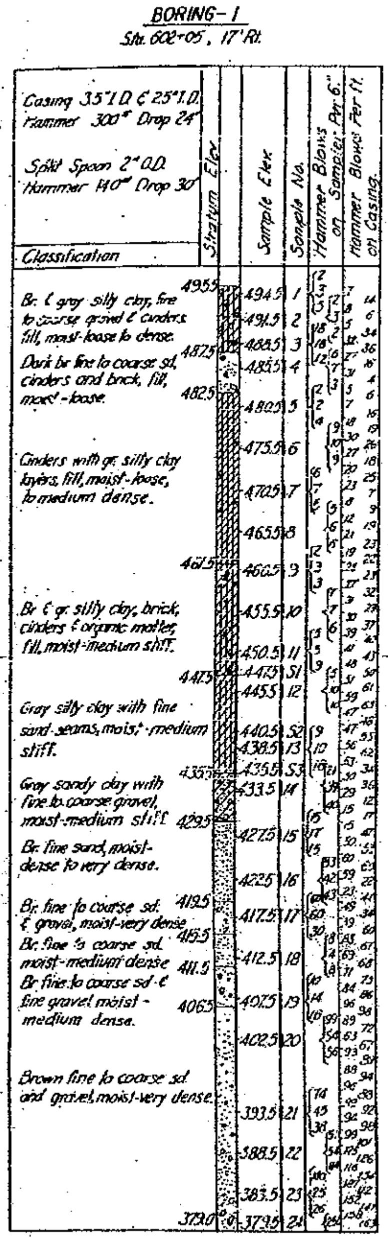


BORING NO.: R-8
CORE BOX NO.: 6 OF 6
DEPTH (ft.): 149.0-161.0
ELEVATION (ft.): 306.7
16/NQ: 151.0’-156.0‘; REC. 100%, RQD 100%
17/NQ: 156.0’-161.0‘; REC. 98%, RQD 98%

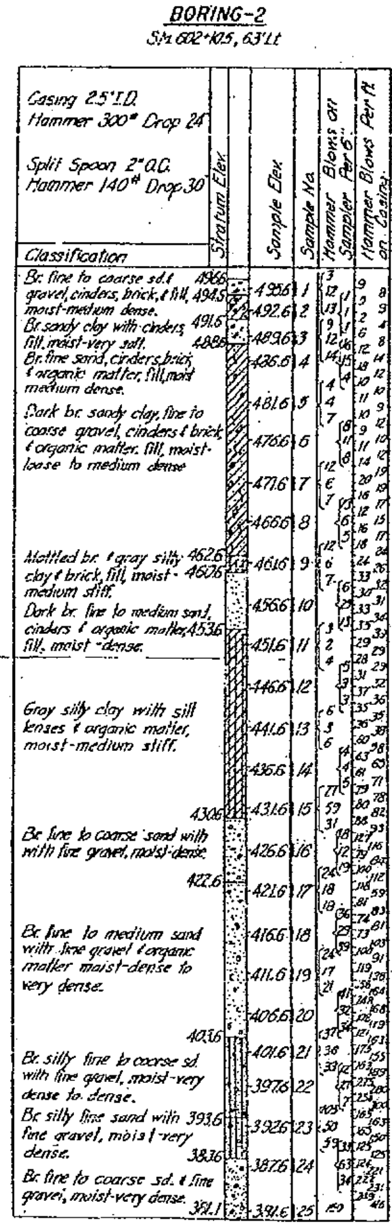
Project Mngr.: AJM Drawn By: TCF Chkd By: DWW Approved By: AJM	PN. N1105070 Scale: As Shown File No. Core D Date: 9-23-10	HCN A Terracon COMPANY 611 LUNKEN PARK DRIVE CINCINNATI, OHIO 45226	ROCK CORE PHOTOGRAPHS BRENT SPENCE BRIDGE REPLACEMENT PARSONS BRINCKERHOFF CINCINNATI, OHIO	BORING R-8

EXHIBIT A-8
EXISTING BRENT SPENCE BRIDGE
TEST BORING LOGS (1958)

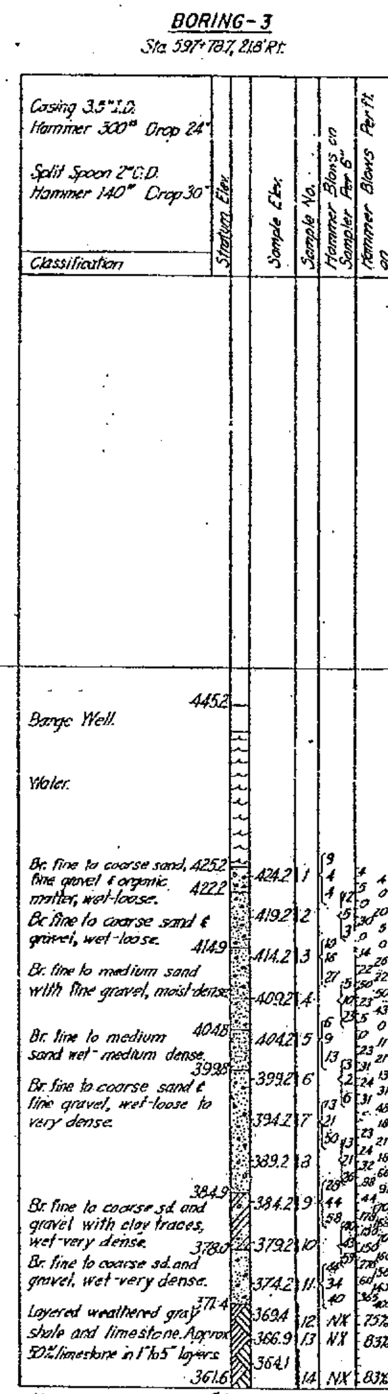
FED ROAD DIST	STATE	FED AID PRO. NO.	FISCAL YEAR	SHEET NO.	TOTAL SHEETS
7	KY	I 75-8	1960	3	



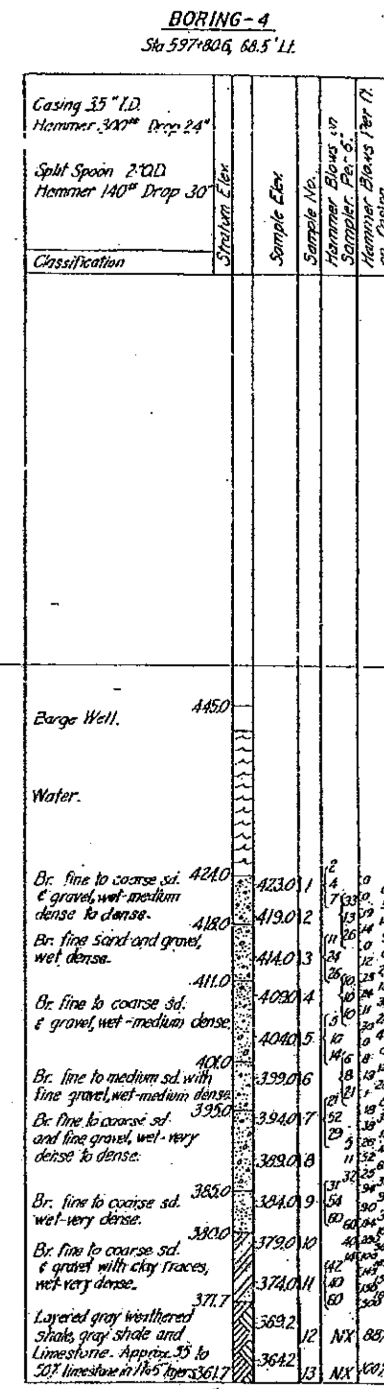
3.5" I.D. Casing driven to 59', 2.5" I.D. Casing inserted at 59'. Sampling unsuccessful at El. 397.5 due to 13' sand upheaval. Casing was driven to refusal at El. 379. This was assumed to be top of rock.
Groundwater elev. = 441.5



No recovery on sample #25
Groundwater elev. = 441.6



No recovery on sample #2



No recovery on sample #5

El. 450.00

SOIL TEST DATA
BORING-1

Sample No.	Elevation	% Sand, Silt & Clay	% Moisture	Natural (or Wet) Weight for Cu. Ft. lbs/cu. ft.	Unconfined Compressive Strength lbs/sq. ft.	Failure Strain	Atterburg Limits	Classification U.S.C.S.	Classification AASHO
S2	491.5	36% Sa. 49% Si. 15% Cl.	31.3	115.8	2130	7.5%	27.8% LL 18.2% PL 9.6 Pl. Index	CL	A4

NOTES

Borings were made in October and November 1958 by W.G. Nutting Company of Cincinnati, Ohio.

Jar samples, rock core samples, and sealed tube laboratory samples have been delivered to the Kentucky Department of Highways, District Highway Building, Covington, Kentucky.

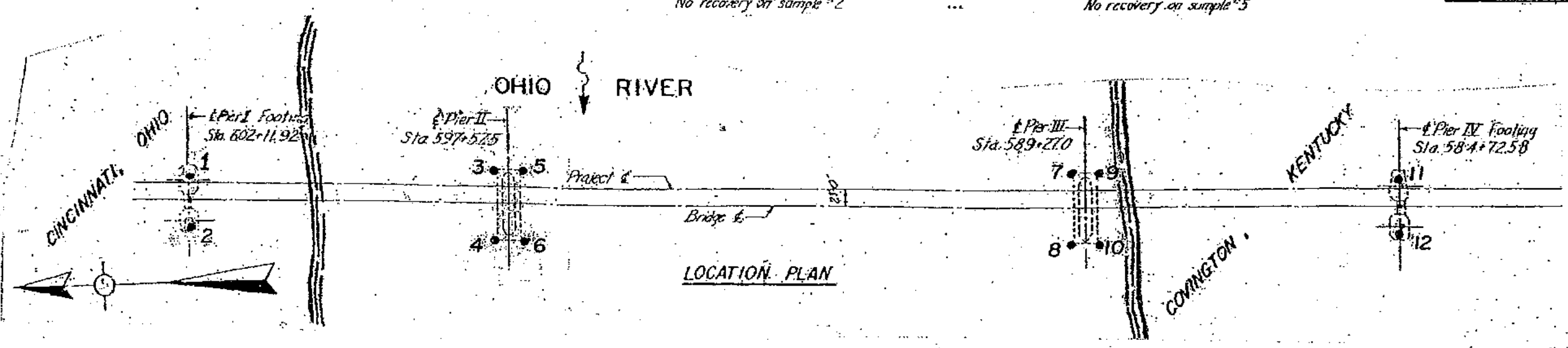
NX - Denotes core sample.

S - Denotes undisturbed sealed tube sample.

Drwg. No 14355

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF KENTON
BORINGS 1,2,3 & 4
COVINGTON-CINCINNATI OHIO RIVER BRIDGE
ROAD COVINGTON-LEXINGTON
STATION 584+74.0 TO 602+10.5 PROJECT NO. I 75-8
MODJESKI & MASTERS, ENGINEERS
HARRISBURG, PENNSYLVANIA
No. 6

TEST BORINGS
DRILLED IN 1958
BY H.C. NUTTING CO.



LOCATION PLAN

H.C. NUTTING COMPANY
CORPORATE OFFICE — 611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

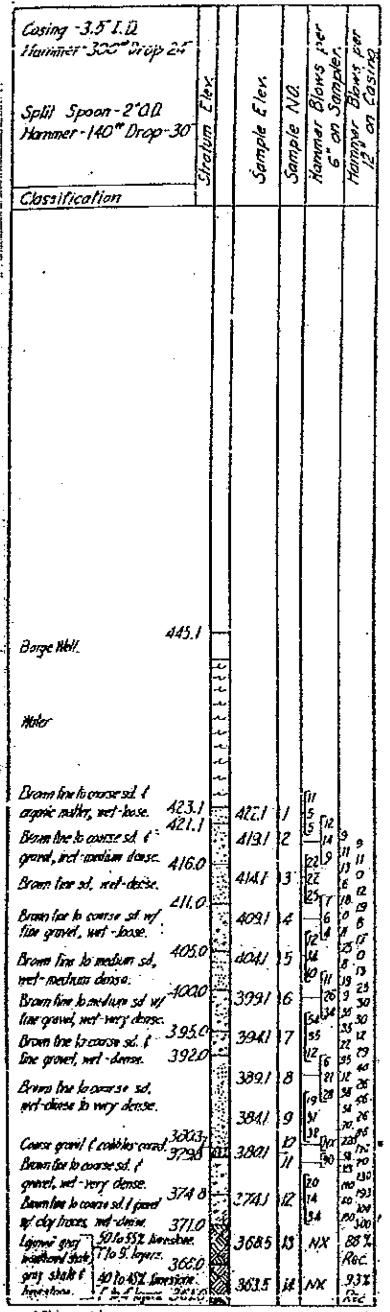
EXISTING BRIDGE DATA (B-1 To B-4)

CLIENT: PARSONS BRINCKERHOFF
PROPOSED QUEENSGATE ALIGNMENT
BRENT SPENCE BRIDGE REPLACEMENT

W.O. 10974.054

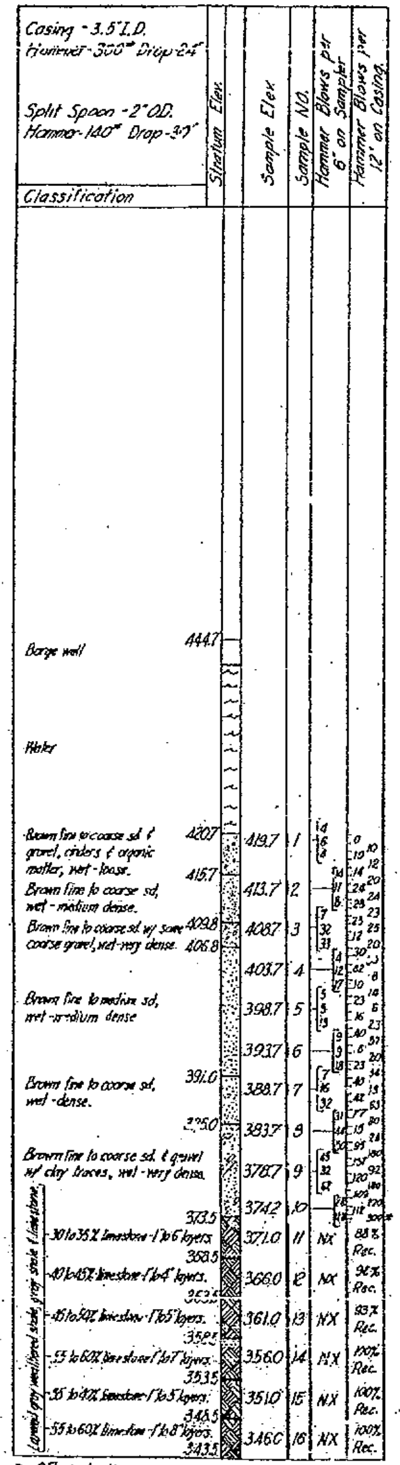
JUNE 2007		FIGURE 4A

BORING-5
Sta. 597+41.6-19.1' Rt.



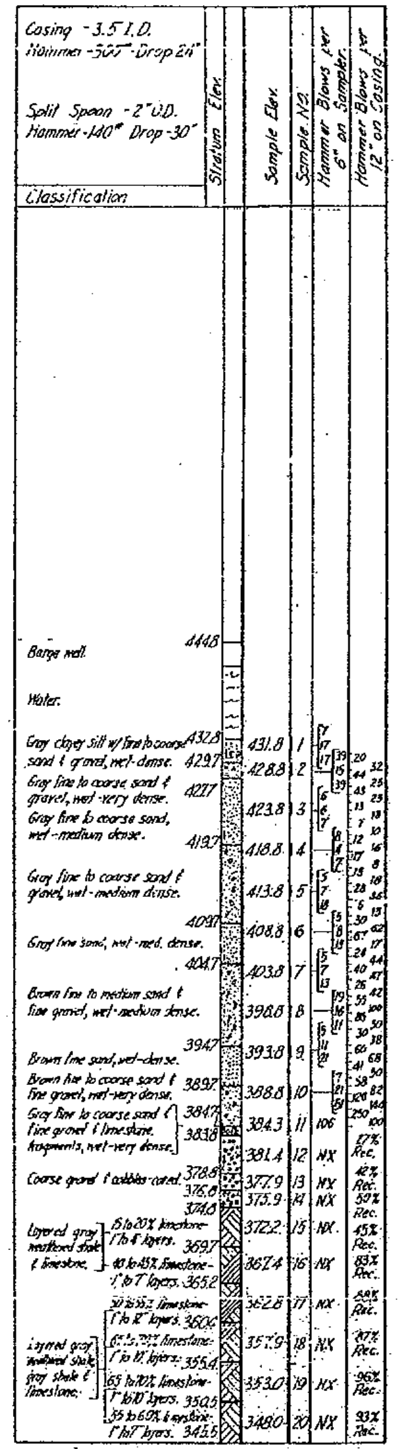
* - 0.5' penetration.
+ - 0.3' penetration.

BORING-6
Sta. 597+39.2-67.3' Lt.

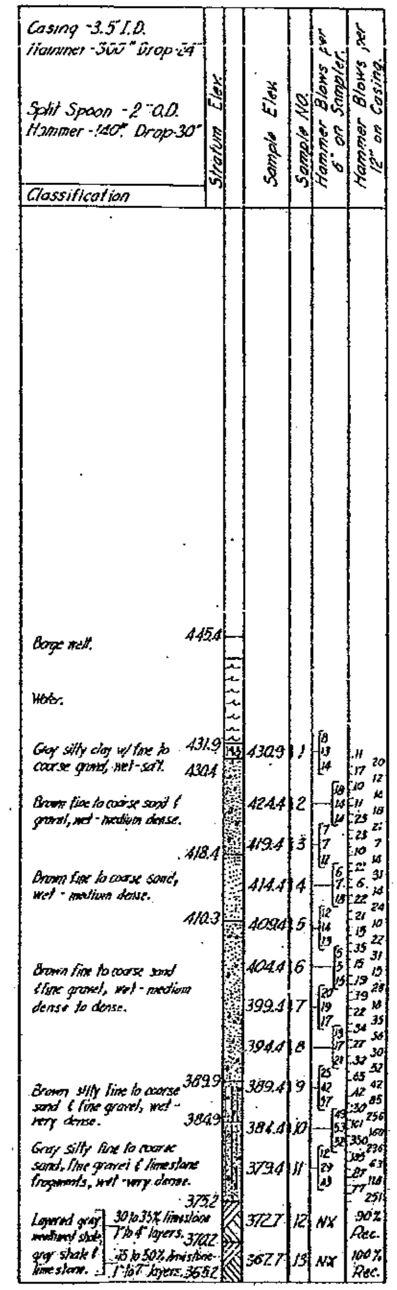


* - 0.5' penetration.

BORING-7
Sta. 589+46.2-23.7' Rt.



BORING-8
Sta. 589+47-68' Lt.

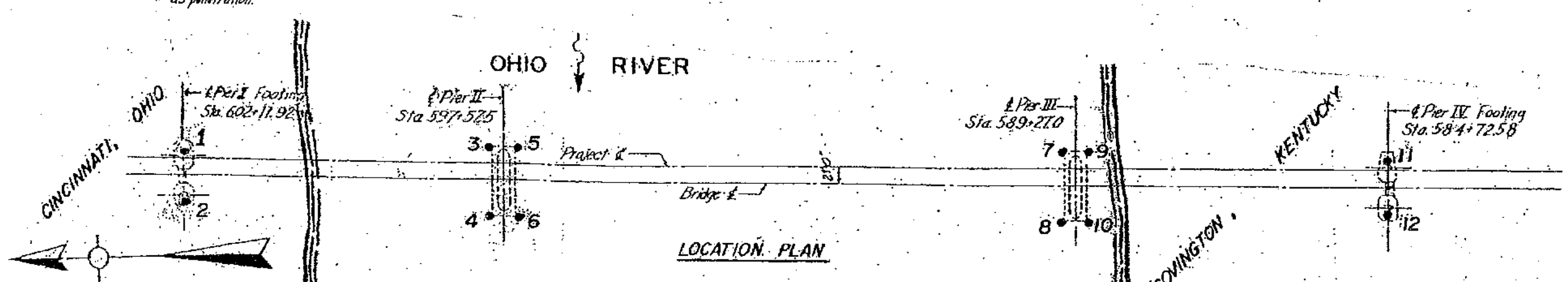


For Notes, see Sheet No. 7.

Drwg. No. 14355

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF
KENTON
BORINGS 5,6,7 & 8
COVINGTON-CINCINNATI OHIO RIVER BRIDGE
ROAD-COVINGTON-LEXINGTON
STATION 584+74.0 TO 602+10.5 PROJECT NO. 175-8
MODJESKI & MASTERS, ENGINEERS
HARRISBURG, PENNSYLVANIA
SUPPLEMENTARY DRAWING 7

TEST BORINGS
DRILLED IN 1958
BY H.C. NUTTING CO.



LOCATION PLAN

H.C. NUTTING COMPANY
CORPORATE OFFICE — 611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

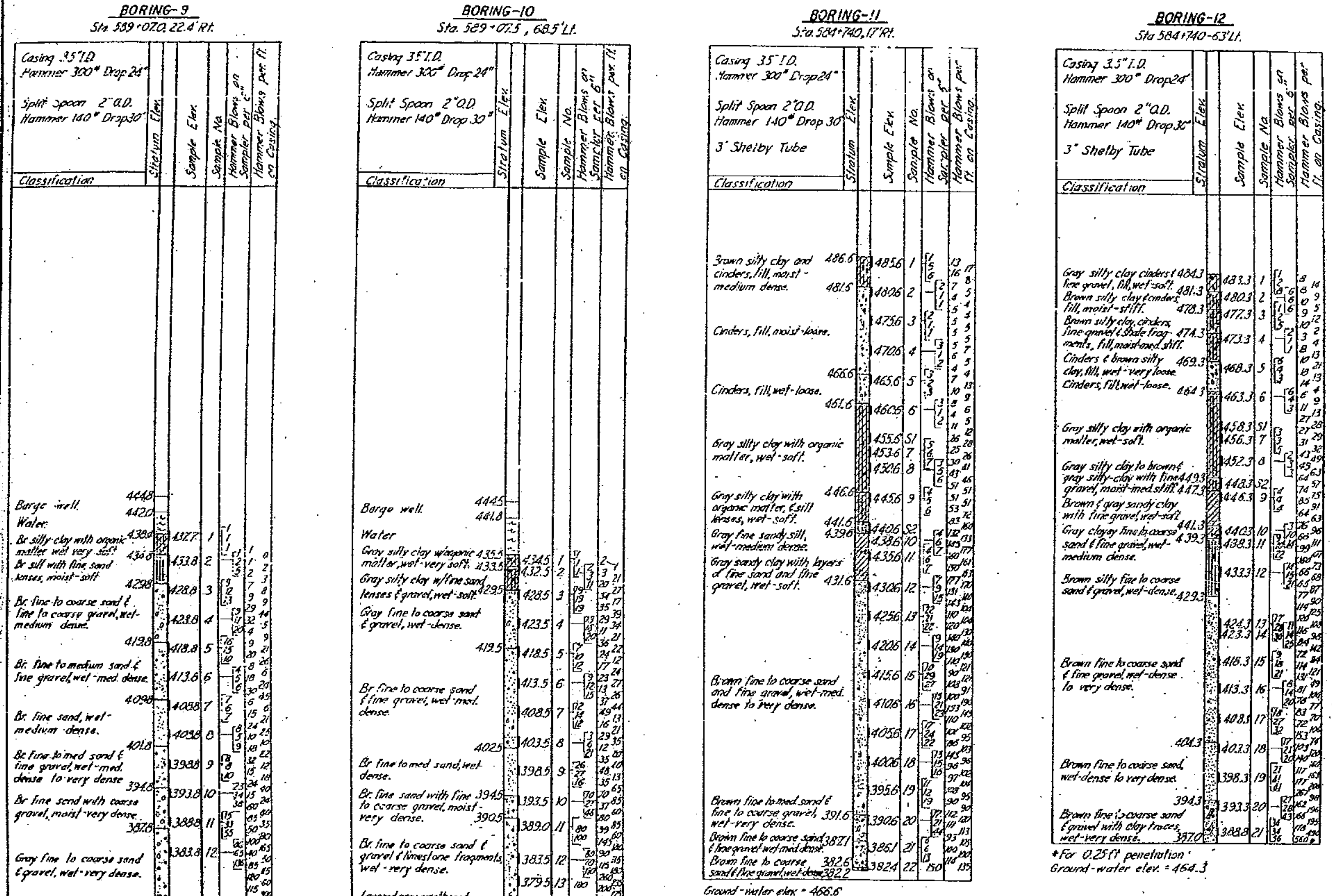
EXISTING BRIDGE DATA (B-5 To B-8)

CLIENT: PARSONS BRINCKERHOFF
PROPOSED QUEENSGATE ALIGNMENT
BRENT SPENCE BRIDGE REPLACEMENT

W.O. 10974.054

JUNE 2007

FIGURE 4 B



Undisturbed Shelby Tube Samples (Boring-11)

Sample No.	Elev.	Dead Weight (Tools etc.)	Hydraulic Pressure	Total Pressure	Time (Sec)
S1	456.6-455.5 455.5-454.5	398# 398#	0# 440#	398# 838#	8
S2	441.6-440.5 440.6-439.6	450# 450#	0# 3400#	450# 3850#	12

Undisturbed Shelby Tube Samples (Boring-12)

Sample No.	Elev.	Dead Weight (Tools etc.)	Hydraulic Pressure	Total Pressure	Time (Sec)
S1	459.3-458.3 458.3-457.3	346# 346#	580# 1220#	926# 1566#	10
S2	449.3-448.3 448.3-447.3	398# 398#	1780# 2720#	2177# 3118#	14

Federal Road Dist.	State	Fed. Aid Proj. No.	Fiscal Year	Sheet No.	Total Sheets
7	KY	175-8	1960	8	

SOIL TEST DATA

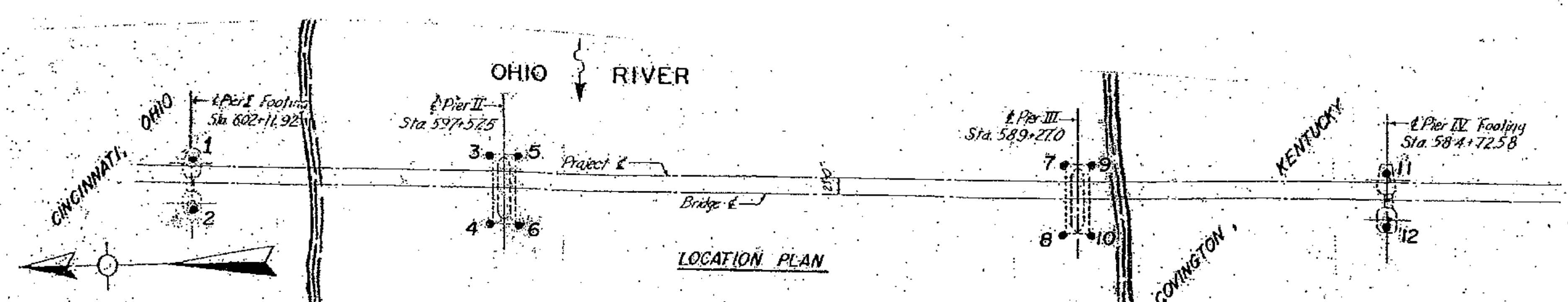
Sample No.	Elevation	% Sand, Silt & Clay	% Moisture	Natural (or Wet) Weight for Cu. ft. lbs./cu. ft.	Unconfined Compressive Strength lbs./Sq. ft.	Failure Strain	Atterburg Limits	Classification U.S.C.S.	Classification AASHO
BORING-11									
S1	485.6	1% Sd. 57% Si. 42% Cl.	32.9	113.5	2130	11.8	44.7% LL 23.9% PL 20.8 Pl. Index	CL-ML	A7-6
BORING-12									
S2	480.3	9.7% Sd. 56.5% Si. 35% Cl.	24.7	123	5130	9.8	38.6% LL 20.5% PL 18.1 Pl. Index	CL	A6

For Notes, see Sheet No. 7.

Drwg. No. 14355

COMMONWEALTH OF KENTUCKY
DEPARTMENT OF HIGHWAYS
FRANKFORT
COUNTY OF KENTON
BORINGS 9, 10, 11, & 12
COVINGTON-CINCINNATI OHIO RIVER BRIDGE
ROAD: COVINGTON-LEXINGTON
STATION 584+74.0 TO 602+10.5 PROJECT NO. 175-8
MODJESKI & MASTERS, ENGINEERS HARRISBURG, PENNSYLVANIA
SUPPLEMENTARY DRAWING No. 8

TEST BORINGS DRILLED IN 1958 BY H.C. NUTTING CO.



H.C. NUTTING COMPANY
CORPORATE OFFICE – 611 LUNKEN PARK DRIVE
CINCINNATI, OHIO 45226
(513) 321-5816
EMPLOYEE OWNED
GEOTECHNICAL, ENVIRONMENTAL AND TESTING ENGINEERS

EXISTING BRIDGE DATA (B-9 To B-12)
CLIENT: PARSONS BRINCKERHOFF
PROPOSED QUEENSGATE ALIGNMENT
BRENT SPENCE BRIDGE REPLACEMENT
W.O. 10974.054

JUNE 2007 | FIGURE 4C

EXHIBIT A-9
QUEENSGATE ALIGNMENT
TEST BORING LOGS (2007)

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory

LOG OF BORING

Date Started 10/4/06 | Sampler: Type SS Dia. 1.375" I.D. | Water Elev. Immediate 462.3 Ft.

Date Completed 10/6/06 | Casing: Length 5' HSA Dia. 3.375" I.D.

Core Barrel: Type NQ/NX Size 2" O.D.

Boring No. B-1 Latitude/Longitude N 39°05'11.9" / W 84°31'29.4" Surface Elev. 517.3 ft

Project Identification: HAM-71/75-0.00/0.22
Brent Spence Queensgate Alignment
Preliminary Geotechnical Study
Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./ RQD	Rec. (ft)	Loss (ft)	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
517.3 / 516.5	0					0 – 0.75 [0.75] ASPHALT (9")										
	2	2 / 4 / 5	0.4		18	0.75 – 7.5 [6.75] Brown, dark brown, and gray silty clay, little to some sand, little gravel, trace cinders and brick fragments (FILL), moist–stiff to medium stiff —Brick fragments from 2.5' to 4'	1	–	–	–	–	–	–	–	14	VISUAL
	4	4 / 6 / 6	0.3		28/8		2	–	–	–	–	–	–	–	–	VISUAL
	6	3 / 4 / 4	0.5		43/4		3	–	–	–	–	–	–	–	16	VISUAL
	8	2 / 3 / 3	1.5		482/290	7.5–11 [2.5] Brown to gray clay, little sand, trace gravel, cinders and brick fragments (FILL), moist–medium stiff	4	3	4	7	32	54	41	20	27	A-7-6(12)
506.3	10	3 / 6	1.0		530/260		5	–	–	–	–	–	–	–	23	VISUAL
504.8	12	10	0.5		69/23	11 – 12.5 [1.5] Brown SILT AND CLAY, little sand, trace root matter (WEATHERED TILL), moist–very stiff	5A	–	–	–	–	–	–	–	16	VISUAL
	14	20 / 11 / 10	1.5		9	12.5 – 25 [12.5] Brown SILT AND CLAY, little sand, trace to little gravel (GLACIAL TILL), moist–very stiff	6	–	–	–	–	–	–	–	18	VISUAL
	16	5 / 10 / 50/0.1'	1.1		13		7	–	–	–	–	–	–	–	16	VISUAL
	18	7 / 10 / 11	1.5		2		8	6	8	7	35	44	33	13	16	A-6a(9)
	20, 22, 24	7 / 6 / 10	1.5		13		9	–	–	–	–	–	–	–	16	VISUAL
492.3	26	3 / 4 / 6	1.5		5	25 – 35 [10] Gray and trace brown CLAY, trace to little sand, trace gravel (GLACIAL TILL), moist–stiff to hard	10	5	3	3	27	62	41	20	19	A-7-6(12)
	28, 30, 32, 34	11 / 15 / 17	1.5		5		11	–	–	–	–	–	–	–	17	VISUAL
482.3	36	6 / 26 / 42	1.5		9	35 – 48.8 [13.8] Gray SILTY CLAY, little to some gravel, trace to little sand (GLACIAL TILL), moist–hard to stiff	12	–	–	–	–	–	–	–	16	VISUAL
	38, 40, 42	10 / 20 / 18	1.5		3		13	26	5	4	15	50	36	17	16	A-6b(9)
	44, 46	3 / 7 / 7	1.5		3		14	–	–	–	–	–	–	–	19	VISUAL
468.5	48	SHELBY TUBE	0.7				15	–	–	–	–	–	–	–	15	VISUAL
466.8	50	11 / 12 / 22	0.6		3	48.8 – 50.5 [1.7] Gray CLAY, little sand, trace gravel (GLACIAL TILL), moist–hard	16	–	–	–	–	–	–	–	18	VISUAL
	52, 54	3 / 7 / 10	1.5		4	50.5 – 55 [4.5] Gray and trace brown SANDY SILT, some sand and gravel, little rock fragments (GLACIAL TILL), very moist–very stiff	17	–	–	–	–	–	–	–	–	VISUAL
462.3	56, 58, 60	70/0.3'	0.2		6	55 – 61.5 [6.5] Gray SHALE, some limestone rock fragments, moist–soft	18	–	–	–	–	–	–	–	–	VISUAL
455.8	62, 64, 66	RQD = 24	5.0	0.0		61.5 – 86.3 [24.8] Interbedded SHALE AND LIMESTONE; Shale is gray, medium tough to tough, calcareous, occupies 52% of matrix, Limestone is light gray, hard, occasionally fossiliferous and argillaceous, evenly distributed in 1/2" to 9 1/2" layers, occupies 48% of matrix.	19	–	–	–	–	–	–	–	–	VISUAL
	68, 70	RQD = 94	5.0	0.0			20	–	–	–	–	–	–	–	–	VISUAL
	72, 74, 76	RQD = 77	4.9	0.1			21	–	–	–	–	–	–	–	–	VISUAL
	78, 80	RQD = 87	5.0	0.0			22	–	–	–	–	–	–	–	–	VISUAL
	82, 84, 86	RQD = 88	5.0	0.0			23	–	–	–	–	–	–	–	–	VISUAL
431.0	88, 90	RQD = 97	5.0	0.0		86.3 – 101.5 [15.2] Gray SHALE, medium tough to tough, calcareous, occasional limestone seams (less than 1/2")	24	–	–	–	–	–	–	–	–	VISUAL
	92, 94, 96	RQD = 96	5.0	0.0			25	–	–	–	–	–	–	–	–	VISUAL
	98, 100	RQD = 100	5.0	0.0			26	–	–	–	–	–	–	–	–	VISUAL
415.8	102, 104	RQD = 95	5.0	0.0		101.5 – 106.5 [5] Interbedded SHALE AND LIMESTONE; Shale is gray, tough, calcareous, occupies 54% of matrix, Limestone is light gray, hard, fossiliferous, evenly distributed in 2" to 8 1/2" layers, occupies 46% of matrix.	27	–	–	–	–	–	–	–	–	VISUAL
410.8	106					Boring completed at 106.5 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00–0.42mm, Fine Sand = 0.42–0.074mm, Silt = 0.074–0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory

LOG OF BORING

Date Started 9/28/06 Sampler: Type SS Dia. 1.375" I.D. Water Elev. Immediate 465.5 Ft. Project Identification: HAM-71/75-0.00/0.22
Date Completed 10/3/06 Casing: Length 5' HSA Dia. 3.375" I.D. Brent Spence Queensgate Alignment
Core Barrel: Type NQ/NX Size 2" O.D. Preliminary Geotechnical Study
Boring No. B-2 Latitude/Longitude N 39°05' 20.0" / W 84°31'31.7" Surface Elev. 487.5 ft Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./ RQD	Rec. (ft)	Loss (ft)	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	LL	P.I.	W.C.	ODOT Class
487.5	0	2 / 5 / 5	1.5		<1	0 – 2.5 [2.5] Dark brown sandy silt and brick fragments, some sand, little cinders (FILL), moist–stiff/loose	1	-	-	-	-	-	-	-	-	VISUAL
485.0	2	9	0.5		<1	2.5 – 3 [0.5] Dark brown sandy silt, trace cinders and brick fragments (FILL), moist–very stiff	2	-	-	-	-	-	-	-	13	VISUAL
484.5	4	10 / 7	1.0		<1		2A	-	-	-	-	-	-	-	13	VISUAL
	6	6 / 4 / 6	1.5		<19	3 – 7.5 [4.5] Brown CLAY, trace sand and root matter (ALLUVIUM), slightly moist–very stiff to stiff	3	0	0	10	34	56	44	19	19	A-7-6(12)
480.0	8	5 / 6 / 6	1.5		9	7.5 – 17.5 [10] Brown SILTY CLAY, trace to little sand (ALLUVIUM), moist–medium stiff to stiff	4	-	-	-	-	-	-	-	16	VISUAL
	10	5 / 4 / 4	1.5		19/<1		5	0	0	9	44	47	36	16	18	A-6b(10)
	12	2 / 3 / 4	1.5		1		6	-	-	-	-	-	-	-	21	VISUAL
	14, 16	2 / 5 / 5	1.5		67/<1		7	-	-	-	-	-	-	-	22	VISUAL
470.0	18	2 / 3 / 4	1.5		21/<1	17.5 – 30 [12.5] Brown SILT AND CLAY, some sand (ALLUVIUM), very moist–medium stiff to very soft —Occasional wet sand/silt seams	8	-	-	-	-	-	-	-	22	VISUAL
	20	SHELBY TUBE	2.0				9	-	-	-	-	-	-	-	23	VISUAL
	22	2 / 1 / 2	1.5		19/<1		10	-	-	-	-	-	-	-	23	VISUAL
	24, 26, 28	1/1.5'	1.5		4		11	0	1	20	39	40	31	12	29	A-6a(9)
457.5	30, 32	1/1.0' / 2	1.5		9	30 – 42 [12] Gray SANDY SILT (ALLUVIUM), very moist–very soft to medium stiff —Occasional wet sand/silt seams	12	-	-	-	-	-	-	-	25	VISUAL
445.5	34, 36, 38	2 / 2 / 3	1.5		9		13	0	0	30	36	34	30	10	32	A-4a(7)
	40	SHELBY TUBE	2.0				14	-	-	-	-	-	-	-	34	VISUAL
445.5	42, 44	10 / 15 / 15	0.6		21/<1	42 – 45 [3] Brown GRAVEL WITH SAND AND SILT, little rock fragments (OUTWASH), wet–medium dense	15	-	-	-	-	-	-	-	-	VISUAL
442.5	46, 48	2 / 1/1.0'	1.0		2	45 – 50 [5] Brown COARSE AND FINE SAND, little gravel (OUTWASH), wet–very loose	16	-	-	-	-	-	-	-	-	VISUAL
437.5	50, 52, 54	24 / 7 / 11	0.5		7	50 – 55 [5] Brown GRAVEL WITH SAND AND SILT, little rock fragments (OUTWASH), wet–medium dense	17	51	12	8	-29-		0	0	-	A-2-4(0)
432.5	56, 58	5 / 35 / 8	1.0		19/<1	55 – 60 [5] Brown GRAVEL WITH SAND (OUTWASH), wet–dense	18	-	-	-	-	-	-	-	-	VISUAL
427.5	60	5	0.5		3	60 – 60.5 [0.5] Brown GRAVEL WITH SAND AND SILT (OUTWASH), wet–medium dense	19	25	17	27	-31-		0	0	-	A-2-4(0)
427.0	62, 64	32 / 70/0.2'	0.5		35/13	60.5 – 65 [4.5] Gray SHALE, moist–soft	19A	-	-	-	-	-	-	-	-	VISUAL
422.5	66	RQD = 0	0.8	0.3		65 – 81.3 [16.3] Interbedded SHALE AND LIMESTONE; Shale is gray, medium tough to tough, calcareous, occupies 71% of matrix, Limestone is light gray, hard, fossiliferous and argillaceous, evenly distributed in 1/2" to 9" layers, occupies 29% of matrix.	20	-	-	-	-	-	-	-	-	VISUAL
	68	RQD = 20	4.0	1.0			21	-	-	-	-	-	-	-	-	VISUAL
	70, 72	RQD = 25	5.0	0.0			22	-	-	-	-	-	-	-	-	VISUAL
	74, 76, 78, 80	RQD = 72	5.0	0.0			23	-	-	-	-	-	-	-	-	VISUAL
406.2	82	RQD = 70	5.0	0.0		81.3 – 83.6 [2.3] Gray SHALE, tough, calcareous, occasional limestone seams (less than 1/4"), soft zone from 83.3' to 83.6'	24	-	-	-	-	-	-	-	-	VISUAL
403.9	84, 86, 88	RQD = 40	4.7	0.3		83.6 – 93.1 [9.5] Interbedded SHALE AND LIMESTONE; Shale is gray, tough, calcareous, occupies 52% of matrix, Limestone is light gray, hard, occasionally argillaceous and shaley, evenly distributed in 1/4" to 8" layers, occupies 48% of matrix.	25	-	-	-	-	-	-	-	-	VISUAL
	90, 92	RQD = 72	5.0	0.0			26	-	-	-	-	-	-	-	-	VISUAL
394.4	94					93.1 – 94.9 [1.8] Light gray LIMESTONE, hard, argillaceous, occasionally fossiliferous, occasional shale seams (less than 1/4")										
392.6	96, 98	RQD = 68	4.8	0.2		94.9 – 110.1 [15.2] Interbedded SHALE AND LIMESTONE; Shale is gray, tough, calcareous, occupies 51% of matrix, Limestone is light gray, hard, occasionally argillaceous, shaley, and fossiliferous, evenly distributed in 1/4" to 10" layers, occupies 49% of matrix.	27	-	-	-	-	-	-	-	-	VISUAL
	100, 102, 104	RQD = 92	5.0	0.1			28	-	-	-	-	-	-	-	-	VISUAL
	106, 108	RQD = 90	3.8	0.3			29	-	-	-	-	-	-	-	-	VISUAL
377.4	110					Boring completed at 110.1 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00–0.42mm, Fine Sand = 0.42–0.074mm, Silt = 0.074–0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory
LOG OF BORING

5/6

Date Started 11/30/06 — Sampler: Type SS — Dia. 1.375" — Water Elev. 453.0 Ft.
Date Completed 11/30/06 — Casing: Length 5' HSA — Dia. 3.375" I.D.
Core Barrel: Type NQ/NX — Size 2" O.D.
Boring No. B-3 — Latitude/Longitude N 39°05'24.3" / W 84°31'32.9" — Surface Elev. 458.0 ft

Project Identification: HAM-71/75-0.00/0.22
Brent Spence Queensgate Alignment
Preliminary Geotechnical Study
Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./ RQD	Rec. (ft)	Loss (ft)M	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
458.0	0					0 – 5 [5] Barge Platform										
453.0 ▽	5					5 – 29 [24] Water (Ohio River)										
429.0	29	70 / 50/0.3'	0.8		7/3	29 – 30.5 [1.5] Gray SHALE, some limestone fragments/layers, wet–medium tough	1									Visual
427.5	30.5	RQD = 0	1.2	3.3		30.5 – 35.2 [4.7] Dark gray SHALE, soft to medium tough, calcareous, highly fractured	2									Visual
422.8	35.2	RQD = 64	9.6	0.4		35.2 – 75 [39.8] Interbedded SHALE AND LIMESTONE; Shale is gray, medium to very tough, calcareous, occasional soft/fractured zones up to 2" thick, occupies 69% of matrix. Limestone is light gray and gray, hard, occasionally fossiliferous and shaley, vertical fracture @ 47.6', very steep fracture @ 57.3', 1/16" to 1/8" cavities @ 72.2', evenly distributed in 1/2" to 8 1/2" layers, occupies 31% of matrix.	3									Visual
	45	RQD = 70	10.0	0.0			4									Visual
	55	RQD = 86	9.8	0.2			5									Visual
	65	RQD = 91	10.0	0.0			6									Visual
383.0	75					Boring completed at 75.0 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00–0.42mm, Fine Sand = 0.42–0.074mm, Silt = 0.074–0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory
LOG OF BORING

Date Started 11/29/06
Date Completed 11/30/06
Boring No. B-4

Sampler: Type SS Dia. 1.375"
Casing: Length 5' HSA Dia. 3.375" I.D.
Core Barrel: Type NQ/NX Size 2" O.D.
Latitude/Longitude N 39°05'37.4" / W 84°31'35.9"

Water Elev. 453.0 Ft.
Surface Elev. 458.0 ft

Project Identification: HAM-71/75-0.00/0.22
Brent Spence Queensgate Alignment
Preliminary Geotechnical Study
Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./ RQD	Rec. (ft)	Loss (ft)M	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
458.0	0					0 – 5 [5] Barge platform										
453.0 ▽	5					5 – 32.5 [27.5] Water (Ohio River)										
425.5	32.5	41	0.5			32.5 – 33 [0.5]	1A									Visual
425.0	33	36 / 101	1.0		80/3	Brown GRAVEL WITH SAND, some silt (OUTWASH), saturated–very dense 33 – 35.3 [2.3]	1B									Visual
		50/0.3'				Gray SHALE, little limestone fragments, wet, medium tough	2									Visual
422.7	35.3	RQD = 0	1.2	0.9		35.3 – 82.3 [47] Interbedded SHALE AND LIMESTONE; Shale is dark gray to gray, medium to very tough, calcareous, occasional soft fractured zones up to 2" thick, occupies 29% of matrix. Limestone is light gray, hard, occasionally to frequently shaley, fossiliferous from 80' to 82.3', evenly distributed in 1/4" to 10" layers, occupies 29% of matrix.	3									Visual
	38	RQD = 72	5.0	0.0			4									Visual
	43	RQD = 73	10.0	0.0			5									Visual
	53	RQD = 61	9.8	0.2			6									Visual
	63	RQD = 82	4.9	0.1			7									Visual
	68	RQD = 84	4.4	0.6			8									Visual
	73	RQD = 87	10.0	0.0			9									Visual
375.7	82.3					Boring completed at 82.3 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00–0.42mm, Fine Sand = 0.42–0.074mm, Silt = 0.074–0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory
LOG OF BORING

Date Started: 10/4/06 | Sampler: Type: SS | Dia. 1.375" I.D. | Water Elev. Immediate 461.5 Ft.
Date Completed: 10/6/06 | Casing: Length: 5" HSA | Dia. 3.375" I.D.
Core Barrel: Type: NQ/NX | Size 2" O.D.
Boring No. B-5 | Latitude/Longitude N 39°05'36.6" / W 84°31'35.8" | Surface Elev. 491.5 ft

Project Identification: HAM-71/75-0.00/0.22
Brent Spence Queensgate Alignment
Preliminary Geotechnical Study
Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./ RQD	Rec. (ft)	Loss (ft)	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	LL	PL	W.C.	ODOT Class
491.5	0	10 / 20 / 30	1.5		NR	0 – 5 [5] Black coal (FILL), moist–very dense	1	–	–	–	–	–	–	–	–	VISUAL
		50	0.5		NR		2	–	–	–	–	–	–	–	–	VISUAL
486.5	5	50	0.5		NR	5 – 10 [5] Gray gravel and red brick fragments, some sand (FILL), moist–very dense to medium dense	3	–	–	–	–	–	–	–	–	VISUAL
		3 / 5 / 7	0.5		14		4	–	–	–	–	–	–	–	–	VISUAL
481.5	10	6 / 4 / 4	1.5		90	10 – 20 [10] Black coal and gray gravel, trace brick fragments (FILL), moist–medium dense to very loose	5	–	–	–	–	–	–	–	–	VISUAL
		5 / 7 / 7	1.5				6	–	–	–	–	–	–	–	–	VISUAL
		2 / 2 / 2	1.5		<1		7	–	–	–	–	–	–	–	–	VISUAL
		3 / 7 / 3	1.5		60		8	–	–	–	–	–	–	–	–	VISUAL
471.5	20	1 / 2 / 3	1.5		50	20 – 25 [5] Dark brown sandy silt, trace gravel, cinders, and brick fragments (FILL), moist–medium stiff —Petroleum odor from 20' to 21.5'	9	–	–	–	–	–	–	–	20	VISUAL
466.5	25	3 / 2 / 2	1.5		30	25 – 30 [5] Black coal, trace gravel (FILL), moist–very loose	10	–	–	–	–	–	–	–	–	VISUAL
461.5 ▽	30	2 / 1 / 2	1.5		370/90	30 – 35 [5] Dark brown sandy silt, trace organics and cinders (FILL), very moist–soft —Organic odor	11	6	19	27	27	21	27	3	29	A-4a(3)
456.5	35	2 / 2 / 2	1.5		560/105	35 – 40 [5] Dark brown and gray SILT AND CLAY, some sand, trace gravel and organics (ALLUVIUM), very moist–medium stiff	12	–	–	–	–	–	–	–	32	VISUAL
		SHELBY TUBE	2.0				13	5	8	14	36	37	36	12	32	A-6a(9)
451.5	40	2 / 2 / 3	1.5		3040/912	40 – 45 [5] Dark gray SILT AND CLAY, some organics and wood fragments (ALLUVIUM), moist–medium stiff —Loss-on-ignition=21%	14	–	–	–	–	–	–	–	60	VISUAL
446.5	45	3 / 4 / 6	1.5		4382/5105	45 – 55 [10] Dark brown and gray SANDY SILT, trace organics (ALLUVIUM), very moist–stiff to medium stiff	15	–	–	–	–	–	–	–	32	VISUAL
		2 / 2 / 4	1.5		576/705		16	0	1	32	34	33	28	6	34	A-4a(8)
436.5	55	4 / 3 / 7	1.54		2015/2660	55 – 60 [5] Dark brown SANDY SILT, little gravel, trace wood fragments (ALLUVIUM), very moist–stiff —Loss-on-ignition=6%	17	–	–	–	–	–	–	–	33	VISUAL
431.5	60	2 / 3 / 4	1.5		60/15	60 – 65 [5] Brown GRAVEL and sand (OUTWASH), wet loose	18	50	21	16	–12–		0	0	–	A-1-a(0)
426.5	65	4 / 43 / 27	1.5		315/133	65 – 75 [10] Brown to gray GRAVEL, some sand (OUTWASH), wet–very dense to medium dense	19	–	–	–	–	–	–	–	–	VISUAL
		11 / 14 / 16	1.5		126/58		20	67	13	9	–11–		0	0	–	A-1-a(0)
							21	–	–	–	–	–	–	–	–	VISUAL
416.5	75	50/0.0"	0		NR											
		RQD = 24	1.0	0.7		75 – 99 [24] Interbedded SHALE AND LIMESTONE: Shale is gray, medium tough to tough, calcareous, completely softened from 79.6' to 79.8', occupies 55% of matrix. Limestone is light gray, hard, frequently fossiliferous and argillaceous, evenly distributed in 1/4" to 7" layers, occupies 45% of matrix.	22	–	–	–	–	–	–	–	–	VISUAL
		RQD = 59	9.5	0.5			23	–	–	–	–	–	–	–	–	VISUAL
		RQD = 92	10.0	0.0			24	–	–	–	–	–	–	–	–	VISUAL
		RQD = 92	9.9	0.1			25	–	–	–	–	–	–	–	–	VISUAL
392.5	99					99 – 101.2 [2.2] Gray SHALE, tough, calcareous										
390.3	101.2					101.2 – 120.7 [19.5] Interbedded SHALE AND LIMESTONE: Shale is gray, tough, calcareous, softened from 109.5' to 109.8', occupies 57% of matrix. Limestone is light gray, hard, occasionally fossiliferous and argillaceous, evenly distributed in 1/2" to 12 1/2" layers, occupies 43% of matrix.										
		RQD = 84	9.9	0.1			26	–	–	–	–	–	–	–	–	VISUAL
		RQD = 80	4.0	1.0			27	–	–	–	–	–	–	–	–	VISUAL
370.8	120					Boring completed at 120.7 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00–0.42mm, Fine Sand = 0.42–0.074mm, Silt = 0.074–0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

W.O. 10974.054

State of Ohio
Department of Transportation
Division of Highways
Testing Laboratory
LOG OF BORING

Date Started 10/2/06
Date Completed 10/3/06
Boring No. B-6   Latitude/Longitude N 39°05'45.3" / W 84°31'37.9"

Sampler: Type SS   Dia. 1.375" I.D.
Casing: Length 5' HSA   Dia. 3.375" I.D.
Core Barrel: Type NQ/NX   Size 2" O.D.

Water Elev. Immediate 462.1 Ft.
Surface Elev. 487.1 ft

Project Identification: HAM-71/75-0.00/0.22
Brent Spence Queensgate Alignment
Preliminary Geotechnical Study
Cincinnati/Covington, OH/KY

Elev. (ft)	Depth (ft)	Std. Pen./RQD	Rec. (ft)	Loss (ft)	FID (ppm v/v)	Description	Sample No.	% Agg	% C.S.	% F.S.	% Silt	% Clay	L.L.	P.I.	W.C.	ODOT Class
487.1	0	7 / 24 / 8	1.5		22/12	0 – 5 [5] Dark brown silt and clay with brick fragments, little sand and cinders, trace organics (FILL), moist–hard	1	–	–	–	–	–	–	–	18	VISUAL
		17 / 21 / 21	1.5		13		2	–	–	–	–	–	–	–	–	VISUAL
482.1		3 / 2 / 3	1.5		6	5 – 12.5 [7.5] Dark brown sandy silt, trace gravel, cinders, and brick fragments (FILL), moist–medium stiff	3	3	1	24	40	32	24	7	18	A-4a(7)
		SHELBY TUBE	2.0				4	–	–	–	–	–	–	–	27	VISUAL
		2 / 2 / 5	1.5		<1		5	–	–	–	–	–	–	–	27	VISUAL
474.6		2 / 4 / 5	0.3		6	12.5 – 18 [5.5] Brown and trace gray SILT AND CLAY, trace sand and gravel (ALLUVIUM), moist–stiff to medium stiff	6	1	1	4	42	52	37	15	25	A-6a(10)
		2 / 3 / 5	1.5		<1		7	–	–	–	–	–	–	–	24	VISUAL
		SHELBY TUBE	2.0			18 – 25 [7] Brown SANDY SILT (ALLUVIUM), moist–medium stiff	8	0	0	28	39	33	27	9	24	A-4a(7)
467.1	20	2 / 2 / 3	1.5		<1		9	–	–	–	–	–	–	–	26	VISUAL
462.1 ▽		WOH / WOH / 1	1.5		4	25 – 30 [5] Brown SANDY SILT (ALLUVIUM), wet–very loose	10	0	1	43	–56–		0	0	–	A-4a(4)
457.1	30	2 / 1 / 1	1.5		<1	30 – 35 [5] Brown SANDY SILT (ALLUVIUM), wet–very loose	11	0	1	17	46	36	29	9	29	A-4a(8)
452.1	36	2 / 1 / 2	1.5		<1	35 – 40 [5] Gray SANDY SILT (ALLUVIUM), very moist–soft	12	–	–	–	–	–	–	–	28	VISUAL
447.1	40	3 / 3 / 4	0.8		<1	40 – 50 [10] Brown GRAVEL WITH SAND, little clay (OUTWASH), wet–loose to medium dense	13	–	–	–	–	–	–	–	–	VISUAL
	46	8 / 6 / 7	1.5		<1		14	50	29	10	–11–		0	0	–	A-1-b(0)
437.1	50	11 / 11 / 18	1.5		<1	50 – 60 [10] Brown COARSE AND FINE SAND, little silt and clay, trace gravel (OUTWASH), wet–medium dense	15	17	25	41	–17–		0	0	–	A-3a(0)
	56	15 / 14 / 11	1.2		3		16	–	–	–	–	–	–	–	–	VISUAL
427.1	60	80	0.5		65/5	60 – 60.5 [0.5] Gray SHALE, moist–soft	17	–	–	–	–	–	–	–	–	VISUAL
426.6		RQD = 0	0.5	0.8		60.5 – 82.8 [22.3] Interbedded SHALE AND LIMESTONE; Shale is gray, medium tough to tough, calcareous, occupies 75% of matrix, Limestone is light gray, occasionally fossiliferous and argillaceous, evenly distributed in 1/2" to 8" layers, occupies 25% of matrix.	18	–	–	–	–	–	–	–	–	VISUAL
		RQD = 53	9.3	0.7			19	–	–	–	–	–	–	–	–	VISUAL
	72	RQD = 90	9.2	0.8			20	–	–	–	–	–	–	–	–	VISUAL
404.3	82	RQD = 100	10.0	0.0			21	–	–	–	–	–	–	–	–	VISUAL
						82.8 – 85.6 [2.8] Gray SHALE, tough, calcareous										
401.5	86					85.6 – 106.3 [20.7] Interbedded SHALE AND LIMESTONE; Shale is gray, tough, calcareous, softened from 93.5' to 93.9', occupies 40% of matrix, Limestone is light gray, hard, frequently fossiliferous, shaley, and argillaceous, evenly distributed in 1/2" to 10" layers, occupies 60% of matrix.										
	92	RQD = 73	9.7	0.3			22	–	–	–	–	–	–	–	–	VISUAL
	102	RQD = 100	4.5	0.0			23	–	–	–	–	–	–	–	–	VISUAL
380.8	106					Boring completed at 106.3 feet										

Particle Sizes: Agg => 2.00mm, Coarse Sand = 2.00-0.42mm, Fine Sand = 0.42-0.074mm, Silt = 0.074-0.005mm, Clay =< 0.005mm.

Form TE-151 Revised 9/94

EXHIBIT A-10

ENVIRONMENTAL SCREENING RESULTS

ENIVRONMENTAL SCREENING RESULTS

Boring	Sample Number	Reading (ppm-V/V)
L-1	1	2
L-1	2	<1
L-1	3	<1
L-1	4	-
L-1	5	<1
L-1	6	57/53
L-1	6A	53/13
L-1	7	<1
L-1	7A	<1
L-1	8	1
L-1	9	10/6
L-1	10	8
L-1	11	5
L-1	12	56/21
L-1	13	67/20
L-1	14	19/15
L-1	15	1
L-1	16	45/17
L-1	17	3
L-1	18	5
L-1	19	45/18
L-1	20	1
L-1	21	<1
L-1	22	25/14
L-1	23	77/27
L-1	24	14/12
L-1	25	9
L-1	26	69/16
L-1	27	15/5
L-1	28	8
L-1A	1	<1
L-1A	2	2
L-1A	3	23/22
L-1A	4	-
L-1A	5	45/23
L-1A	6	5

Boring	Sample Number	Reading (ppm-V/V)
L-1A	7	10/13
L-1A	8	-
L-1A	9	9
L-1A	9A	<1
L-1A	10	5
L-1A	11	<1
L-1A	12	82/1
L-1A	13	27/39
L-1A	14	<1
L-1A	15	<1
L-1A	16	<1
L-1A	17	<1
L-1A	18	24/22
L-1A	19	<1
L-1A	20	14/10
L-1A	21	16/1
L-1A	22	<1
L-1A	23	<1
L-1A	24	30/17
L-1A	25	<1
L-1A	26	20/13
L-1A	27	5
L-1A	28	55/17
L-1A	28A	37/40
L-2A	1	5
L-2A	2	3
L-2A	3	158/27
L-2A	4	16/<1
L-2A	5	3
L-2A	6	9
L-2A	7	-
L-2A	8	92/25
L-2A	9	30/20
L-2A	10	3900/3000
L-2A	11	430/514
L-2A	12	-

Boring	Sample Number	Reading (ppm-V/V)
L-2A	13	10/8
L-2A	14	20/9
L-2A	15	20/6
L-2A	16	<1
L-2A	17	8
L-2A	18	13/1
L-2A	19	<1
L-2A	20	77/35
L-2A	21	16/17
L-2A	22	33/5
L-2A	23	22/19
L-2A	24	36/22
L-2A	25	27/25
L-2A	26	32/16
L-2A	27	42/26
L-2A	28	39/17
L-2A	29	43/19
L-2A	30	6
L-3	1	-
L-3	2	2730/1100
L-3	3	2730/850
L-3	4	-
L-3	5	-
L-3	6	15/4
L-3	7	3
L-3	8	<1
L-3	9	<1
L-3	10	13/3
L-3	11	6
L-3	12	11/2
L-3	13	16/5
L-3	14	19/14
L-3	15	7
L-3	16	13/4
L-3	17	-
L-4	1	648/114
L-4	2	5
L-4	3	<1

Boring	Sample Number	Reading (ppm-V/V)
L-4	4	447/112
L-4	5	47/19
L-4	6	49/23
L-4	7	-
L-4	8	181/48
L-4	9	1914/165
L-4	10	3900/1500
L-4	11	-
L-4	12	1270/819
L-4	13	9
L-4	14	<1
L-4	15	125/69
L-4	16	161/65
L-4	17	2
L-4	18	25/17
L-4	19	82/31
L-4	20	<1
L-4	21	29/13
L-4	22	34/15
L-4	23	11/5
L-4	24	30/10
L-4	25	75/17
L-4	26	-
L-4	27	-
L-5	1	<1
L-5	2	<1
L-5	3	<1
L-5	4	<1
L-5	5	2
L-5	6	2
L-5	7	14/33
L-5	8	108/78
L-5	9	2
L-5	10	15/12
L-5	11	12/7
L-5	12	40/22
L-5	13	13/3
L-5	14	26/7

Boring	Sample Number	Reading (ppm-V/V)
L-5	15	43/20
L-5	16	68/23
L-5	17	5
L-5	18	23/21
L-5	19	31/14
L-5	20	40/29
L-5	21	58/38
L-5	22	-
L-6	1	<1
L-6	2	-
L-6	3	<1
L-6	4	<1
L-6	5	<1
L-6	6	-
L-6	7	2
L-6	8	7
L-6	9	<1
L-6	10	-
L-6	11	<1
L-6	12	<1
L-6	13	<1
L-6	14	<1
L-6	15	<1
L-6	16	<1
L-6	17	<1
L-6	18	-
L-6	19	<1
L-6	20	-
L-6	21	14/5
L-6	22	-
L-7	1	340/190
L-7	2	400/364
L-7	3	54/31
L-7	4	No sample
L-7	5	<1
L-7	6	2
L-7	7	No sample
L-7	8	7

Boring	Sample Number	Reading (ppm-V/V)
L-7	9	2750/2700
L-7	10	2800/2600
L-7	11	No Sample
L-7	12	74/74
L-7	13	2
L-7	14	<1
L-7	15	62/40
L-7	16	50/20
L-7	17	103/66
L-7	18	3
L-7	19	73/42
L-7	20	<1
R-1	1	1
R-1	2	8
R-1	3	-
R-1	4	2
R-1	5	-
R-1	6	1
R-1	7	2
R-1	8	5
R-1	9	6
R-1	10	-
R-1	11	<1
R-1	12	2
R-1	13	8
R-1	14	8
R-1	15	7
R-1	16	
R-2	1	<1
R-2	2	10/6
R-2	3	7
R-2	4	11/5
R-2	5	-
R-2	6	18/11
R-2	7	-
R-2	8	8
R-2	9	9
R-2	10	4

Boring	Sample Number	Reading (ppm-V/V)
R-2	11	24/3
R-2	12	13/11
R-2	13	13/10
R-2	14	3
R-2	15	40/13
R-2	16	64/39
R-3	1	-
R-3	2	-
R-3	3	3
R-3	4	2
R-3	5	-
R-3	6	2
R-3	7	8
R-3	8	1
R-3	9	<1
R-3	10	3
R-3	11	-
R-3	12	3
R-3	13	1
R-3	14	2
R-3	15	5
R-3	16	14/4
R-3	17	-
R-4	1	<1
R-4	2	11/7
R-4	3	4700/3900
R-4	4	42/63
R-4	5	3
R-4	6	3
R-4	7	6
R-4	8	<1
R-4	9	-
R-4	10	<1
R-4	11	12/14
R-4	12	9
R-4	13	16/11
R-4	14	19/10
R-4	15	16/12

Boring	Sample Number	Reading (ppm-V/V)
R-4	16	1
R-5	1	5
R-5	2	2/4800
R-5	3	2/6000
R-5	4	5844/-
R-5	5	-
R-5	6	3620/5800
R-5	7	5700/5900
R-5	8	154/196
R-5	9	2
R-5	10	52/34
R-5	11	12/13
R-5	12	104/62
R-5	13	62/32
R-5	14	67/33
R-5	15	95/51
R-5	16	2
R-5	17	20.7
R-5	18	28/23
R-5	19	27/19
R-6	1	1
R-6	2	5
R-6	3	-
R-6	4	13/10
R-6	5	6
R-6	6	<1
R-6	7	-
R-6	8	1
R-6	9	70/76
R-6	10	2
R-6	11	25/17
R-6	12	58/26
R-6	13	9
R-6	14	59/28
R-6	15	19/7
R-6	16	40/8
R-6	17	38/10
R-6	18	37/14

Boring	Sample Number	Reading (ppm-V/V)
R-6	19	39/21
R-6	20	43/18
R-6	21	93/43
R-7	2	3500/3000
R-7	3	8300/8300
R-7	4	2500/3576
R-7	5	2100/2200
R-7	6	-
R-7	7	-
R-7	8	21/30
R-7	9	-
R-7	10	28/61
R-7	11	37/101
R-7	12	29/51
R-7	13	64/84
R-7	14	79/142
R-7	15	35/39
R-7	16	50/48
R-7	17	40/27

EXHIBIT A-11

GEOVISION SUSPENSION LOGGING REPORT

SUSPENSION PS VELOCITIES
BORINGS L-1, L-4 AND R-2A

BRENT SPENCE BRIDGE REPLACEMENT
CINCINNATI, OHIO

Report 10261-01 rev a

September 22, 2010

SUSPENSION PS VELOCITIES
BORINGS L-1, L-4 AND R-2A

BRENT SPENCE BRIDGE REPLACEMENT
CINCINNATI, OHIO

Report 10261-01 rev a

September 22, 2010

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APPENDICES

INTRODUCTION

Boring geophysical measurements were collected in three cased borings for the Brent Spence Bridge Replacement project in Cincinnati, Ohio. Geophysical data acquisition was performed in two on-land borings on August 3, 20010 by Victor Gonzalez and one boring in the Ohio River on September 2, 2010 by Chuck Carter of **GEO***Vision*. Data analysis was performed by Victor Gonzalez and Chuck Carter and reviewed by Robert Steller of **GEO***Vision*. Report preparation was performed by Victor Gonzalez and reviewed by Robert Steller of **GEO***Vision*. The work was performed under subcontract with H.C. Nutting (HCN) with Bill Meadows serving as the point of contact for HCN.

This report describes the field measurements, data analysis, and results of this work.

SCOPE OF WORK

This report presents the results of boring geophysical measurements collected on August 3, 2010 on land and on September 2, 2010 in the Ohio River in three 3-inch PVC cased borings, as detailed in Table 1. The purpose of the study was to acquire shear wave velocities and compressional wave velocities as a function of depth.

BORING DESIGNATION	DATES LOGGED	ELEVATION - FEET MSL [1]	COORDINATES – FEET [1]	
			NORTHING	EASTING
L-1	08/03/2010	494.59	39.093833610	84.522929480
L-4	08/03/2010	479.97	39.088805640	84.523275430
R-2A	09/02/2010	457.64 (DECK LEVEL)	NA	NA

[1] Coordinates and elevations provided by HCN

Table 1 Boring locations and logging dates

The OYO Suspension Logging System was used to obtain in-situ horizontal shear and compressional wave velocity measurements at 1.6-foot intervals. The acquired data were analyzed and a profile of velocity versus depth was produced for both compressional and horizontally polarized shear waves.

A detailed reference for the velocity measurement techniques used in this study is:

> Guidelines for Determining Design Basis Ground Motions, Report TR-102293, Electric Power Research Institute, Palo Alto, California, November 1993, Sections 7 and 8.

INSTRUMENTATION

Suspension Instrumentation

Suspension soil velocity measurements were performed in all borings using the PS suspension logging system, manufactured by OYO Corporation. This system directly determines the average velocity of a 3.3-foot high segment of the soil column surrounding the boring of interest by measuring the elapsed time between arrivals of a wave propagating upward through the soil column. The receivers that detect the wave, and the source that generates the wave, are moved as a unit in the boring producing relatively constant amplitude signals at all depths.

The suspension system probe consists of a combined reversible polarity solenoid horizontal shear-wave source (S_H) and compressional-wave source (P), joined to two biaxial receivers by a flexible isolation cylinder, as shown in Figure 1. The separation of the two receivers is 3.3 feet, allowing average wave velocity in the region between the receivers to be determined by inversion of the wave travel time between the two receivers. The total length of the probe as used in these surveys is 19 feet, with the center point of the receiver pair 12.1 feet above the bottom end of the probe.

The probe receives control signals from, and sends the receiver signals to, instrumentation on the surface via an armored 4 or 7 conductor cable. The cable is wound onto the drum of a winch and is used to support the probe. Cable travel is measured to provide probe depth data, using a 3.28-foot circumference sheave fitted with a digital rotary encoder.

The entire probe is suspended in the boring by the cable, therefore, source motion is not coupled directly to the boring walls; rather, the source motion creates a horizontally propagating impulsive pressure wave in the fluid filling the boring and surrounding the source. This pressure wave is converted to P and S_H-waves in the surrounding soil and rock as it impinges upon the wall of the boring. These waves propagate through the soil and rock surrounding the boring, in turn causing a pressure wave to be generated in the fluid surrounding the receivers as the soil waves pass their location. Separation of the P and S_H-waves at the receivers is performed using the following steps:

1. Orientation of the horizontal receivers is maintained parallel to the axis of the source, maximizing the amplitude of the recorded S_H -wave signals.
2. At each depth, S_H-wave signals are recorded with the source actuated in opposite directions, producing S_H-wave signals of opposite polarity, providing a characteristic S_H-wave signature distinct from the P-wave signal.
3. The 6.3-foot separation of source and receiver 1 permits the P-wave signal to pass and damp significantly before the slower S_H-wave signal arrives at the receiver. In saturated soils, the received P-wave signal is typically of much higher frequency than the received S_H -wave signal, permitting additional separation of the two signals by low pass filtering.
4. Direct arrival of the original pressure pulse in the fluid is not detected at the receivers because the wavelength of the pressure pulse in fluid is significantly greater than the dimension of the fluid annulus surrounding the probe, preventing significant energy transmission through the fluid medium.

In operation, a distinct, repeatable pattern of impulses is generated at each depth as follows:

1. The source is fired in one direction producing dominantly horizontal shear with some vertical compression, and the signals from the horizontal receivers situated parallel to the axis of motion of the source are recorded.
2. The source is fired again in the opposite direction and the horizontal receiver signals are recorded.
3. The source is fired again and the vertical receiver signals are recorded. The repeated source pattern facilitates the picking of the P and S_H-wave arrivals; reversal of the source changes the polarity of the S_H-wave pattern but not the P-wave pattern.

The data from each receiver during each source activation is recorded as a different channel on the recording system. The Suspension PS system has six channels (two simultaneous recording channels), each with a 1024 sample record. The recorded data are displayed as six channels with a common time scale. Data are stored on disk for further processing. Up to 8 sampling sequences can be summed to improve the signal to noise ratio of the signals.

Review of the displayed data on the recorder or computer screen allows the operator to set the gains, filters, delay time, pulse length (energy), sample rate, and summing number to optimize the quality of the data before recording. Verification of the calibration of the Suspension PS digital recorder is performed every twelve months using a NIST traceable frequency source and counter, as outlined in Appendix B.

MEASUREMENT PROCEDURES

Suspension Measurement Procedures

Three 4 7/8-inch borings containing 3-inch schedule 40 PVC casing filled with fresh water were logged. Measurements followed the **GEO***Vision* Procedure for P-S Suspension Seismic Velocity Logging, revision 1.4. Prior to each logging run, the probe was positioned with the top of the probe at the top of the barge deck, ground surface, or other stationary reference point. Subsequently, the electronic depth counter was set to 6.56 feet, the distance between the mid-point of the receiver and the top of the probe, minus the height of the stationary reference point, as verified with a tape measure, and recorded on the field logs. The probe was lowered to the bottom of the boring or until the probe descent was inhibited, stopping at 1.6-foot intervals to collect data, as summarized in Table 2.

At each measurement depth the measurement sequence of two opposite horizontal records and one vertical record was performed, and the gains were adjusted as required. The data from each depth were viewed on the computer display, checked, and recorded on disk before moving to the next depth.

Upon completion of the measurements, the probe zero depth indication at the stationary reference point was verified and recorded on the field logs prior to removal from the boring. Field data were backed up to USB flash drive each day upon completion of data acquisition.

BORING NUMBER	TOOL AND RUN NUMBER	DEPTH RANGE (FEET)	DEPTH TO BOTTOM OF BORING (FEET)	SAMPLE INTERVAL (FEET)	DATE LOGGED
L-1	SUSPENSION PS 1	6.56 – 167.32	182	1.6	08/03/2010
L-4	SUSPENSION PS 1	6.56 – 139.44	154	1.6	08/03/2010
R-2A	SUSPENSION PS 1	3.28 – 123.03	139	1.6	09/02/2010

Table 2. Logging dates and depth ranges

DATA ANALYSIS

Suspension Analysis

Using the proprietary OYO program PSLOG.EXE version 1.0, the recorded digital waveforms were analyzed to locate the most prominent first minima, first maxima, or first break on the vertical axis records, indicating the arrival of P-wave energy. The difference in travel time between receiver 1 and receiver 2 (R1-R2) arrivals was used to calculate the P-wave velocity for that 3.3-foot segment of the soil column. When observable, P-wave arrivals on the horizontal axis records were used to verify the velocities determined from the vertical axis data. The time picks were then transferred into an EXCEL template to complete the velocity calculations based on the arrival time picks made in PSLOG.

The P-wave velocity over the 6.33-foot interval from source to receiver 1 (S-R1) was also picked using PSLOG, and calculated and plotted in EXCEL, for quality assurance of the velocity derived from the travel time between receivers. In this analysis, the depth values as recorded were increased by 4.53 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the P-wave signal at receiver 1 and subtracting 4.0 milliseconds, the calculated and experimentally verified delay from source trigger pulse (beginning of record) to source impact. This delay corresponds to the duration of acceleration of the solenoid before impact.

As with the P-wave records, using PSLOG, the recorded digital waveforms were analyzed to locate the presence of clear S_H-wave pulses, as indicated by opposite polarity pulses on each pair of horizontal records. Ideally, the S_H-wave signals from the 'normal' and 'reverse' source pulses are very nearly inverted images of each other. Digital FFT - IFFT lowpass filtering can be used to remove the higher frequency P-wave signal from the S_H-wave signal, if present.

Generally, the first maxima were picked for the 'normal' signals and the first minima for the 'reverse' signals, although other points on the waveform were used if the first pulse was distorted.

The absolute arrival time of the 'normal' and 'reverse' signals may vary by +/- 0.2 milliseconds, due to differences in the actuation time of the solenoid source caused by constant mechanical bias in the source or by boring inclination. This variation does not affect the R1-R2 velocity determinations, as the differential time is measured between arrivals of waves created by the same source actuation. The final velocity value is the average of the values obtained from the 'normal' and 'reverse' source actuations.

As with the P-wave data, S_H-wave velocity calculated from the travel time over the 6.33-foot interval from source to receiver 1 was calculated and plotted for verification of the velocity derived from the travel time between receivers. In this analysis, the depth values were increased by 4.53 feet to correspond to the mid-point of the 6.33-foot S-R1 interval. Travel times were obtained by picking the first break of the S_H-wave signal at the near receiver and subtracting 4.0 milliseconds, the calculated and experimentally verified delay from the beginning of the record at the source trigger pulse to source impact.

These data and analysis were reviewed by Robert Steller as a component of **GEO***Vision*'s in-house QA-QC program.

Figure 2 shows an example of R1 - R2 measurements on a sample filtered suspension record. In Figure 2, the time difference over the 3.3-foot interval of 1.88 milliseconds for the horizontal signals is equivalent to an S_H-wave velocity of 1745 feet/second. Whenever possible, time differences were determined from several phase points on the S_H-waveform records to verify the data obtained from the first arrival of the S_H-wave pulse. Figure 3 displays the same record before filtering of the S_H-waveform record with a 1400 Hz FFT - IFFT digital lowpass filter, illustrating the presence of higher frequency P-wave energy at the beginning of the record, and distortion of the lower frequency S_H-wave by residual P-wave signal.

RESULTS

Suspension Results

Suspension R1-R2 P-wave and S_H-wave velocities are plotted in Figures 4, 5 and 6. The suspension velocity data presented in these figures are presented in Tables 3, 4 and 5, respectively. These plots and data are included in the EXCEL analysis files in the boring specific directories on the data disk (CD-R) that accompanies this report.

P- and S_H-wave velocity data from R1-R2 analysis and quality assurance analysis of S-R1 data are plotted together in Figures A-1 through A-3 to aid in visual comparison. It should be noted that R1-R2 data are an average velocity over a 3.3-foot segment of the soil column; S-R1 data are an average over 6.33 feet, creating a significant smoothing relative to the R1-R2 plots. S-R1 data are presented in Tables A-1 through A-3, and included in the EXCEL analysis files.

Calibration procedures and records for the suspension PS measurement system are presented in Appendix B.

SUMMARY

Discussion of Suspension Results

Suspension PS velocity data are ideally collected in an uncased fluid filled boring, drilled with rotary mud (rotary wash) methods. The data collected in these uncased borings were of fair over-all quality.

Suspension PS velocity data quality is judged based upon 5 criteria:

	Criteria	Results for L-1, L-4, and R-2A
1	Consistent data between receiver to receiver (R1 – R2) and source to receiver (S – R1) data.	Data tracks fairly well between R1-R2 and S-R1 data. This correlation is generally degraded slightly in cased borings such as these.
2	Consistency between data from adjacent depth intervals.	All three borings show moderate scatter between adjacent depth intervals. This is expected in thinly interbedded sediments and fractured rock. This may be the case at this site, but the soil logs do not present sufficient detail to ascertain if this is indeed the case.
3	Consistent relationship between P-wave and S_H -wave (excluding transition to saturated soils)	Relationship between P-wave and S_H –wave is consistent, except above 50 feet in L-1. This drop of P-wave velocity below 5000 feet/sec is indicative of gases trapped in organic materials. Poisson's Ratio is within expected ranges for these materials.
4	Clarity of P-wave and S_H-wave onset, as well as damping of later oscillations.	Clarity of P-wave and S_H-wave onsets are poor in some sections of the softer sediments, which may indicate an enlarged boring filled with grout. Particularly in L-4 above 45 feet, the arrivals are very consistent, which may indicate signal arriving through the grout column. There are no low frequency un-damped signals that would indicate un-coupled casing.
5	Consistency of profile between adjacent borings, if available.	Similar S_H –wave velocity profiles are seen in similar units in all three borings. One exception is the section of bedrock between 104 and 119 feet in L-4. This presents a very low velocity for bedrock, particularly since it is overlaid by a much faster layer of stone fragments with sand. This may be due to weathering of the rock, or the presence of weaker shale. This velocity inversion is present in both P-wave and S_H-wave, and in both R1-R2 and S-R1 data, substantiating its presence.

Quality Assurance

These boring geophysical measurements were performed using industry-standard or better methods for measurements and analyses. All work was performed under **GEO***Vision* quality assurance procedures, which include:

- Use of NIST-traceable calibrations, where applicable, for field and laboratory instrumentation
- Use of standard field data logs
- Use of independent verification of velocity data by comparison of receiver-to-receiver and source-to-receiver velocities
- Independent review of calculations and results by a registered professional engineer, geologist, or geophysicist.

Suspension Data Reliability

P- and S_H-wave velocity measurement using the Suspension Method gives average velocities over a 3.3-foot interval of depth. This high resolution results in the scatter of values shown in the graphs. In uncased borings, individual measurements are very reliable, with estimated precision of +/- 5%. In cased borings, with uncertain grout bond, estimated precision is +/- 15%. Standardized field procedures and quality assurance checks contribute to the reliability of the data.

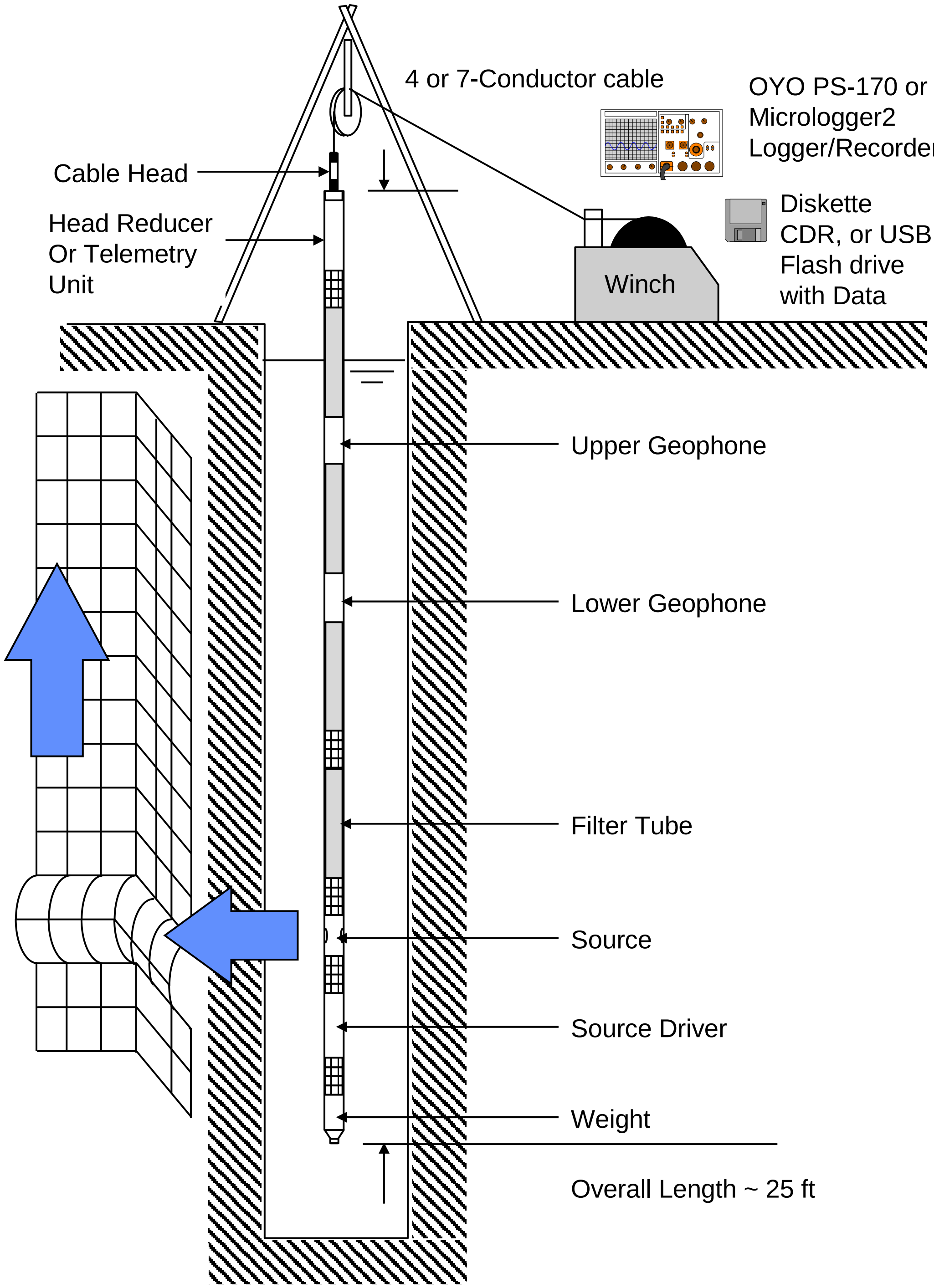


Figure 1: Concept illustration of P-S logging system

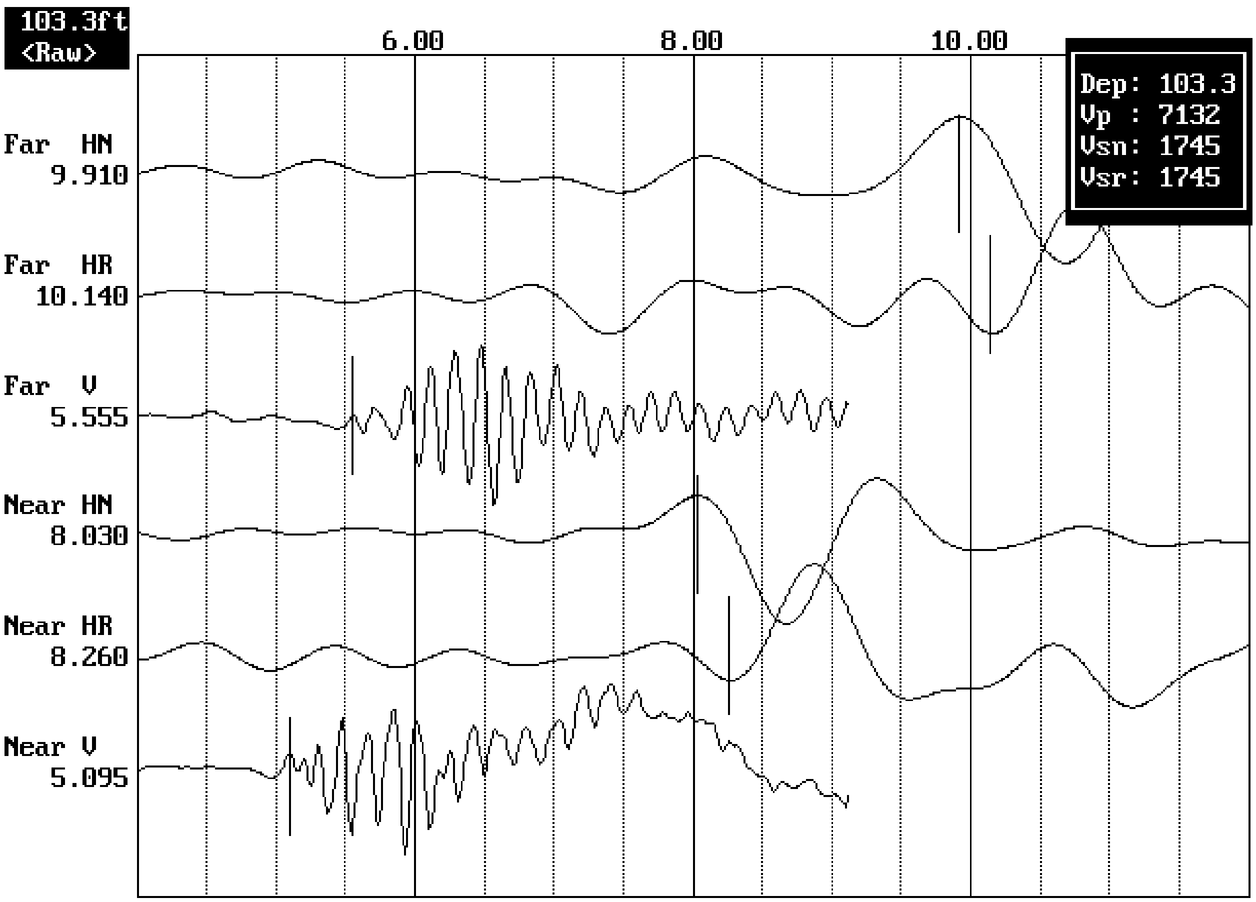


Figure 2: Example of filtered (1400 Hz lowpass) record

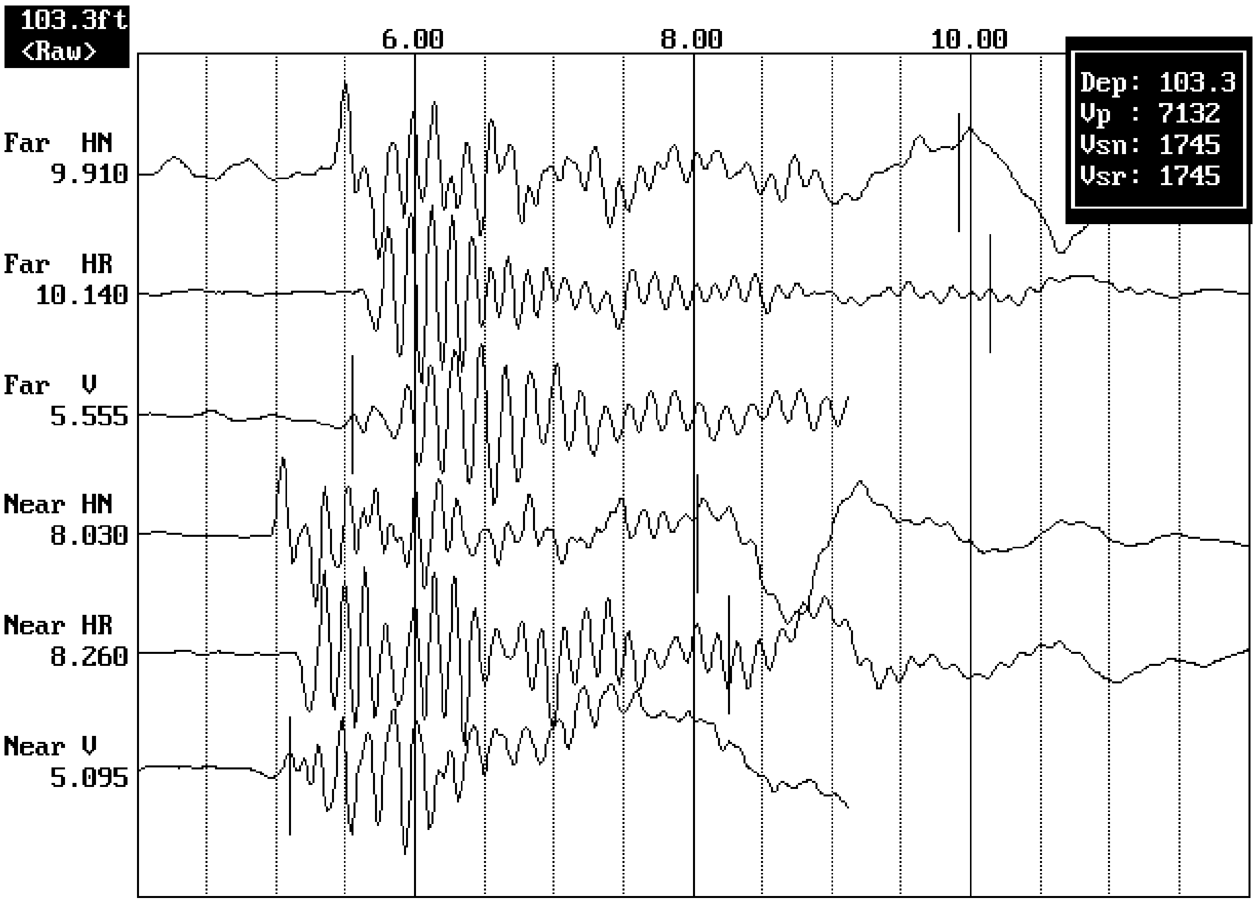


Figure 3. Example of unfiltered record

BRENT SPENCE BRIDGE REPLACEMENT BORING L-1

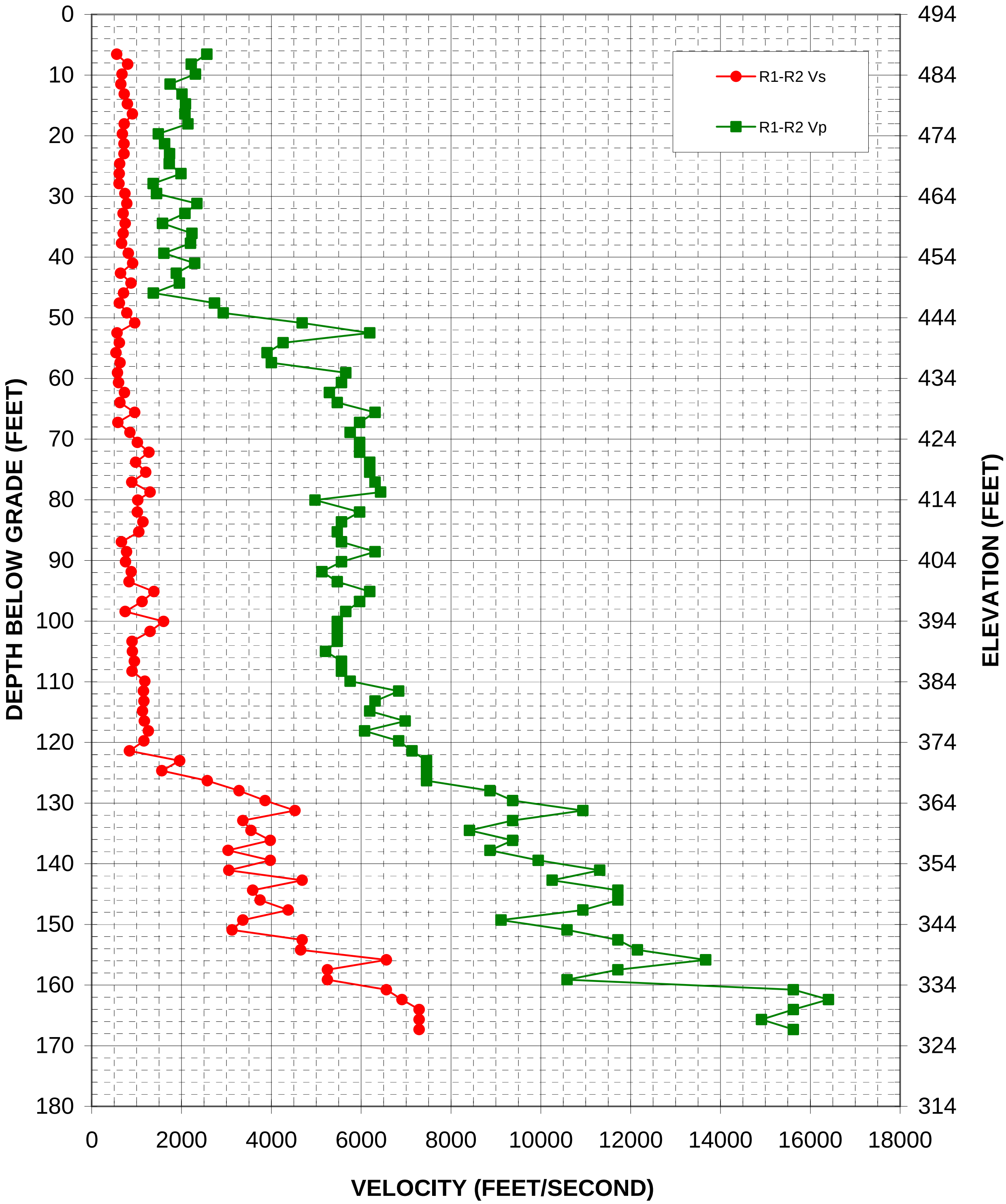


Figure 4: Boring L-1, Suspension R1-R2 P- and S_H-wave velocities

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
6.6	488.0	556	2563	0.48
8.2	486.4	800	2217	0.43
9.8	484.7	676	2310	0.45
11.5	483.1	650	1745	0.42
13.1	481.5	725	2013	0.43
14.8	479.8	795	2090	0.42
16.4	478.2	905	2076	0.38
18.0	476.5	725	2144	0.44
19.7	474.9	684	1485	0.37
21.3	473.3	721	1624	0.38
23.0	471.6	721	1736	0.40
24.6	470.0	625	1727	0.42
26.2	468.3	613	1988	0.45
27.9	466.7	608	1367	0.38
29.5	465.1	741	1445	0.32
31.2	463.4	781	2343	0.44
32.8	461.8	698	2076	0.44
34.4	460.1	746	1577	0.36
36.1	458.5	702	2232	0.45
37.7	456.9	666	2202	0.45
39.4	455.2	815	1608	0.33
41.0	453.6	911	2294	0.41
42.7	451.9	643	1886	0.43
44.3	450.3	875	1953	0.37
45.9	448.7	709	1373	0.32
47.6	447.0	616	2734	0.47
49.2	445.4	781	2929	0.46
50.9	443.7	958	4687	0.48
52.5	442.1	566	6190	0.50
54.1	440.5	616	4261	0.49
55.8	438.8	540	3906	0.49
57.4	437.2	631	4001	0.49
59.1	435.5	576	5657	0.49
60.7	433.9	599	5561	0.49
62.3	432.3	729	5292	0.49
64.0	430.6	628	5468	0.49
65.6	429.0	958	6309	0.49
67.3	427.3	583	5965	0.50
68.9	425.7	852	5756	0.49
70.5	424.1	1017	5965	0.49
72.2	422.4	1274	5965	0.48
73.8	420.8	979	6190	0.49
75.5	419.1	1204	6190	0.48
77.1	417.5	893	6309	0.49
78.7	415.8	1299	6433	0.48
80.1	414.5	1025	4971	0.48
82.0	412.6	1017	5965	0.49
83.7	410.9	1141	5561	0.48
85.3	409.3	1050	5468	0.48
86.9	407.6	663	5561	0.49
88.6	406.0	777	6309	0.49
90.2	404.4	754	5561	0.49
91.9	402.7	881	5126	0.48
93.5	401.1	831	5468	0.49
95.1	399.4	1384	6190	0.47
96.8	397.8	1122	5965	0.48
98.4	396.2	746	5657	0.49
100.1	394.5	1600	5468	0.45
101.7	392.9	1299	5468	0.47
103.3	391.2	899	5468	0.49
105.0	389.6	905	5208	0.48
106.6	388.0	951	5561	0.48
108.3	386.3	899	5561	0.49
109.9	384.7	1182	5756	0.48
111.5	383.0	1151	6835	0.49
113.2	381.4	1161	6309	0.48
114.8	379.8	1131	6190	0.48
116.5	378.1	1172	6981	0.49
118.1	376.5	1262	6076	0.48
119.8	374.8	1161	6835	0.49
121.4	373.2	841	7132	0.49
123.0	371.6	1959	7456	0.46
124.7	369.9	1562	7456	0.48
126.3	368.3	2573	7456	0.43
128.0	366.6	3281	8867	0.42
129.6	365.0	3860	9374	0.40
131.2	363.4	4525	10936	0.40
132.9	361.7	3365	9374	0.43
134.5	360.1	3547	8412	0.39
136.2	358.4	3977	9374	0.39
137.8	356.8	3038	8867	0.43
139.4	355.2	3977	9942	0.40
141.1	353.5	3052	11313	0.46
142.7	351.9	4687	10253	0.37
144.4	350.2	3586	11717	0.45
146.0	348.6	3750	11717	0.44
147.6	347.0	4374	10936	0.40
149.3	345.3	3365	9113	0.42
150.9	343.7	3125	10583	0.45
152.6	342.0	4687	11717	0.40
154.2	340.4	4654	12151	0.41
155.8	338.8	6562	13670	0.35
157.5	337.1	5249	11717	0.37
159.1	335.5	5249	10583	0.34
160.8	333.8	6562	15623	0.39
162.4	332.2	6907	16404	0.39
164.0	330.5	7291	15623	0.36
165.7	328.9	7291	14913	0.34
167.3	327.3	7291	15623	0.36

Table 3. Boring L-1, Suspension R1-R2 depths and P- and S_H-wave velocities

BRENT SPENCE BRIDGE REPLACEMENT BORING L-4

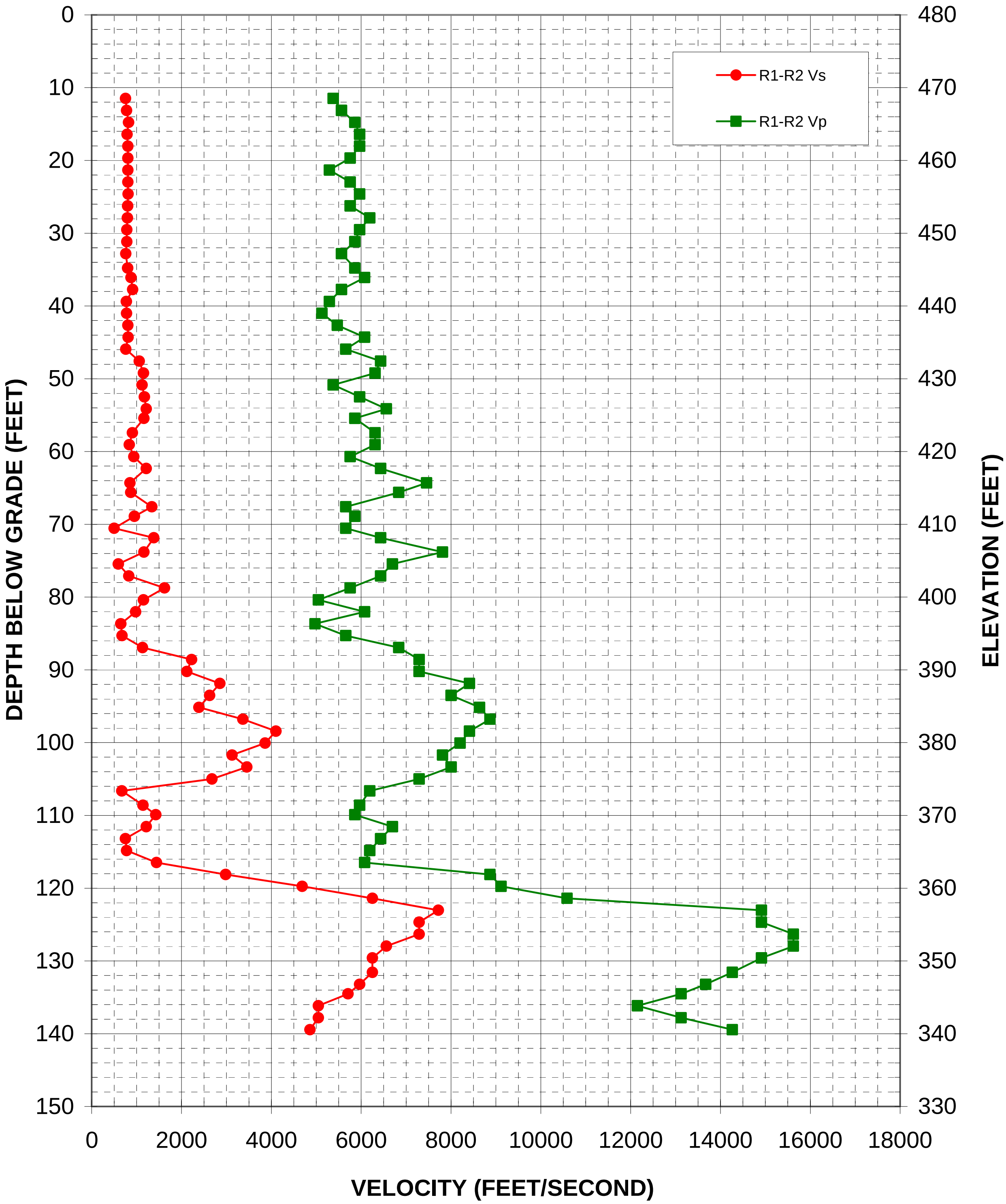


Figure 5: Boring L-4, Suspension R1-R2 P- and S_H-wave velocities

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
7.2	472.8			
8.2	471.8			
9.8	470.1			
11.5	468.5	754	5378	0.49
13.1	466.8	777	5561	0.49
14.8	465.2	821	5859	0.49
16.4	463.6	786	5965	0.49
18.0	461.9	805	5965	0.49
19.7	460.3	805	5756	0.49
21.3	458.6	805	5292	0.49
23.0	457.0	805	5756	0.49
24.6	455.4	810	5965	0.49
26.2	453.7	800	5756	0.49
27.9	452.1	795	6190	0.49
29.5	450.4	781	5965	0.49
31.2	448.8	781	5859	0.49
32.8	447.2	759	5561	0.49
34.8	445.2	800	5859	0.49
36.1	443.9	875	6076	0.49
37.7	442.2	911	5561	0.49
39.4	440.6	772	5292	0.49
41.0	439.0	777	5126	0.49
42.7	437.3	805	5468	0.49
44.3	435.7	810	6076	0.49
45.9	434.0	759	5657	0.49
47.6	432.4	1058	6433	0.49
49.2	430.8	1151	6309	0.48
50.9	429.1	1122	5378	0.48
52.5	427.5	1172	5965	0.48
54.1	425.8	1215	6562	0.48
55.4	424.5	1161	5859	0.48
57.4	422.6	905	6309	0.49
59.1	420.9	836	6309	0.49
60.7	419.3	937	5756	0.49
62.3	417.6	1215	6433	0.48
64.3	415.7	852	7456	0.49
65.6	414.4	869	6835	0.49
67.6	412.4	1339	5657	0.47
68.9	411.1	951	5859	0.49
70.5	409.4	501	5657	0.50
71.9	408.1	1381	6433	0.48
73.8	406.2	1161	7812	0.49
75.5	404.5	594	6696	0.50
77.1	402.9	825	6433	0.49
78.7	401.2	1620	5756	0.46
80.4	399.6	1151	5047	0.47
82.0	397.9	979	6076	0.49
83.7	396.3	646	4971	0.49
85.3	394.7	676	5657	0.49
86.9	393.0	1131	6835	0.49
88.6	391.4	2224	7291	0.45
90.2	389.7	2117	7291	0.45
91.9	388.1	2853	8412	0.44
93.5	386.5	2625	8002	0.44
95.1	384.8	2386	8634	0.46
96.8	383.2	3365	8867	0.42
98.4	381.5	4101	8412	0.34
100.1	379.9	3860	8202	0.36
101.7	378.3	3125	7812	0.40
103.3	376.6	3454	8002	0.39
105.0	375.0	2678	7291	0.42
106.6	373.3	670	6190	0.49
108.6	371.4	1141	5965	0.48
109.9	370.1	1426	5859	0.47
111.5	368.4	1215	6696	0.48
113.2	366.8	750	6433	0.49
114.8	365.1	777	6190	0.49
116.5	363.5	1442	6076	0.47
118.1	361.9	2983	8867	0.44
119.8	360.2	4687	9113	0.32
121.4	358.6	6249	10583	0.23
123.0	356.9	7720	14913	0.32
124.7	355.3	7291	14913	0.34
126.3	353.7	7291	15623	0.36
128.0	352.0	6562	15623	0.39
129.6	350.4	6249	14913	0.39
131.6	348.4	6249	14265	0.38
133.2	346.8	5965	13670	0.38
134.5	345.5	5706	13123	0.38
136.2	343.8	5047	12151	0.40
137.8	342.2	5047	13123	0.41
139.4	340.5	4861	14265	0.43

Table 4. Boring L-4, Suspension R1-R2 depths and P- and S_H-wave velocities

BRENT SPENCE BRIDGE REPLACEMENT BORING R-2A

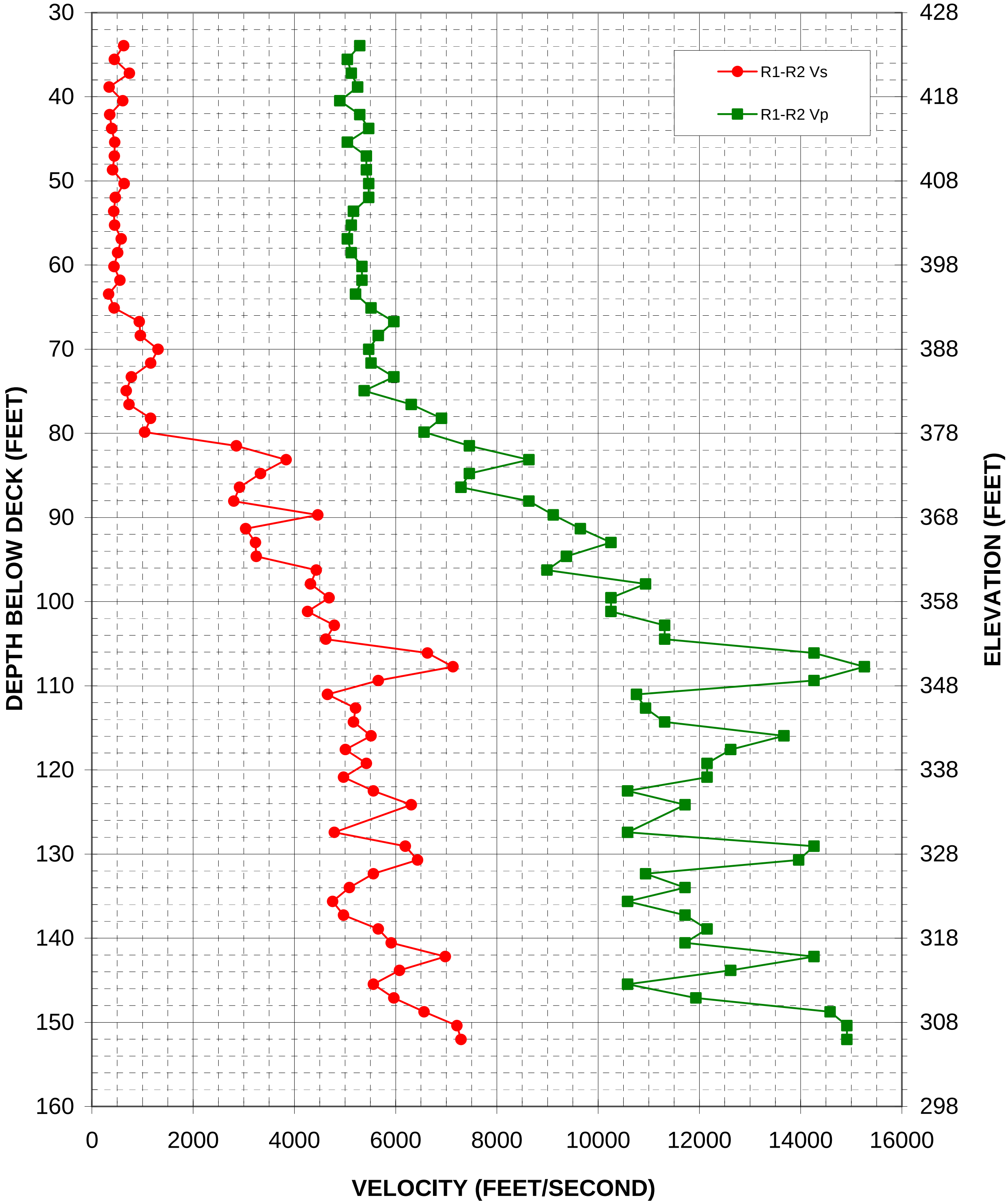


Figure 6: Boring R-2A, Suspension R1-R2 S_H-wave velocities

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
33.9	423.7	628	5292	0.49
35.6	422.1	443	5047	0.50
37.2	420.4	741	5126	0.49
38.8	418.8	341	5249	0.50
40.5	417.2	610	4897	0.49
42.1	415.5	352	5292	0.50
43.8	413.9	392	5468	0.50
45.4	412.2	451	5047	0.50
47.0	410.6	443	5423	0.50
48.7	409.0	409	5423	0.50
50.3	407.3	637	5468	0.49
52.0	405.7	462	5468	0.50
53.6	404.0	432	5167	0.50
55.2	402.4	449	5126	0.50
56.9	400.8	581	5047	0.49
58.5	399.1	509	5126	0.50
60.2	397.5	437	5335	0.50
61.8	395.8	554	5335	0.49
63.4	394.2	331	5208	0.50
65.1	392.6	440	5514	0.50
66.7	390.9	937	5965	0.49
68.4	389.3	958	5657	0.49
70.0	387.6	1307	5468	0.47
71.7	386.0	1161	5514	0.48
73.3	384.3	781	5965	0.49
74.9	382.7	680	5378	0.49
76.6	381.1	734	6309	0.49
78.2	379.4	1159	6907	0.49
79.9	377.8	1042	6562	0.49
81.5	376.1	2853	7456	0.41
83.1	374.5	3837	8634	0.38
84.8	372.9	3331	7456	0.38
86.4	371.2	2916	7291	0.40
88.1	369.6	2804	8634	0.44
89.7	367.9	4464	9113	0.34
91.3	366.3	3038	9650	0.44
93.0	364.7	3232	10253	0.44
94.6	363.0	3248	9374	0.43
96.3	361.4	4434	8989	0.34
97.9	359.7	4317	10936	0.41
99.5	358.1	4687	10253	0.37
101.2	356.5	4261	10253	0.40
102.8	354.8	4790	11313	0.39
104.5	353.2	4621	11313	0.40
106.1	351.5	6628	14265	0.36
107.7	349.9	7132	15260	0.36
109.4	348.3	5657	14265	0.41
111.0	346.6	4654	10757	0.38
112.7	345.0	5208	10936	0.35
114.3	343.3	5167	11313	0.37
115.9	341.7	5514	13670	0.40
117.6	340.1	5009	12619	0.41
119.2	338.4	5423	12151	0.38
120.9	336.8	4971	12151	0.40
122.5	335.1	5561	10583	0.31
124.1	333.5	6309	11717	0.30
127.4	330.2	4790	10583	0.37
129.1	328.6	6190	14265	0.38
130.7	326.9	6433	13961	0.37
132.3	325.3	5561	10936	0.33
134.0	323.7	5087	11717	0.38
135.6	322.0	4755	10583	0.37
137.3	320.4	4971	11717	0.39
138.9	318.7	5657	12151	0.36
140.5	317.1	5911	11717	0.33
142.2	315.5	6981	14265	0.34
143.8	313.8	6076	12619	0.35
145.5	312.2	5561	10583	0.31
147.1	310.5	5965	11930	0.33
148.8	308.9	6562	14581	0.37
150.4	307.2	7211	14913	0.35
152.0	305.6	7291	14913	0.34

Table 5. Boring R-2A, Suspension R1-R2 depths and S_H-wave velocities

APPENDIX A

SUSPENSION VELOCITY MEASUREMENT QUALITY ASSURANCE SUSPENSION SOURCE TO RECEIVER ANALYSIS RESULTS

BRENT SPENCE BRIDGE REPLACEMENT BORING L-1

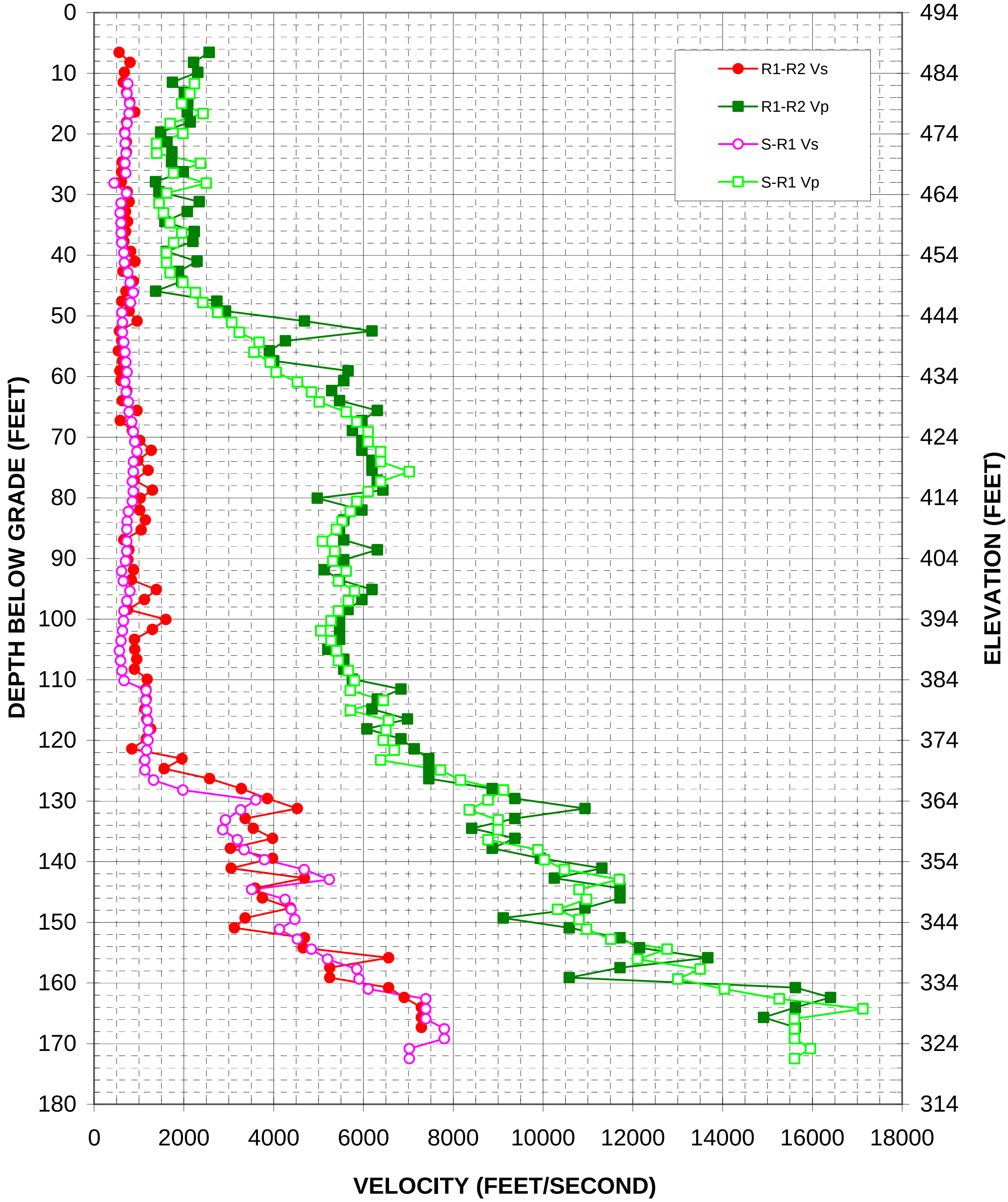


Figure A-1. Boring L-1, R1 - R2 high resolution analysis and S - R1 quality assurance analysis P- and S_H-wave data

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
11.7	482.9	751	2236	0.44
13.4	481.2	735	2134	0.43
15.0	479.6	793	1950	0.40
16.6	478.0	780	2429	0.44
18.3	476.3	735	1696	0.38
19.9	474.7	685	1978	0.43
21.6	473.0	695	1393	0.33
23.2	471.4	713	1396	0.32
24.8	469.8	688	2372	0.45
26.5	468.1	706	1769	0.41
28.1	466.5	447	2499	0.48
29.8	464.8	725	1618	0.37
31.4	463.2	600	1448	0.40
33.0	461.6	580	1546	0.42
34.7	459.9	598	1688	0.43
36.3	458.3	603	1956	0.45
38.0	456.6	616	1773	0.43
39.6	455.0	665	1614	0.40
41.2	453.3	675	1614	0.39
42.9	451.7	751	1696	0.38
44.5	450.1	802	1978	0.40
46.2	448.4	872	2258	0.41
47.8	446.8	812	2421	0.44
49.4	445.1	619	2753	0.47
51.1	443.5	633	3066	0.48
52.7	441.9	633	3235	0.48
54.4	440.2	662	3676	0.48
56.0	438.6	688	3564	0.48
57.6	436.9	709	3922	0.48
59.3	435.3	735	4058	0.48
60.9	433.7	685	4530	0.49
62.6	432.0	717	4842	0.49
64.2	430.4	763	5015	0.49
65.8	428.7	780	5617	0.49
67.5	427.1	836	5851	0.49
69.1	425.5	872	6105	0.49
70.8	423.8	906	6105	0.49
72.4	422.2	955	6383	0.49
74.0	420.5	878	6383	0.49
75.7	418.9	872	7021	0.49
77.3	417.3	851	6383	0.49
79.0	415.6	872	6105	0.49
80.6	414.0	851	5851	0.49
82.3	412.3	767	5708	0.49
83.9	410.7	735	5528	0.49
85.2	409.4	731	5401	0.49
87.2	407.4	731	5088	0.49
88.8	405.8	728	5360	0.49
90.5	404.1	699	5319	0.49
92.1	402.5	611	5617	0.49
93.7	400.9	650	5443	0.49
95.4	399.2	798	5802	0.49
97.0	397.6	731	5662	0.49
98.7	395.9	669	5443	0.49
100.3	394.3	656	5279	0.49
101.9	392.7	633	5051	0.49
103.6	391.0	598	5279	0.49
105.2	389.4	565	5401	0.49
106.9	387.7	590	5443	0.49
108.5	386.1	619	5662	0.49
110.1	384.5	669	5802	0.49
111.8	382.8	1160	5708	0.48
113.4	381.2	1151	6441	0.48
115.1	379.5	1170	5708	0.48
116.7	377.9	1190	6562	0.48
118.3	376.3	1211	6501	0.48
120.0	374.6	1200	6441	0.48
121.6	373.0	1170	6687	0.48
123.3	371.3	1132	6383	0.48
124.9	369.7	1132	7715	0.49
126.5	368.0	1325	8164	0.49
128.2	366.4	1978	9118	0.48
129.8	364.8	3601	8776	0.40
131.5	363.1	3266	8358	0.41
133.1	361.5	2925	9001	0.44
134.7	359.8	2866	9001	0.44
136.4	358.2	3191	8776	0.42
138.0	356.6	3343	9889	0.44
139.7	354.9	3795	10030	0.42
141.3	353.3	4681	10479	0.38
142.9	351.6	5240	11702	0.37
144.6	350.0	3510	10802	0.44
146.2	348.4	4255	10970	0.41
147.9	346.7	4388	10325	0.39
149.5	345.1	4472	10802	0.40
151.1	343.4	4130	10970	0.42
152.8	341.8	4530	11510	0.41
154.4	340.2	4842	12765	0.42
156.1	338.5	5201	12105	0.39
157.7	336.9	5851	13502	0.38
159.4	335.2	5900	13002	0.37
161.0	333.6	6105	14042	0.38
162.6	332.0	7391	15263	0.35
164.3	330.3	7391	17124	0.39
165.9	328.7	7391	15602	0.36
167.6	327.0	7801	15602	0.33
169.2	325.4	7801	15602	0.33
170.8	323.8	7021	15957	0.38
172.5	322.1	7021	15602	0.37

Table A-1. Boring L-1, S - R1 quality assurance analysis P- and S_H-wave data

BRENT SPENCE BRIDGE REPLACEMENT BORING L-4

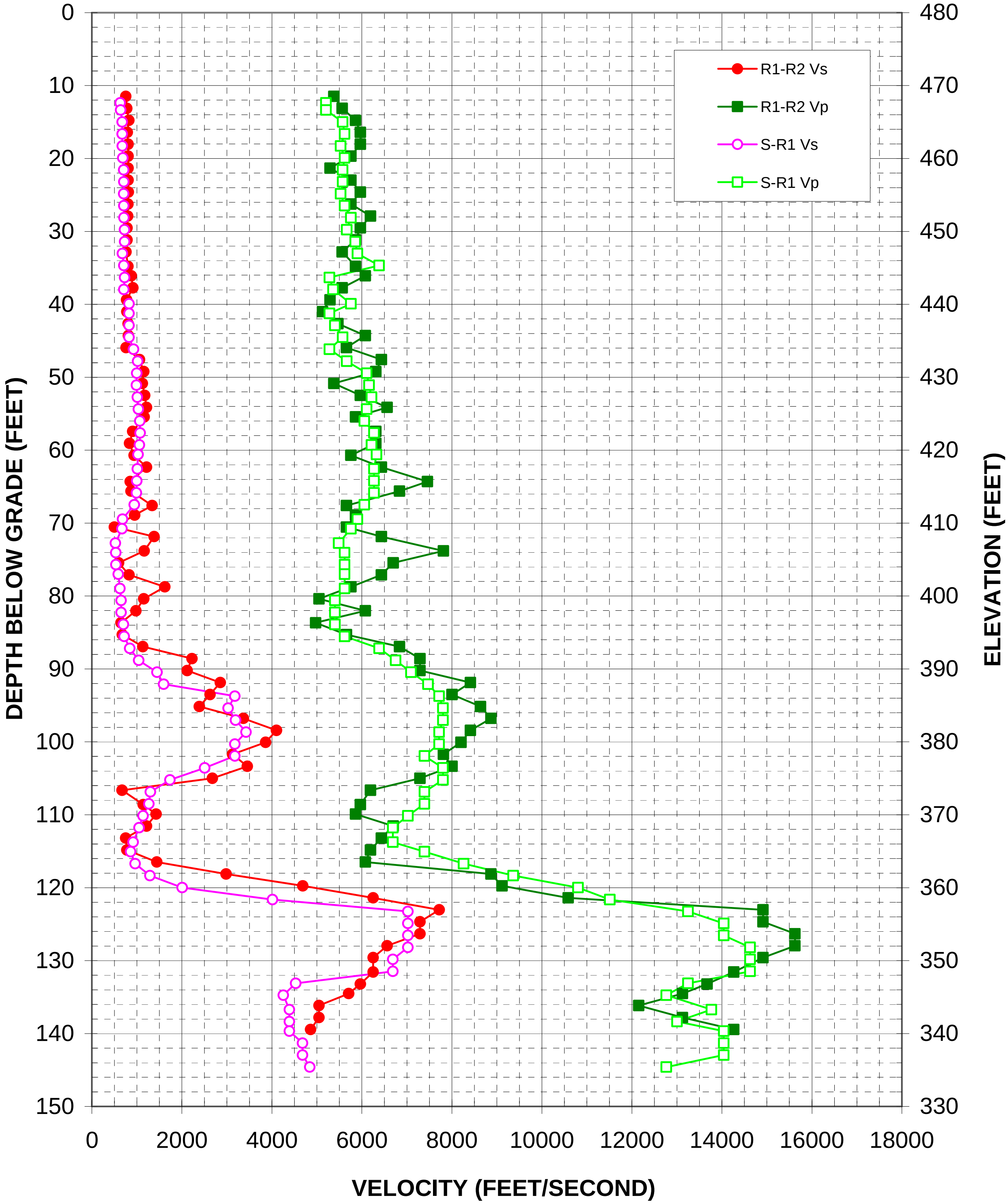


Figure A-2. Boring L-4, R1 - R2 high resolution analysis and S - R1 quality assurance analysis P- and S_H-wave data

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
12.4	467.6	630	5201	0.49
13.4	466.6	638	5201	0.49
15.0	465.0	674	5572	0.49
16.6	463.3	674	5617	0.49
18.3	461.7	675	5528	0.49
19.9	460.1	690	5617	0.49
21.6	458.4	706	5572	0.49
23.2	456.8	709	5572	0.49
24.8	455.1	706	5528	0.49
26.5	453.5	709	5617	0.49
28.1	451.9	713	5755	0.49
29.8	450.2	724	5662	0.49
31.4	448.6	728	5851	0.49
33.0	446.9	678	5900	0.49
34.7	445.3	709	6383	0.49
36.3	443.7	728	5279	0.49
38.0	442.0	709	5360	0.49
39.9	440.0	826	5755	0.49
41.2	438.7	826	5279	0.49
42.9	437.1	826	5401	0.49
44.5	435.4	826	5572	0.49
46.2	433.8	924	5279	0.48
47.8	432.2	1018	5662	0.48
49.4	430.5	996	6105	0.49
51.1	428.9	989	6159	0.49
52.7	427.2	1010	6213	0.49
54.4	425.6	1032	6105	0.49
56.0	424.0	1064	6053	0.48
57.6	422.3	1072	6269	0.48
59.3	420.7	1056	6213	0.49
60.6	419.4	1025	6325	0.49
62.6	417.4	1010	6269	0.49
64.2	415.8	996	6269	0.49
65.8	414.1	989	6269	0.49
67.5	412.5	942	6053	0.49
69.5	410.5	682	5900	0.49
70.8	409.2	669	5755	0.49
72.7	407.2	522	5485	0.50
74.0	405.9	532	5617	0.50
75.7	404.3	534	5617	0.50
77.0	403.0	585	5617	0.49
79.0	401.0	624	5617	0.49
80.6	399.4	650	5401	0.49
82.3	397.7	650	5401	0.49
83.9	396.1	699	5401	0.49
85.5	394.4	716	5617	0.49
87.2	392.8	841	6383	0.49
88.8	391.2	1040	6751	0.49
90.5	389.5	1448	7092	0.48
92.1	387.9	1596	7469	0.48

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
93.7	386.2	3177	7715	0.40
95.4	384.6	3026	7801	0.41
97.0	383.0	3191	7801	0.40
98.7	381.3	3425	7715	0.38
100.3	379.7	3177	7715	0.40
101.9	378.0	3177	7391	0.39
103.6	376.4	2507	7801	0.44
105.2	374.8	1734	7801	0.47
106.9	373.1	1300	7391	0.48
108.5	371.5	1265	7391	0.48
110.1	369.8	1138	7021	0.49
111.8	368.2	1048	6687	0.49
113.7	366.2	918	6687	0.49
115.1	364.9	859	7391	0.49
116.7	363.3	962	8260	0.49
118.3	361.6	1288	9361	0.49
120.0	360.0	2006	10802	0.48
121.6	358.3	4012	11510	0.43
123.3	356.7	7021	13247	0.30
124.9	355.1	7021	14042	0.33
126.5	353.4	7021	14042	0.33
128.2	351.8	7021	14627	0.35
129.8	350.1	6687	14627	0.37
131.5	348.5	6687	14627	0.37
133.1	346.9	4530	13247	0.43
134.7	345.2	4255	12765	0.44
136.7	343.3	4388	13767	0.44
138.4	341.6	4388	13002	0.44
139.7	340.3	4388	14042	0.45
141.3	338.7	4681	14042	0.44
142.9	337.0	4681	14042	0.44
144.6	335.4	4842	12765	0.42

Table A-2. Boring L-4, S - R1 quality assurance analysis P- and S_H-wave data

BRENT SPENCE BRIDGE REPLACEMENT BORING R-2A

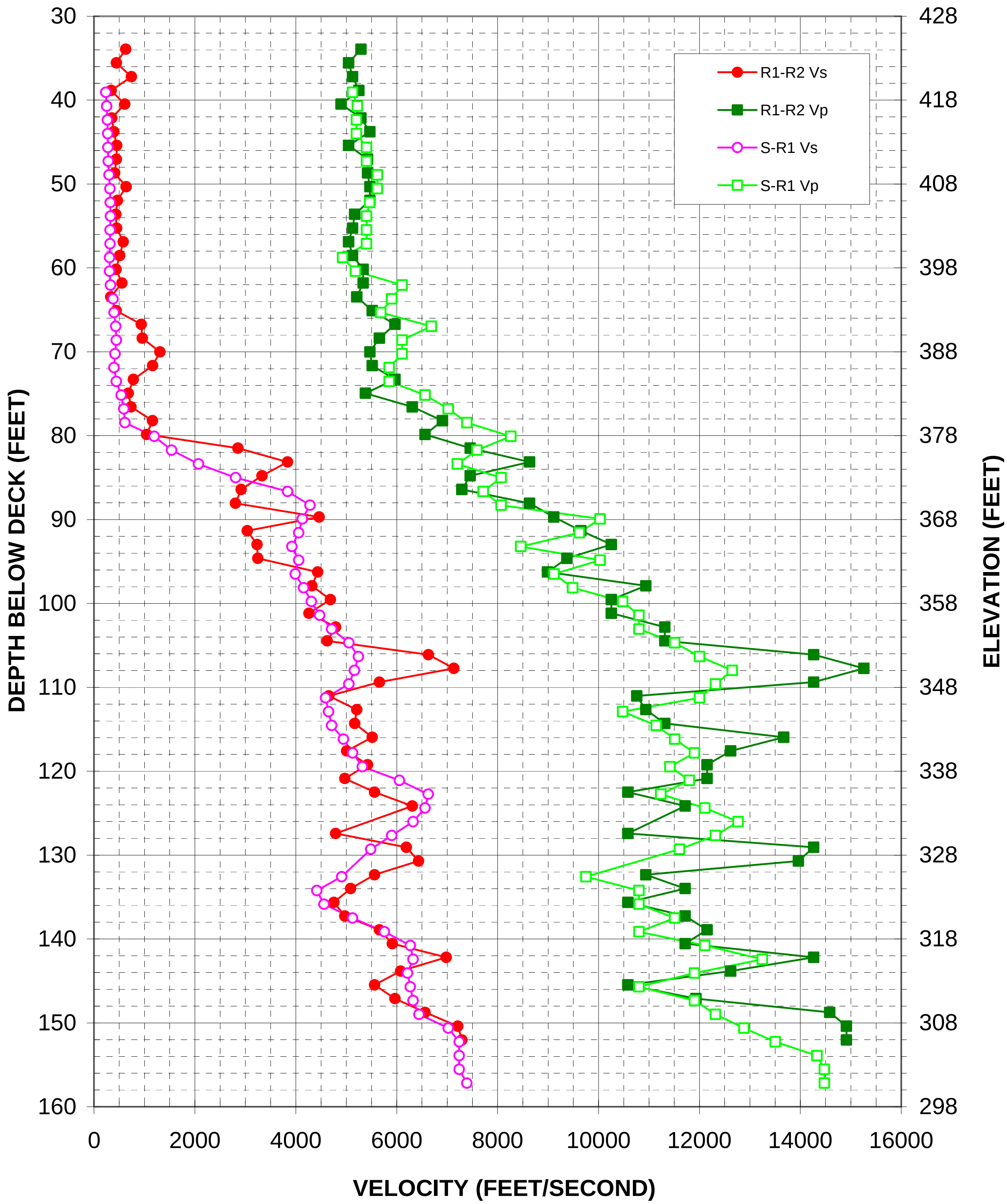


Figure A-3. Boring R-2A, R1 - R2 high resolution analysis and S - R1 quality assurance analysis S_H-wave data

Depth (feet)	Elevation (feet)	V_s (feet/sec)	V_p (feet/sec)	Poisson's Ratio
39.1	418.6	235	5125	0.50
40.7	416.9	250	5220	0.50
42.4	415.3	264	5201	0.50
44.0	413.6	271	5201	0.50
45.6	412.0	277	5401	0.50
47.3	410.4	284	5401	0.50
48.9	408.7	298	5617	0.50
50.6	407.1	315	5617	0.50
52.2	405.4	323	5464	0.50
53.8	403.8	326	5401	0.50
55.5	402.2	319	5401	0.50
57.1	400.5	317	5401	0.50
58.8	398.9	309	4927	0.50
60.4	397.2	309	5182	0.50
62.0	395.6	327	6105	0.50
63.7	394.0	376	5900	0.50
65.3	392.3	399	5685	0.50
67.0	390.7	429	6687	0.50
68.6	389.0	442	6105	0.50
70.2	387.4	417	6105	0.50
71.9	385.8	397	5851	0.50
73.5	384.1	442	5851	0.50
75.2	382.5	539	6562	0.50
76.8	380.8	592	7021	0.50
78.4	379.2	613	7391	0.50
80.1	377.6	1195	8260	0.49
81.7	375.9	1536	7590	0.48
83.4	374.3	2071	7201	0.45
85.0	372.6	2808	8070	0.43
86.6	371.0	3837	7715	0.34
88.3	369.4	4281	8070	0.30
89.9	367.7	4130	10030	0.40
91.6	366.1	4058	9618	0.39
93.2	364.4	3922	8459	0.36
94.8	362.8	4058	10030	0.40
96.5	361.2	3989	9118	0.38
98.1	359.5	4154	9488	0.38
99.8	357.9	4307	10479	0.40
101.4	356.2	4472	10802	0.40
103.0	354.6	4712	10802	0.38
104.7	353.0	5051	11510	0.38
106.3	351.3	5240	12002	0.38
108.0	349.7	5162	12650	0.40
109.6	348.0	5051	12318	0.40
111.3	346.4	4589	12002	0.41
112.9	344.7	4650	10479	0.38
114.5	343.1	4712	11144	0.39
116.2	341.5	4944	11510	0.39
117.8	339.8	5125	11900	0.39
119.5	338.2	5319	11416	0.36
121.1	336.5	6053	11800	0.32
122.7	334.9	6624	11234	0.23
124.4	333.3	6562	12105	0.29
126.0	331.6	6325	12765	0.34
127.7	330.0	5900	12318	0.35
129.3	328.3	5485	11605	0.36
132.6	325.1	4910	9751	0.33
134.2	323.4	4416	10802	0.40
135.9	321.8	4559	10802	0.39
137.5	320.1	5125	11510	0.38
139.1	318.5	5755	10802	0.30
140.8	316.9	6269	12105	0.32
142.4	315.2	6325	13247	0.35
144.1	313.6	6213	11900	0.31
145.7	311.9	6269	10802	0.25
147.3	310.3	6325	11900	0.30
149.0	308.7	6441	12318	0.31
150.6	307.0	7021	12883	0.29
152.3	305.4	7238	13502	0.30
153.9	303.7	7238	14329	0.33
155.5	302.1	7238	14476	0.33
157.2	300.5	7391	14476	0.32

Table A-3. Boring R-2A, S - R1 quality assurance analysis S_H-wave data

APPENDIX B

GEOPHYSICAL LOGGING SYSTEMS –
NIST TRACEABLE CALIBRATION
PROCEDURES AND CALIBRATION RECORDS

GEOVision SUSPENSION PS SEISMIC LOGGER/RECORDER

CALIBRATION PROCEDURE

Reviewed 7/21/08

Objective

The timing/sampling accuracy of seismic recorders or data loggers is required for several GEOVision field procedures including Seismic Refraction, Downhole P-S Seismic Velocity Logging, and Suspension P-S Seismic Velocity Logging. This procedure describes the method for measuring the timing accuracy of a seismic data logger, such as the OYO Model 170 or OYO/Robertson Model 3403. The objective of this procedure is to verify that the timing accuracy of the recorder is accurate to within 1%.

Frequency of Calibration

The calibration of each GEOVision seismic data logger is twelve (12) months. In the case of rented seismic logger/recorders, calibration must be performed prior to use.

Test Equipment Required

The following equipment is required. Item #2 must have current NIST traceable calibration.

1. Function generator, Krohn Hite 5400B or equivalent
2. Frequency counter, HP 5315A or equivalent
3. Test cables, from item 1 to item 2, and from item 1 to subject data logger.

Procedure

This procedure is designed to be performed using the accompanying Suspension P-S Seismic Logger/Recorder Calibration Data Form with the same revision number. All data must be entered and the procedure signed by the technician performing the test.

1. Record all identification data on the form provided.
2. Connect function generator to data logger (such as OYO Model 170) using test cable
3. Connect the function generator to the frequency counter using test cable.
4. Set signal generator to target frequency specified on data form, 0.25 volt (amplitude is approximate, modify as necessary to yield less than full scale waveforms on

logger display) peak sine wave. Verify frequency using the counter and note actual frequency on the data form.

5. Set data logger to file length specified on data form and record a data file to disk. Note file name on data form.

6. Measure the duration of 9 complete sine wave cycles on the data file. This measurement must be made using the analysis program PSLOG.EXE version 1.00, and saved as a .sps pick file. Note the duration in milliseconds in the spaces provided on the data form. Calculate average recorded sine wave frequency for each channel pair (Hn, Hr, V) by dividing the duration by 9. Note the average frequency of each channel pair on the data form.

7. Repeat steps 4 through 6 until all target frequencies have been recorded, producing 6 separate data and pick files.

Criteria

The average frequency for the nine cycles (obtained by dividing 9 cycles by the duration in seconds) must be within plus or minus 1% of the actual frequency for each of the 6 records.

If the results are outside this range, the data logger must be marked with a GEOVision REJECT tag until it can be repaired and retested.

If results are acceptable affix label indicating the initials of the person performing the calibration, the date of calibration, and the due date for the next calibration (12 months).

Procedure Approval

Approved by:

John G. Diehl — President

Name — Title

[signature] — July 21, 2008

Signature — Date

Calibration Laboratory Approval (if required):

Name — Title

Signature — Date

MICRO PRECISION CALIBRATION, INC.
12686 HOOVER STREET
GARDEN GROVE CA. 92841-1823
714.901.5659

Certificate of Calibration

Date: 10/16/2009 **Lab #** AC-1274 **Certificate #: 749437**

Customer:
GEOVISION
1124 OLYMPIC DRIVE
CORONA, CA, 92881

		Purchase Order:	9333-100601-001
		Work Order:	61143
MPC Control #:	AM6767	Serial Number:	160023
Asset ID:	160023	Department:	N/A
Gage Type:	LOGGER	Performed By:	KYU HAN
Manufacturer:	OYO	Received Condition:	IN TOLERANCE
Model Number:	3403	Returned Condition:	IN TOLERANCE
Size:	N/A	Cal Date:	October 12, 2009
Temp./RH:	73 °F / 45 %	Cal. Interval:	12 MONTHS
		Cal. Due Date:	October 12, 2010

Found conditions meet or exceed manufacturer specifications.

***Calibration Notes:**

The UUT (unit under test) was calibrated using the customers procedures in our Garden Grove lab. The UUT was operated by the customers personnel and data collection was observed by MPC personnel. The UUT was found to be in tolerance to customer supplied specifications. The reference standards used are in complience with ISO/IEC 17025:2005, ISO9001:2000, ANSI/NCSL Z540-1-1994 and laboratory accreditiation for lab code 935.11. Frequency is accredited. Measurement uncertainity is 0.2 x E12 Hz. Please see attached data sheet.

Standards Used To Calibrate Equipment

I.D.	Description	Model	Serial	Manufacturer	Cal. Due Date	Traceability #
AM4000	WAVEFORM GENERATOR	33250A	MY40000703	AGILENT	7/15/2010	662404
T1100	COUNTER	53131A	3546A09912	HEWLETT PACKARD	1/12/2010	646688

Calibrating Technician: KYU HAN

QC Approval: Tammy Webster

Unless Otherwise Noted, Uncertainty Estimated at >= 4 to 1. Uncertainties have been estimated at a 95 percent confidence level (k=2). Services rendered comply with ISO 17025:2005, ISO 9001:2000, ANSI/NCSL Z540-1, MPC Quality Manual, MPC CSD and with customer purchase order instructions.

Calibration cycles and resulting due dates were submitted/approved by the customer. Any number of factors may cause an instrument to drift out of tolerance before the next scheduled calibration. Recalibration cycles should be based on frequency of use, environmental conditions and customer's established systematic accuracy. The information on this report, pertains only to the instrument identified.

AM 6767

GEOVision geophysical services

SUSPENSION PS SEISMIC LOGGER/RECORDER CALIBRATION DATA FORM

INSTRUMENT DATA

System mfg.:	Oyo	Model no.:	3403
Serial no.:	160023	Calibration date:	10/12/2009
By:	Charles Carter	Due date:	10/12/2010
Counter mfg.:	Hewlett-Packard	Model no.:	53131A
Serial no.:	3546a09912	Calibration date:	1/12/2009
By:	Microprecision	Due date:	1/12/2010
Signal generator mfg.:	Agilent	Model no.:	33250A
Serial no.:	MY40000703	Calibration date:	7/15/2009
By:	Microprecision	Due date:	7/15/2010

SYSTEM SETTINGS:

Gain:	2
Filter	10KHz
Range:	See sample period in table below
Delay:	0
Stack (1 std)	1
System date = correct date and time	10/12/2009

PROCEDURE:

Set sine wave frequency to target frequency with amplitude of approximately 0.25 volt peak
Note actual frequency on data form.
Set sample period and record data file to disk. Note file name on data form.
Pick duration of 9 cycles using PSLOG.EXE program, note duration on data form, and save as .sps file. Calculate average frequency for each channel pair and note on data form.

Average frequency must be within +/- 1% of actual frequency at all data points.

Maximum error ((AVG-ACT)/ACT*100)% As found + 0.20% As left + 0.20%

Target Frequency (Hz)	Actual Frequency (Hz)	Sample Period (microS)	File Name	Time for 9 cycles Hn (msec)	Average Frequency Hn (Hz)	Time for 9 cycles Hr (msec)	Average Frequency Hr (Hz)	Time for 9 cycles V (msec)	Average Frequency V (Hz)
50.00	50.00	200	2	180.2	49.94	179.8	50.06	180.2	49.94
100.0	100.0	100	3	90.00	100.0	90.10	99.9	90.00	100.0
200.0	200.0	50	4	44.95	200.2	44.95	200.2	44.95	200.2
500.0	500.0	20	5	18.00	500.0	18.00	500.0	18.00	500.0
1000	1000	10	6	9.000	1000	8.990	1001.1	9.000	1000.0
2000	2000	5	7	4.495	2002	4.505	1998	4.500	2000

Calibrated by:	Charles Carter	10/12/2009	Charles Carter
	Name	Date	Signature
Witnessed by:	Kyu Han	10/12/2009	[signature]
	Name	Date	Signature

Suspension PS Seismic Recorder/Logger Calibration Data Form Rev 2.0 July 21, 2008

EXHIBIT A-12

PHOTO SCIENCE GEOSPATIAL SOLUTIONS REPORT

September 14, 2010

REVISED

Survey Report of BSB River Boring Locations

For

Parson Brinckerhoff Americas, Inc.
312 Elm Street Suite 2500
Cincinnati, OH 45202

PSI NO. 7069-005

presented by

PHOTO SCIENCE
Geospatial Solutions

2670 Wilhite Drive
Lexington, KY 40503
859-277-8700

PHOTO SCIENCE
Geospatial Solutions

2670 Wilhite Drive
Lexington, Kentucky 40503
Phone 859-277-8700 + Fax 859-277-8901
www.photoscience.com

September 14, 2010
Revision

Report of Field Survey

BSB River Boring Locations

PSI Project Number 7069-005

Purpose of this revision is to include three additional Borings that were conducted by H.C. Nutting after the original survey report was submitted.

One River Boring (R2-A) and two Land Borings (L1-2 and L1A-2) were surveyed on August 27, 2010.

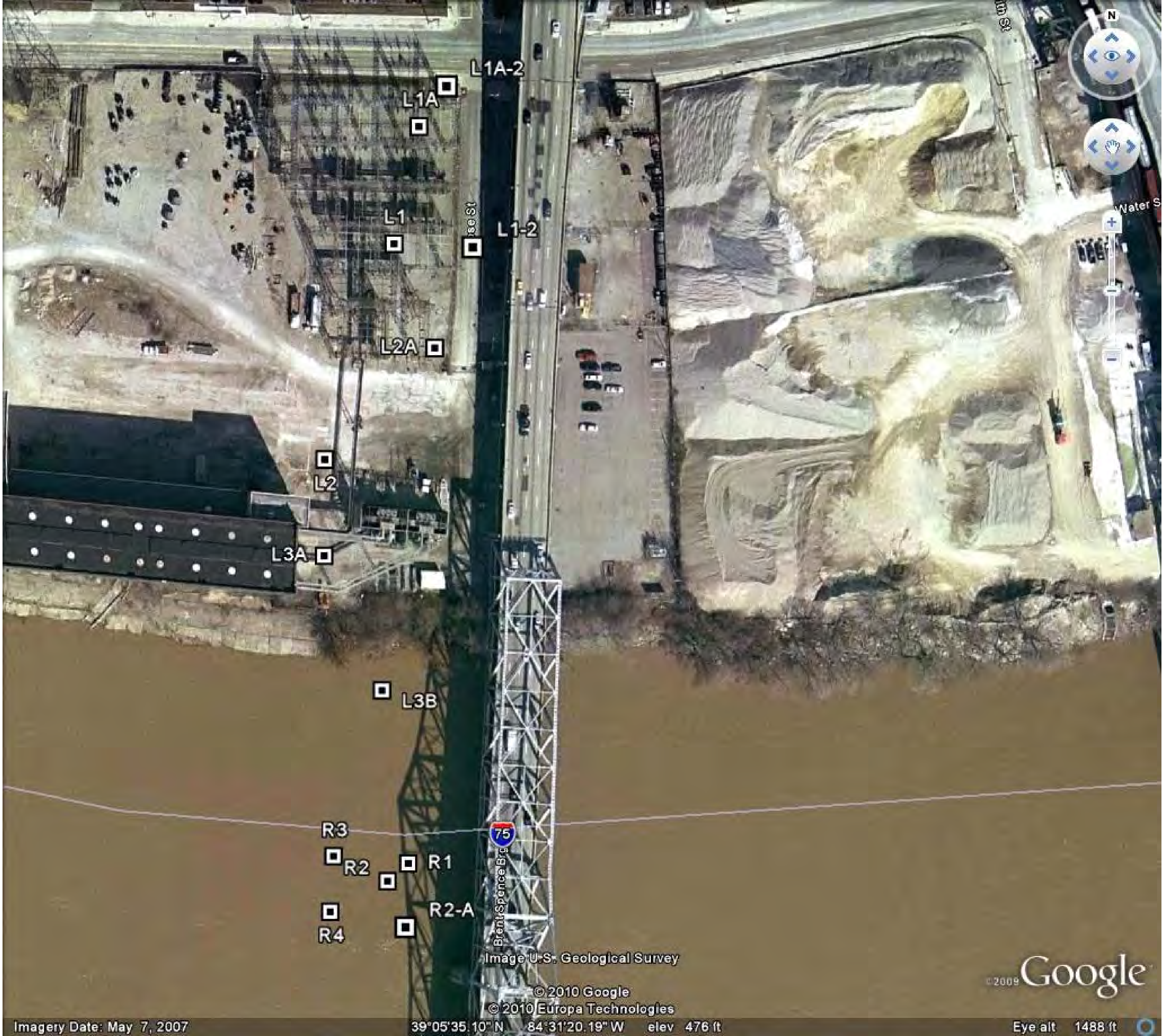
NAD83 KY Single Zone USFeet NAVD88

	Northing	Easting	Elevation	
L1A-2	4288504.15	5269616.44	489.72	Ground
L1-2	4288344.49	5269644.75	494.59	Ground
R2-A	4287656.44	5269581.56	457.64	Top of Deck

NAD83 Ohio South Zone USFeet NAVD88

	Northing	Easting	Elevation	
L1A-2	404978.23	1394463.06	489.72	Ground
L1-2	404817.69	1394485.67	494.59	Ground
R2-A	404132.45	1394398.08	457.64	Top of Deck

** It should be noted that no Borings were conducted at original sites L1 and L1A. These locations were within Duke Energy's property and not accessible for H.C. Nutting Drilling Rigs.



L1A-2
L1A
L1
L1-2
L2A
L2
L3A
L3B
R3
R2
R1
R2-A
R4
Water St
75
Image U.S. Geological Survey
© 2010 Google
© 2010 Europa Technologies
Google
Imagery Date: May 7, 2007
39°05'35.10" N 84°31'20.19" W elev 476 ft
Eye alt 1488 ft


L-IA-2


L-1-2


BSB
R-2A


BSB
R-2A

July 16, 2010

Survey Report of
BSB River Boring Locations

For

Parson Brinckerhoff Americas, Inc.
312 Elm Street Suite 2500
Cincinnati, OH 45202

PSI NO. 7069-005

presented by

PHOTO SCIENCE
Geospatial Solutions

2670 Wilhite Drive
Lexington, KY 40503
859-277-8700

2670 Wilhite Drive
Lexington, Kentucky 40503
Phone 859-277-8700 + Fax 859-277-8901
www.photoscience.com

July 16, 2010

Report of Field Survey

BSB River Boring Locations

PSI Project Number 7069-005

Photo Science, Inc. was given permission to proceed on June 29, 2010 by Duane Phelps of Parsons Brinckerhoff Americas, Inc for field surveying services at the Brent Spence Bridge Boring Site under Task Order 7.1.10.5. The field survey for this project was to locate approximately eighteen boring locations within the project area and a large culvert on the Kentucky side of the Ohio River.

A two-person RTK (real time kinematics) GPS crew was mobilized to the site on July 1, 2010. The crew was equipped with dual-frequency Trimble 5700 Base, Trimble R8 Rover GPS units, and Trimble TRIMMARK 3 Radio, to establish horizontal and vertical control values for the Boring Locations. The crew used BSB/PSI's control monuments 11 and 12 as base known positions.

Both RTK and Traditional surveying techniques were used in locating the Boring's. All River Borings were located with a TOPCON GTS223 Total Station by making use of two control points set by RTK near the River's Edge. When allowable, boring locations on land were located by direct RTK occupation. If the boring location wasn't suitable for direct occupation, a pair of control points were established nearby and then located with the total station.

At this time the culvert on the Kentucky side has not been surveyed. Photo Science is waiting on additional information from Mr. Phelps as to the location of the culvert. The Surveying Crew made a thorough search of the river bank for evidence of said culvert without uncovering any indication of its location. It's possible the culvert is below the waterline or is covered with debris.

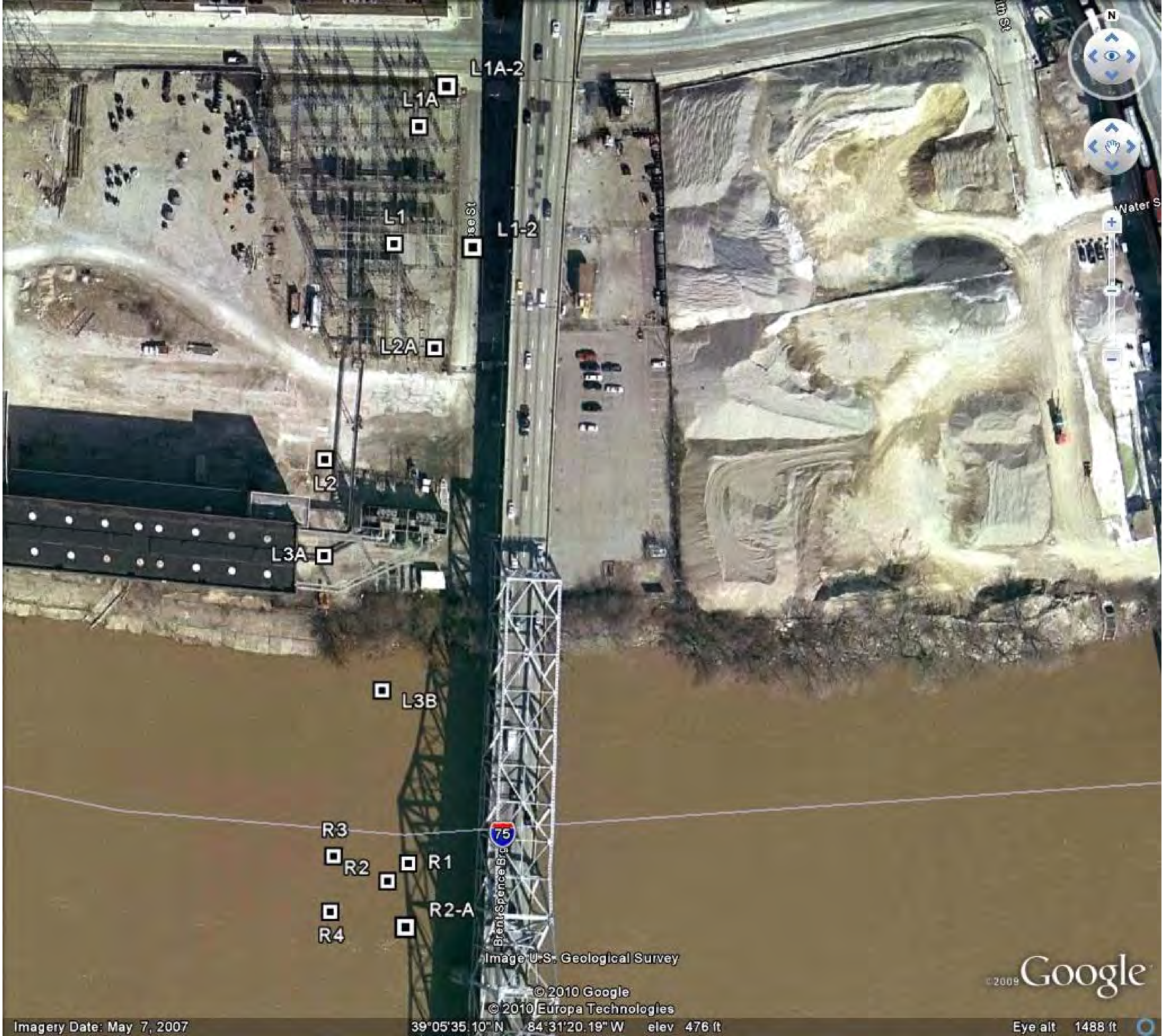
Final position summary sheet is provided for both, Kentucky State Plane Single and Ohio South Zones. Also included is a photo of each "survey setup" on the boring locations.

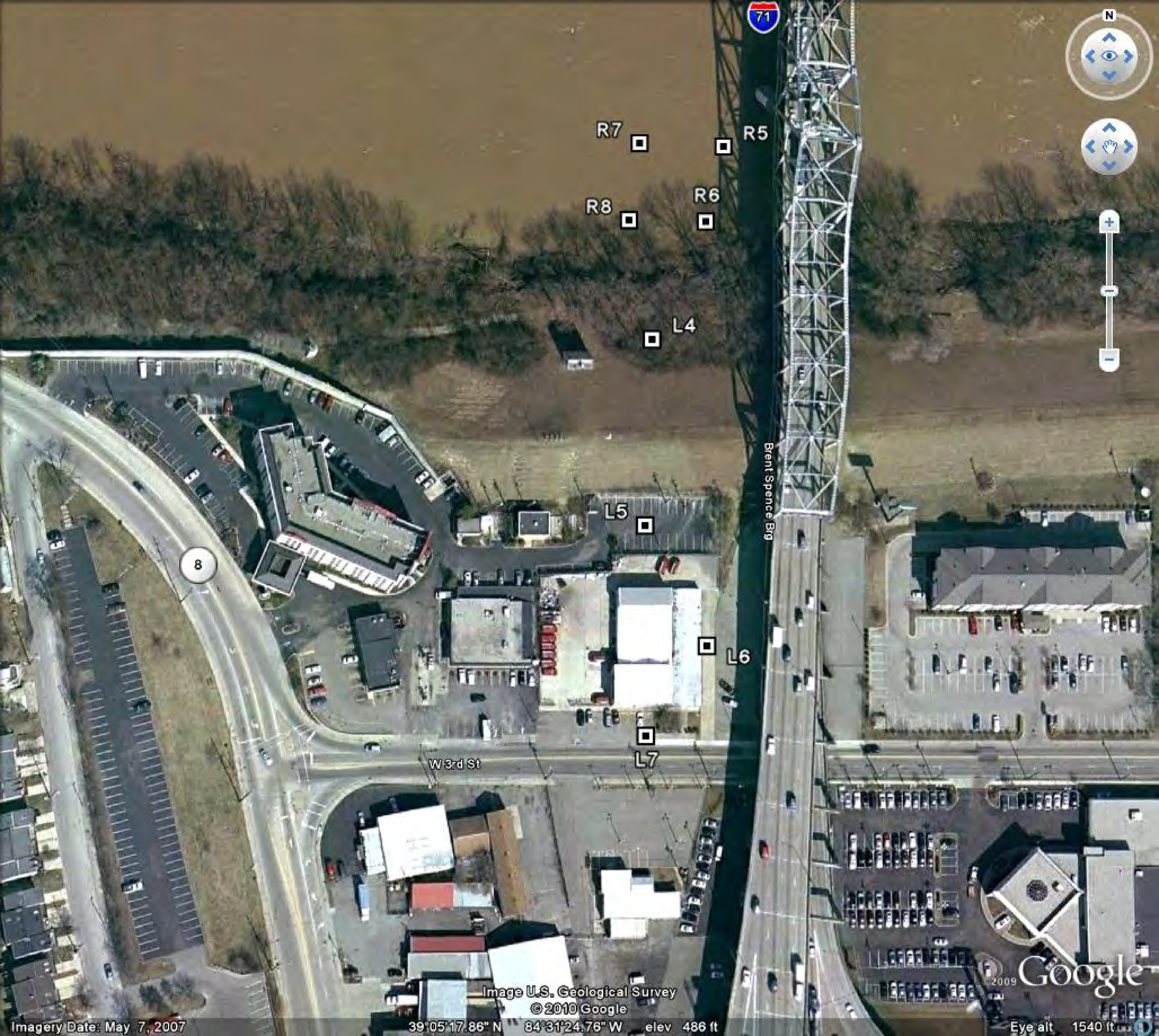
The horizontal datum is based on NAD 83 (2007) while the vertical datum is NAVD 88.

PHOTO SCIENCE, INC.
BSB BORE HOLES
KENTUCKY-OHIO
PSI #7069-005

PT#	Northing(Y) (SPC KY SINGLE) US FEET	Easting(X) (SPC KY SINGLE) US FEET	Elev(Z) NAVD 88 US FEET	Description
L2	4288131.73	5269499.24	496.26	L2 GROUND
L2A	4288244.14	5269607.61	494.50	L2A GROUND
L3A	4288035.34	5269496.11	496.05	L3A GROUND
L3B	4287897.88	5269553.98	458.66	L3B TOP OF DECK
L4	4286513.60	5269492.16	479.97	L4 GROUND
L5	4286320.80	5269488.42	486.33	L5 GROUND
L6	4286195.50	5269554.96	485.69	L6 GROUND
L7	4286100.55	5269491.85	484.41	L7 GROUND
R1	4287721.26	5269583.28	458.04	R1 TOP OF DECK
R2	4287702.96	5269562.17	458.10	R2 TOP OF DECK
R3	4287727.53	5269506.27	458.01	R3 TOP OF DECK
R4	4287670.82	5269503.75	457.98	R4 TOP OF DECK
R5	4286731.27	5269570.19	458.59	R5 TOP OF DECK
R6	4286646.07	5269550.32	457.04	R6 GROUND
R7	4286733.89	5269479.10	458.46	R7 TOP OF DECK
R8	4286646.68	5269468.06	455.70	R8 GROUND

PT#	Northing(Y) (SPC OH S) US FEET	Easting(X) (SPC OH S) US FEET	Elev(Z) NAVD 88 US FEET	Description
L2	404610.28	1394332.72	496.26	L2 GROUND
L2A	404718.74	1394445.00	494.50	L2A GROUND
L3A	404514.08	1394326.17	496.05	L3A GROUND
L3B	404374.67	1394379.10	458.66	L3B TOP OF DECK
L4	402993.71	1394268.14	479.97	L4 GROUND
L5	402801.20	1394257.55	486.33	L5 GROUND
L6	402673.64	1394319.58	485.69	L6 GROUND
L7	402581.01	1394253.15	484.41	L7 GROUND
R1	404197.15	1394402.10	458.04	R1 TOP OF DECK
R2	404179.62	1394380.36	458.10	R2 TOP OF DECK
R3	404206.16	1394325.38	458.01	R3 TOP OF DECK
R4	404149.58	1394320.85	457.98	R4 TOP OF DECK
R5	403208.43	1394353.84	458.59	R5 TOP OF DECK
R6	403124.01	1394330.96	457.04	R6 GROUND
R7	403214.29	1394262.92	458.46	R7 TOP OF DECK
R8	403127.54	1394248.79	455.70	R8 GROUND


L1A-2
L1A
L1
L1-2
L2A
L2
L3A
L3B
R3
R2
R1
R2-A
R4
75
Brent Spence Br
Water S
Image U.S. Geological Survey
© 2010 Google
© 2010 Europa Technologies
©2009 Google
Imagery Date: May 7, 2007
39°05'35.10" N 84°31'20.19" W elev 476 ft
Eye alt 1488 ft


71
R7
R5
R6
R8
L4
Brent Spence Brg
L5
8
L6
L7
W 3rd St
Image U.S. Geological Survey
© 2010 Google
©2009 Google
Imagery Date: May 7, 2007
39°05'17.86" N 84°31'24.76" W elev 486 ft
Eye alt 1540 ft


BSB
R-4


BSB
CORE HOLE
L2


BSB
CORE HOLE
L2A


BSB
CORE HOLE
L2A


BSB
CORE HOLE
L3A

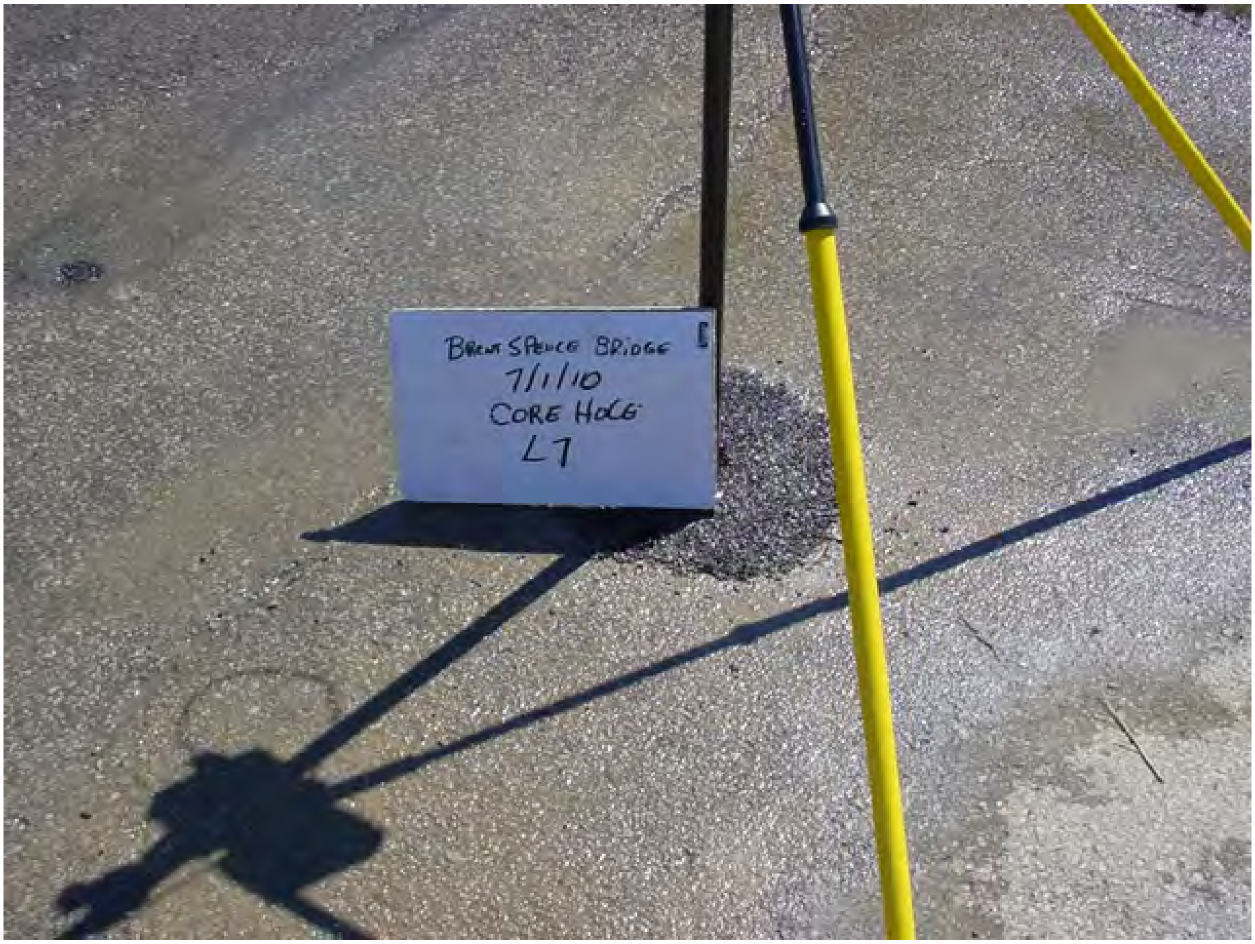

BSB
L3A


BSB
CORE HOLE
L4

Trimble
BRENT SPENCE BRIDGE
7/1/10
CORE HOLE
L5


SPENCE BRIDGE
7/1/10
CORE HOLE
L6


Core Hole
LG


BRENT SPENCE BRIDGE
7/1/10
CORE HOLE
L7


BSB
R-2


BSB
R-2


BSB
CORE HOLE
R-5


BSB
CORE HOLE
R-5


R-6
BSB
CORE HOLE
R-6


BSB
R-6


BSB
CORE HERE
R-7


BSB
Core Here
R-7


BSB
Core Here
R-8


BSB
Core Here
R-8


BSB
L-3B


BSB
L-3B


BSB
R-1


BSB
R-1


BSB
R-3


BSB
R-3


BSB
R-4

Kentucky Transportation Cabinet
Ohio Department of Highways
I-75 I-71 Control

Control Monument
Information Sheet

Site/Quad	Station Description (Description is to be complete) (type, size, depth set, etc.)	Station Designation
Covington	Concrete Monument 5/8"Rebar 2 1/2" Aluminum Cap 24" Concrete	MON 11
Locality/County: Kenton		Stamping on Mark: I75-I71 CONTROL

Date Set or Found (Date, with S or F)	Latitude	Longitude	Horiz. Datum	Zone	Vert.Datum
2/22/2010 S	39ø05'11.19732"N	84ø31'17.39378"W	NAD83	KY Single	1988
	Northing (KY SP1Z) (US Survey Feet): 4,285,659.91	Easting (KY SP1Z) (US Survey Feet): 5,270,007.96	Elevation: 496.51	Derived From: Level	Order Accuracy: 3rd
Person filling out form: AFS	Northing (OH SPSZ) (US Survey Feet): 402122.39	Easting (OH SPSZ) (US Survey Feet): 1394753.18	Geoid Model: Geoid 09	Ellipsoid Ht.: 384.64	Other Info.

Established by Agency	Project Factor	Back Station I.D.	Datum Azimuth - Distance to back station
Photo Science, Inc.			° ' " (ft)
Scale Factor: 1.00012828	Elev. Factor: 0.99998164	Ahead Station I.D.	Datum Azimuth - Distance to ahead station: ° ' " (ft)

Kentucky Registered Land Surveyor in charge of monumentation		Ky. Registration No.
	Anthony F. Stith	1877

Give a complete sketch and location description so that monument may be recovered by others

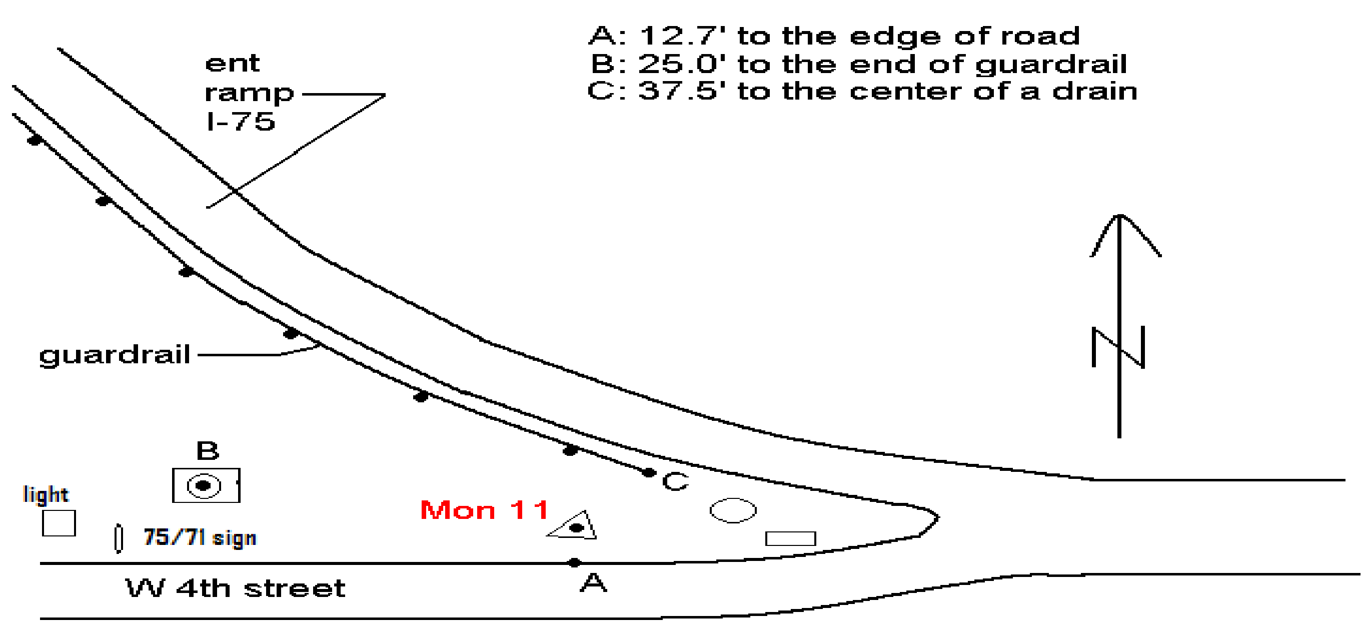


PHOTO SCIENCE
Geospatial Solutions

GPS CONTROL SURVEY
FIELD DATA SHEET

PAGE:
2

JOB REFERENCE	POINT ID:	Mon 11
I-75 &I-71 Control	Proj. No.:	7069-004

2670 Wilhite Drive
Lexington, KY 40503
859-277-8700 voice 859-277-8901 fax

PHOTO:

PHOTO:

PHOTO:

PHOTO:

Kentucky Transportation Cabinet
Ohio Department of Highways
I-75 I-71 Control

Control Monument
Information Sheet

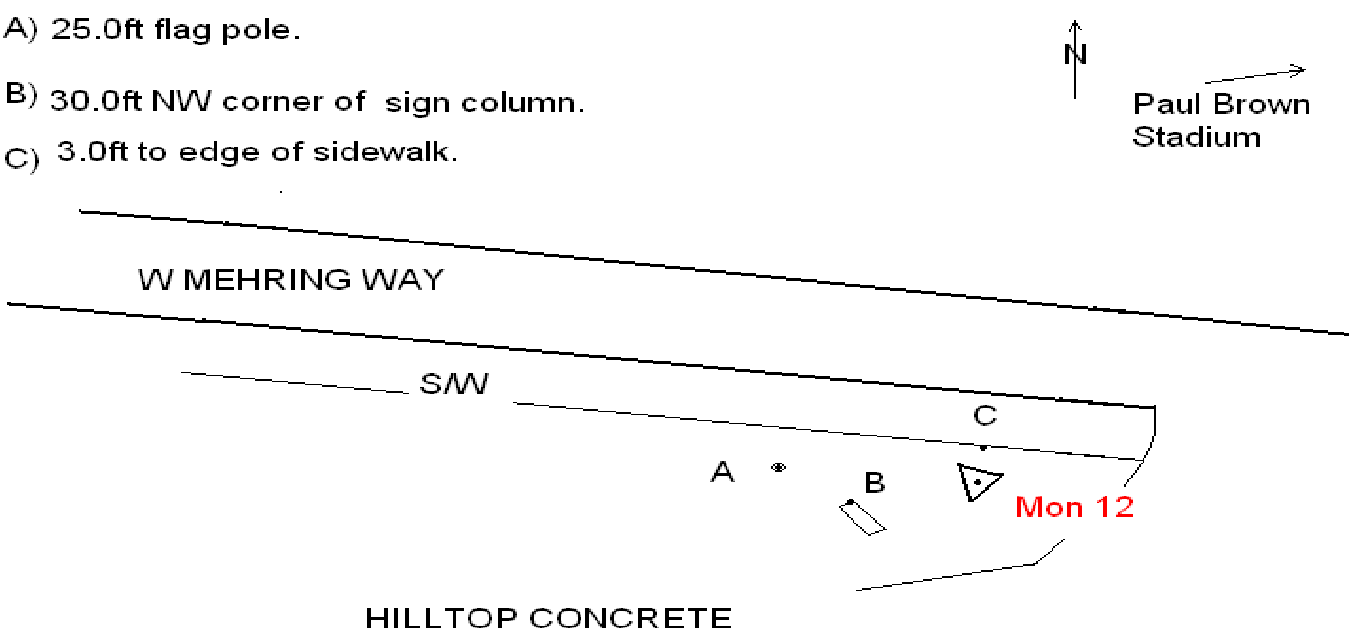
Site/Quad	Station Description (Description is to be complete) (type, size, depth set, etc.)	Station Designation
Covington	Concrete Monument 5/8"Rebar 2 1/2" Aluminum Cap 24" Concrete	MON 12
Locality/County: Hamilton		Stamping on Mark: I75-I71 CONTROL

Date Set or Found (Date, with S or F)	Latitude	Longitude	Horiz. Datum	Zone	Vert.Datum
2/22/2010 S	39ø05'39.45612"N	84ø31'08.62875"W	NAD83	KY Single	1929
	Northing (KY SP1Z) (US Survey Feet): 4,288,528.22	Easting (KY SP1Z) (US Survey Feet): 5,270,661.40	Elevation: 486.48	Derived From: Level	Order Accuracy: 3rd
Person filling out form: AFS	Northing (OH SPSZ) (US Survey Feet): 404965.15	Easting (OH SPSZ) (US Survey Feet): 1395508.02	Geoid Model: Geoid 09	Ellipsoid Ht.: 374.55	Other Info.
Established by Agency: Photo Science, Inc.	Project Factor	Back Station I.D.	Datum Azimuth - Distance to back station: ° ' " (ft)		
Scale Factor: 1.00013119	Elev. Factor: 0.99998212	Ahead Station I.D.	Datum Azimuth - Distance to ahead station: ° ' " (ft)		

Kentucky Registered Land Surveyor in charge of monumentation		Ky. Registration No.
	Anthony F. Stith	1877

Give a complete sketch and location description so that monument may be recovered by others

A) 25.0ft flag pole.
B) 30.0ft NW corner of sign column.
C) 3.0ft to edge of sidewalk.



GPS CONTROL SURVEY FIELD DATA SHEET		PAGE:	2
JOB REFERENCE	I-75 & I-71 Control	POINT ID:	Mon 12
		Proj. No.:	7069-004

2670 Wilhite Drive
Lexington, KY 40503
859-277-8700 voice 859-277-8901 fax

PHOTO:

PHOTO:

PHOTO:

PHOTO:

APPENDIX B
LABORATORY TESTING

EXHIBIT B-1
LABORATORY TEST RESULTS
(Sieve, Hydrometer, Atterberg Limits, Moisture)

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-1	1	6	7.5									17.2		
	2	7.5	9	16.9	4.7	27.3	35.7	15.4	27	17	10	30.4	A-4a(3)	
	3	10	11.5	0.0	0.0	36.5	46.6	16.9	24	16	8	19.0	A-6a(6)	
	5	14	16.5	0.0	0.4	23.9	55.8	19.9	27	17	10	22.4	A-6b(8)	
	6	17.5	19									22.9		1.60
	7	20	21.5	0.0	0.0	43.5	40.3	16.2	20	18	2	20.6	A-6a(4)	
	8	25	26.5									18.9		
	9	30	31.5	0.0	1.4	35.9	44.3	18.4	23	17	6	18.7	A-4a(6)	
	10	35	36.5									27.9		
	11	40	41.5	37.3	19.1	17.3	19.5	6.8	24	16	8	14.5	A-2-4(0)	
	12	45	46.5	61.4	19.7	11.1	6.1	1.7	NP	NP	NP	10.3	A-1-a(0)	
	13	50	51.5									15.1		
	14	55	56.5	11.6	32.1	38.6	13.5	4.2	NP	NP	NP	16.2	A-3a(0)	
	15	60	61.5									18.5		
	16	65	66.5	6.1	27.1	53.9	8.8	4.1	NP	NP	NP	19.1	A-3a(0)	
	17	70	71.5									10.7		
	18	75	76.5	32.5	20.3	37.6	6.2	3.4	NP	NP	NP	15.4	A-1-b(0)	
	19	80	81.5									59.2		
	20	85	86.5	0.9	20.6	70.7	4.1	3.7	NP	NP	NP	21.7	A-3(0)	
	21	90	91.5									11.3		
	20	85	86.5	*										
	21	90	91.5	*										
	22	95	96.5	47.8	25.5	19.9	5.1	1.7	NP	NP	NP	11.4	A-1-b(0)	
	23	100	101.5									23.0		
	24	105	106.5									22.7		
	25	110	11.5	58.0	21.2	12.6	7.1	1.1	NP	NP	NP	12.1	A-1-a(0)	
	26	115	116.5									9.5		
	27	120	121.5	10.5	52.4	29.2	6.6	1.3	NP	NP	NP	15.4	A-1-b(0)	
	28	125	126.5									9.6		

H.C. Nutting, a Terracon Company
611 Lunken Park Drive, Cincinnati, Ohio 45226
P [513] 321.5816 F [513] 321.4540

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-1A	1	5	6.5									28.6		
	2	7.5	9	0.0	0.0	32.4	49.0	18.6	26	16	10	19.4	A-4a(7)	
	3	10	11.5	0.4	1.8	17.8	51.9	18.1	24	16	8	20.0	A-4a(7)	
	5	15	16.5	0.0	2.0	48.2	38.5	11.3	NP	NP	NP	22.0	A-4a(0)	
	6	17.5	19									21.4		
	7	20	21.5	0.0	0.0	29.9	51.1	19.0	25	16	9	21.9	A-4b(7)	
	9	25	26.5									24.9		
	10	30	31.5	0.0	0.0	15.4	62.6	22.0	28	18	10	29.0	A-4b(8)	
	11	35	36.5	0.0	0.0	30.1	54.2	15.7	27	20	7	26.1	A-4b(7)	
	12	40	41.5	52.3	25.1	93.0	9.6	3.7	NP	NP	NP	8.1	A-1-a(0)	
	13	45	46.5	18.6	27.3	36.9	12.8	4.4	NP	NP	NP	17.6	A-3a(0)	
	14	50	51.5	0.6	22.3	59.5	13.1	4.5	NP	NP	NP	22.2	A-3a(0)	
	15	55	56.5									18.3		
	16	60	61.5	21.8	13.3	33.9	20.9	10.1	NP	NP	NP	20.8	A-2-4(0)	
	17	65	66.5									17.4		
	18	70	71.5	2.6	32.2	57.2	3.8	4.2	NP	NP	NP	19.7	A-3(0)	
	19	75	76.5									14.0		
	20	80	81.5	7.3	28.2	55.5	4.9	4.1	NP	NP	NP	18.5	A-3(0)	
	21	85	86.5									20.9		
	22	90	91.5	39.1	20.5	33.8	4.0	2.6	NP	NP	NP	13.9	A-1-b(0)	
	23	95	96.5									20.1		
	24	100	101.5	32.2	30.9	27.9	6.0	3.0	NP	NP	NP	15.1	A-1-b(0)	
	25	105	106.5									15.8		
	26	110	111.5	43.0	34.8	15.9	3.9	2.4	NP	NP	NP	9.2	A-1-b(0)	
	27	115	116.5									8.1		
	28	120	121.5	74.2	7.0	11.6	5.1	2.1	Insufficient Sample			8.5	A-1-a(0)	

Classification Test Data															
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-2	3	15	16.5									35.1		
	4	17.5	19									36.9		
	5	20	21.5									34.2		
	6	25	26.5									40.4		
	7	30	31.5	27.0	23.0	9.5	20.5	20.0	35	17	18	15.7	A-6b(3)	
	8	35	36.5									30.3		
	9	40	41.5	0.0	0.2	7.7	60.5	31.6	48	29	19	38.2	A-7-6(13)	
	11	46.5	48									31.6		
	12	50	51.5									23.7		
	14	60	61.5									21.5		
	15	65	66.5	7.8	45.4	37.6	6.1	3.1				24.8	A-1-b(0)	
	17	75	76.5									31.0		
	20	90	91.5									31.7		
	21	95	96.5	3.2	33.7	54.4	5.9	2.8				20.1	A-3(0)	
	22	100	101.5									14.7		
	23	105	106.5									8.3		
	24	110	111.5									19.0		

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Classification Test Data

Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-2A	1	3	4.5									8.8		
	2	5	6.5									9.4		
	3	7.5	9									45.8		
	4	10	11.5									21.9		
	5	12.5	14									14.8		
	6	15	16.5									14.4		
	7	18	20	0.0	0.2	38.1	49.5	12.2	NP	NP	NP	32.4	A-4a(0)	4.90
	8	20	21.5									23.8		
	9	25	26.5									28.6		
	10	30	31.5									44.1		
	11	35	36.5	47.8	23.9	16.5	8.9	2.9	NP	NP	NP	12.6	A-1-b(0)	
	13	41.5	43									21.5		
	14	45	46									7.6		
	15	50	51.5	30.3	30.3	26.0	9.9	3.5	NP	NP	NP	15.0	A-1-b(0)	
	16	55	56.5									16.5		
	17	60	61.5									14.8		
	18	65	66.5									15.7		
	19	70	71.5	0.7	35.3	54.0	5.8	4.2	NP	NP	NP	18.8	A-3(0)	
	20	75	76.5	37.3	32.5	21.3	6.4	2.5				10.5	A-1-b(0)	
	21	80	81.5									9.5		
	22	85	86.5	49.6	34.0	9.8	4.1	2.5	NP	NP	NP	11.3	A-1-b(0)	
	23	90	91.5									22.0		
	24	95	96.5	21.2	39.9	29.2	7.4	2.3				15.4	A-1-b(00	
	25	100	101.5									10.4		
	26	105	106.5	1.0	3.4	79.6	12.2	3.8	NP	NP	NP	23.7	A-3a(0)	
	27	110	111.5									22.7		
	28	115	116.5	53.9	22.1	13.6	7.3	3.1				8.2	A-1-a(0)	
	29	120	121.5									6.8		
	30	125	126.5									6.5		

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Classification Test Data															
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-3	2	21	22.5									29.3		
	3	22.5	24	29.1	36.0	24.3	6.1	4.5	NP	NP	NP	25.7	A-1-b(0)	
	6	30	31.5									7.5		
	7	32.5	34	6.1	25.9	63.0	2.6	2.4	NP	NP	NP	23.1	A-3(0)	
	8	35	36.5									52.0		
	9	40	41.5	1.1	38.6	56.3	2.0	2.0	NP	NP	NP	24.6	A-3(0)	
	10	45	46.5									4.6		
	11	50	51.5	61.7	13.8	15.9	6.2	2.4	NP	NP	NP	10.4	A-1-a(0)	
	12	55	56.5	4.1	31.9	54.1	6.0	3.9	NP	NP	NP	24.2	A-3(0)	
	13	60	61.5	11.7	27.9	54.8	3.4	2.2	NP	NP	NP	20.6	A-3(0)	
	14	65	66.5	27.2	36.1	28.9	5.2	2.6	NP	NP	NP	14.5	A-1-b(0)	
	15	70	71.5	21.7	58.0	14.6	3.0	2.7	NP	NP	NP	16.3	A-1-b(0)	
	16	75	76.5	9.3	52.2	32.8	3.5	2.2	NP	NP	NP	18.8	A-1-b(0)	

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Classification Test Data

Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-3A	1	7.5	9									24.6		
	2	10.0	11.5									21.6		
	3	12.5	14.0									35.8		
	4	15.0	16.5									39.2		
	5	17.5	19.0	49.1	30.5	9.8	6.5	4.1	Insufficient Sample			64.5	A-1-b	
	6	20.0	21.5									39.9		
	7	25.0	26.5									43.3		
	8	30.0	31.5	25.2	32.4	24.6	9.0	8.8	Insufficient Sample			61.8	A-1-b	
	9	35.0	36.5									46.5		
	10	40.0	41.5									85.8		
	11	45.0	46.5									29.1		
	12	50.0	51.5	17.2	17.2	21.5	24.0	20.1	26	17	9	26.6	A-4a(2)	
	14	60.0	61.5									17.9		
	15	65.0	66.5									28.6		
	16	70.0	71.5									17.8		
	17	75.0	76.5	6.6	38.4	45.2	4.6	5.2	NP	NP	NP	20.3	A-3	
	18	80.0	81.5									18.7		
	19	85.0	86.5									14.0		
	20	90.0	91.5									23.6		
	21	95.0	96.5									18.2		
	23	105.0	106.5	38.7	33.1	23.4	1.4	3.4	NP	NP	NP	17.7	A-1-b	
	24	110.0	111.5									14.2		
	25	115.0	116.5									16.7		
	26	120.0	121.5	59.1	17.2	16.8	3.9	3.0	NP	NP	NP	12.2	A-1-a	
	27	122.5	124.0									19.6		

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-4	1	0.0	1.5									37.7		
	2	2.5	4.0									18.7		
	3	5.0	6.5									26.1		
	4	7.5	9.0	10.0	24.1	18.2	31.2	16.5	35	24	11	26.9	A-6a(3)	
	5	10.0	11.5									132.4		
	6	12.5	14.0									17.6		
	8	17.5	19.0									23.5		
	9	20.0	21.5									27.8		
	10	25.0	26.5	0.0	0.2	1.8	61.6	36.4	50	29	21	44.3	A-7-6(14)	5.40
	ST/11	30.0	32.0	0.0	0.0	0.6	62.1	37.3	46	25	21		A-7-6(14)	
	12	32.0	33.5									43.6		
	13	35.0	36.5									31.9		
	14	40.0	41.5									24.4		
	15	45.0	46.5									7.0		
	16	50.0	51.5									10.0		
	17	55.0	56.5	53.4	9.2	27.2	7.2	3.0	NP	NP	NP	12.2	A-1-b(0)	
	18	60.0	61.5									13.5		
	19	65.0	66.5									7.4		
	20	70.0	71.5	56.0	24.5	13.3	3.7	2.5	NP	NP	NP	10.5	A-1-a(0)	
	21	75.0	76.5									9.6		
	22	80.0	81.5	19.1	63.1	11.5	3.4	2.9	NP	NP	NP	17.5	A-1-b(0)	
	23	85.0	86.5									15.2		
	24	90.0	91.5									5.6		
	25	95.0	96.5									11.4		

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-5	1	5.0	6.5									37.8		
	2	7.5	9.0									19.6		
	3	10.0	11.5									20.5		
	4	12.5	14.0									16.5		
	5	15.0	16.5									25.8		
	7	20.0	21.5									22.8		
	8	23.0	25.0	0.0	0.2	15.6	45.1	38.1	29	17	12		A-6a(9)	
	9	25.0	26.5									22.1		
	10	30.0	31.5									26.8		
	11	35.0	36.5									27.4		
	12	38.0	40.0	0.0	0.2	20.9	47.7	31.2	29	19	10		A-4a(8)	
	13	40.0	41.5									28.0		
	14	45.0	46.5									13.4		
	15	50.0	51.5									17.7		
	16	55.0	56.5									7.8		
	17	60.0	61.5									13.9		
	18	65.0	66.5									10.8		
	19	70.0	71.5									10.8		
	20	75.0	76.5	55.4	17.5	17.2	6.8	3.1	NP	NP	NP	11.5	A-1-a(0)	
	21	80.0	81.5									10.1		
	22	85.0	86.5									8.6		
	23	90.0	91.5									9.8		
	24	95.0	96.5									7.3		

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-6	1	7.5	9.0									20.8		
	2	10.0	11.5	0.0	0.0	10.4	53.0	36.6	33	19	14	22.4	A-6a(10)	
	3	12.5	14.5									26.3		
	3/ST	14.5	16.0	0.0	0.0	8.8	54.2	37.0	33	19	14		A-6a(10)	
	4	14.5	16.0									24.2		
	5	20.0	21.5									24.7		
	6	25.0	26.5	0.0	0.0	5.3	61.3	33.4	32	10	12	26.5	A-6a(9)	
	6/ST	30.0	32.0	0.0	0.2	9.6	55.8	34.4	30	19	11		A-6a(8)	
	7	35.0	36.5									26.6		
	8	40.0	41.5									27.1		
	9	45.0	46.5									20.6		
	10	50.0	51.5	13.5	6.0	9.4	57.1	14.0	NP	NP	NP	22.8	A-4b(00	
	11	55.0	56.5									20.5		
	12	60.0	61.5									7.7		
	13	65.0	66.5									15.6		
	14	70.0	71.5									9.3		
	15	75.0	76.5									10.6		
	16	80.0	81.5									8.8		
	17	85.0	86.5									12.3		
	18	90.0	91.5	31.7	28.1	26.5	10.0	3.7	NP	NP	NP	9.0	A-1-b(0)	
	19	95.0	96.5									8.2		
	20	100.0	101.5									7.3		
	21	105.0	106.5									10.2		
	22	108.5	110.0									NO. REC.		

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
L-7	1	5	6.5									25.8		
	2	7.5	9	23.3	17.1	21.6	20.1	17.9	NP	NP	NP	18.9	A-4a(0)	
	3	10	11.5									22.9		
	4	13	15	32.0	1.6	7.5	29.5	29.4	29	17	12		A-6a(6)	
	5	15	16.5									27.2		
	6	17.5	19									22.6		
	7	20	22	0.0	0.0	18.3	43.9	37.8	31	17	14		A-6a(10)	
	8	22	23.5	0.0	0.2	17.1	47.9	34.8	32	18	14	23.4	A-6a(10)	
	9	25	26.5									30.1		
	10	30	31.5									23.7		
	11	33	35	32.3	4.0	14.3	25.6	23.8	31	17	14		A-6a(4)	
	12	35	36.5									22.1		
	13	40	41.5									26.9		
	14	45	46.5									32.6		
	15	50	51.5									15.5		
	16	55	56.5									10.4		
	17	60	61.5	55.4	27.8	10.3	4.1	2.4	NP	NP	NP	10.3	A-1-a(0)	
	18	65	66.5									19.3		
	19	70	71.5									12.3		
	20	75	76.5	60.8	18.4	12.7	5.5	2.6	NP	NP	NP	8.9	A-1-a(0)	
	21	80	81.5									9.0		
	22	85	85.5									103.0		
	23	90	90.4	52.6	14.4	16.5	10.6	5.9	Insufficient Sample			8.2	A-1-b(00	
	24	95	96.5									9.9		
R-1	1	32	33.5									12.0		
	2	32.5	35	69.5	20.8	7.3	1.2	1.2	NP	NP	NP	12.5	A-1-a(0)	
	4	36.5	38	3.1	63.5	28.6	1.9	2.9				23.0	A-1-b(0)	
	6	39.5	41	2.1	23.9	69.1	1.9	3.0	NP	NP	NP	23.0	A-3(0)	
	7	41	42.5									23.2		
	8	42.5	44									20.8		
	9	45	46.5									20.4		
	11	55	56.5	1.7	32.2	61.4	2.3	2.4				21.2	A-3(0)	
	12	60	61.5									25.8		
	13	65	66.5									13.8		
	14	70	71.5									14.0		
	15	75	76.5									18.9		

Classification Test Data															
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
R-2	1	32	33.5	87.5	10.8	1.5	0.0	0.2	Insufficient Sample			12.6	A-1-a(0)	
	2	33.5	35	21.6	57.9	16.3	2.4	1.8	NP	NP	NP	17.0	A-1-b(0)	
	3	35	36.5	61.8	19.5	14.3	2.0	2.4	Insufficient Sample			14.7	A-1-a(0)	
	4	36.5	38	23.3	20.9	51.0	1.5	3.3	NP	NP	NP	17.6	A-3(0)	
	6	39.5	41	55.1	13.0	26.4	4.0	1.5	NP	NP	NP	14.7	A-1-b(0)	
	8	45	46.5	9.4	17.5	68.8	1.6	2.7	NP	NP	NP	21.5	A-3(0)	
	9	47.5	49									27.6		
	10	50	51.5	0.0	6.6	86.8	3.6	3.0	NP	NP	NP	24.5	A-3(0)	
	11	55	56.5									26.5		
	12	60	61.5	1.0	12.3	80.1	3.1	3.5	NP	NP	NP	20.3	A-3(0)	
	13	65	66.5									24.4		
	14	70	71.5	25.0	46.0	23.3	3.0	2.7	NP	NP	NP	14.9	A-1-b(0)	
	15	75	76.5	17.4	39.6	37.1	2.9	3.0	NP	NP	NP	16.8	A-1-b(0)	

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Classification Test Data															
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
R-2A	2	29	30.5									21.2		
	3	30.5	32	45.4	37.2	6.1	10.0	1.3	NP	NP	NP	17.6	A-1-b(0)	
	5	35	36.5									16.4		
	6	36.5	38									27.5		
	7	38	39.5									27.3		
	8	42.5	44									18.1		
	9	45	46.5	32.3	35.6	27.2	3.0	1.9	NP	NP	NP	16.5	A-1-b(0)	
	11	50	51.5									29.3		
	12	55	56.5	3.9	29.6	61.9	2.9	1.7	NP	NP	NP	21.3	A-3(0)	
	13	60	61.5									24.4		
	14	65	66.5	60.6	14.5	16.8	6.3	1.8	NP	NP	NP	13.2	A-1-a(0)	
	15	70	71.5									13.5		
	16	75	76.5									13.2		
R-3	3	34	35.5	38.3	37.3	20.0	3.3	1.1	NP	NP	NP	14.9	A-1-b(0)	
	4	35.5	37	57.8	33.5	5.9	1.3	1.5	NP	NP	NP	16.7	A-1-a(0)	
	5	37	38.5	6.3	68.9	17.5	3.1	4.2	NP	NP	NP	18.1	A-1-b(00	
	6	38.5	40		No Sample at this depth									
	7	40	41.5	1.1	39.2	54.6	2.6	2.5	NP	NP	NP	18.8	A-3(0)	
	8	42.5	44									24.5		
	9	45	46.5	8.4	48.0	39.4	2.5	1.7	NP	NP	NP	20.3	A-1-b(0)	
	10	47.5	49	25.4	36.0	34.1	1.4	3.1	NP	NP	NP	18.7	A-1-b(0)	
	12	55	56.5									22.4		
	13	60	61.5	32.1	34.5	28.6	2.3	2.5	NP	NP	NP	19.2	A-1-b(0)	
	14	65	66.5	70.3	12.6	13.4	2.3	1.4	NP	NP	NP	10.0	A-1-a(0)	
	15	70	71.5	56.7	29.4	9.1	2.6	2.2	NP	NP	NP	15.5	A-1-a(0)	
	16	75	76.5	52.1	29.4	12.1	3.8	2.6	NP	NP	NP	13.0	A-1-a(0)	
	17	80	81.5									No Samp.		

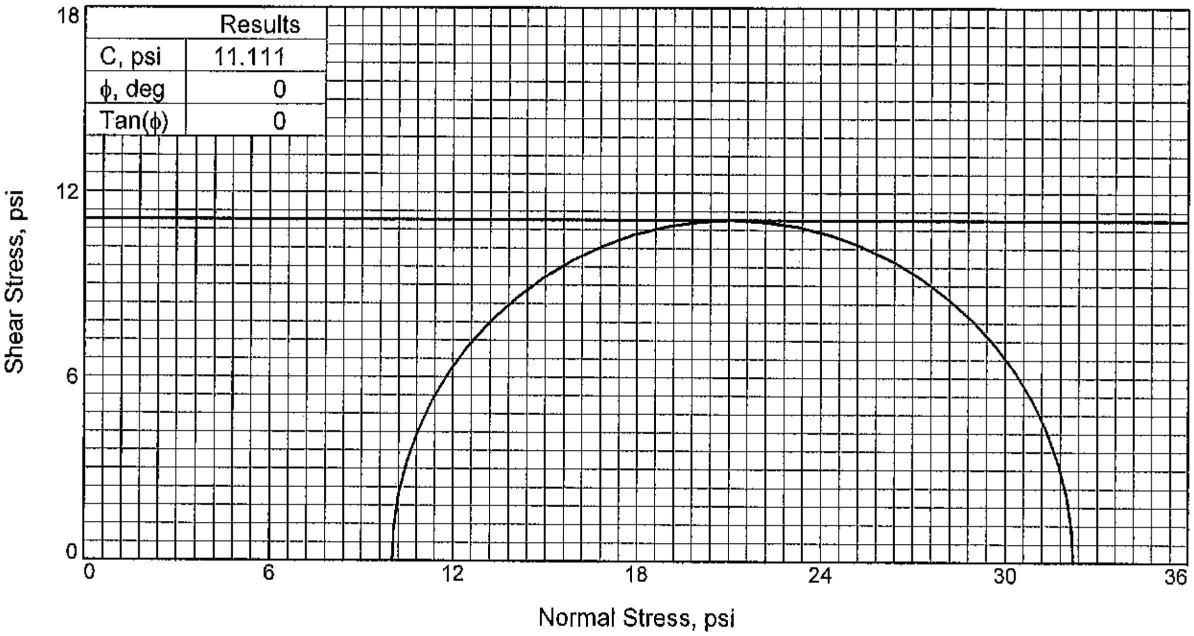
Classification Test Data															
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
R-4	1	33.5	35	18.0	67.9	9.5	2.9	1.7	NP	NP	NP	17.4	A-1-b(0)	
	2	35	36.5	37.3	51.7	9.7	0.1	1.2	NP	NP	NP	19.4	A-1-b(0)	
	3	36.5	38	28.6	59.6	9.1	1.3	1.4	NP	NP	NP	13.9	A-1-b(0)	
	4	38	39.5	2.1	84.8	11.4	0.2	1.5	NP	NP	NP	21.1	A-3a(0)	
	5	39.5	41	38.1	42.2	14.4	3.2	2.1	NP	NP	NP	16.6	A-1-b(0)	
	6	41	42,5	10.5	54.9	27.9	3.5	3.2	NP	NP	NP	25.3	A-1-b(0)	
	7	42.5	44									25.2		
	8	45	46.5	5.1	17.7	74.4	0.4	2.4	NP	NP	NP	21.8	A-3(0)	
	9	47.5	49									No Samp.		
	10	50	51.5	1.8	22.4	71.7	0.7	3.4	NP	NP	NP	26.0	A-3(0)	
	11	55	56.5	3.5	7.3	81.7	3.5	4.0	NP	NP	NP	21.8	A-3(0)	
	12	60	61.5									23.9		
	13	65	66.5									13.4		
	14	70	71.5	77.8	12.8	6.1	1.6	1.8	NP	NP	NP	16.2	A-1-a(0)	
	15	75	76.5	51.9	36.5	6.9	2.5	2.2	NP	NP	NP	15.8	A-1-a(0)	
	16	80	81.5									No Samp		
R-5	1	16	17.5	18.5	16.3	49.5	9.9	5.8	NP	NP	NP	27.5	A-3a(0)	
	2	17.5	19	30.4	17.6	40.9	6.9	4.2	NP	NP	NP	39.2	A-3a(0)	
	3	19	20.5	36.3	18.5	35.7	5.1	4.4	NP	NP	NP	39.5	A-1-b(0)	
	4	20.5	22	0.0	0.4	24.4	52.3	22.9	31	20	11	30.3	A-6a(8)	
	6	23.5	25	0.0	0.2	9.3	53.5	37.0	36	21	15	43.8	A-6a(10)	
	7	25	26.5									49.2		
	8	27.5	29									10.2		
	9	30	31.5	44.8	38.2	9.7	4.0	3.3	NP	NP	NP	10.2	A-1-b(0)	
	10	32.5	34									7.1		
	11	35	36.5									21.2		
	12	40	41.5	47.5	30.3	12.7	6.3	3.2	NP	NP	NP	13.6	A-1-b(0)	
	13	45	46.5									23.3		
	14	50	51.5	44.2	39.0	10.3	4.1	2.4	NP	NP	NP	16.3	A-1-b(0)	
	15	55	56.5	50.3	30.4	10.2	5.4	3.7	NP	NP	NP	18.1	A-1-a(0)	
	16	60	61.5									12.9		
	17	65	66.5	12.6	20.7	5.9	3.2	4.5	NP	NP	NP	19.4	A-3(0)	
	18	70	71.5									7.3		
	19	75	75.5	49.2	20.4	10.7	13.2	19.7	Insufficient Sample			16.4	A-1-b(0)	

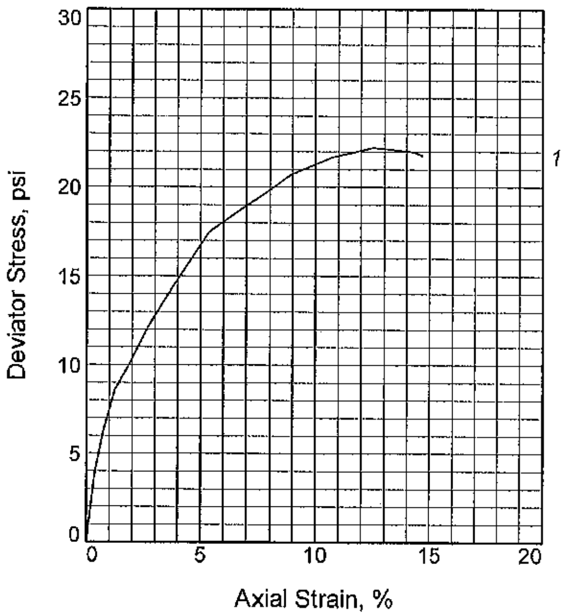
Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
R-6	1	0	1.5	0.5	1.3	24.9	49.8	23.5	28	19	9	28.7	A-4a(8)	
	2	2.5	4	0.0	0.2	2.2	67.1	30.5	35	22	13	32.2	A-6a(9)	
	ST/3	5	7	0.3	0.4	5.9	60.5	32.9	38	22	16		A-6b(10)	
	4	7	8.5	0.0	0.2	21.5	52.3	26.0	30	20	10	33.1	A-4b(8)	
	5	10	11.56	0.0	0.2	30.8	41.9	27.1	30	17	13	26.0	A-6a(8)	
	6	12.5	14	0.0	0.0	39.0	38.7	22.3	26	17	9	24.0	A-4a(5)	
	ST/7	15	17	0.0	0.4	18.1	54.9	26.6	34	23	11		A-6a(8)	
	8	17	18.5	5.9	0.8	43.7	42.9	18.8	26	17	9	26.3	A-4a(3)	
	9	20	21.5	0.4	0.6	31.7	40.6	26.7	33	23	10	48.2	A-4a(6)	
	10	25	26.5	84.3	4.4	2.2	6.5	2.6	Insufficient Sample			10.0	A-1-a(0)	
	11	30	31.5									8.4		
	12	35	36.5	58.0	20.2	11.9	6.0	3.9	NP	NP	NP	9.3	A-1-a(0)	
	13	40	41.5									8.5		
	14	45	46.5	33.5	38.8	20.2	4.9	2.6	NP	NP	NP	13.0	A-1-b(0)	
	15	50	51.5	55.9	21.2	18.2	3.1	1.6	NP	NP	NP	17.1	A-1-a(0)	
	16	55	56.5									14.1		
	17	60	61.5									21.7		
	18	65	66.5	54.3	18.4	19.7	5.2	2.4	NP	NP	NP	14.7	A-1-a(0)	
	19	70	71.5									6.4		
	20	75	75.9	57.6	12.6	17.4	8.8	3.6	NP	NP	NP	8.7	A-1-a(0)	
	21	80	80.4									5.8		
	22	84										No Samp.		
R-7	2	22.5	24	5.7	2.0	9.5	44.5	38.3	42	22	20	45.6	A-7-6(12)	
	3	24	25.5	32.9	3.3	20.6	25.3	17.9	32	18	14	24.4	A-6a(3)	
	4	22.5	27	34.5	3.5	23.8	22.0	16.2	Insufficient Sample			31.4	A-4a(0)	
	5	27	28.5	35.5	3.9	20.6	24.8	15.2	Insufficient Sample			33.2	A-4a(0)	
	8	32.5	34	67.8	19.3	5.0	5.3	2.6	Insufficient Sample			12.3	A-1-a(0)	
	9	35	36.2	No sample at this depth										
	10	37.5	39									10.0		
	11	40	41.5	61.4	16.4	11.0	7.4	3.8	NP	NP	NP	10.2	A-1-a(0)	
	12	45	46.5									7.0		
	13	50	51.5	57.2	26.9	84.0	4.6	2.9	NP	NP	NP	13.1	A-1-a(0)	
	14	55	56.5									13.2		
	15	60	61.5	59.6	15.6	14.5	5.3	5.0	NP	NP	NP	10.0	A-1-a(0)	
	16	65	66.5									83.8		
	17	70	70.4	71.1	12.1	8.2	4.5	4.1	Insufficient Sample			12.8	A-1-a(0)	

Classification Test Data														
Boring No.	Sample ID	Top Depth (feet)	Bottom Depth (feet)	Gradation (%)					Atterberg			Moisture Content (%)	ODOT Classification (GI)	LOI (%)
				Gravel	Coarse Sand	Fine Sand	Silt	Clay	Liquid Limit	Plastic Limit	Plasticity Index			
R-8	1	0	1.5									35.7		
	2	1.5	3	0.0	0.2	5.8	62.9	31.1	36	21	15	33.0	A-6a(10)	
	3	3	4.5									29.1		
	4	4.5	6									28.3		
	5	6	7.5									28.0		
	6	7.5	9									29.5		
	7	9	10.5									28.3		
	8	12.5	14									30.7		
	9	15	16.5	0.0	0.4	23.9	51.9	23.8	30	21	9	39.6	A-4a(8)	
	10	17.5	19									35.6		
	11	20	21.5									32.6		
	12	25	26.5									33.9		
	13	30	31.5									8.7		
	14	35	36.5									9.8		
	15	40	41.5	38.3	34.9	20.5	3.9	2.4	NP	NP	NP	17.0	A-1-b(0)	
	16	45	56.5									9.7		
	17	50	51.5									13.9		
	18	55	56.5	27.4	48.7	14.8	6.3	2.8	NP	NP	NP	12.2	A-1-b(0)	
	19	60	61.5									16.7		
	20	65	66.5									13.8		

EXHIBIT B-2
TRIAXIAL TESTING RESULTS

	Results
C, psi	11.111
ϕ, deg	0
Tan(ϕ)	0





	Sample No.	1
Initial	Water Content, %	19.3
	Dry Density, pcf	105.4
	Saturation, %	86.9
	Void Ratio	0.5989
	Diameter, in.	2.800
	Height, in.	5.590
At Test	Water Content, %	19.3
	Dry Density, pcf	105.4
	Saturation, %	86.9
	Void Ratio	0.5989
	Diameter, in.	2.800
	Height, in.	5.590
	Strain rate, in./min.	0.055
	Back Pressure, psi	0.00
	Cell Pressure, psi	10.00
	Fail. Stress, psi	22.22
	Ult. Stress, psi	
	σ_1 Failure, psi	32.22
	σ_3 Failure, psi	10.00

Type of Test:
Unconsolidated Undrained

Sample Type: UU

Description: BROWN SANDY LEAN CLAY NOTED GRAVEL, MOIST - STIFF

Assumed Specific Gravity= 2.70

Remarks: Lab No. 7251

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-1 **Depth:** 12-14'

Sample Number: ST/4

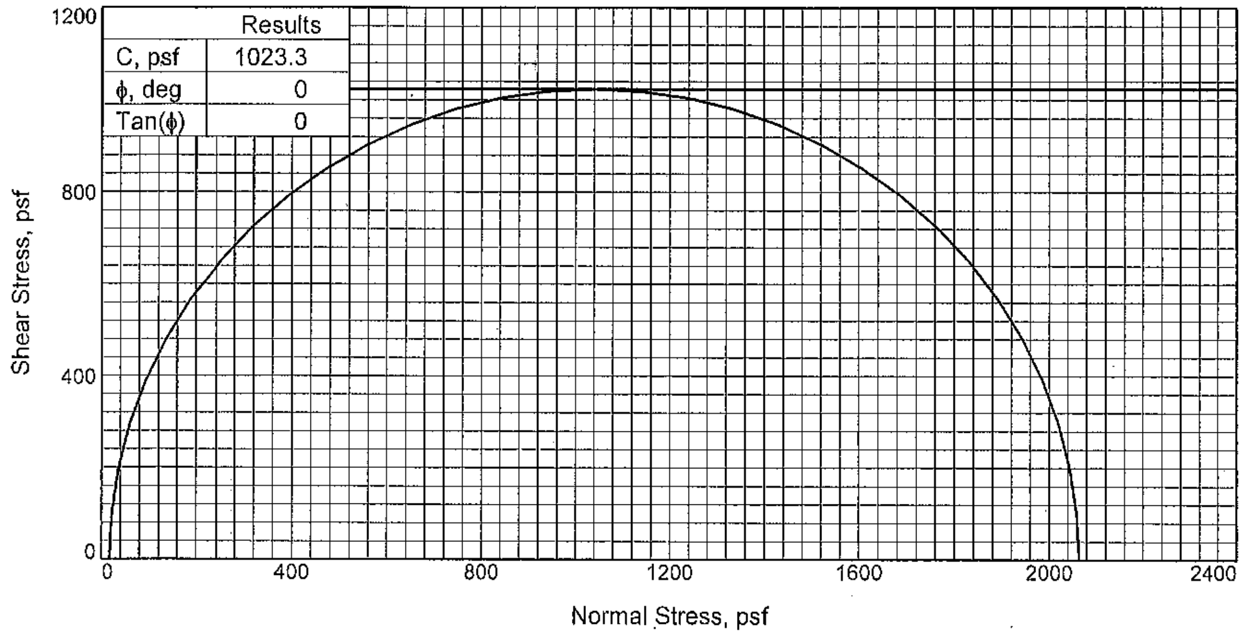
Proj. No.: N1105070 **Date Sampled:** 8-23-10

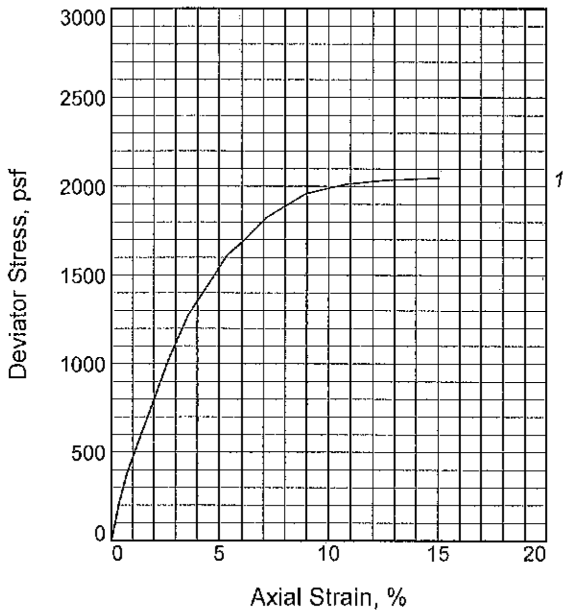
TRIAXIAL SHEAR TEST REPORT

H.C. Nutting

A Terracon Company

Tested By: SV **Checked By:** GS





Sample No.		1
Initial	Water Content,	36.9
	Dry Density, pcf	82.1
	Saturation,	94.7
	Void Ratio	1.0519
	Diameter, in.	2.860
	Height, in.	5.580
At Test	Water Content,	36.9
	Dry Density, pcf	82.1
	Saturation,	94.7
	Void Ratio	1.0519
	Diameter, in.	2.860
	Height, in.	5.580
Strain rate, in./min.		0.055
Back Pressure, psf		0.0
Cell Pressure, psf		17.0
Fail. Stress, psf		2046.7
Ult. Stress, psf		
σ_1 Failure, psf		2063.7
σ_3 Failure, psf		17.0

Type of Test:
Unconsolidated Undrained
Sample Type: ST
Description: BROWN CLAY, MOIST - STIFF

LL= 46 **PL=** 25 **PI=** 21
Assumed Specific Gravity= 2.70
Remarks: Lab No. 6500

Figure ________

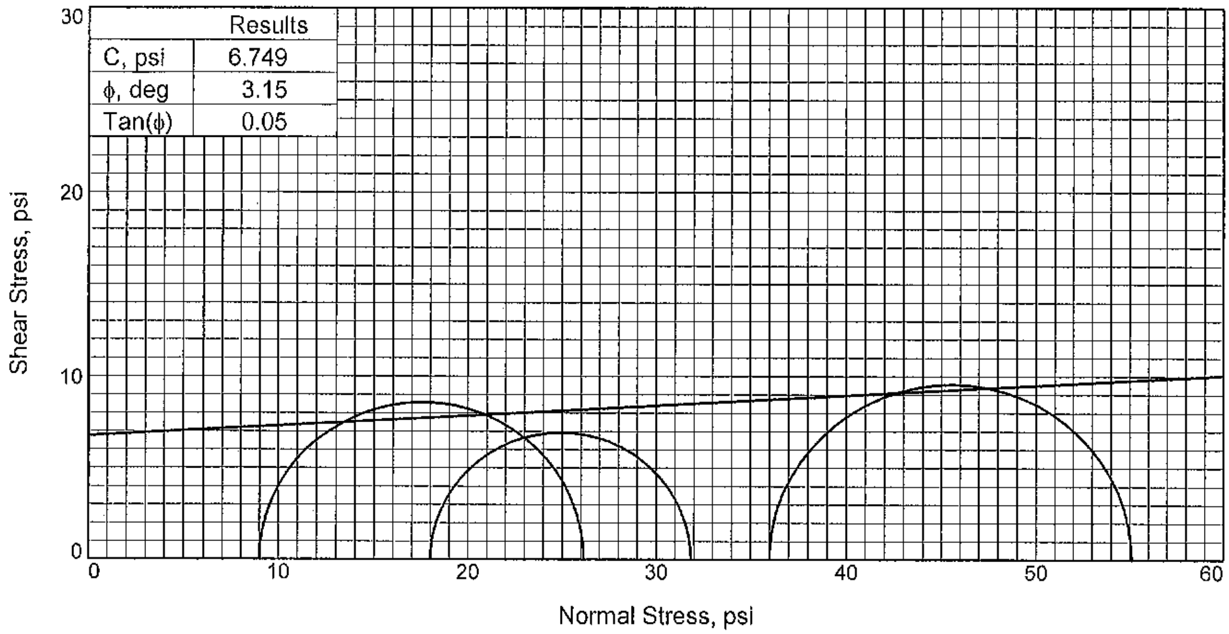
Client: PARSONS BRINCKERHOFF

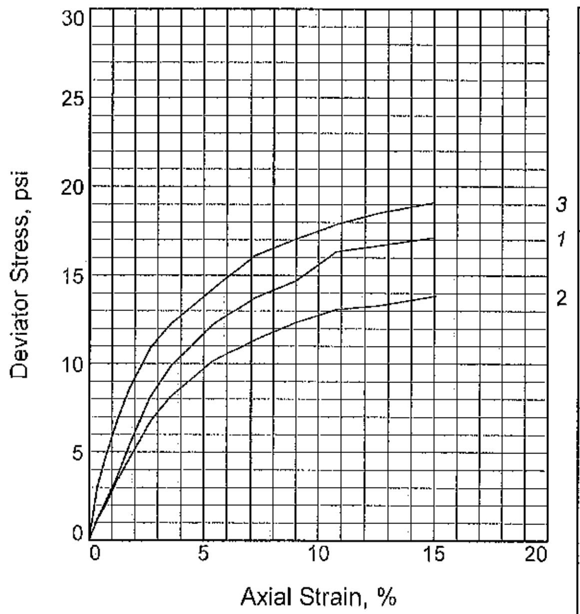
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-4 **Depth:** 30-32'
Sample Number: ST/11
Proj. No.: N1105070 **Date:** 9-23-10

TRIAXIAL SHEAR TEST REPORT
H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2	3
Initial	Water Content,	24.9	23.6	21.7
	Dry Density, pcf	99.8	102.7	106.0
	Saturation,	97.8	98.5	98.4
	Void Ratio	0.6885	0.6480	0.5962
	Diameter, in.	2.840	2.850	2.840
	Height, in.	5.580	5.570	5.590
At Test	Water Content,	24.9	23.6	21.7
	Dry Density, pcf	99.8	102.7	106.0
	Saturation,	97.8	98.5	98.4
	Void Ratio	0.6885	0.6480	0.5962
	Diameter, in.	2.840	2.850	2.840
	Height, in.	5.580	5.570	5.590
Strain rate, in./min.		0.055	0.055	0.055
Back Pressure, psi		0.00	0.00	0.00
Cell Pressure, psi		9.00	18.00	36.00
Fail. Stress, psi		17.16	13.85	19.10
Ult. Stress, psi				
σ_1 Failure, psi		26.16	31.85	55.10
σ_3 Failure, psi		9.00	18.00	36.00

Type of Test:
Unconsolidated Undrained

Sample Type: ST

Description: BROWN SILT AND CLAY, MOIST - MED STIFF

LL= 29 **PL=** 17 **PI=** 12

Assumed Specific Gravity= 2.70

Remarks: Lab No. 5689

Figure ______

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-5 **Depth:** 23-25'

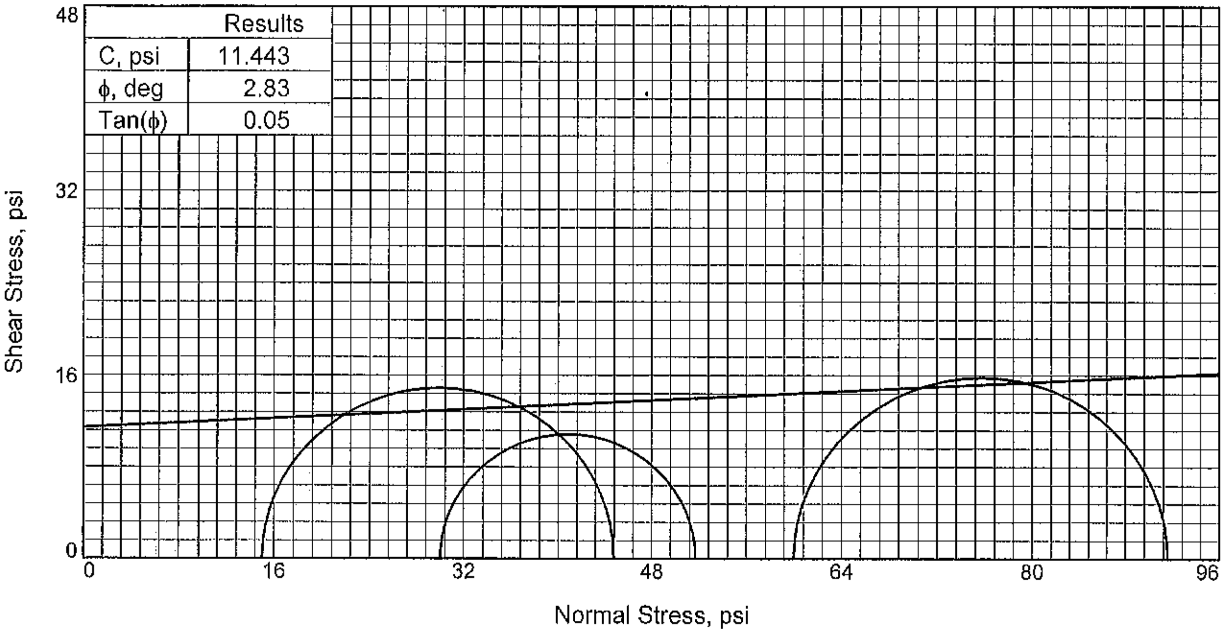
Sample Number: 8

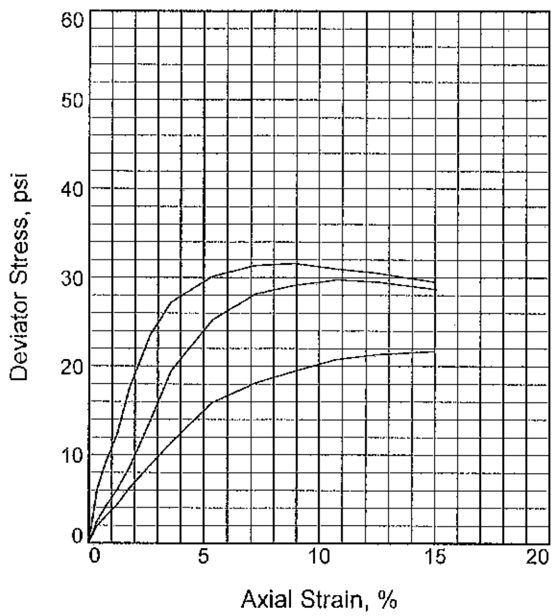
Proj. No.: N1105070 **Date:** 9-21-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2	3
Initial	Water Content,	30.8	27.6	27.4
	Dry Density, pcf	90.3	95.6	96.0
	Saturation,	95.9	97.7	97.9
	Void Ratio	0.8670	0.7625	0.7562
	Diameter, in.	2.840	2.840	2.840
	Height, in.	5.580	5.600	5.600
At Test	Water Content,	30.8	27.5	27.4
	Dry Density, pcf	90.3	95.6	96.0
	Saturation,	95.9	97.5	97.9
	Void Ratio	0.8670	0.7625	0.7562
	Diameter, in.	2.840	2.840	2.840
	Height, in.	5.580	5.600	5.600
Strain rate, in./min.		0.055	0.056	0.056
Back Pressure, psi		0.00	0.00	0.00
Cell Pressure, psi		15.00	30.00	60.00
Fail. Stress, psi		29.76	21.70	31.58
Ult. Stress, psi				
σ_1 Failure, psi		44.76	51.70	91.58
σ_3 Failure, psi		15.00	30.00	60.00

Type of Test:
Unconsolidated Undrained

Sample Type: ST

Description: GRAY SANDY SILT, MOIST - STIFF

LL= 29 **PL=** 19 **PI=** 10

Assumed Specific Gravity= 2.70

Remarks: Lab No. 5693

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-5 **Depth:** 38-40'

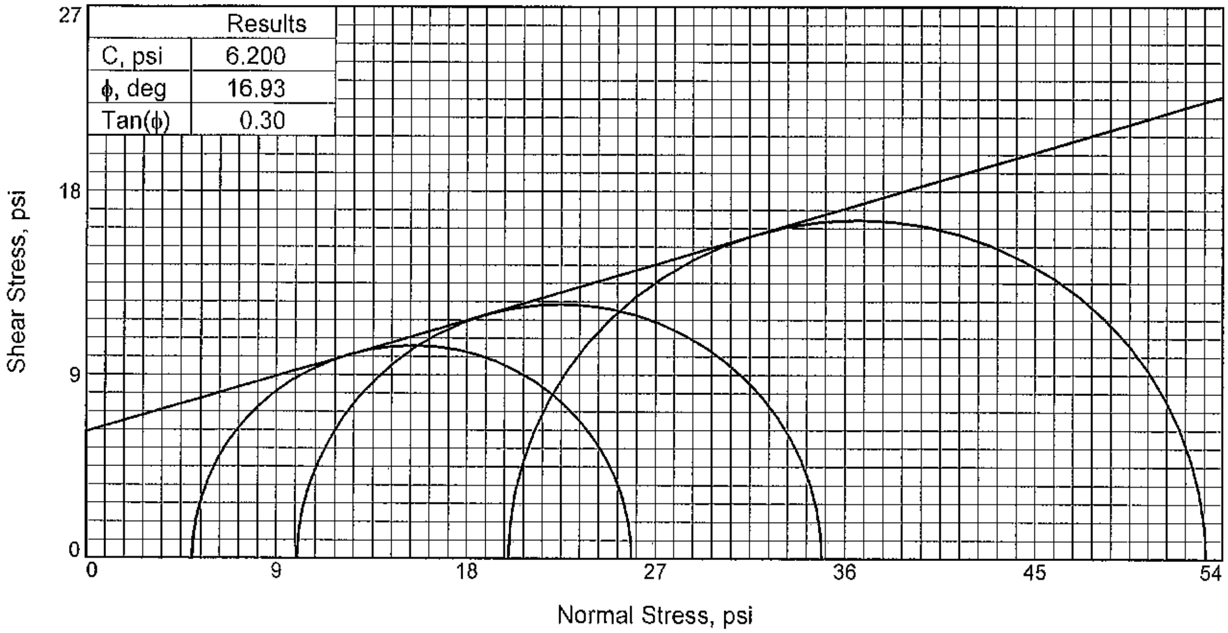
Sample Number: 12

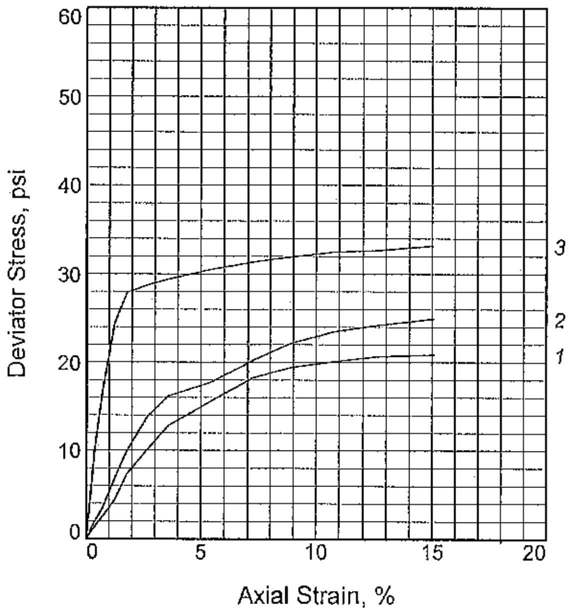
Proj. No.: N1105070 **Date:** 9-21-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





	Sample No.	1	2	3
Initial	Water Content,	24.7	24.2	22.8
	Dry Density, pcf	100.9	101.1	103.4
	Saturation,	99.5	97.9	97.7
	Void Ratio	0.6699	0.6666	0.6307
	Diameter, in.	2.840	2.840	2.840
	Height, in.	5.570	5.580	5.590
At Test	Water Content,	24.7	24.2	22.8
	Dry Density, pcf	100.9	101.1	103.4
	Saturation,	99.5	97.9	97.7
	Void Ratio	0.6699	0.6666	0.6307
	Diameter, in.	2.840	2.840	2.840
	Height, in.	5.570	5.580	5.590
	Strain rate, in./min.	0.055	0.055	0.055
	Back Pressure, psi	0.00	0.00	0.00
	Cell Pressure, psi	5.00	10.00	20.00
	Fail. Stress, psi	20.87	24.92	33.19
	Ult. Stress, psi			
	σ_1 Failure, psi	25.87	34.92	53.19
	σ_3 Failure, psi	5.00	10.00	20.00

Type of Test:
Unconsolidated Undrained

Sample Type: ST

Description: BROWN SILT AND CLAY, MOIST - STIFF

LL= 33 **PL=** 19 **PI=** 14

Assumed Specific Gravity= 2.70

Remarks: Lab No. 5728

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-6 **Depth:** 14.5-16.0'

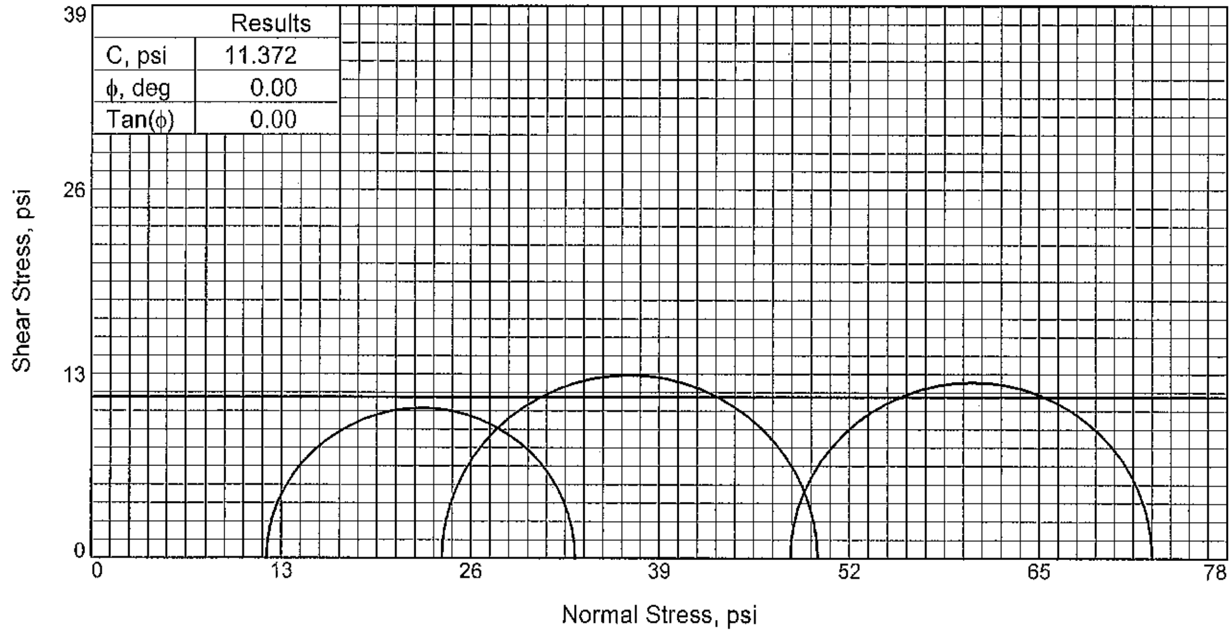
Sample Number: 3/ST

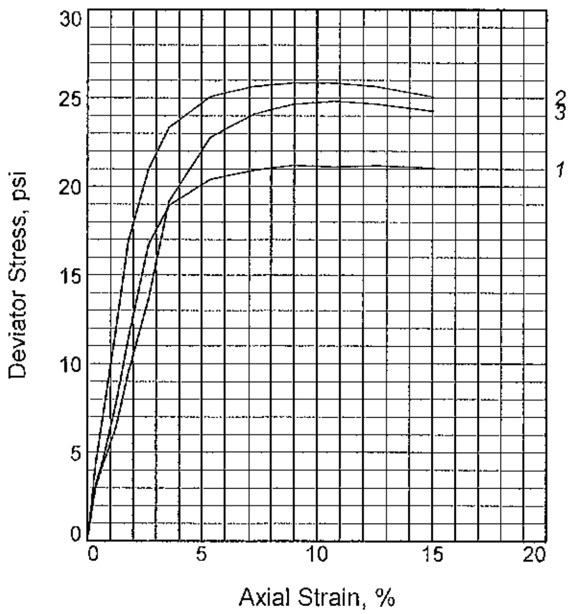
Proj. No.: N1105070 **Date:** 9-21-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2	3
Initial	Water Content,	28.0	26.4	26.2
	Dry Density, pcf	95.9	98.0	98.4
	Saturation,	99.8	98.9	99.2
	Void Ratio	0.7568	0.7207	0.7126
	Diameter, in.	2.850	2.850	2.860
	Height, in.	5.580	5.590	5.590
At Test	Water Content,	28.0	26.4	26.2
	Dry Density, pcf	95.9	98.0	98.4
	Saturation,	99.8	98.9	99.2
	Void Ratio	0.7568	0.7207	0.7126
	Diameter, in.	2.850	2.850	2.860
	Height, in.	5.580	5.590	5.590
Strain rate, in./min.		0.055	0.055	0.055
Back Pressure, psi		0.00	0.00	0.00
Cell Pressure, psi		12.00	24.00	48.00
Fail. Stress, psi		21.18	25.84	24.82
Ult. Stress, psi				
σ_1 Failure, psi		33.18	49.84	72.82
σ_3 Failure, psi		12.00	24.00	48.00

Type of Test:
Unconsolidated Undrained
Sample Type: ST
Description: GRAY SILT AND CLAY, MOIST - STIFF
LL= 30 **PL=** 19 **PI=** 11
Assumed Specific Gravity= 2.70
Remarks: Lab No. 5729

Figure ________

Client: PARSONS BRINCKERHOFF

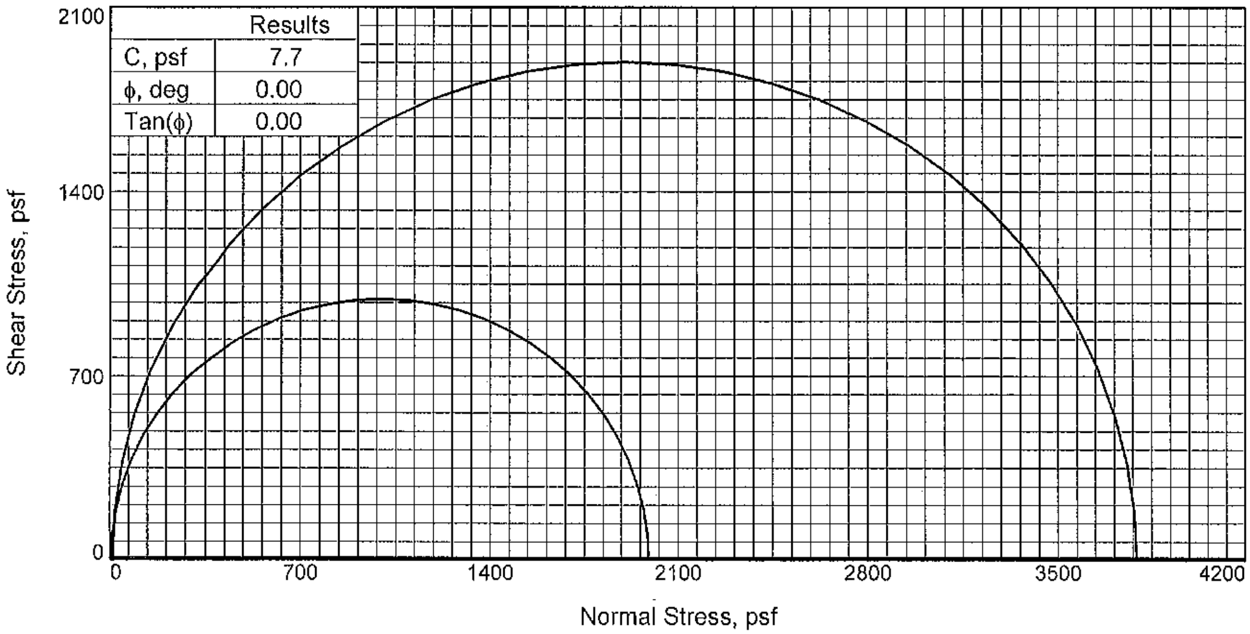
Project: BRENT SPENCE BRIDGE REPLACEMENT

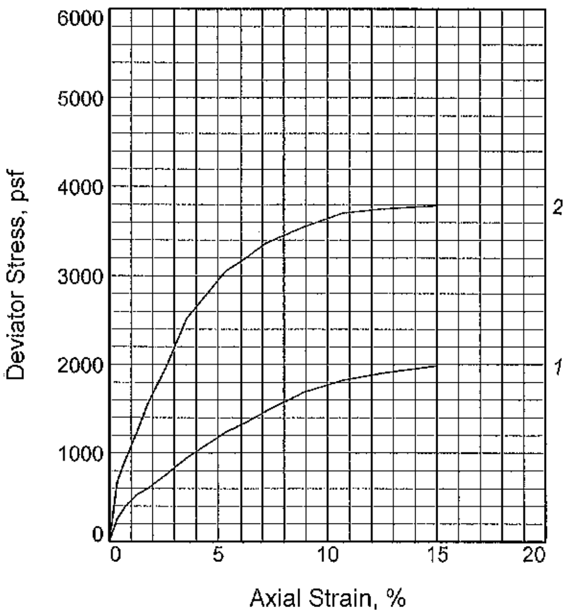
Source of Sample: L-6 **Depth:** 30-32'
Sample Number: 6/ST
Proj. No.: N1105070 **Date:** 9-21-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2
Initial	Water Content,	20.8	20.8
	Dry Density, pcf	104.3	107.7
	Saturation,	91.3	99.2
	Void Ratio	0.6157	0.5655
	Diameter, in.	2.840	2.820
	Height, in.	5.590	5.600
At Test	Water Content,	20.8	20.8
	Dry Density, pcf	104.3	107.7
	Saturation,	91.3	99.2
	Void Ratio	0.6157	0.5655
	Diameter, in.	2.840	2.820
	Height, in.	5.590	5.600
Strain rate, in./min.		0.055	0.056
Back Pressure, psf		0.0	0.0
Cell Pressure, psf		5.0	10.0
Fail. Stress, psf		1986.6	3784.3
Ult. Stress, psf			
σ_1 Failure, psf		1991.6	3794.3
σ_3 Failure, psf		5.0	10.0

Type of Test:
Unconsolidated Undrained

Sample Type: ST

Description: BROWN GRAY SILT, CLAY AND GRAVEL, MOIST - STIFF

LL= 29 **PL=** 17 **PI=** 12

Assumed Specific Gravity= 2.70

Remarks: Lab No. 5733

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-7 **Depth:** 13-15'

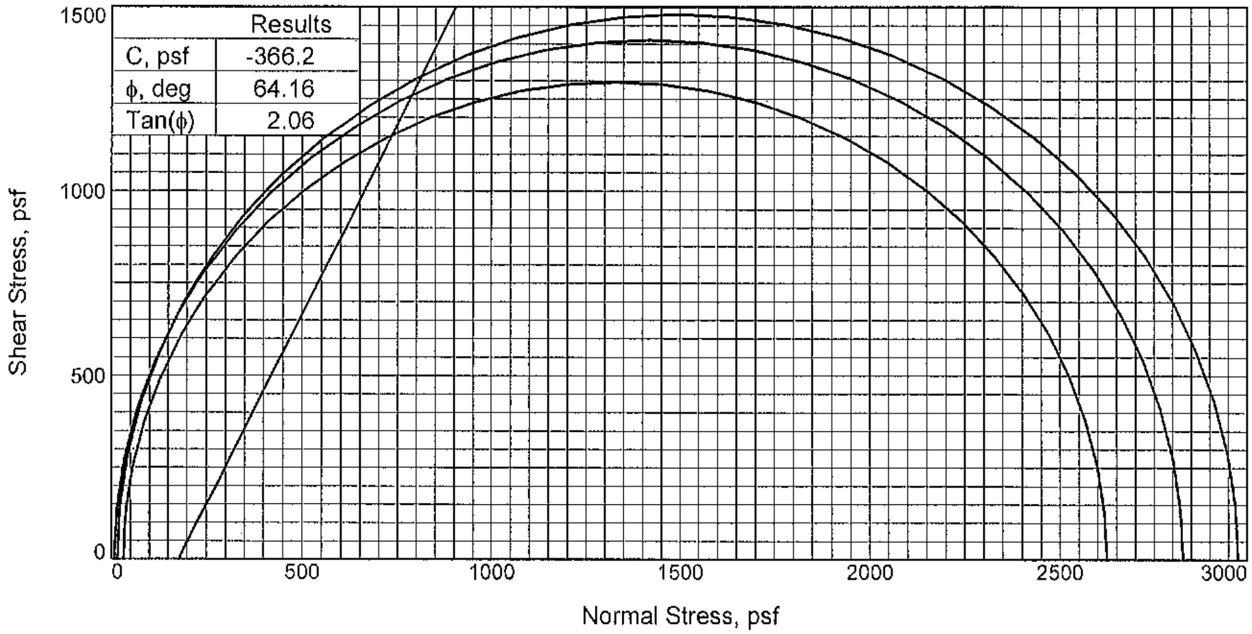
Sample Number: 4

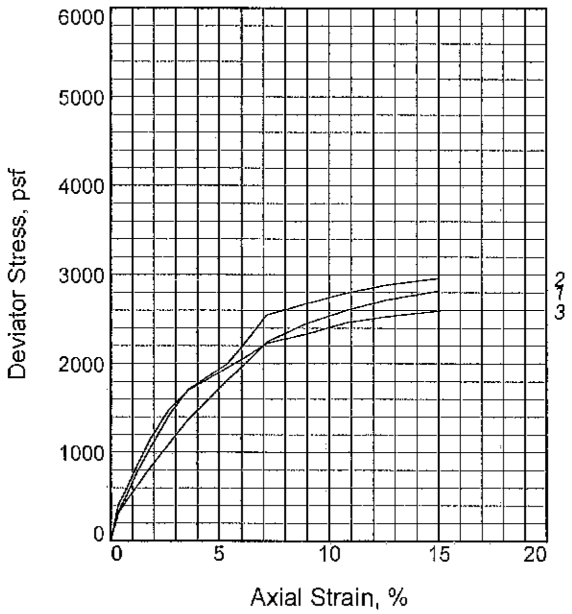
Proj. No.: N1105070 **Date:** 9-21-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2	3
Initial	Water Content,	22.7	23.0	23.2
	Dry Density, pcf	103.5	103.3	103.1
	Saturation,	97.4	98.2	98.8
	Void Ratio	0.6289	0.6322	0.6354
	Diameter, in.	2.840	2.840	2.850
	Height, in.	5.580	5.580	5.580
At Test	Water Content,	22.7	23.0	23.2
	Dry Density, pcf	103.5	103.3	103.1
	Saturation,	97.4	98.2	98.8
	Void Ratio	0.6289	0.6322	0.6354
	Diameter, in.	2.840	2.840	2.850
	Height, in.	5.580	5.580	5.580
Strain rate, in./min.		0.055	0.055	0.055
Back Pressure, psf		0.0	0.0	0.0
Cell Pressure, psf		8.0	16.0	32.0
Fail. Stress, psf		2819.9	2959.8	2591.9
Ult. Stress, psf				
σ_1 Failure, psf		2827.9	2975.8	2623.9
σ_3 Failure, psf		8.0	16.0	32.0

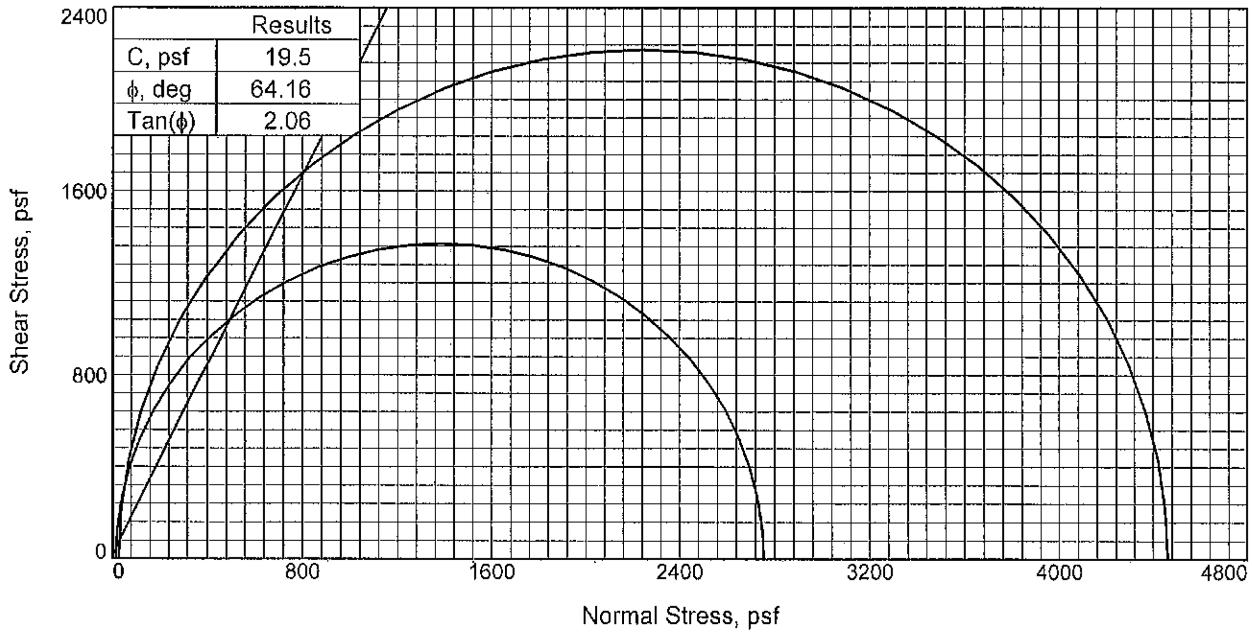
Type of Test:
Unconsolidated Undrained
Sample Type: ST
Description: BROWN SILT AND CLAY, MOIST - STIFF
LL= 31 **PL=** 17 **PI=** 14
Assumed Specific Gravity= 2.70
Remarks: Lab No. 5736

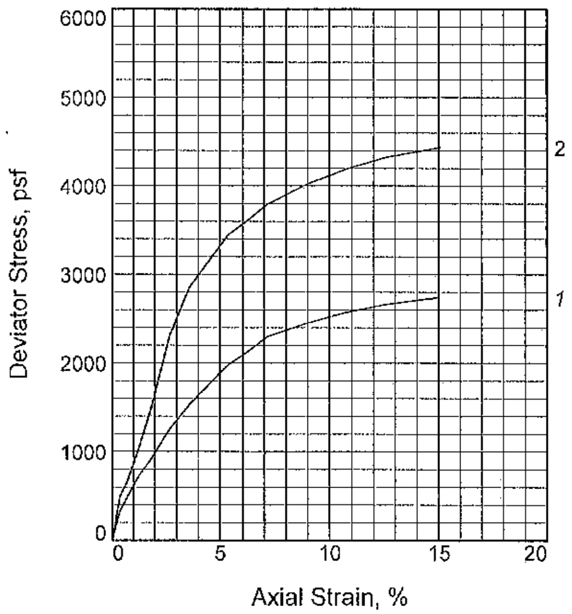
Figure ________

Client: PARSONS BRINCKERHOFF
Project: BRENT SPENCE BRIDGE REPLACEMENT
Source of Sample: L-7 **Depth:** 20-22'
Sample Number: 7
Proj. No.: N1105070 **Date:** 9-22-10

TRIAXIAL SHEAR TEST REPORT
H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1	2
Initial	Water Content,	25.3	24.9
	Dry Density, pcf	99.1	100.8
	Saturation,	97.5	100.0
	Void Ratio	0.7009	0.6720
	Diameter, in.	2.870	2.830
	Height, in.	5.610	5.570
At Test	Water Content,	25.3	24.9
	Dry Density, pcf	99.1	100.8
	Saturation,	97.5	100.0
	Void Ratio	0.7009	0.6720
	Diameter, in.	2.870	2.830
	Height, in.	5.610	5.570
Strain rate, in./min.		0.056	0.055
Back Pressure, psf		0.0	0.0
Cell Pressure, psf		14.0	28.0
Fail. Stress, psf		2741.0	4434.4
Ult. Stress, psf			
σ_1 Failure, psf		2755.0	4462.4
σ_3 Failure, psf		14.0	28.0

Type of Test:
Unconsolidated Undrained
Sample Type: ST
Description: BROWN GRAY SILT AND CLAY AND GRAVEL, MOIST - MED STIFF
LL= 31 **PL=** 17 **PI=** 14
Assumed Specific Gravity= 2.70
Remarks: Lab No. 5740
Only 2pt test, Insufficient for 3rd pt

Figure ______

Client: PARSONS BRINCKERHOFF

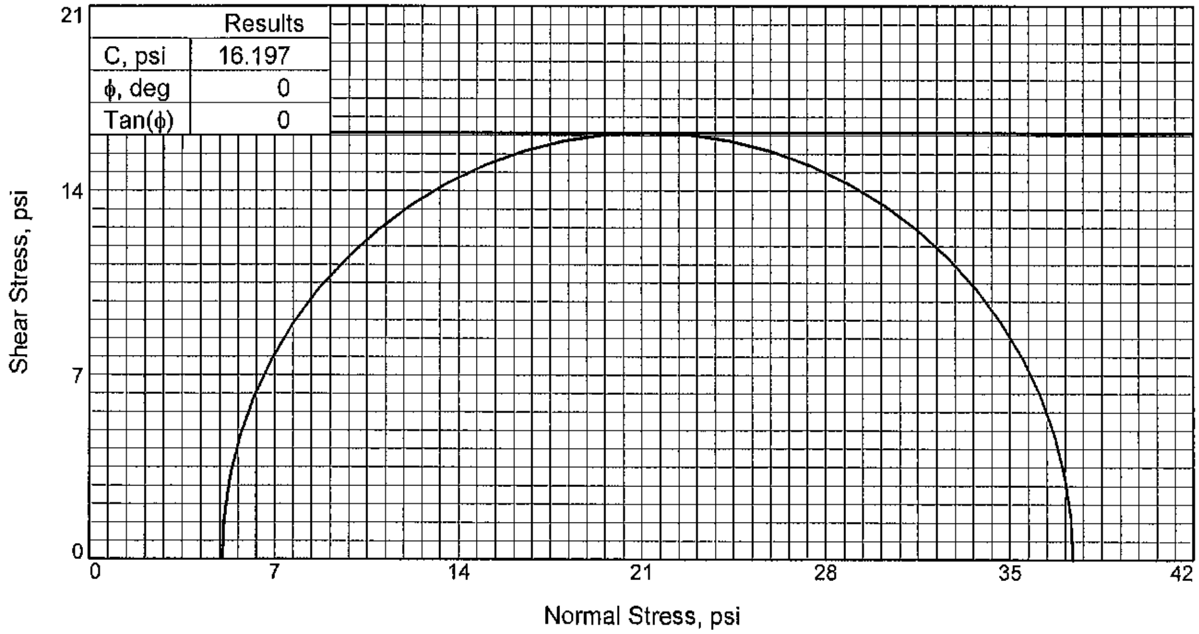
Project: BRENT SPENCE BRIDGE REPLACEMENT

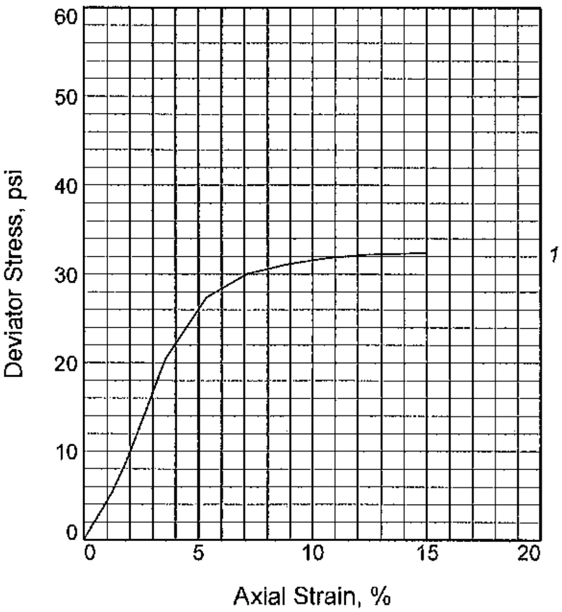
Source of Sample: L-7 **Depth:** 33-35'
Sample Number: 11
Proj. No.: N1105070 **Date:** 9-22-10

TRIAXIAL SHEAR TEST REPORT

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS





Sample No.		1
Initial	Water Content, %	35.2
	Dry Density, pcf	87.6
	Saturation, %	102.6
	Void Ratio	0.9247
	Diameter, in.	2.790
	Height, in.	5.600
At Test	Water Content, %	35.2
	Dry Density, pcf	87.6
	Saturation, %	102.6
	Void Ratio	0.9247
	Diameter, in.	2.790
	Height, in.	5.600
Strain rate, in./min.		0.056
Back Pressure, psi		0.00
Cell Pressure, psi		5.00
Fail. Stress, psi		32.39
Ult. Stress, psi		
σ_1 Failure, psi		37.39
σ_3 Failure, psi		5.00

Type of Test:
Unconsolidated Undrained
Sample Type: ST
Description: GRAY BROWN LEAN CLAY W/ SAND LENSES, MOIST - STIFF

Assumed Specific Gravity= 2.70
Remarks: Lab No. 6610

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

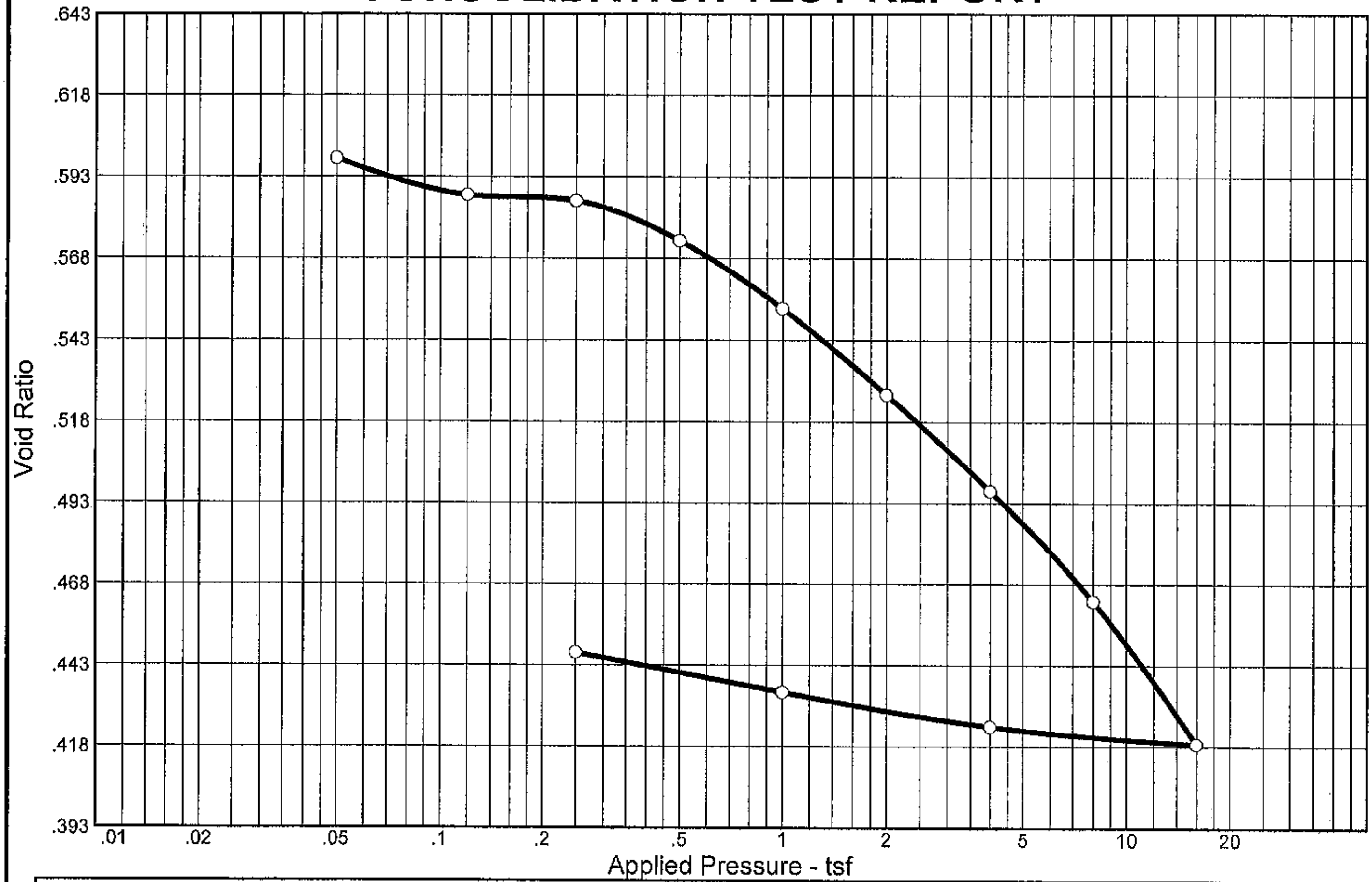
Source of Sample: R-6 **Depth:** 5-7'
Sample Number: 3/ST
Proj. No.: N1105070 **Date Sampled:** 9-17-10

TRIAXIAL SHEAR TEST REPORT
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

EXHIBIT B-3
CONSOLIDATION TESTING RESULTS

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.2/day)	C_α	No.	Load (tsf)	C_v (ft.2/day)	C_α	No.	Load (tsf)	C_v (ft.2/day)	C_α
2	0.12	1.30									
3	0.25	0.42									
4	0.50	0.37									
5	1.00	0.66									
6	2.00	0.56									
7	4.00	0.52									
8	8.00	0.58									
9	16.00	1.01									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
91.5 %	20.3 %	105.5			2.702		1.34	0.15	0.02			0.599

MATERIAL DESCRIPTION	USCS	AASHTO
BROWN SANDY LEAN CLAY NOTED GRAVEL, MOIST - STIFF		

Project No. N1105070 **Client:** PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1 **Sample No.:** ST/4 **Elev./Depth:** 12-14'

Remarks:
Lab No. 7251

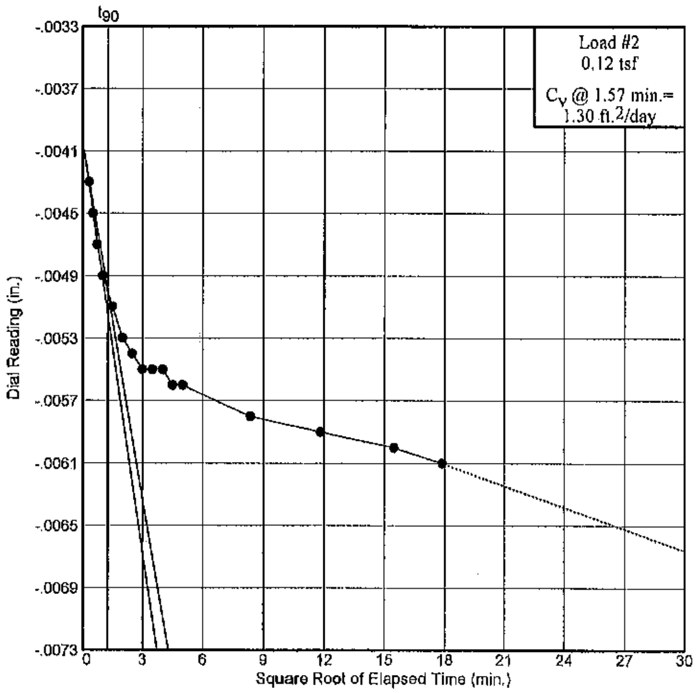
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

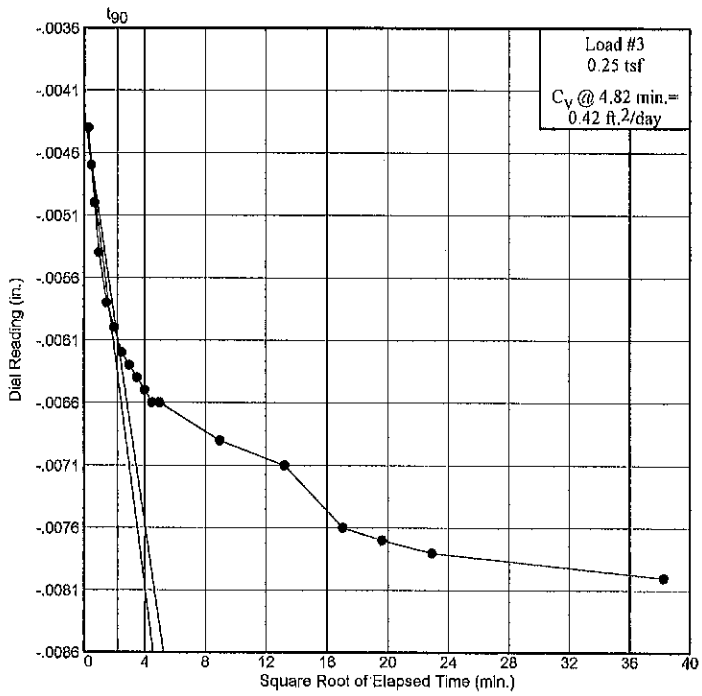
Figure

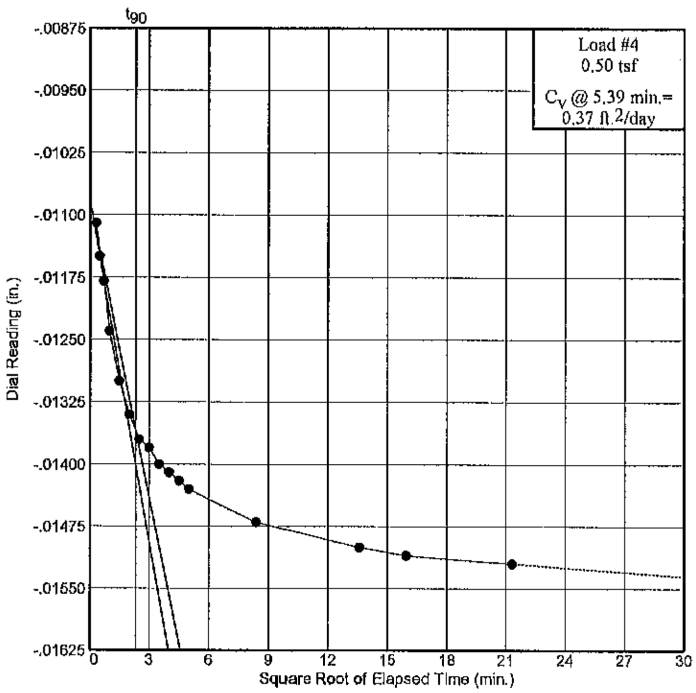
Dial Reading vs. Time

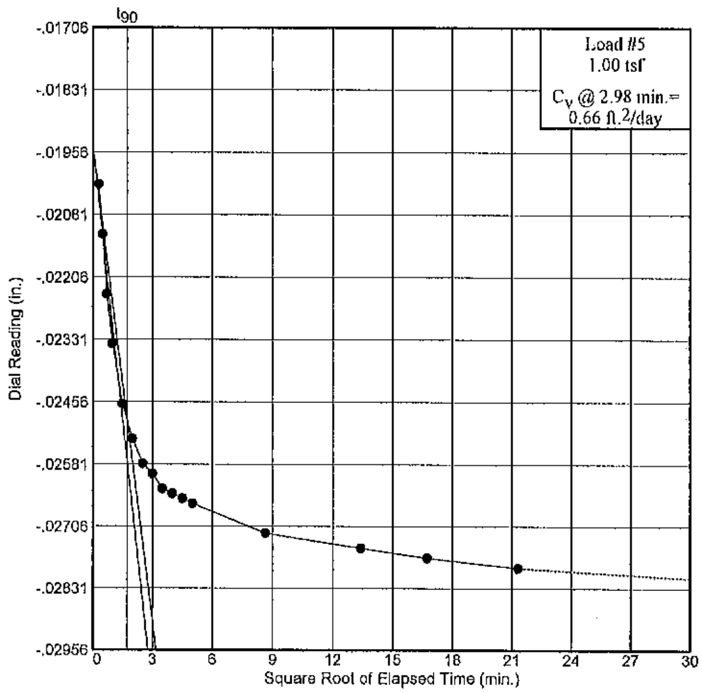
Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1 Sample No.: ST/4 Elev./Depth: 12-14'









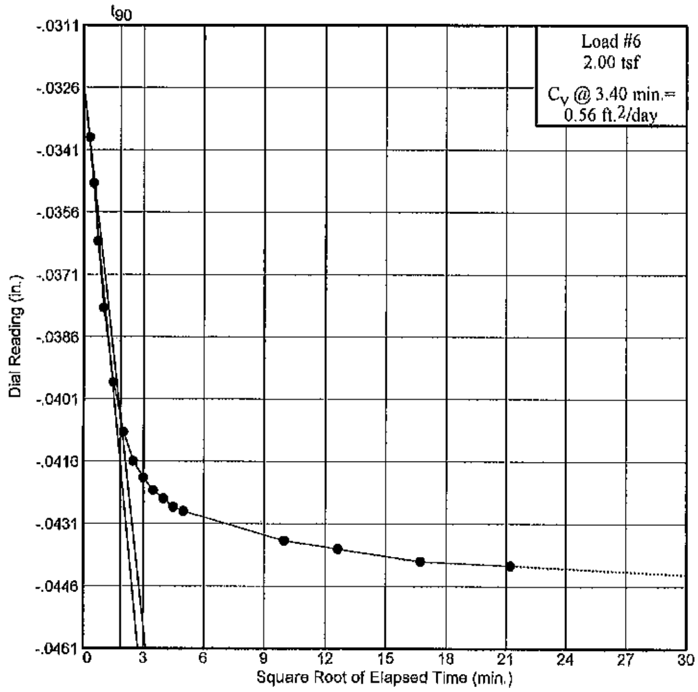
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

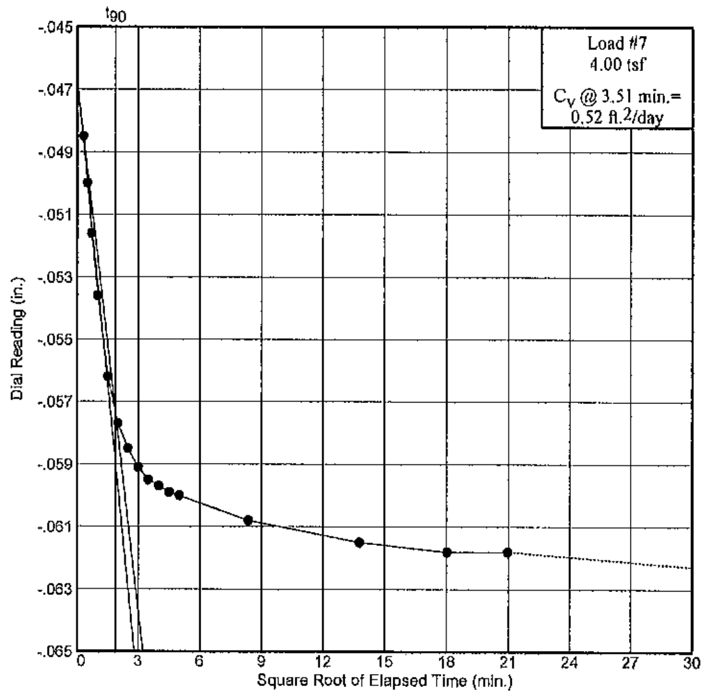
Figure

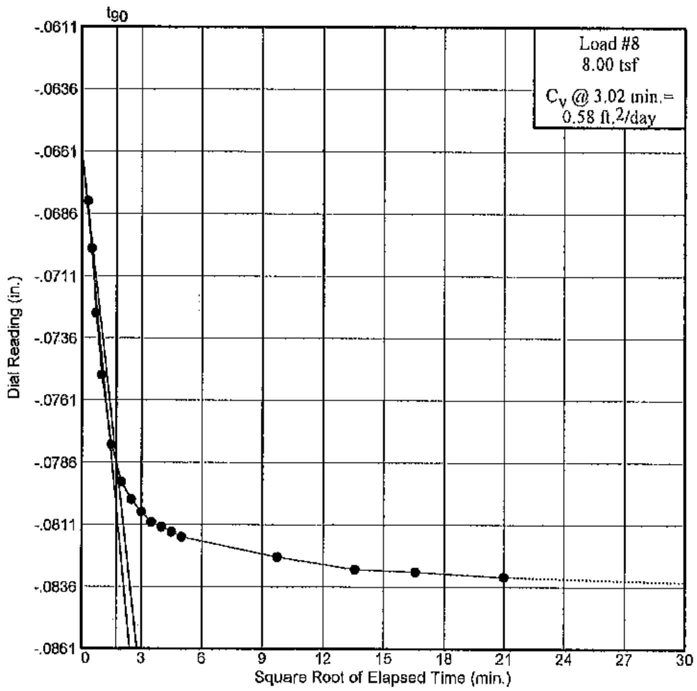
Dial Reading vs. Time

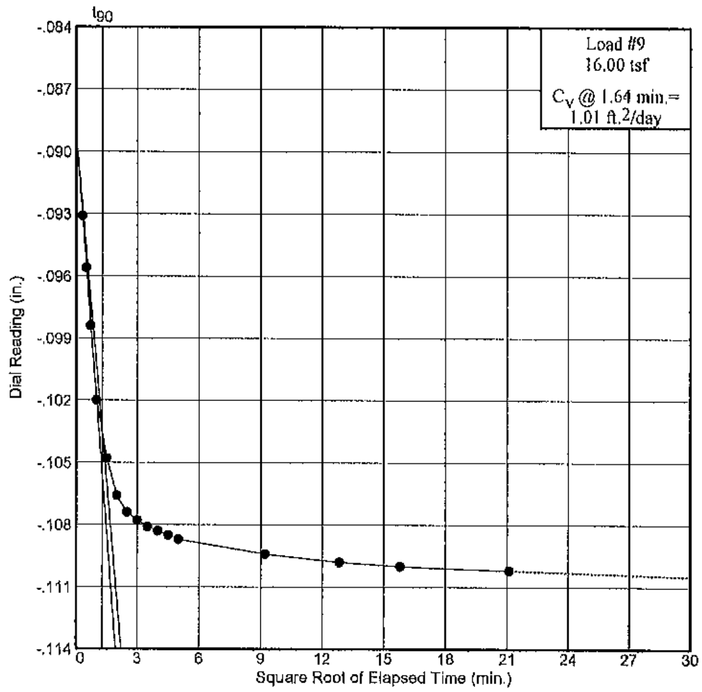
Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1 Sample No.: ST/4 Elev./Depth: 12-14'









H.C. Nutting
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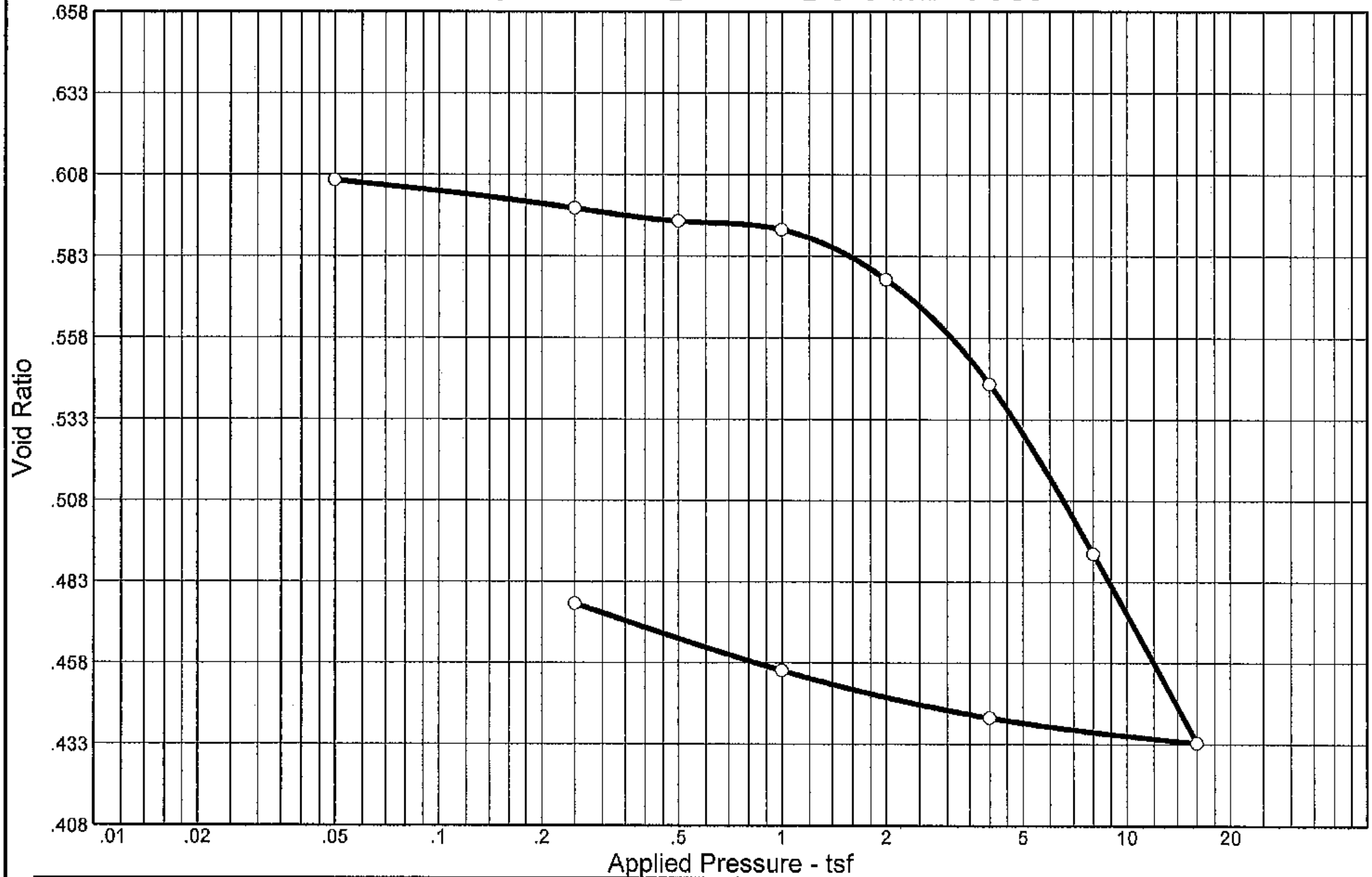
Cincinnati, Ohio

Figure

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation

No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.25	0.42									
3	0.50	0.20									
4	1.00	0.38									
5	2.00	0.37									
6	4.00	0.76									
7	8.00	0.75									
8	16.00	0.32									

Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
95.4 %	21.5 %	104.6			2.690		2.61	0.19	0.02			0.606

MATERIAL DESCRIPTION	USCS	AASHTO
BROWN LEAN CLAY, MOIST - STIFF		

Project No. N1105070 **Client:** PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1A **Sample No.:** ST/4 **Elev./Depth:** 13-15'

Remarks:
Lab No. 7279

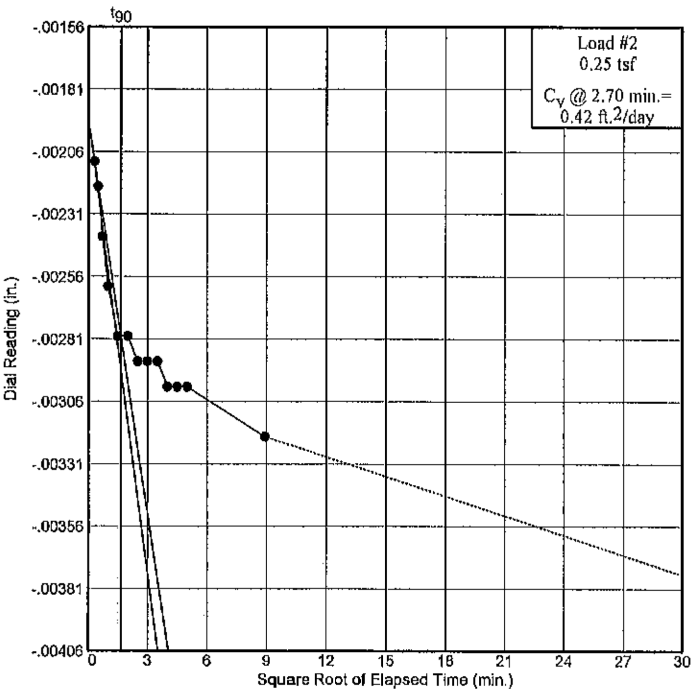
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

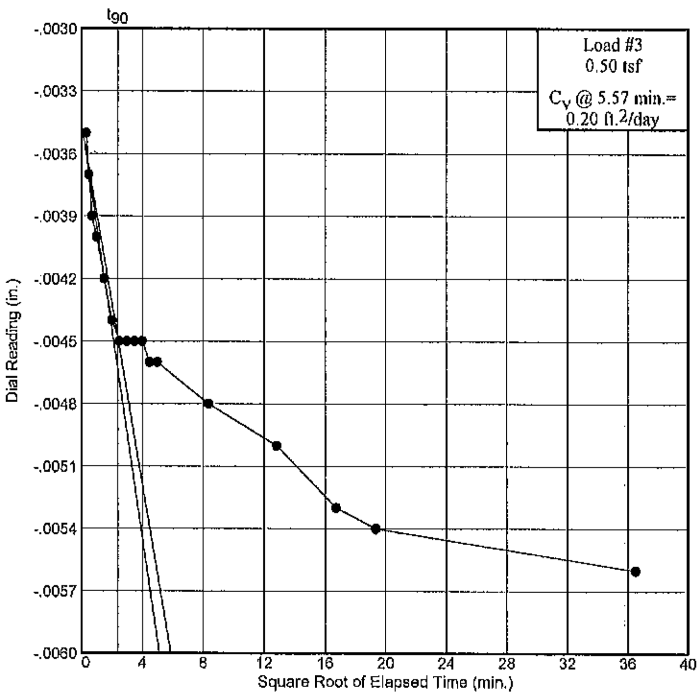
Figure

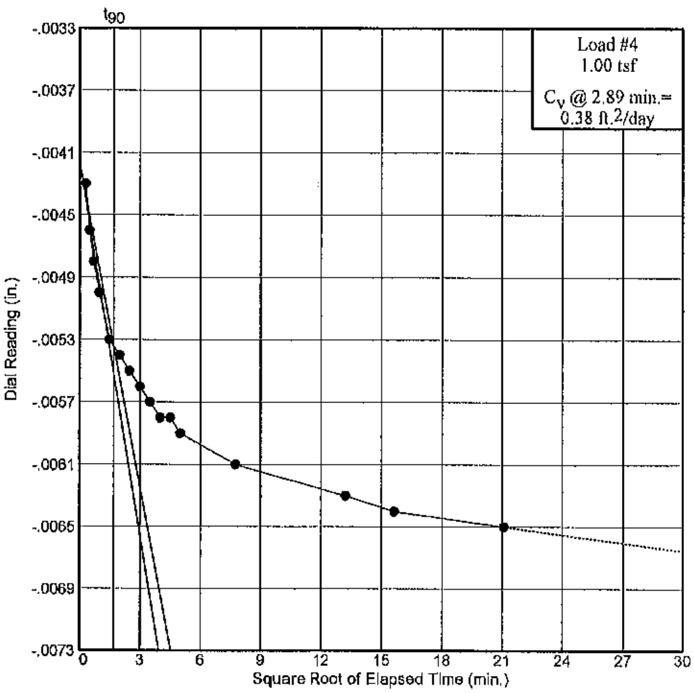
Dial Reading vs. Time

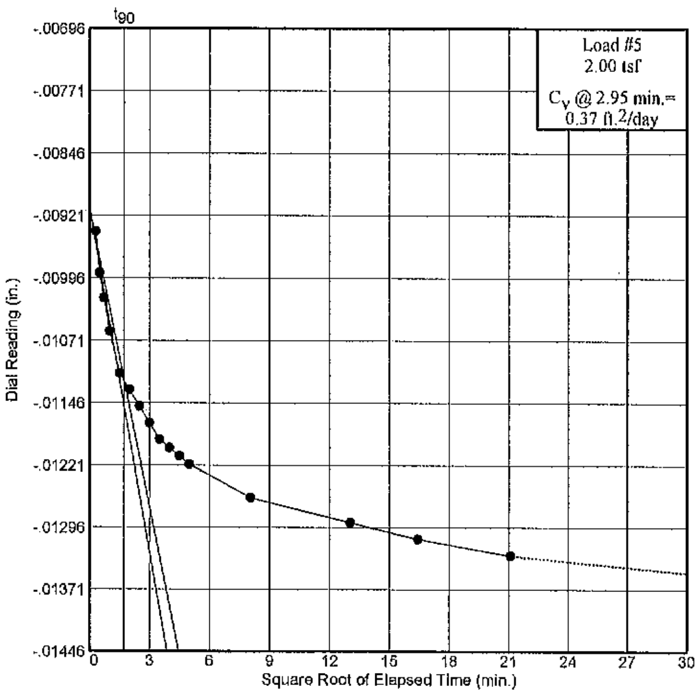
Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1A | Sample No.: ST/4 | Elev./Depth: 13-15'









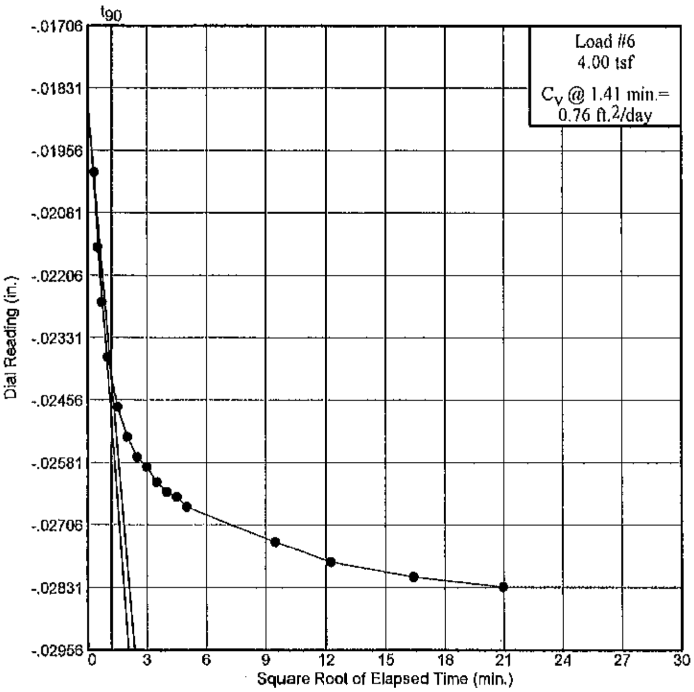
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

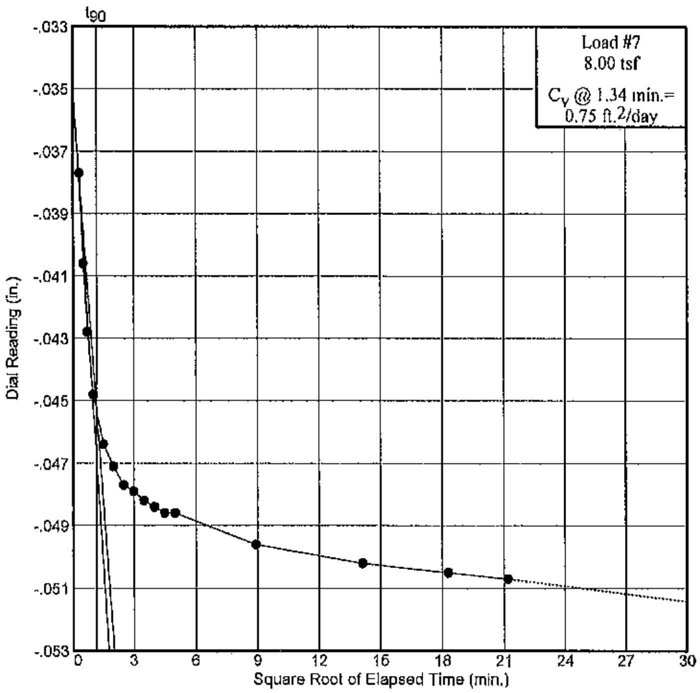
Figure

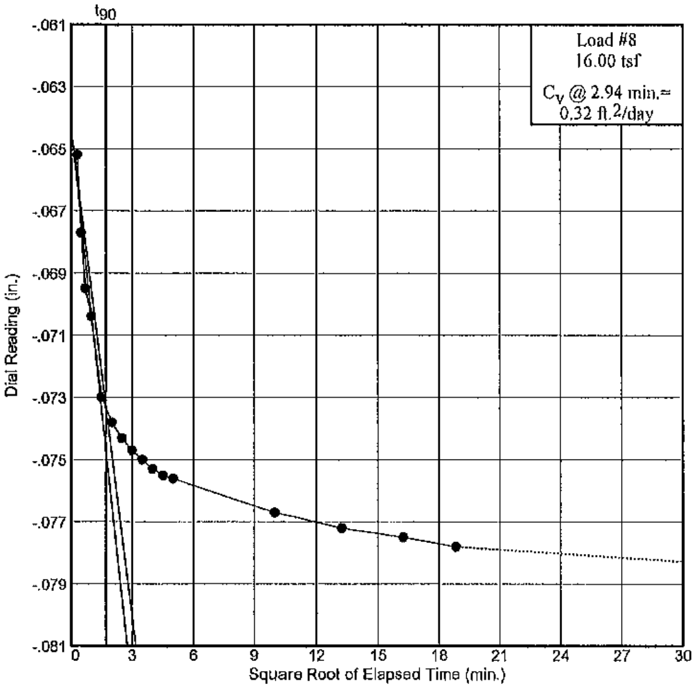
Dial Reading vs. Time

Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: L-1A　　Sample No.: ST/4　　Elev./Depth: 13-15'



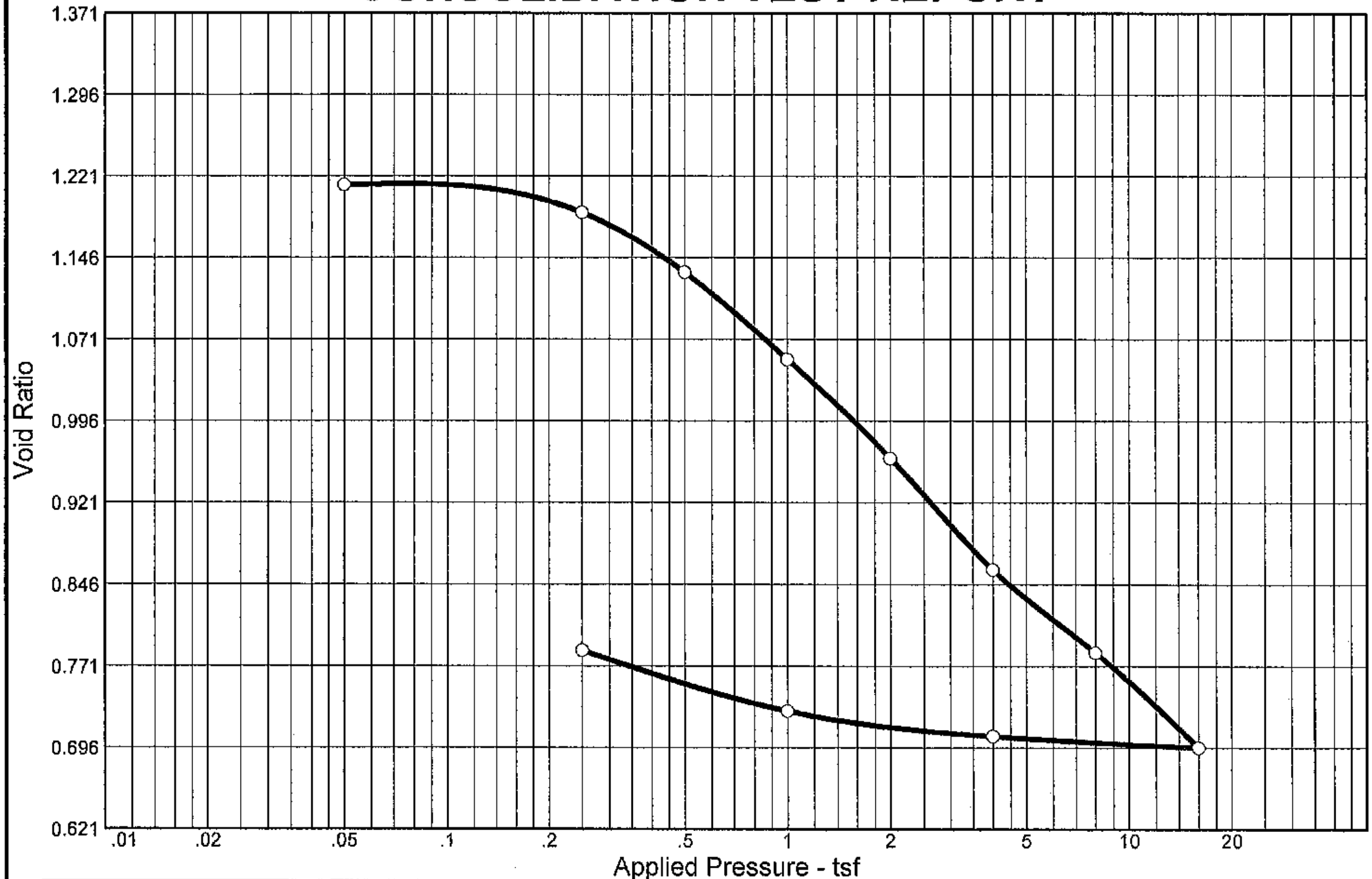




H.C. Nutting
A Terracon Company
Cincinnati, Ohio

Figure

CONSOLIDATION TEST REPORT



Coefficients of Consolidation and Secondary Consolidation											
No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α	No.	Load (tsf)	C_v (ft.^{2}/day)	C_α
2	0.25	0.95									
3	0.50	0.15									
4	1.00	0.14									
5	2.00	0.30									
6	4.00	0.16									
7	8.00	2.28									
8	16.00	0.49									

Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (tsf)	P_c (tsf)	C_c	C_r	Swell Press. (tsf)	Swell %	e_o
Sat.	Moist.											
75.1 %	34.3 %	75.2	34	11	2.671		5.05	0.29	0.05			1.218

MATERIAL DESCRIPTION	USCS	AASHTO
BROWN SILT AND CLAY, MOIST - MED STIFF		ODOT=A-6a(8)

Project No. N1105070 **Client:** PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: R-6 **Sample No.:** 7/ST **Elev./Depth:** 15-17'

Remarks:

Lab No. 6614

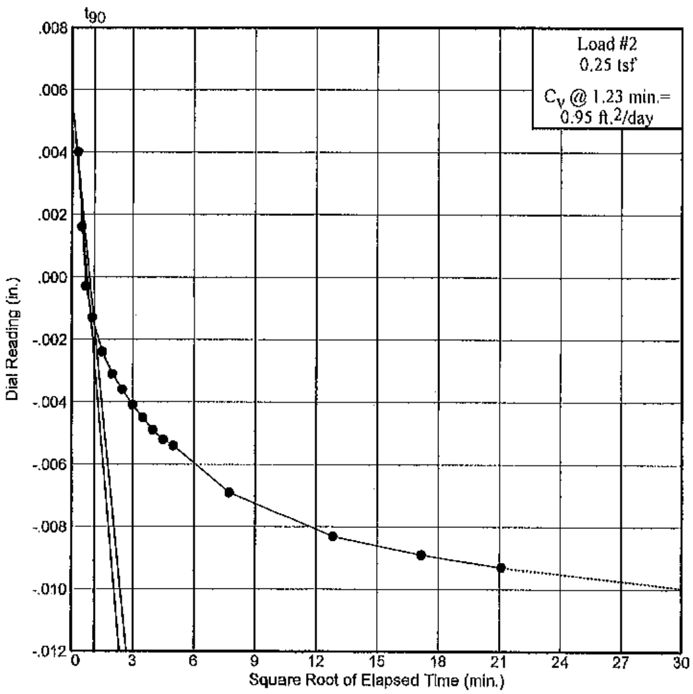
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

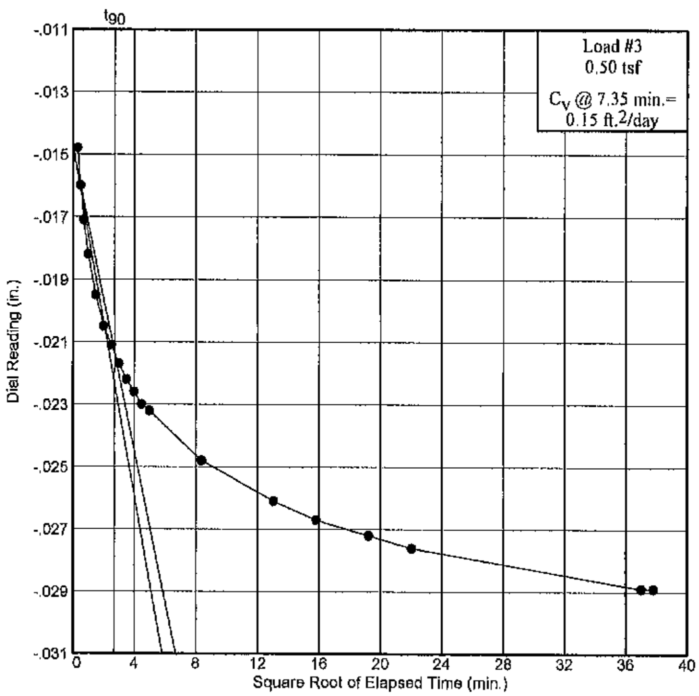
Figure

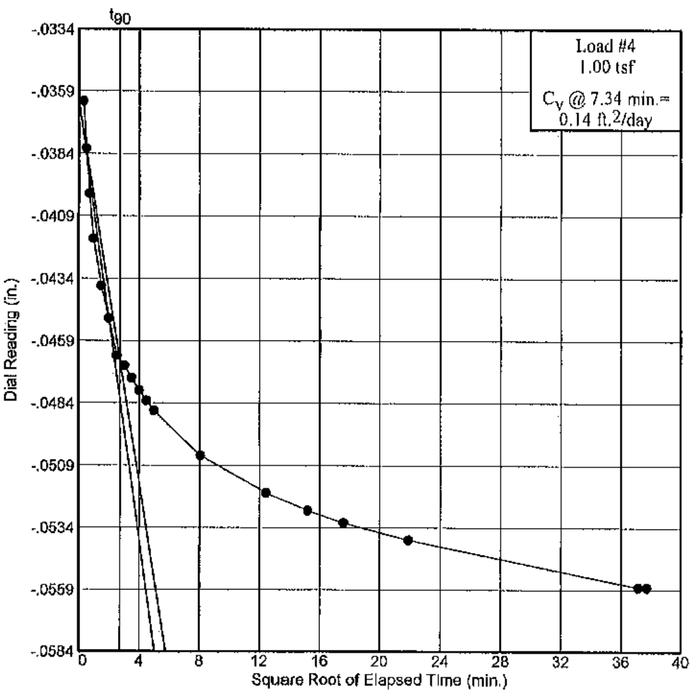
Dial Reading vs. Time

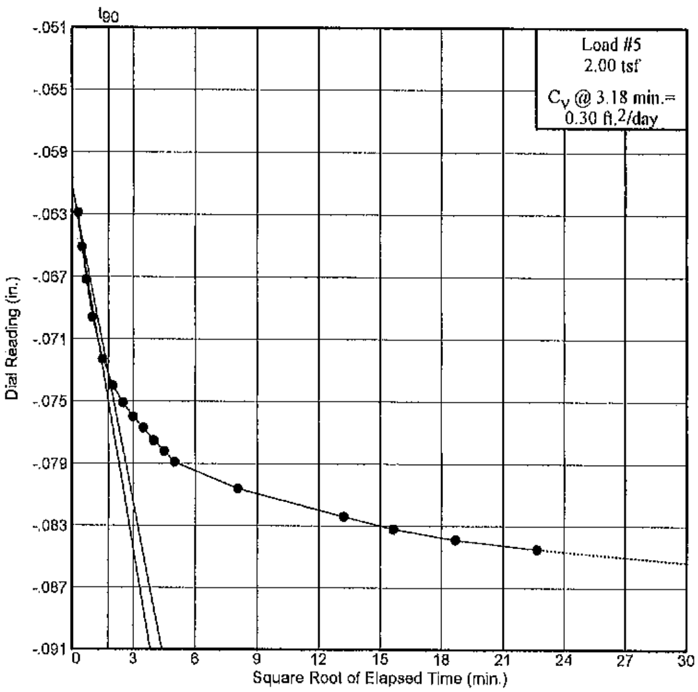
Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: R-6 Sample No.: 7/ST Elev./Depth: 15-17'









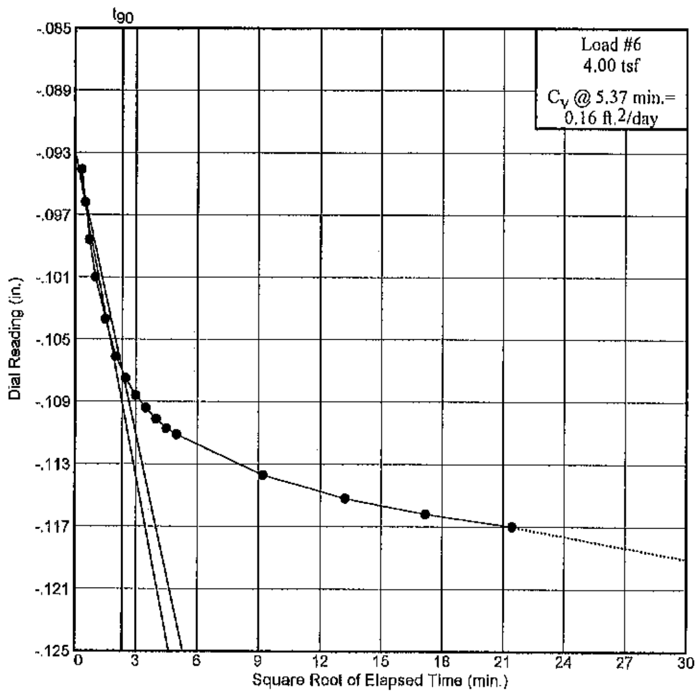
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

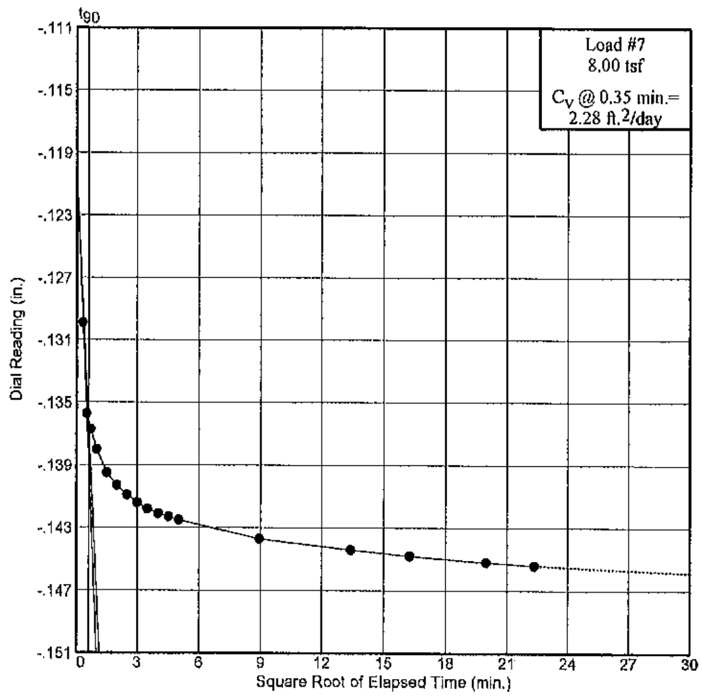
Figure

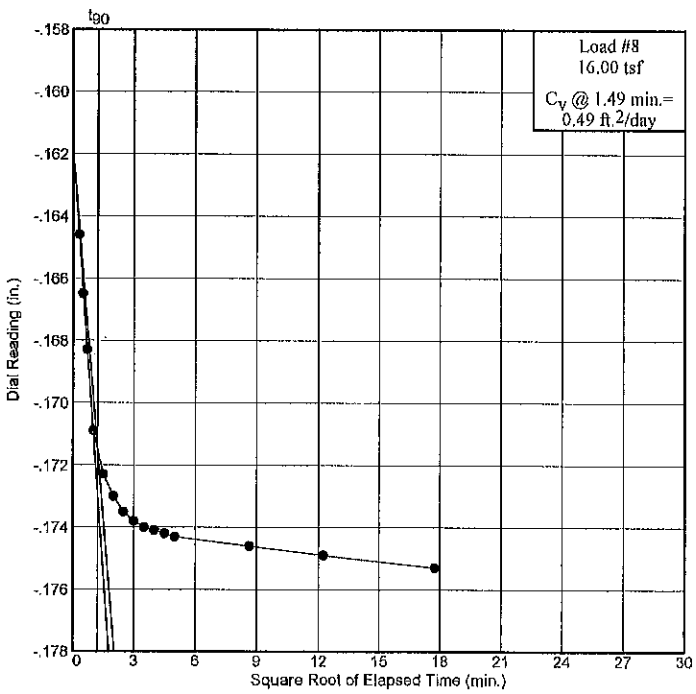
Dial Reading vs. Time

Project No.: N1105070
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source: R-6 | Sample No.: 7/ST | Elev./Depth: 15-17'



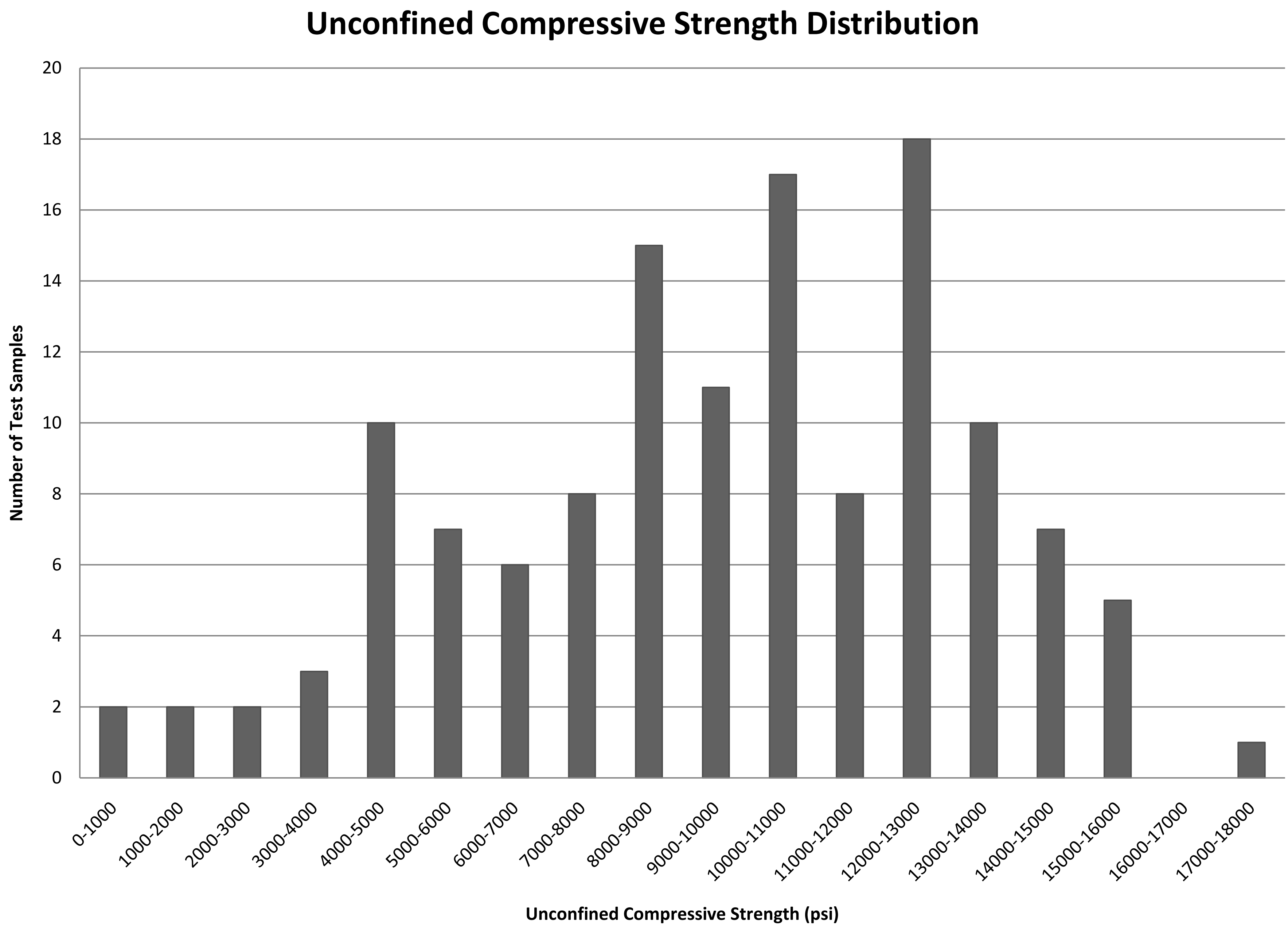




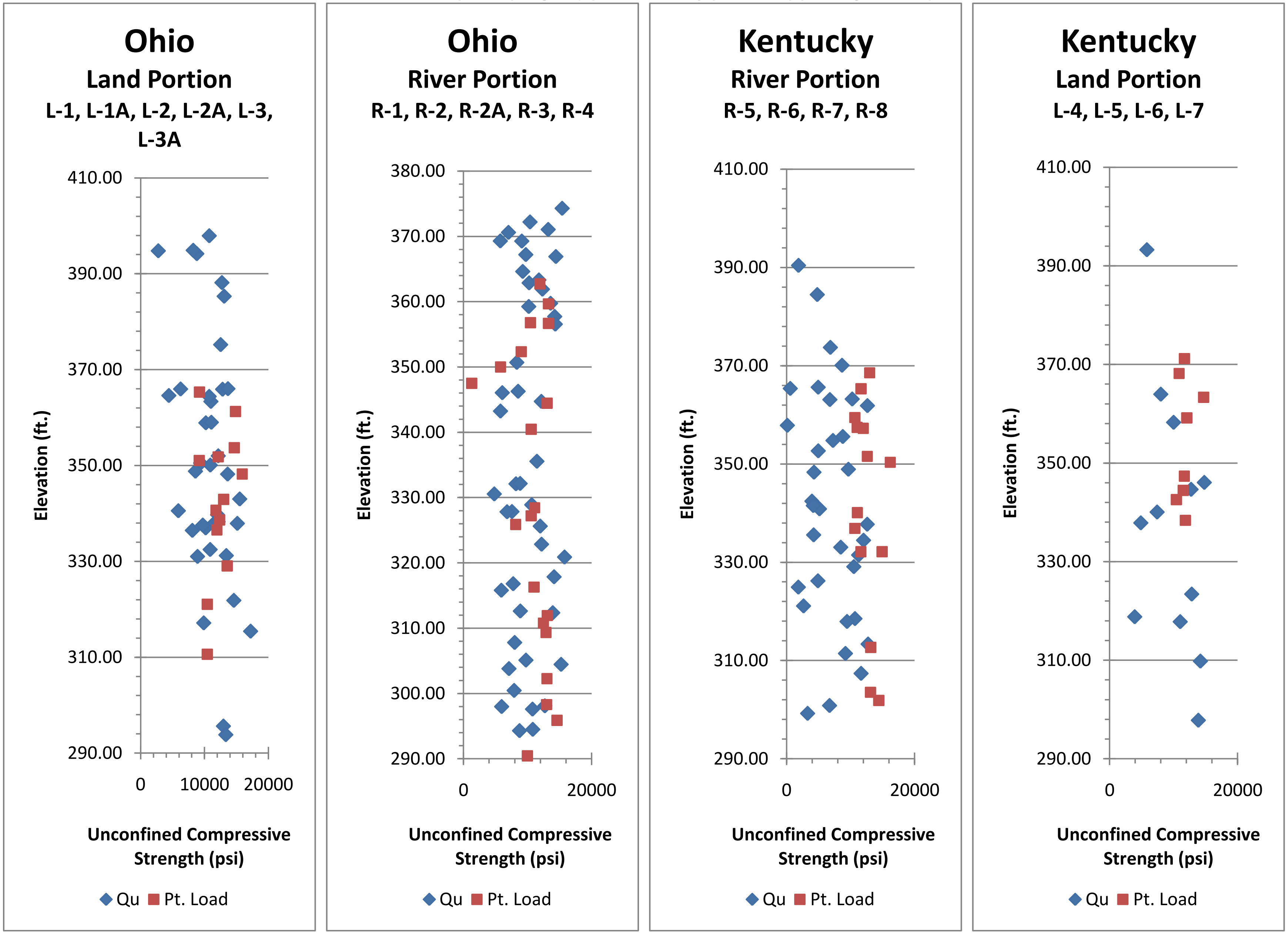
H.C. Nutting
A Terracon Company
Cincinnati, Ohio

Figure

UNCONFINED COMPRESSIVE STRENGTH TESTING RESULTS AND FIGURES


Unconfined Compressive Strength Distribution
Number of Test Samples
0
2
4
6
8
10
12
14
16
18
20
0-1000
1000-2000
2000-3000
3000-4000
4000-5000
5000-6000
6000-7000
7000-8000
8000-9000
9000-10000
10000-11000
11000-12000
12000-13000
13000-14000
14000-15000
15000-16000
16000-17000
17000-18000
Unconfined Compressive Strength (psi)

ELEVATION VS. UNCONFINED COMPRESSIVE STRENGTH



UNCONFINED COMPRESSION TEST RESULTS

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Unconfined Strength (psf)	Unconfined Strength (psi)	Water Content (%)	Rock Type
L-1	129.9	363.56	130.3	363.16	1581417	10982	0.2	Limestone
L-1	142.7	350.76	143.2	350.26	1349949	9375	0.4	Limestone
L-1	153.5	339.96	154	339.46	1731345	12023	0.5	Limestone
L-1	156	337.46	157	336.46	1463837	10166	0.2	Limestone
L-1	162.5	330.96	163	330.46	1245899	8652	1.3	Limestone/Shale
L-1A	123.1	368.35	123.7	367.75	1467597	10192	0.1	Limestone
L-1A	132.3	359.15	132.8	358.65	1958018	13597	0.2	Limestone
L-1A	143	348.45	143.5	347.95	848262	5891	1.2	Limestone
L-1A	150.7	340.75	151.1	340.35	1928313	13391	0.2	Limestone
L-1A	160	331.45	160.5	330.95	634960	4409	1.9	Limestone
L-2	126.7	369.56	127	369.26	1844685	12810	2.4	Limestone
L-2	130	366.26	130.7	365.56	1591219	11050	0.8	Limestone
L-2	137	359.26	137.5	358.76	1746923	12131	1.1	Limestone
L-2	144	352.26	144.5	351.76	2229975	15486	1.1	Limestone
L-2	148.2	348.06	148.5	347.76	599284	4162	2.4	Shale
L-2	153	343.26	153.5	342.76	1398280	9710	3.6	Limestone
L-2	154.5	341.76	155	341.26	1564616	10865	1.1	Limestone
L-2	158.5	337.76	158.9	337.36	1280499	8892	1.2	Limestone
L-2	163.6	332.66	164	332.26	899381	6246	1.7	Limestone
L-2	165.1	331.16	165.4	330.86	1542930	10715	1.1	Limestone
L-2A	130.1	364.40	130.5	364.00	1164152	8084	0.1	Limestone
L-2A	131.5	363.00	132.2	362.30	1264643	8782	0.3	Limestone
L-2A	137	357.50	137.4	357.10	267975	1861	4.5	Shale
L-2A	157.8	336.70	158.3	336.20	1233462	8566	0.7	Limestone
L-3	97.6	361.06	98	360.66	471917	3277	1.9	Limestone/Shale
L-3	100.2	358.46	100.4	358.26	1863379	12940	0.7	Limestone
L-3	103.8	354.86	104.4	354.26	1917187	13314	1.4	Limestone
L-3	121.2	337.46	121.8	336.86	965311	6704	1.3	Limestone/Shale
L-3	124.6	334.06	125.2	333.46	569305	3954	1.2	Limestone/Shale
L-3	145.2	313.46	146.2	312.46	1661279	11537	0.7	Limestone
L-3	145.6	313.06	146.1	312.56	1358505	9434	1.5	Limestone/Shale
L-3	158.7	299.96	160.2	298.46	2475252	17189	0.4	Limestone
L-3	162.8	295.86	163.3	295.36	1744346	12114	0.2	Limestone
L-3	164.5	294.16	165.2	293.46	2176614	15115	0.8	Limestone

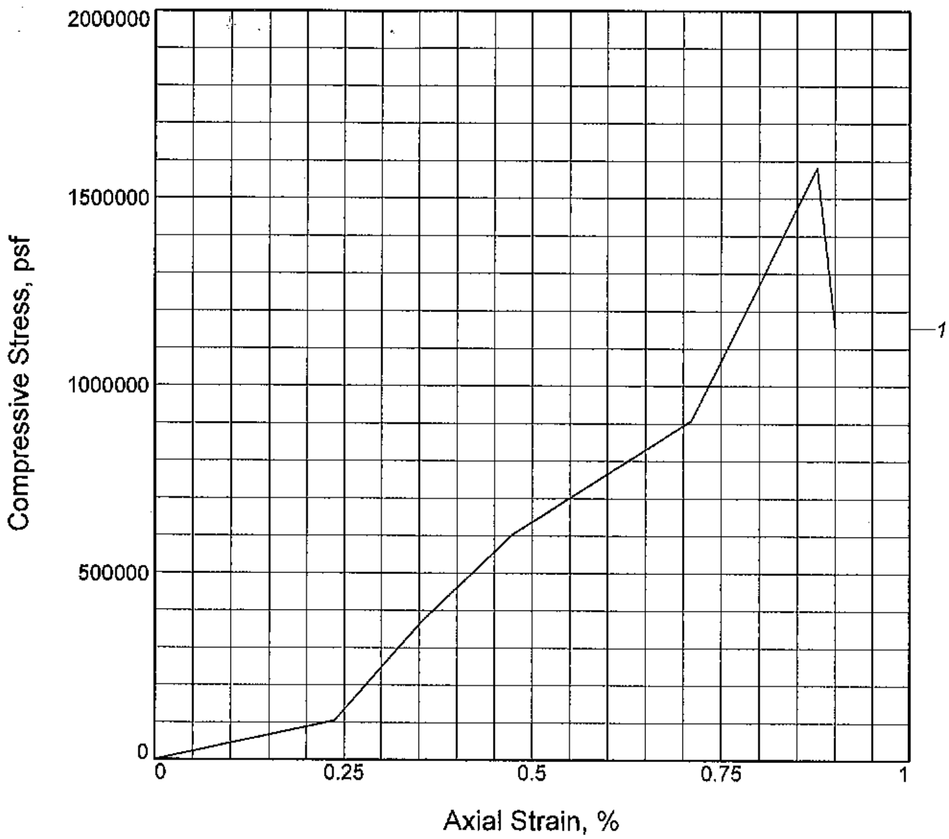
Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Unconfined Strength (psf)	Unconfined Strength (psi)	Water Content (%)	Rock Type
L-3A	126.5	369.55	126.75	369.30	82023	570	6.3	Shale
L-3A	142.3	353.80	142.5	353.55	615192	4272	2	Shale
L-3A	157.7	338.35	158	338.05	397254	2759	0.8	Limestone
L-4	116	363.97	116.5	363.47	1965081	13646	0.5	Limestone
L-4	120.4	359.57	120.9	359.07	1829521	12705	1.1	Limestone
L-4	140.5	339.47	141	338.97	1880122	13056	0.8	Limestone
L-4	143	336.97	143.5	336.47	1801226	12509	0.4	Limestone
L-5	113.5	372.83	114	372.33	972696	6755	2.4	Limestone/Shale
L-5	120.2	366.13	120.6	365.73	1567920	10888	0.2	Limestone
L-5	130.3	356.03	131	355.33	738738	5130	0.3	Limestone/Shale
L-5	133.3	353.03	133.8	352.53	1217480	8455	1.7	Limestone/Shale
L-6	112	373.69	112.4	373.29	703969	4889	0.3	Limestone/Shale
L-6	120.5	365.19	121	364.69	2097849	14568	0.2	Limestone
L-6	130.5	355.19	130.9	354.79	1420383	9864	0.2	Limestone
L-6	147.5	338.19	148	337.69	1544585	10726	0.2	Limestone
L-7	101	383.41	101.5	382.91	1183176	8217	0.3	Limestone
L-7	113.7	370.71	114.2	370.21	842027	5847	0.3	Limestone
L-7	132.5	351.91	133.2	351.21	689715	4790	0.6	Limestone/Shale
R-1	91.5	366.54	92.1	365.94	1837107	12758	0.4	Limestone
R-1	94.3	363.74	95	363.04	706054	4903	2.4	Limestone
R-1	104.5	353.54	105	353.04	568922	3951	2.4	Limestone
R-1	123	335.04	123.5	334.54	1443507	10024	0.8	Limestone
R-1	136	322.04	136.5	321.54	2134074	14820	0.6	Limestone
R-1	145.3	312.74	145.7	312.34	1072646	7449	1.3	Limestone
R-1	153	305.04	153.6	304.44	1850857	12853	0.6	Limestone
R-1	159.1	298.94	159.9	298.14	1592203	11057	0.7	Limestone
R-1	163.5	294.54	164.2	293.84	2046785	14214	1.2	Limestone
R-1	168.2	289.84	168.9	289.14	2000122	13890	1.2	Limestone
R-2	87.5	370.60	88	370.10	1893232	13147	0.2	Limestone
R-2	89.3	368.80	89.7	368.40	1387302	9634	0.6	Limestone
R-2	90.7	367.40	91.6	366.50	1848338	12836	0.4	Limestone
R-2	93.7	364.40	94	364.10	709761	4929	2.2	Shale
R-2	99.8	358.30	100.1	358.00	1155667	8025	0.8	Limestone
R-2	112.9	345.20	113.9	344.20	2034883	14131	0.2	Limestone
R-2	119.8	338.30	120.6	337.50	2005345	13926	0.4	Limestone
R-2	139	319.10	139.5	318.60	1138483	7906	1.2	Limestone
R-2A	99.5	358.14	100.1	357.54	2075031	14410	0.6	Limestone

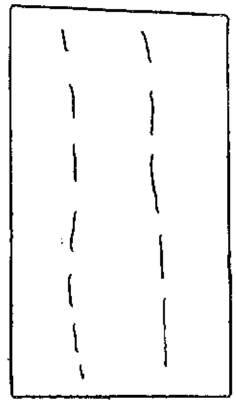
Unconfined Compression Test Results
Parsons Brinckerhoff ■ Brent Spence Bridge Replacement ■ Cincinnati, Ohio
March 11, 2011 ■ HCN/Terracon Project No. N1105070

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Unconfined Strength (psf)	Unconfined Strength (psi)	Water Content (%)	Rock Type
R-2A	111.8	345.84	112.2	345.44	1773180	12314	1.1	Limestone
R-2A	117.8	339.84	118.2	339.44	872331	6058	1.9	Limestone
R-2A	120.5	337.13	121	336.64	607979	4222	1.7	Limestone/Shale
R-2A	134.4	323.24	134.9	322.74	1089495	7566	1.2	Limestone
R-2A	140	317.64	140.5	317.14	1117004	7757	0.7	Limestone
R-2A	148	309.64	148.5	309.14	2192551	15226	0.2	Limestone
R-2A	160	297.64	160.5	297.14	1550817	10770	0.6	Limestone
R-2A	175.8	281.84	176.3	281.34	1495031	10382	0.8	Limestone
R-2A	179.8	277.84	180.3	277.34	1902575	13212	0.1	Limestone
R-2A	183.5	274.14	184	273.64	1400566	9726	0.1	Limestone
R-3	92.3	365.71	92.7	365.31	1331115	9244	0.7	Limestone
R-3	93.8	364.21	94.5	363.51	1474639	10241	0.2	Limestone
R-3	102.7	355.31	103.1	354.91	1041933	7236	1.7	Limestone/Shale
R-3	106.5	351.51	107.1	350.91	1322957	9187	2.2	Limestone/Shale
R-3	123.8	334.21	124.7	333.31	983924	6833	0.9	Limestone/Shale
R-3	140	318.01	140.5	317.51	1310334	9100	0.5	Limestone
R-3	145.5	312.51	146	312.01	1694430	11767	1.1	Limestone
R-3	157.3	300.71	158	300.01	2048572	14226	0.5	Limestone
R-4	90.5	367.48	91	366.98	1198015	8320	4.2	Limestone
R-4	95.5	362.48	96	361.98	832099	5778	4.2	Limestone
R-4	102.8	355.18	103.3	354.68	380693	2644	1.9	Limestone/Shale
R-4	111.3	346.68	111.9	346.08	857943	5958	1.5	Limestone
R-4	121.9	336.08	122.3	335.68	2216031	15389	1.1	Limestone
R-4	129.6	328.38	130	327.98	828577	5754	1.5	Limestone
R-4	140.6	317.38	141.1	316.88	1956363	13586	0.4	Limestone
R-4	152.8	305.18	153.6	304.38	1534100	10653	1.1	Limestone
R-4	159.6	298.38	160.5	297.48	2269771	15762	0.5	Limestone
R-5	85.2	373.39	85.7	372.89	1022251	7099	2.9	Limestone
R-5	86.4	372.19	86.8	371.79	1556479	10809	0.3	Limestone
R-5	90.1	368.49	90.8	367.79	1011411	7024	0.5	Limestone
R-5	92.2	366.39	92.8	365.79	16945	118	8.6	Shale
R-5	93	365.59	93.8	364.79	2062678	14324	0.6	Limestone
R-5	95	363.59	95.3	363.29	1179728	8193	0.5	Limestone
R-5	103	355.59	103.5	355.09	692912	4812	1.5	Limestone/Shale
R-5	146.2	312.39	147	311.59	1753704	12179	0.5	Limestone
R-6	84.1	372.94	84.5	372.54	851152	5911	0.6	Limestone
R-6	88.5	368.54	89	368.04	1150291	7988	0.7	Limestone

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Unconfined Strength (psf)	Unconfined Strength (psi)	Water Content (%)	Rock Type
R-6	94.5	362.54	94.9	362.14	1403296	9745	0.1	Limestone
R-6	100.1	356.94	100.5	356.54	1828127	12695	0.1	Limestone
R-6	107.1	349.94	107.5	349.54	1259226	8745	0.5	Limestone
R-6	114.5	342.54	115	342.04	1466508	10184	0.2	Limestone
R-6	136.5	320.54	137.3	319.74	1649615	11456	0.3	Limestone
R-6	159.8	297.24	160.2	296.84	1273413	8843	0.8	Limestone
R-7	83.5	374.96	83.9	374.56	1277541	8872	0.3	Limestone
R-7	88.4	370.06	89	369.46	1748783	12144	0.3	Limestone
R-7	98	360.46	98.5	359.96	979514	6802	1.0	Limestone
R-7	121.1	337.36	121.4	337.06	263952	1833	3.5	Shale
R-7	128.7	329.76	129.5	328.96	1227670	8525	0.5	Limestone
R-7	136.6	321.86	137.6	320.86	1724247	11974	0.4	Limestone
R-7	154.5	303.96	155.1	303.36	1812415	12586	0.5	Limestone
R-7	163.7	294.76	164.5	293.96	1263171	8772	0.4	Limestone
R-8	87.8	367.90	88.2	367.50	1388903	9645	0.1	Limestone
R-8	100.5	355.20	101	354.70	1618490	11240	0.6	Limestone
R-8	101.8	353.90	102.3	353.40	711870	4944	1	Limestone/Shale
R-8	126.3	329.40	126.7	329.00	1674834	11631	0.7	Limestone
R-8	127.8	327.90	128.3	327.40	1537026	10674	0.3	Limestone
R-8	135.5	320.20	136	319.70	1511267	10495	0.5	Limestone
R-8	141	314.70	141.5	314.20	1831836	12721	0.3	Limestone
R-8	149	306.70	149.5	306.20	1817085	12619	0.3	Limestone
R-8	151.8	303.90	152.1	303.60	1475126	10244	0.2	Limestone
R-8	158.7	297.00	159.2	296.50	1729572	12011	0.2	Limestone

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1581417.8			
Undrained shear strength, psf	790708.9			
Failure strain, %	0.9			
Strain rate, in./min.	0.042			
Water content, %	0.2			
Wet density, pcf	166.4			
Dry density, pcf	166.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.220			
Height/diameter ratio	2.14			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-29-10
Remarks:
Lab No. 6003

Figure ________

Client: PARSONS BRINCKERHOFF

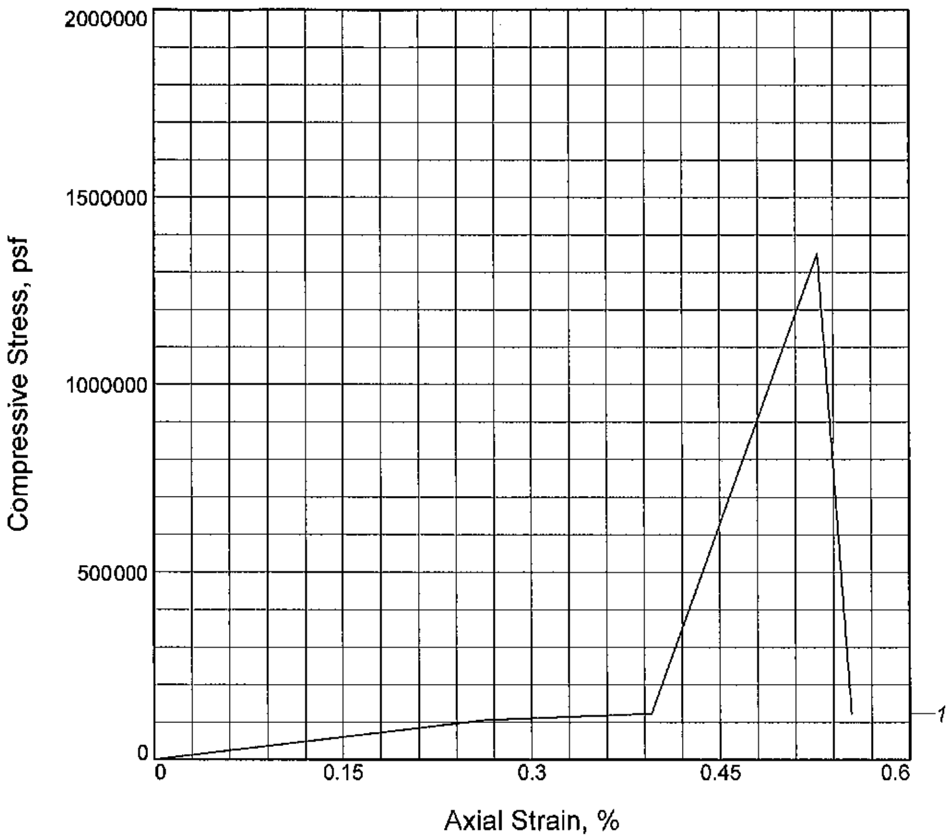
Project: BRENT SPENCE BRIDGE REPLACEMENT

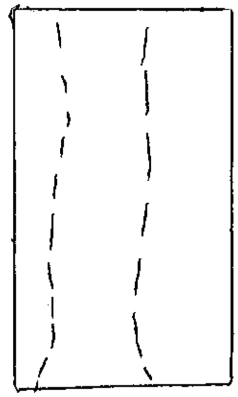
Source of Sample: L-1 **Depth:** 129.9-130.3'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1349949.2			
Undrained shear strength, psf	674974.6			
Failure strain, %	0.5			
Strain rate, in./min.	0.037			
Water content, %	0.4			
Wet density, pcf	167.3			
Dry density, pcf	166.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.790			
Height/diameter ratio	1.92			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-29-10
Remarks:
Lab No. 6006

Figure ________

Client: PARSONS BRINCKERHOFF

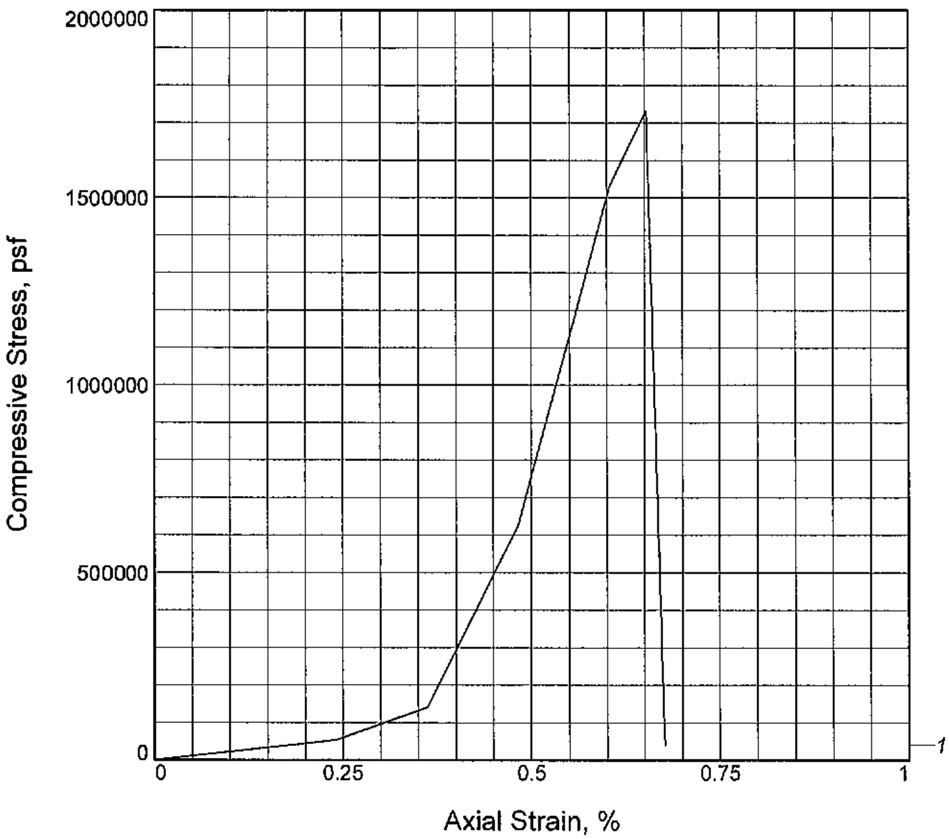
Project: BRENT SPENCE BRIDGE REPLACEMENT

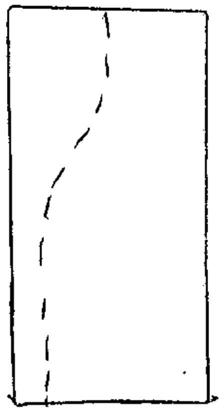
Source of Sample: L-1 **Depth:** 142.7-143.2'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1731345.1			
Undrained shear strength, psf	865672.5			
Failure strain, %	0.7			
Strain rate, in./min.	0.041			
Water content, %	0.5			
Wet density, pcf	167.6			
Dry density, pcf	166.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.140			
Height/diameter ratio	2.10			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-29-10
Remarks:
Lab No. 6010

Figure ________

Client: PARSONS BRINCKERHOFF

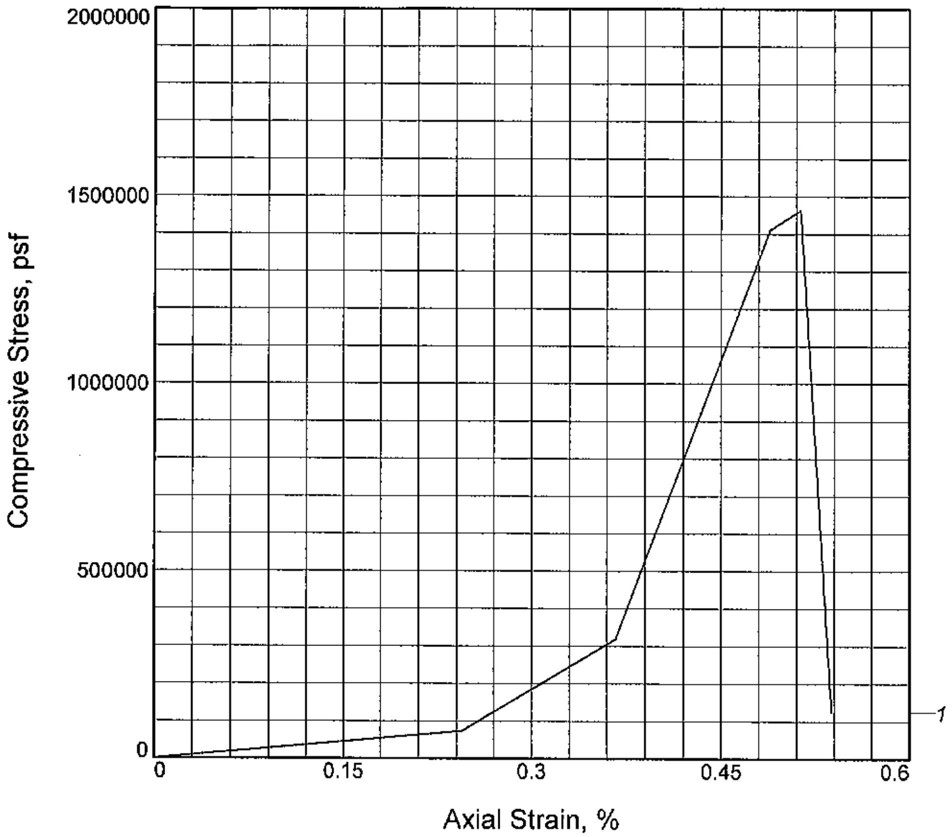
Project: BRENT SPENCE BRIDGE REPLACEMENT

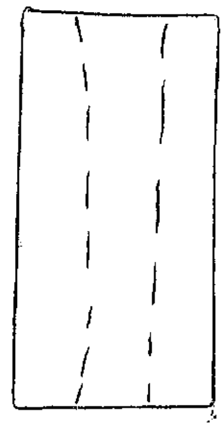
Source of Sample: L-1 **Depth:** 153.5-154'
Sample Number: 8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1463837.5			
Undrained shear strength, psf	731918.8			
Failure strain, %	0.5			
Strain rate, in./min.	0.040			
Water content, %	0.2			
Wet density, pcf	168.0			
Dry density, pcf	167.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.090			
Height/diameter ratio	2.08			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-29-10

Remarks:
Lab No. 6011

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

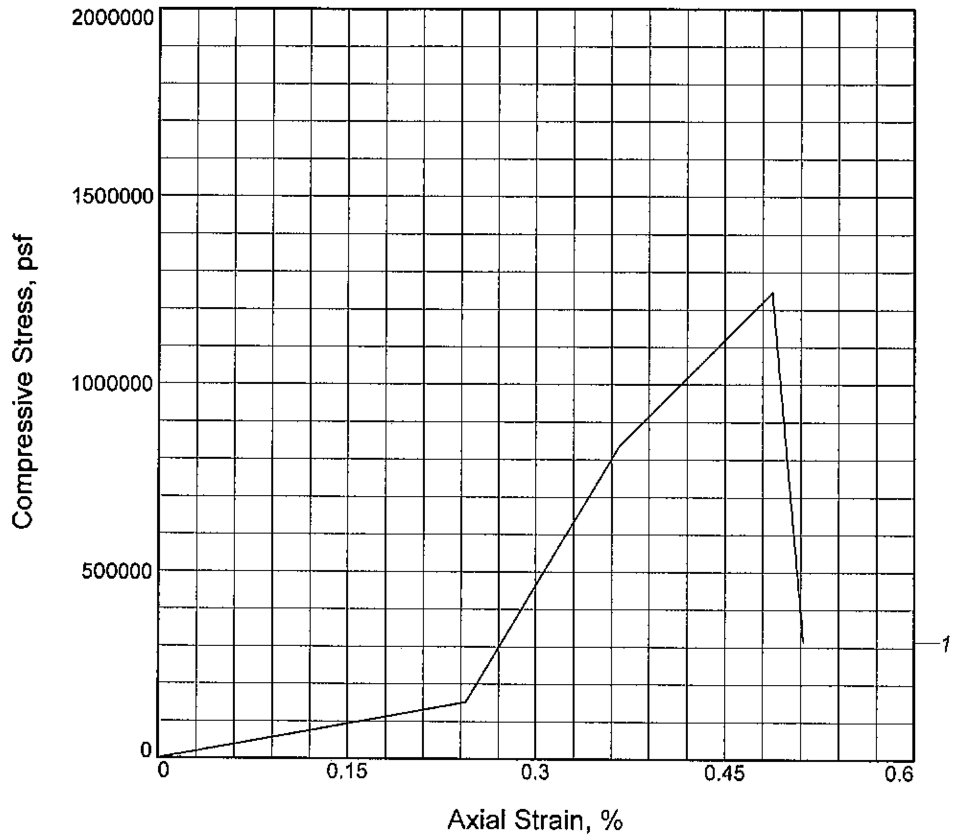
Source of Sample: L-1 **Depth:** 156-157'
Sample Number: 9

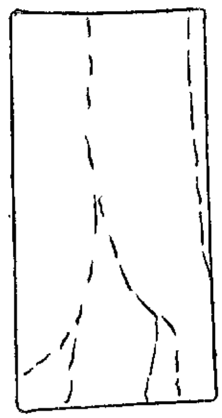
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1245899.8			
Undrained shear strength, psf	622949.9			
Failure strain, %	0.5			
Strain rate, in./min.	0.041			
Water content, %	1.3			
Wet density, pcf	167.0			
Dry density, pcf	164.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.100			
Height/diameter ratio	2.07			

Description: LIMESTONE AND SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone and Shale

Project No.: N1105070

Date Sampled: 7-29-10

Remarks:
Lab No. 6013

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-1 **Depth:** 162.5-163'

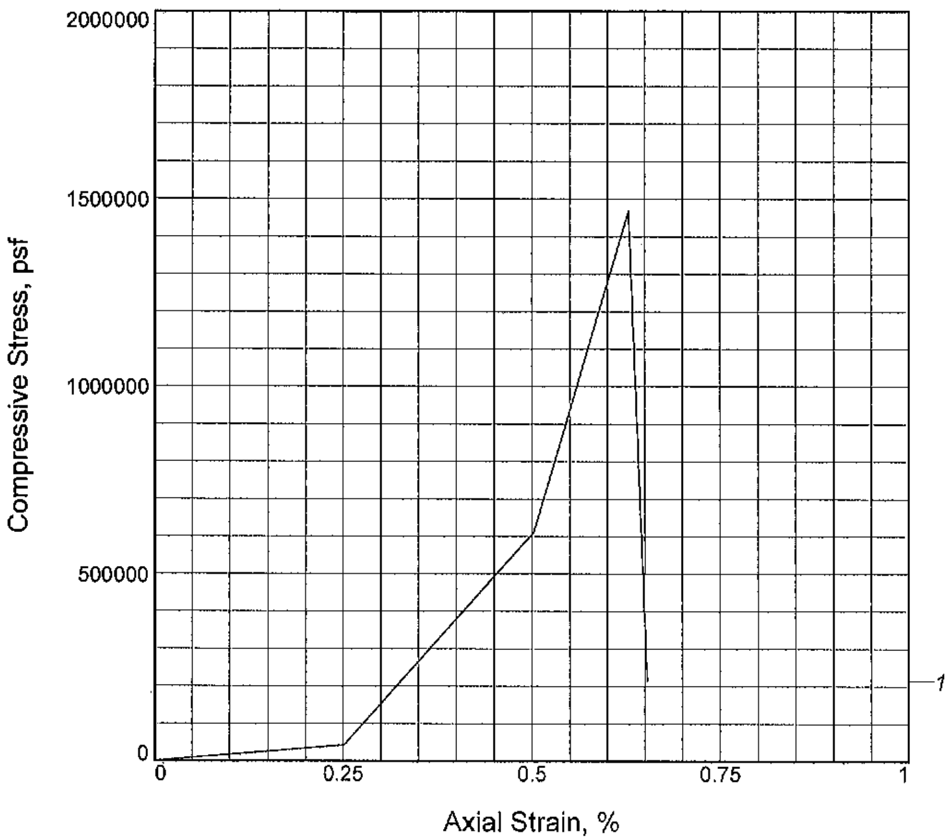
Sample Number: 11

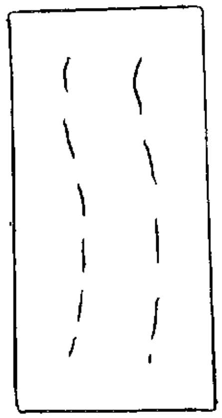
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1467597.4			
Undrained shear strength, psf	733798.7			
Failure strain, %	0.6			
Strain rate, in./min.	0.039			
Water content, %	0.1			
Wet density, pcf	164.2			
Dry density, pcf	164.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.980			
Height/diameter ratio	2.00			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-27-10
Remarks:
Lab No. 7314

Figure ________

Client: PARSONS BRINCKERHOFF

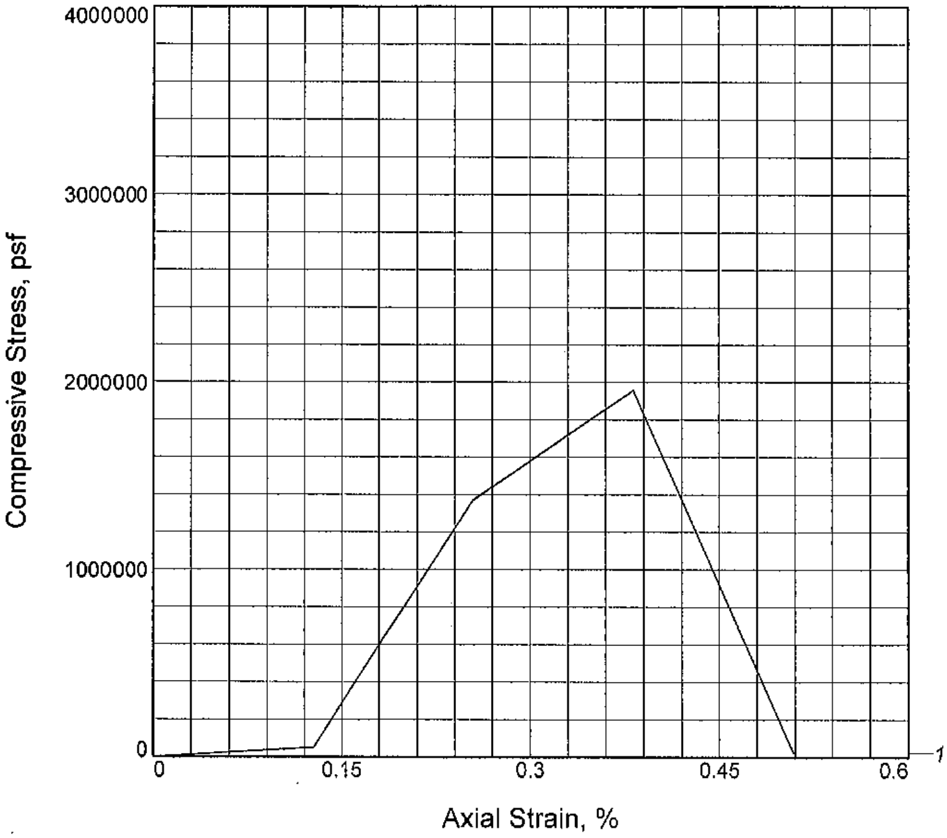
Project: BRENT SPENCE BRIDGE REPLACEMENT

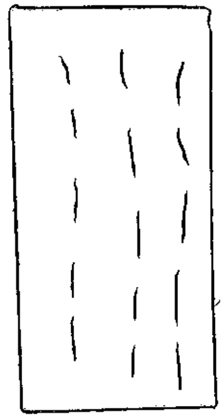
Source of Sample: L-1A **Depth:** 123.1-123.7'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1958018.3			
Undrained shear strength, psf	979009.1			
Failure strain, %	0.4			
Strain rate, in./min.	0.039			
Water content, %	0.2			
Wet density, pcf	164.6			
Dry density, pcf	164.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.930			
Height/diameter ratio	1.97			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-27-10
Remarks:
Lab No. 7315

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

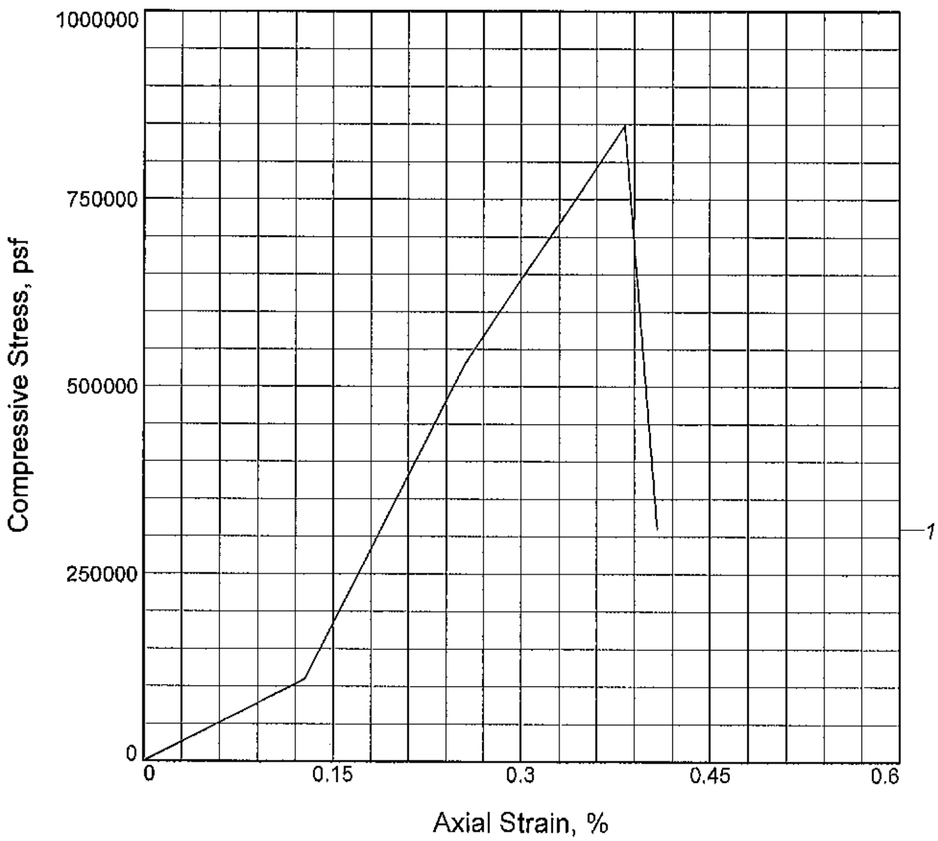
Source of Sample: L-1A **Depth:** 132.3-132.8'
Sample Number: 2

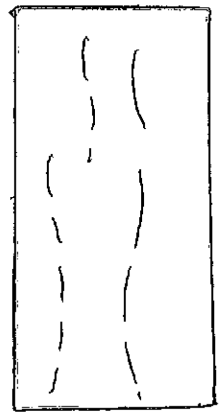
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	848262.7			
Undrained shear strength, psf	424131.4			
Failure strain, %	0.4			
Strain rate, in./min.	0.039			
Water content, %	1.2			
Wet density, pcf	162.0			
Dry density, pcf	160.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.920			
Height/diameter ratio	1.97			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-27-10
Remarks:
Lab No. 7317

Figure ________

Client: PARSONS BRINCKERHOFF

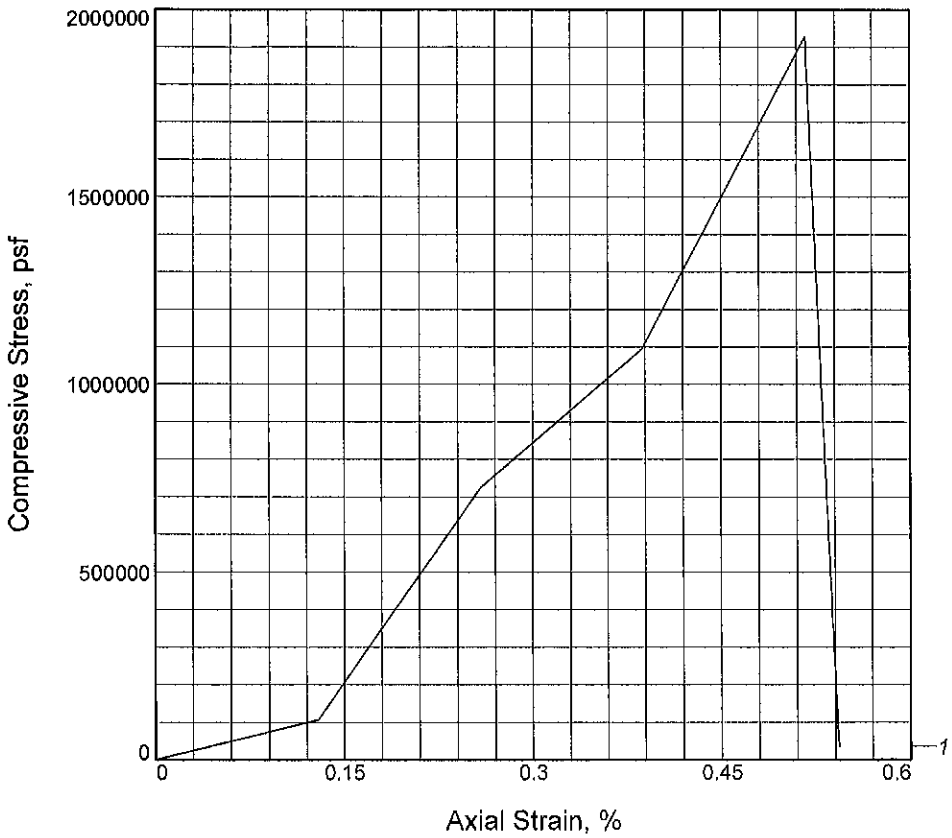
Project: BRENT SPENCE BRIDGE REPLACEMENT

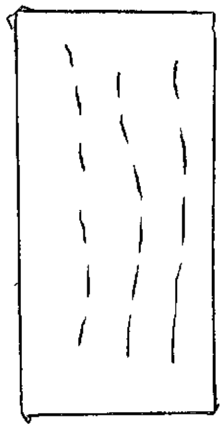
Source of Sample: L-1A **Depth:** 143-143.5'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1928313.7			
Undrained shear strength, psf	964156.9			
Failure strain, %	0.5			
Strain rate, in./min.	0.038			
Water content, %	0.2			
Wet density, pcf	167.5			
Dry density, pcf	167.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.870			
Height/diameter ratio	1.94			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-27-10
Remarks:
Lab No. 7319

Figure ________

Client: PARSONS BRINCKERHOFF

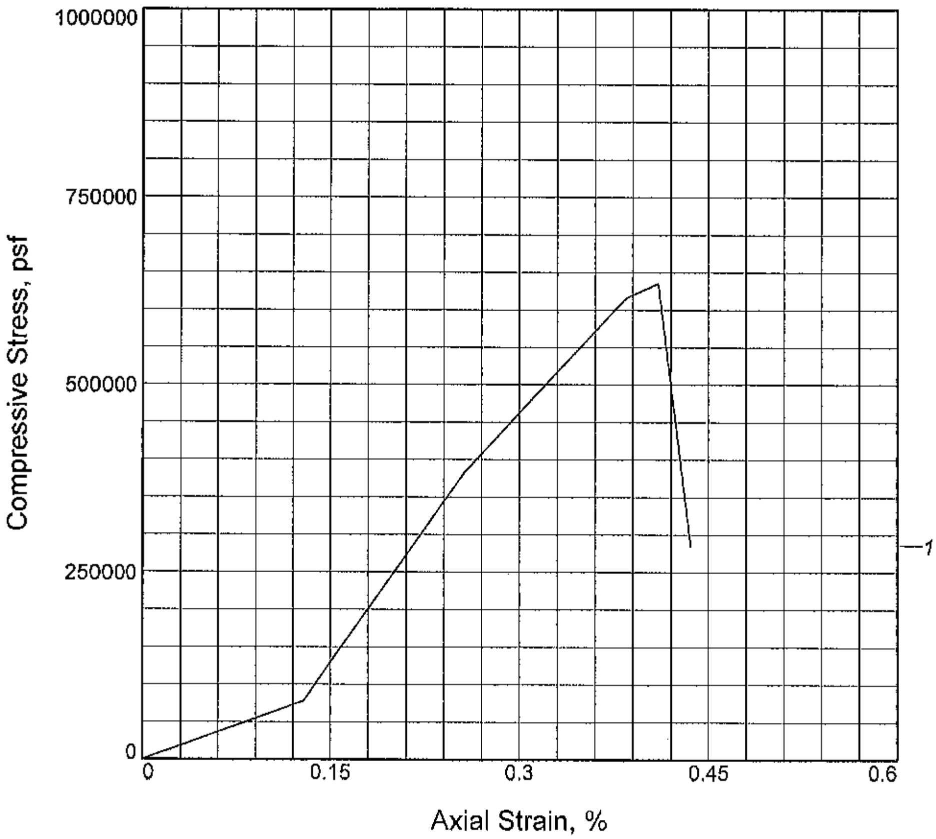
Project: BRENT SPENCE BRIDGE REPLACEMENT

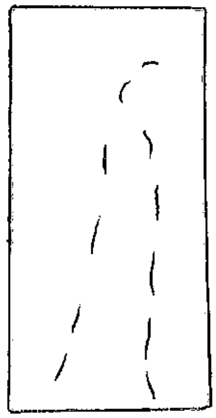
Source of Sample: L-1A **Depth:** 150.7-151.2'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	634960.3			
Undrained shear strength, psf	317480.1			
Failure strain, %	0.4			
Strain rate, in./min.	0.039			
Water content, %	1.9			
Wet density, pcf	162.7			
Dry density, pcf	159.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.900			
Height/diameter ratio	1.96			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date Sampled: 8-27-10

Remarks:
Lab No. 7322

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

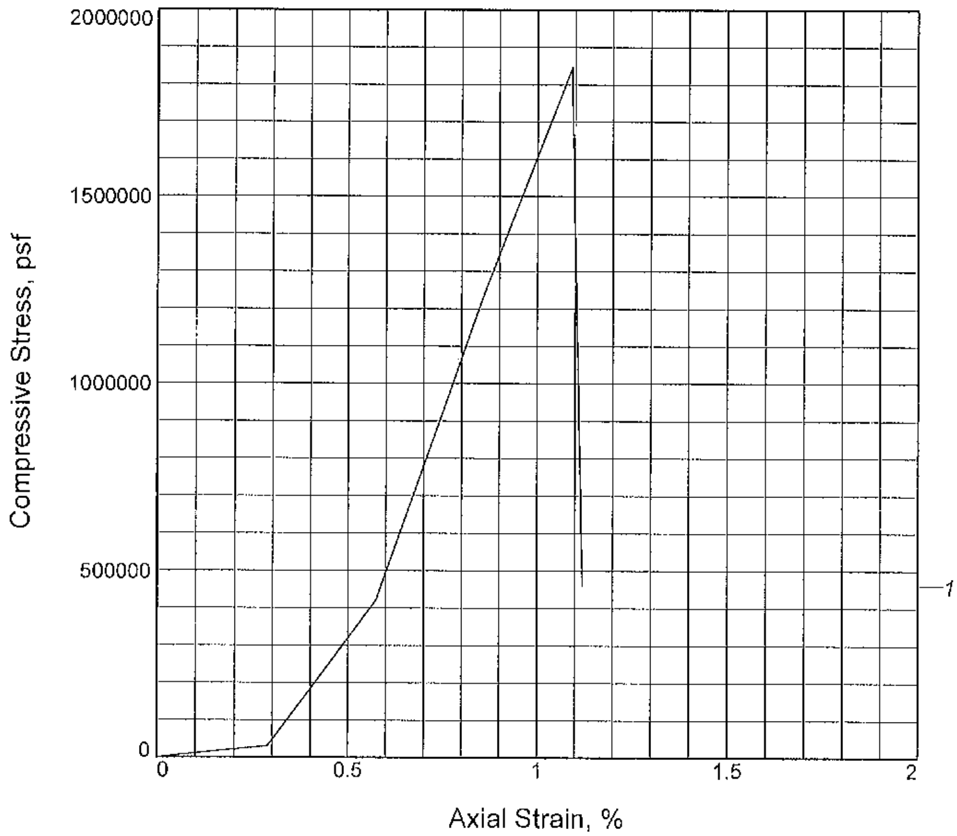
Source of Sample: L-1A **Depth:** 160-160.5'
Sample Number: 9

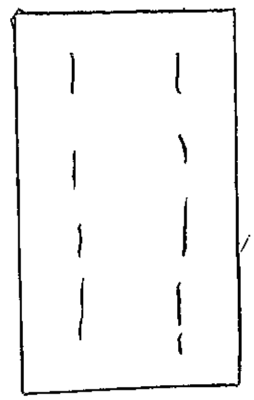
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1844685.1			
Undrained shear strength, psf	922342.5			
Failure strain,	1.1			
Strain rate, in./min.	0.034			
Water content, %	2.4			
Wet density, pcf	160.9			
Dry density, pcf	157.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.480			
Height/diameter ratio	1.73			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4882

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

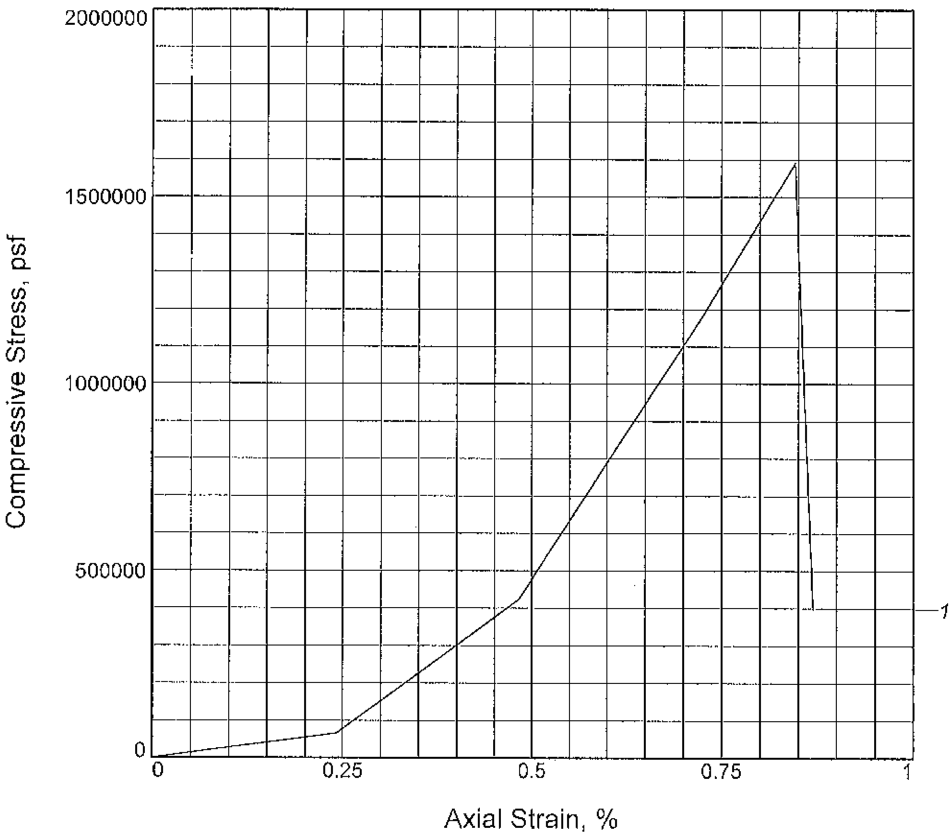
Source of Sample: L-2 **Depth:** 126.7-127'
Sample Number: 1/NQ

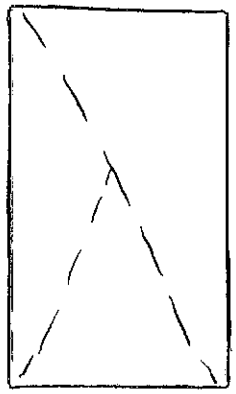
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1591219.7			
Undrained shear strength, psf	795609.8			
Failure strain,	0.8			
Strain rate, in./min.	0.041			
Water content, %	0.8			
Wet density, pcf	163.4			
Dry density, pcf	162.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	4.140			
Height/diameter ratio	2.06			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4884

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

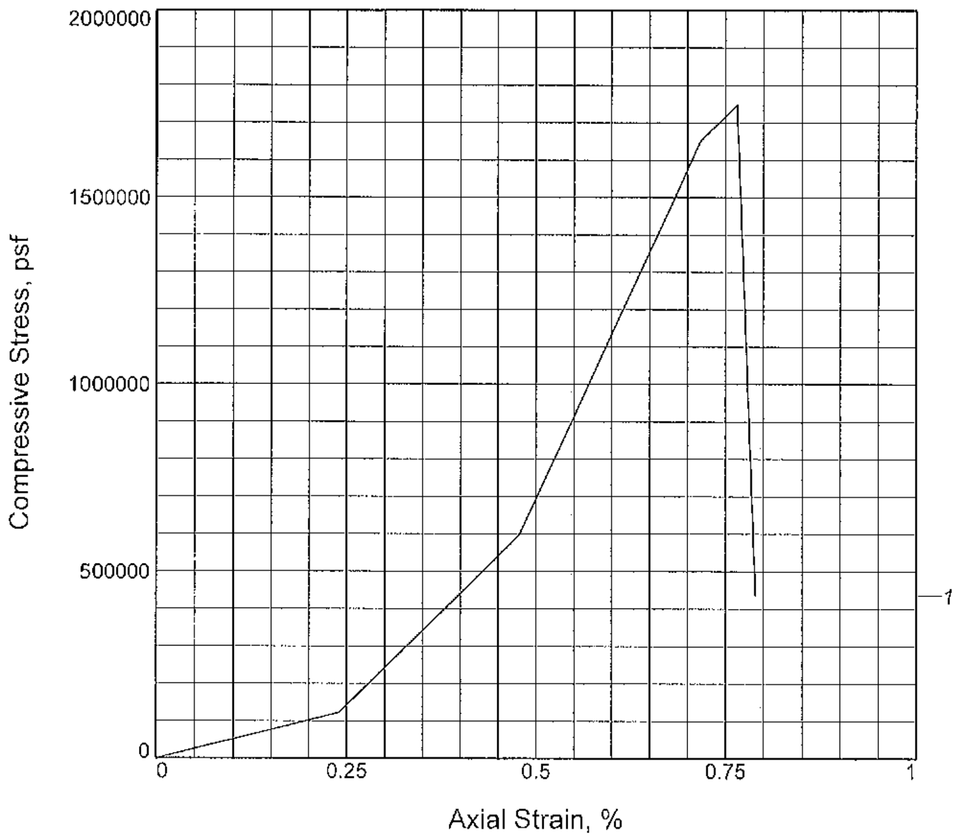
Source of Sample: L-2 **Depth:** 130-130.7'
Sample Number: 4/NQ

UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1746923.7			
Undrained shear strength, psf	873461.9			
Failure strain,	0.8			
Strain rate, in./min.	0.041			
Water content, %	1.1			
Wet density, pcf	163.7			
Dry density, pcf	161.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	4.180			
Height/diameter ratio	2.08			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date: 6-15-10

Remarks:
Lab No. 4885

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-2 **Depth:** 137-137.5'

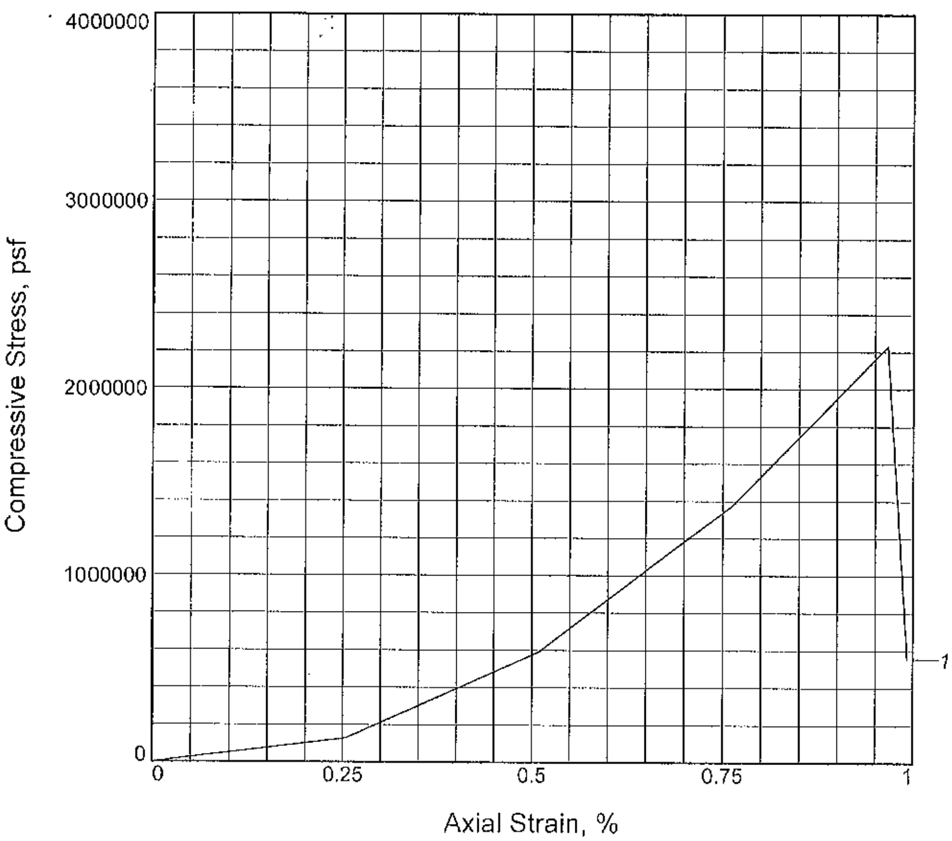
Sample Number: 3/NQ

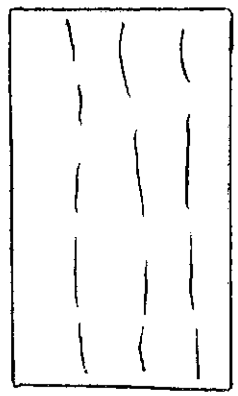
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2229975.2			
Undrained shear strength, psf	1114987.6			
Failure strain,	1.0			
Strain rate, in./min.	0.039			
Water content, %	1.1			
Wet density, pcf	164.8			
Dry density, pcf	163.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.930			
Height/diameter ratio	1.96			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4887

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

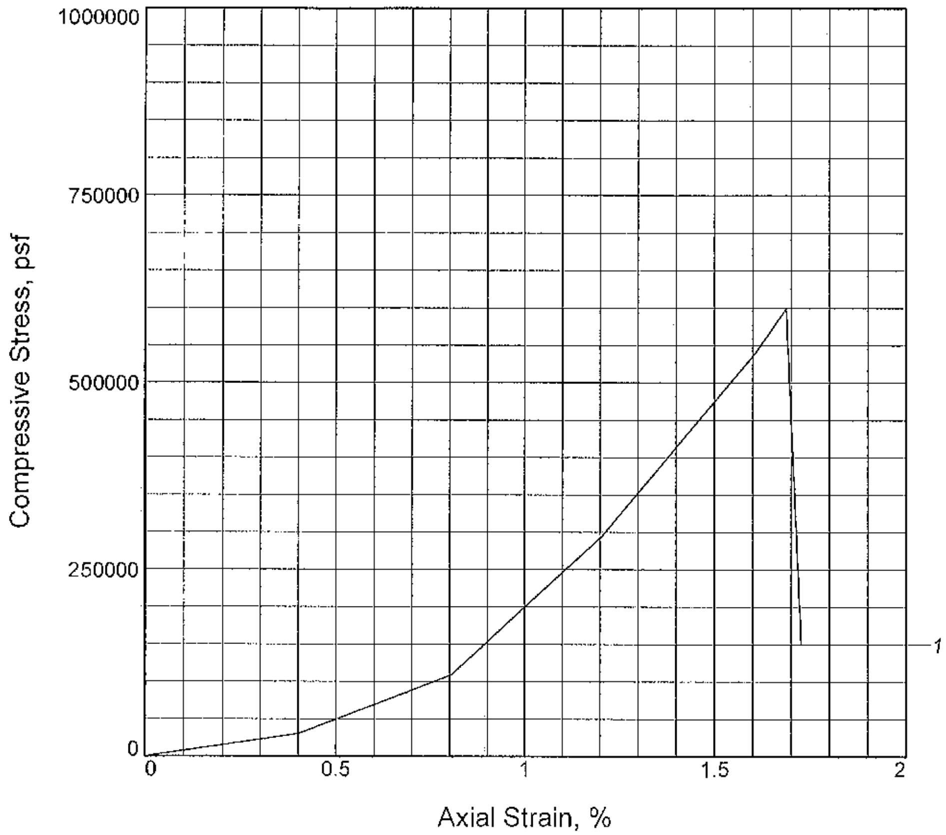
Source of Sample: L-2 **Depth:** 144-144.5'
Sample Number: 5/NQ

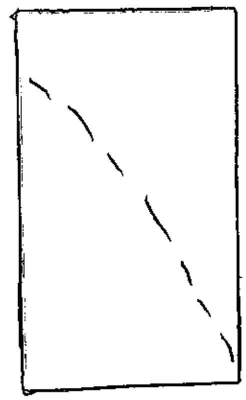
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	599284.8			
Undrained shear strength, psf	299642.4			
Failure strain,	1.7			
Strain rate, in./min.	0.024			
Water content, %	2.4			
Wet density, pcf	144.2			
Dry density, pcf	140.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	2.490			
Height/diameter ratio	1.24			

Description: SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4888

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

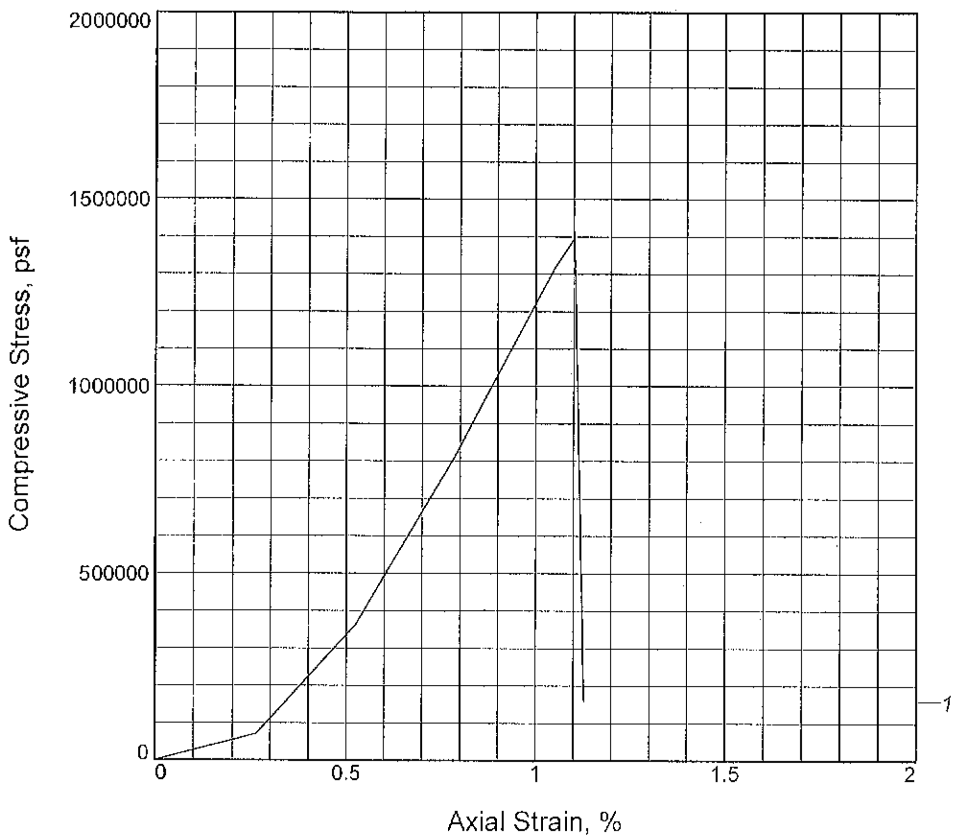
Source of Sample: L-2 **Depth:** 148.2-148.5'
Sample Number: 5/NQ

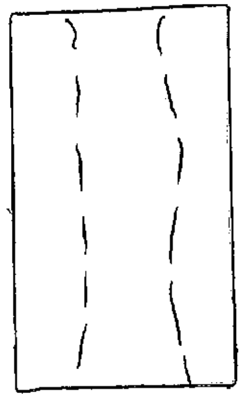
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1398280.1			
Undrained shear strength, psf	699140.0			
Failure strain,	1.1			
Strain rate, in./min.	0.038			
Water content, %	3.6			
Wet density, pcf	161.9			
Dry density, pcf	156.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.810			
Height/diameter ratio	1.90			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date: 6-15-10

Remarks:
Lab No. 4889

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

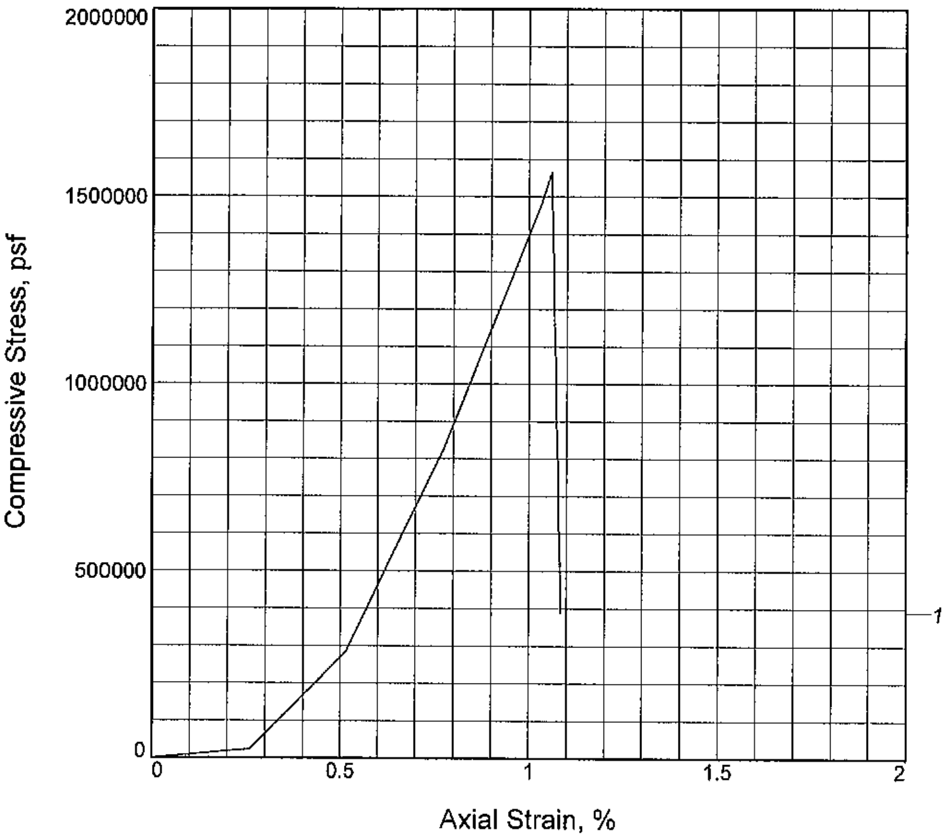
Source of Sample: L-2 **Depth:** 153-153.5'
Sample Number: 6/NQ

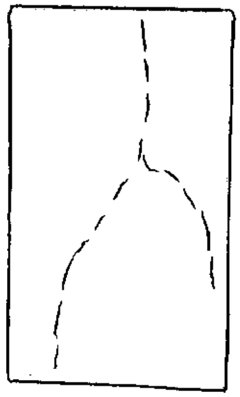
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1564616.2			
Undrained shear strength, psf	782308.1			
Failure strain, %	1.1			
Strain rate, in./min.	0.038			
Water content, %	1.1			
Wet density, pcf	162.7			
Dry density, pcf	160.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.870			
Height/diameter ratio	1.93			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 6-15-10
Remarks:
Lab No. 4890

Figure ________

Client: PARSONS BRINCKERHOFF

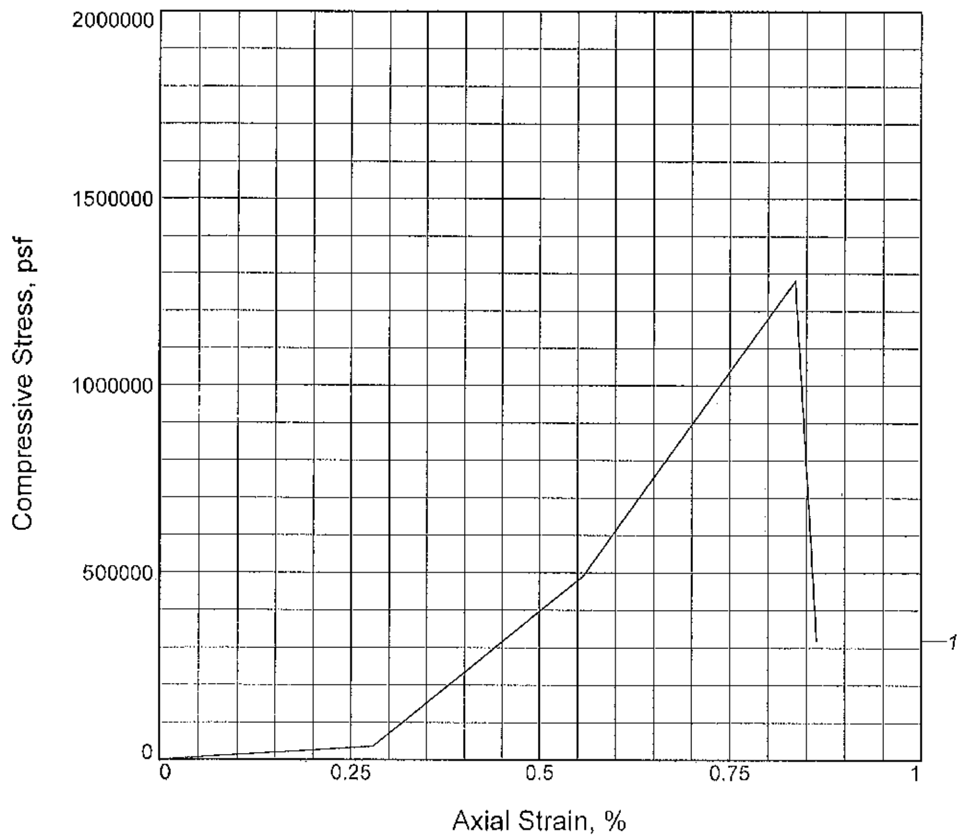
Project: BRENT SPENCE BRIDGE REPLACEMENT

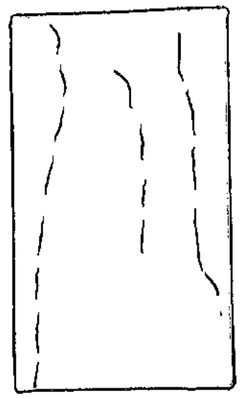
Source of Sample: L-2 **Depth:** 154.5-155'
Sample Number: 7/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1280499.4			
Undrained shear strength, psf	640249.7			
Failure strain,	0.8			
Strain rate, in./min.	0.035			
Water content, %	1.2			
Wet density, pcf	159.3			
Dry density, pcf	157.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.590			
Height/diameter ratio	1.79			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4891

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-2 **Depth:** 158.5-158.9'
Sample Number: 8/NQ

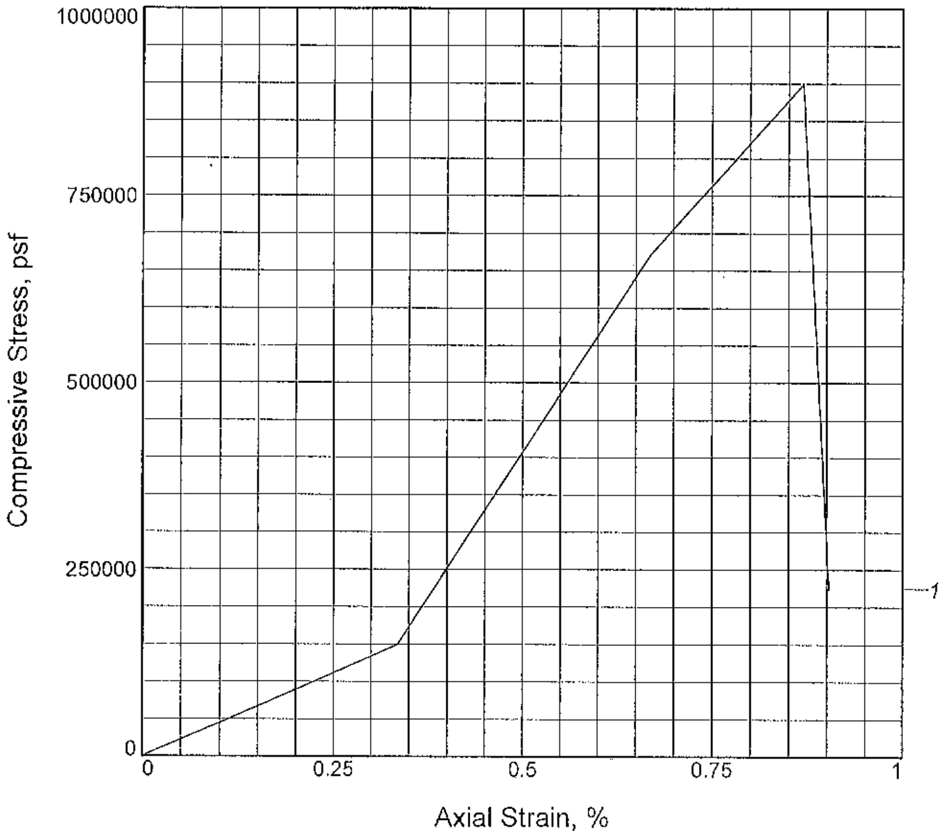
UNCONFINED COMPRESSION TEST

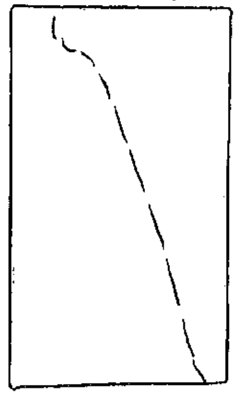
H. C. NUTTING COMPANY

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	899381.1			
Undrained shear strength, psf	449690.5			
Failure strain,	0.9			
Strain rate, in./min.	0.029			
Water content, %	1.7			
Wet density, pcf	160.4			
Dry density, pcf	157.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	2.990			
Height/diameter ratio	1.49			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4892

Figure ________

Client: PARSONS BRINCKERHOFF

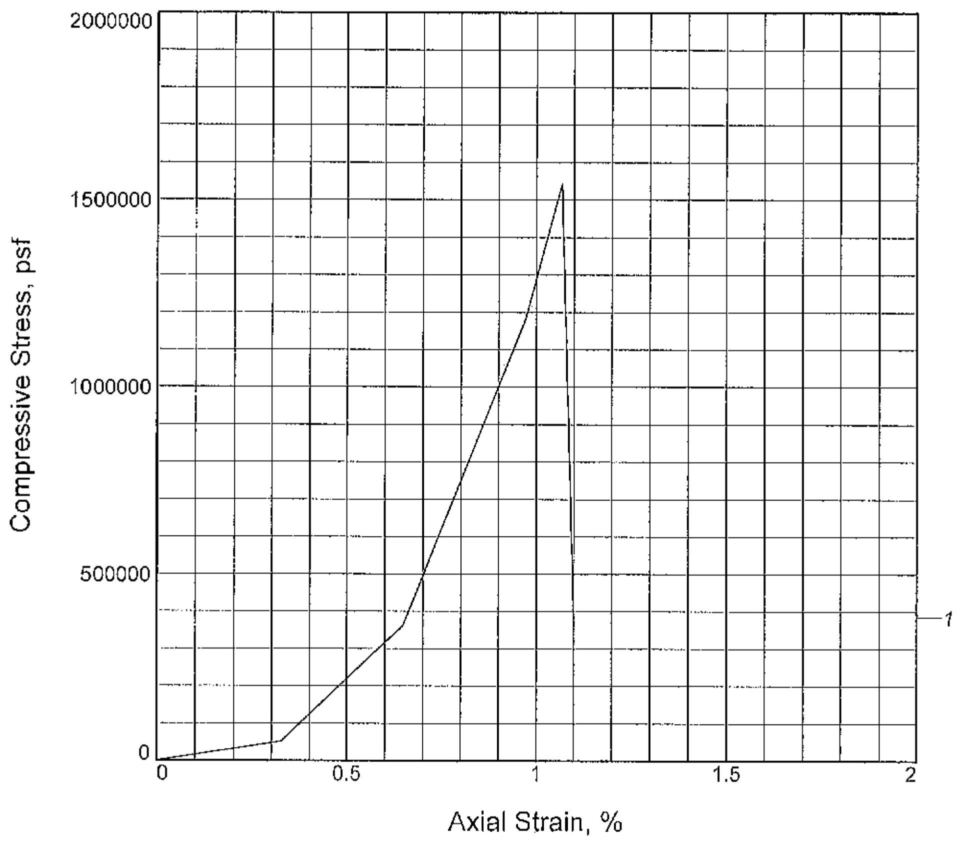
Project: BRENT SPENCE BRIDGE REPLACEMENT

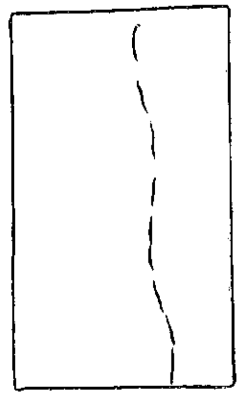
Source of Sample: L-2 **Depth:** 163.6-164'
Sample Number: 9/NQ

UNCONFINED COMPRESSION TEST
H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1542930.8			
Undrained shear strength, psf	771465.4			
Failure strain,	1.1			
Strain rate, in./min.	0.030			
Water content, %	1.1			
Wet density, pcf	163.8			
Dry density, pcf	162.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.010			
Specimen height, in.	3.090			
Height/diameter ratio	1.54			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date: 6-15-10
Remarks:
Lab No. 4893

Figure _______

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

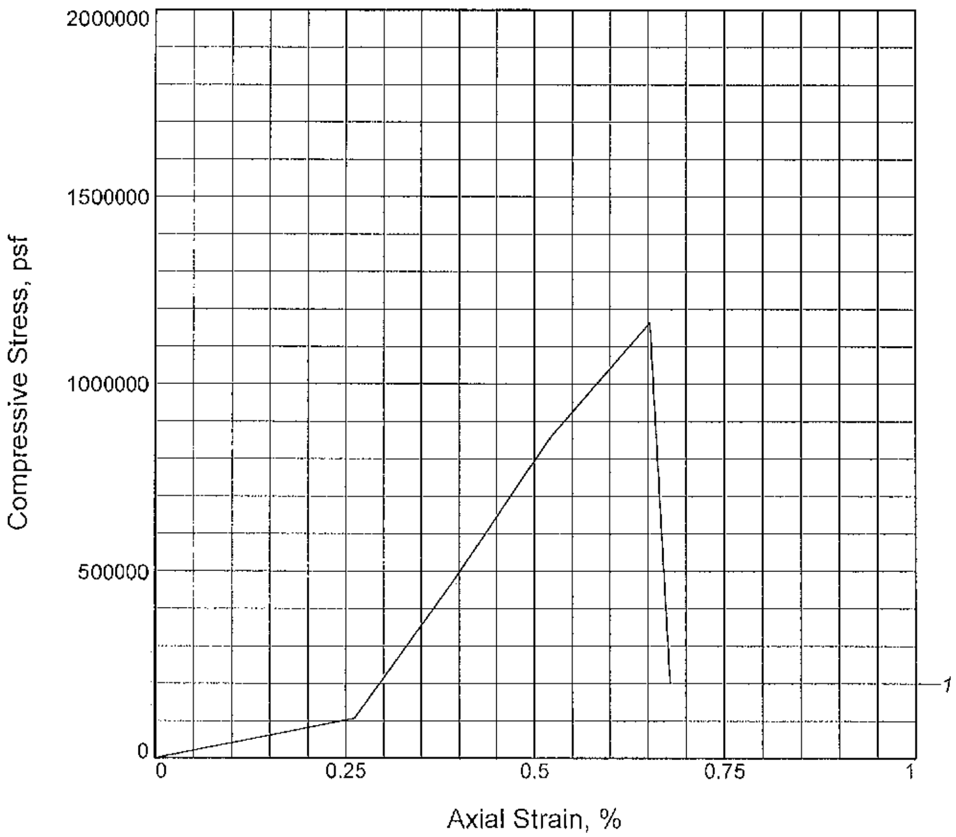
Source of Sample: L-2 **Depth:** 165.1-165.4'
Sample Number: 9/NQ

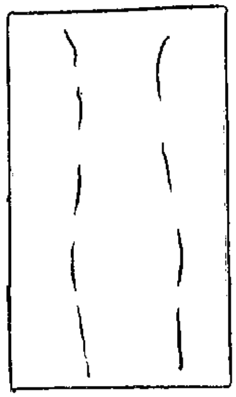
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1164152.5			
Undrained shear strength, psf	582076.2			
Failure strain,	0.7			
Strain rate, in./min.	0.038			
Water content, %	0.1			
Wet density, pcf	165.7			
Dry density, pcf	165.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.830			
Height/diameter ratio	1.93			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date: 7-28-10

Remarks:
Lab No. 5914

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-2A **Depth:** 130.1-130.5'

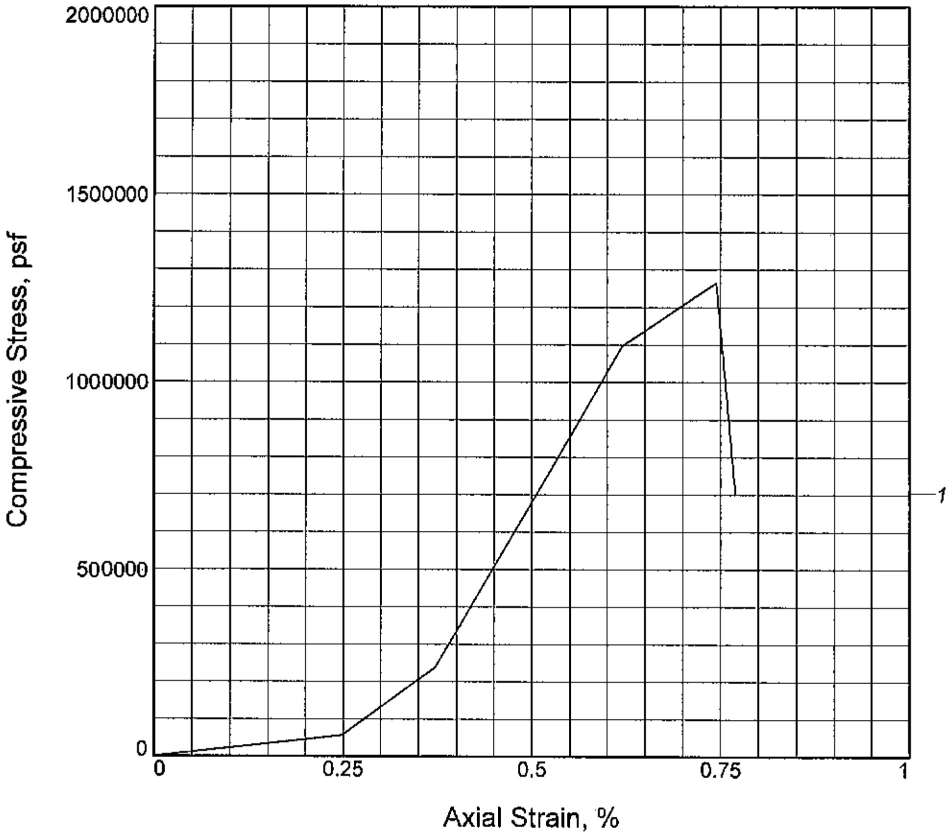
Sample Number: 1/NQ

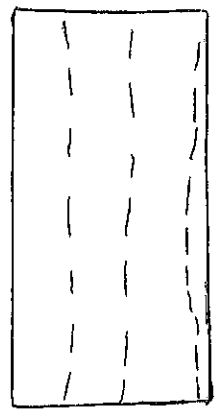
UNCONFINED COMPRESSION TEST

H. C. NUTTING COMPANY

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1264643.3			
Undrained shear strength, psf	632321.6			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	0.3			
Wet density, pcf	167.7			
Dry density, pcf	167.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.030			
Height/diameter ratio	2.04			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-28-10
Remarks:
lab No. 5915

Figure _______

Client: PARSONS BRINCKERHOFF

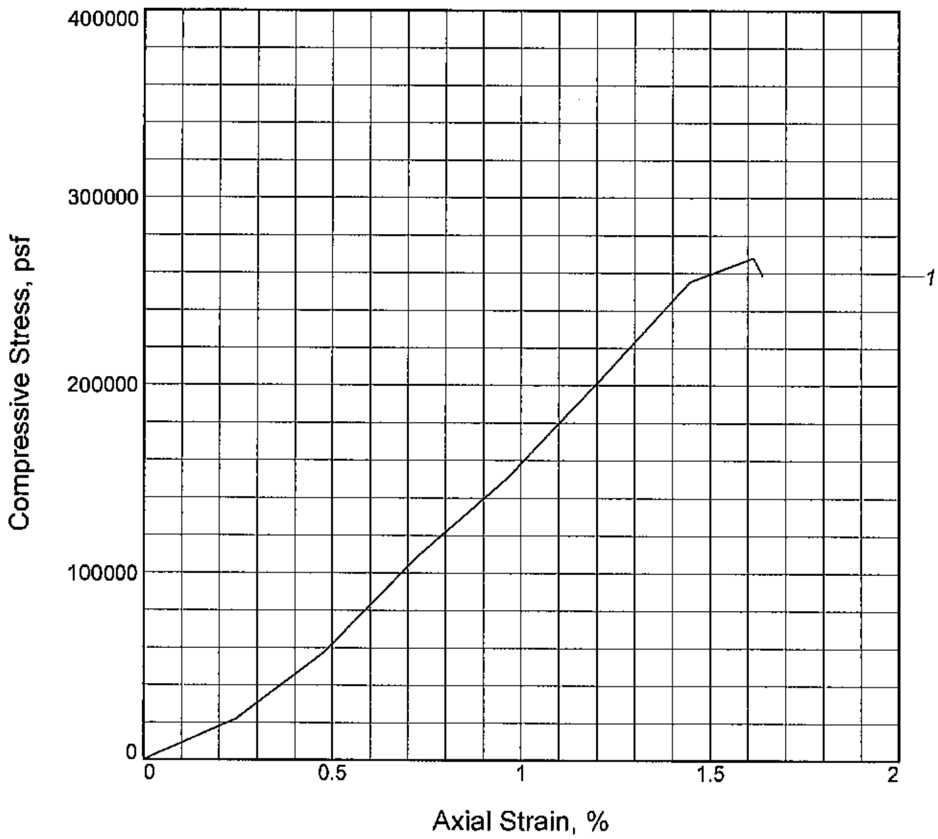
Project: BRENT SPENCE BRIDGE REPLACEMENT

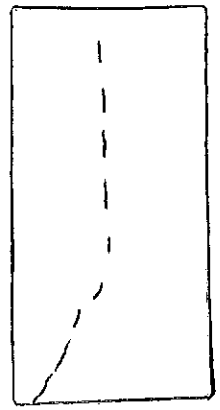
Source of Sample: L-2A **Depth:** 131.5-132.2'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	267975.4			
Undrained shear strength, psf	133987.7			
Failure strain, %	1.6			
Strain rate, in./min.	0.041			
Water content, %	4.5			
Wet density, pcf	158.4			
Dry density, pcf	151.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.150			
Height/diameter ratio	2.10			

Description: SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Shale

Project No.: N1105070
Date Sampled: 7-28-10
Remarks:
Lab No. 5916

Figure ________

Client: PARSONS BRINCKERHOFF

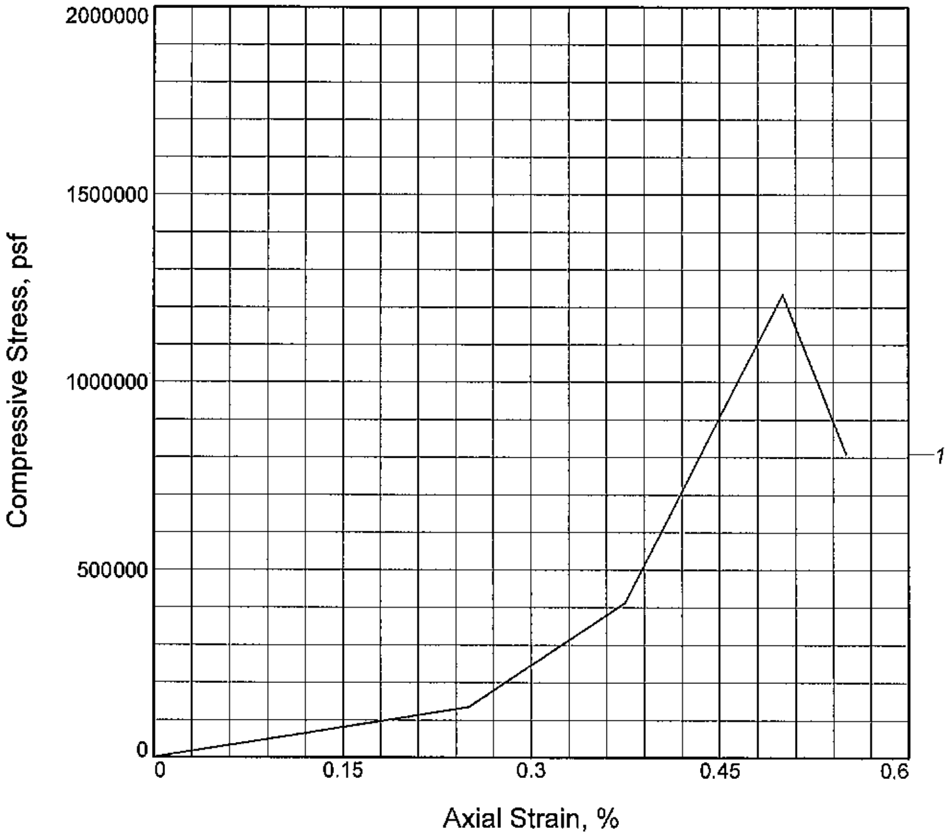
Project: BRENT SPENCE BRIDGE REPLACEMENT

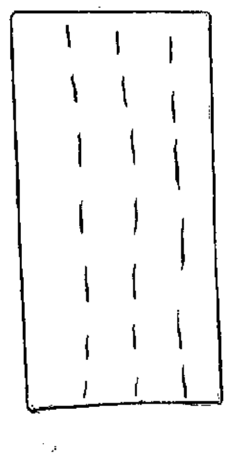
Source of Sample: L-2A **Depth:** 137-137.4'
Sample Number: 3/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1233462.3			
Undrained shear strength, psf	616731.2			
Failure strain, %	0.5			
Strain rate, in./min.	0.040			
Water content, %	0.7			
Wet density, pcf	167.6			
Dry density, pcf	166.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.000			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 5921
Remarks:
Lab No. 5921

Figure ________

Client: PARSONS BRINCKERHOFF

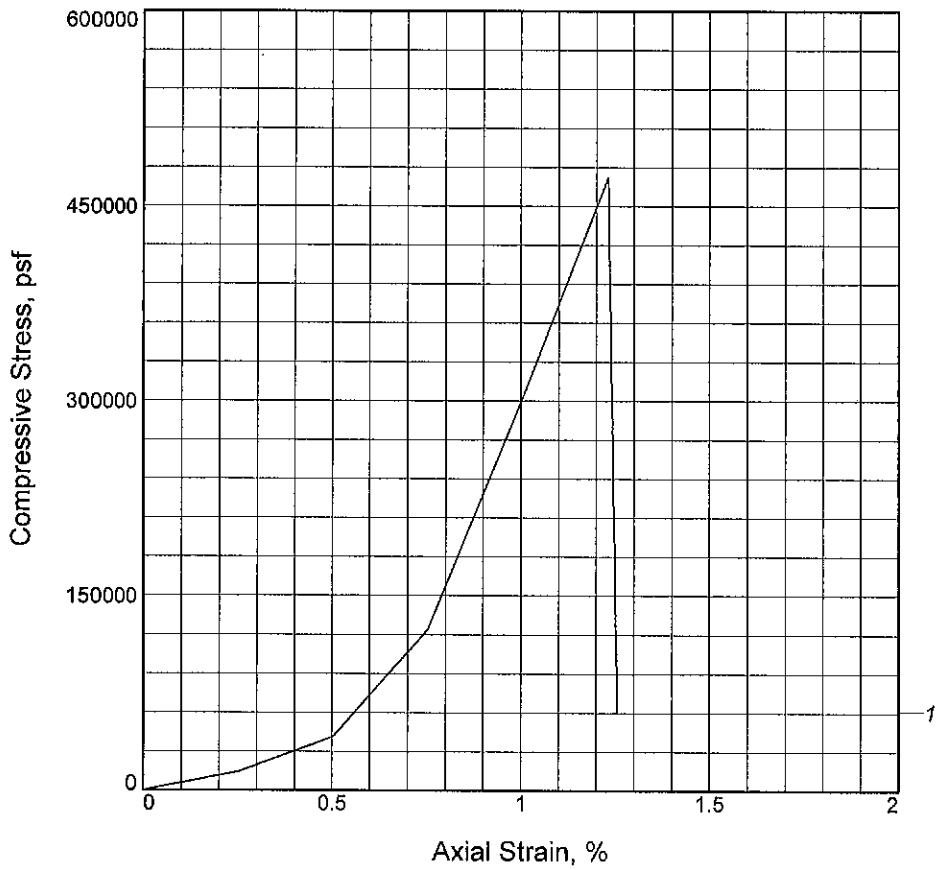
Project: BRENT SPENCE BRIDGE REPLACEMENT

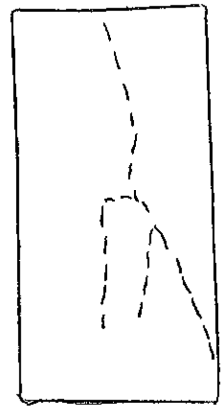
Source of Sample: L-2A **Depth:** 157.8-158.3'
Sample Number: 7/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	471917.2			
Undrained shear strength, psf	235958.6			
Failure strain, %	1.2			
Strain rate, in./min.	0.039			
Water content, %	1.9			
Wet density, pcf	164.0			
Dry density, pcf	161.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.980			
Height/diameter ratio	2.02			

Description: SHALE & LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Shale & Limestone

Project No.: N1105070

Date Sampled:

Remarks:

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-3 **Depth:** 97.6-98'

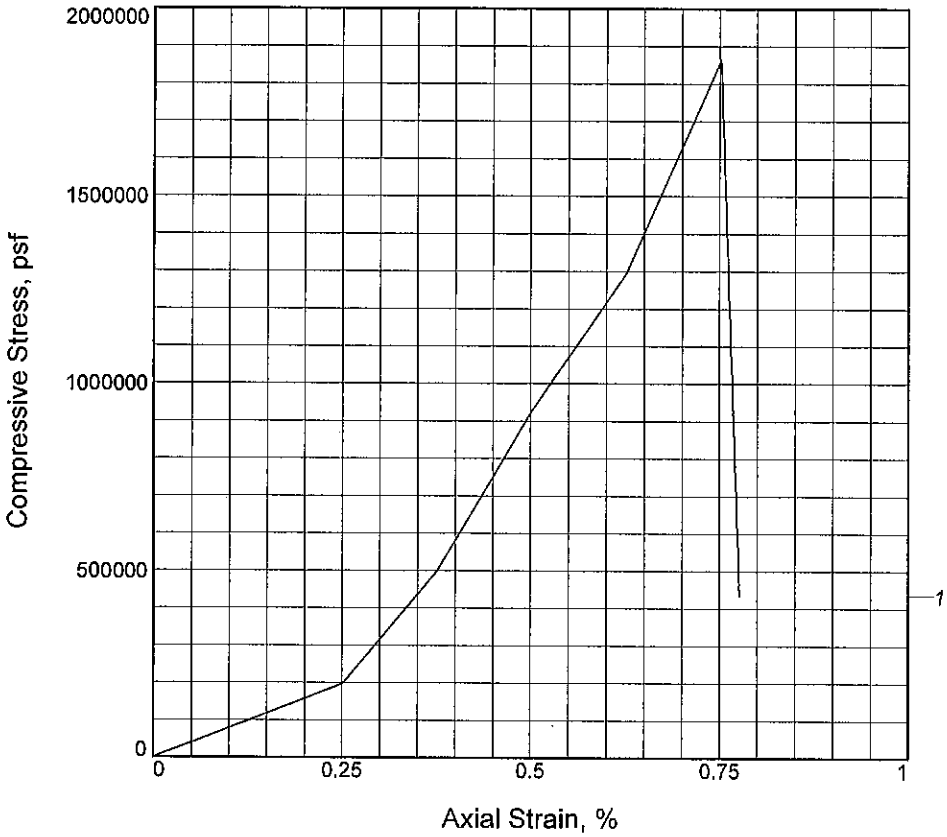
Sample Number: 2

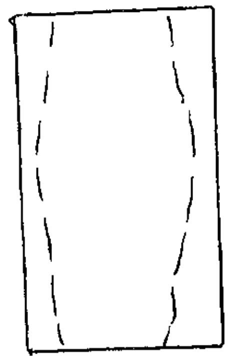
UNCONFINED COMPRESSION TEST

H.C. Nutting

A Terracon Company

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1863379.2			
Undrained shear strength, psf	931689.6			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.7			
Wet density, pcf	N/A			
Dry density, pcf	N/A			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.990			
Height/diameter ratio	2.03			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6029

Figure ________

Client: PARSONS BRINCKERHOFF

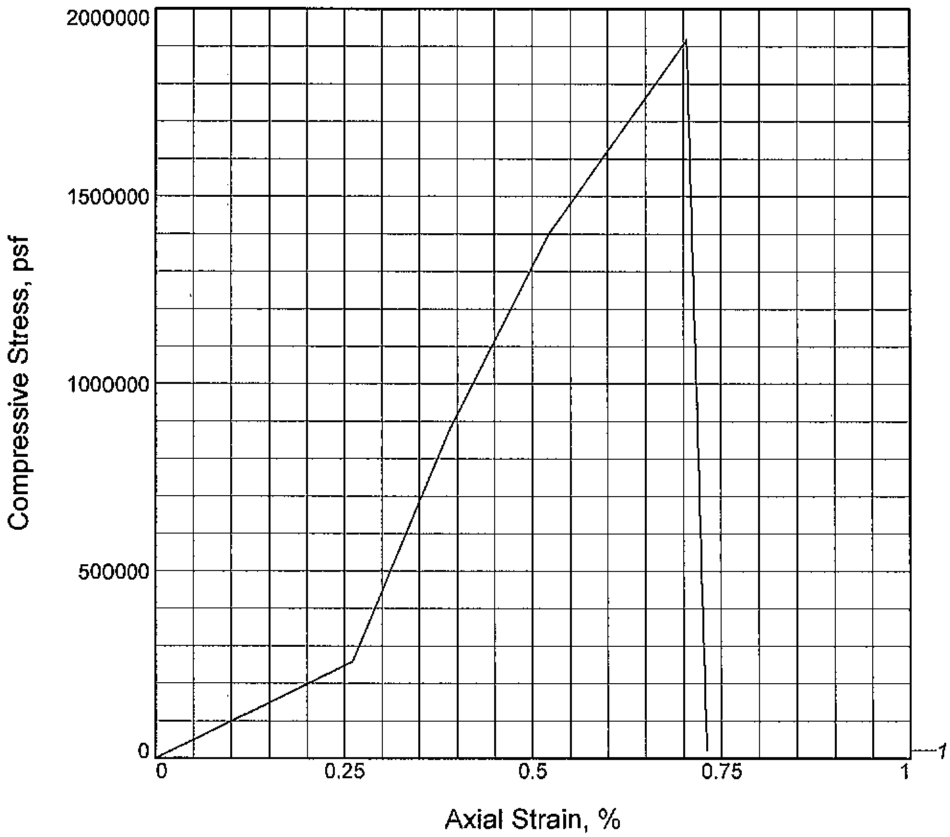
Project: BRENT SPENCE BRIDGE REPLACEMENT

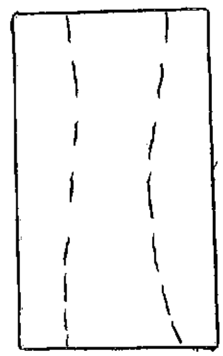
Source of Sample: L-3 **Depth:** 100.2-100.4'
Sample Number: 3

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1917187.9			
Undrained shear strength, psf	958594.0			
Failure strain, %	0.7			
Strain rate, in./min.	0.038			
Water content, %	1.4			
Wet density, pcf	168.3			
Dry density, pcf	165.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.830			
Height/diameter ratio	1.94			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6030

Figure ________

Client: PARSONS BRINCKERHOFF

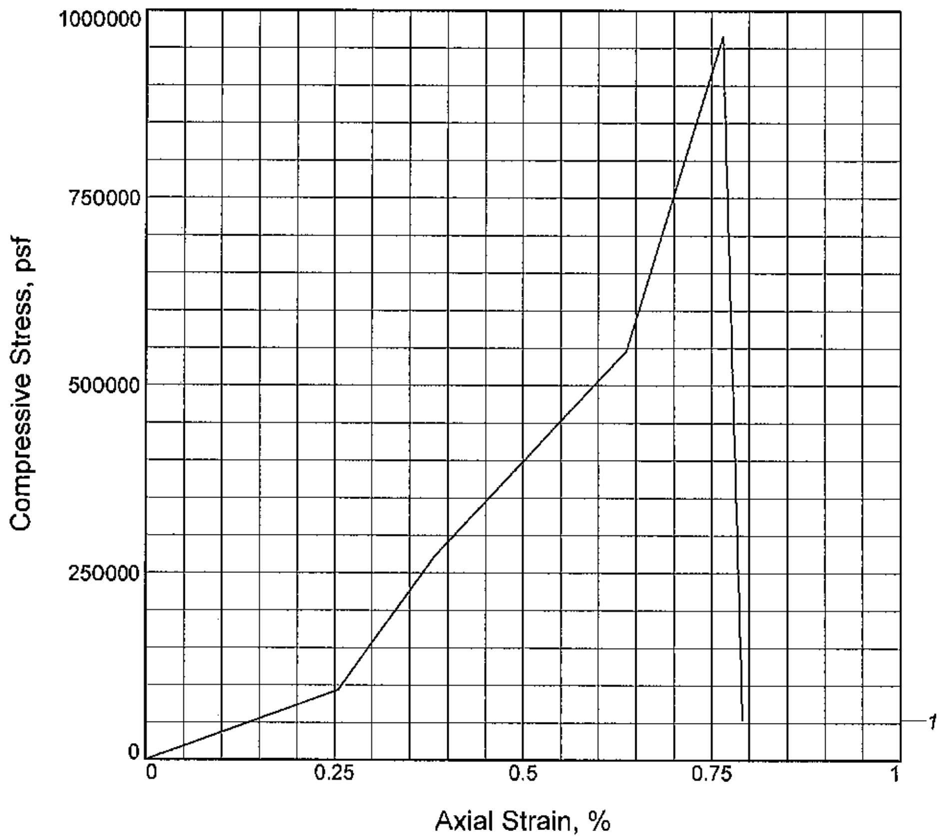
Project: BRENT SPENCE BRIDGE REPLACEMENT

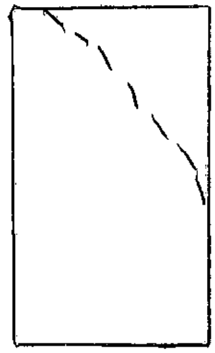
Source of Sample: L-3 **Depth:** 103.8-104.4'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	965311.3			
Undrained shear strength, psf	482655.6			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	1.3			
Wet density, pcf	165.8			
Dry density, pcf	163.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.920			
Height/diameter ratio	1.98			

Description: LIMESTONE & SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone & Shale

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6033

Figure ________

Client: PARSONS BRINCKERHOFF

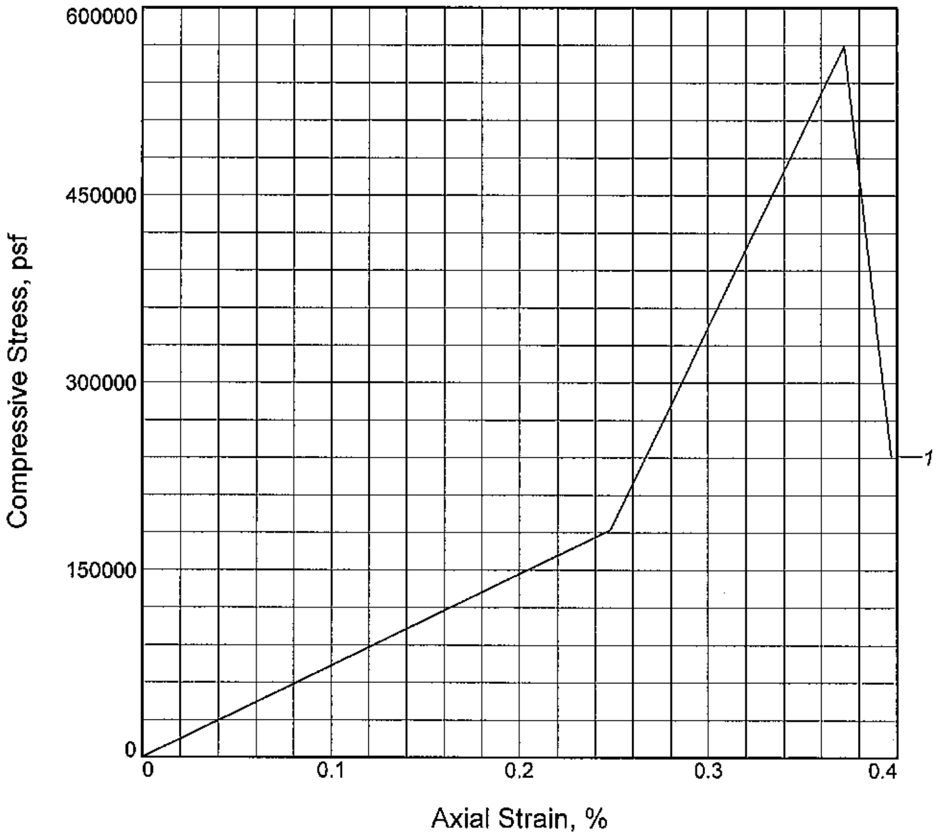
Project: BRENT SPENCE BRIDGE REPLACEMENT

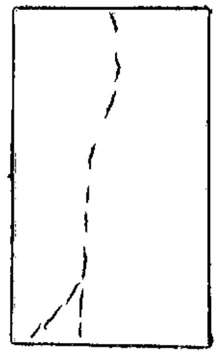
Source of Sample: L-3 **Depth:** 121.2-121.8'
Sample Number: 7

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	569305.4			
Undrained shear strength, psf	284652.7			
Failure strain, %	0.4			
Strain rate, in./min.	0.040			
Water content, %	1.2			
Wet density, pcf	166.8			
Dry density, pcf	164.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	4.030			
Height/diameter ratio	2.04			

Description: LIMESTONE W/SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone w/Shale

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6034

Figure _______

Client: PARSONS BRINCKERHOFF

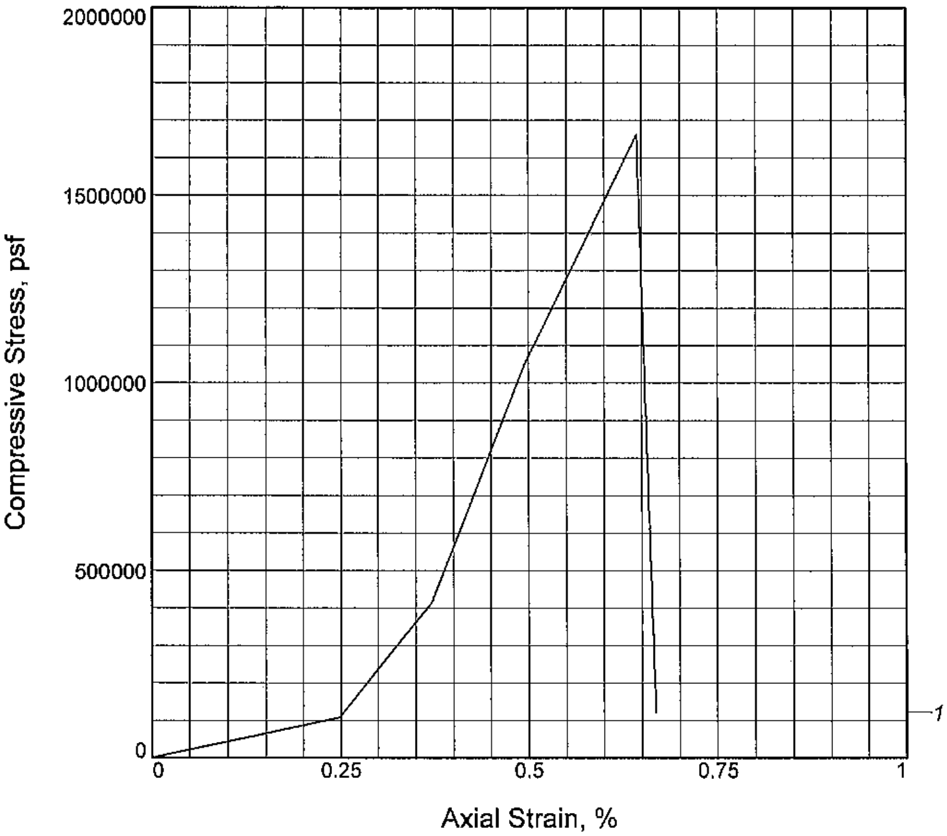
Project: BRENT SPENCE BRIDGE REPLACEMENT

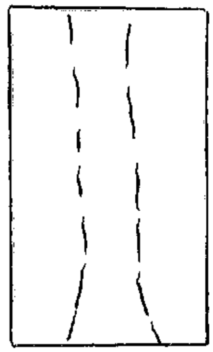
Source of Sample: L-3 **Depth:** 124.6-125.2'
Sample Number: 8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1661279.5			
Undrained shear strength, psf	830639.7			
Failure strain, %	0.6			
Strain rate, in./min.	0.040			
Water content, %	0.7			
Wet density, pcf	166.7			
Dry density, pcf	165.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	4.040			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6038

Figure ________

Client: PARSONS BRINCKERHOFF

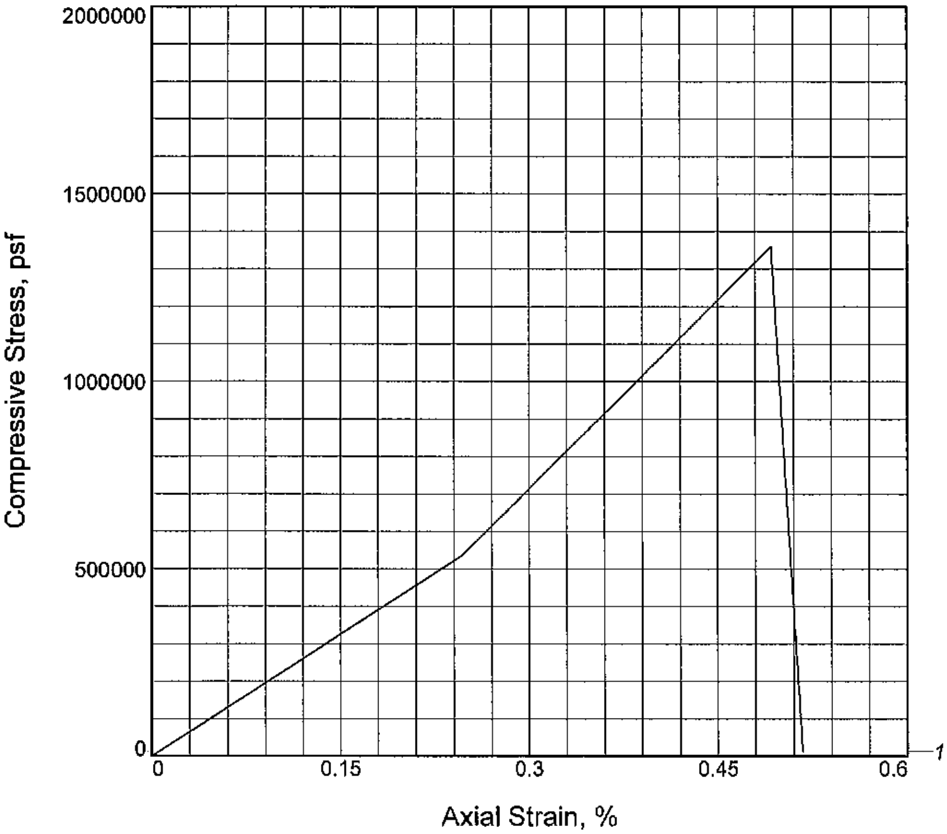
Project: BRENT SPENCE BRIDGE REPLACEMENT

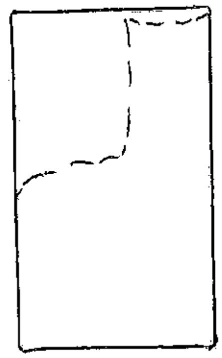
Source of Sample: L-3 **Depth:** 145.2-146.2'
Sample Number: 12

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1358505.5			
Undrained shear strength, psf	679252.8			
Failure strain, %	0.5			
Strain rate, in./min.	0.040			
Water content, %	1.5			
Wet density, pcf	165.7			
Dry density, pcf	163.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.060			
Height/diameter ratio	2.05			

Description: LIMESTONE W/SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone w/Shale

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6037

Figure ________

Client: PARSONS BRINCKERHOFF

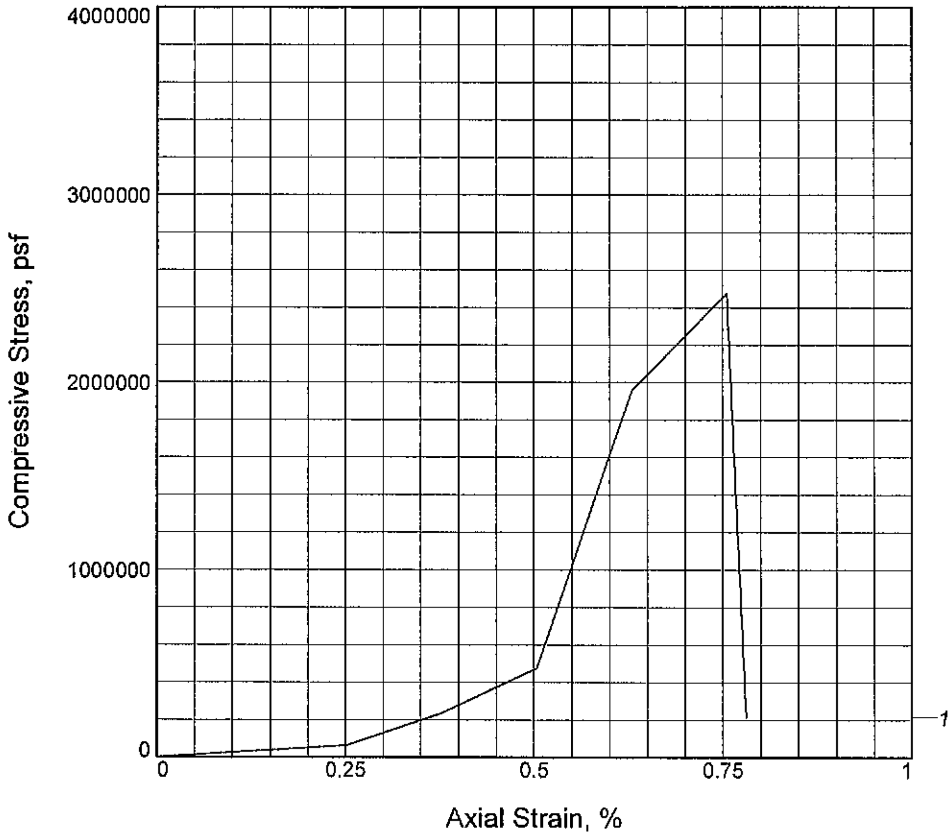
Project: BRENT SPENCE BRIDGE REPLACEMENT

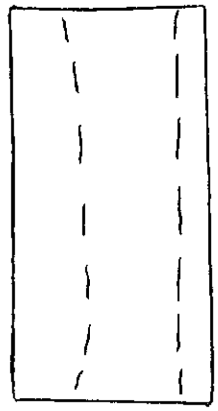
Source of Sample: L-3 **Depth:** 145.6-146.1'
Sample Number: 11

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2475252.6			
Undrained shear strength, psf	1237626.3			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.4			
Wet density, pcf	165.7			
Dry density, pcf	165.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.970			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No.6040

Figure ________

Client: PARSONS BRINCKERHOFF

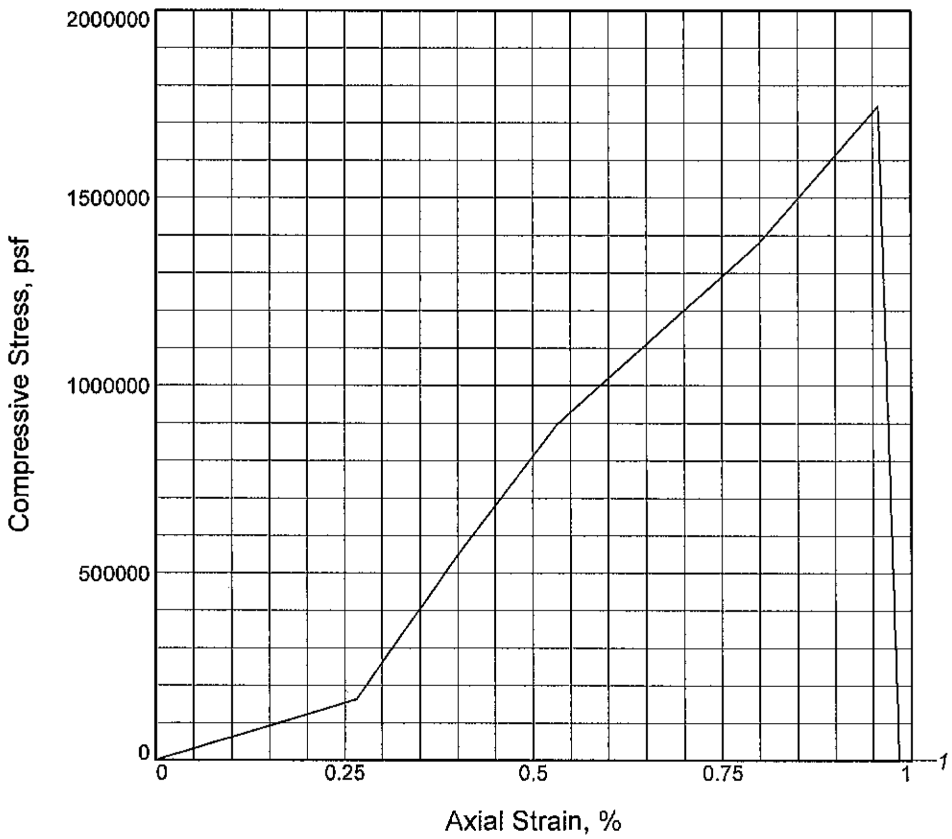
Project: BRENT SPENCE BRIDGE REPLACEMENT

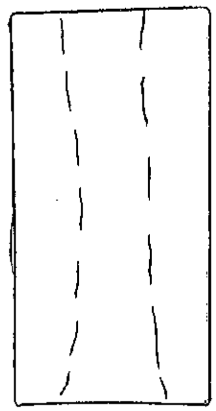
Source of Sample: L-3 **Depth:** 158.7-160.2'
Sample Number: 14

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1744346.8			
Undrained shear strength, psf	872173.4			
Failure strain, %	1.0			
Strain rate, in./min.	0.037			
Water content, %	0.2			
Wet density, pcf	165.1			
Dry density, pcf	164.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.760			
Height/diameter ratio	1.90			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6041

Figure ______

Client: PARSONS BRINCKERHOFF

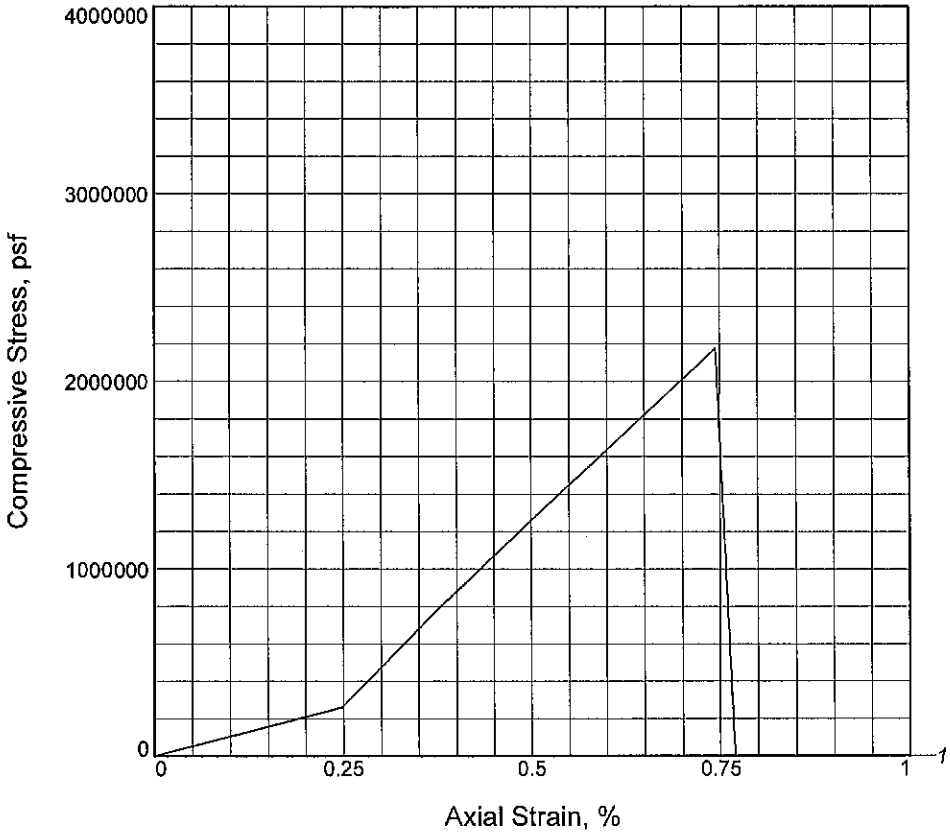
Project: BRENT SPENCE BRIDGE REPLACEMENT

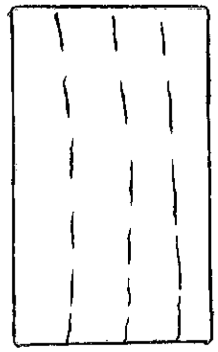
Source of Sample: L-3 **Depth:** 162.8-163.3'
Sample Number: 15

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2176614.5			
Undrained shear strength, psf	1088307.3			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	0.8			
Wet density, pcf	164.7			
Dry density, pcf	163.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.030			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6042

Figure ________

Client: PARSONS BRINCKERHOFF

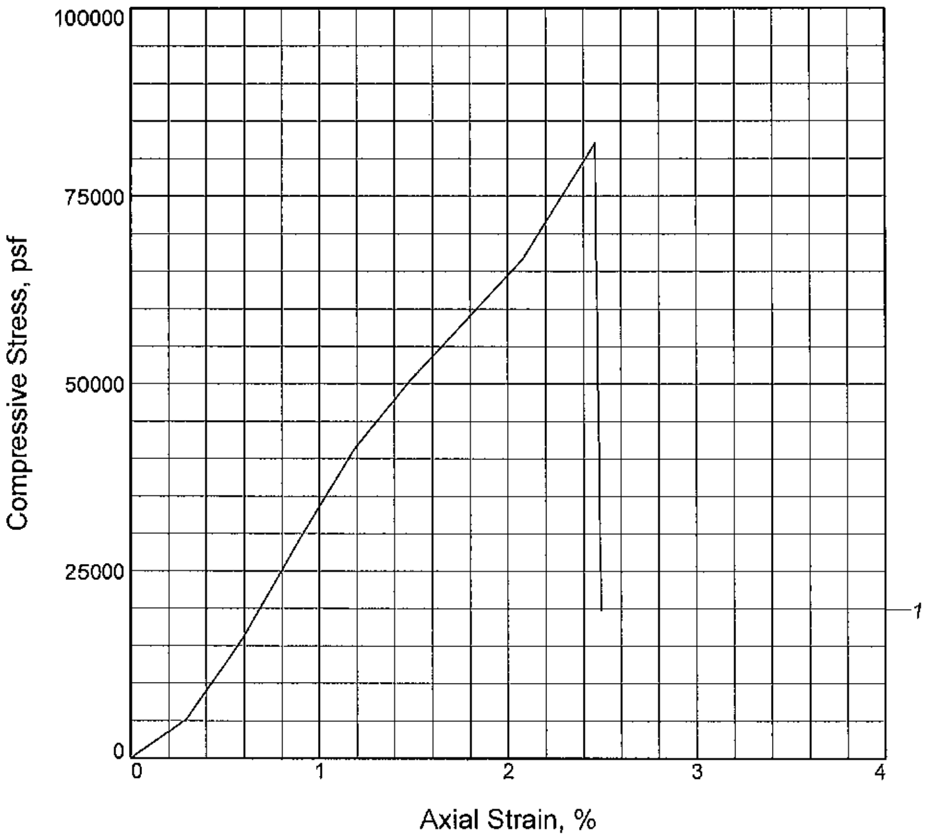
Project: BRENT SPENCE BRIDGE REPLACEMENT

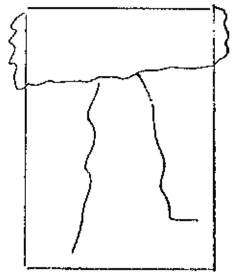
Source of Sample: L-3 **Depth:** 164.5-165.2'
Sample Number: 16

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	82023.1			
Undrained shear strength, psf	41011.5			
Failure strain, %	2.5			
Strain rate, in./min.	0.033			
Water content, %	6.3			
Wet density, pcf	153.9			
Dry density, pcf	144.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.960			
Specimen height, in.	3.370			
Height/diameter ratio	1.72			

Description: GRAY MED TOUGH SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070

Date Sampled: 5-27-10

Remarks:
Lab No. 4190

Figure _______

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-3A **Depth:** 126.5-126.75'

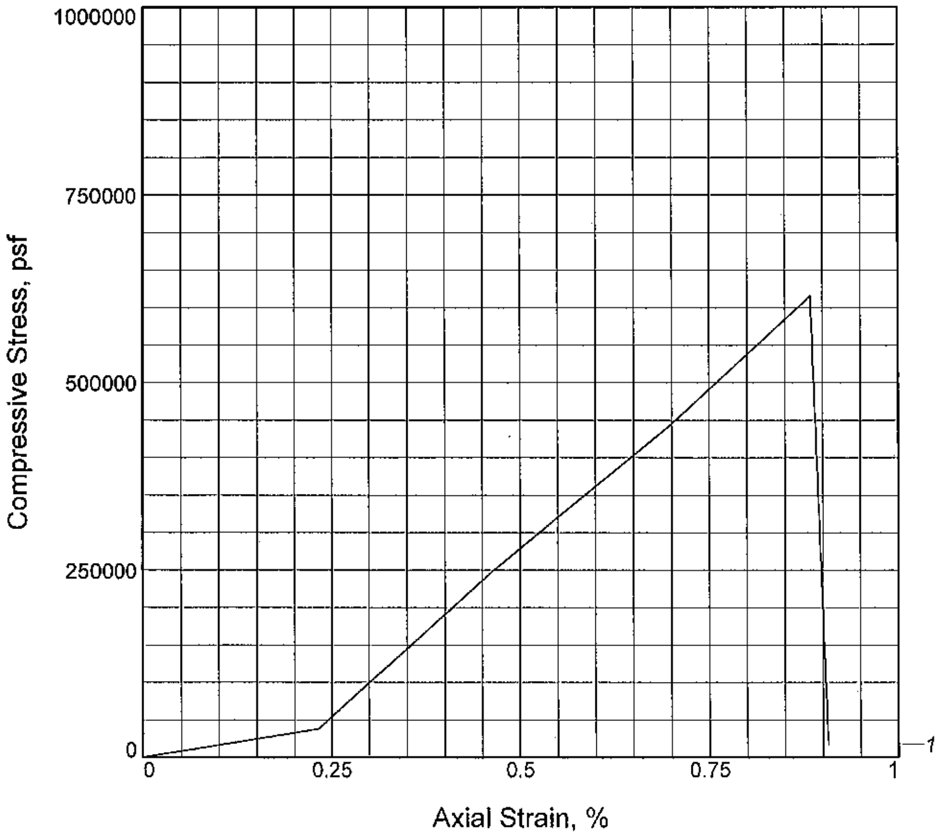
Sample Number: 3A-1

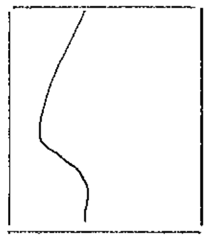
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	615192.0			
Undrained shear strength, psf	307596.0			
Failure strain, %	0.9			
Strain rate, in./min.	0.043			
Water content, %	2.0			
Wet density, pcf	164.2			
Dry density, pcf	161.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	4.300			
Height/diameter ratio	2.16			

Description: GRAY TOUGH SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070

Date Sampled: 5-27-10

Remarks:
Lab No. 4193

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: L-3A **Depth:** 142.25-142.5'

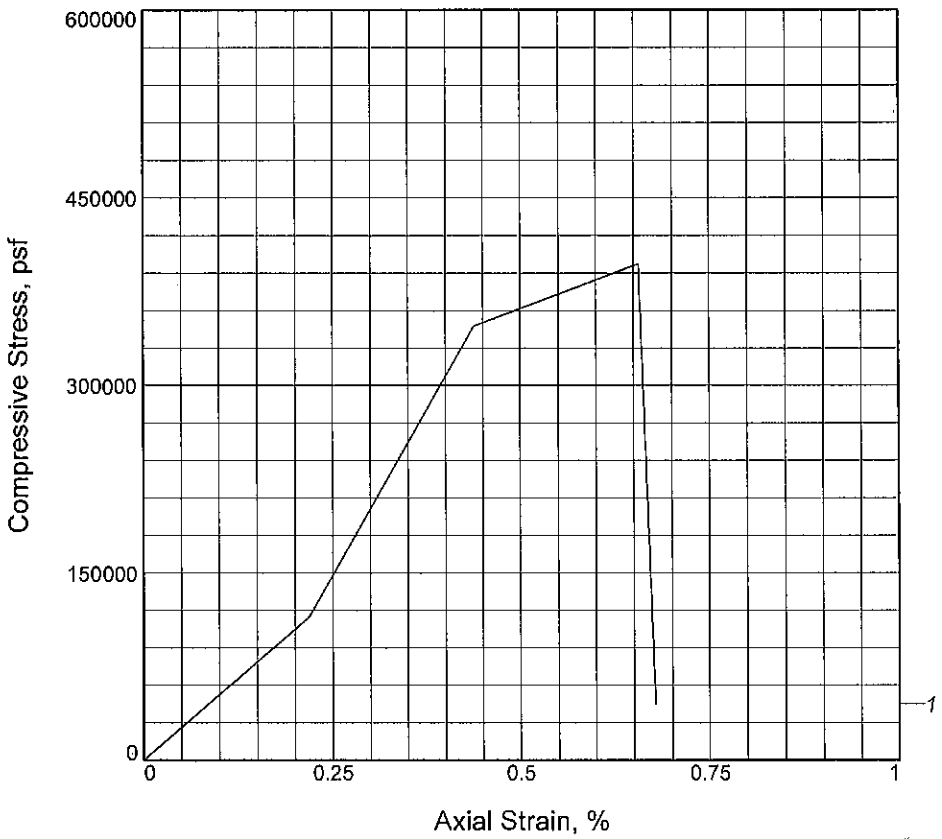
Sample Number: 3A-4

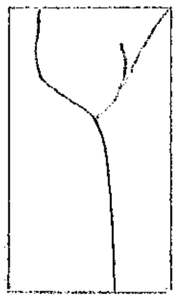
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	397254.7			
Undrained shear strength, psf	198627.3			
Failure strain, %	0.7			
Strain rate, in./min.	0.045			
Water content, %	0.8			
Wet density, pcf	165.2			
Dry density, pcf	163.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	4.570			
Height/diameter ratio	2.30			

Description: GRAY SHALE & LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 5-27-10
Remarks:
Lab No. 4197

Figure ________

Client: PARSONS BRINCKERHOFF

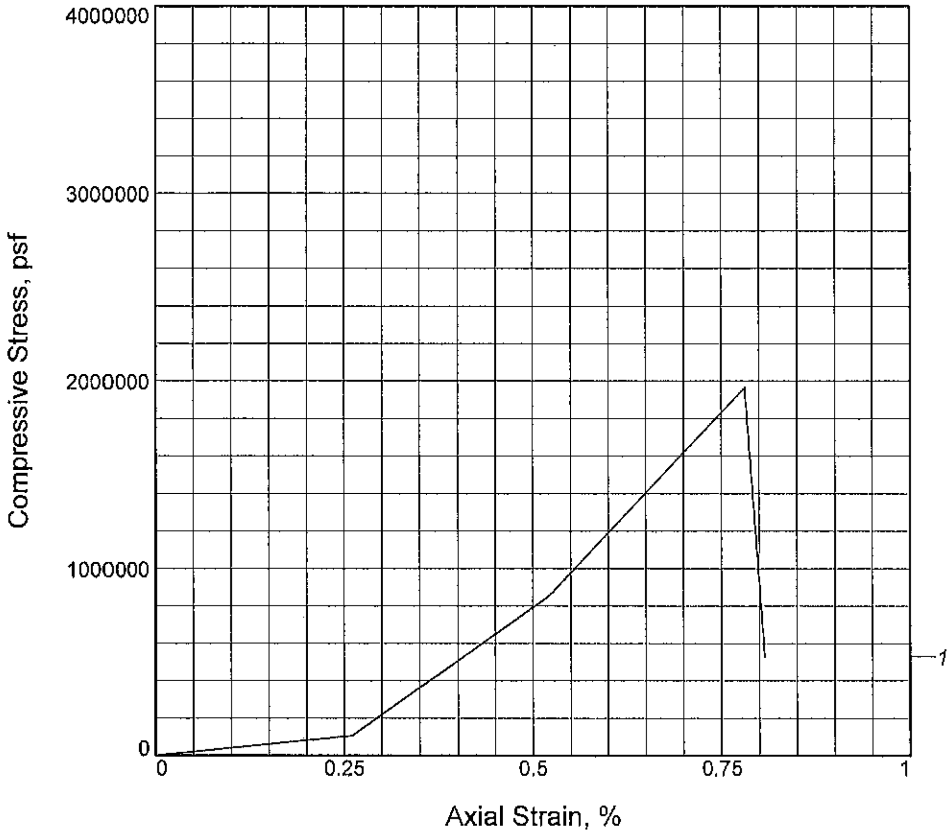
Project: BRENT SPENCE BRIDGE REPLACEMENT

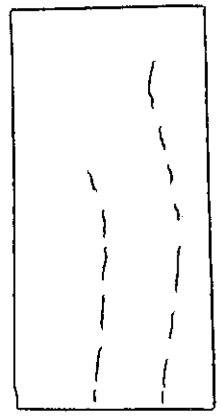
Source of Sample: L-3A **Depth:** 157.7-158.0'
Sample Number: 3A-8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1965081.6			
Undrained shear strength, psf	982540.8			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	0.5			
Wet density, pcf	163.5			
Dry density, pcf	162.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.840			
Height/diameter ratio	1.92			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 5987

Figure ________

Client: PARSONS BRINCKERHOFF

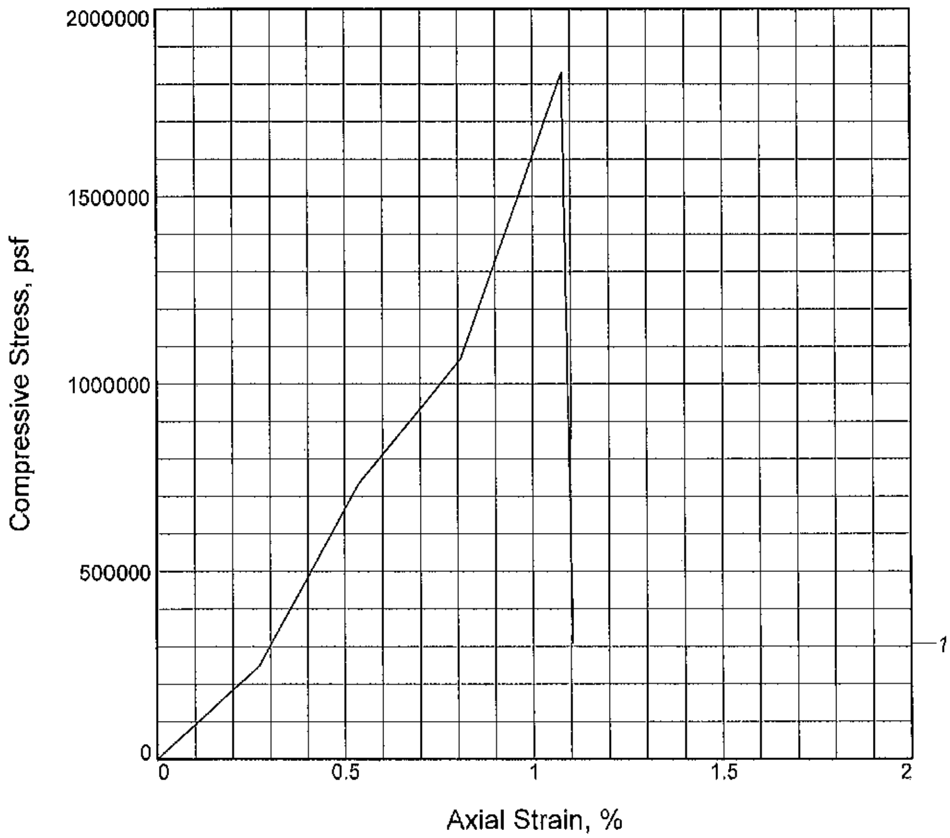
Project: BRENT SPENCE BRIDGE REPLACEMENT

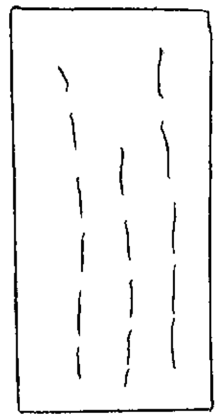
Source of Sample: L-4 **Depth:** 116-116.5'
Sample Number: 2

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1829521.6			
Undrained shear strength, psf	914760.8			
Failure strain, %	1.1			
Strain rate, in./min.	0.037			
Water content, %	1.1			
Wet density, pcf	165.1			
Dry density, pcf	163.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.710			
Height/diameter ratio	1.85			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 5988

Figure ________

Client: PARSONS BRINCKERHOFF

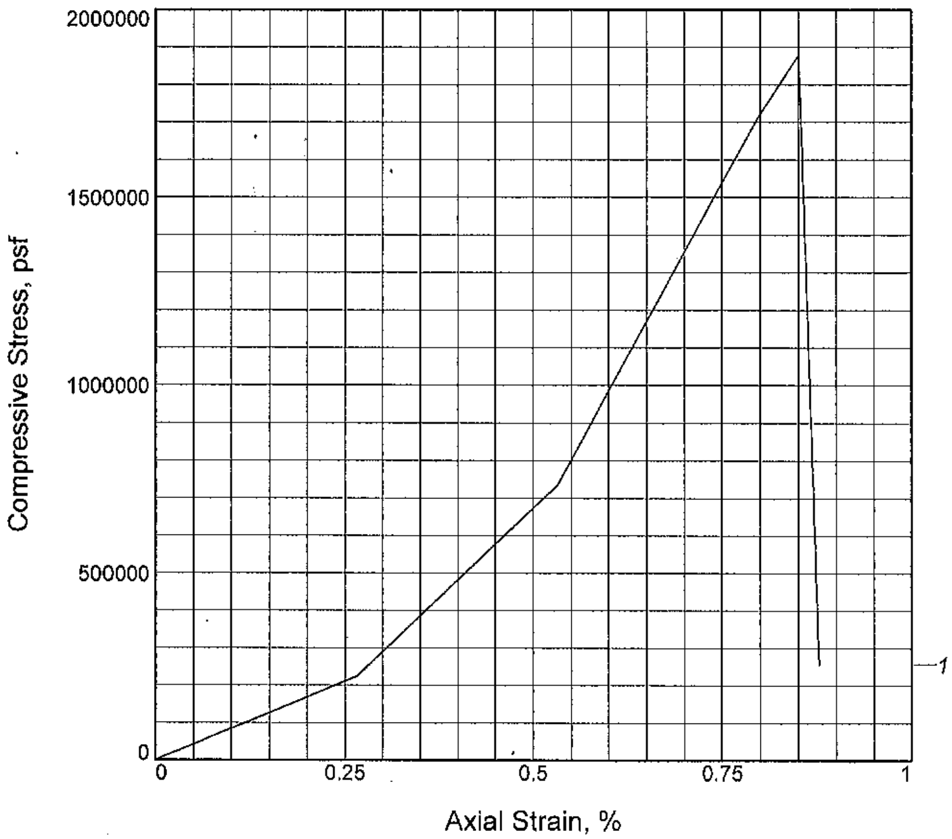
Project: BRENT SPENCE BRIDGE REPLACEMENT

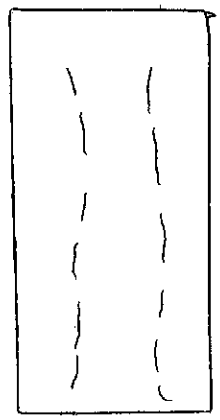
Source of Sample: L-4 **Depth:** 120.4-120.9'
Sample Number: 3

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1880122.7			
Undrained shear strength, psf	940061.4			
Failure strain, %	0.9			
Strain rate, in./min.	0.037			
Water content, %	0.8			
Wet density, pcf	166.2			
Dry density, pcf	164.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.760			
Height/diameter ratio	1.88			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 5991

Figure ________

Client: PARSONS BRINCKERHOFF

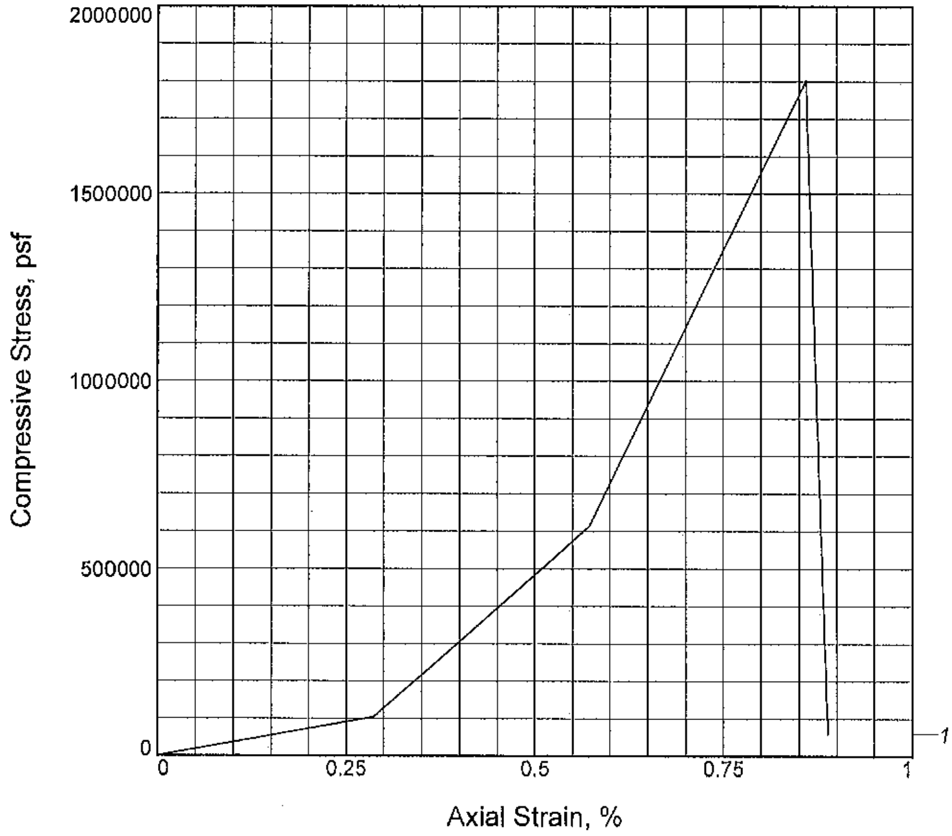
Project: BRENT SPENCE BRIDGE REPLACEMENT

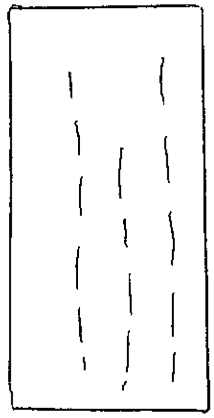
Source of Sample: L-4 **Depth:** 140.5-141'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1801226.2			
Undrained shear strength, psf	900613.1			
Failure strain, %	0.9			
Strain rate, in./min.	0.034			
Water content, %	0.4			
Wet density, pcf	165.7			
Dry density, pcf	165.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.490			
Height/diameter ratio	1.75			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 5993

Figure ________

Client: PARSONS BRINCKERHOFF

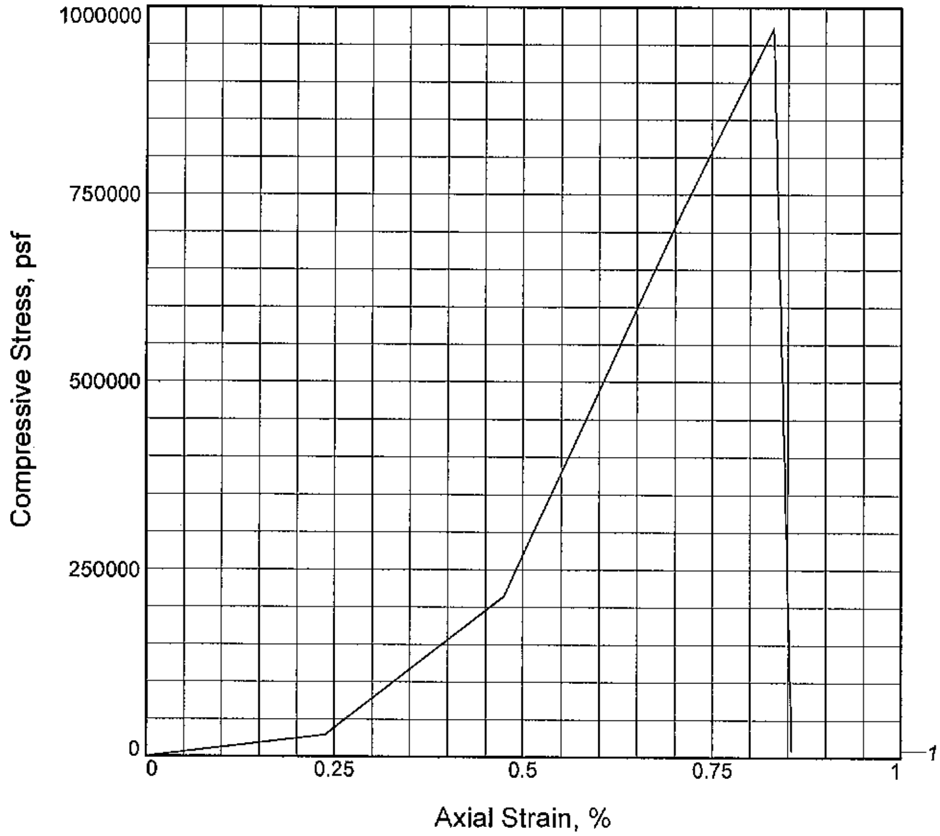
Project: BRENT SPENCE BRIDGE REPLACEMENT

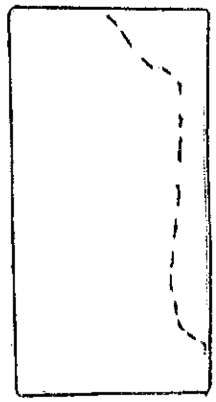
Source of Sample: L-4 **Depth:** 143-143.5'
Sample Number: 8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	972696.1			
Undrained shear strength, psf	486348.0			
Failure strain, %	0.8			
Strain rate, in./min.	0.042			
Water content, %	2.4			
Wet density, pcf	166.5			
Dry density, pcf	162.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.210			
Height/diameter ratio	2.13			

Description: GRAY LIMESTONE W/SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone w/shale

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5574

Figure ________

Client: PARSONS BRINCKERHOFF

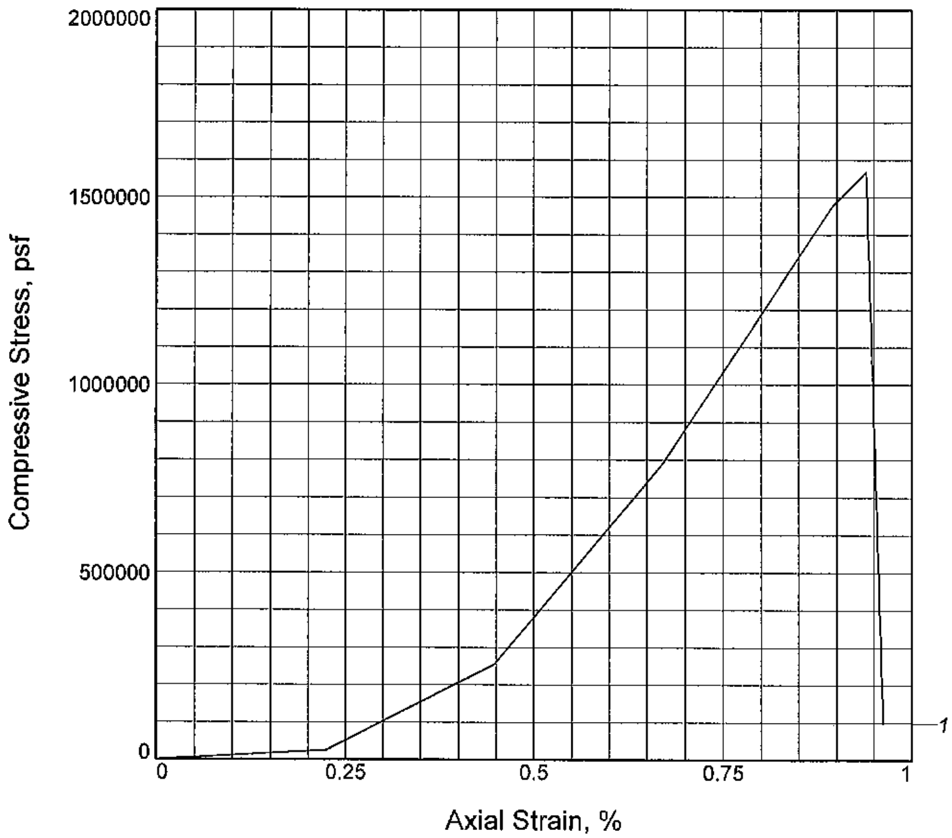
Project: BRENT SPENCE BRIDGE REPLACEMENT

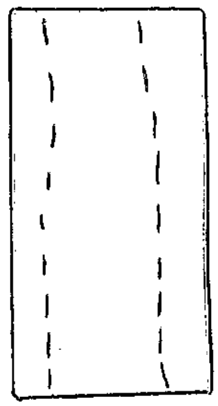
Source of Sample: L-5 **Depth:** 113.5-114'
Sample Number: 2

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1567920.4			
Undrained shear strength, psf	783960.2			
Failure strain, %	0.9			
Strain rate, in./min.	0.044			
Water content, %	0.2			
Wet density, pcf	167.4			
Dry density, pcf	167.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.470			
Height/diameter ratio	2.26			

Description: GRAY LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-7-10

Remarks:
Lab No. 5576

Figure ______

Client: PARSONS BRINCKERHOFF

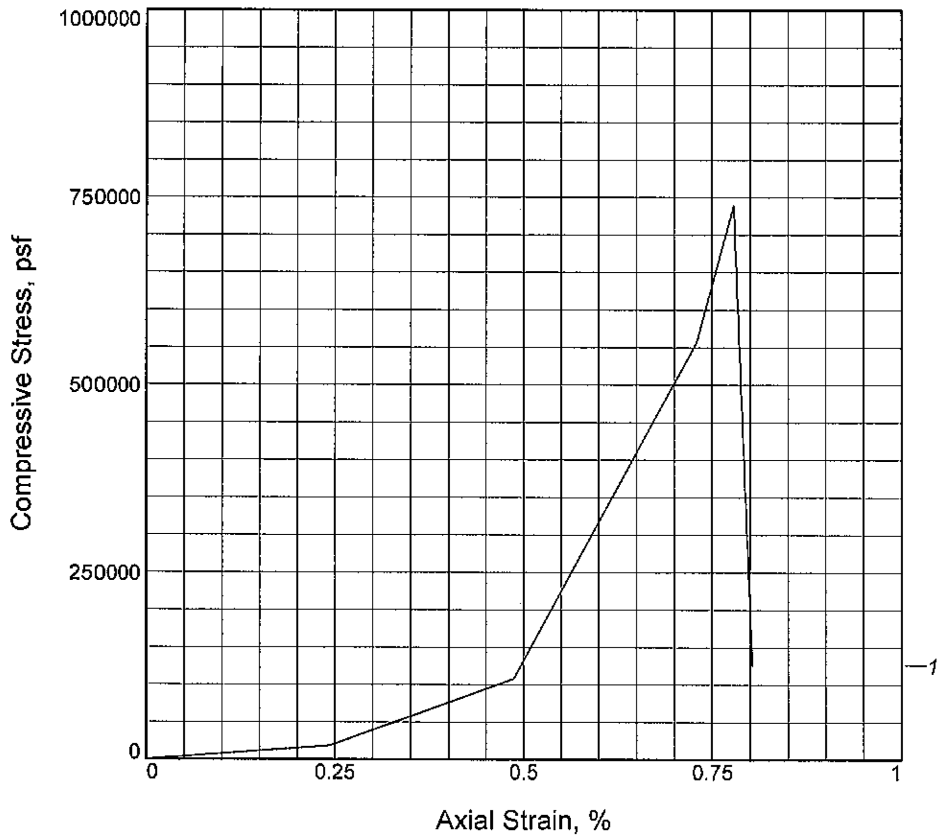
Project: BRENT SPENCE BRIDGE REPLACEMENT

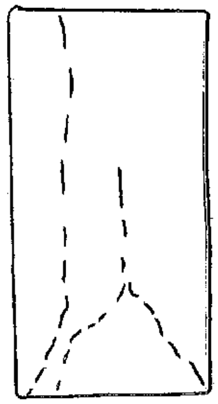
Source of Sample: L-5 **Depth:** 120.2-120.6'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	738738.4			
Undrained shear strength, psf	369369.2			
Failure strain, %	0.8			
Strain rate, in./min.	0.041			
Water content, %	0.3			
Wet density, pcf	166.2			
Dry density, pcf	165.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.110			
Height/diameter ratio	2.08			

Description: GRAY LIMESTONE W/SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone w/shale

Project No.: N1105070
Date Sampled: 7-7-10

Remarks:
Lab No. 5578

Figure _______

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

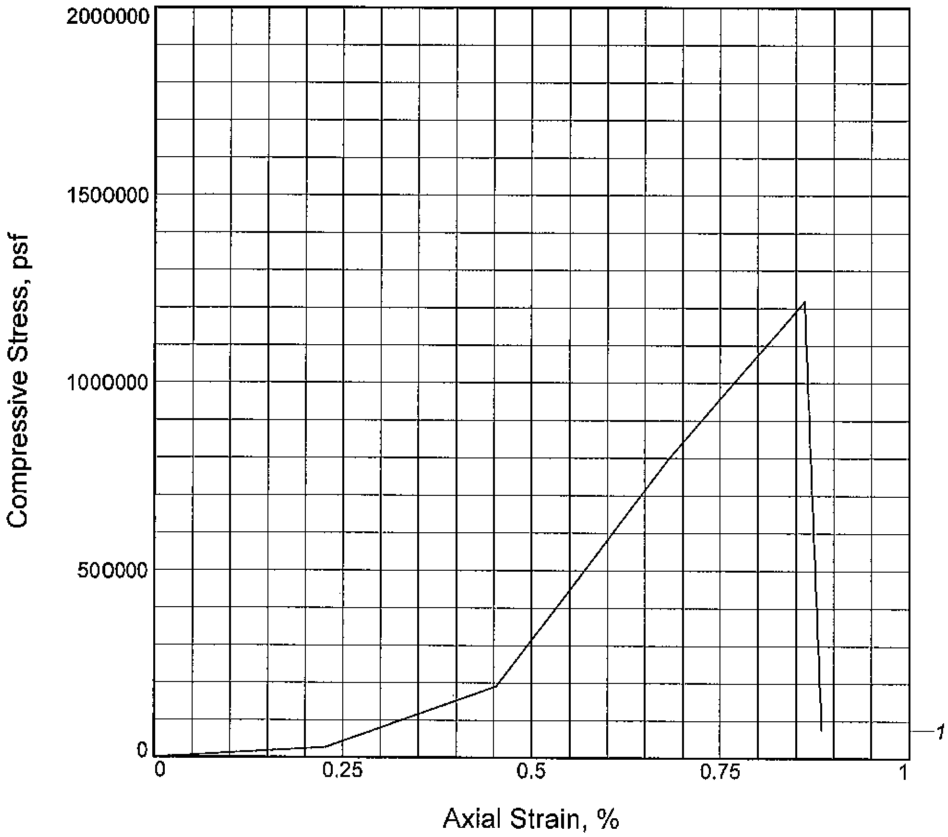
Source of Sample: L-5 **Depth:** 130.3-131'
Sample Number: 6

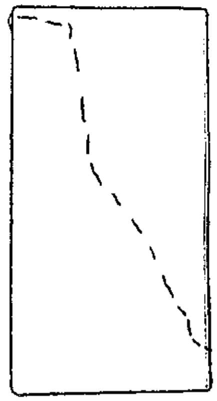
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1217480.4			
Undrained shear strength, psf	608740.2			
Failure strain, %	0.9			
Strain rate, in./min.	0.044			
Water content, %	1.7			
Wet density, pcf	167.1			
Dry density, pcf	164.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.410			
Height/diameter ratio	2.23			

Description: GRAY LIMESTONE W/SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone w/shale

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5579

Figure ________

Client: PARSONS BRINCKERHOFF

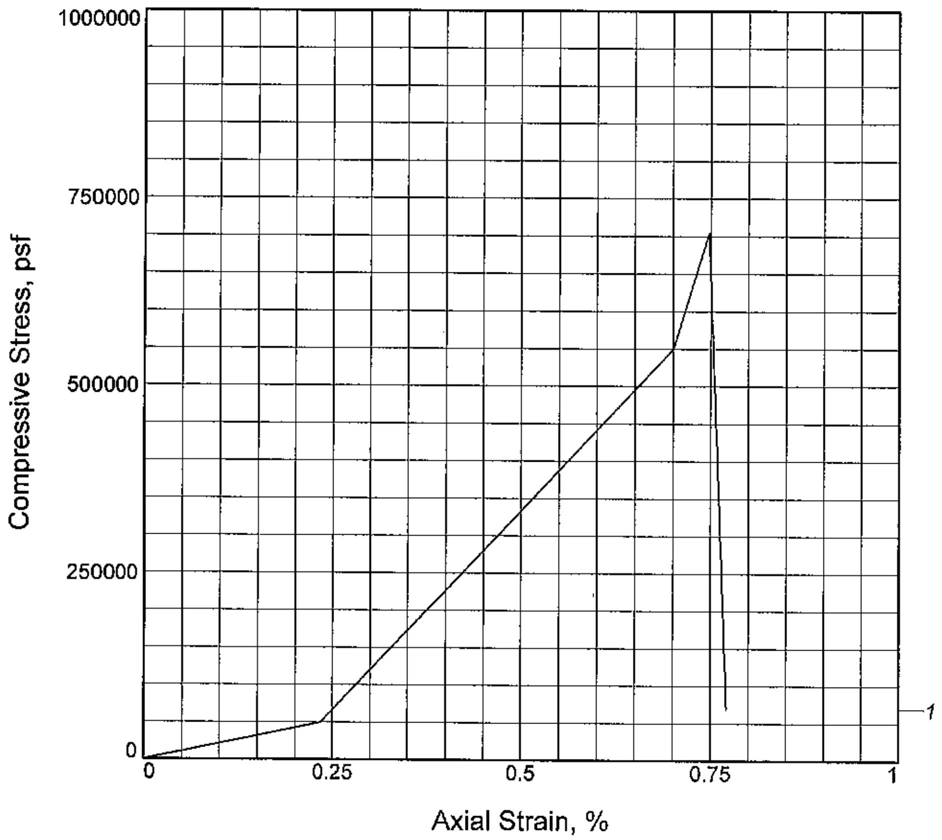
Project: BRENT SPENCE BRIDGE REPLACEMENT

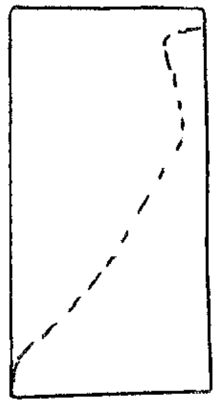
Source of Sample: L-5 **Depth:** 133.3-133.8'
Sample Number: 7

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	703969.8			
Undrained shear strength, psf	351984.9			
Failure strain, %	0.7			
Strain rate, in./min.	0.042			
Water content, %	0.3			
Wet density, pcf	167.2			
Dry density, pcf	166.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.280			
Height/diameter ratio	2.16			

Description: GRAY LIMESTONE AND SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone and Shale

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5557

Figure ________

Client: PARSONS BRINCKERHOFF

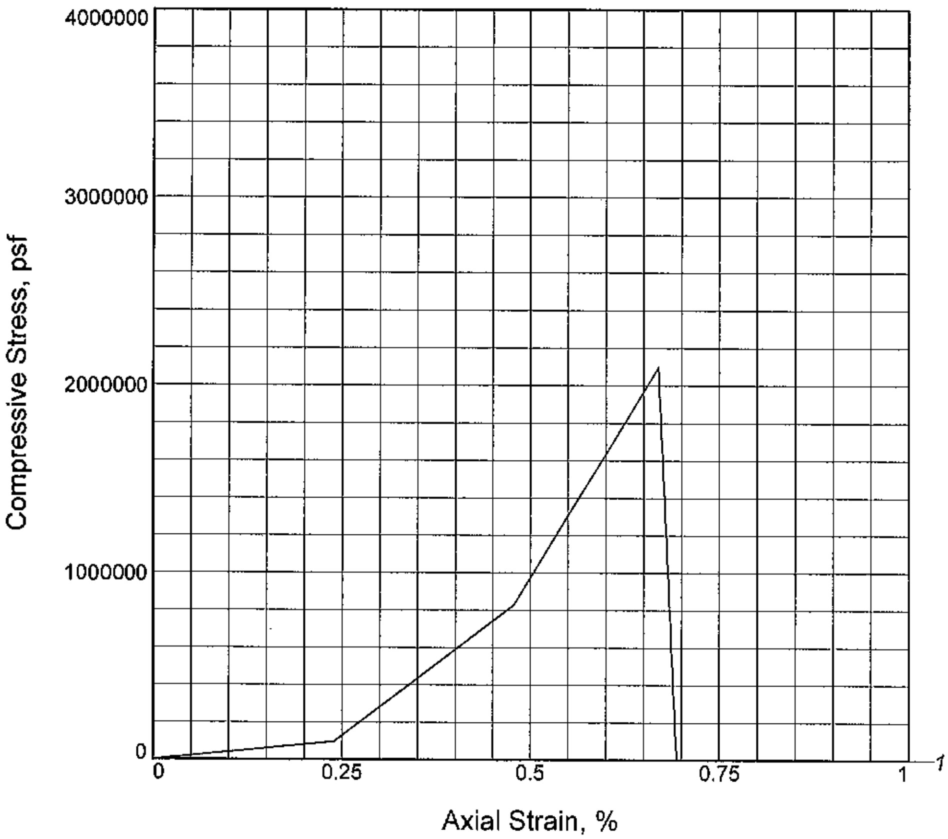
Project: BRENT SPENCE BRIDGE REPLACEMENT

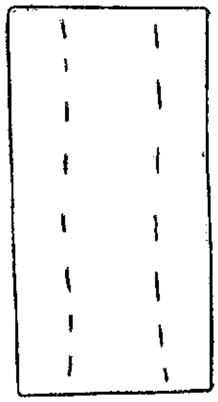
Source of Sample: L-6 **Depth:** 112-112.4'
Sample Number: 2

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2097849.1			
Undrained shear strength, psf	1048924.5			
Failure strain, %	0.7			
Strain rate, in./min.	0.041			
Water content, %	0.2			
Wet density, pcf	168.8			
Dry density, pcf	168.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	4.180			
Height/diameter ratio	2.10			

Description: GRAY LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5558

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

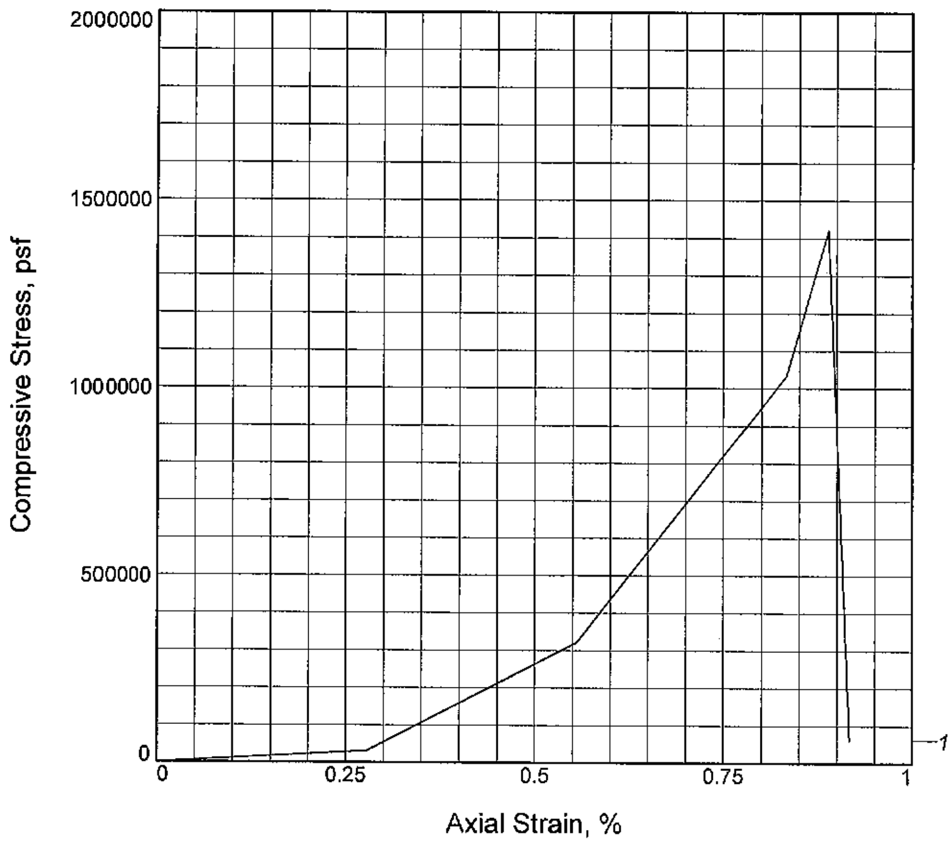
Source of Sample: L-6 **Depth:** 120.5-121'
Sample Number: 4

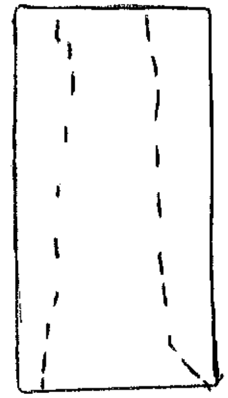
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure _______

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1420383.4			
Undrained shear strength, psf	710191.7			
Failure strain, %	0.9			
Strain rate, in./min.	0.036			
Water content, %	0.2			
Wet density, pcf	164.6			
Dry density, pcf	164.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.600			
Height/diameter ratio	1.81			

Description: GRAY LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5560

Figure ________

Client: PARSONS BRINCKERHOFF

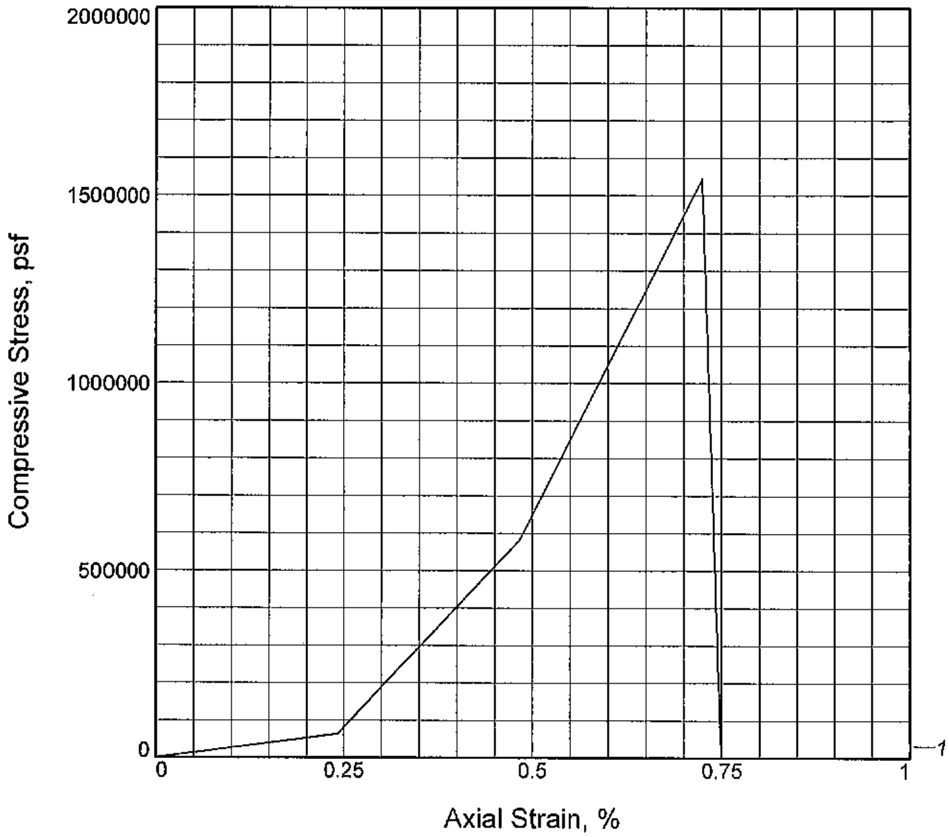
Project: BRENT SPENCE BRIDGE REPLACEMENT

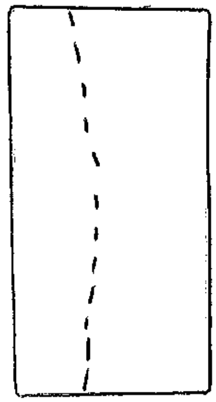
Source of Sample: L-6 **Depth:** 130.5-130.9'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1544585.1			
Undrained shear strength, psf	772292.5			
Failure strain, %	0.7			
Strain rate, in./min.	0.041			
Water content, %	0.2			
Wet density, pcf	169.6			
Dry density, pcf	169.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	4.140			
Height/diameter ratio	2.08			

Description: GRAY LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5562

Figure ________

Client: PARSONS BRINCKERHOFF

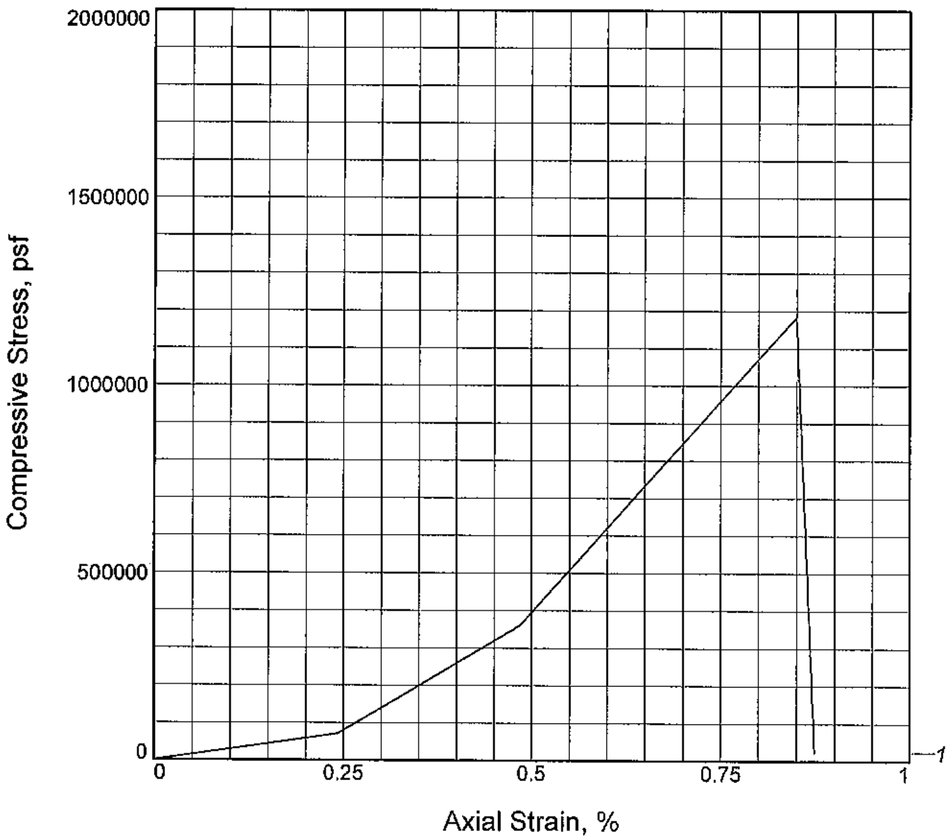
Project: BRENT SPENCE BRIDGE REPLACEMENT

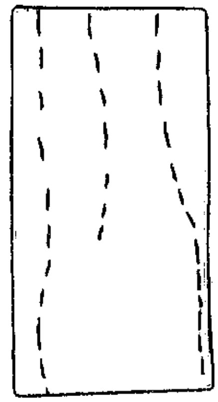
Source of Sample: L-6 **Depth:** 147.5-148'
Sample Number: 8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1183176.9			
Undrained shear strength, psf	591588.4			
Failure strain, %	0.8			
Strain rate, in./min.	0.041			
Water content, %	0.3			
Wet density, pcf	168.5			
Dry density, pcf	168.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.120			
Height/diameter ratio	2.08			

Description: GRAY LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5564

Figure ________

Client: PARSONS BRINCKERHOFF

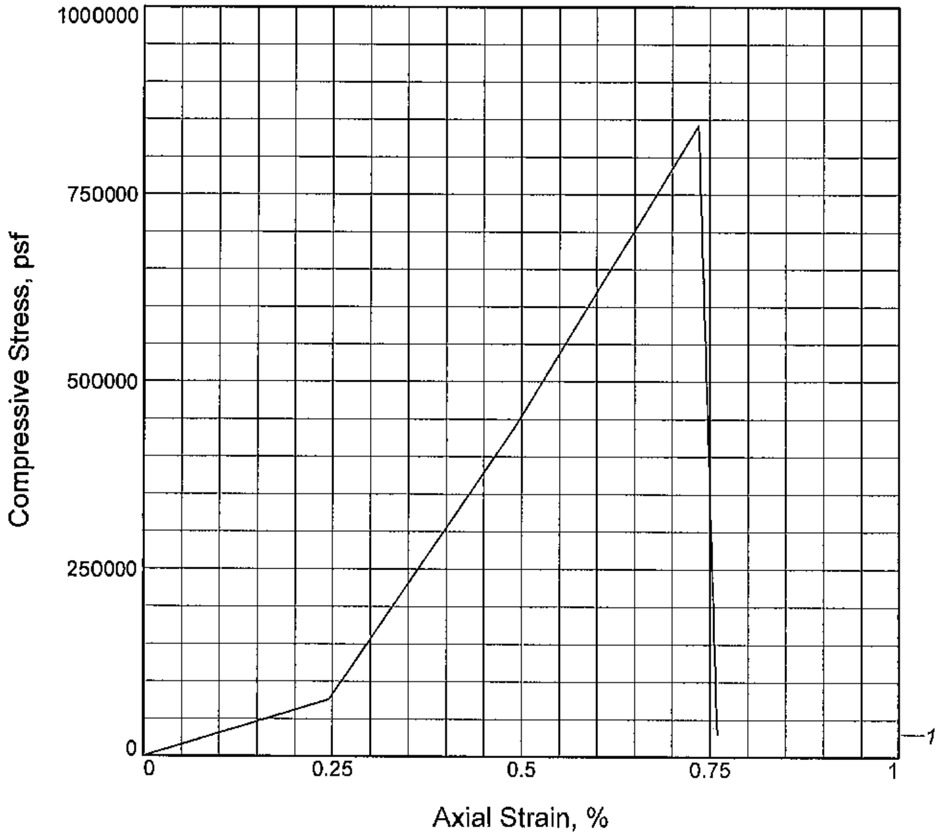
Project: BRENT SPENCE BRIDGE REPLACEMENT

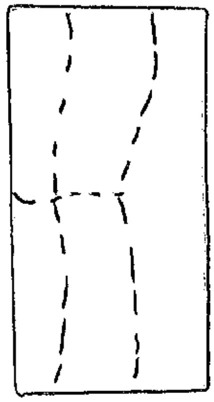
Source of Sample: L-7 **Depth:** 101-101.5'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	842027.9			
Undrained shear strength, psf	421014.0			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	0.3			
Wet density, pcf	166.4			
Dry density, pcf	165.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.080			
Height/diameter ratio	2.06			

Description: GRAY LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5567

Figure ________

Client: PARSONS BRINCKERHOFF

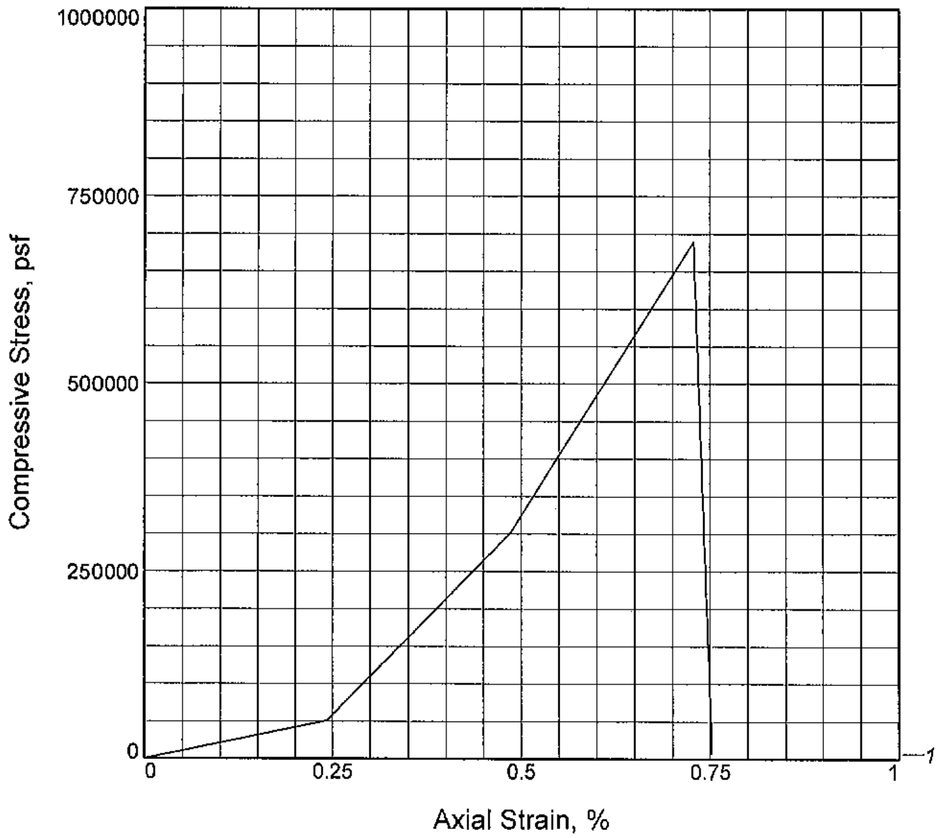
Project: BRENT SPENCE BRIDGE REPLACEMENT

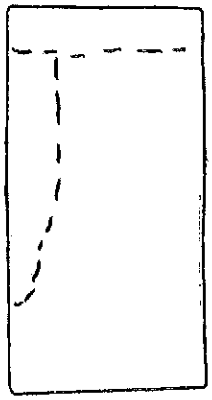
Source of Sample: L-7 **Depth:** 113.7-114.2'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	689715.9			
Undrained shear strength, psf	344857.9			
Failure strain, %	0.7			
Strain rate, in./min.	0.041			
Water content, %	0.6			
Wet density, pcf	167.1			
Dry density, pcf	166.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.120			
Height/diameter ratio	2.08			

Description: GRAY LIMESTONE W/ SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone w/shale

Project No.: N1105070
Date Sampled: 7-7-10
Remarks:
Lab No. 5571

Figure _______

Client: PARSONS BRINCKERHOFF

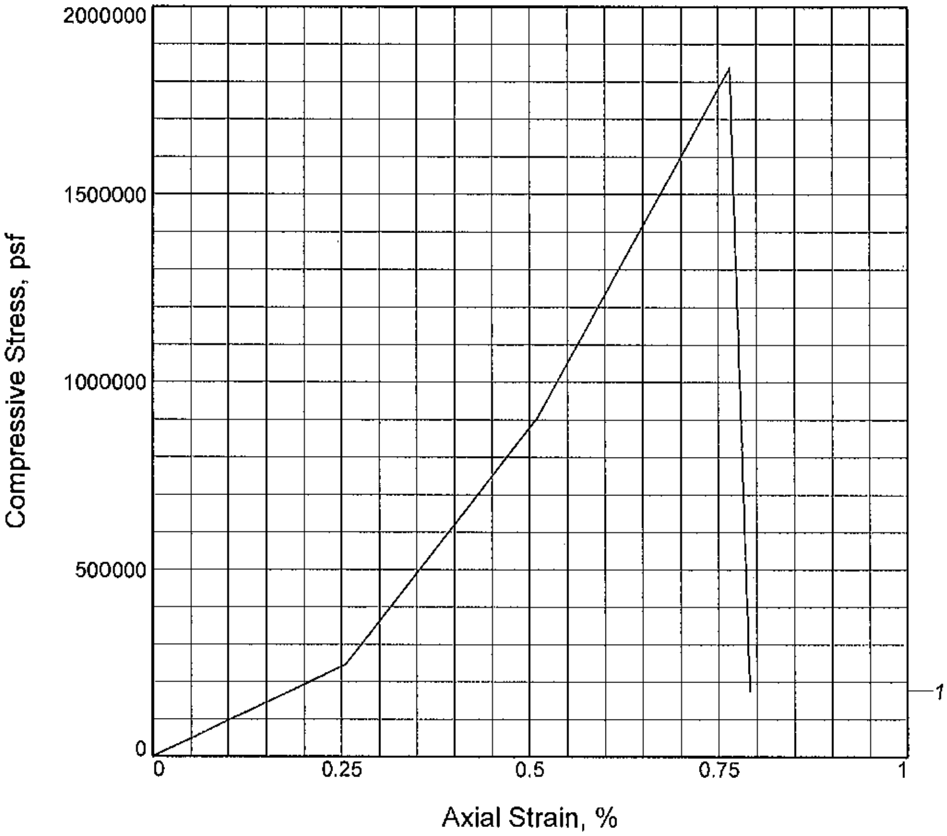
Project: BRENT SPENCE BRIDGE REPLACEMENT

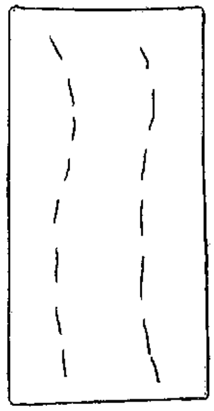
Source of Sample: L-7 **Depth:** 132.5-133.2'
Sample Number: 8

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1837107.7			
Undrained shear strength, psf	918553.9			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.4			
Wet density, pcf	167.8			
Dry density, pcf	167.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.920			
Height/diameter ratio	1.99			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-13-10
Remarks:
Lab No. 6052

Figure ________

Client: PARSONS BRINCKERHOFF

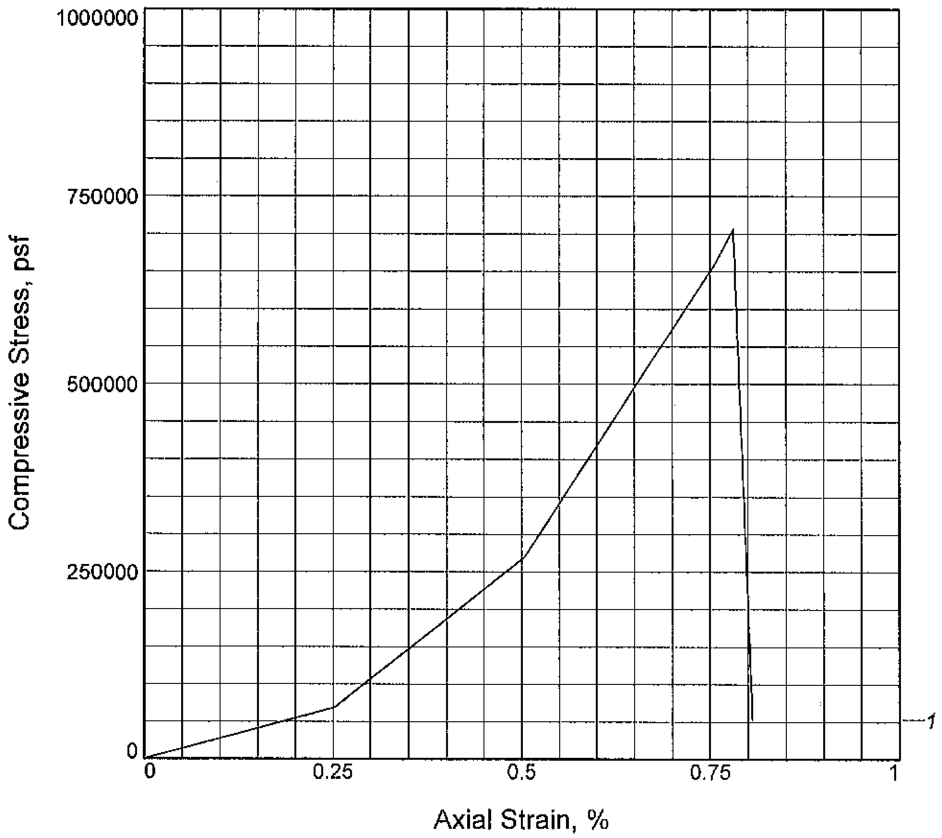
Project: BRENT SPENCE BRIDGE REPLACEMENT

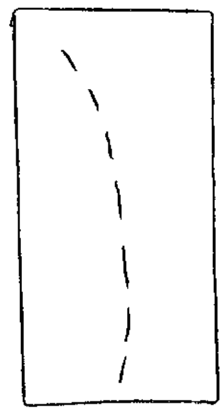
Source of Sample: R-1 **Depth:** 91.5-92.1'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	706054.5			
Undrained shear strength, psf	353027.2			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	2.4			
Wet density, pcf	164.3			
Dry density, pcf	160.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.970			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-13-10
Remarks:
Lab No. 6053

Figure ________

Client: PARSONS BRINCKERHOFF

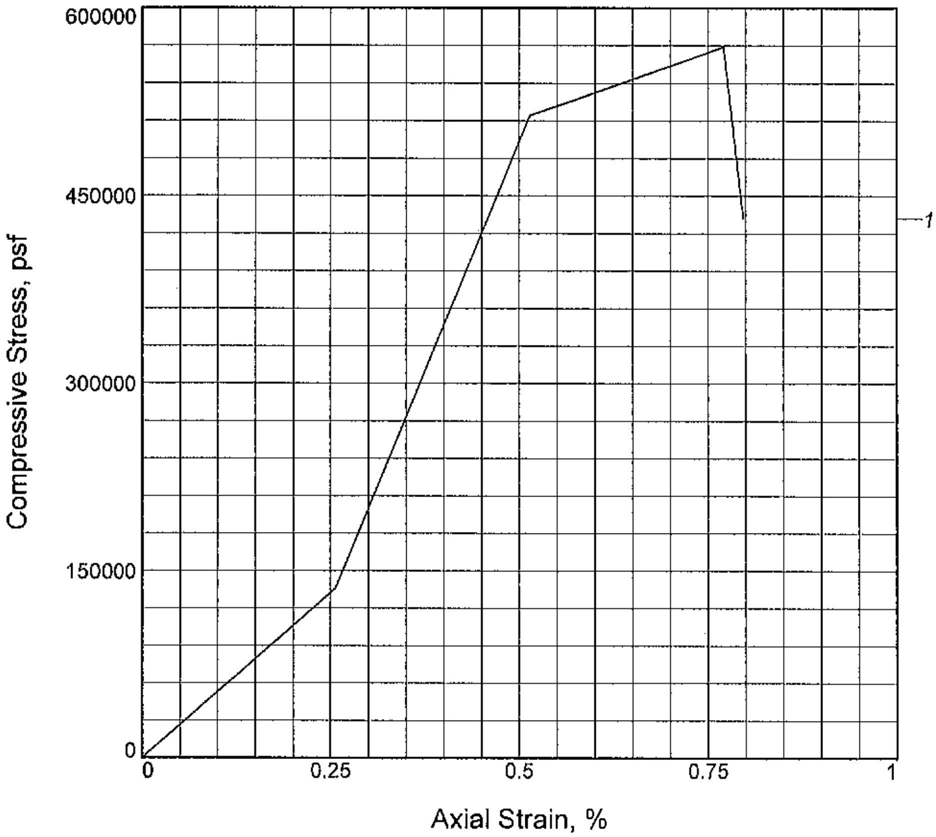
Project: BRENT SPENCE BRIDGE REPLACEMENT

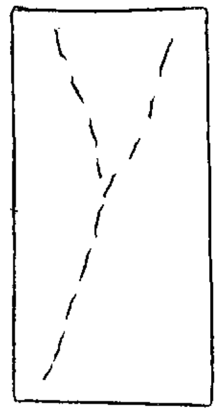
Source of Sample: R-1 **Depth:** 94.3-95'
Sample Number: 2

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	568922.9			
Undrained shear strength, psf	284461.5			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	2.4			
Wet density, pcf	164.6			
Dry density, pcf	160.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.890			
Height/diameter ratio	1.97			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-13-10

Remarks:
Lab No. 6055

Figure ________

Client: PARSONS BRINCKERHOFF

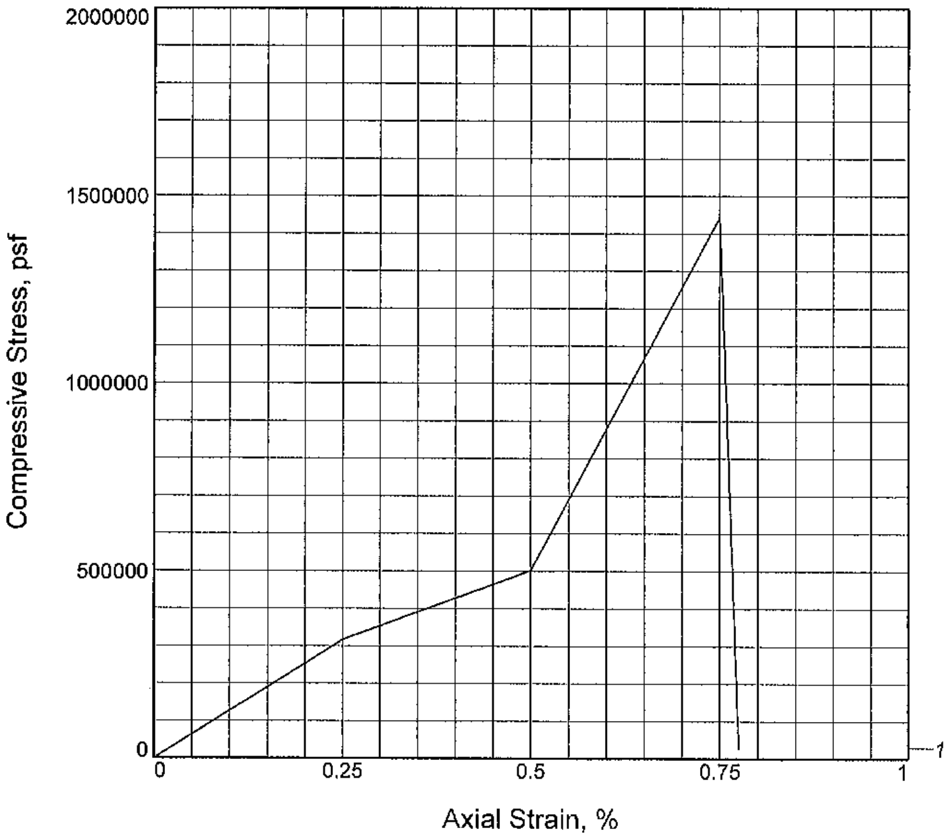
Project: BRENT SPENCE BRIDGE REPLACEMENT

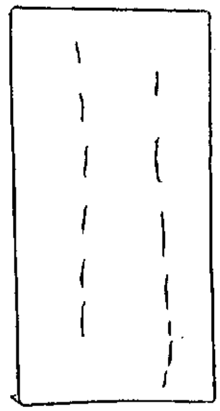
Source of Sample: R-1 **Depth:** 104.5-105'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1443507.9			
Undrained shear strength, psf	721754.0			
Failure strain, %	0.8			
Strain rate, in./min.	0.040			
Water content, %	0.8			
Wet density, pcf	162.6			
Dry density, pcf	161.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.000			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-13-10
Remarks:
Lab No. 6058

Figure ________

Client: PARSONS BRINCKERHOFF

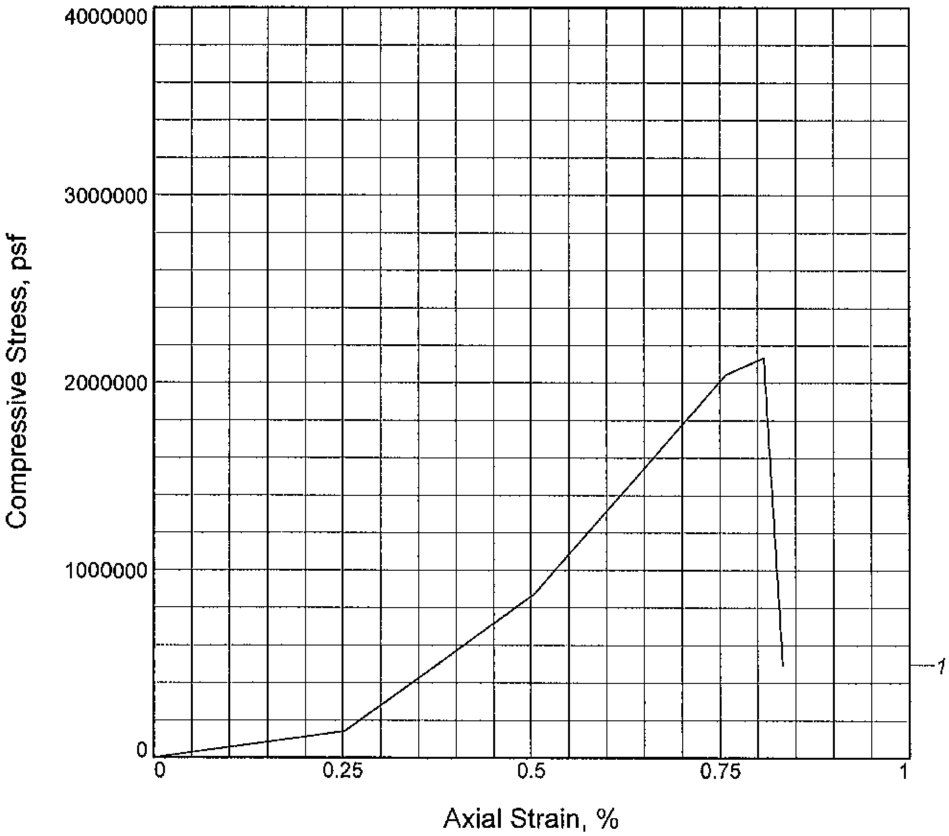
Project: BRENT SPENCE BRIDGE REPLACEMENT

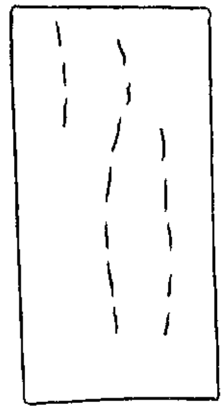
Source of Sample: R-1 **Depth:** 123-123.5'
Sample Number: 7

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2134074.0			
Undrained shear strength, psf	1067037.0			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.6			
Wet density, pcf	166.2			
Dry density, pcf	165.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.960			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-13-10
Remarks:
Lab No. 6060

Figure _______

Client: PARSONS BRINCKERHOFF

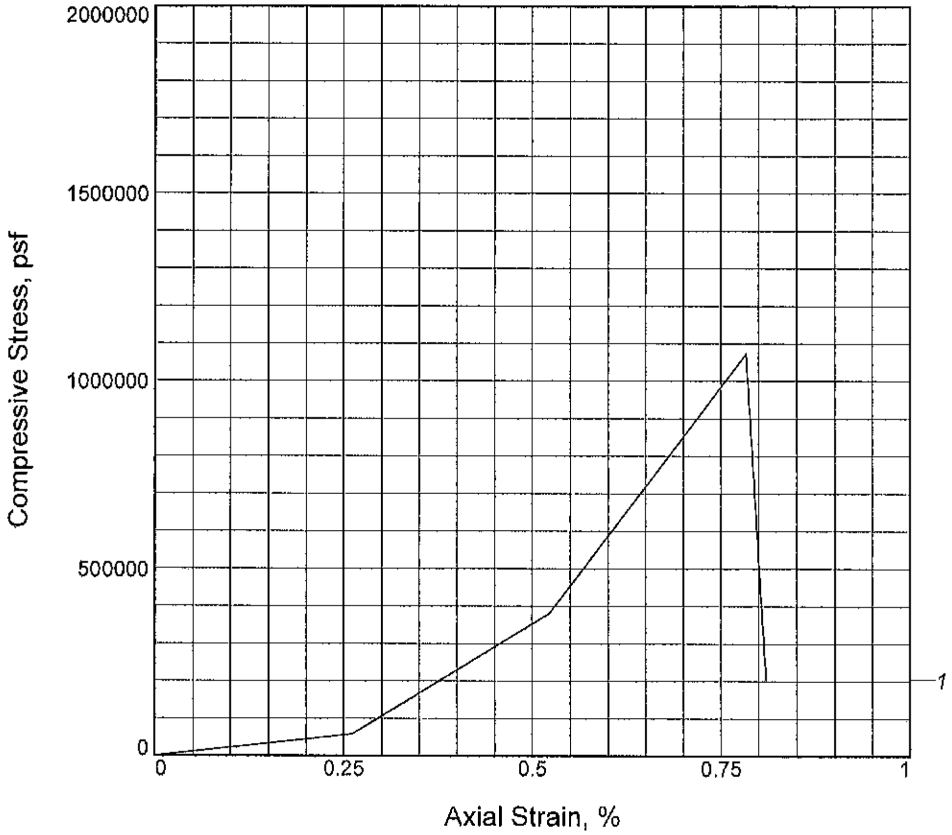
Project: BRENT SPENCE BRIDGE REPLACEMENT

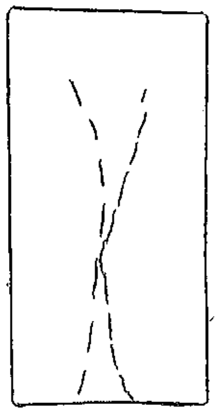
Source of Sample: R-1 **Depth:** 136-136.5'
Sample Number: 9

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1072646.9			
Undrained shear strength, psf	536323.4			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	1.3			
Wet density, pcf	162.4			
Dry density, pcf	160.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.830			
Height/diameter ratio	1.92			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6062

Figure ________

Client: PARSONS BRINCKERHOFF

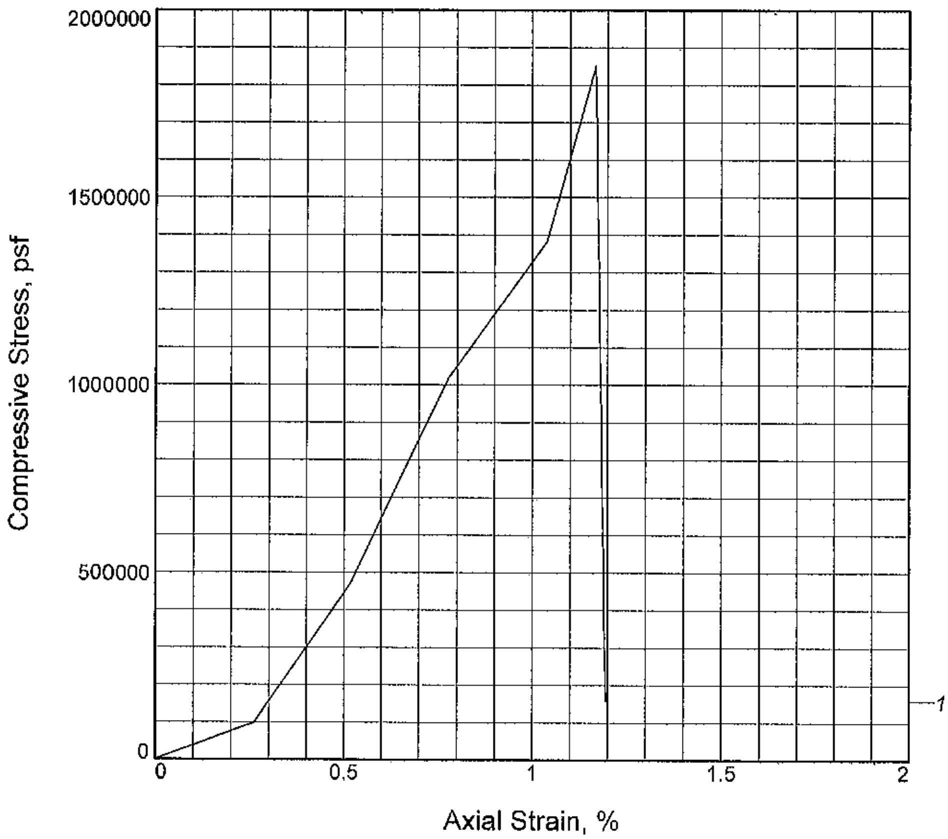
Project: BRENT SPENCE BRIDGE REPLACEMENT

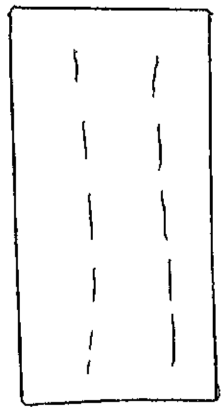
Source of Sample: R-1 **Depth:** 145.3-145.7'
Sample Number: 11

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1850857.7			
Undrained shear strength, psf	925428.8			
Failure strain, %	1.2			
Strain rate, in./min.	0.038			
Water content, %	0.6			
Wet density, pcf	161.9			
Dry density, pcf	160.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.850			
Height/diameter ratio	1.93			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6065

Figure ________

Client: PARSONS BRINCKERHOFF

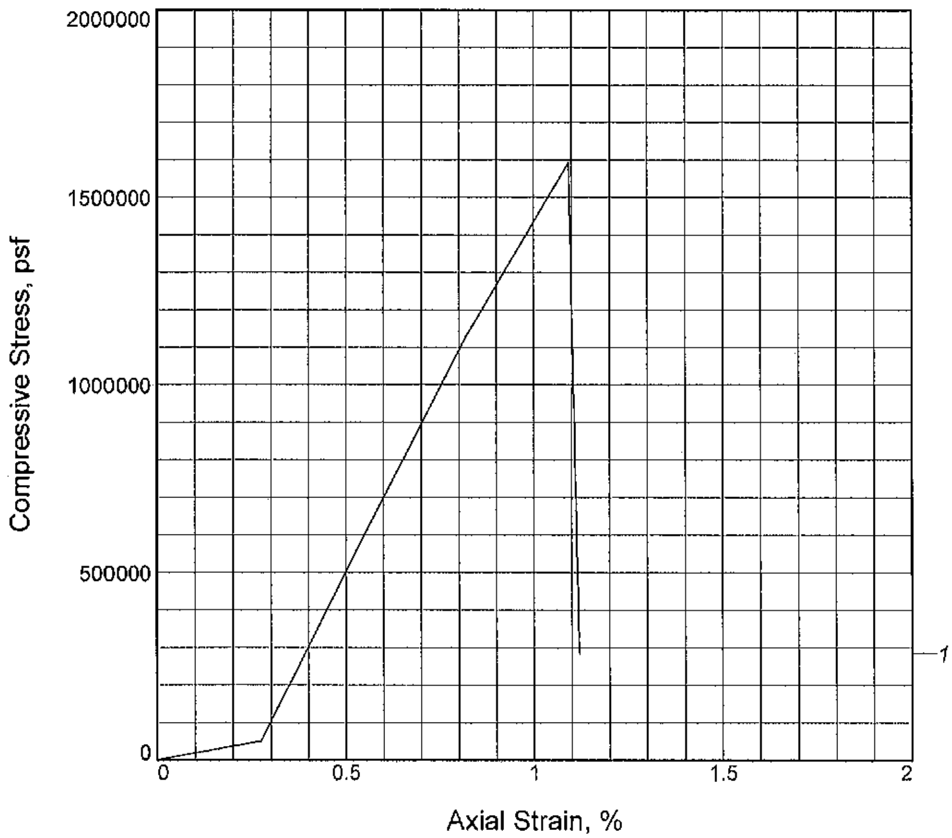
Project: BRENT SPENCE BRIDGE REPLACEMENT

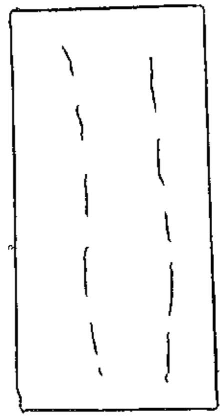
Source of Sample: R-1 **Depth:** 153-153.6'
Sample Number: 14

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1592203.7			
Undrained shear strength, psf	796101.9			
Failure strain, %	1.1			
Strain rate, in./min.	0.036			
Water content, %	0.7			
Wet density, pcf	162.4			
Dry density, pcf	161.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.660			
Height/diameter ratio	1.84			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6066

Figure ________

Client: PARSONS BRINCKERHOFF

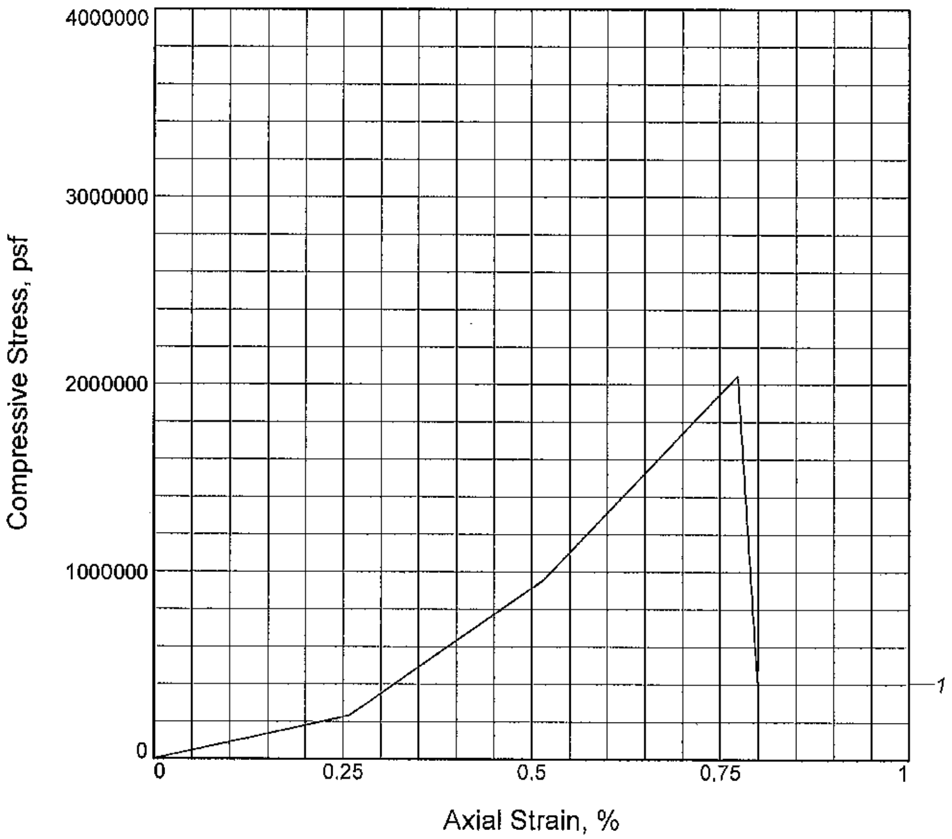
Project: BRENT SPENCE BRIDGE REPLACEMENT

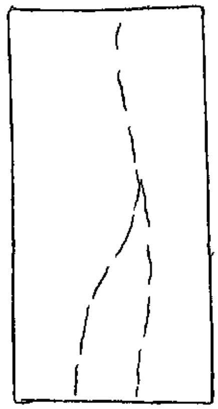
Source of Sample: R-1 **Depth:** 159.1-159.9'
Sample Number: 15

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2046785.9			
Undrained shear strength, psf	1023392.9			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	1.2			
Wet density, pcf	161.3			
Dry density, pcf	159.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.880			
Height/diameter ratio	1.95			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6068

Figure ________

Client: PARSONS BRINCKERHOFF

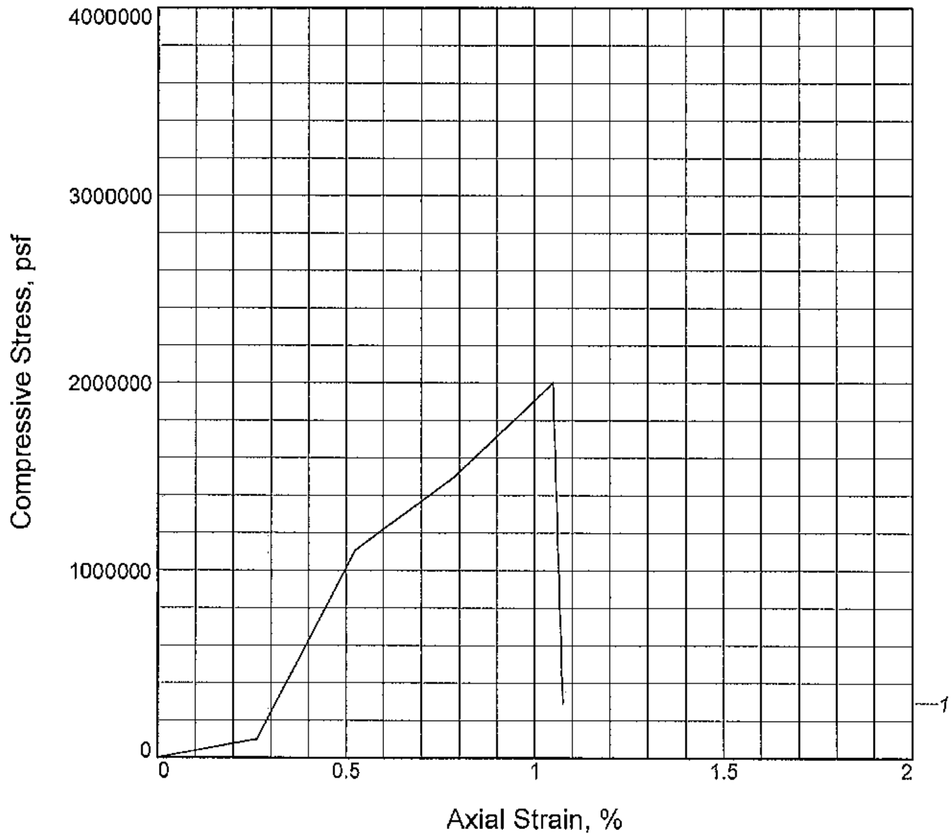
Project: BRENT SPENCE BRIDGE REPLACEMENT

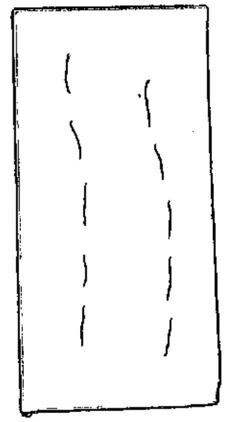
Source of Sample: R-1 **Depth:** 163.5-164.2'
Sample Number: 17

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2000122.6			
Undrained shear strength, psf	1000061.3			
Failure strain, %	1.0			
Strain rate, in./min.	0.038			
Water content, %	1.2			
Wet density, pcf	168.6			
Dry density, pcf	166.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.810			
Height/diameter ratio	1.91			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10

Remarks:
Lab No. 6069

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

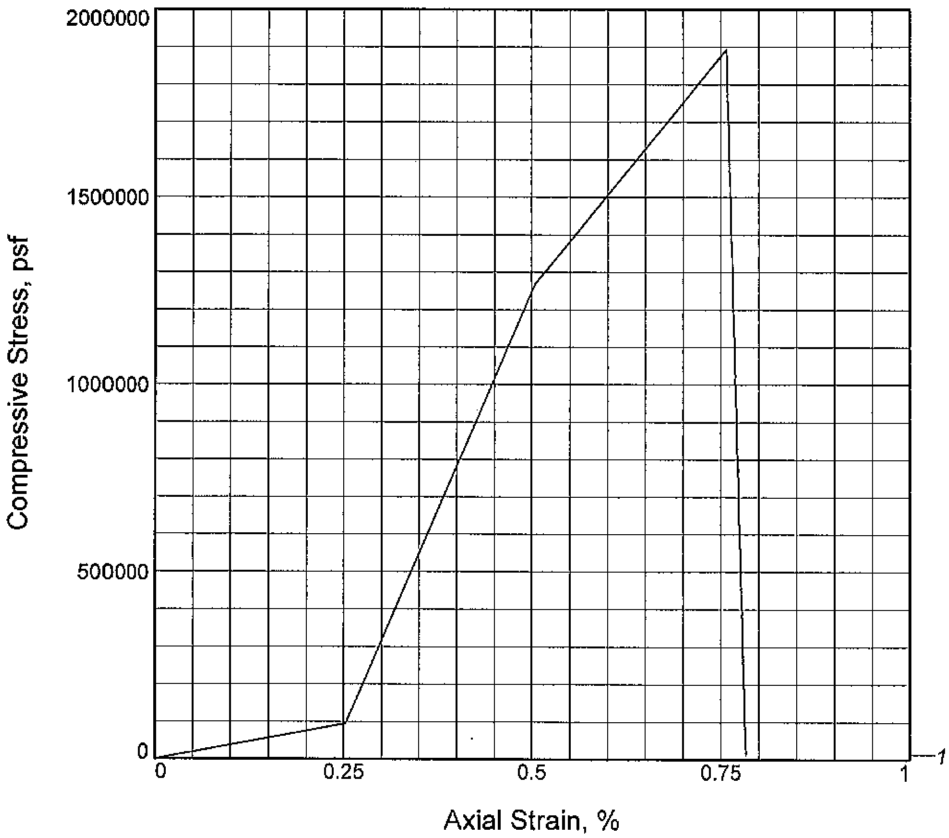
Source of Sample: R-1 **Depth:** 168.2-168.9'
Sample Number: 18

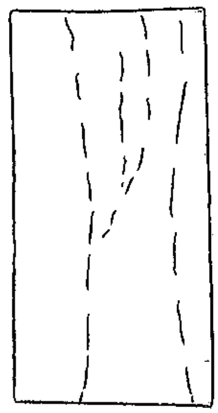
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1893232.5			
Undrained shear strength, psf	946616.2			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.2			
Wet density, pcf	167.9			
Dry density, pcf	167.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.960			
Height/diameter ratio	2.00			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5878

Figure ________

Client: PARSONS BRINCKERHOFF

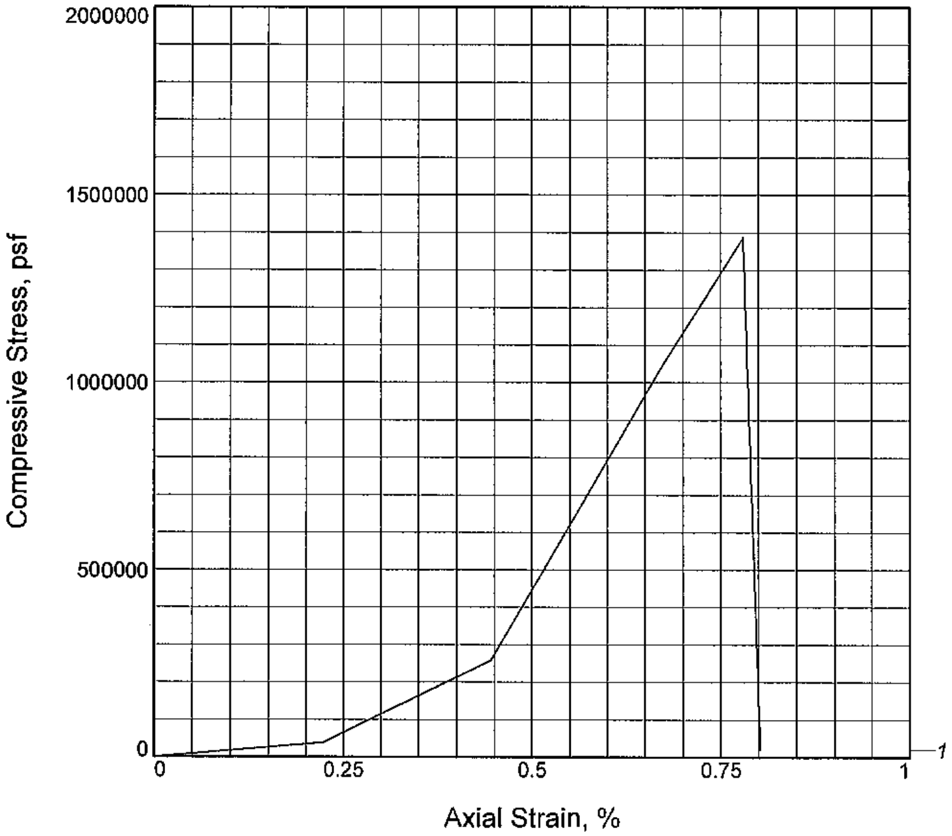
Project: BRENT SPENCE BRIDGE REPLACEMENT

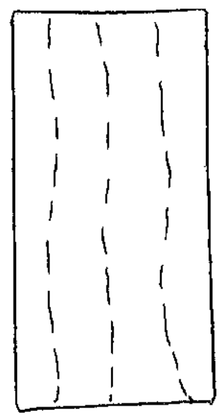
Source of Sample: R-2 **Depth:** 87.5-88'
Sample Number: 1/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1387302.6			
Undrained shear strength, psf	693651.3			
Failure strain, %	0.8			
Strain rate, in./min.	0.044			
Water content, %	0.6			
Wet density, pcf	165.2			
Dry density, pcf	164.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.490			
Height/diameter ratio	2.27			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5881

Figure ________

Client: PARSONS BRINCKERHOFF

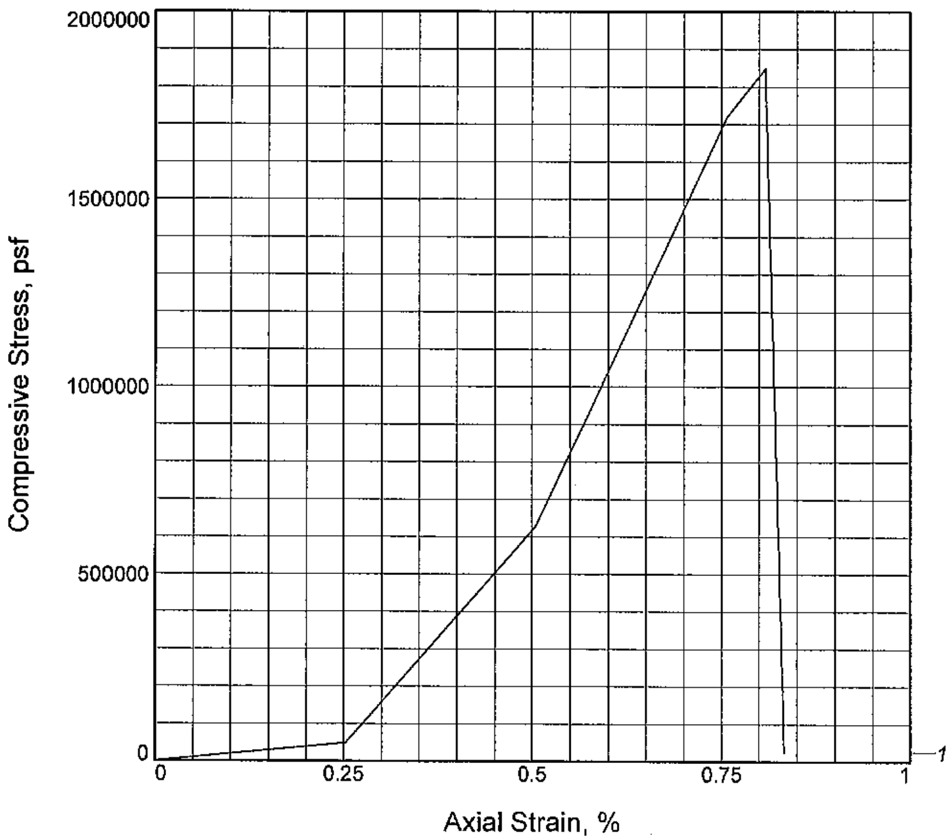
Project: BRENT SPENCE BRIDGE REPLACEMENT

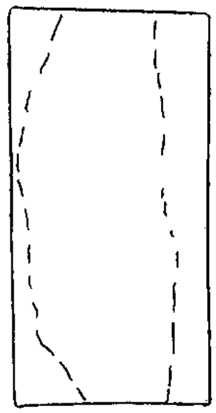
Source of Sample: R-2 **Depth:** 89.3-89.7'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1848338.3			
Undrained shear strength, psf	924169.1			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.4			
Wet density, pcf	167.5			
Dry density, pcf	166.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.960			
Height/diameter ratio	2.00			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5882

Figure ________

Client: PARSONS BRINCKERHOFF

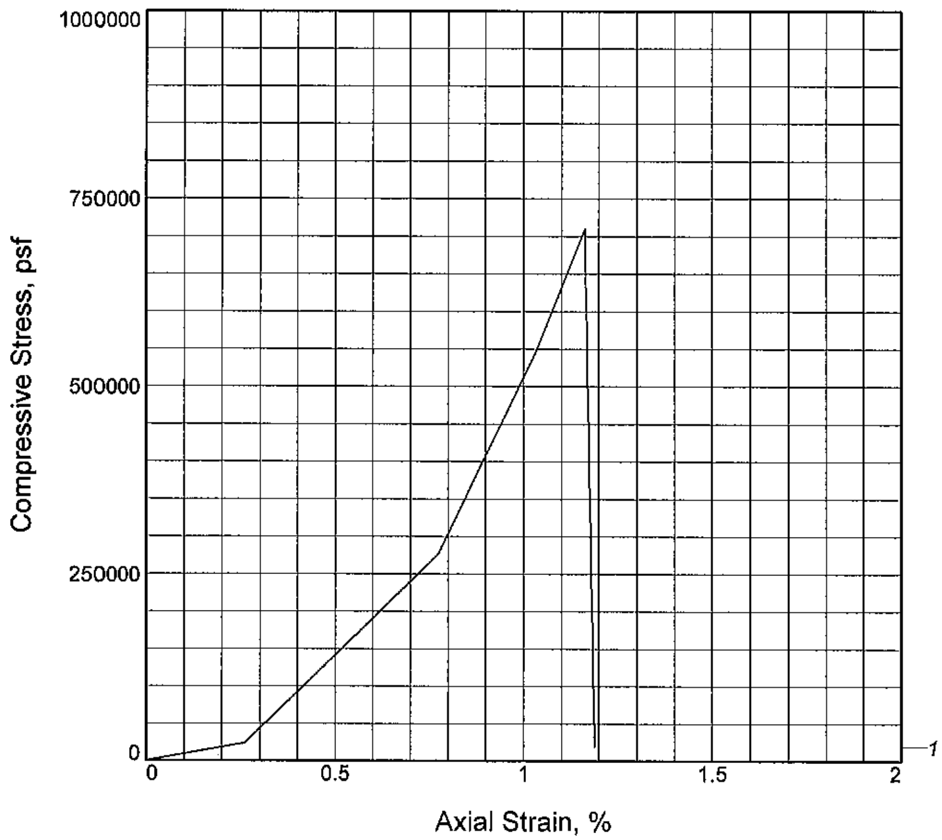
Project: BRENT SPENCE BRIDGE REPLACEMENT

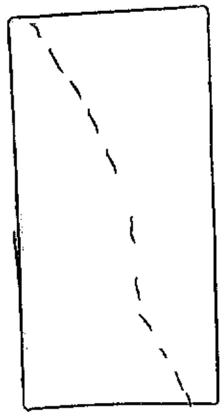
Source of Sample: R-2 **Depth:** 90.7-91.6'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	709761.6			
Undrained shear strength, psf	354880.8			
Failure strain, %	1.2			
Strain rate, in./min.	0.038			
Water content, %	2.2			
Wet density, pcf	160.9			
Dry density, pcf	157.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.870			
Height/diameter ratio	1.95			

Description: SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5883

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-2 **Depth:** 93.7-94'
Sample Number: 2/NQ

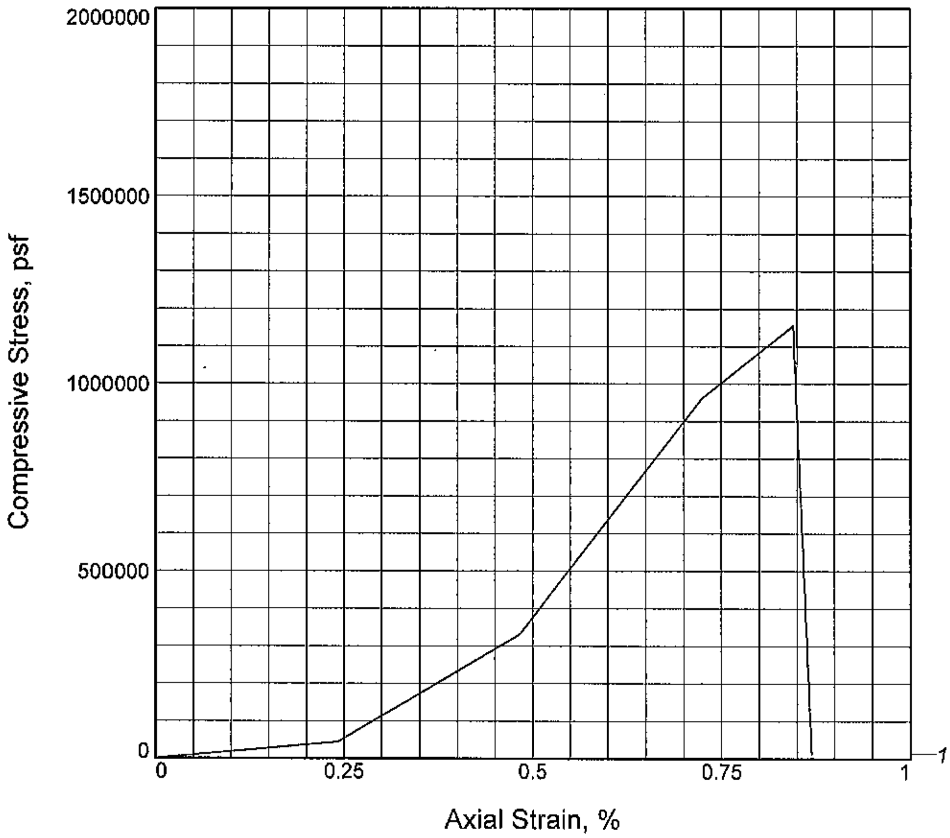
UNCONFINED COMPRESSION TEST

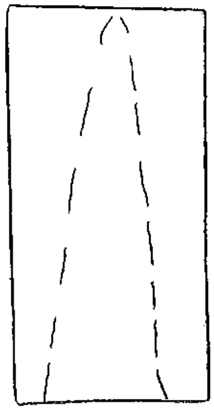
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1155667.4			
Undrained shear strength, psf	577833.7			
Failure strain, %	0.8			
Strain rate, in./min.	0.041			
Water content, %	0.8			
Wet density, pcf	165.1			
Dry density, pcf	163.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	4.140			
Height/diameter ratio	2.10			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5884

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

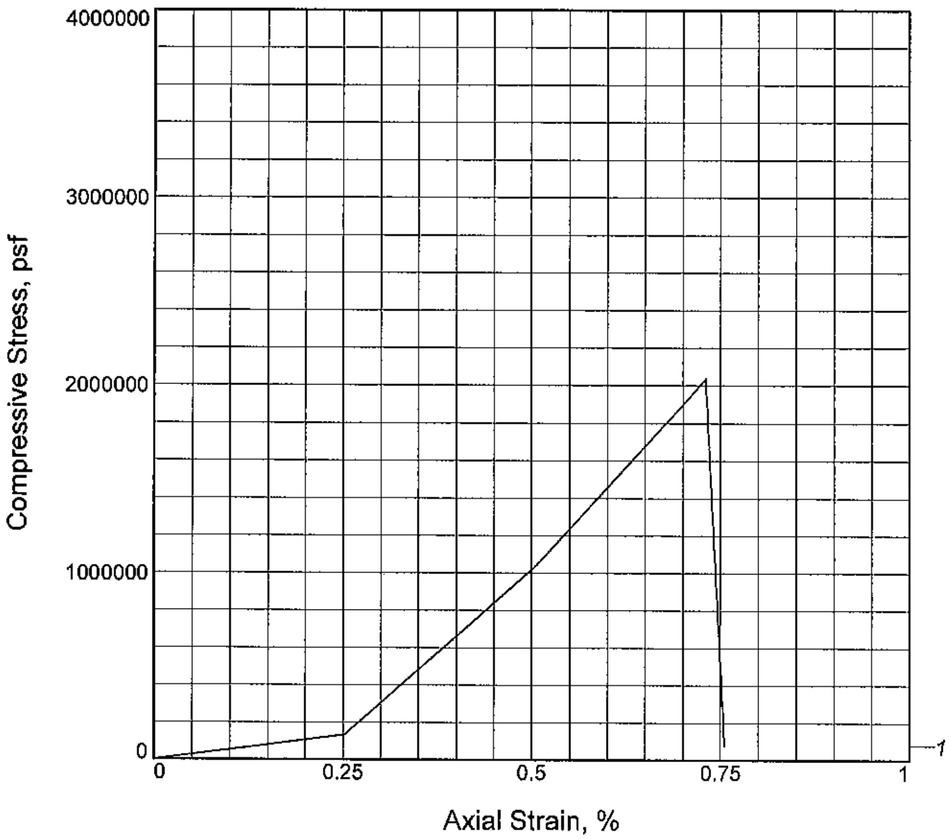
Source of Sample: R-2 **Depth:** 99.8-100.1'
Sample Number: 3/NQ

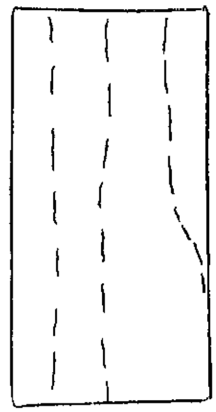
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2034883.4			
Undrained shear strength, psf	1017441.7			
Failure strain, %	0.7			
Strain rate, in./min.	0.039			
Water content, %	0.2			
Wet density, pcf	168.4			
Dry density, pcf	168.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.970			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date Sampled: 7-20-10

Remarks:
Lab No. 5887

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-2 **Depth:** 112.9-113.9'

Sample Number: 6/NQ

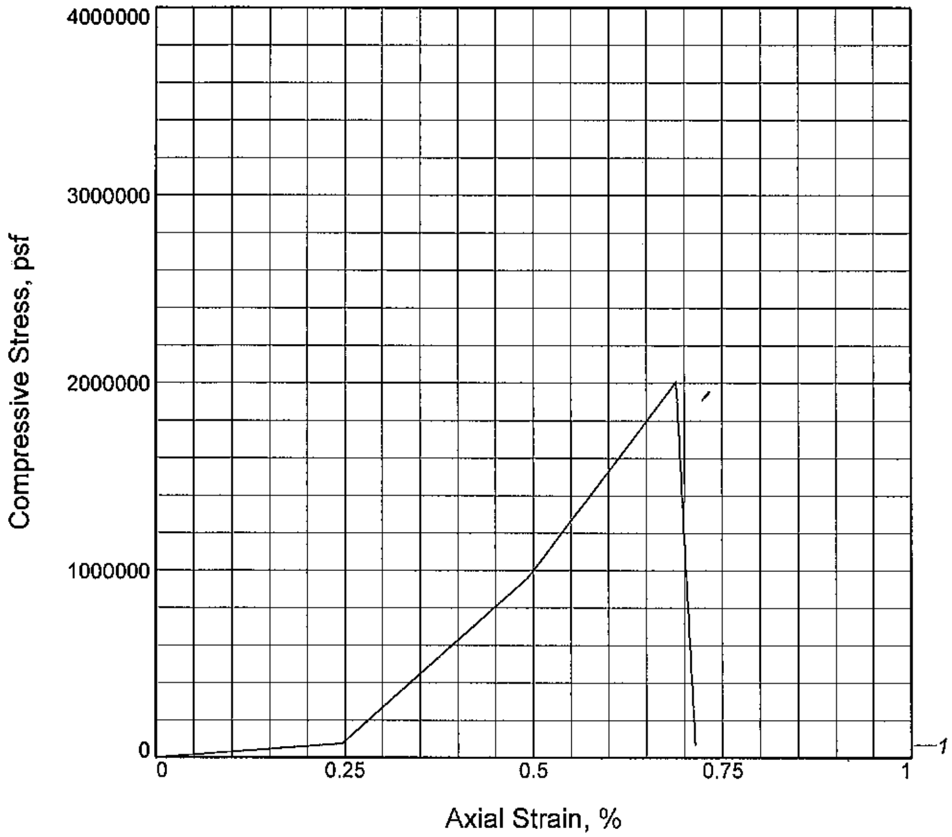
UNCONFINED COMPRESSION TEST

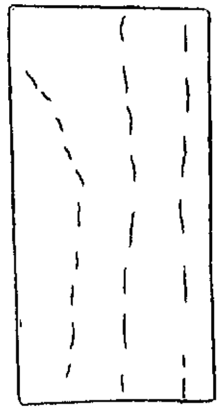
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2005345.4			
Undrained shear strength, psf	1002672.7			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	0.4			
Wet density, pcf	169.1			
Dry density, pcf	168.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.060			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5888

Figure _______

Client: PARSONS BRINCKERHOFF

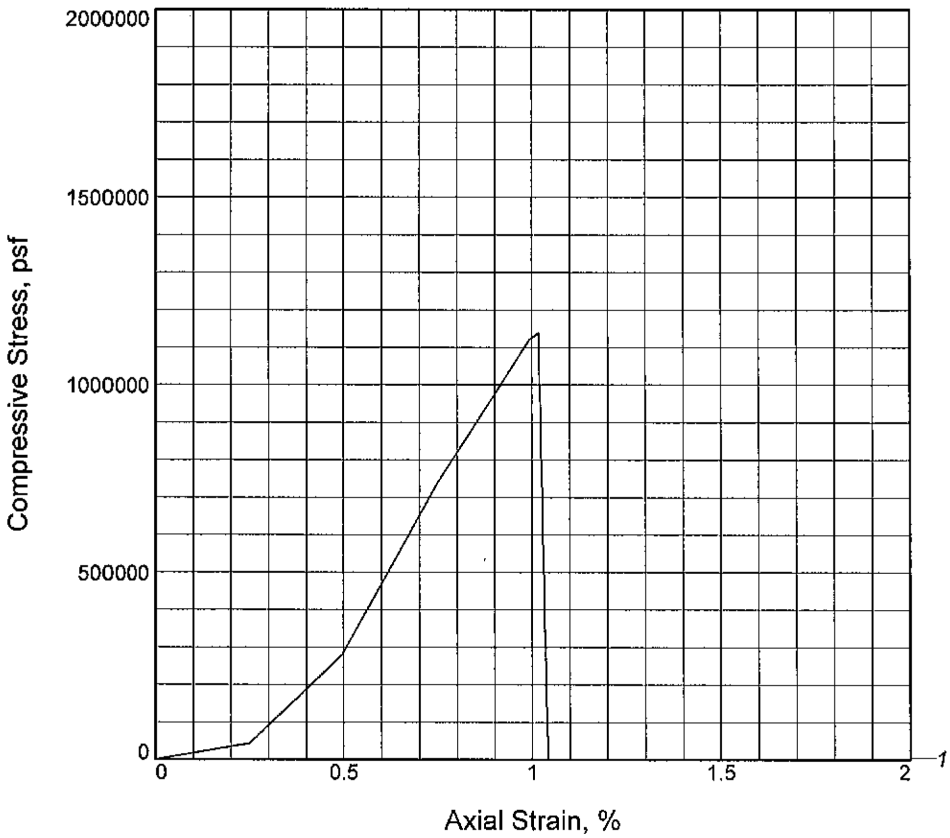
Project: BRENT SPENCE BRIDGE REPLACEMENT

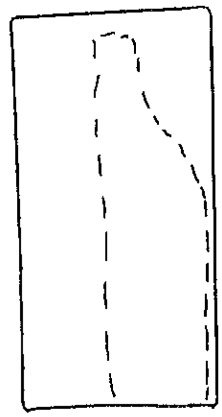
Source of Sample: R-2 **Depth:** 119.8-120.6'
Sample Number: 8/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1138483.6			
Undrained shear strength, psf	569241.8			
Failure strain, %	1.0			
Strain rate, in./min.	0.040			
Water content, %	1.2			
Wet density, pcf	165.7			
Dry density, pcf	163.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.030			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-20-10
Remarks:
Lab No. 5891

Figure ______

Client: PARSONS BRINCKERHOFF

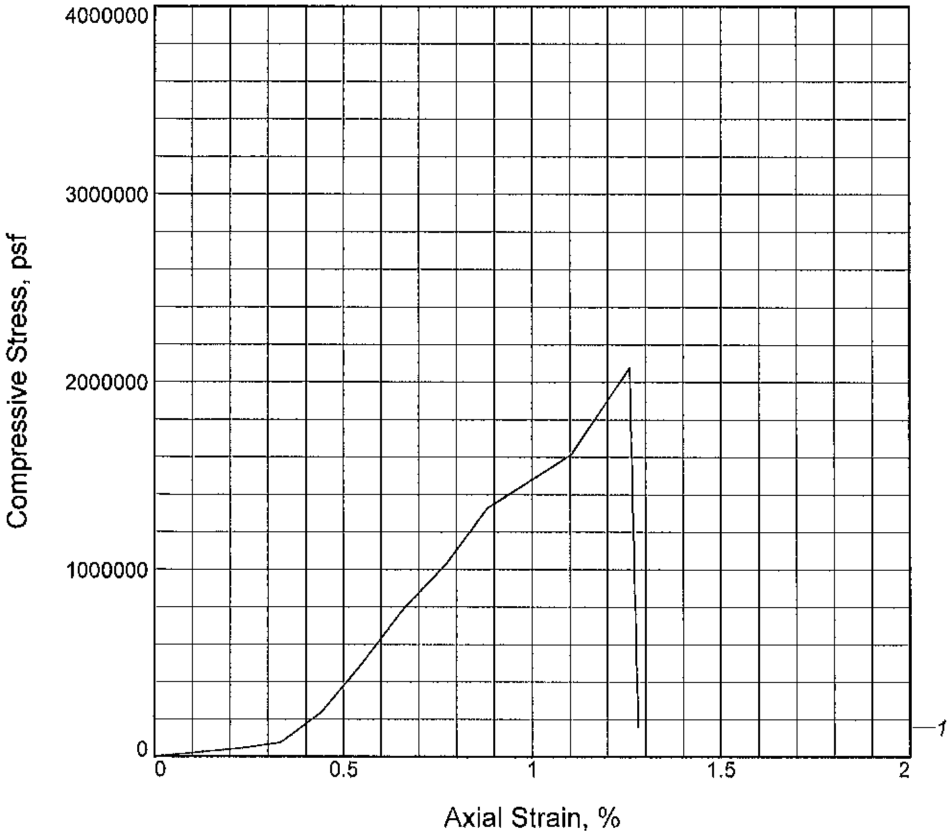
Project: BRENT SPENCE BRIDGE REPLACEMENT

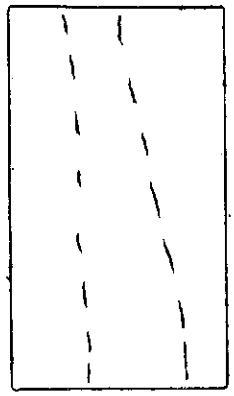
Source of Sample: R-2 **Depth:** 139-139.5'
Sample Number: 12/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2075031.3			
Undrained shear strength, psf	1037515.7			
Failure strain, %	1.3			
Strain rate, in./min.	0.045			
Water content, %	0.6			
Wet density, pcf	167.6			
Dry density, pcf	166.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	4.530			
Height/diameter ratio	1.90			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10

Remarks:
Lab No. 9697

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

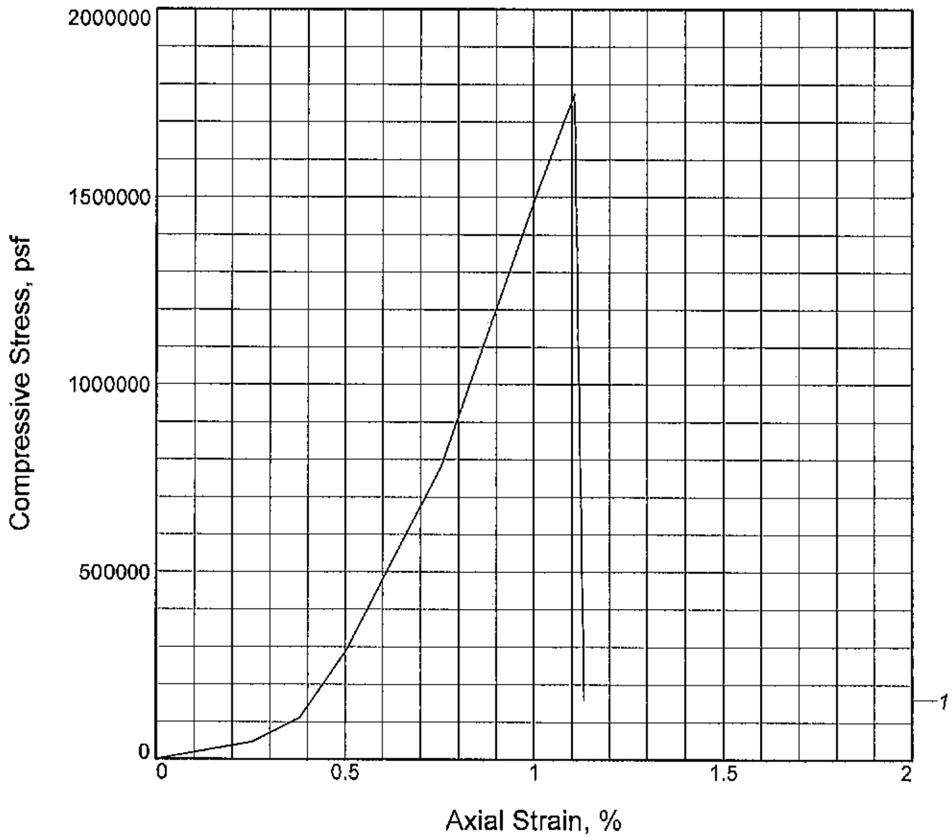
Source of Sample: R-2A **Depth:** 99.5-100.1'
Sample Number: 1

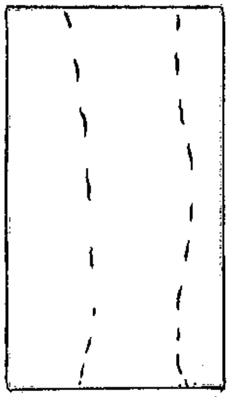
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1773180.9			
Undrained shear strength, psf	886590.5			
Failure strain, %	1.1			
Strain rate, in./min.	0.039			
Water content, %	1.1			
Wet density, pcf	169.3			
Dry density, pcf	167.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.375			
Specimen height, in.	3.970			
Height/diameter ratio	1.67			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10

Remarks:
Lab No. 9699

Figure _______

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

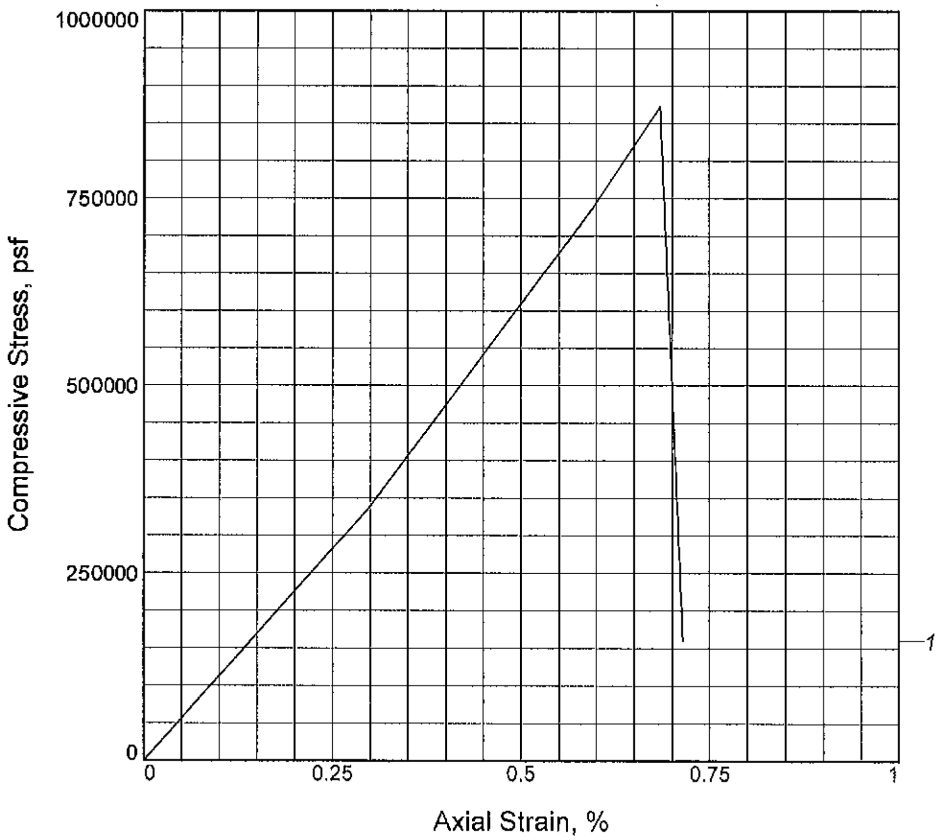
Source of Sample: R-2A **Depth:** 111.8-112.2'
Sample Number: 3

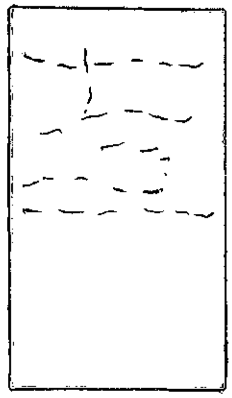
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	872331.3			
Undrained shear strength, psf	436165.7			
Failure strain, %	0.7			
Strain rate, in./min.	0.033			
Water content, %	1.9			
Wet density, pcf	161.1			
Dry density, pcf	158.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	3.360			
Height/diameter ratio	1.41			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9701

Figure ________

Client: PARSONS BRINCKERHOFF

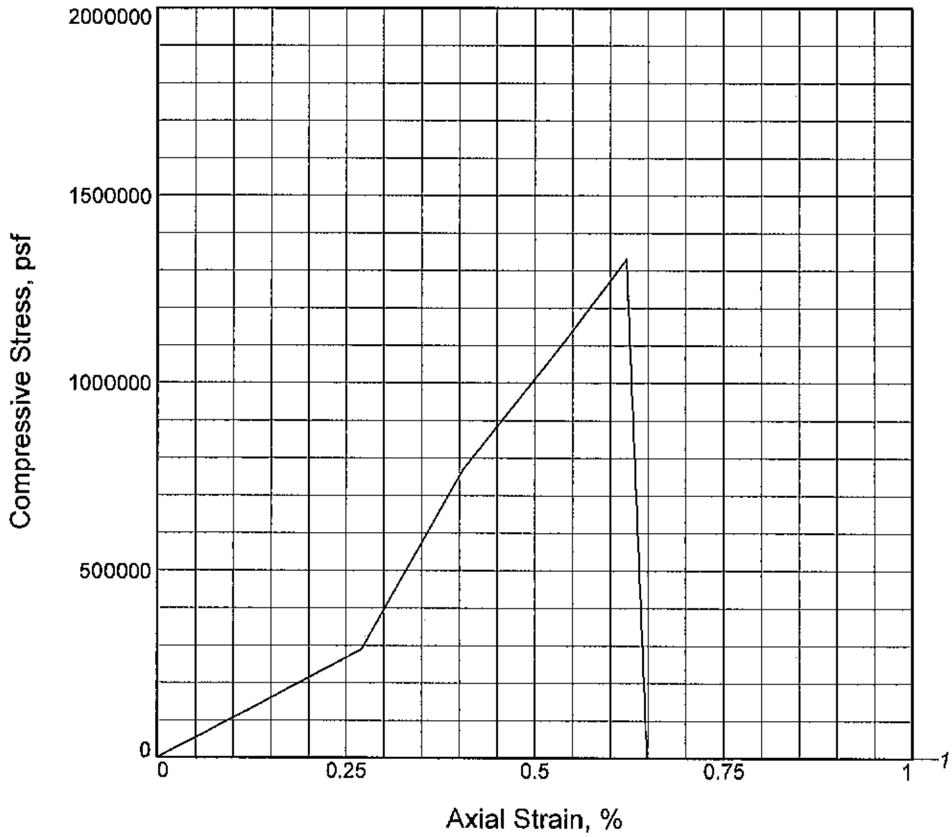
Project: BRENT SPENCE BRIDGE REPLACEMENT

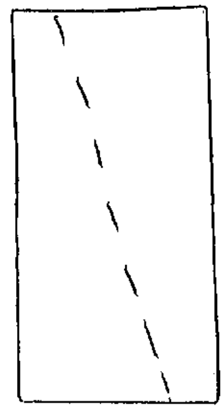
Source of Sample: R-2A **Depth:** 117.8-118.2'
Sample Number: 5

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1331115.6			
Undrained shear strength, psf	665557.8			
Failure strain, %	0.6			
Strain rate, in./min.	0.037			
Water content, %	0.7			
Wet density, pcf	164.3			
Dry density, pcf	163.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.700			
Height/diameter ratio	1.88			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6014

Figure ________

Client: PARSONS BRINCKERHOFF

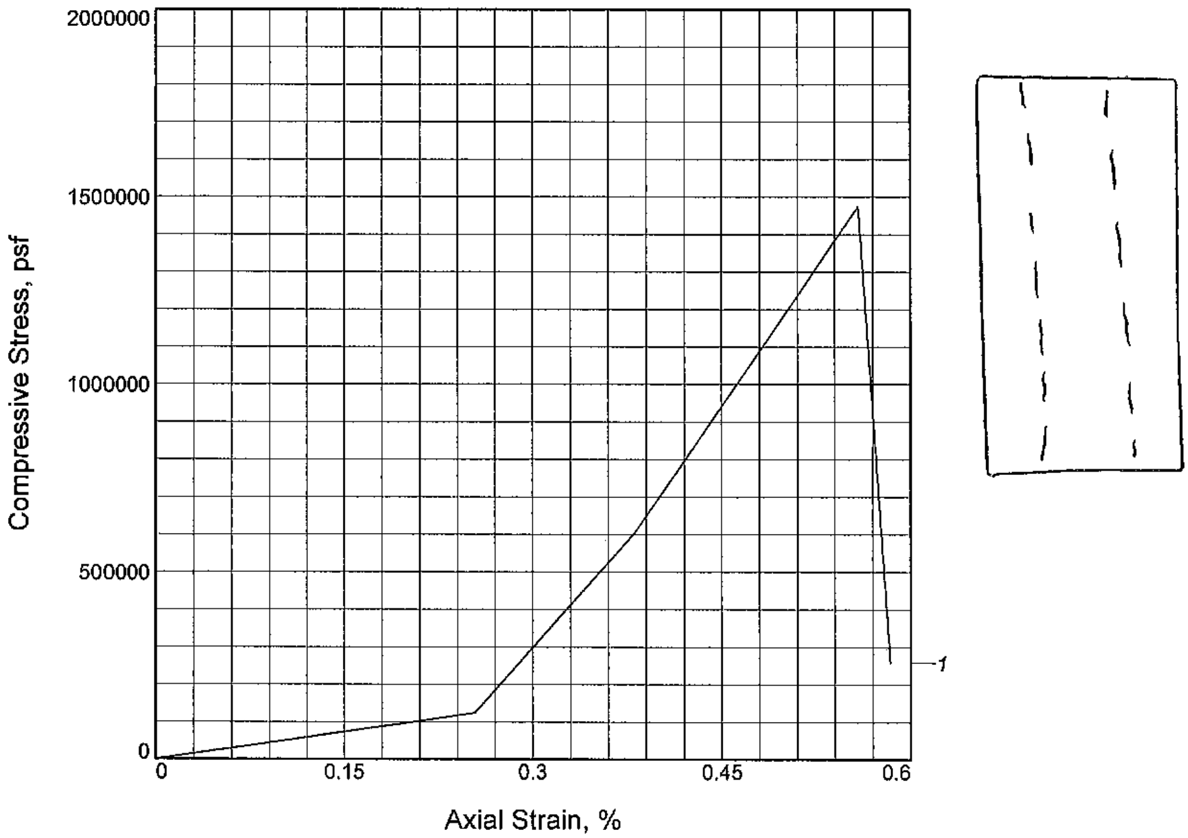
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-3 **Depth:** 92.3-92.7'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1474639.5			
Undrained shear strength, psf	737319.7			
Failure strain, %	0.6			
Strain rate, in./min.	0.039			
Water content, %	0.2			
Wet density, pcf	170.4			
Dry density, pcf	170.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.940			
Height/diameter ratio	2.00			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6015

Figure ________

Client: PARSONS BRINCKERHOFF

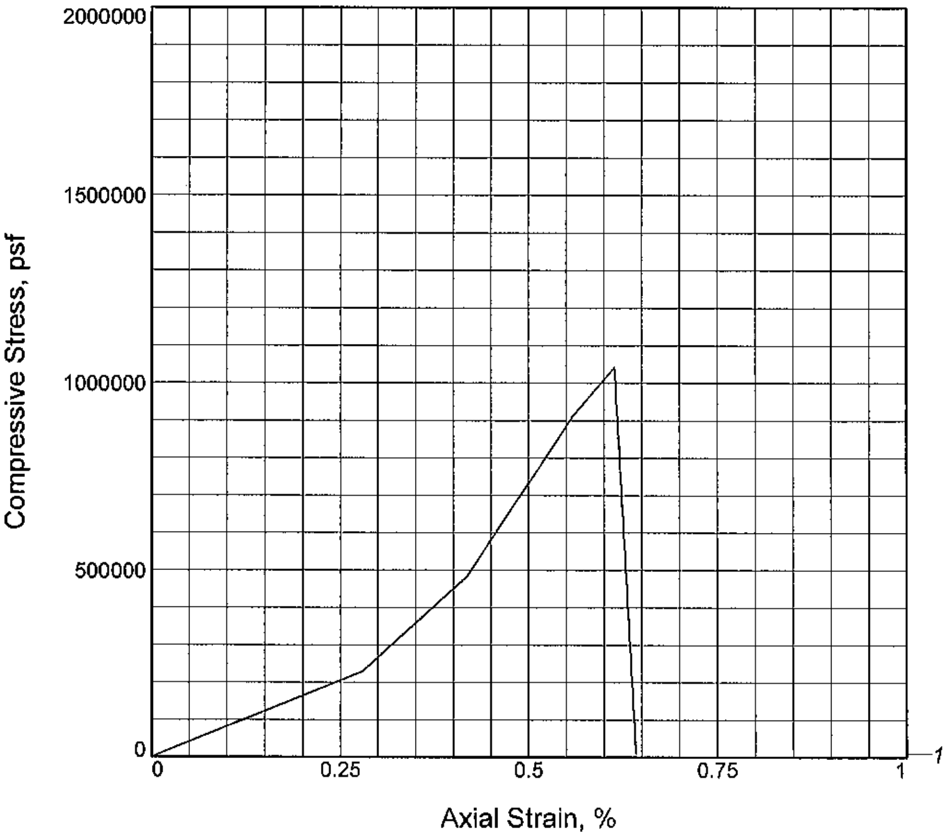
Project: BRENT SPENCE BRIDGE REPLACEMENT

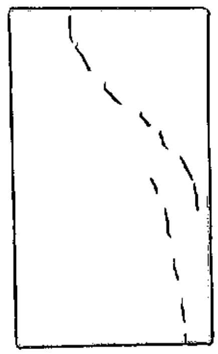
Source of Sample: R-3 **Depth:** 93.8-94.5'
Sample Number: 2

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1041933.4			
Undrained shear strength, psf	520966.7			
Failure strain, %	0.6			
Strain rate, in./min.	0.035			
Water content, %	1.7			
Wet density, pcf	165.2			
Dry density, pcf	162.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.580			
Height/diameter ratio	1.82			

Description: LIMESTONE & SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone and Shale

Project No.: N1105070
Date Sampled: 7-30-10
Remarks:
Lab No. 6017

Figure ______

Client: PARSONS BRINCKERHOFF

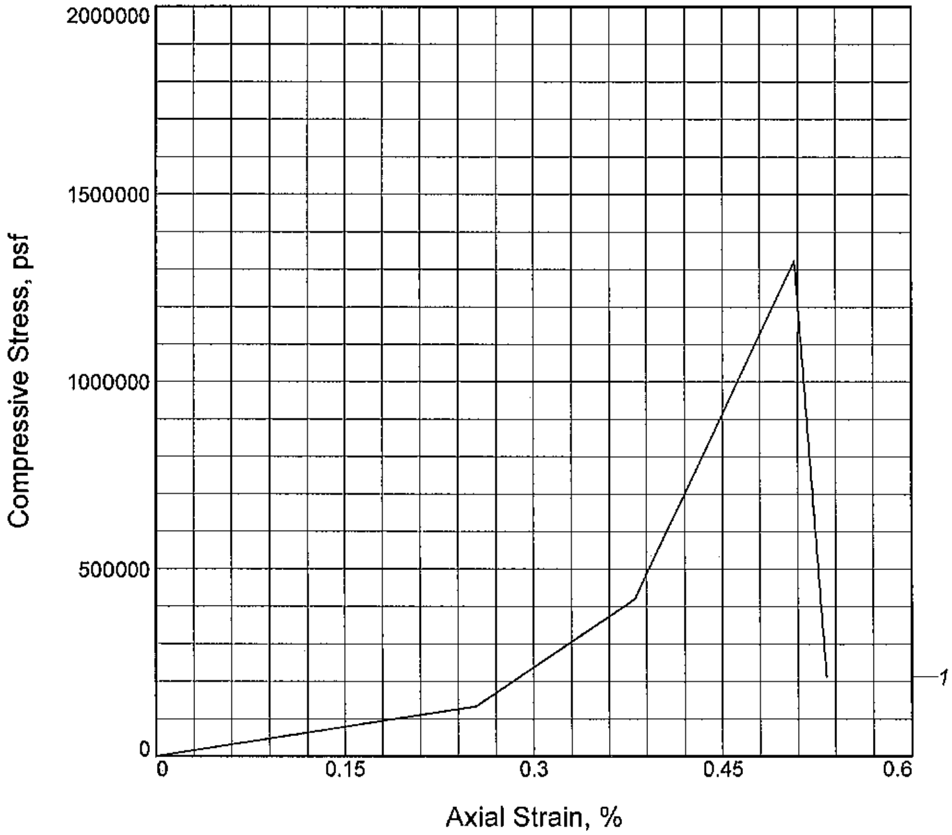
Project: BRENT SPENCE BRIDGE REPLACEMENT

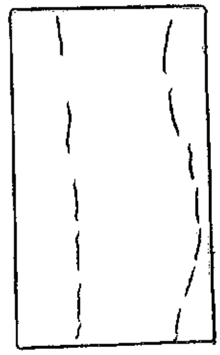
Source of Sample: R-3 **Depth:** 102.7
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1322957.4			
Undrained shear strength, psf	661478.7			
Failure strain, %	0.5			
Strain rate, in./min.	0.039			
Water content, %	2.2			
Wet density, pcf	167.7			
Dry density, pcf	164.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.940			
Height/diameter ratio	1.99			

Description: LIMESTONE & SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone & Shale

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6019

Figure ________

Client: PARSONS BRINCKERHOFF

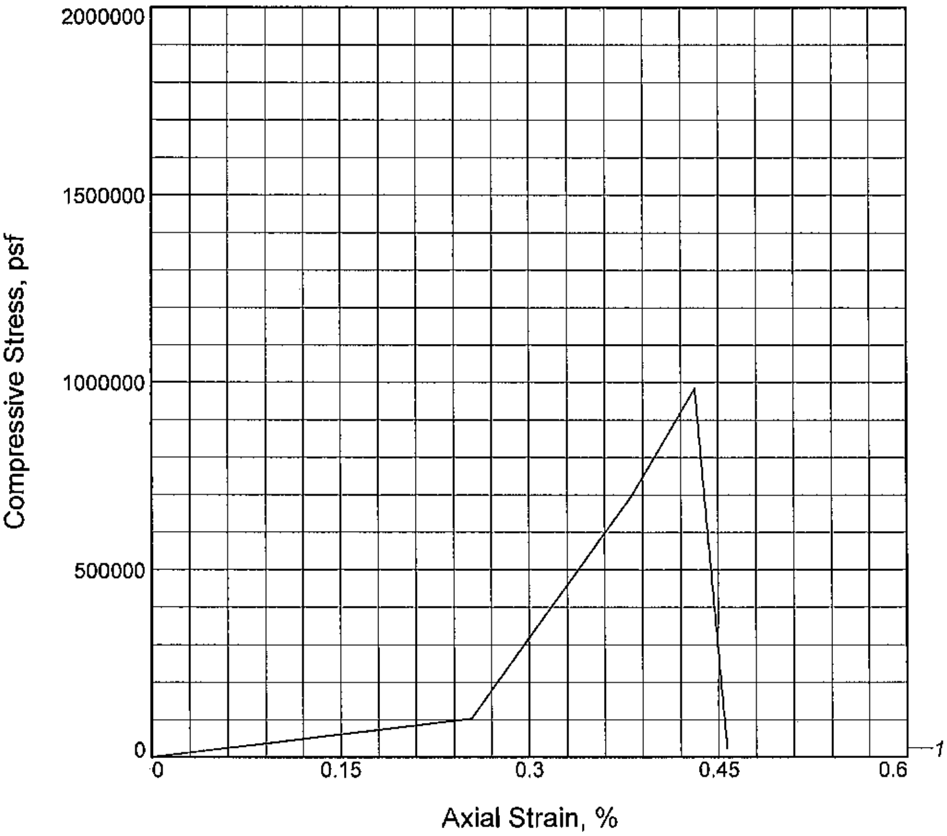
Project: BRENT SPENCE BRIDGE REPLACEMENT

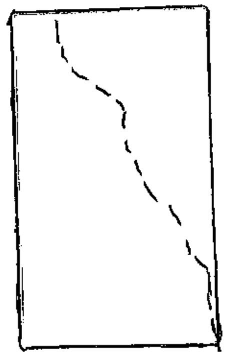
Source of Sample: R-3 **Depth:** 106.5-107.1
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	983924.2			
Undrained shear strength, psf	491962.1			
Failure strain, %	0.4			
Strain rate, in./min.	0.039			
Water content, %	0.9			
Wet density, pcf	165.8			
Dry density, pcf	164.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.940			
Height/diameter ratio	2.00			

Description: LIMESTONE & SHALE

LL =	PL =	PI =	Assumed GS=	Type: Limestone & Shale

Project No.: N1105070

Date Sampled: 8-3-10

Remarks:
Lab No. 6022

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

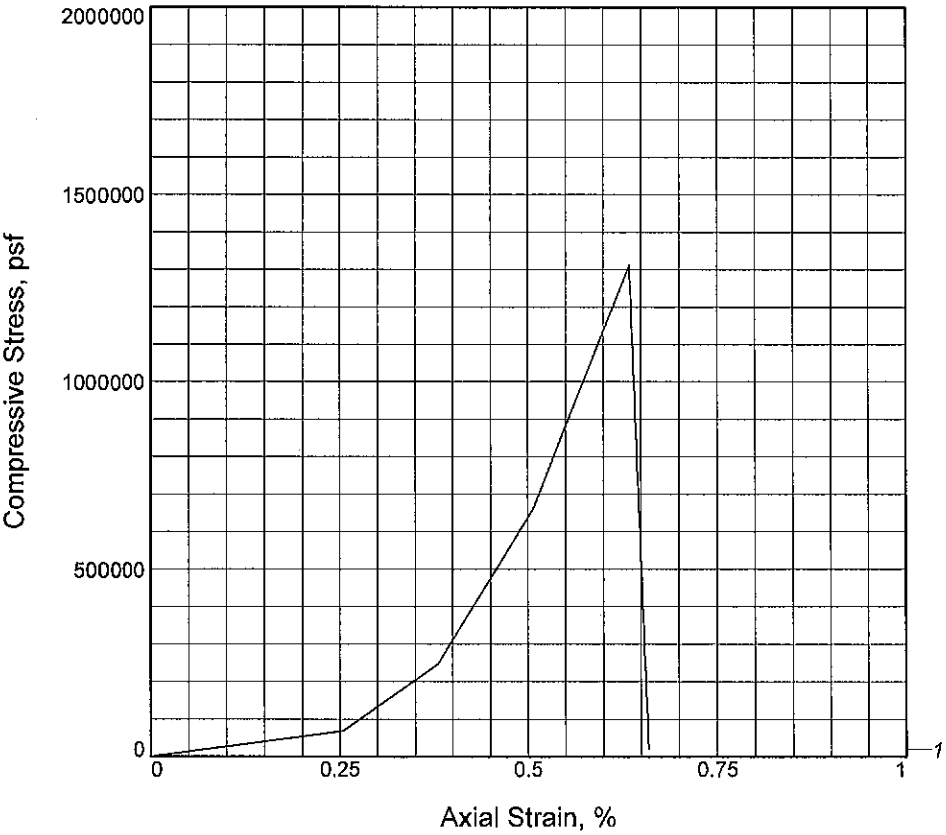
Source of Sample: R-3 **Depth:** 123.8-124.7'
Sample Number: 9

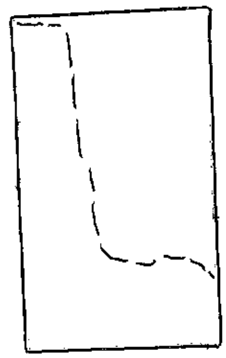
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1310334.6			
Undrained shear strength, psf	655167.3			
Failure strain, %	0.6			
Strain rate, in./min.	0.039			
Water content, %	0.5			
Wet density, pcf	166.3			
Dry density, pcf	165.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.940			
Height/diameter ratio	2.00			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6024

Figure ________

Client: PARSONS BRINCKERHOFF

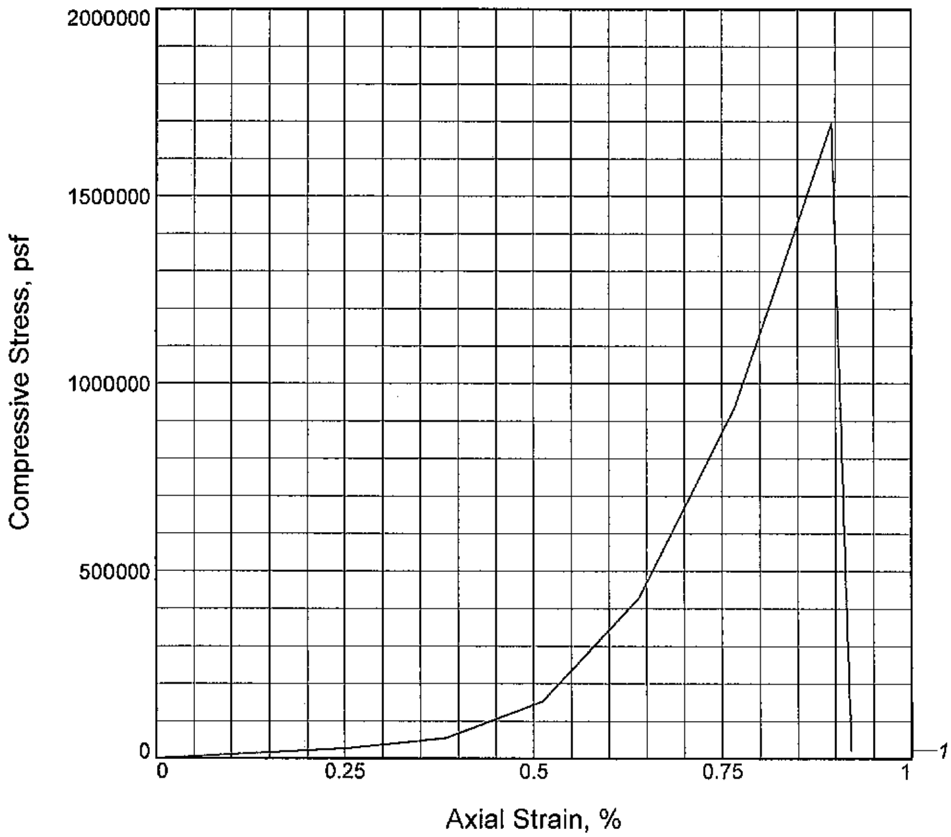
Project: BRENT SPENCE BRIDGE REPLACEMENT

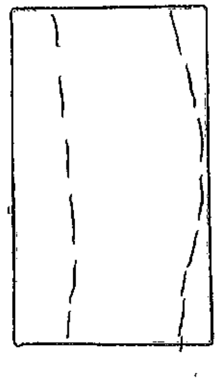
Source of Sample: R-3 **Depth:** 140-140.5'
Sample Number: 11

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1694430.2			
Undrained shear strength, psf	847215.1			
Failure strain, %	0.9			
Strain rate, in./min.	0.039			
Water content, %	1.1			
Wet density, pcf	167.8			
Dry density, pcf	165.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.910			
Height/diameter ratio	1.98			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6025

Figure ________

Client: PARSONS BRINCKERHOFF

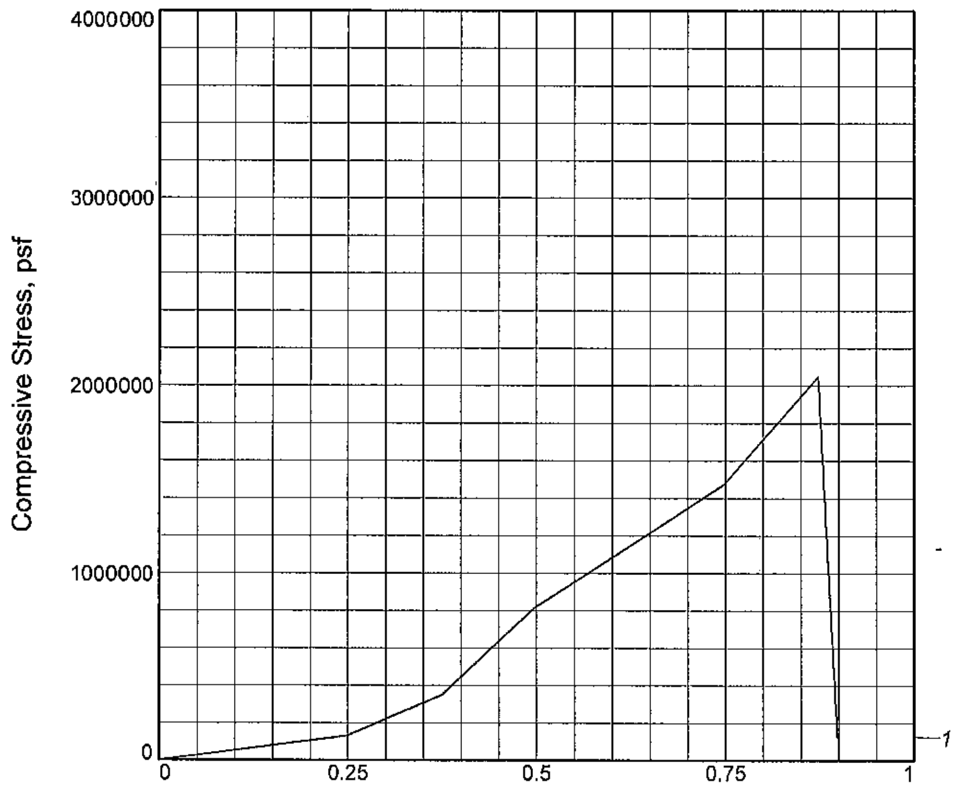
Project: BRENT SPENCE BRIDGE REPLACEMENT

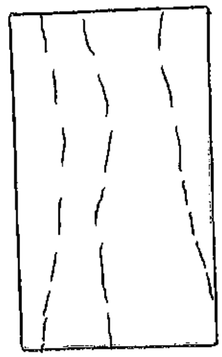
Source of Sample: R-3 **Depth:** 145.5-146'
Sample Number: 12

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2048572.8			
Undrained shear strength, psf	1024286.4			
Failure strain, %	0.9			
Strain rate, in./min.	0.040			
Water content, %	0.5			
Wet density, pcf	168.3			
Dry density, pcf	167.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.010			
Height/diameter ratio	2.04			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-3-10
Remarks:
Lab No. 6026

Figure ________

Client: PARSONS BRINCKERHOFF

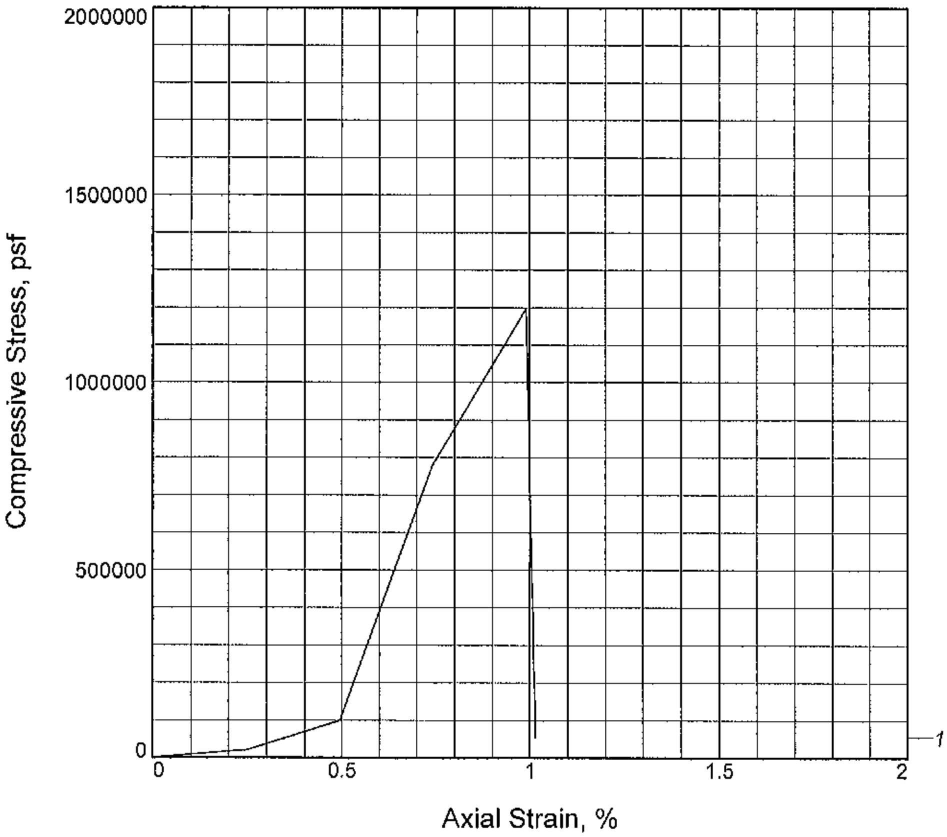
Project: BRENT SPENCE BRIDGE REPLACEMENT

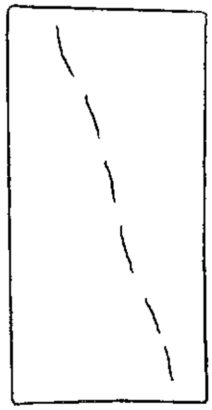
Source of Sample: R-3 **Depth:** 157.3-158'
Sample Number: 13

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1198015.0			
Undrained shear strength, psf	599007.5			
Failure strain, %	1.0			
Strain rate, in./min.	0.040			
Water content, %	4.2			
Wet density, pcf	162.0			
Dry density, pcf	155.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	4.040			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6070

Figure ________

Client: PARSONS BRINCKERHOFF

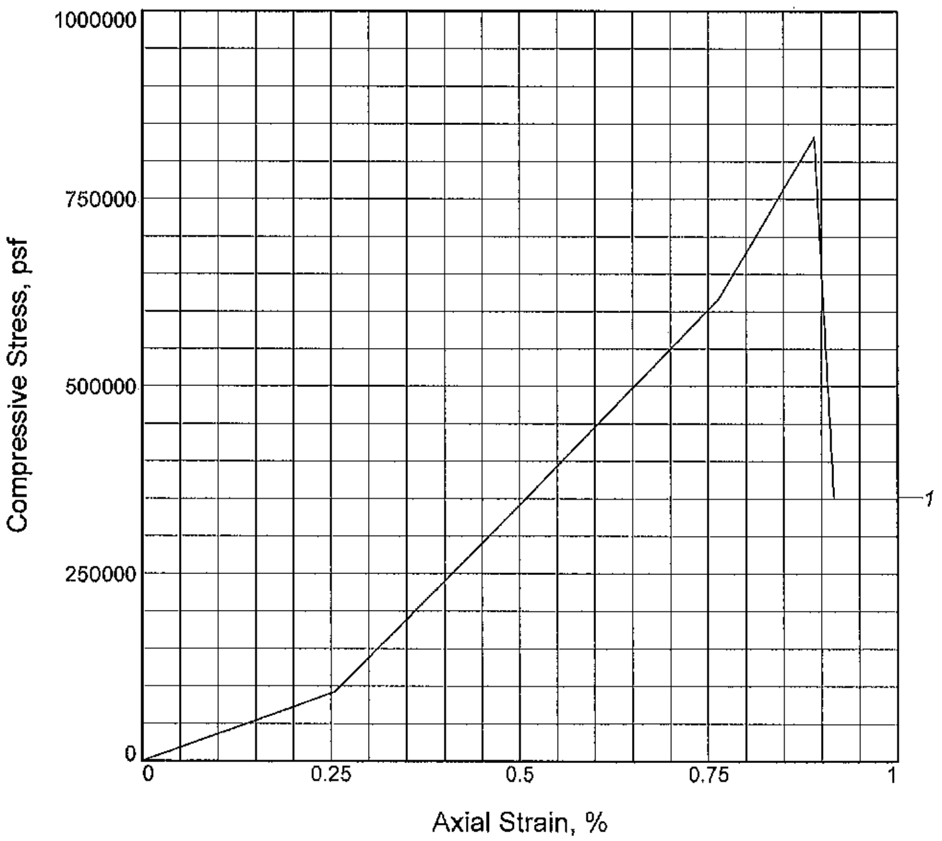
Project: BRENT SPENCE BRIDGE REPLACEMENT

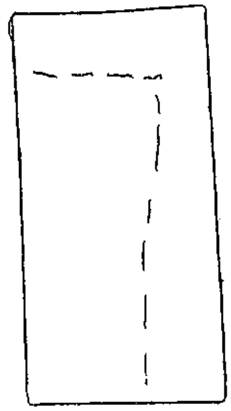
Source of Sample: R-4 **Depth:** 90.5-91.0'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	832099.0			
Undrained shear strength, psf	416049.5			
Failure strain, %	0.9			
Strain rate, in./min.	0.039			
Water content, %	4.2			
Wet density, pcf	159.3			
Dry density, pcf	152.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.930			
Height/diameter ratio	1.97			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6072

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

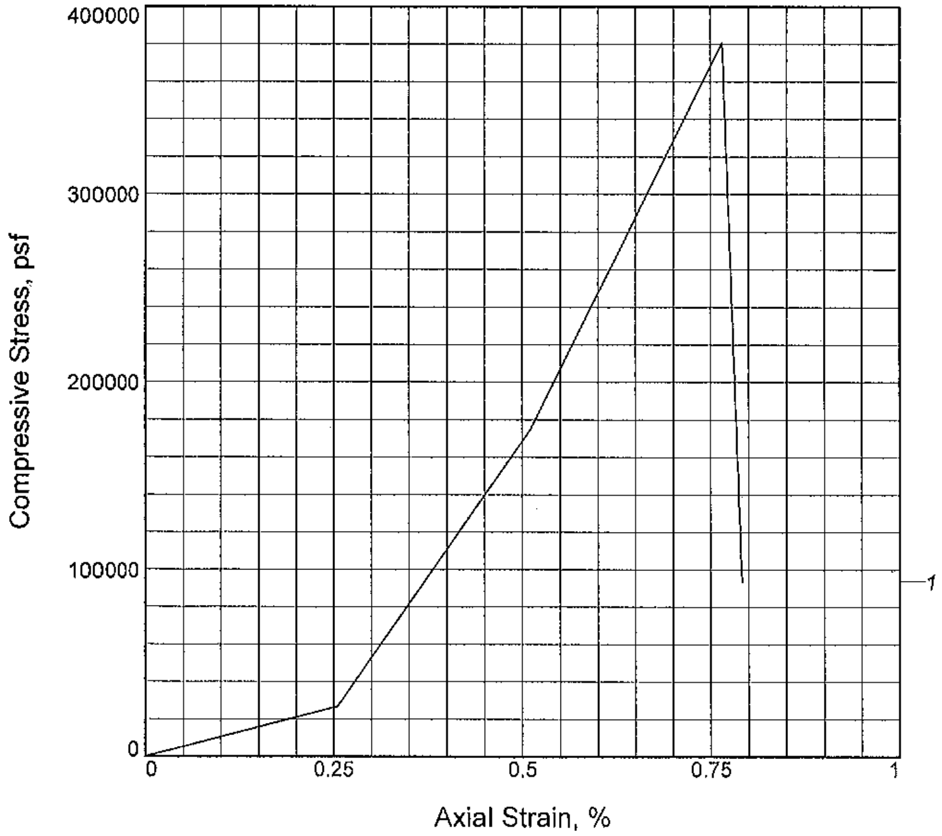
Source of Sample: R-4 **Depth:** 95.5-96'
Sample Number: 3

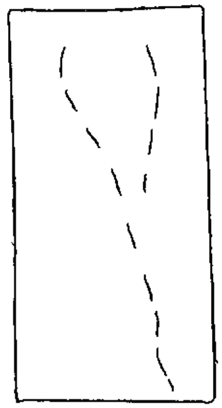
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	380693.0			
Undrained shear strength, psf	190346.5			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	1.9			
Wet density, pcf	161.3			
Dry density, pcf	158.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.920			
Height/diameter ratio	1.97			

Description: LIMESTONE AND SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone and Shale

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6074

Figure ________

Client: PARSONS BRINCKERHOFF

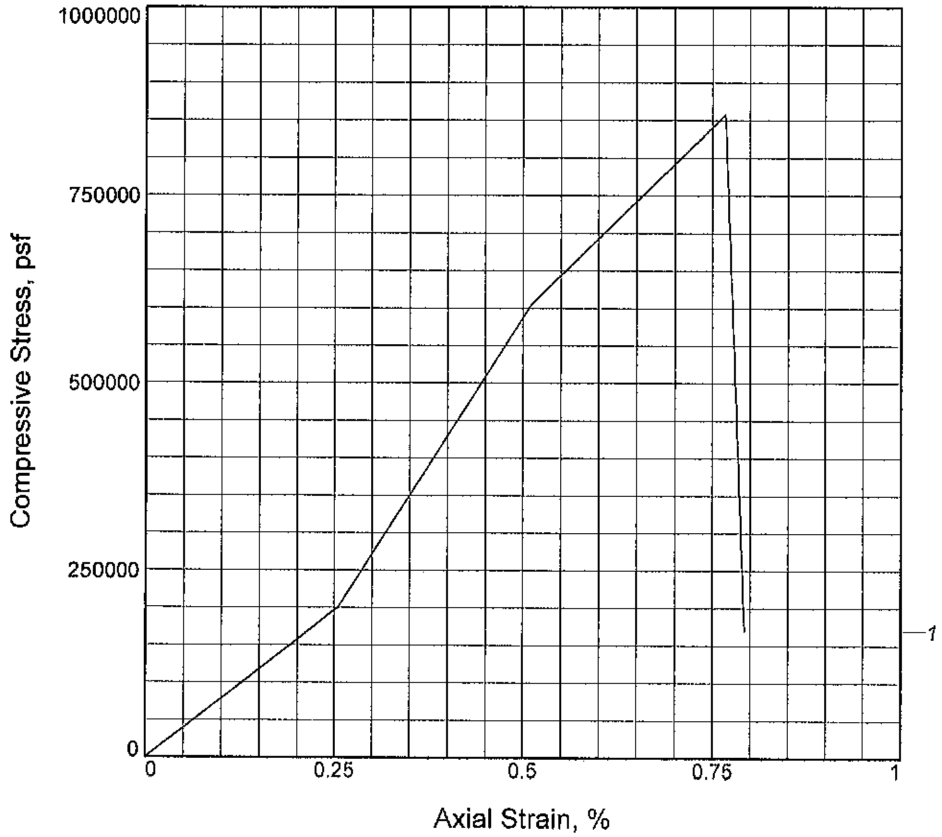
Project: BRENT SPENCE BRIDGE REPLACEMENT

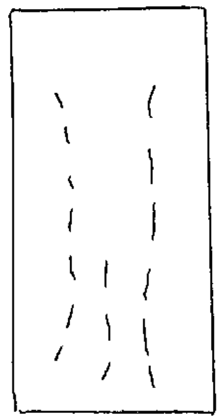
Source of Sample: R-4 **Depth:** 102.8-103.3'
Sample Number: 5

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	857943.7			
Undrained shear strength, psf	428971.8			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	1.5			
Wet density, pcf	161.9			
Dry density, pcf	159.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.910			
Height/diameter ratio	1.96			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6075

Figure ________

Client: PARSONS BRINCKERHOFF

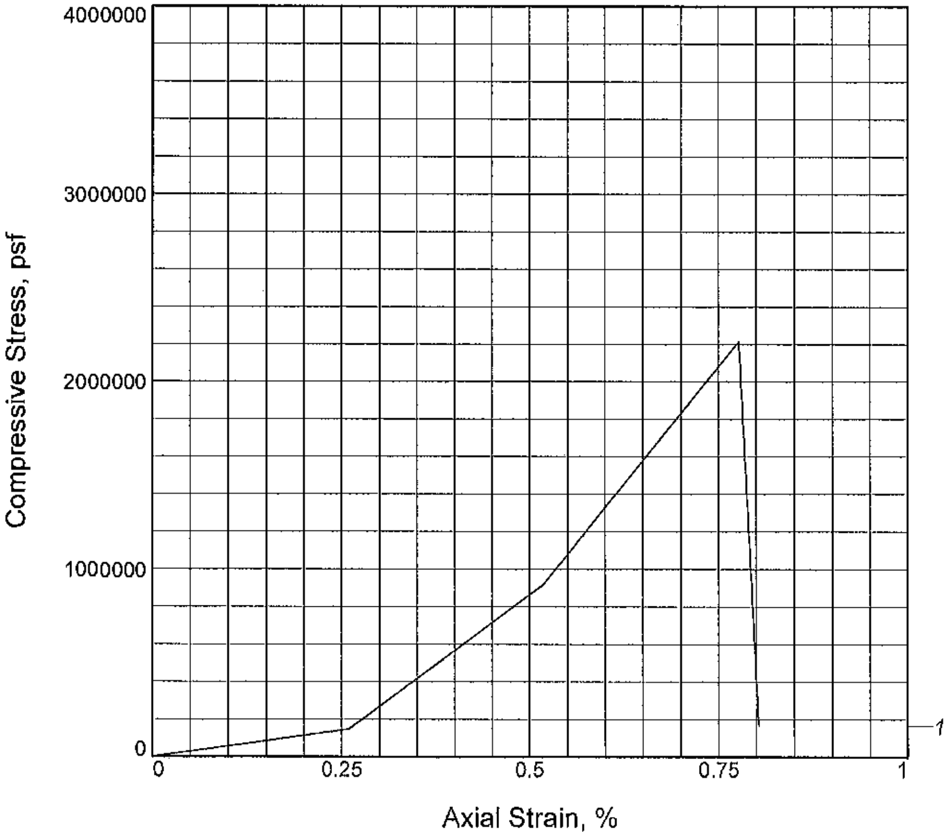
Project: BRENT SPENCE BRIDGE REPLACEMENT

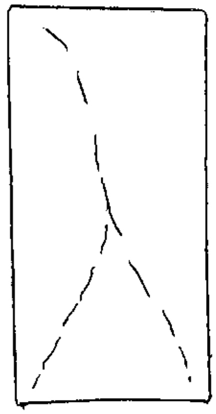
Source of Sample: R-4 **Depth:** 111.3-111.9'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2216031.5			
Undrained shear strength, psf	1108015.8			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	1.1			
Wet density, pcf	161.2			
Dry density, pcf	159.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.860			
Height/diameter ratio	1.93			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date Sampled: 8-23-10

Remarks:
Lab No. 6077

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

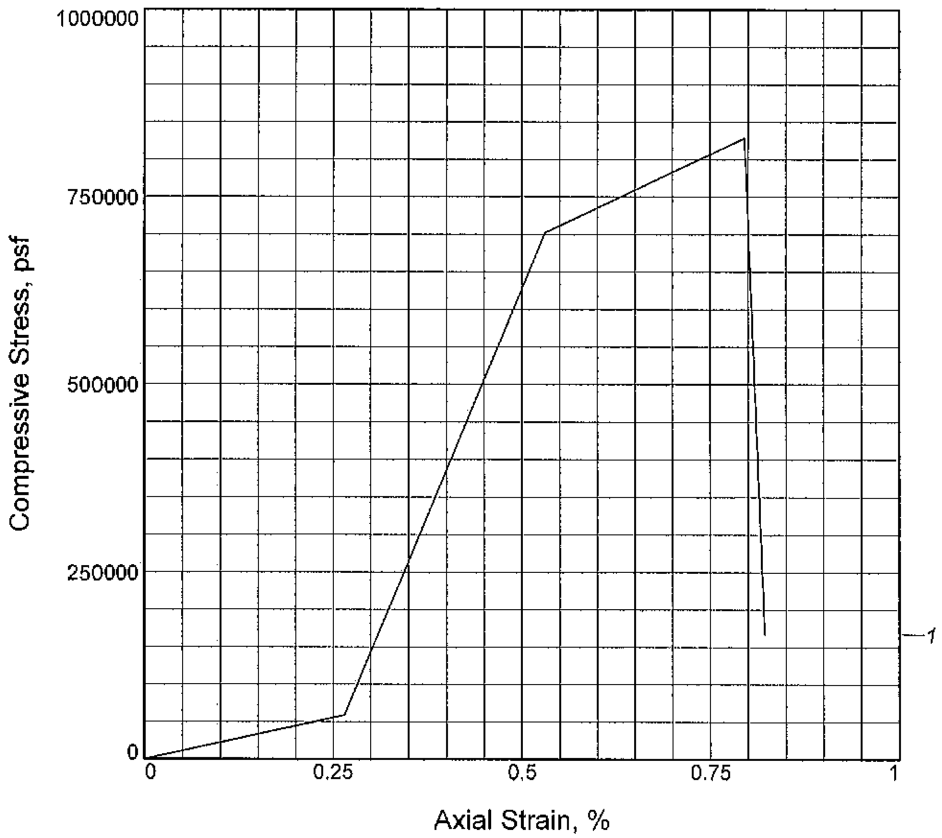
Source of Sample: R-4 **Depth:** 121.9-122.3'
Sample Number: 8

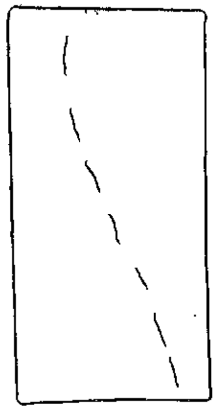
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	828577.7			
Undrained shear strength, psf	414288.9			
Failure strain, %	0.8			
Strain rate, in./min.	0.037			
Water content, %	1.5			
Wet density, pcf	160.7			
Dry density, pcf	158.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.770			
Height/diameter ratio	1.89			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6078

Figure ________

Client: PARSONS BRINCKERHOFF

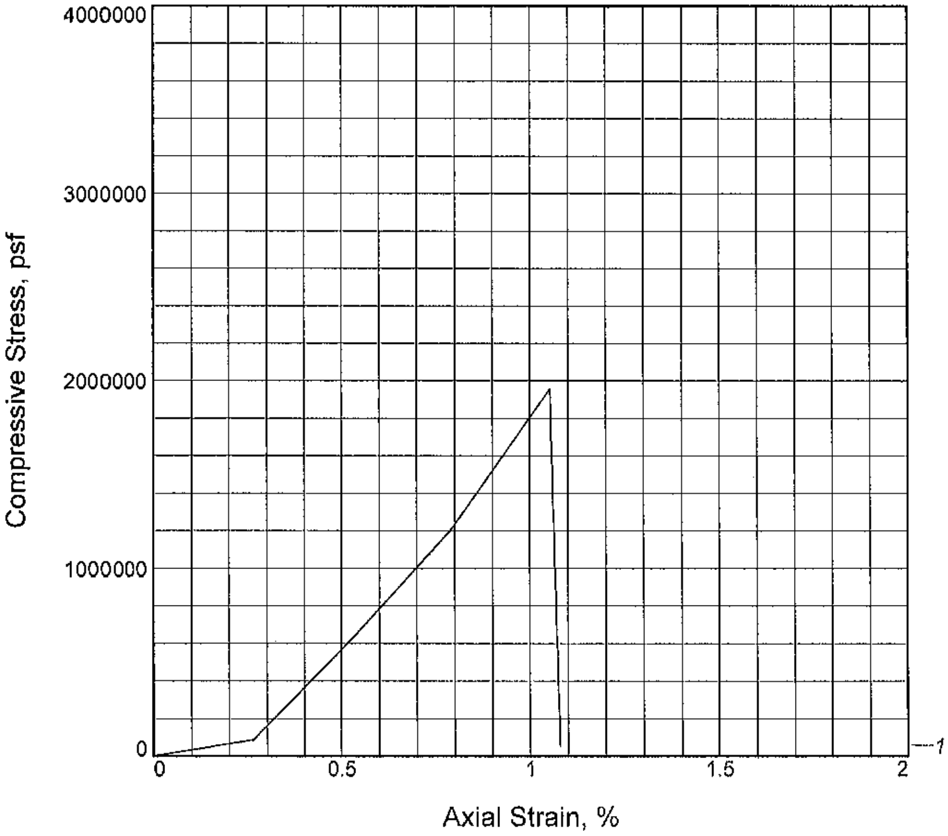
Project: BRENT SPENCE BRIDGE REPLACEMENT

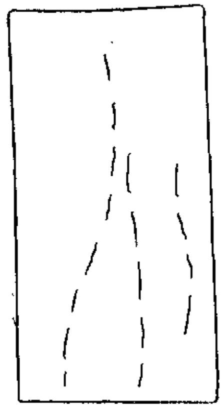
Source of Sample: R-4 **Depth:** 129.6-130'
Sample Number: 9

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1956363.0			
Undrained shear strength, psf	978181.5			
Failure strain, %	1.1			
Strain rate, in./min.	0.038			
Water content, %	0.4			
Wet density, pcf	165.8			
Dry density, pcf	165.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.800			
Height/diameter ratio	1.91			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6080

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

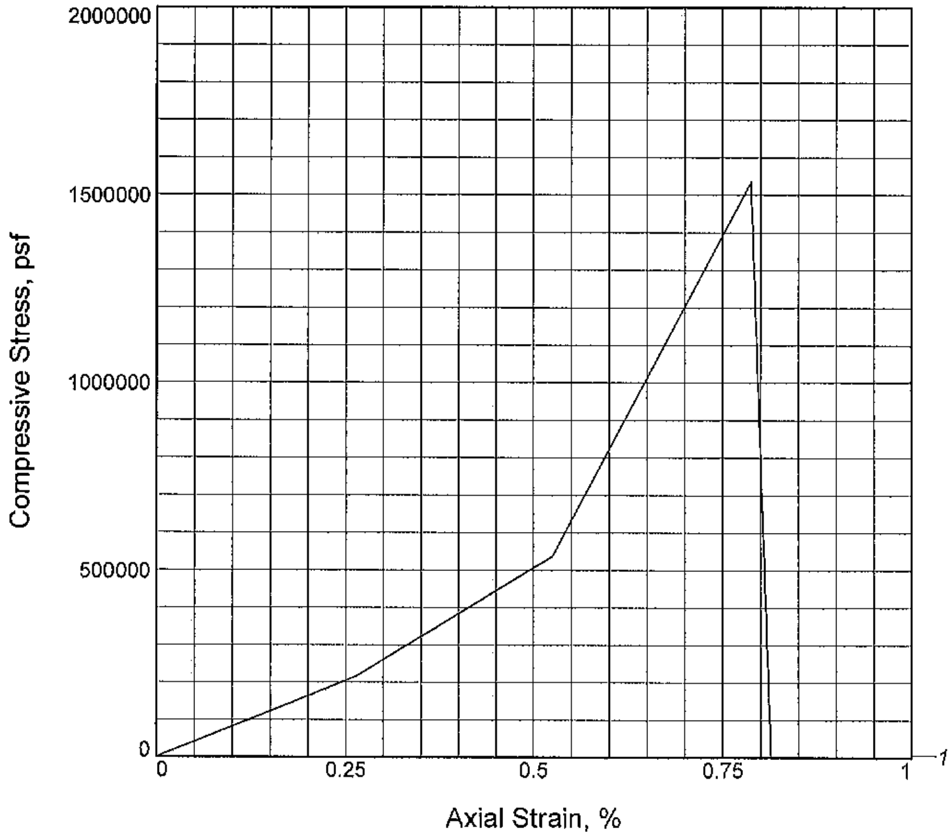
Source of Sample: R-4 **Depth:** 140.6-141.1'
Sample Number: 11

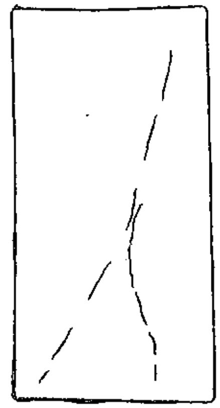
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1534100.3			
Undrained shear strength, psf	767050.1			
Failure strain, %	0.8			
Strain rate, in./min.	0.038			
Water content, %	1.1			
Wet density, pcf	162.0			
Dry density, pcf	160.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.990			
Specimen height, in.	3.810			
Height/diameter ratio	1.91			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070

Date Sampled: 8-23-10

Remarks:
Lab No. 6082

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-4 **Depth:** 152.8-153.6'

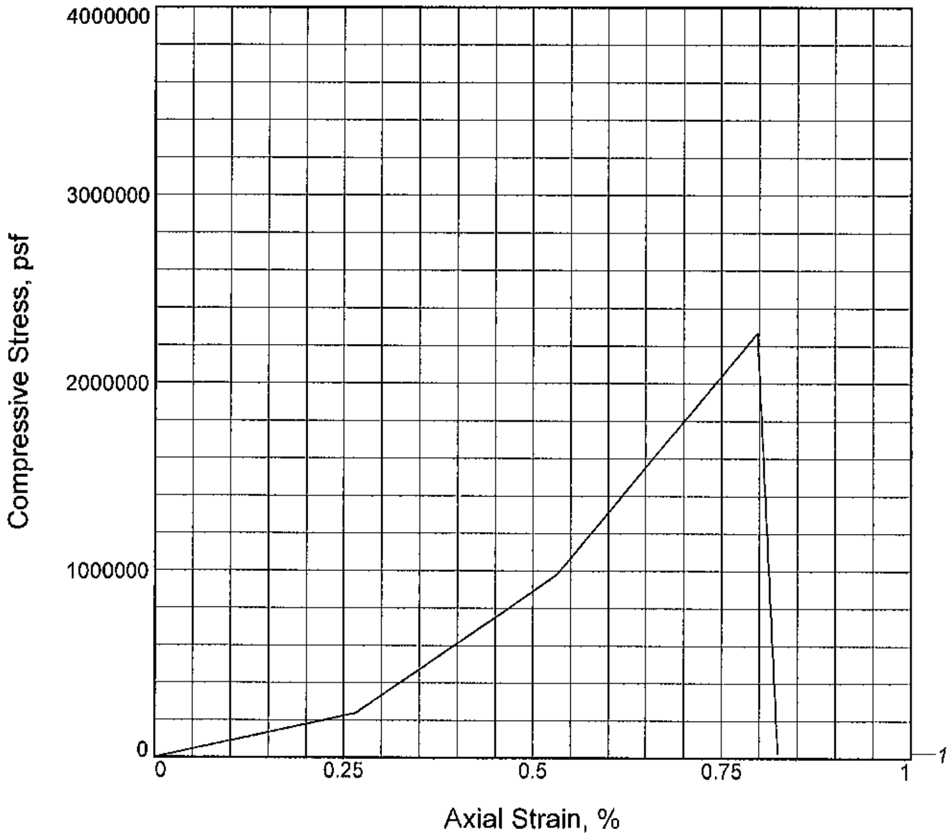
Sample Number: 13

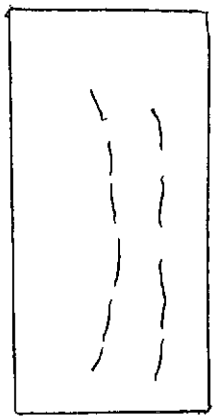
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2269771.2			
Undrained shear strength, psf	1134885.6			
Failure strain, %	0.8			
Strain rate, in./min.	0.037			
Water content, %	0.5			
Wet density, pcf	162.3			
Dry density, pcf	161.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.000			
Specimen height, in.	3.760			
Height/diameter ratio	1.88			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 8-23-10
Remarks:
Lab No. 6084

Figure ________

Client: PARSONS BRINCKERHOFF

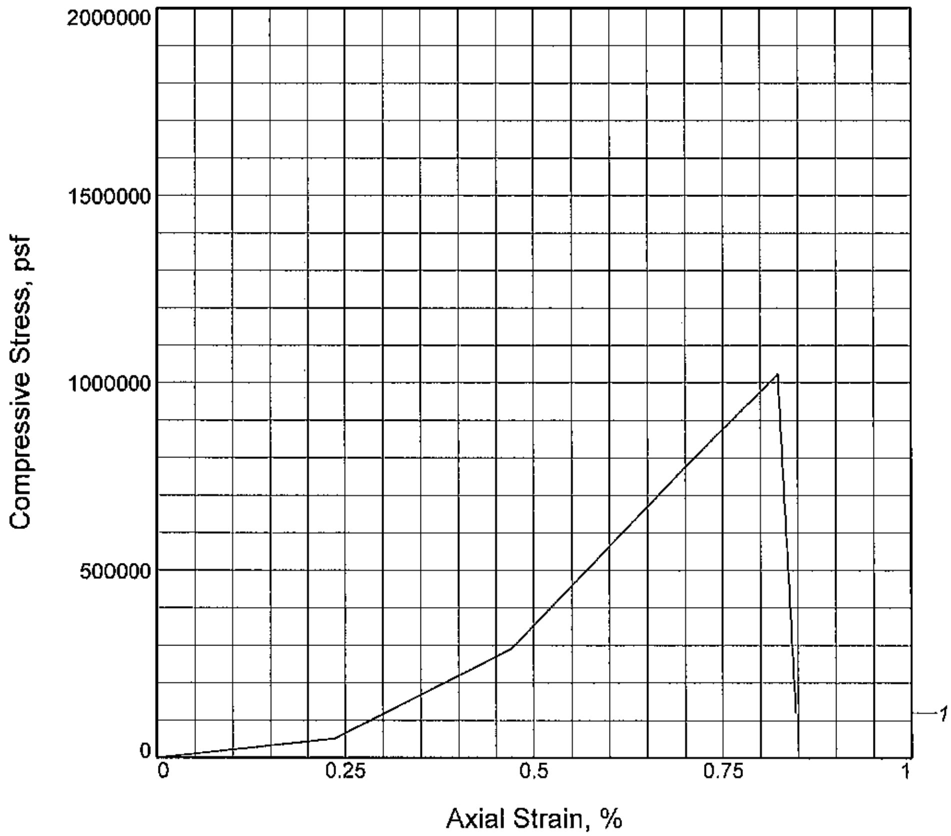
Project: BRENT SPENCE BRIDGE REPLACEMENT

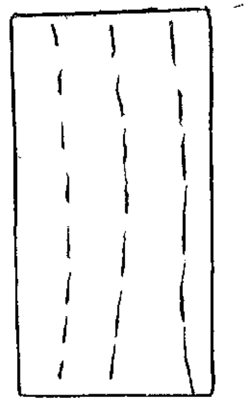
Source of Sample: R-4 **Depth:** 159.6-160.5'
Sample Number: 15

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1022251.4			
Undrained shear strength, psf	511125.7			
Failure strain, %	0.8			
Strain rate, in./min.	0.042			
Water content, %	2.9			
Wet density, pcf	165.0			
Dry density, pcf	160.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.964			
Specimen height, in.	4.250			
Height/diameter ratio	2.16			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5840

Figure ______

Client: PARSONS BRINCKERHOFF

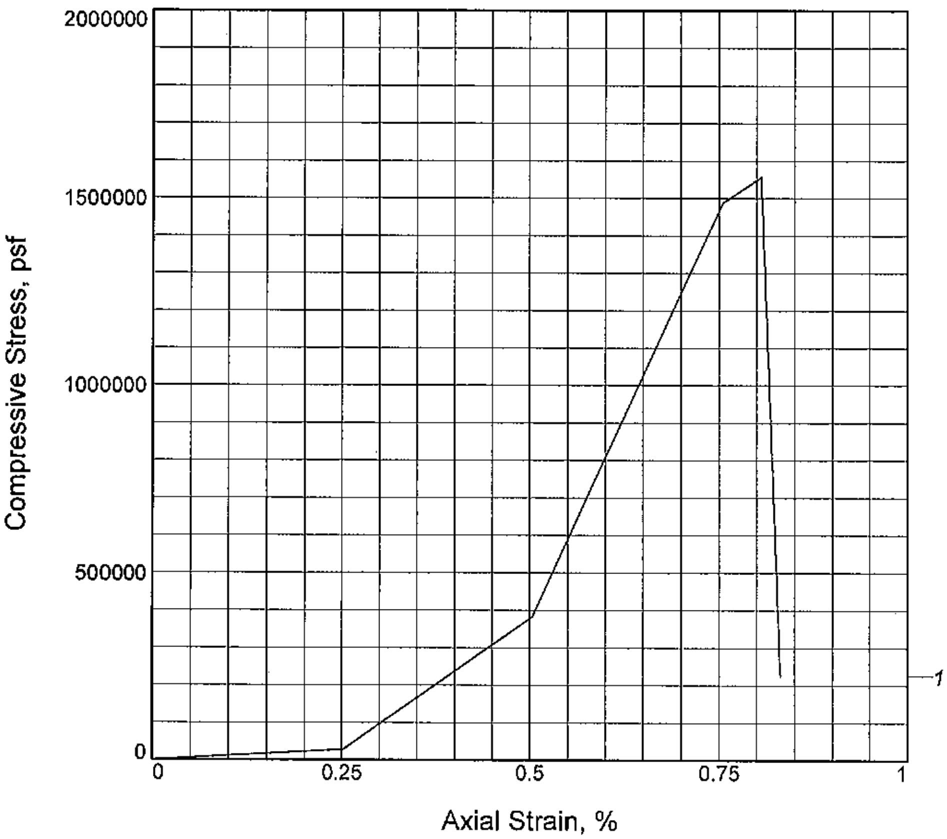
Project: BRENT SPENCE BRIDGE REPLACEMENT

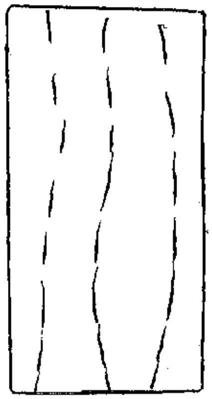
Source of Sample: R-5 **Depth:** 85.2-85.7'
Sample Number: 1/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1556479.7			
Undrained shear strength, psf	778239.9			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.3			
Wet density, pcf	167.1			
Dry density, pcf	166.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.973			
Specimen height, in.	3.970			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5841

Figure ________

Client: PARSONS BRINCKERHOFF

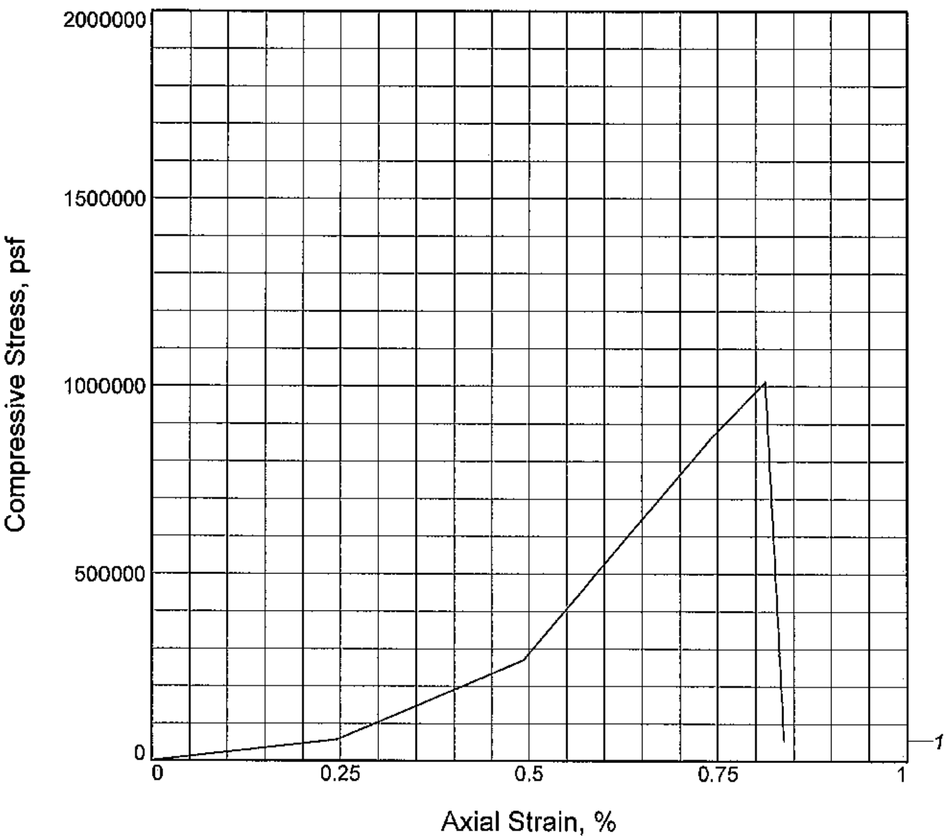
Project: BRENT SPENCE BRIDGE REPLACEMENT

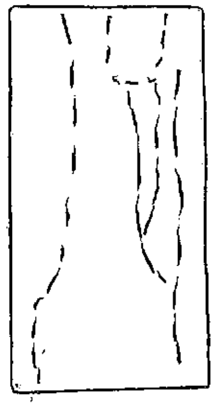
Source of Sample: R-5 **Depth:** 86.4-86.8'
Sample Number: 1/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1011411.0			
Undrained shear strength, psf	505705.5			
Failure strain, %	0.8			
Strain rate, in./min.	0.040			
Water content, %	0.5			
Wet density, pcf	166.3			
Dry density, pcf	165.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.060			
Height/diameter ratio	2.06			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10

Remarks:
Lab No. 5842

Figure ________

Client: PARSONS BRINCKERHOFF

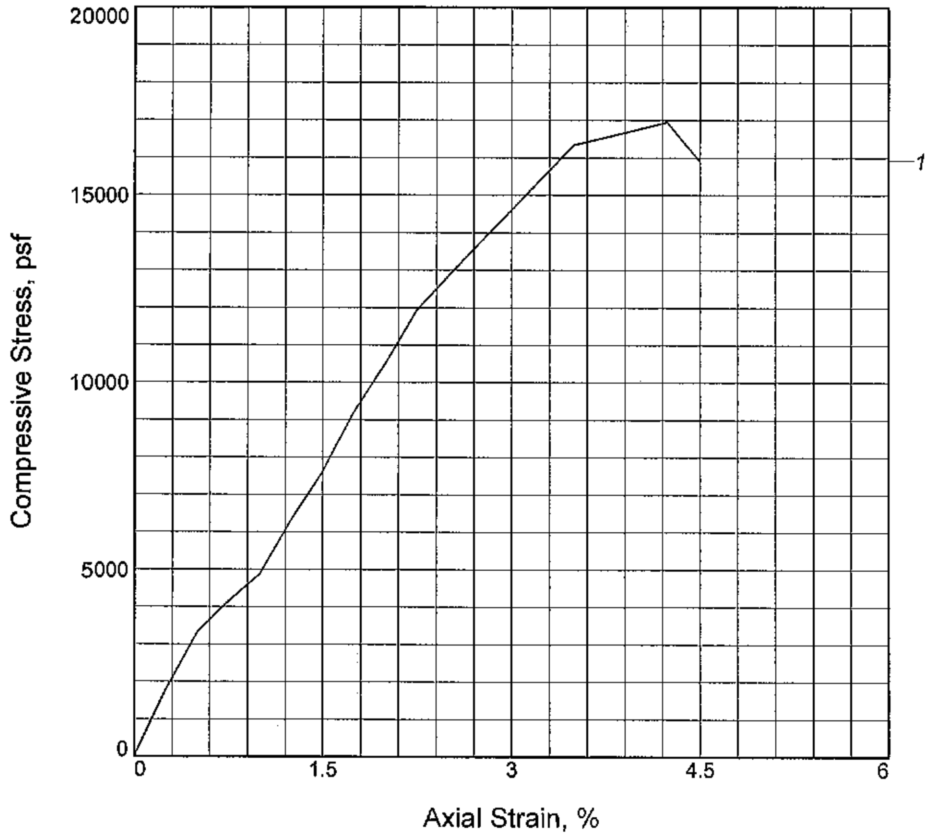
Project: BRENT SPENCE BRIDGE REPLACEMENT

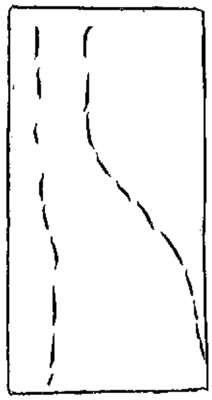
Source of Sample: R-5 **Depth:** 90.1-90.8'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	16945.6			
Undrained shear strength, psf	8472.8			
Failure strain, %	4.2			
Strain rate, in./min.	0.040			
Water content, %	8.6			
Wet density, pcf	150.0			
Dry density, pcf	138.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.950			
Specimen height, in.	4.010			
Height/diameter ratio	2.06			

Description: SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070

Date Sampled: 7-16-10

Remarks:
Lab No. 5843

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-5 **Depth:** 92.2-92.8'

Sample Number: 2/NQ

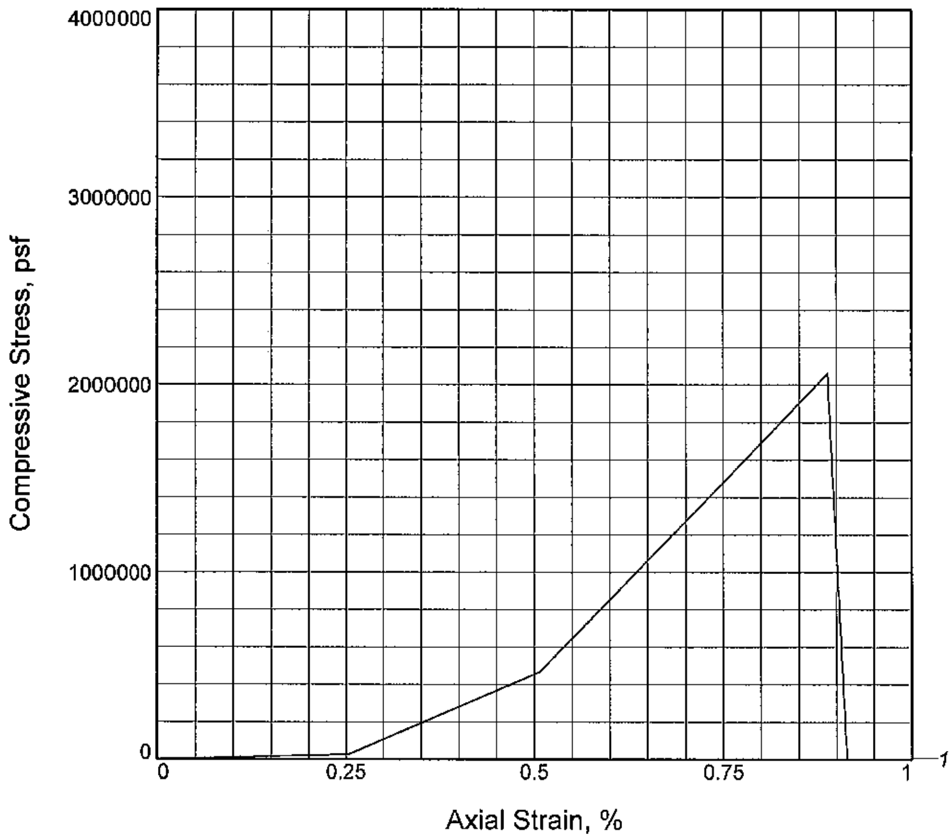
UNCONFINED COMPRESSION TEST

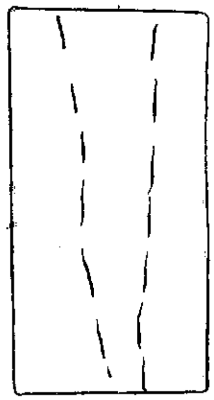
H.C. Nutting
A Terracon Company

Figure ________

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2062678.8			
Undrained shear strength, psf	1031339.4			
Failure strain, %	0.9			
Strain rate, in./min.	0.039			
Water content, %	0.6			
Wet density, pcf	167.4			
Dry density, pcf	166.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	3.940			
Height/diameter ratio	1.99			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5844

Figure ______

Client: PARSONS BRINCKERHOFF

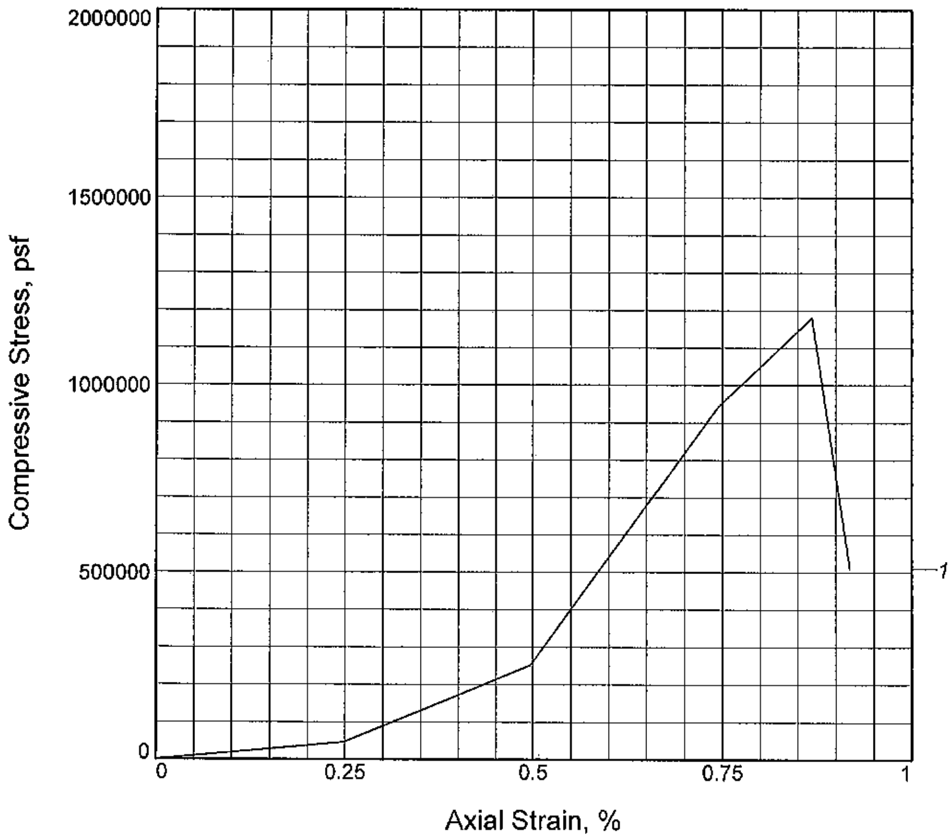
Project: BRENT SPENCE BRIDGE REPLACEMENT

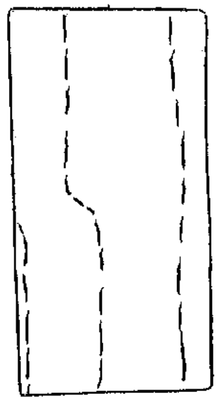
Source of Sample: R-5 **Depth:** 93-93.8'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1179728.7			
Undrained shear strength, psf	589864.4			
Failure strain, %	0.9			
Strain rate, in./min.	0.040			
Water content, %	0.5			
Wet density, pcf	166.9			
Dry density, pcf	166.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.968			
Specimen height, in.	4.030			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5845

Figure ________

Client: PARSONS BRINCKERHOFF

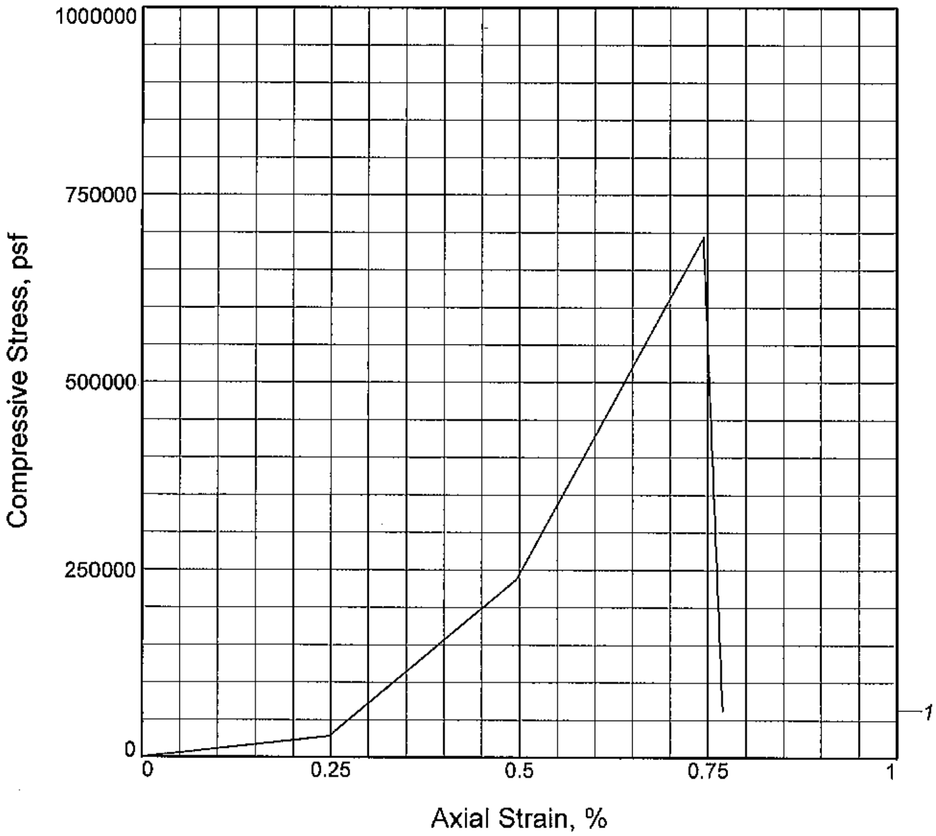
Project: BRENT SPENCE BRIDGE REPLACEMENT

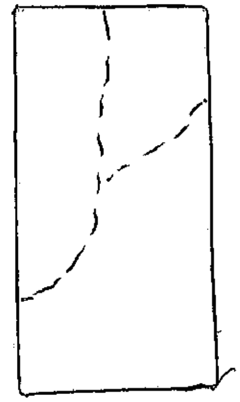
Source of Sample: R-5 **Depth:** 95-95.3'
Sample Number: 3/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	692912.3			
Undrained shear strength, psf	346456.2			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	1.5			
Wet density, pcf	167.8			
Dry density, pcf	165.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.975			
Specimen height, in.	4.030			
Height/diameter ratio	2.04			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5848

Figure ________

Client: PARSONS BRINCKERHOFF

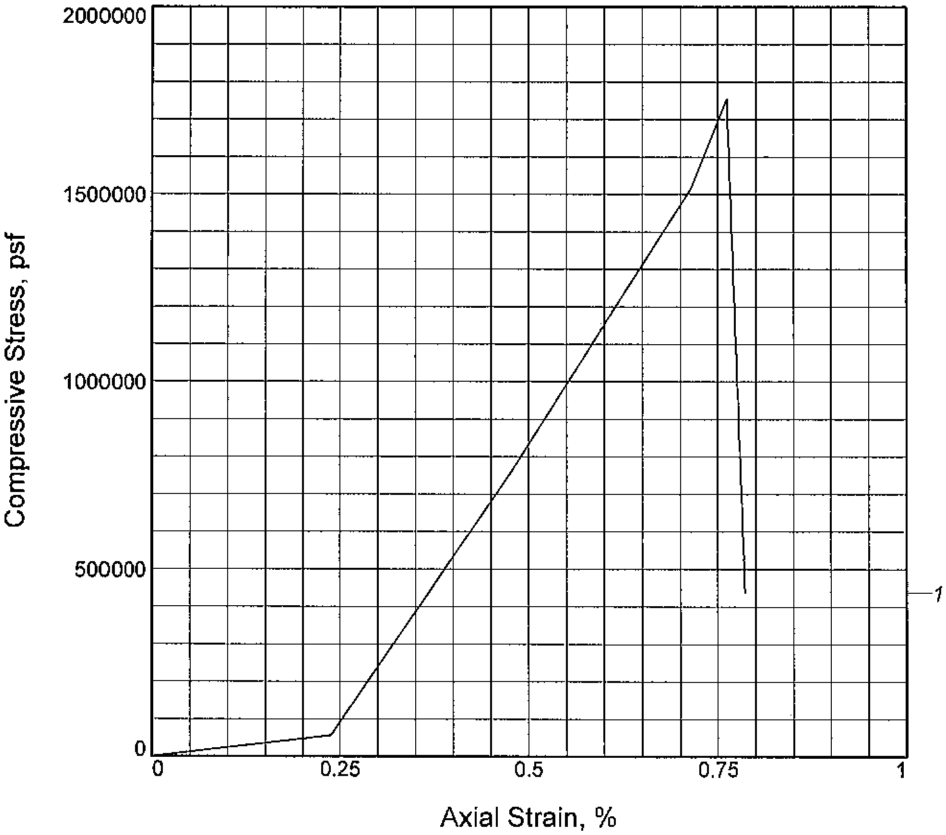
Project: BRENT SPENCE BRIDGE REPLACEMENT

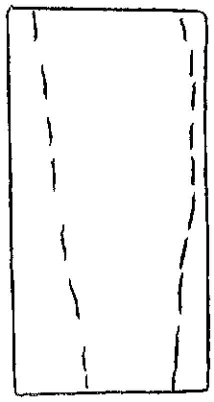
Source of Sample: R-5 **Depth:** 103-103.5'
Sample Number: 3/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1753704.3			
Undrained shear strength, psf	876852.2			
Failure strain, %	0.8			
Strain rate, in./min.	0.042			
Water content, %	0.5			
Wet density, pcf	165.1			
Dry density, pcf	164.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.977			
Specimen height, in.	4.200			
Height/diameter ratio	2.12			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-16-10
Remarks:
Lab No. 5853

Figure ________

Client: PARSONS BRINCKERHOFF

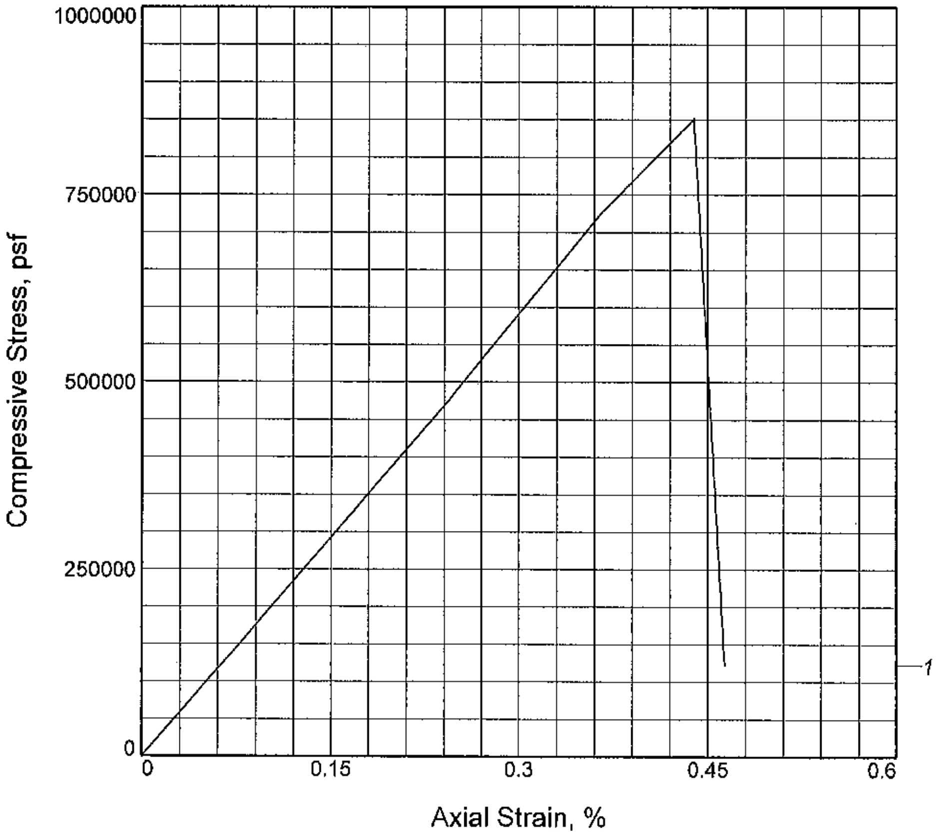
Project: BRENT SPENCE BRIDGE REPLACEMENT

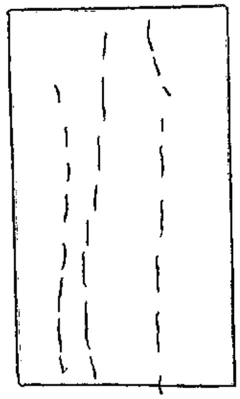
Source of Sample: R-5 **Depth:** 146.2-147'
Sample Number: 13/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	851152.9			
Undrained shear strength, psf	425576.4			
Failure strain, %	0.4			
Strain rate, in./min.	0.041			
Water content, %	0.6			
Wet density, pcf	165.5			
Dry density, pcf	164.5			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.100			
Height/diameter ratio	2.07			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5897

Figure ________

Client: PARSONS BRINCKERHOFF

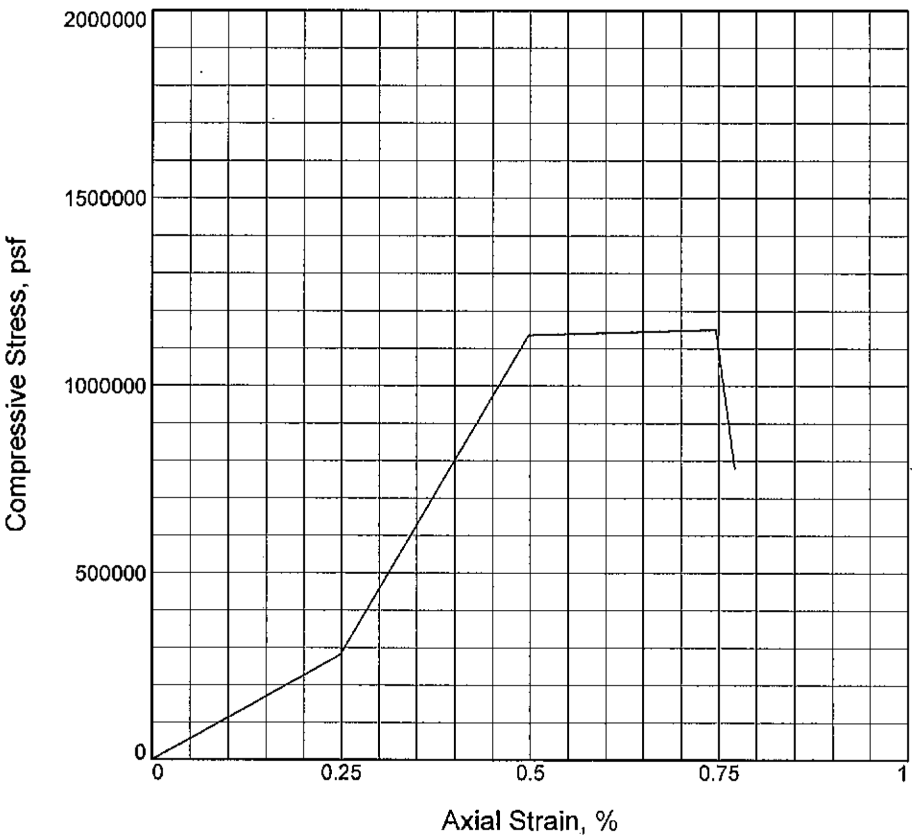
Project: BRENT SPENCE BRIDGE REPLACEMENT

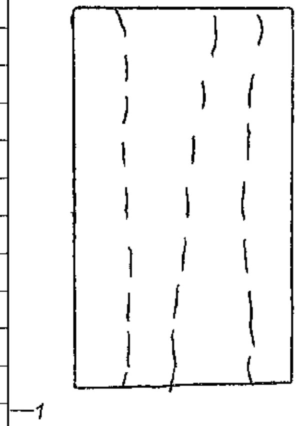
Source of Sample: R-6 **Depth:** 84.1-84.5'
Sample Number: 1/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1150291.5			
Undrained shear strength, psf	575145.7			
Failure strain, %	0.7			
Strain rate, in./min.	0.040			
Water content, %	0.7			
Wet density, pcf	167.9			
Dry density, pcf	166.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.020			
Height/diameter ratio	2.03			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5898

Figure ________

Client: PARSONS BRINCKERHOFF

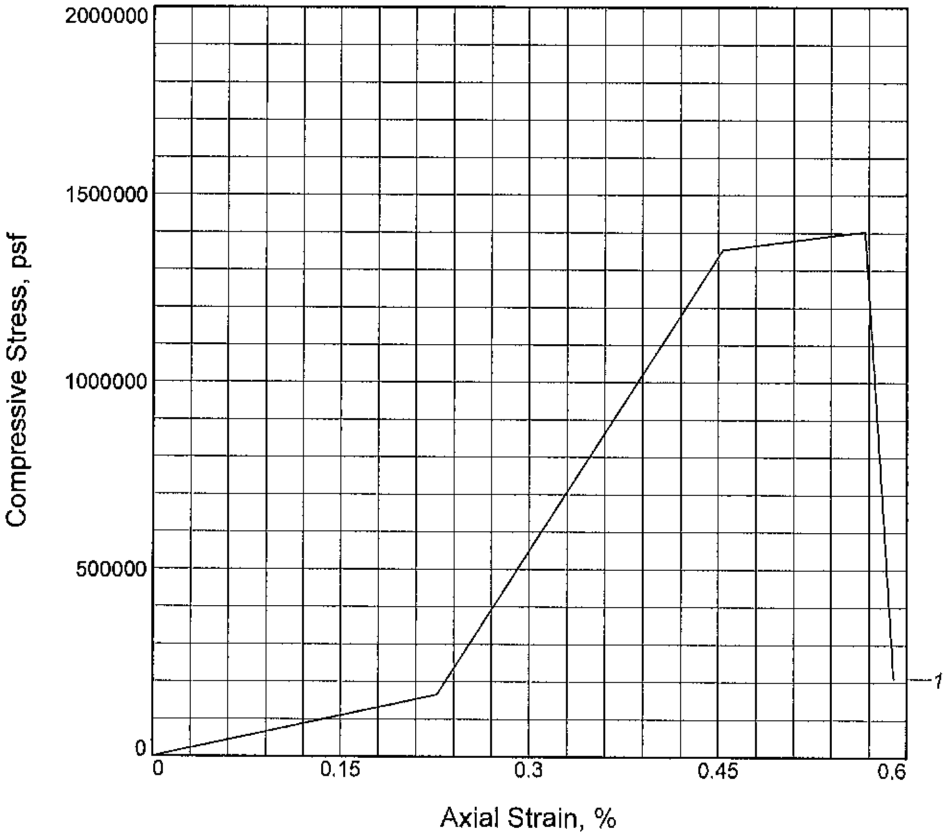
Project: BRENT SPENCE BRIDGE REPLACEMENT

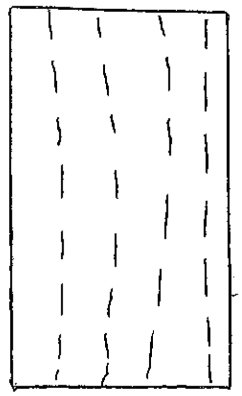
Source of Sample: R-6 **Depth:** 88.5-89'
Sample Number: 2/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1403296.0			
Undrained shear strength, psf	701648.0			
Failure strain, %	0.6			
Strain rate, in./min.	0.044			
Water content, %	0.1			
Wet density, pcf	168.6			
Dry density, pcf	168.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.410			
Height/diameter ratio	2.23			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5901

Figure ________

Client: PARSONS BRINCKERHOFF

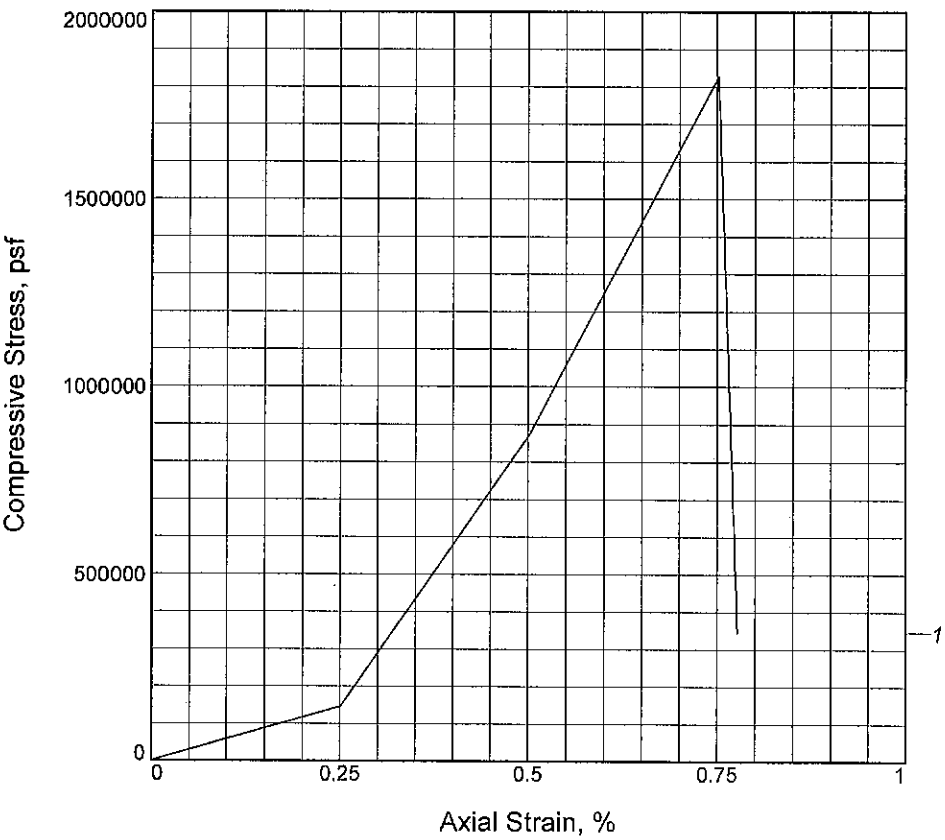
Project: BRENT SPENCE BRIDGE REPLACEMENT

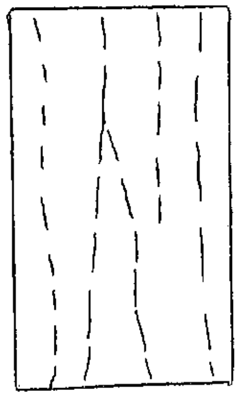
Source of Sample: R-6 **Depth:** 94.5-94.9'
Sample Number: 3/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1828127.1			
Undrained shear strength, psf	914063.6			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.1			
Wet density, pcf	167.8			
Dry density, pcf	167.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.990			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5903

Figure ________

Client: PARSONS BRINCKERHOFF

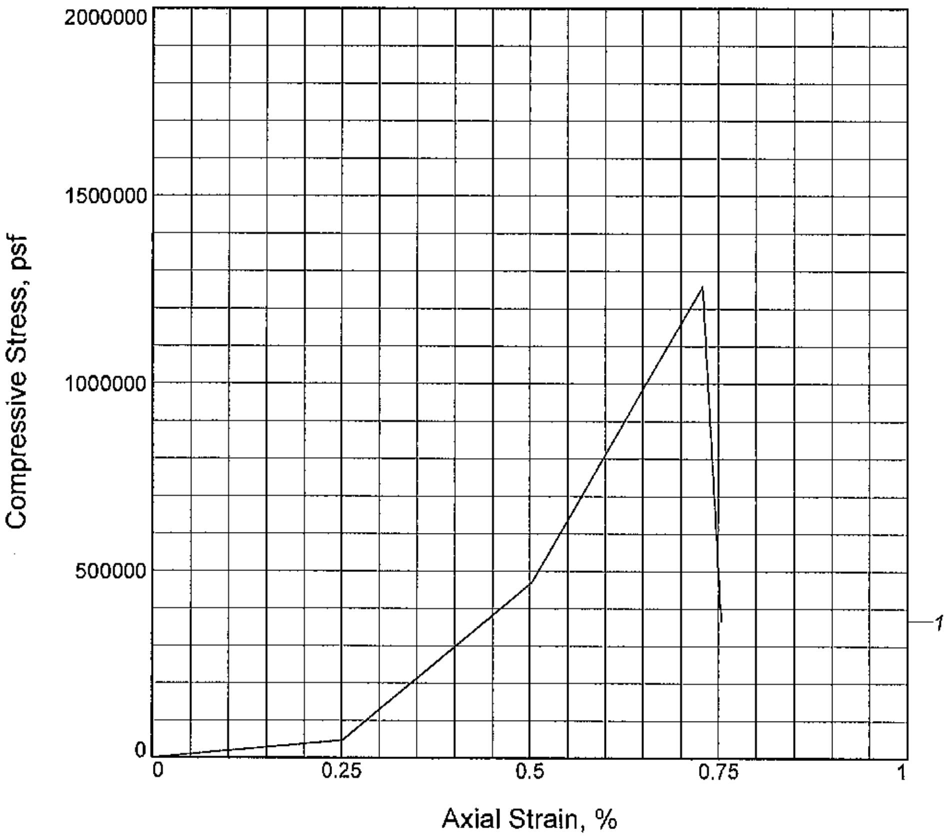
Project: BRENT SPENCE BRIDGE REPLACEMENT

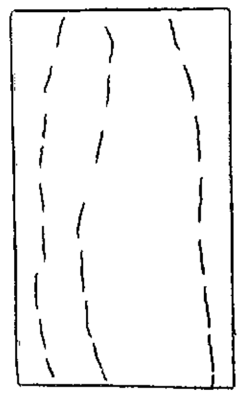
Source of Sample: R-6 **Depth:** 100.1-100.5'
Sample Number: 4/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1259226.7			
Undrained shear strength, psf	629613.3			
Failure strain, %	0.7			
Strain rate, in./min.	0.039			
Water content, %	0.5			
Wet density, pcf	168.0			
Dry density, pcf	167.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.980			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5906

Figure ________

Client: PARSONS BRINCKERHOFF

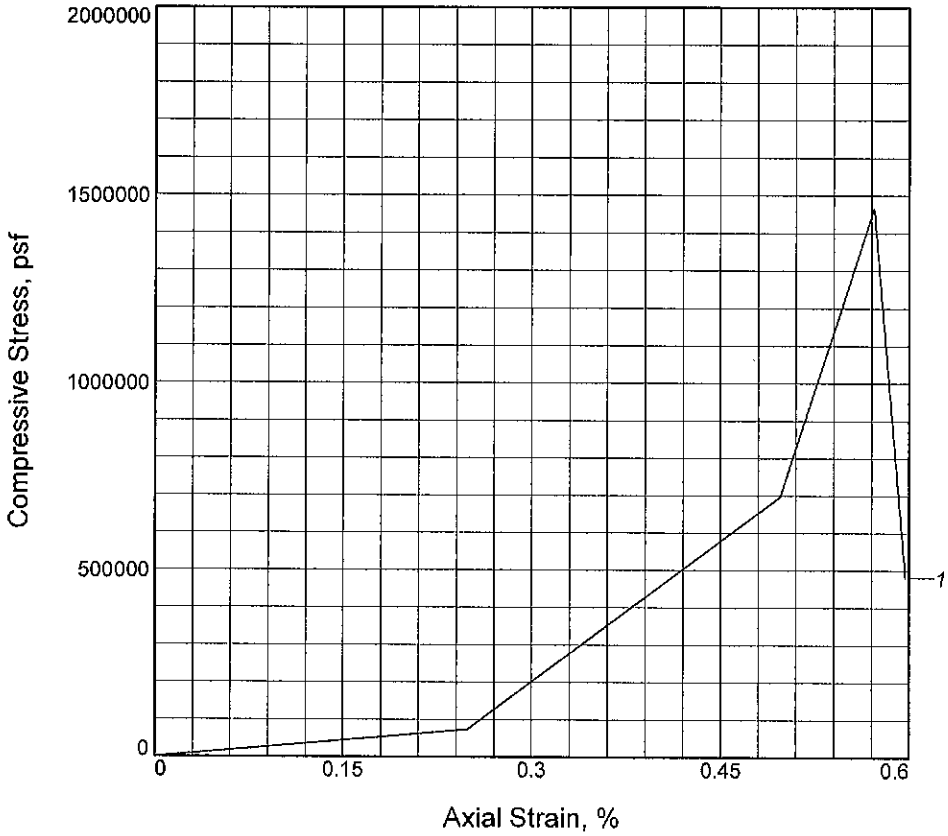
Project: BRENT SPENCE BRIDGE REPLACEMENT

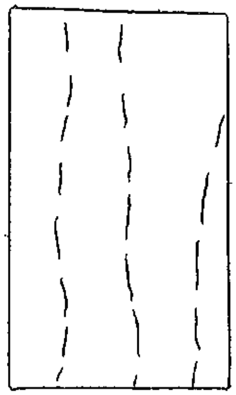
Source of Sample: R-6 **Depth:** 107.1-107.5'
Sample Number: 6/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1466508.1			
Undrained shear strength, psf	733254.1			
Failure strain, %	0.6			
Strain rate, in./min.	0.040			
Water content, %	0.2			
Wet density, pcf	169.2			
Dry density, pcf	168.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.020			
Height/diameter ratio	2.03			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-27-10

Remarks:
Lab No. 5907

Figure ________

Client: PARSONS BRINCKERHOFF

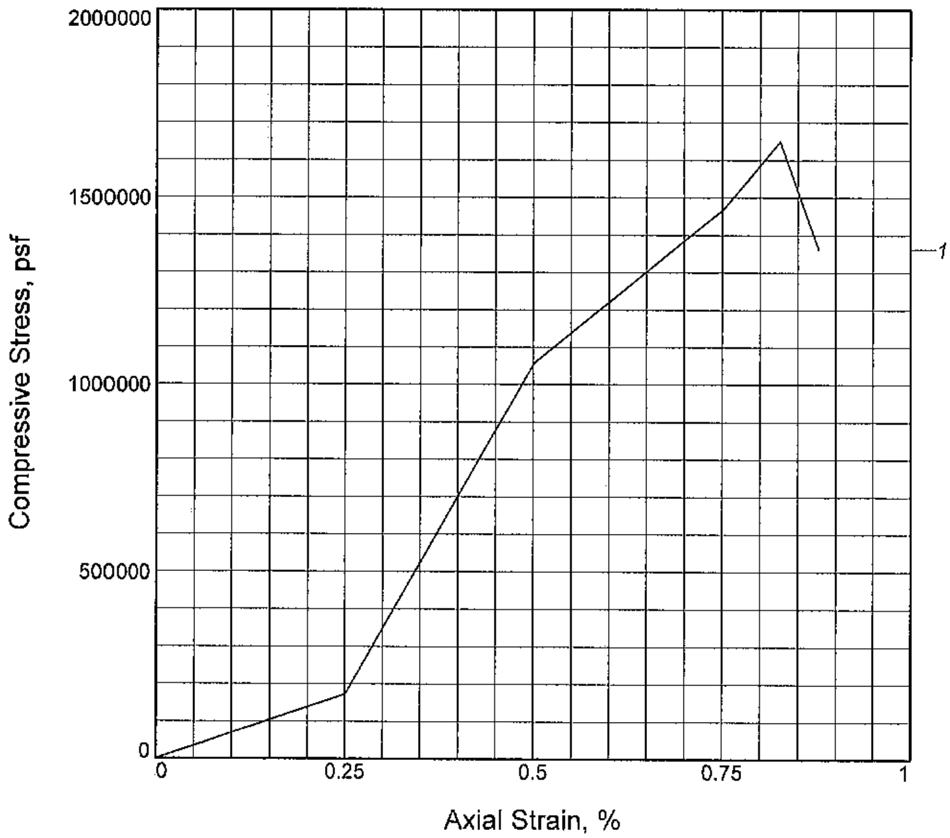
Project: BRENT SPENCE BRIDGE REPLACEMENT

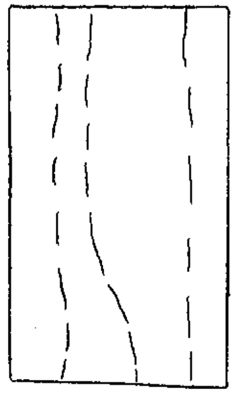
Source of Sample: R-6 **Depth:** 114.5-115'
Sample Number: 7/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1649615.1			
Undrained shear strength, psf	824807.5			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.3			
Wet density, pcf	167.3			
Dry density, pcf	166.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.990			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5909

Figure ______

Client: PARSONS BRINCKERHOFF

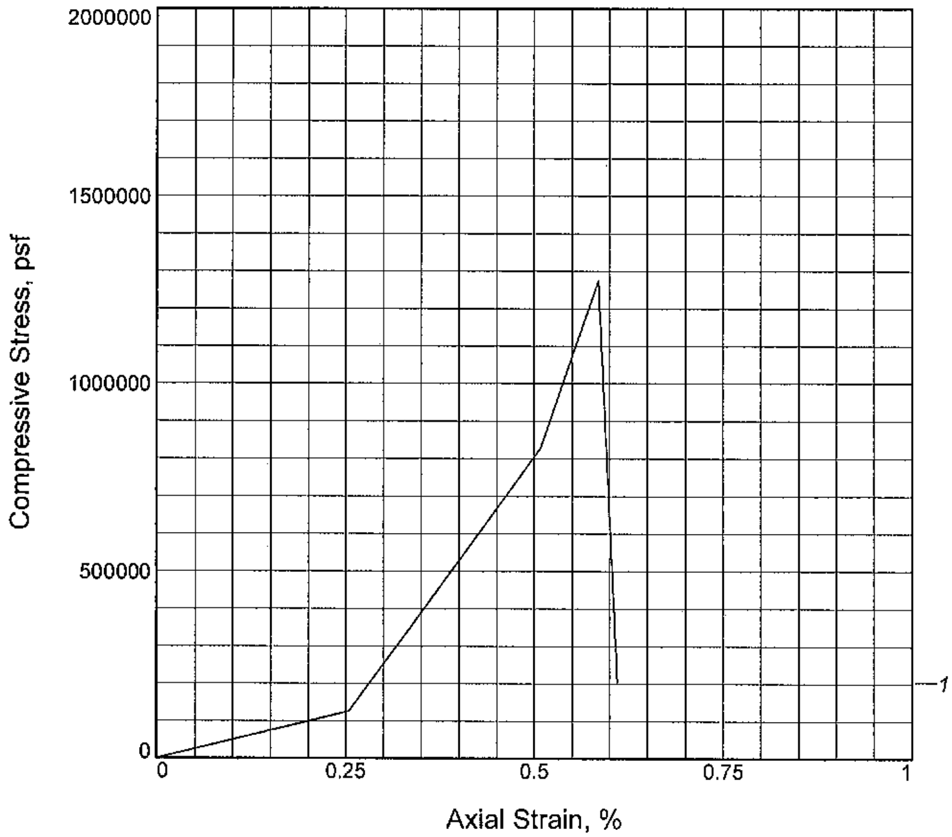
Project: BRENT SPENCE BRIDGE REPLACEMENT

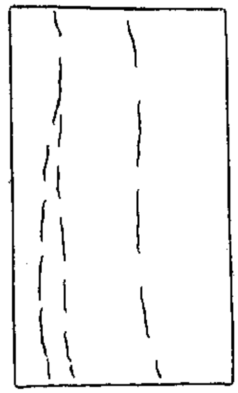
Source of Sample: R-6 **Depth:** 136.5-137.3'
Sample Number: 12/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1273413.0			
Undrained shear strength, psf	636706.5			
Failure strain, %	0.6			
Strain rate, in./min.	0.039			
Water content, %	0.8			
Wet density, pcf	167.0			
Dry density, pcf	165.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.930			
Height/diameter ratio	1.98			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-27-10
Remarks:
Lab No. 5912

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

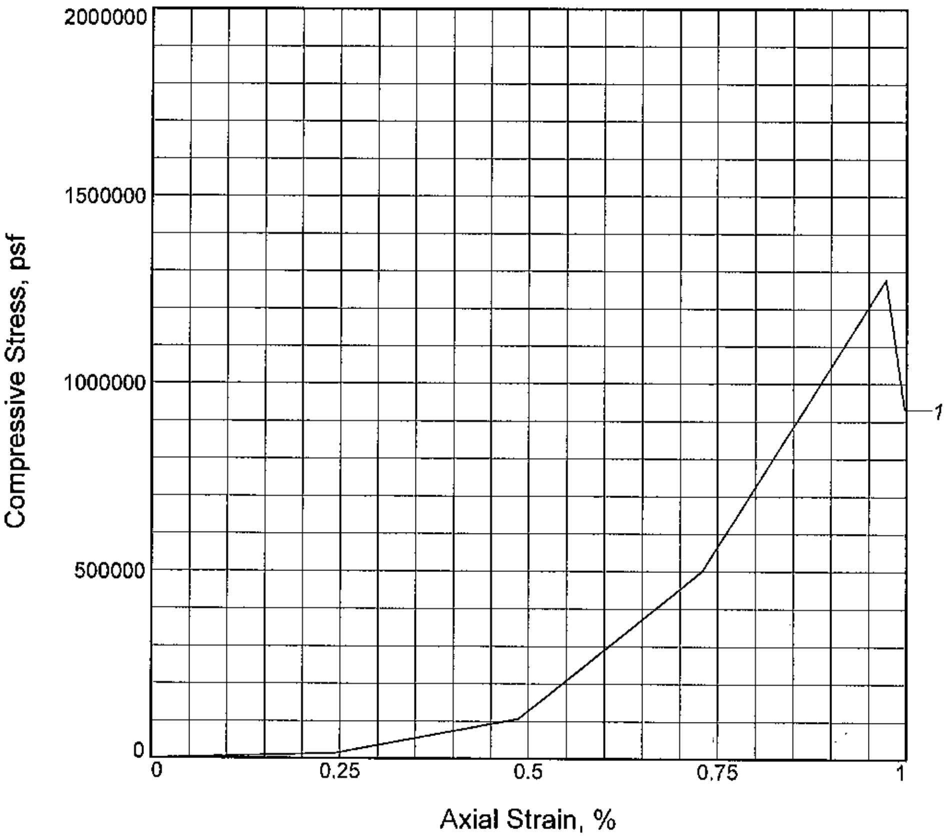
Source of Sample: R-6 **Depth:** 159.8-160.2'
Sample Number: 16/NQ

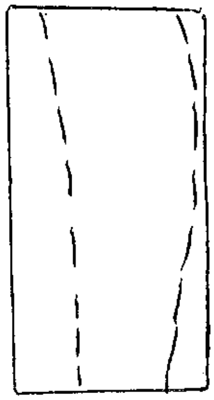
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1277541.7			
Undrained shear strength, psf	638770.9			
Failure strain, %	1.0			
Strain rate, in./min.	0.041			
Water content, %	0.3			
Wet density, pcf	168.1			
Dry density, pcf	167.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.972			
Specimen height, in.	4.110			
Height/diameter ratio	2.08			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10
Remarks:
Lab No. 5857

Figure ________

Client: PARSONS BRINCKERHOFF

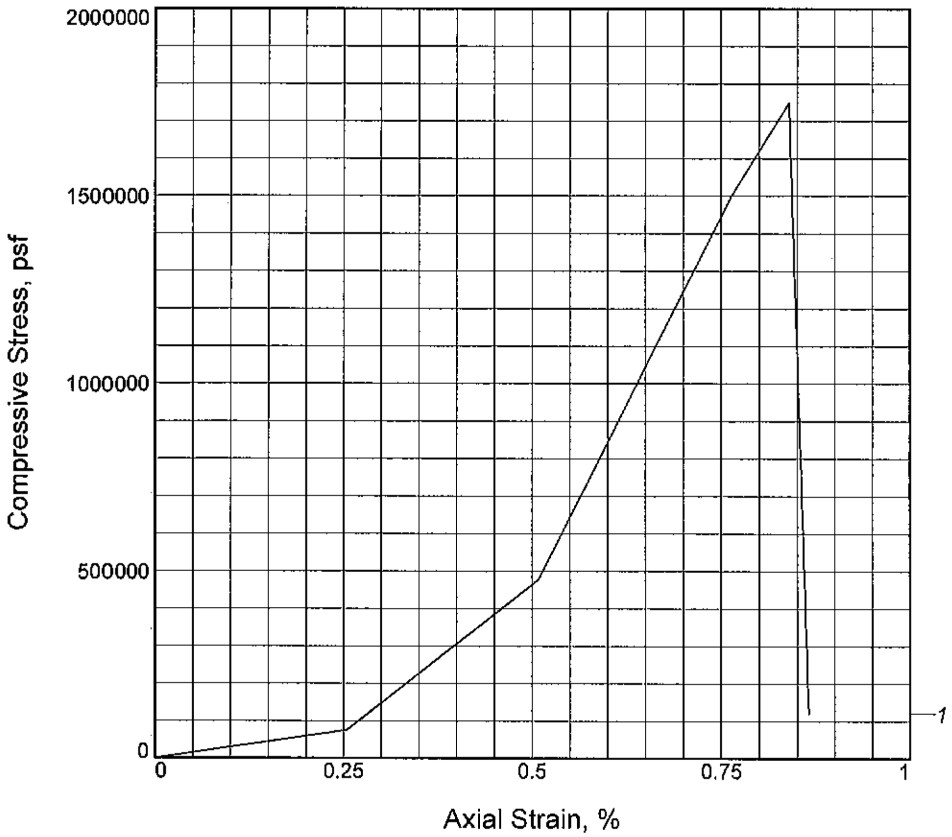
Project: BRENT SPENCE BRIDGE REPLACEMENT

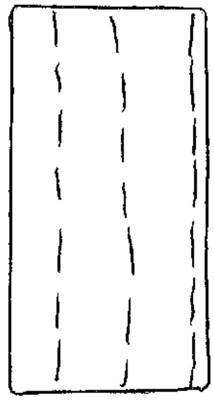
Source of Sample: R-7 **Depth:** 83.5-83.9'
Sample Number: 1/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1748783.3			
Undrained shear strength, psf	874391.7			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.3			
Wet density, pcf	168.9			
Dry density, pcf	168.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.930			
Height/diameter ratio	1.99			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10

Remarks:
Lab No. 5858

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

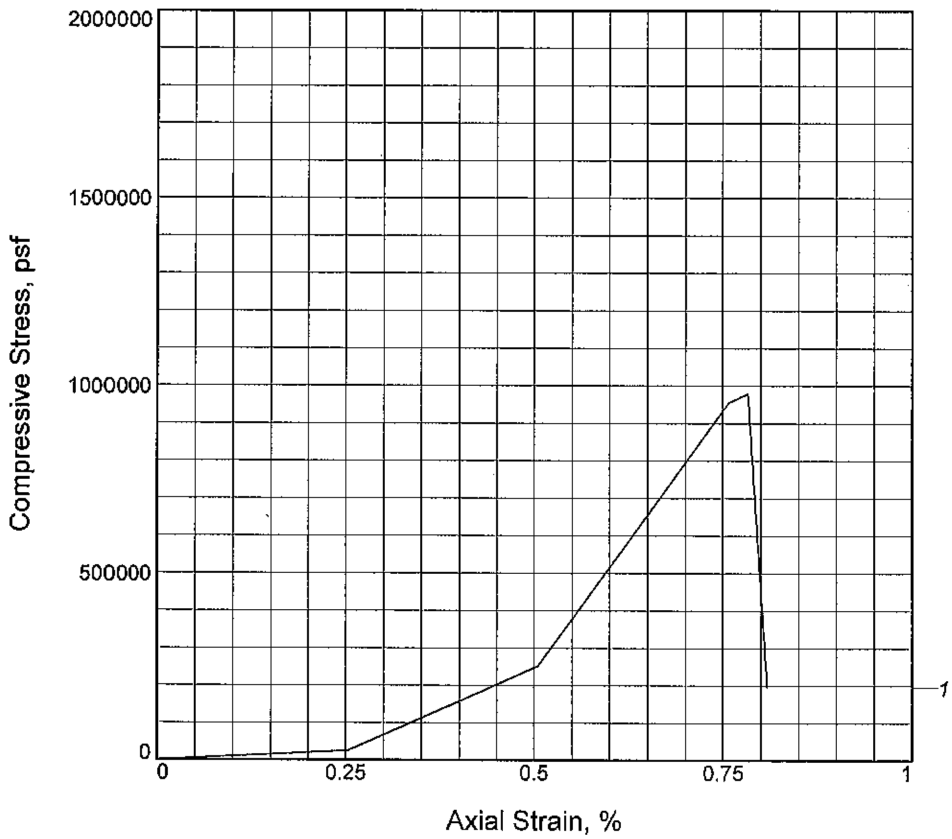
Source of Sample: R-7 **Depth:** 88.4-89'
Sample Number: 2/NQ

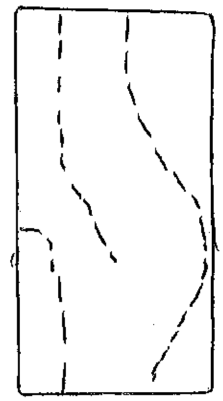
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	979514.7			
Undrained shear strength, psf	489757.3			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	1.0			
Wet density, pcf	170.6			
Dry density, pcf	168.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.960			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10

Remarks:
Lab No. 5862

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

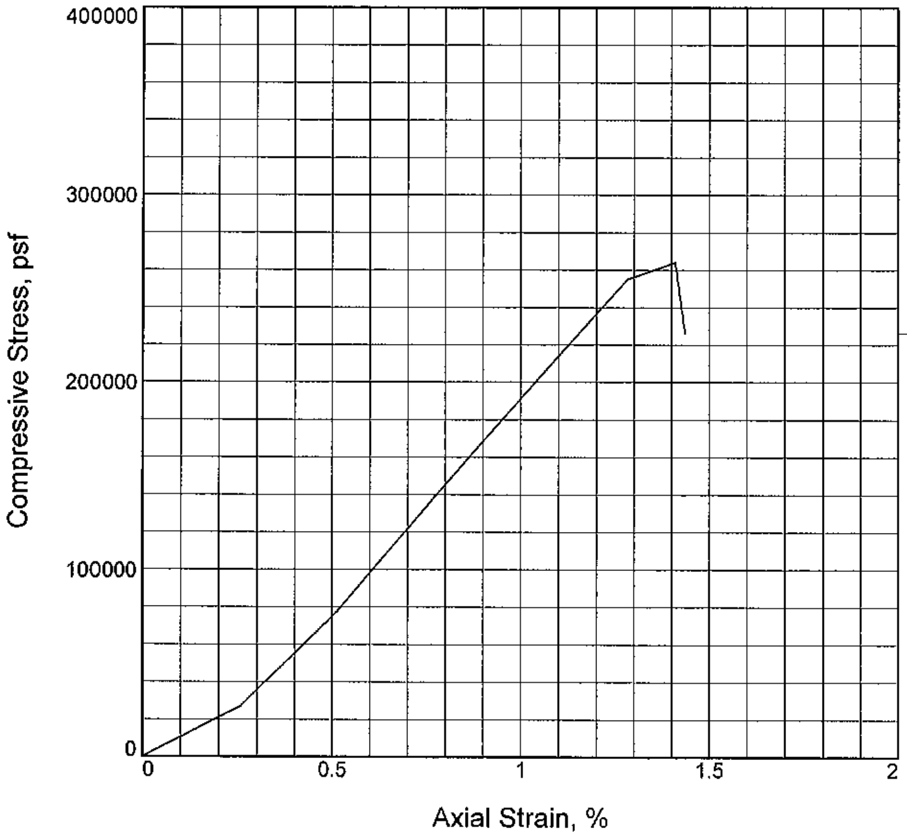
Source of Sample: R-7 **Depth:** 98-98.5'
Sample Number: 5/NQ

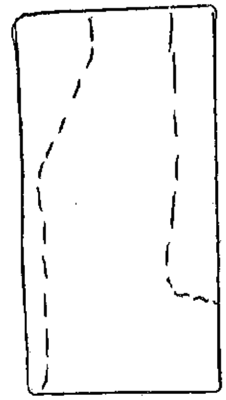
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	263952.1			
Undrained shear strength, psf	131976.0			
Failure strain, %	1.4			
Strain rate, in./min.	0.039			
Water content, %	3.5			
Wet density, pcf	161.0			
Dry density, pcf	155.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.900			
Height/diameter ratio	1.98			

Description: SHALE

LL =	PL =	PI =	Assumed GS=	Type: Shale

Project No.: N1105070
Date Sampled: 7-19-10
Remarks:
Lab No. 5866

Figure ________

Client: PARSONS BRINCKERHOFF

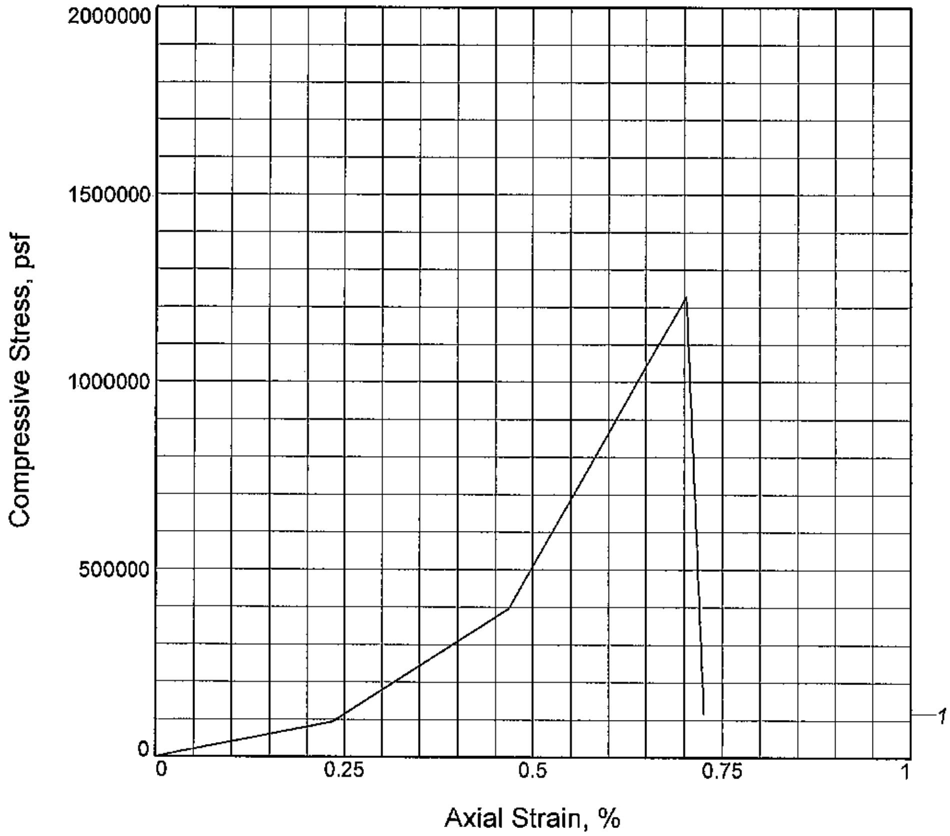
Project: BRENT SPENCE BRIDGE REPLACEMENT

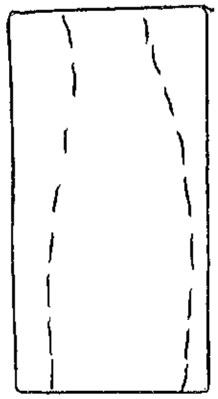
Source of Sample: R-7 **Depth:** 121.1-121.4'
Sample Number: 9/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1227670.6			
Undrained shear strength, psf	613835.3			
Failure strain, %	0.7			
Strain rate, in./min.	0.042			
Water content, %	0.5			
Wet density, pcf	165.0			
Dry density, pcf	164.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.270			
Height/diameter ratio	2.17			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10
Remarks:
Lab No. 5868

Figure ________

Client: PARSONS BRINCKERHOFF

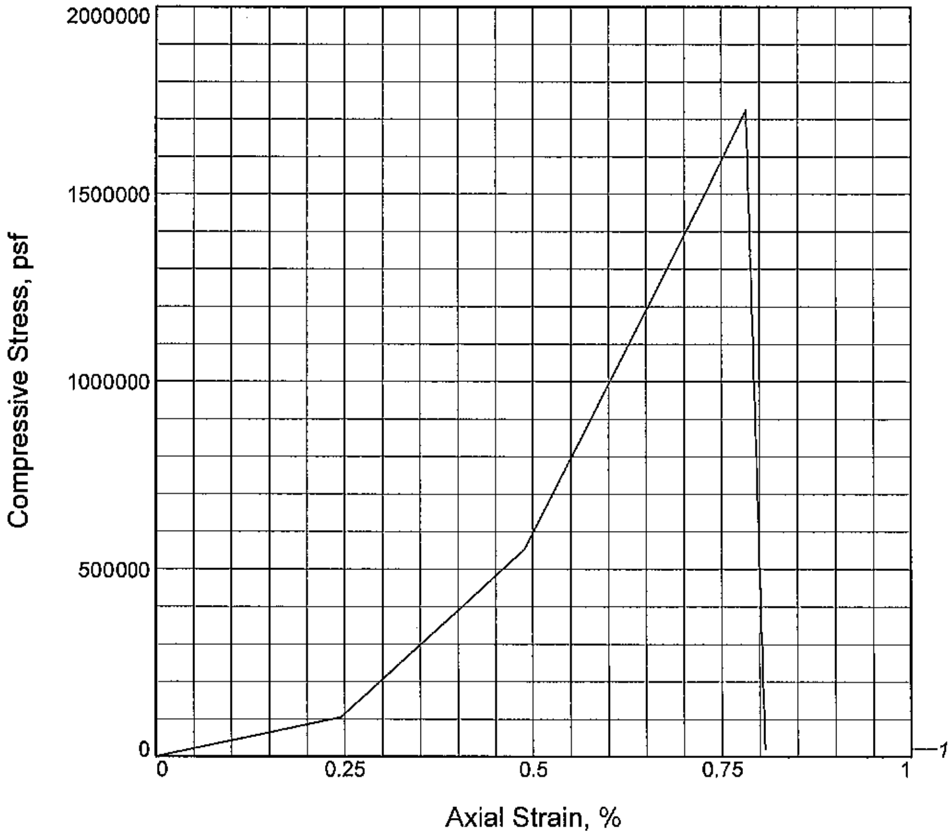
Project: BRENT SPENCE BRIDGE REPLACEMENT

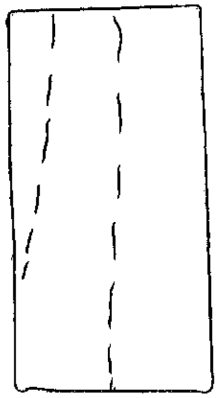
Source of Sample: R-7 **Depth:** 128.7-129.5'
Sample Number: 10/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1724247.7			
Undrained shear strength, psf	862123.8			
Failure strain, %	0.8			
Strain rate, in./min.	0.040			
Water content, %	0.4			
Wet density, pcf	168.4			
Dry density, pcf	167.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.090			
Height/diameter ratio	2.08			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10

Remarks:
Lab No. 5869

Figure ________

Client: PARSONS BRINCKERHOFF

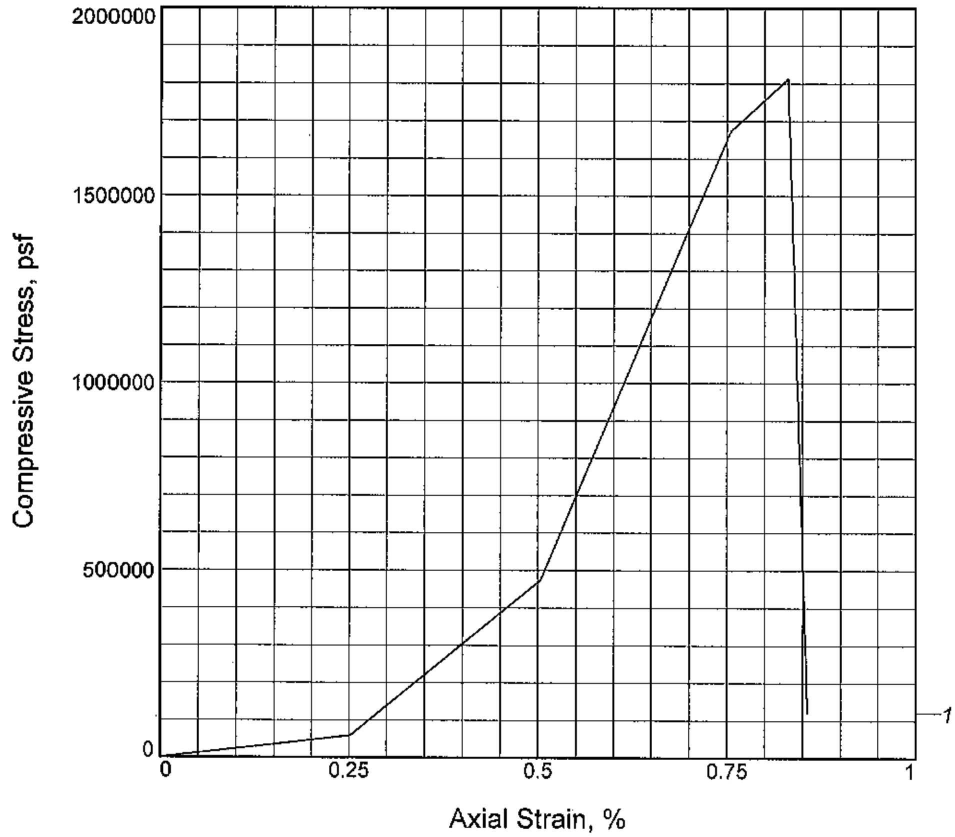
Project: BRENT SPENCE BRIDGE REPLACEMENT

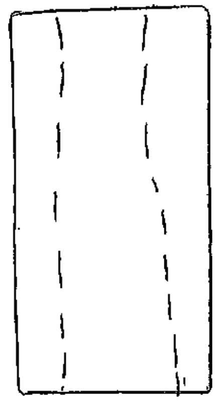
Source of Sample: R-7 **Depth:** 136.6-137.6'
Sample Number: 12/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1812415.1			
Undrained shear strength, psf	906207.5			
Failure strain, %	0.8			
Strain rate, in./min.	0.039			
Water content, %	0.5			
Wet density, pcf	165.8			
Dry density, pcf	165.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.970			
Height/diameter ratio	2.02			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 7-19-10
Remarks:
Lab No. 5871

Figure ________

Client: PARSONS BRINCKERHOFF

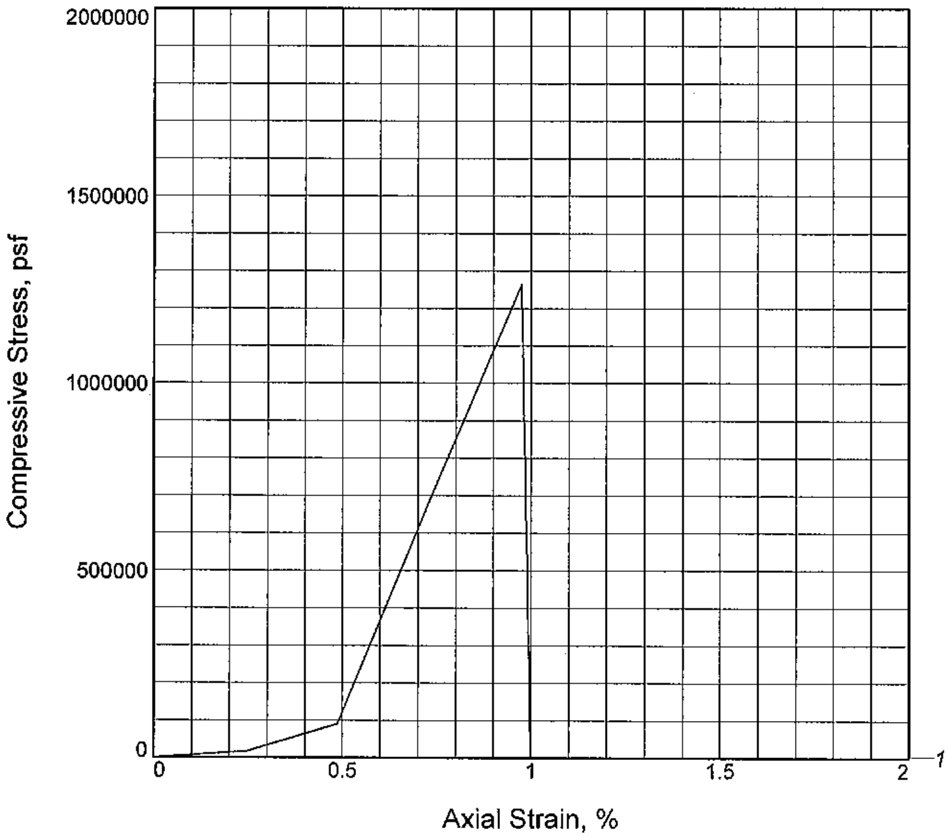
Project: BRENT SPENCE BRIDGE REPLACEMENT

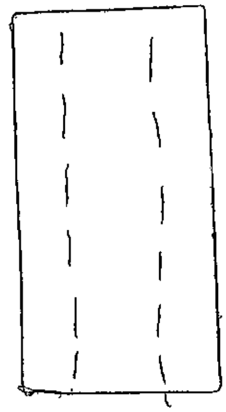
Source of Sample: R-7 **Depth:** 154.5-155.1
Sample Number: 16/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1263171.1			
Undrained shear strength, psf	631585.6			
Failure strain, %	1.0			
Strain rate, in./min.	0.041			
Water content, %	0.4			
Wet density, pcf	165.4			
Dry density, pcf	164.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	4.100			
Height/diameter ratio	2.08			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 7-19-10
Remarks:
Lab No. 5872

Figure _______

Client: PARSONS BRINCKERHOFF

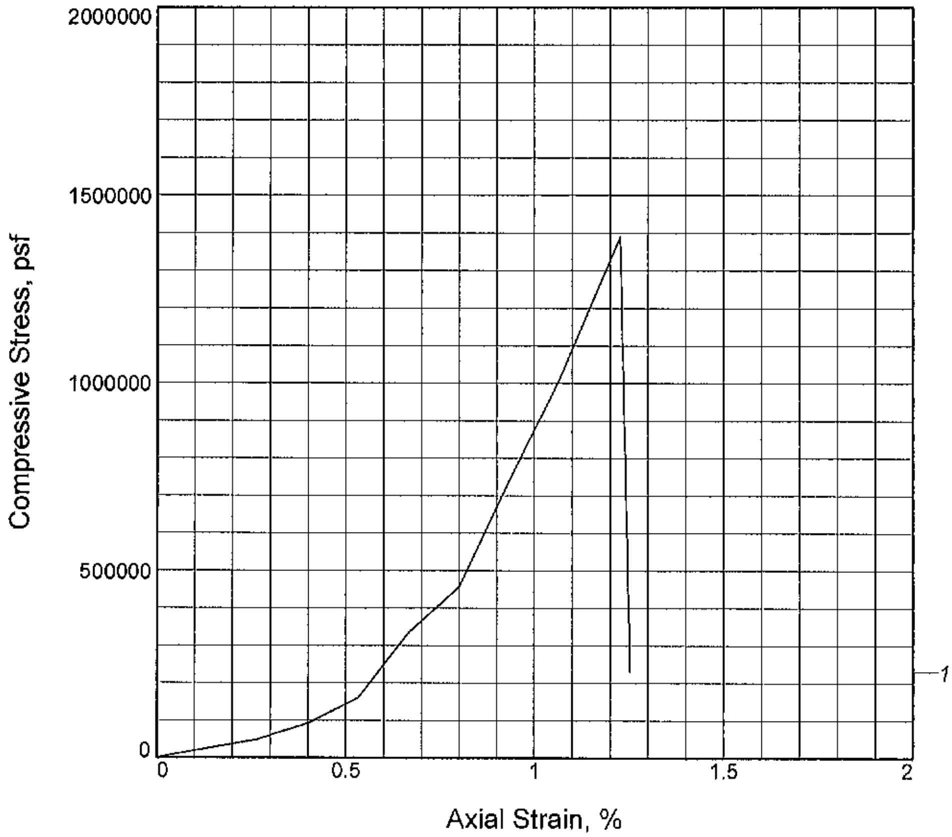
Project: BRENT SPENCE BRIDGE REPLACEMENT

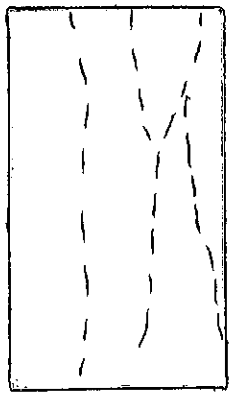
Source of Sample: R-7 **Depth:** 163.7-164.5'
Sample Number: 17/NQ

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: SV **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1388903.4			
Undrained shear strength, psf	694451.7			
Failure strain, %	1.2			
Strain rate, in./min.	0.037			
Water content, %	0.1			
Wet density, pcf	166.9			
Dry density, pcf	166.8			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.750			
Height/diameter ratio	1.89			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9682

Figure ________

Client: PARSONS BRINCKERHOFF

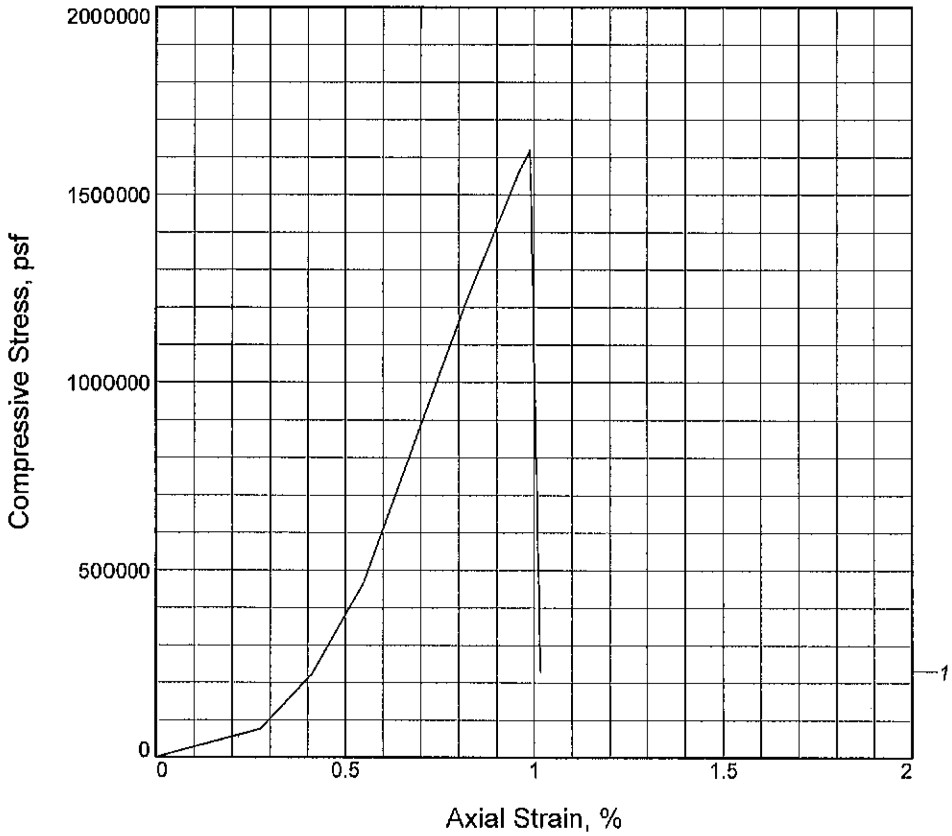
Project: BRENT SPENCE BRIDGE REPLACEMENT

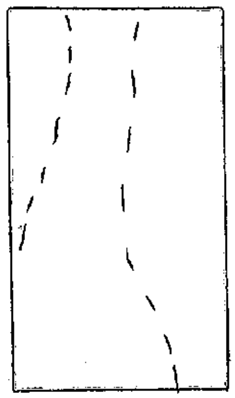
Source of Sample: R-8 **Depth:** 87.8-88.2'
Sample Number: 1

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1618490.2			
Undrained shear strength, psf	809245.1			
Failure strain, %	1.0			
Strain rate, in./min.	0.036			
Water content, %	0.6			
Wet density, pcf	167.4			
Dry density, pcf	166.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.640			
Height/diameter ratio	1.84			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9685

Figure ________

Client: PARSONS BRINCKERHOFF

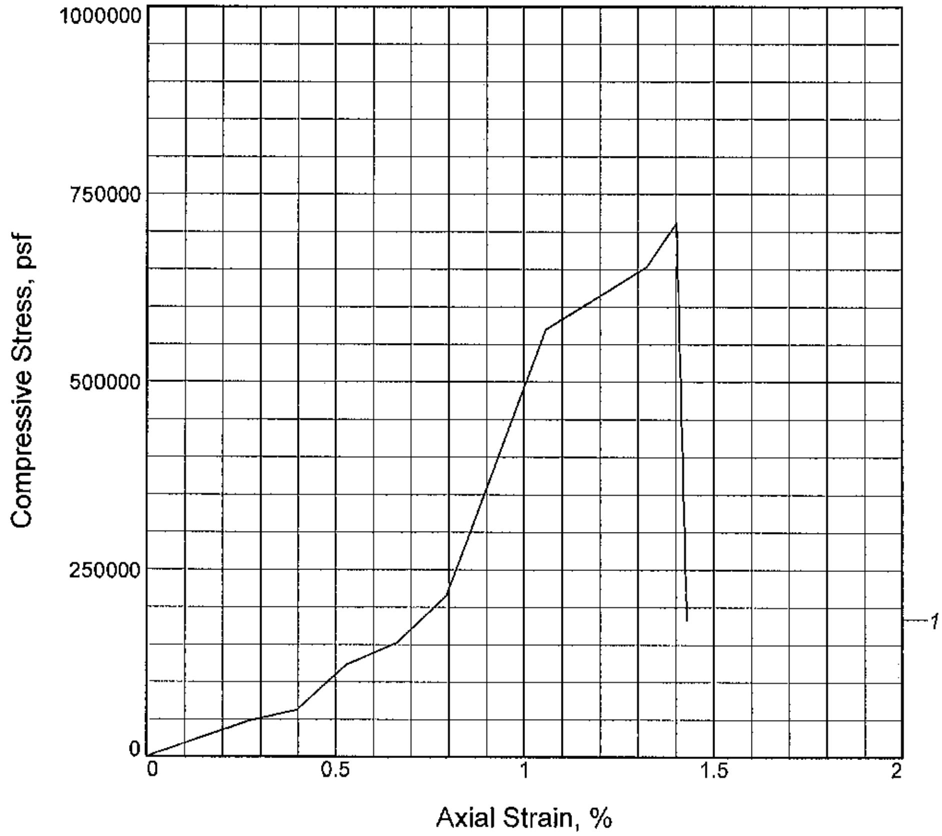
Project: BRENT SPENCE BRIDGE REPLACEMENT

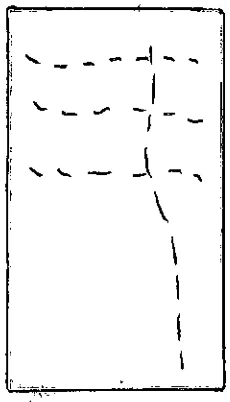
Source of Sample: R-8 **Depth:** 100.5-101'
Sample Number: 4

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	711870.3			
Undrained shear strength, psf	355935.1			
Failure strain, %	1.4			
Strain rate, in./min.	0.037			
Water content, %	1.0			
Wet density, pcf	164.3			
Dry density, pcf	162.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.780			
Height/diameter ratio	1.91			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9687

Figure ________

Client: PARSONS BRINCKERHOFF

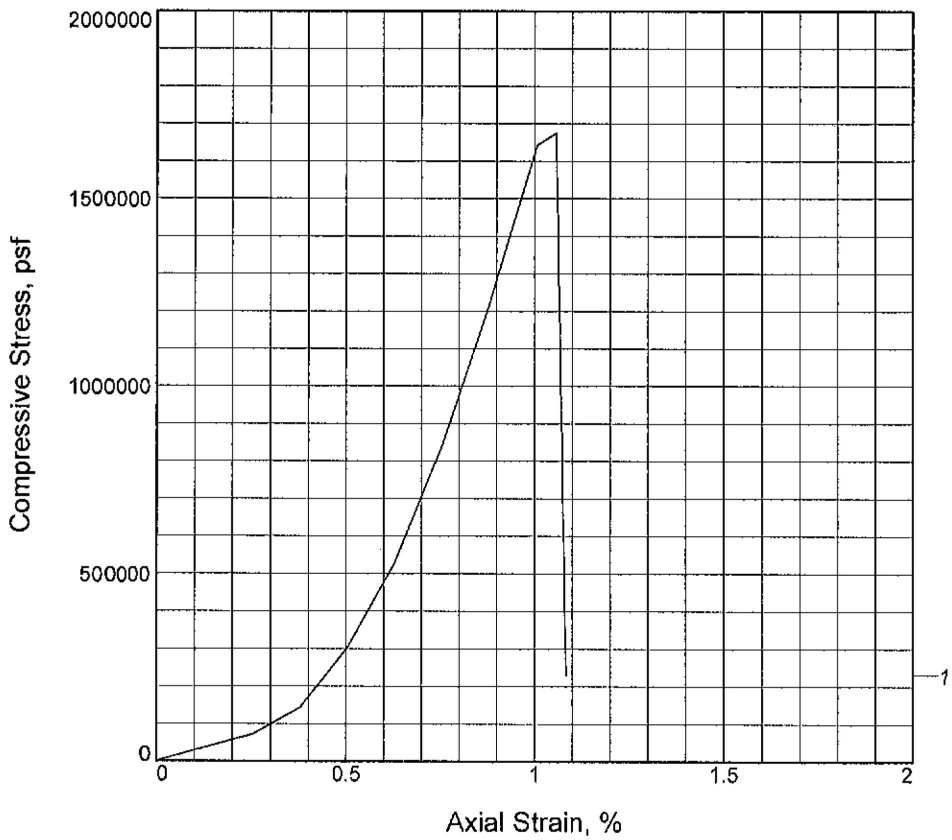
Project: BRENT SPENCE BRIDGE REPLACEMENT

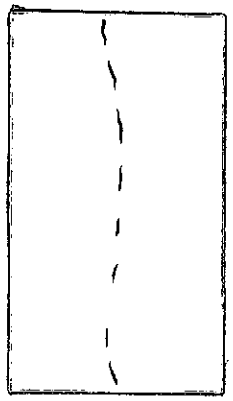
Source of Sample: R-8 **Depth:** 101.8-102.3'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1674834.1			
Undrained shear strength, psf	837417.0			
Failure strain, %	1.1			
Strain rate, in./min.	0.039			
Water content, %	0.7			
Wet density, pcf	166.4			
Dry density, pcf	165.3			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.970			
Height/diameter ratio	2.01			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9690

Figure _______

Client: PARSONS BRINCKERHOFF

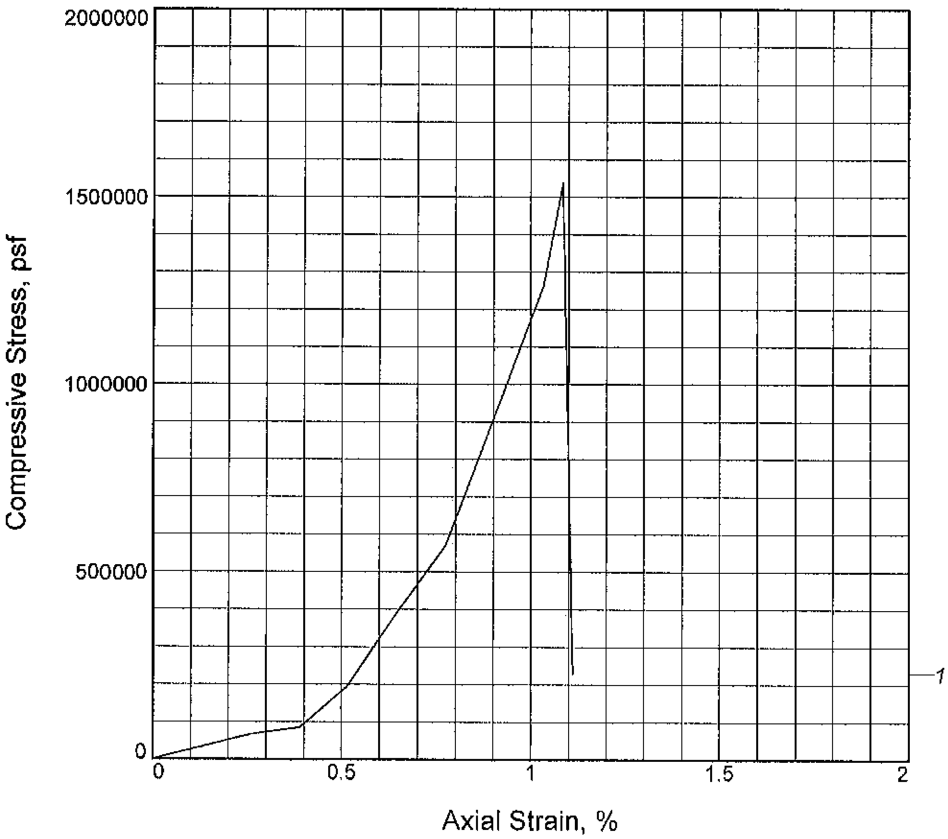
Project: BRENT SPENCE BRIDGE REPLACEMENT

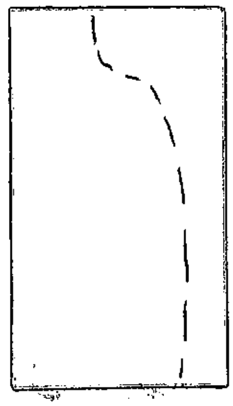
Source of Sample: R-8 **Depth:** 126.3-126.7'
Sample Number: 9

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1537026.1			
Undrained shear strength, psf	768513.1			
Failure strain, %	1.1			
Strain rate, in./min.	0.038			
Water content, %	0.3			
Wet density, pcf	166.6			
Dry density, pcf	166.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.870			
Height/diameter ratio	1.95			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9691

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

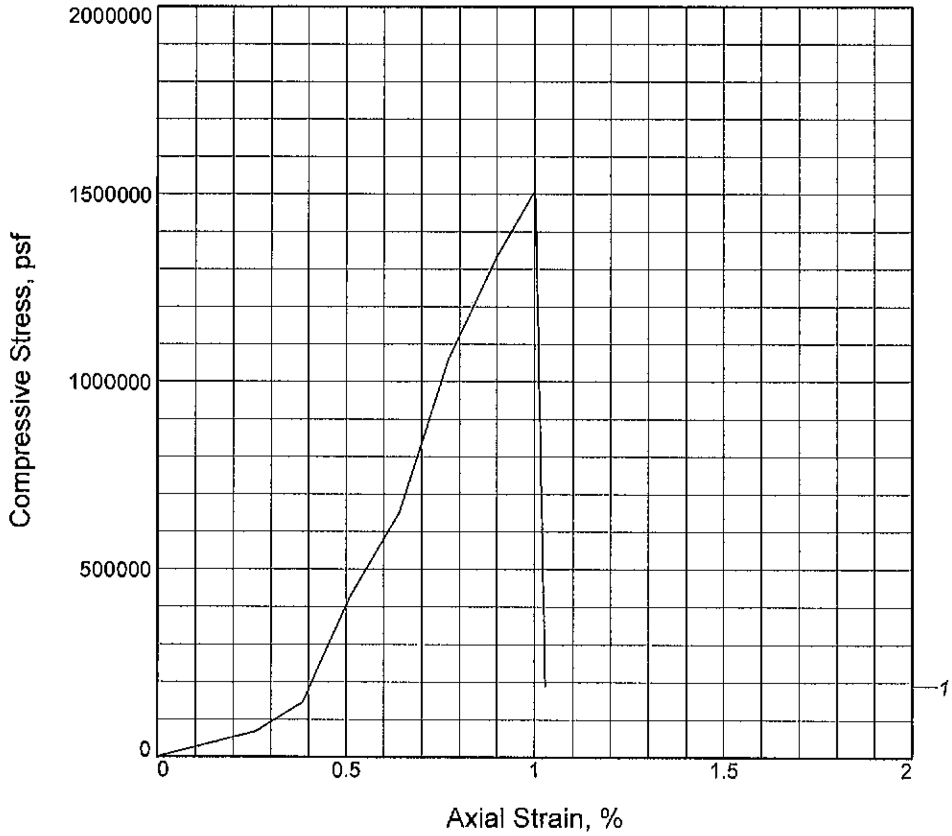
Source of Sample: R-8 **Depth:** 127.8-128.3'
Sample Number: 10

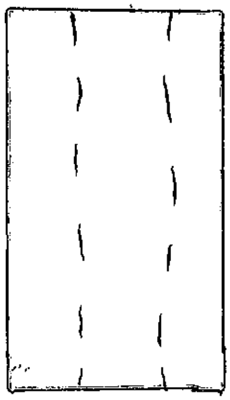
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1511267.5			
Undrained shear strength, psf	755633.8			
Failure strain, %	1.0			
Strain rate, in./min.	0.038			
Water content, %	0.5			
Wet density, pcf	162.5			
Dry density, pcf	161.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.970			
Specimen height, in.	3.890			
Height/diameter ratio	1.97			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9692

Figure ________

Client: PARSONS BRINCKERHOFF

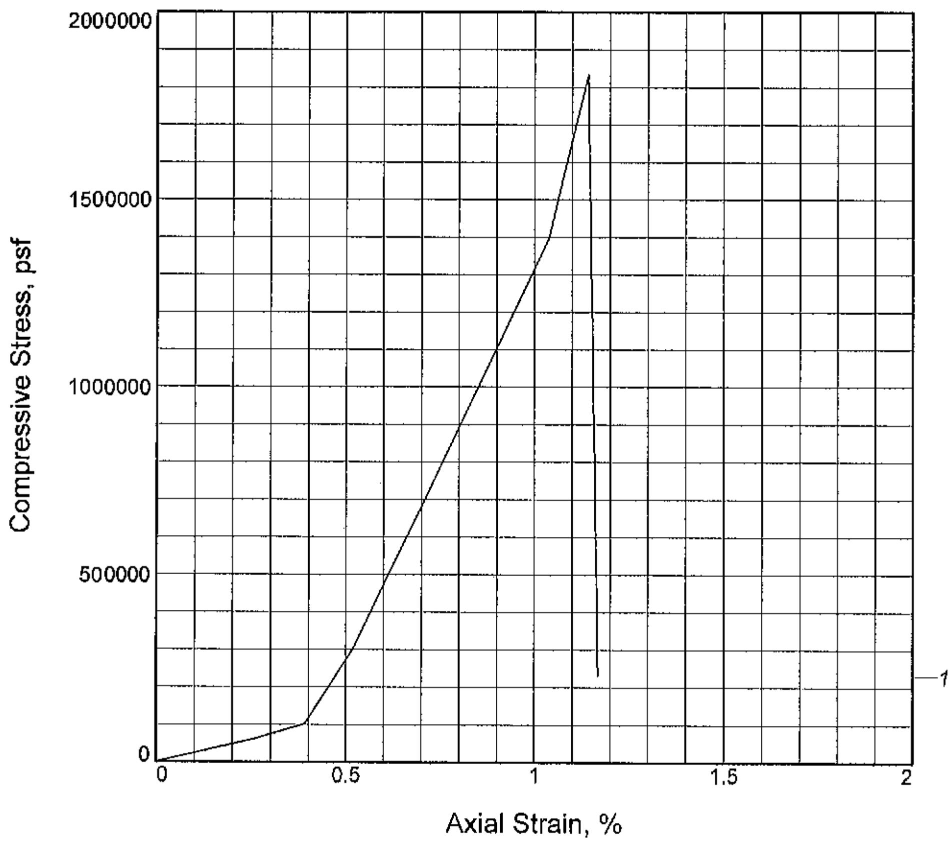
Project: BRENT SPENCE BRIDGE REPLACEMENT

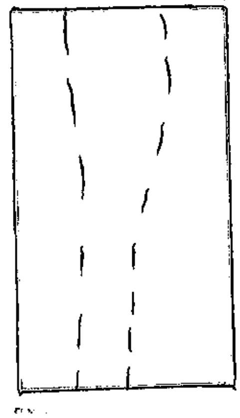
Source of Sample: R-8 **Depth:** 135.5-136'
Sample Number: 11

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1831836.4			
Undrained shear strength, psf	915918.2			
Failure strain, %	1.1			
Strain rate, in./min.	0.038			
Water content, %	0.3			
Wet density, pcf	162.5			
Dry density, pcf	162.0			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.850			
Height/diameter ratio	1.94			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10

Remarks:
Lab No. 9693

Figure ________

Client: PARSONS BRINCKERHOFF

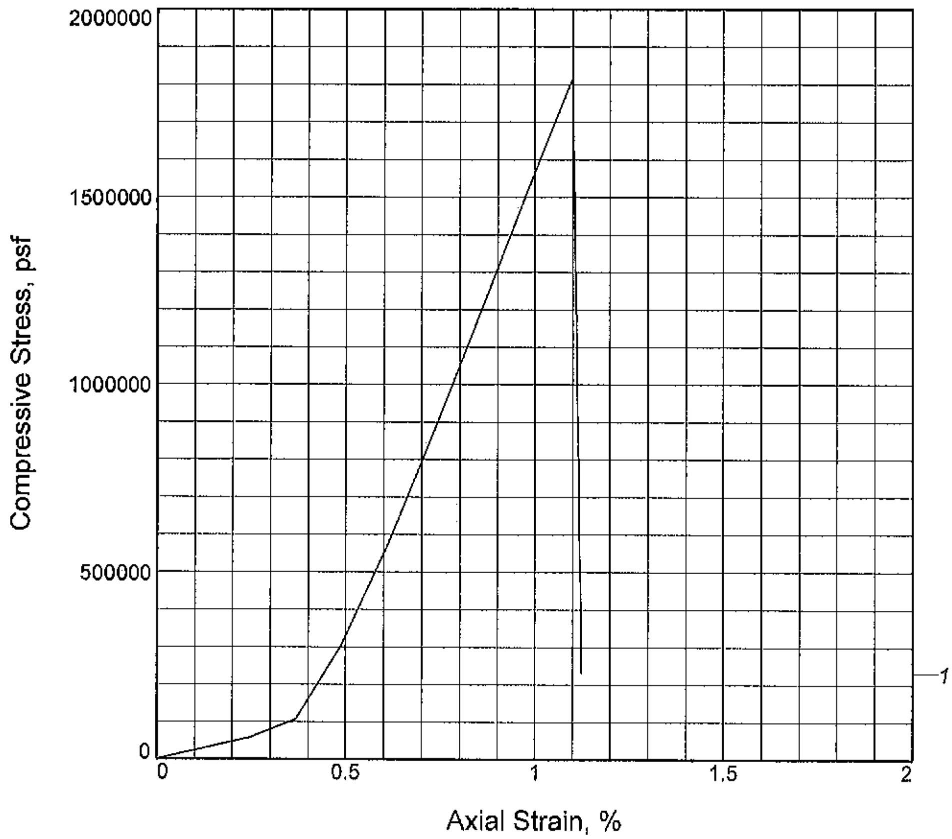
Project: BRENT SPENCE BRIDGE REPLACEMENT

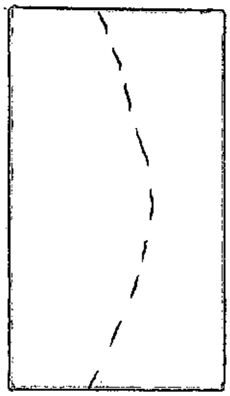
Source of Sample: R-8 **Depth:** 141-141.5'
Sample Number: 12

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1817085.1			
Undrained shear strength, psf	908542.5			
Failure strain, %	1.1			
Strain rate, in./min.	0.040			
Water content, %	0.3			
Wet density, pcf	160.0			
Dry density, pcf	159.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	4.090			
Height/diameter ratio	2.07			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9694

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

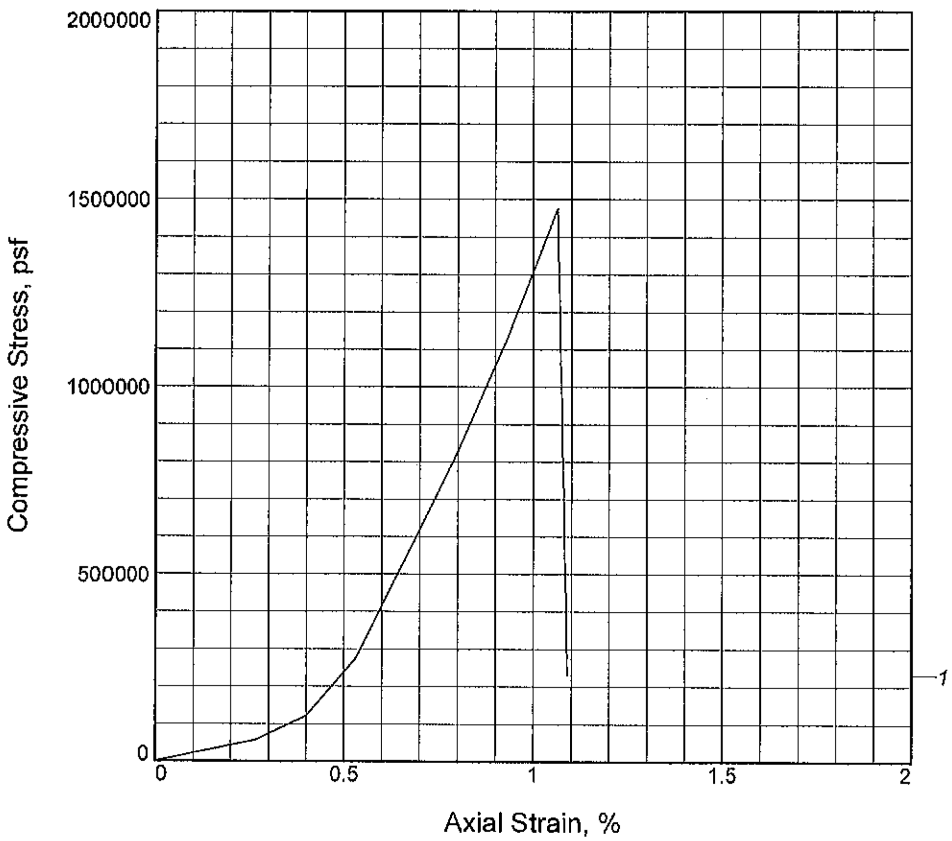
Source of Sample: R-8 **Depth:** 149-149.5'
Sample Number: 13

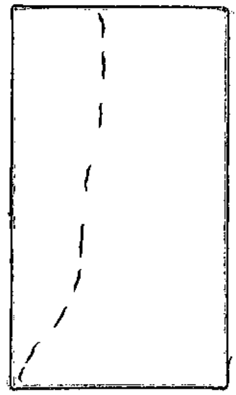
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1475126.5			
Undrained shear strength, psf	737563.2			
Failure strain, %	1.1			
Strain rate, in./min.	0.037			
Water content, %	0.2			
Wet density, pcf	164.4			
Dry density, pcf	164.2			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.760			
Height/diameter ratio	1.90			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9695

Figure ________

Client: PARSONS BRINCKERHOFF

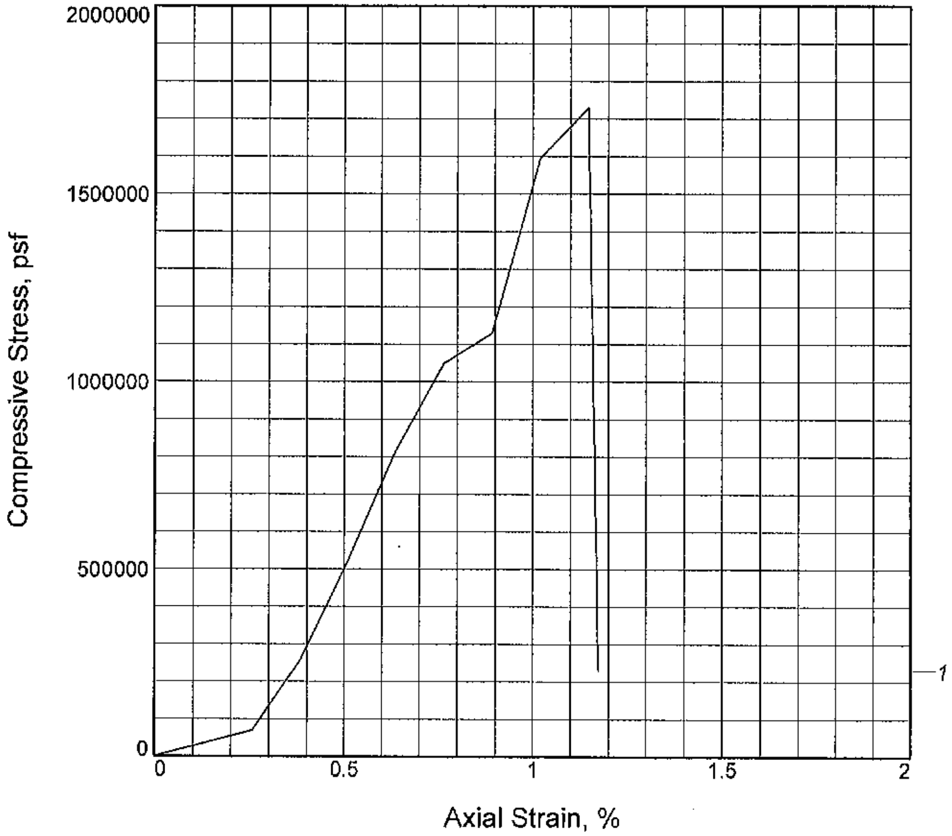
Project: BRENT SPENCE BRIDGE REPLACEMENT

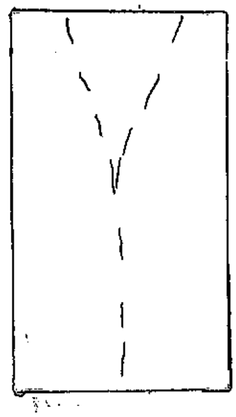
Source of Sample: R-8 **Depth:** 151.8-152.1'
Sample Number: 14

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1729572.6			
Undrained shear strength, psf	864786.3			
Failure strain, %	1.1			
Strain rate, in./min.	0.039			
Water content, %	0.2			
Wet density, pcf	164.9			
Dry density, pcf	164.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	1.980			
Specimen height, in.	3.920			
Height/diameter ratio	1.98			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-5-10
Remarks:
Lab No. 9696

Figure _______

Client: PARSONS BRINCKERHOFF

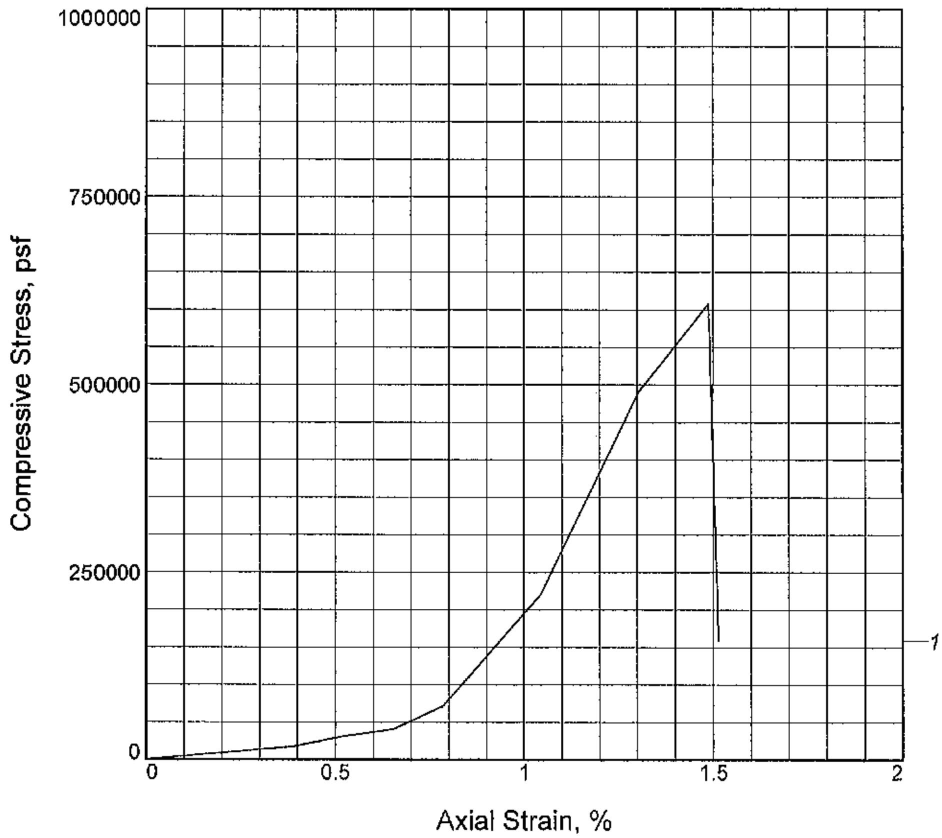
Project: BRENT SPENCE BRIDGE REPLACEMENT

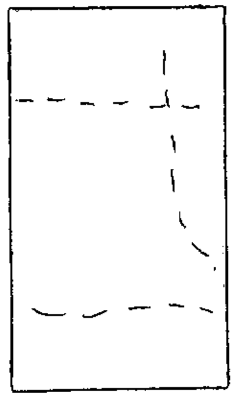
Source of Sample: R-8 **Depth:** 158.7-159.2'
Sample Number: 15

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	607979.7			
Undrained shear strength, psf	303989.9			
Failure strain, %	1.5			
Strain rate, in./min.	0.038			
Water content, %	1.7			
Wet density, pcf	162.3			
Dry density, pcf	159.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	3.830			
Height/diameter ratio	1.61			

Description: LIMESTONE W/SHALE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone w/Shale

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9702

Figure ________

Client: PARSONS BRINCKERHOFF

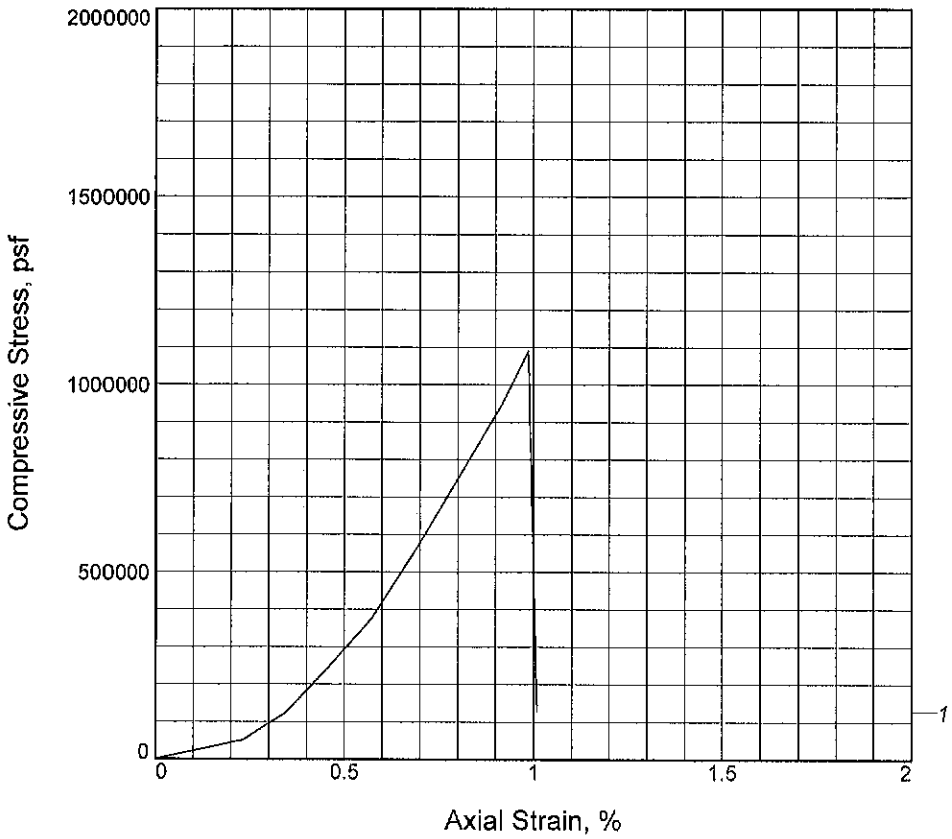
Project: BRENT SPENCE BRIDGE REPLACEMENT

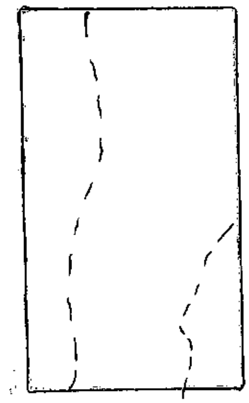
Source of Sample: R-2A **Depth:** 120.51-121'
Sample Number: 6

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1089495.1			
Undrained shear strength, psf	544747.5			
Failure strain, %	1.0			
Strain rate, in./min.	0.043			
Water content, %	1.2			
Wet density, pcf	162.6			
Dry density, pcf	160.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.375			
Specimen height, in.	4.360			
Height/diameter ratio	1.84			

Description: LIMESTONE

LL =	**PL =**	**PI =**	**Assumed GS=**	**Type:** Limestone

Project No.: N1105070
Date Sampled: 10-7-10

Remarks:
Lab No. 9705

Figure ________

Client: PARSONS BRINCKERHOFF

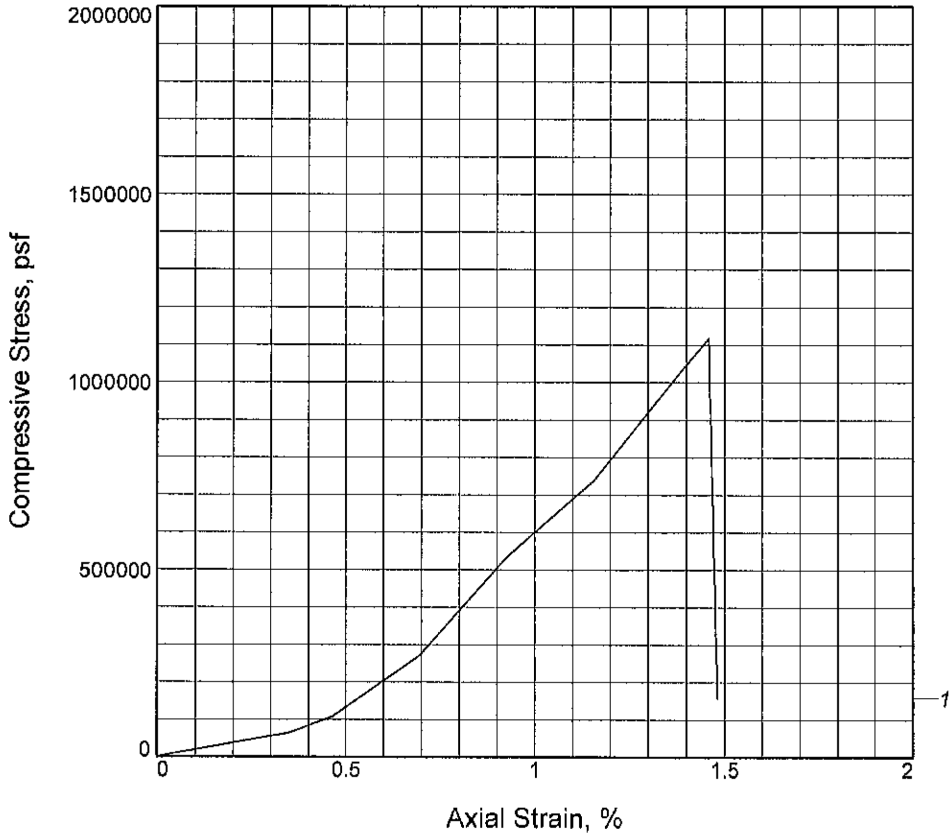
Project: BRENT SPENCE BRIDGE REPLACEMENT

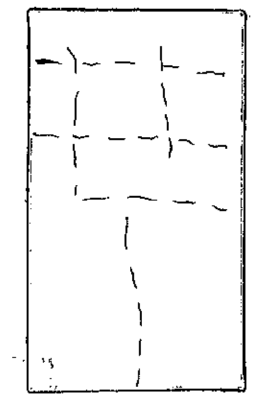
Source of Sample: R-2A **Depth:** 134.4-134.9'
Sample Number: 9

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1117004.0			
Undrained shear strength, psf	558502.0			
Failure strain, %	1.5			
Strain rate, in./min.	0.043			
Water content, %	0.7			
Wet density, pcf	162.0			
Dry density, pcf	160.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	4.320			
Height/diameter ratio	1.82			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9706

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

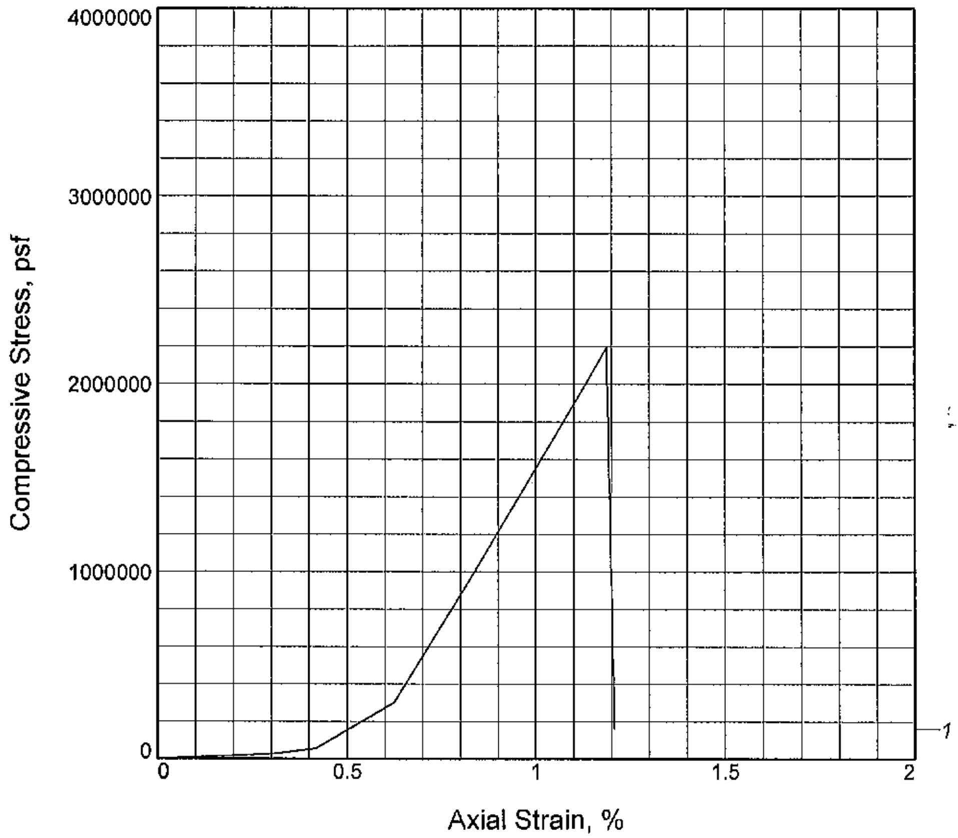
Source of Sample: R-2A **Depth:** 140-140.5'
Sample Number: 10

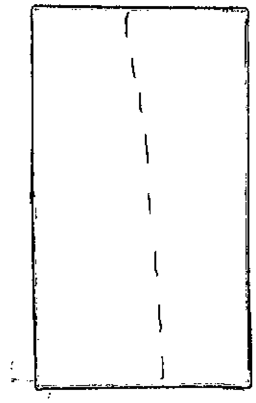
UNCONFINED COMPRESSION TEST

H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	2192551.5			
Undrained shear strength, psf	1096275.7			
Failure strain, %	1.2			
Strain rate, in./min.	0.048			
Water content, %	0.2			
Wet density, pcf	168.3			
Dry density, pcf	167.9			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.370			
Specimen height, in.	4.800			
Height/diameter ratio	2.03			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9708

Figure ________

Client: PARSONS BRINCKERHOFF

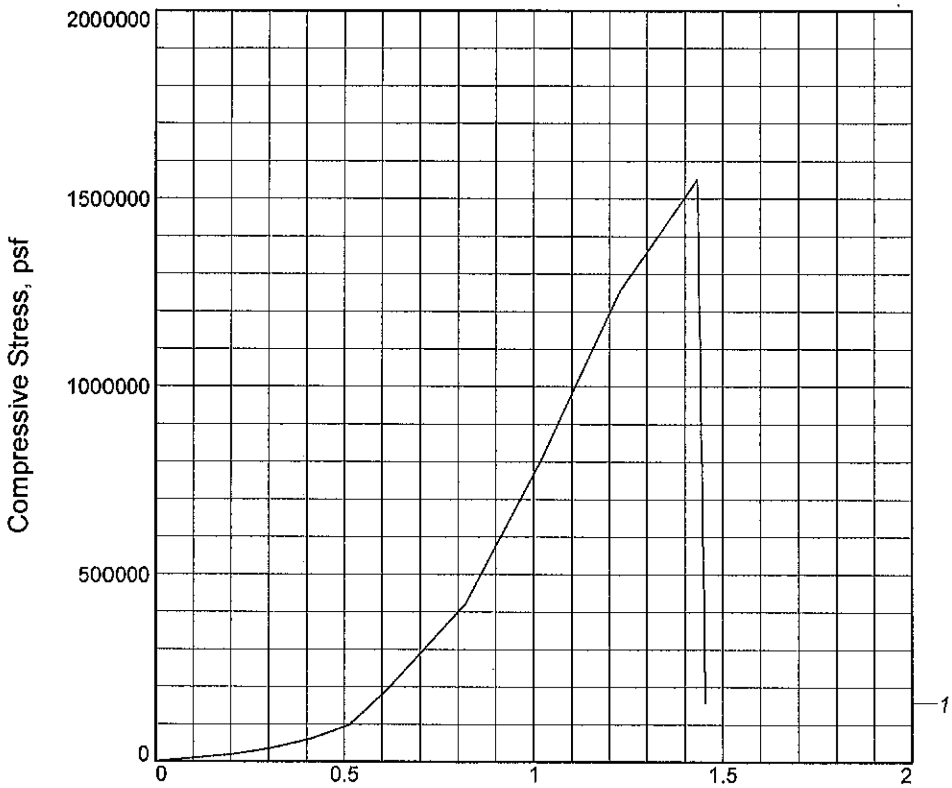
Project: BRENT SPENCE BRIDGE REPLACEMENT

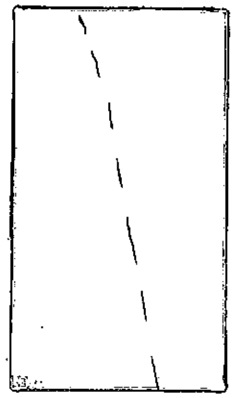
Source of Sample: R-2A **Depth:** 148-148.5'
Sample Number: 12

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1550817.3			
Undrained shear strength, psf	775408.7			
Failure strain, %	1.4			
Strain rate, in./min.	0.048			
Water content, %	0.6			
Wet density, pcf	162.0			
Dry density, pcf	161.1			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	4.880			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9709

Figure ________

Client: PARSONS BRINCKERHOFF

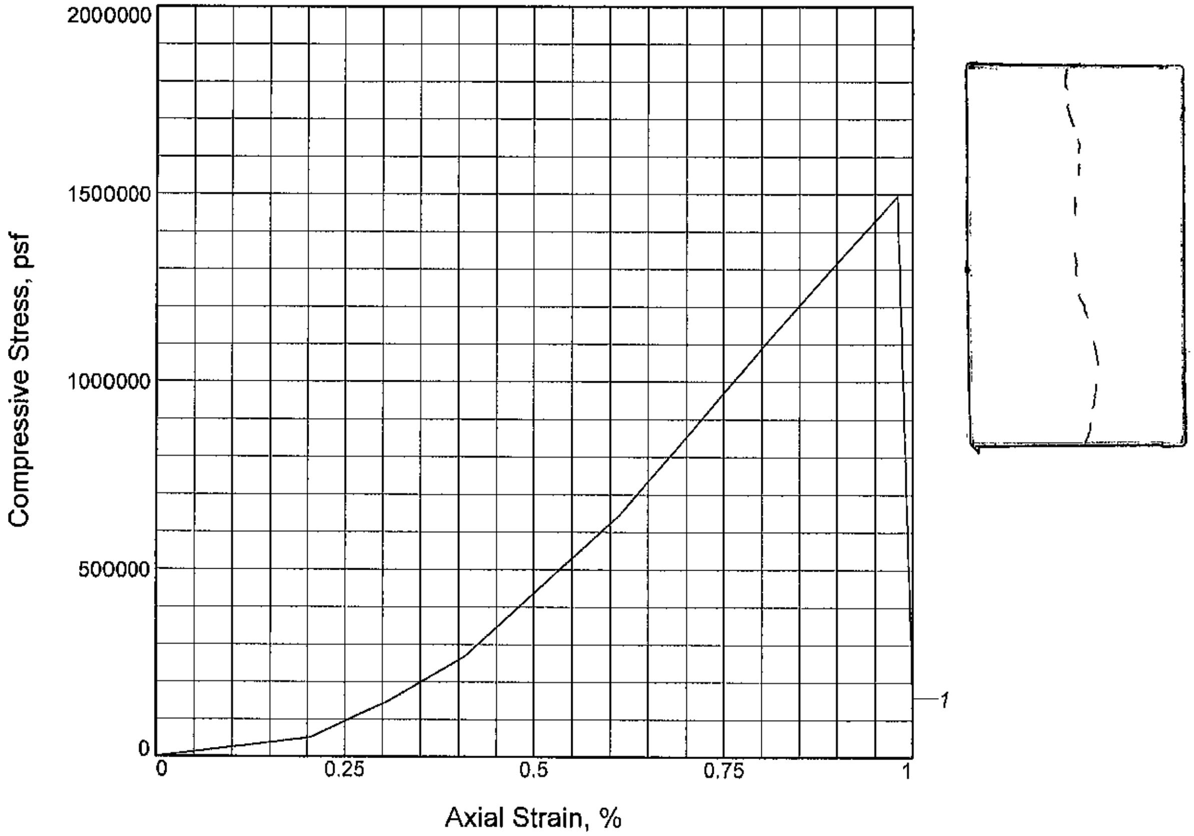
Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-2A **Depth:** 160-160.5'
Sample Number: 13

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, psf	1495031.6			
Undrained shear strength, psf	747515.8			
Failure strain, %	1.0			
Strain rate, in./min.	0.049			
Water content, %	0.8			
Wet density, pcf	162.8			
Dry density, pcf	161.6			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.390			
Specimen height, in.	4.900			
Height/diameter ratio	2.05			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9711

Figure ________

Client: PARSONS BRINCKERHOFF

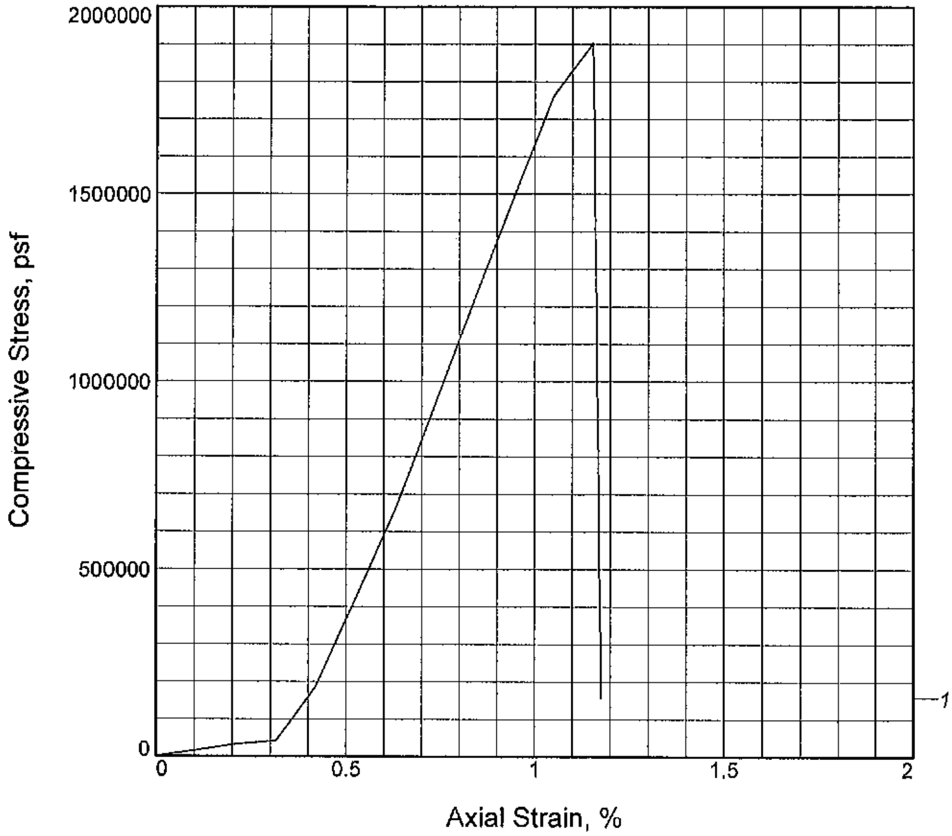
Project: BRENT SPENCE BRIDGE REPLACEMENT

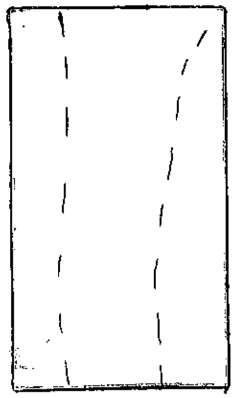
Source of Sample: R-2A **Depth:** 175.8-176.3'
Sample Number: 15

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1902575.1			
Undrained shear strength, psf	951287.5			
Failure strain, %	1.2			
Strain rate, in./min.	0.047			
Water content, %	0.1			
Wet density, pcf	165.6			
Dry density, pcf	165.4			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.390			
Specimen height, in.	4.760			
Height/diameter ratio	1.99			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9712

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

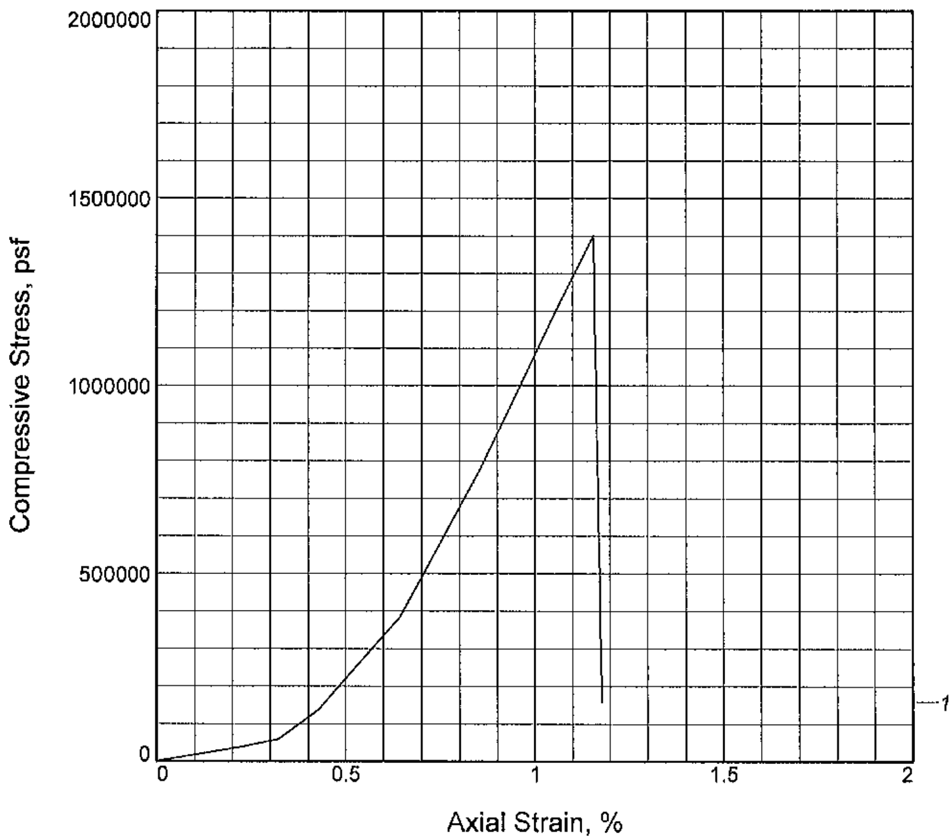
Source of Sample: R-2A **Depth:** 179.8-180.3'
Sample Number: 16

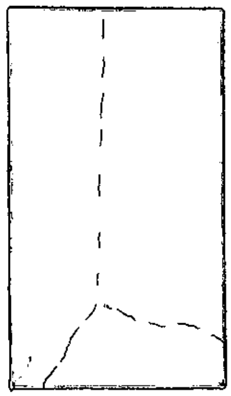
UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Figure ________

Tested By: MRE **Checked By:** GS

UNCONFINED COMPRESSION TEST





Sample No.	1			
Unconfined strength, psf	1400566.4			
Undrained shear strength, psf	700283.2			
Failure strain, %	1.2			
Strain rate, in./min.	0.046			
Water content, %	0.1			
Wet density, pcf	168.8			
Dry density, pcf	168.7			
Saturation, %	N/A			
Void ratio	N/A			
Specimen diameter, in.	2.380			
Specimen height, in.	4.670			
Height/diameter ratio	1.96			

Description: LIMESTONE

LL =	PL =	PI =	Assumed GS=	Type: Limestone

Project No.: N1105070
Date Sampled: 10-7-10
Remarks:
Lab No. 9713

Figure ________

Client: PARSONS BRINCKERHOFF

Project: BRENT SPENCE BRIDGE REPLACEMENT

Source of Sample: R-2A **Depth:** 183.5-184'
Sample Number: 17

UNCONFINED COMPRESSION TEST
H.C. Nutting
A Terracon Company

Tested By: MRE **Checked By:** GS

POINT LOAD TESTING RESULTS

POINT LOAD TEST RESULTS

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Is (psi)	UCS (psi)
L-1	139.5	353.96	140	353.46	733	14651
L-1	145	348.46	145.5	347.96	795	15893
L-1A	140.1	351.35	140.7	350.75	458	9157
L-1A	152.6	338.85	153	338.45	617	12346
L-1A	154.5	336.95	155.3	336.15	597	11932
L-2A	142.5	352.00	142.9	351.60	607	12131
L-2A	165.2	329.30	165.7	328.80	677	13547
L-3	93.1	365.56	93.6	365.06	460	9195
L-3	117.6	341.06	118.4	340.26	589	11786
L-3	137.2	321.46	138	320.66	522	10439
L-3	147.8	310.86	148.3	310.36	522	10434
L-3A	134.6	361.45	135	361.05	742	14846
L-3A	152.75	343.30	153.5	342.55	649	12976
L-4	132.4	347.57	132.9	347.07	585	11696
L-4	141.4	338.57	141.8	338.17	593	11853
L-5	122.7	363.63	123.3	363.03	736	14712
L-5	143.5	342.83	144	342.33	523	10455
L-6	114	371.69	115	370.69	586	11720
L-6	126.3	359.39	126.8	358.89	604	12087
L-7	116	368.41	116.5	367.91	544	10879
L-7	139.7	344.71	140.2	344.21	576	11517
R-1	101	357.04	101.5	356.54	523	10455
R-1	110.2	347.84	110.9	347.14	64	1282
R-1	129.4	328.64	129.8	328.24	555	11103
R-1	145.7	312.34	146.5	311.54	655	13095
R-1	161.8	296.24	162.5	295.54	731	14614
R-2	107.7	350.40	108.5	349.60	289	5783
R-2	130.7	327.40	131.1	327.00	529	10575
R-2	148.5	309.60	149	309.10	644	12884
R-2	159.5	298.60	160.1	298.00	648	12962
R-2A	105.1	352.54	105.5	352.14	451	9027
R-2A	131.5	326.14	132	325.64	407	8142
R-2A	141.2	316.44	141.5	316.14	551	11014
R-2A	166.9	290.74	167.5	290.14	499	9985

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	Is (psi)	UCS (psi)
R-3	98	360.01	98.7	359.31	664	13271
R-3	113.3	344.71	113.8	344.21	652	13042
R-3	117.2	340.81	117.9	340.11	528	10568
R-4	95	362.98	95.5	362.48	596	11920
R-4	101	356.98	101.6	356.38	664	13271
R-4	147	310.98	147.4	310.58	624	12473
R-4	155.5	302.48	155.9	302.08	652	13035
R-5	100.8	357.79	101.5	357.09	551	11011
R-5	108	350.59	108.5	350.09	810	16192
R-5	118.2	340.39	118.8	339.79	553	11057
R-5	156.4	302.19	157	301.59	720	14406
R-6	91.5	365.54	92	365.04	582	11637
R-6	105	352.04	106	351.04	630	12607
R-6	124.7	332.34	125.1	331.94	580	11607
R-6	153.1	303.94	153.9	303.14	655	13102
R-7	89.7	368.76	90.1	368.36	649	12982
R-7	100.8	357.66	101.6	356.86	599	11981
R-7	125.9	332.56	126.7	331.76	746	14914
R-7	145.5	312.96	146.1	312.36	657	13149
R-8	96	359.70	96.5	359.20	533	10656
R-8	118.6	337.10	119	336.70	533	10656

ELASTIC MODULUS TESTING RESULTS

ELASTIC MODULUS TEST RESULTS

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft)	Bottom Elevation (ft)	Unconfined Compressive Strength (psi)	Young's Modulus (ksi)	Rock Type
L-1	150.7	342.76	151.3	342.16	21926	9302	Limestone
L-1A	150	341.45	150.4	341.05	21828	9750	Limestone
L-2A	150.9	343.60	151.4	343.10	16544	8863	Limestone
L-3	113.2	345.46	114.2	344.46	21169	9808	Limestone
L-3	121.8	336.86	122.8	335.86	13540	6323	Limestone
L-3A	155	341.05	155.5	340.55	16975	7601	Limestone
L-4	127.5	352.47	128	351.97	17130	8666	Limestone
L-5	137.3	349.03	138	348.33	20794	9086	Limestone
L-6	138	347.69	138.3	347.39	25530	9219	Limestone
L-7	125.7	358.71	126.2	358.21	23281	9443	Limestone
R-1	115	343.04	115.9	342.14	12584	8636	Limestone
R-1	137.7	320.34	138.2	319.84	15380	6475	Limestone
R-1	146.5	311.54	147	311.04	20779	10022	Limestone
R-2	143.5	314.60	144	314.10	13836	8461	Limestone
R-2	155.3	302.80	155.6	302.50	26538	7518	Limestone
R-2A	112.7	344.94	113.2	344.44	10771	8474	Limestone
R-2A	125.5	332.14	126	331.64	10193	7020	Limestone
R-3	106	352.01	106.5	351.51	14729	6789	Limestone
R-3	136.5	321.51	137	321.01	24544	6685	Limestone
R-4	120.6	337.38	121.3	336.68	19133	10417	Limestone
R-4	139.6	318.38	140.5	317.48	16884	8452	Limestone
R-5	103.5	355.09	104	354.59	14991	5369	Limestone
R-5	128.1	330.49	128.8	329.79	19640	8454	Limestone
R-6	99.6	357.44	100.1	356.94	14253	6276	Limestone
R-6	158.4	298.64	158.9	298.14	22557	9098	Limestone
R-7	106.2	352.26	106.7	351.76	16419	8899	Limestone
R-8	101.5	354.20	102.2	353.50	10883	7995	Limestone
R-8	122.9	332.80	123.3	332.40	14846	8419	Limestone

SLAKE DURABILITY TESTING RESULTS

SLAKE DURABILITY INDEX TEST RESULTS

Boring	Top Depth (ft.)	Top Elevation (ft.)	Bottom Depth (ft.)	Bottom Elevation (ft.)	SDI
L-1	137.1	356.36	137.8	355.66	68
L-1	147.7	345.76	149.2	344.26	67.3
L-1	157	336.46	157.8	335.66	67.8
L-2	133	363.26	133.5	362.76	89.9
L-2	143.5	352.76	144	352.26	91.4
L-2	148.2	348.06	145.5	350.76	88.2
L-2A	138.1	356.40	138.9	355.60	75.3
L-2A	147.3	347.20	147.9	346.60	40.1
L-3A	134.25	361.80	134.5	361.55	85.3
L-3A	150.5	345.55	150.7	345.35	97.7
L-4	108.5	371.47	109.5	370.47	59.2
L-5	109	377.33	109.4	376.93	50.9
L-5	118.5	367.83	118.8	367.53	48.1
L-6	110	375.69	110.4	375.29	56.9
L-6	117.7	367.99	118	367.69	55.1
L-7	105.5	378.91	105.8	378.61	65.9
L-7	107.5	376.91	107.8	376.61	93.3
L-7	118.6	365.81	11.8	472.61	77
R-2	88.2	369.90	88.5	369.60	67.9
R-2	89	369.10	89.3	368.80	82.5
R-2	93.7	364.40	94	364.10	93.6
R-2	100.4	357.70	101.3	356.80	94.1
R-2	134	324.10	134.3	323.80	91.7
R-5	92.2	366.39	92.8	365.79	57.9
R-5	95.7	362.89	95.9	362.69	52.5
R-5	153	305.59	153.5	305.09	98.8
R-6	85.1	371.94	85.6	371.44	36.9
R-6	91.5	365.54	92	365.04	53.6
R-6	100.5	356.54	101	356.04	91
R-7	93.4	365.06	93.6	364.86	79.5
R-7	95.7	362.76	96	362.46	72.8
R-7	102	356.46	102.2	356.26	92.6
R-7	121.1	337.36	121.4	337.06	80.4
R-8	88.4	367.30	88.9	366.80	66.8

APPENDIX C
SUPPORTING DOCUMENTS

EXHIBIT C-1
ODOT CLASSIFICATION

SOIL AND ROCK SYMBOLOGY

Ohio Department of Transportation

SOIL

SYMBOL	DESCRIPTION	Classification AASHTO	Classification OHIO
	Gravel and/or Stone Fragments	A-1-a	
	Gravel and/or Stone Fragments with Sand	A-1-b	
F S	Fine Sand	A-3	
	Coarse and Fine Sand	--	A-3a
	Gravel and/or Stone Fragments with Sand and Silt	A-2-4	
		A-2-5	
	Gravel and/or Stone Fragments with Sand, Silt and Clay	A-2-6	
		A-2-7	
	Sandy Silt	A-4	A-4a
	Silt	A-4	A-4b
	Elastic Silt and Clay	A-5	
	Silt and Clay	A-6	A-6a
	Silty Clay	A-6	A-6b
	Elastic Clay	A-7-5	
	Clay	A-7-6	
	Organic Silt	A-8	A-8a
	Organic Clay	A-8	A-8b

VISUALLY CLASSIFIED MATERIALS

Uncontrolled Fill (Describe)	Sod and Topsoil
	Pavement or Base
Bouldery Zone	Peat, S-Sedimentary W-Woody F-Fibrous L-Loamy & etc

ROCK

Anhydrite	Limestone
Breccia	Mudstone
Chert	Sandstone
Claystone	Shale
C Coal	Siltstone
Conglomerate	Underclay
Dolomite	Use for highly or severely weathered descriptions. Not applicable to coal, fireclay or underclay
Fireclay	
Flint	
Gypsum	
Halite	
Interbedded Shale and Limestone	
Ironstone	

EXHIBIT C-2
DRILLED SHAFT BASE & SHAFT RESISTANCE CALCULATIONS

Drilled Shaft Side Resistance Calculations

Location	Avg. RQD (%)	Design Unconfined Compressive Strength (q_u, psi)[5]	Design Elastic Modulus (E_I, ksi)	Rock Mass Rating (RMR)[1]		Rock Mass Modulus (E_M, ksi)[2]	Rock Mass Modulus/ Intact Rock Modulus (E_M/E_I)	Jointed Rock Reduction Factor (α_E)[3]	Nominal Shaft Resistance (q_s, ksf)[4]		
									Rock	<	Concrete[6]
Ohio-Land	38%	4000	6043	47	III (Fair Rock)	1220	0.20	0.63	14.3		22.7
Ohio-River	67%	4800	5311	52	III (Fair Rock)	1627	0.31	0.71	17.7		22.7
Kentucky-River	59%	4800	4757	52	III (Fair Rock)	1627	0.34	0.72	17.9		22.7
Kentucky-Land	49%	4000	6073	47	III (Fair Rock)	1220	0.20	0.63	14.3		22.7

Notes

1 Per AASHTO LRFD Bridge Design Specifications Tables 10.4.6.4-1, 10.4.6.4-2, 10.4.6.4-3
2 Per AASHTO LRFD Bridge Design Specifications Equation 10.4.6.5-1
3 Interpolated From AASHTO LRFD Bridge Design Specifications Table 10.8.3.5.4b-1
4 Per AASHTO LRFD Bridge Design Specifications Equation 10.8.3.5.4b-1, Lower value is selected
5 This is the value used in design computation considering all the variable factors
* Average RQD and elastic modulus values from upper 30 ft. of bedrock
* Example calculation and copies of AASHTO tables provided on following pages

EXAMPLE CALCULATION
(Ohio-River Portion)

Step 1: Obtain average RQD, unconfined compressive strength (q_u), and elastic modulus (E_M) data from field/lab testing

Avg. RQD (%)=	67
Design q_u (psi)=	4800
Design E_M (ksi)=	5311

Step 2: Determine Rock Mass Rating (RMR) using Tables 10.4.6.4-1, 10.4.6.4-2, 10.4.6.4-3 in the AASHTO Manual

Criteria	Rating
1) Strength of Rock	*2*
2) RQD	*13*
3) Joint Spacing	*10*
4) Joint Condition	*20*
5) Groundwater	*7*
Joint Orientation Adjustment	*0*
Total (RMR)=	52
Rock Mass Class=	III (Fair Rock)

Step 3: Determine Rock Mass Modulus using Equation 10.4.6.5-1

$$E_M = 145\left(10^{\frac{RMR-10}{40}}\right)$$

$$E_M = 145\left(10^{\frac{52-10}{40}}\right) = 1627 ksi$$

Step 4: Determine Ratio of Rock Mass Modulus (E_M) to Intact Rock Modulus (E_I)

$$\frac{E_M}{E_I} = \frac{1627ksi}{5311ksi} = 0.31$$

Step 5: Interpolate Jointed Rock Reduction Factor from Table 10.8.3.5.4b-1

$$\frac{0.3-0.1}{0.7-0.55} = \frac{0.3-0.31}{0.7-\alpha_E}$$

$$\alpha_E = 0.71$$

Step 6: Calculate Shaft Resistance using Equation 10.8.3.5.4b-1. Select lower of two values calculated

$$q_s = 0.65 \cdot \alpha_E \cdot p_a\left(\frac{q_u}{p_a}\right)^{0.5} < 7.8 \cdot p_a\left(\frac{f'c}{p_a}\right)^{0.5}$$

p_a=2.12 ksf (Atmospheric Pressure)

f'c= 4 ksi (Concrete Compressive Strength)

$$q_S = 0.65 \cdot 0.71 \cdot 2.12 \cdot \left(\frac{(4800\frac{144}{1000}}{2.12}\right)^{0.5} < 7.8 \cdot 2.12 \cdot \left(\frac{4}{2.12}\right)^{0.5}$$

$$q_S = 17.7ksf < 22.7ksf$$

Shaft Resistance (q_s)= 17.7 ksf

Step 7: Calculate Base Resistance using Equation 10.8.3.5.4c-1

$$q_p = 2.5 \cdot q_u$$

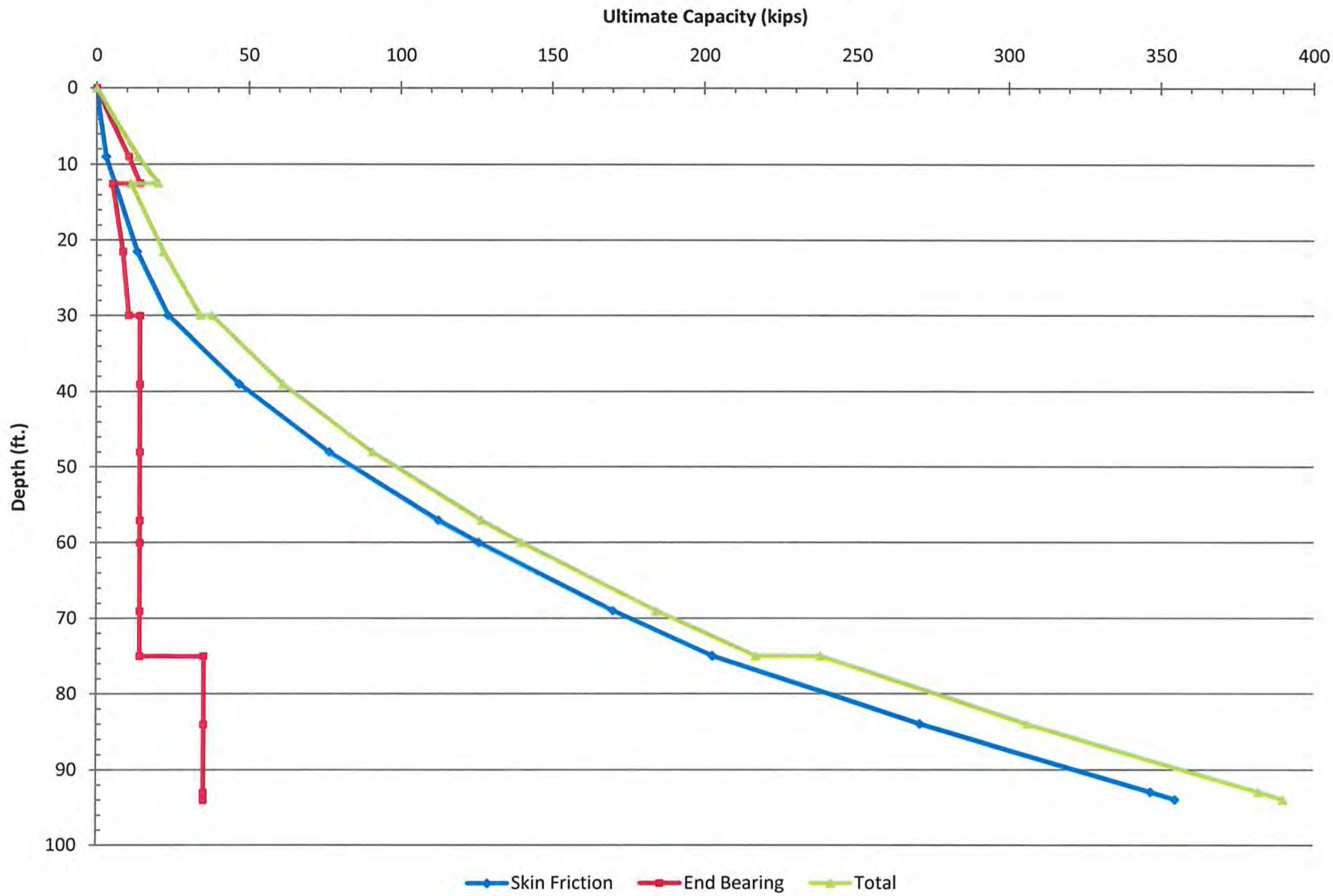
$$q_p = 2.5 \cdot (4800\ psi * \frac{144}{1000})$$

$$q_p = 1728\ ksf$$

A value of 350 ksf has been recommended for use in the design. See report text.

EXHIBIT C-3
DRIVEN PILE CALCULATIONS
(DRIVEN & GRLWEAP)

Ohio-Land
14" Diameter CIP Pile Ultimate Capacity



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\L2A_14.DVN
Project Name: BSB Project Date: 12/07/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 0.00 ft
Diameter of Pile: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	12.50 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
2	Cohesionless	17.50 ft	0.00%	120.00 pcf	24.0/24.0	Nordlund
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	15.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund
5	Cohesionless	30.00 ft	0.00%	125.00 pcf	32.0/32.0	Nordlund
6	Cohesionless	23.50 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.31 psf	19.99	N/A	0.00 Kips
9.01 ft	Cohesionless	282.01 psf	19.99	N/A	3.17 Kips
12.49 ft	Cohesionless	390.94 psf	19.99	N/A	6.10 Kips
12.51 ft	Cohesionless	782.79 psf	16.00	N/A	6.11 Kips
21.51 ft	Cohesionless	1041.99 psf	16.00	N/A	13.40 Kips
29.99 ft	Cohesionless	1286.21 psf	16.00	N/A	23.58 Kips
30.01 ft	Cohesionless	1790.81 psf	19.99	N/A	23.61 Kips
39.01 ft	Cohesionless	2072.51 psf	19.99	N/A	46.91 Kips
48.01 ft	Cohesionless	2354.21 psf	19.99	N/A	76.53 Kips
57.01 ft	Cohesionless	2635.91 psf	19.99	N/A	112.49 Kips
59.99 ft	Cohesionless	2729.19 psf	19.99	N/A	125.79 Kips
60.01 ft	Cohesionless	3668.79 psf	19.99	N/A	125.88 Kips
69.01 ft	Cohesionless	3927.99 psf	19.99	N/A	170.03 Kips
74.99 ft	Cohesionless	4100.21 psf	19.99	N/A	202.59 Kips
75.01 ft	Cohesionless	4532.81 psf	21.33	N/A	202.71 Kips
84.01 ft	Cohesionless	4814.51 psf	21.33	N/A	270.74 Kips
93.01 ft	Cohesionless	5096.21 psf	21.33	N/A	346.73 Kips
102.01 ft	Cohesionless	5377.91 psf	21.33	N/A	430.68 Kips
104.99 ft	Cohesionless	5471.19 psf	21.33	N/A	460.23 Kips
105.01 ft	Cohesionless	6410.81 psf	22.66	N/A	460.45 Kips
114.01 ft	Cohesionless	6692.51 psf	22.66	N/A	575.21 Kips
123.01 ft	Cohesionless	6974.21 psf	22.66	N/A	699.64 Kips
128.49 ft	Cohesionless	7145.74 psf	22.66	N/A	780.13 Kips

ULTIMATE - END BEARING

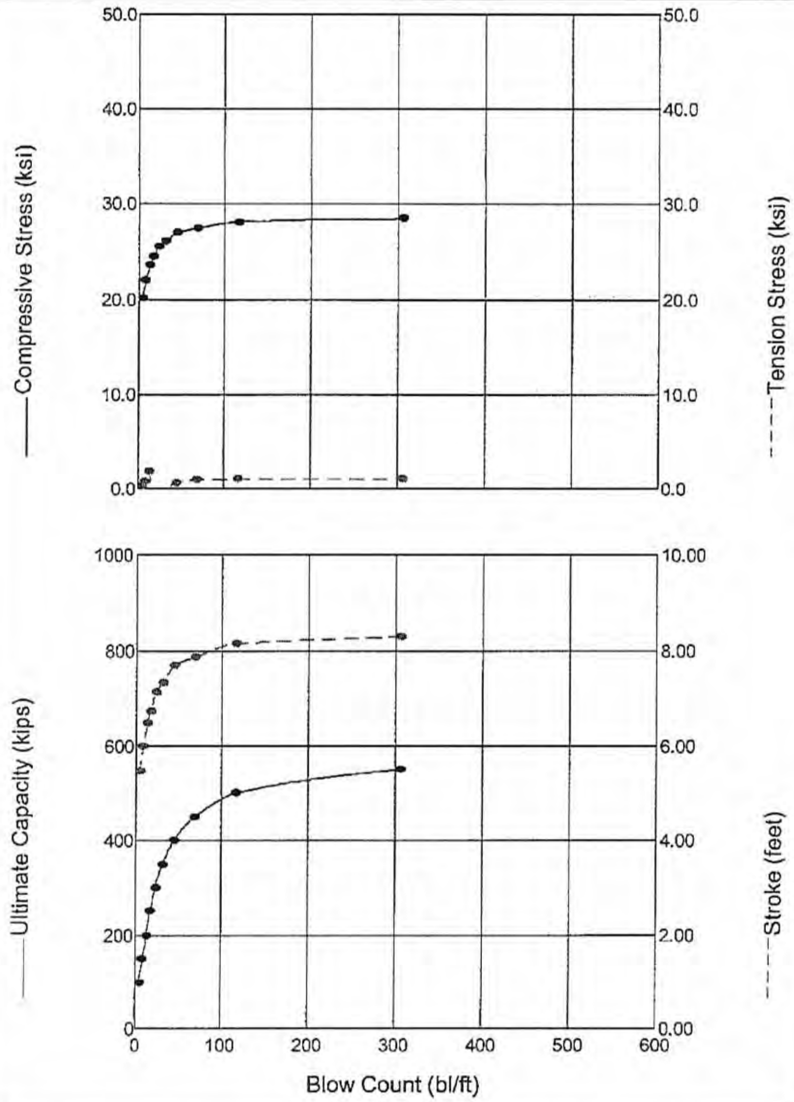
Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.63 psf	30.00	14.24 Kips	0.01 Kips
9.01 ft	Cohesionless	564.03 psf	30.00	14.24 Kips	10.49 Kips
12.49 ft	Cohesionless	781.87 psf	30.00	14.24 Kips	14.24 Kips
12.51 ft	Cohesionless	783.08 psf	13.80	14.24 Kips	5.18 Kips
21.51 ft	Cohesionless	1301.48 psf	13.80	14.24 Kips	8.61 Kips
29.99 ft	Cohesionless	1789.92 psf	13.80	14.24 Kips	10.52 Kips
30.01 ft	Cohesionless	1791.13 psf	30.00	14.24 Kips	14.24 Kips
39.01 ft	Cohesionless	2354.53 psf	30.00	14.24 Kips	14.24 Kips
48.01 ft	Cohesionless	2917.93 psf	30.00	14.24 Kips	14.24 Kips
57.01 ft	Cohesionless	3481.33 psf	30.00	14.24 Kips	14.24 Kips
59.99 ft	Cohesionless	3667.87 psf	30.00	14.24 Kips	14.24 Kips
60.01 ft	Cohesionless	3669.08 psf	30.00	14.24 Kips	14.24 Kips
69.01 ft	Cohesionless	4187.48 psf	30.00	14.24 Kips	14.24 Kips
74.99 ft	Cohesionless	4531.92 psf	30.00	14.24 Kips	14.24 Kips
75.01 ft	Cohesionless	4533.13 psf	40.40	35.28 Kips	35.28 Kips
84.01 ft	Cohesionless	5096.53 psf	40.40	35.28 Kips	35.28 Kips
93.01 ft	Cohesionless	5659.93 psf	40.40	35.28 Kips	35.28 Kips
102.01 ft	Cohesionless	6223.33 psf	40.40	35.28 Kips	35.28 Kips
104.99 ft	Cohesionless	6409.87 psf	40.40	35.28 Kips	35.28 Kips
105.01 ft	Cohesionless	6411.13 psf	55.60	78.59 Kips	78.59 Kips
114.01 ft	Cohesionless	6974.53 psf	55.60	78.59 Kips	78.59 Kips
123.01 ft	Cohesionless	7537.93 psf	55.60	78.59 Kips	78.59 Kips
128.49 ft	Cohesionless	7880.97 psf	55.60	78.59 Kips	78.59 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.01 Kips	0.01 Kips
9.01 ft	3.17 Kips	10.49 Kips	13.66 Kips
12.49 ft	6.10 Kips	14.24 Kips	20.34 Kips
12.51 ft	6.11 Kips	5.18 Kips	11.29 Kips
21.51 ft	13.40 Kips	8.61 Kips	22.01 Kips
29.99 ft	23.58 Kips	10.52 Kips	34.10 Kips
30.01 ft	23.61 Kips	14.24 Kips	37.85 Kips
39.01 ft	46.91 Kips	14.24 Kips	61.15 Kips
48.01 ft	76.53 Kips	14.24 Kips	90.77 Kips
57.01 ft	112.49 Kips	14.24 Kips	126.73 Kips
59.99 ft	125.79 Kips	14.24 Kips	140.03 Kips
60.01 ft	125.88 Kips	14.24 Kips	140.12 Kips
69.01 ft	170.03 Kips	14.24 Kips	184.27 Kips
74.99 ft	202.59 Kips	14.24 Kips	216.82 Kips
75.01 ft	202.71 Kips	35.28 Kips	237.99 Kips
84.01 ft	270.74 Kips	35.28 Kips	306.02 Kips
93.01 ft	346.73 Kips	35.28 Kips	382.01 Kips
102.01 ft	430.68 Kips	35.28 Kips	465.95 Kips
104.99 ft	460.23 Kips	35.28 Kips	495.50 Kips
105.01 ft	460.45 Kips	78.59 Kips	539.04 Kips
114.01 ft	575.21 Kips	78.59 Kips	653.81 Kips
123.01 ft	699.64 Kips	78.59 Kips	778.23 Kips
128.49 ft	780.13 Kips	78.59 Kips	858.72 Kips

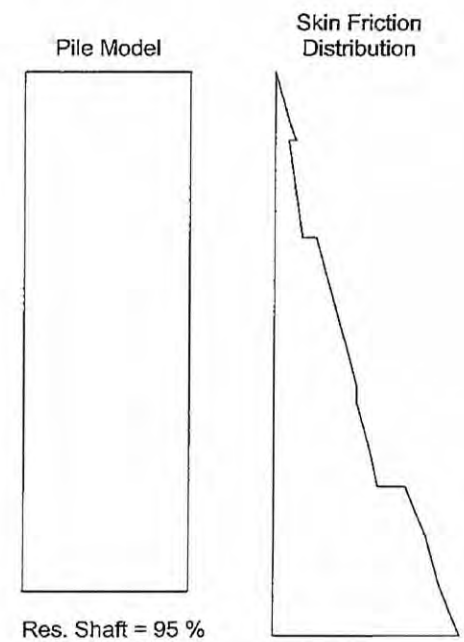
HC Nutting Company
BSB : 12/07/2010 : DWW

15-Dec-2010
GRLWEAP (TM) Version 1998-1



DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	94.00 ft
Pile Top Area	18.33 in2

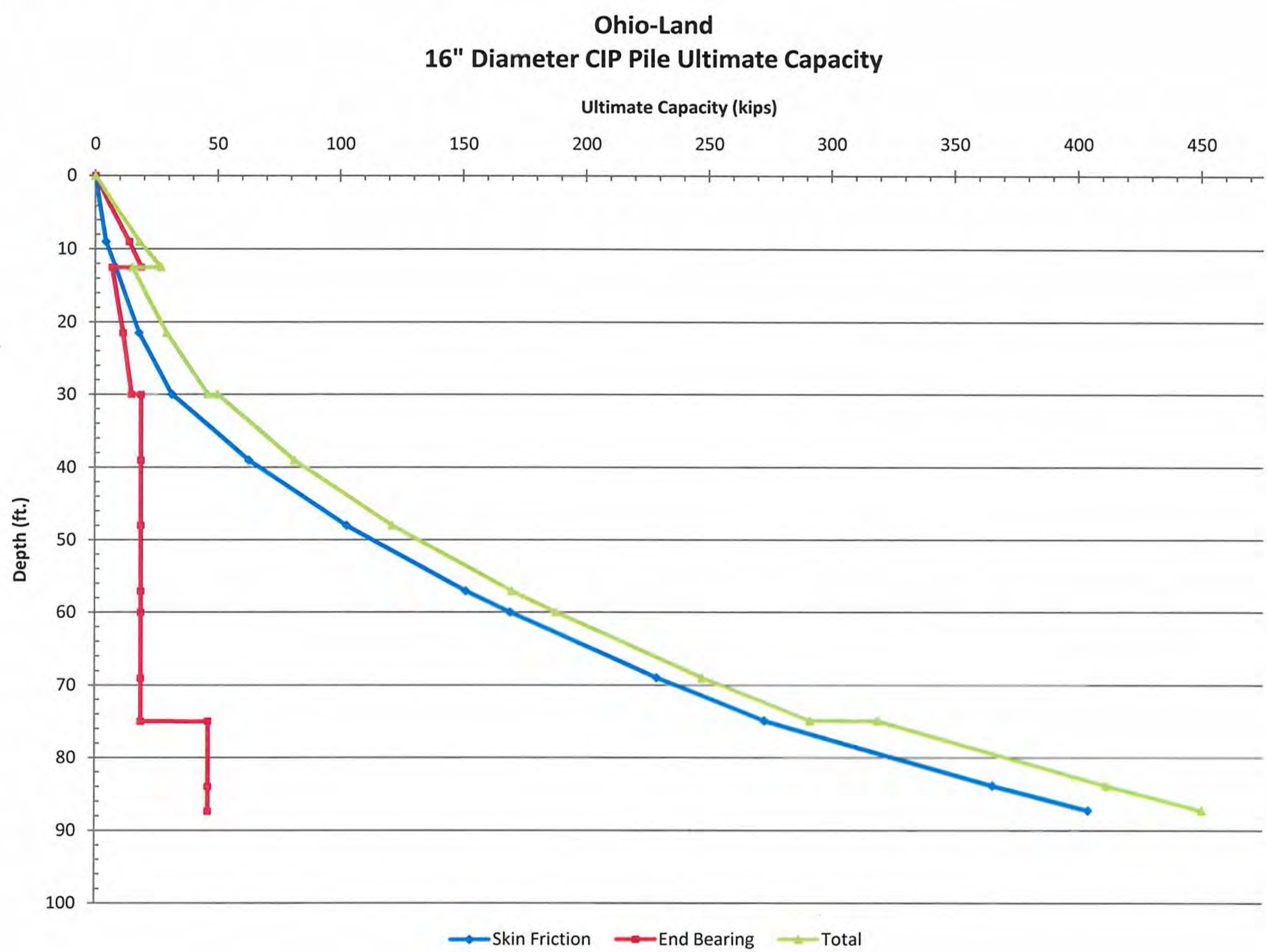


——— 1st Pile
– – – – 2nd Pile

HC Nutting Company
BSB : 12/07/2010 : DWW

15-Dec-2010
GRLWEAP(TM) Version 1998-1

Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
100.0	20.136	0.410	5.4	5.47	33.14
150.0	22.006	0.876	9.0	5.99	29.75
200.0	23.564	1.922	13.9	6.49	28.02
250.0	24.484	0.000	18.2	6.74	26.27
300.0	25.544	0.000	23.8	7.14	26.77
350.0	26.142	0.000	32.7	7.34	26.70
400.0	26.985	0.696	45.3	7.70	27.50
450.0	27.470	0.993	69.5	7.88	27.71
500.0	28.112	1.060	117.1	8.16	28.32
550.0	28.483	1.087	306.8	8.30	28.46


Ohio-Land
16" Diameter CIP Pile Ultimate Capacity
Ultimate Capacity (kips)
0
50
100
150
200
250
300
350
400
450
Depth (ft.)
10
20
30
40
50
60
70
80
90
100
Skin Friction
End Bearing
Total

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\L2A_16.DVN
Project Name: BSB Project Date: 12/07/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 0.00 ft
Diameter of Pile: 16.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	12.50 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
2	Cohesionless	17.50 ft	0.00%	120.00 pcf	24.0/24.0	Nordlund
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	15.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund
5	Cohesionless	30.00 ft	0.00%	125.00 pcf	32.0/32.0	Nordlund
6	Cohesionless	23.50 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

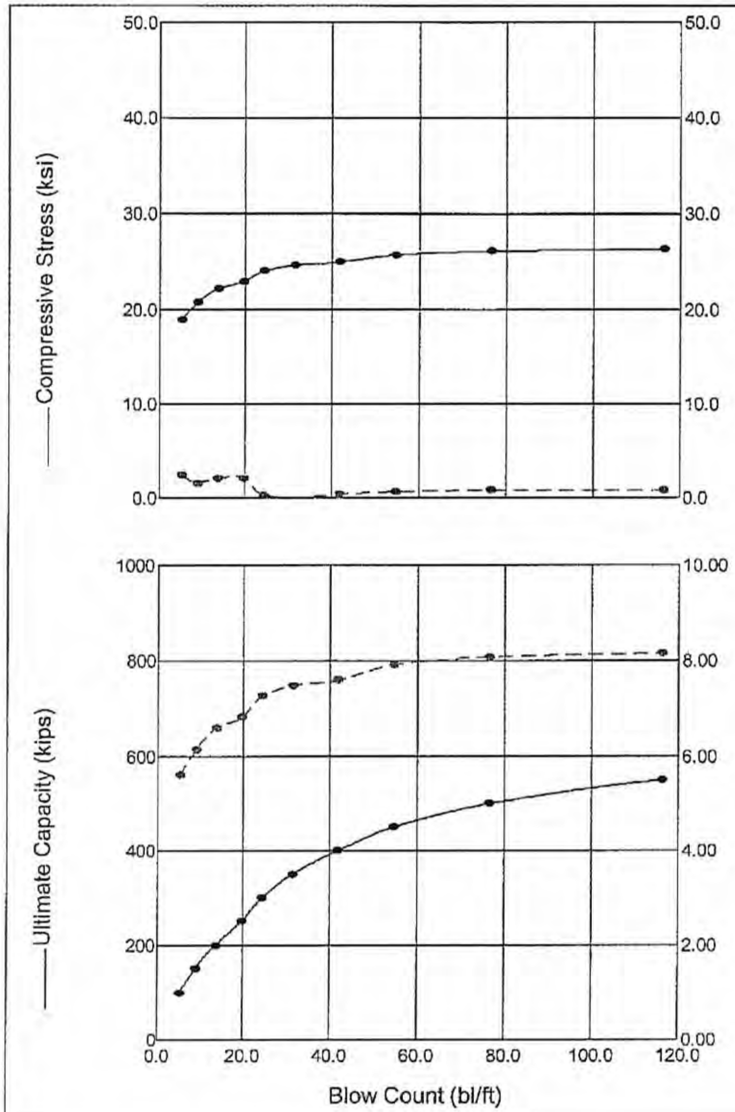
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.31 psf	21.97	N/A	0.00 Kips
9.01 ft	Cohesionless	282.01 psf	21.97	N/A	4.27 Kips
12.49 ft	Cohesionless	390.94 psf	21.97	N/A	8.21 Kips
12.51 ft	Cohesionless	782.79 psf	17.58	N/A	8.24 Kips
21.51 ft	Cohesionless	1041.99 psf	17.58	N/A	17.85 Kips
29.99 ft	Cohesionless	1286.21 psf	17.58	N/A	31.29 Kips
30.01 ft	Cohesionless	1790.81 psf	21.97	N/A	31.34 Kips
39.01 ft	Cohesionless	2072.51 psf	21.97	N/A	62.72 Kips
48.01 ft	Cohesionless	2354.21 psf	21.97	N/A	102.64 Kips
57.01 ft	Cohesionless	2635.91 psf	21.97	N/A	151.09 Kips
59.99 ft	Cohesionless	2729.19 psf	21.97	N/A	169.01 Kips
60.01 ft	Cohesionless	3668.79 psf	21.97	N/A	169.13 Kips
69.01 ft	Cohesionless	3927.99 psf	21.97	N/A	228.61 Kips
74.99 ft	Cohesionless	4100.21 psf	21.97	N/A	272.47 Kips
75.01 ft	Cohesionless	4532.81 psf	23.44	N/A	272.65 Kips
84.01 ft	Cohesionless	4814.51 psf	23.44	N/A	365.28 Kips
93.01 ft	Cohesionless	5096.21 psf	23.44	N/A	468.76 Kips
102.01 ft	Cohesionless	5377.91 psf	23.44	N/A	583.07 Kips
104.99 ft	Cohesionless	5471.19 psf	23.44	N/A	623.31 Kips
105.01 ft	Cohesionless	6410.81 psf	24.90	N/A	623.62 Kips
114.01 ft	Cohesionless	6692.51 psf	24.90	N/A	781.38 Kips
123.01 ft	Cohesionless	6974.21 psf	24.90	N/A	952.43 Kips
128.49 ft	Cohesionless	7145.74 psf	24.90	N/A	1063.09 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.63 psf	30.00	18.60 Kips	0.02 Kips
9.01 ft	Cohesionless	564.03 psf	30.00	18.60 Kips	13.70 Kips
12.49 ft	Cohesionless	781.87 psf	30.00	18.60 Kips	18.60 Kips
12.51 ft	Cohesionless	783.08 psf	13.80	18.60 Kips	6.77 Kips
21.51 ft	Cohesionless	1301.48 psf	13.80	18.60 Kips	11.24 Kips
29.99 ft	Cohesionless	1789.92 psf	13.80	18.60 Kips	14.71 Kips
30.01 ft	Cohesionless	1791.13 psf	30.00	18.60 Kips	18.60 Kips
39.01 ft	Cohesionless	2354.53 psf	30.00	18.60 Kips	18.60 Kips
48.01 ft	Cohesionless	2917.93 psf	30.00	18.60 Kips	18.60 Kips
57.01 ft	Cohesionless	3481.33 psf	30.00	18.60 Kips	18.60 Kips
59.99 ft	Cohesionless	3667.87 psf	30.00	18.60 Kips	18.60 Kips
60.01 ft	Cohesionless	3669.08 psf	30.00	18.60 Kips	18.60 Kips
69.01 ft	Cohesionless	4187.48 psf	30.00	18.60 Kips	18.60 Kips
74.99 ft	Cohesionless	4531.92 psf	30.00	18.60 Kips	18.60 Kips
75.01 ft	Cohesionless	4533.13 psf	40.40	46.08 Kips	46.08 Kips
84.01 ft	Cohesionless	5096.53 psf	40.40	46.08 Kips	46.08 Kips
93.01 ft	Cohesionless	5659.93 psf	40.40	46.08 Kips	46.08 Kips
102.01 ft	Cohesionless	6223.33 psf	40.40	46.08 Kips	46.08 Kips
104.99 ft	Cohesionless	6409.87 psf	40.40	46.08 Kips	46.08 Kips
105.01 ft	Cohesionless	6411.13 psf	55.60	102.65 Kips	102.65 Kips
114.01 ft	Cohesionless	6974.53 psf	55.60	102.65 Kips	102.65 Kips
123.01 ft	Cohesionless	7537.93 psf	55.60	102.65 Kips	102.65 Kips
128.49 ft	Cohesionless	7880.97 psf	55.60	102.65 Kips	102.65 Kips

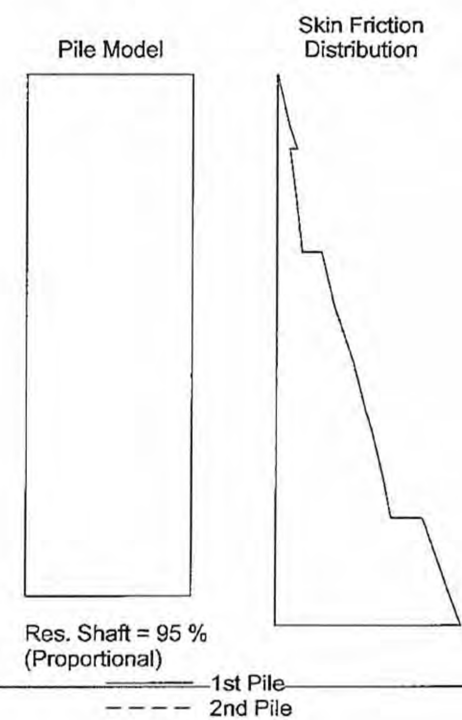
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.02 Kips	0.02 Kips
9.01 ft	4.27 Kips	13.70 Kips	17.98 Kips
12.49 ft	8.21 Kips	18.60 Kips	26.81 Kips
12.51 ft	8.24 Kips	6.77 Kips	15.00 Kips
21.51 ft	17.85 Kips	11.24 Kips	29.10 Kips
29.99 ft	31.29 Kips	14.71 Kips	46.00 Kips
30.01 ft	31.34 Kips	18.60 Kips	49.94 Kips
39.01 ft	62.72 Kips	18.60 Kips	81.32 Kips
48.01 ft	102.64 Kips	18.60 Kips	121.24 Kips
57.01 ft	151.09 Kips	18.60 Kips	169.69 Kips
59.99 ft	169.01 Kips	18.60 Kips	187.61 Kips
60.01 ft	169.13 Kips	18.60 Kips	187.73 Kips
69.01 ft	228.61 Kips	18.60 Kips	247.21 Kips
74.99 ft	272.47 Kips	18.60 Kips	291.07 Kips
75.01 ft	272.65 Kips	46.08 Kips	318.72 Kips
84.01 ft	365.28 Kips	46.08 Kips	411.36 Kips
93.01 ft	468.76 Kips	46.08 Kips	514.83 Kips
102.01 ft	583.07 Kips	46.08 Kips	629.15 Kips
104.99 ft	623.31 Kips	46.08 Kips	669.39 Kips
105.01 ft	623.62 Kips	102.65 Kips	726.27 Kips
114.01 ft	781.38 Kips	102.65 Kips	884.04 Kips
123.01 ft	952.43 Kips	102.65 Kips	1055.09 Kips
128.49 ft	1063.09 Kips	102.65 Kips	1165.74 Kips



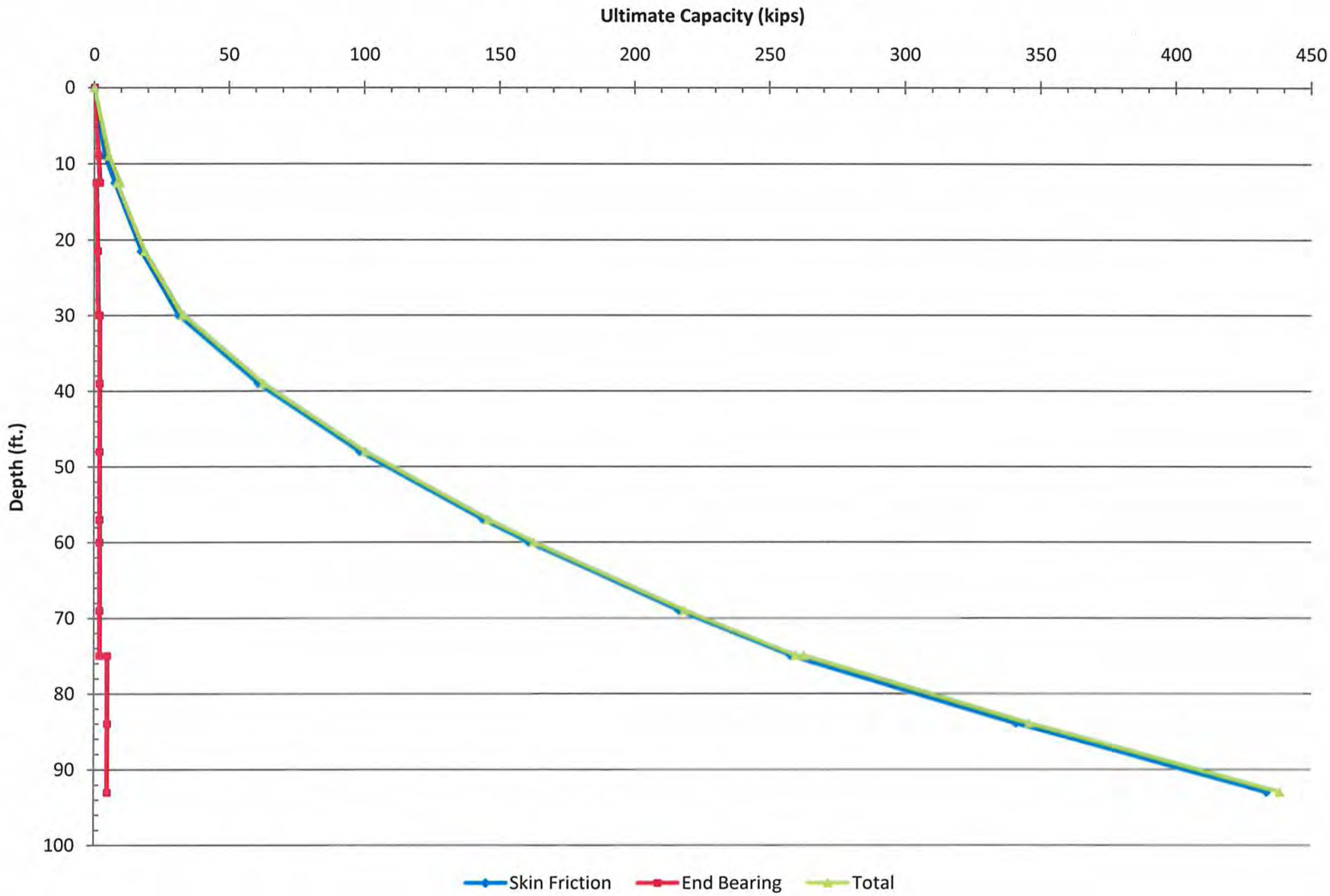
DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	88.00 ft
Pile Top Area	24.35 in2



Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
100.0	18.971	2.373	5.4	5.60	31.98
150.0	20.757	1.547	9.0	6.13	28.44
200.0	22.149	2.042	13.8	6.59	26.85
250.0	22.914	2.081	19.7	6.82	24.83
300.0	24.034	0.274	24.5	7.26	24.30
350.0	24.628	0.000	31.5	7.48	24.19
400.0	24.980	0.414	41.9	7.59	24.09
450.0	25.712	0.637	54.7	7.91	24.79
500.0	26.105	0.834	76.7	8.08	25.01
550.0	26.330	0.835	116.4	8.16	25.02

Ohio-Land
HP 14x73 Pile Ultimate Capacity



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\HPILES\L214X73.DVN
Project Name: BSB Project Date: 12/07/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesionless	12.50 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
2	Cohesionless	17.50 ft	0.00%	120.00 pcf	24.0/24.0	Nordlund
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	15.00 ft	0.00%	120.00 pcf	30.0/30.0	Nordlund
5	Cohesionless	30.00 ft	0.00%	125.00 pcf	32.0/32.0	Nordlund
6	Cohesionless	23.50 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

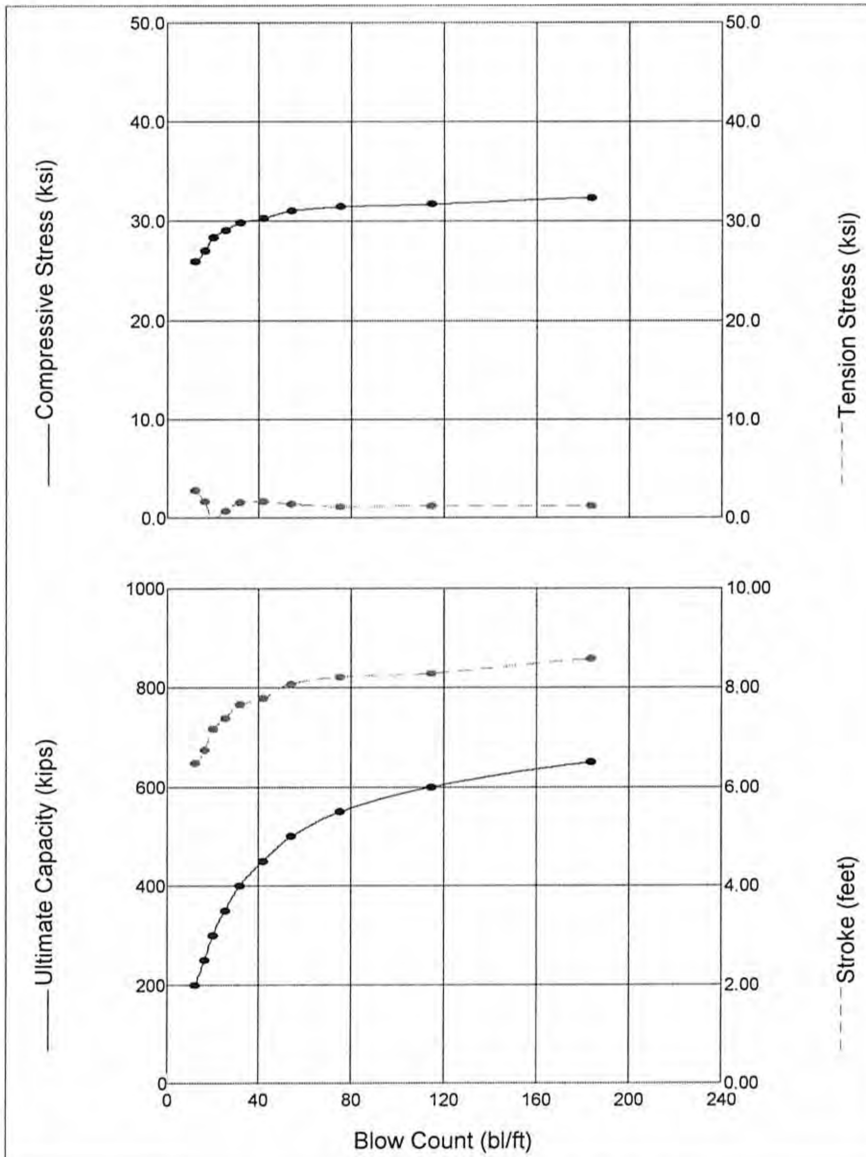
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesionless	0.31 psf	23.58	N/A	0.00 Kips
9.01 ft	Cohesionless	282.01 psf	23.58	N/A	4.01 Kips
12.49 ft	Cohesionless	390.94 psf	23.58	N/A	7.71 Kips
12.51 ft	Cohesionless	782.79 psf	18.86	N/A	7.73 Kips
21.51 ft	Cohesionless	1041.99 psf	18.86	N/A	17.59 Kips
29.99 ft	Cohesionless	1286.21 psf	18.86	N/A	31.37 Kips
30.01 ft	Cohesionless	1790.81 psf	23.58	N/A	31.42 Kips
39.01 ft	Cohesionless	2072.51 psf	23.58	N/A	60.87 Kips
48.01 ft	Cohesionless	2354.21 psf	23.58	N/A	98.33 Kips
57.01 ft	Cohesionless	2635.91 psf	23.58	N/A	143.78 Kips
59.99 ft	Cohesionless	2729.19 psf	23.58	N/A	160.60 Kips
60.01 ft	Cohesionless	3668.79 psf	23.58	N/A	160.72 Kips
69.01 ft	Cohesionless	3927.99 psf	23.58	N/A	216.53 Kips
74.99 ft	Cohesionless	4100.21 psf	23.58	N/A	257.68 Kips
75.01 ft	Cohesionless	4532.81 psf	25.15	N/A	257.84 Kips
84.01 ft	Cohesionless	4814.51 psf	25.15	N/A	340.86 Kips
93.01 ft	Cohesionless	5096.21 psf	25.15	N/A	433.60 Kips
102.01 ft	Cohesionless	5377.91 psf	25.15	N/A	536.05 Kips
104.99 ft	Cohesionless	5471.19 psf	25.15	N/A	572.11 Kips
105.01 ft	Cohesionless	6410.81 psf	26.72	N/A	572.38 Kips
114.01 ft	Cohesionless	6692.51 psf	26.72	N/A	709.29 Kips
123.01 ft	Cohesionless	6974.21 psf	26.72	N/A	857.73 Kips
128.49 ft	Cohesionless	7145.74 psf	26.72	N/A	953.76 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesionless	0.63 psf	30.00	1.98 Kips	0.00 Kips
9.01 ft	Cohesionless	564.03 psf	30.00	1.98 Kips	1.46 Kips
12.49 ft	Cohesionless	781.87 psf	30.00	1.98 Kips	1.98 Kips
12.51 ft	Cohesionless	783.08 psf	13.80	1.98 Kips	0.72 Kips
21.51 ft	Cohesionless	1301.48 psf	13.80	1.98 Kips	1.20 Kips
29.99 ft	Cohesionless	1789.92 psf	13.80	1.98 Kips	1.50 Kips
30.01 ft	Cohesionless	1791.13 psf	30.00	1.98 Kips	1.98 Kips
39.01 ft	Cohesionless	2354.53 psf	30.00	1.98 Kips	1.98 Kips
48.01 ft	Cohesionless	2917.93 psf	30.00	1.98 Kips	1.98 Kips
57.01 ft	Cohesionless	3481.33 psf	30.00	1.98 Kips	1.98 Kips
59.99 ft	Cohesionless	3667.87 psf	30.00	1.98 Kips	1.98 Kips
60.01 ft	Cohesionless	3669.08 psf	30.00	1.98 Kips	1.98 Kips
69.01 ft	Cohesionless	4187.48 psf	30.00	1.98 Kips	1.98 Kips
74.99 ft	Cohesionless	4531.92 psf	30.00	1.98 Kips	1.98 Kips
75.01 ft	Cohesionless	4533.13 psf	40.40	4.90 Kips	4.90 Kips
84.01 ft	Cohesionless	5096.53 psf	40.40	4.90 Kips	4.90 Kips
93.01 ft	Cohesionless	5659.93 psf	40.40	4.90 Kips	4.90 Kips
102.01 ft	Cohesionless	6223.33 psf	40.40	4.90 Kips	4.90 Kips
104.99 ft	Cohesionless	6409.87 psf	40.40	4.90 Kips	4.90 Kips
105.01 ft	Cohesionless	6411.13 psf	55.60	10.93 Kips	10.93 Kips
114.01 ft	Cohesionless	6974.53 psf	55.60	10.93 Kips	10.93 Kips
123.01 ft	Cohesionless	7537.93 psf	55.60	10.93 Kips	10.93 Kips
128.49 ft	Cohesionless	7880.97 psf	55.60	10.93 Kips	10.93 Kips

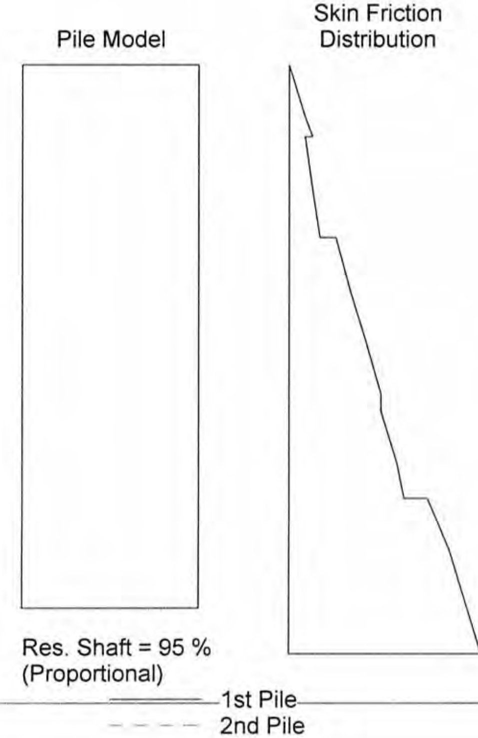
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
9.01 ft	4.01 Kips	1.46 Kips	5.47 Kips
12.49 ft	7.71 Kips	1.98 Kips	9.69 Kips
12.51 ft	7.73 Kips	0.72 Kips	8.45 Kips
21.51 ft	17.59 Kips	1.20 Kips	18.79 Kips
29.99 ft	31.37 Kips	1.50 Kips	32.87 Kips
30.01 ft	31.42 Kips	1.98 Kips	33.40 Kips
39.01 ft	60.87 Kips	1.98 Kips	62.85 Kips
48.01 ft	98.33 Kips	1.98 Kips	100.30 Kips
57.01 ft	143.78 Kips	1.98 Kips	145.76 Kips
59.99 ft	160.60 Kips	1.98 Kips	162.58 Kips
60.01 ft	160.72 Kips	1.98 Kips	162.70 Kips
69.01 ft	216.53 Kips	1.98 Kips	218.51 Kips
74.99 ft	257.68 Kips	1.98 Kips	259.66 Kips
75.01 ft	257.84 Kips	4.90 Kips	262.74 Kips
84.01 ft	340.86 Kips	4.90 Kips	345.77 Kips
93.01 ft	433.60 Kips	4.90 Kips	438.50 Kips
102.01 ft	536.05 Kips	4.90 Kips	540.95 Kips
104.99 ft	572.11 Kips	4.90 Kips	577.01 Kips
105.01 ft	572.38 Kips	10.93 Kips	583.30 Kips
114.01 ft	709.29 Kips	10.93 Kips	720.22 Kips
123.01 ft	857.73 Kips	10.93 Kips	868.66 Kips
128.49 ft	953.76 Kips	10.93 Kips	964.68 Kips



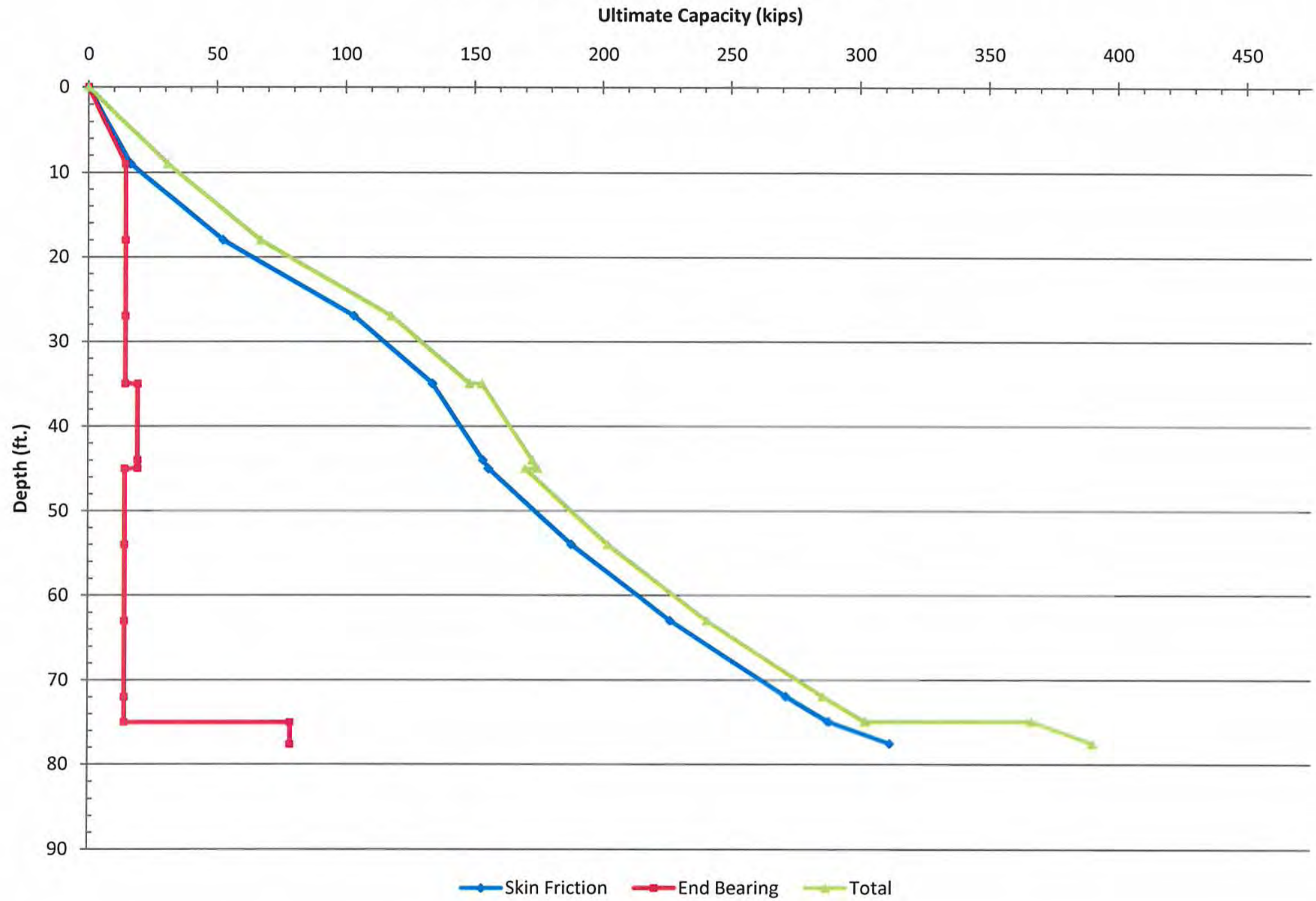
DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.100 sec/ft
Pile Length	94.00 ft
Pile Top Area	21.40 in2



Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
200.0	25.972	2.839	12.6	6.48	31.75
250.0	26.994	1.716	16.5	6.75	30.34
300.0	28.318	0.000	20.2	7.17	30.80
350.0	29.040	0.741	25.5	7.39	31.28
400.0	29.842	1.606	32.0	7.66	32.27
450.0	30.250	1.739	42.0	7.79	32.48
500.0	31.039	1.481	54.1	8.08	33.71
550.0	31.468	1.185	75.2	8.22	34.10
600.0	31.727	1.306	114.7	8.29	34.21
650.0	32.421	1.308	183.7	8.58	35.22

Kentucky-Land
14" Diameter CIP Pile Ultimate Capacity



DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\L5_14.DVN
Project Name: BSB Project Date: 12/09/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 0.00 ft
Diameter of Pile: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	35.00 ft	0.00%	120.00 pcf	1500.00 psf	T-80 Clay
2	Cohesive	10.00 ft	0.00%	120.00 pcf	2000.00 psf	T-80 Clay
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	32.00 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

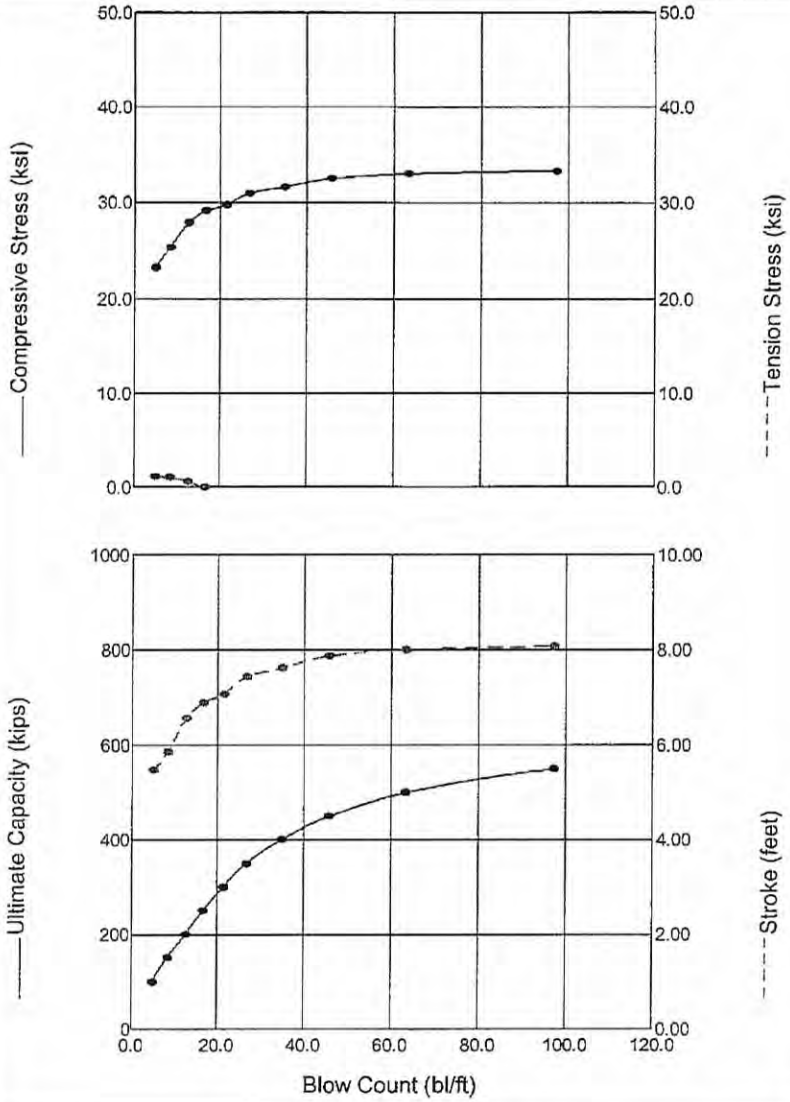
Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	500.59 psf	0.02 Kips
9.01 ft	Cohesive	N/A	N/A	500.59 psf	16.53 Kips
18.01 ft	Cohesive	N/A	N/A	794.62 psf	52.45 Kips
27.01 ft	Cohesive	N/A	N/A	1041.36 psf	103.09 Kips
34.99 ft	Cohesive	N/A	N/A	1041.36 psf	133.55 Kips
35.01 ft	Cohesive	N/A	N/A	595.08 psf	133.61 Kips
44.01 ft	Cohesive	N/A	N/A	595.08 psf	153.24 Kips
44.99 ft	Cohesive	N/A	N/A	595.08 psf	155.38 Kips
45.01 ft	Cohesionless	2592.31 psf	19.99	N/A	155.43 Kips
54.01 ft	Cohesionless	2874.01 psf	19.99	N/A	187.73 Kips
63.01 ft	Cohesionless	3155.71 psf	19.99	N/A	226.37 Kips
72.01 ft	Cohesionless	3437.41 psf	19.99	N/A	271.33 Kips
74.99 ft	Cohesionless	3530.69 psf	19.99	N/A	287.62 Kips
75.01 ft	Cohesionless	4470.31 psf	22.66	N/A	287.76 Kips
84.01 ft	Cohesionless	4752.01 psf	22.66	N/A	369.25 Kips
93.01 ft	Cohesionless	5033.71 psf	22.66	N/A	460.40 Kips
102.01 ft	Cohesionless	5315.41 psf	22.66	N/A	561.21 Kips
106.99 ft	Cohesionless	5471.29 psf	22.66	N/A	621.14 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
27.01 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
34.99 ft	Cohesive	N/A	N/A	N/A	14.43 Kips
35.01 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
44.01 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
44.99 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
45.01 ft	Cohesionless	2592.63 psf	30.00	14.24 Kips	14.24 Kips
54.01 ft	Cohesionless	3156.03 psf	30.00	14.24 Kips	14.24 Kips
63.01 ft	Cohesionless	3719.43 psf	30.00	14.24 Kips	14.24 Kips
72.01 ft	Cohesionless	4282.83 psf	30.00	14.24 Kips	14.24 Kips
74.99 ft	Cohesionless	4469.37 psf	30.00	14.24 Kips	14.24 Kips
75.01 ft	Cohesionless	4470.63 psf	55.60	78.59 Kips	78.59 Kips
84.01 ft	Cohesionless	5034.03 psf	55.60	78.59 Kips	78.59 Kips
93.01 ft	Cohesionless	5597.43 psf	55.60	78.59 Kips	78.59 Kips
102.01 ft	Cohesionless	6160.83 psf	55.60	78.59 Kips	78.59 Kips
106.99 ft	Cohesionless	6472.57 psf	55.60	78.59 Kips	78.59 Kips

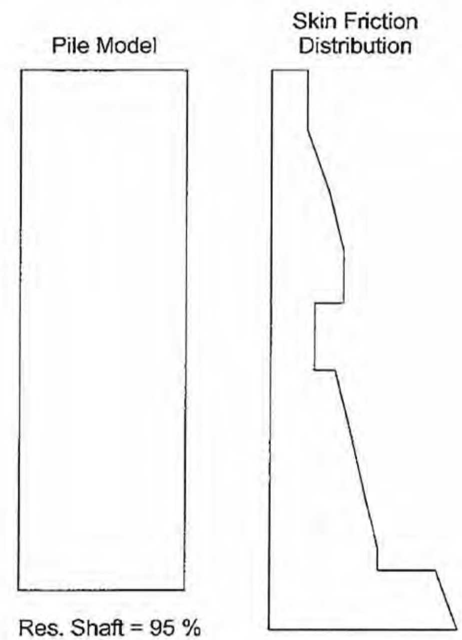
ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.02 Kips	14.43 Kips	14.45 Kips
9.01 ft	16.53 Kips	14.43 Kips	30.96 Kips
18.01 ft	52.45 Kips	14.43 Kips	66.88 Kips
27.01 ft	103.09 Kips	14.43 Kips	117.52 Kips
34.99 ft	133.55 Kips	14.43 Kips	147.98 Kips
35.01 ft	133.61 Kips	19.24 Kips	152.85 Kips
44.01 ft	153.24 Kips	19.24 Kips	172.48 Kips
44.99 ft	155.38 Kips	19.24 Kips	174.62 Kips
45.01 ft	155.43 Kips	14.24 Kips	169.67 Kips
54.01 ft	187.73 Kips	14.24 Kips	201.97 Kips
63.01 ft	226.37 Kips	14.24 Kips	240.61 Kips
72.01 ft	271.33 Kips	14.24 Kips	285.57 Kips
74.99 ft	287.62 Kips	14.24 Kips	301.86 Kips
75.01 ft	287.76 Kips	78.59 Kips	366.35 Kips
84.01 ft	369.25 Kips	78.59 Kips	447.84 Kips
93.01 ft	460.40 Kips	78.59 Kips	538.99 Kips
102.01 ft	561.21 Kips	78.59 Kips	639.80 Kips
106.99 ft	621.14 Kips	78.59 Kips	699.73 Kips



DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	78.00 ft
Pile Top Area	18.33 in2



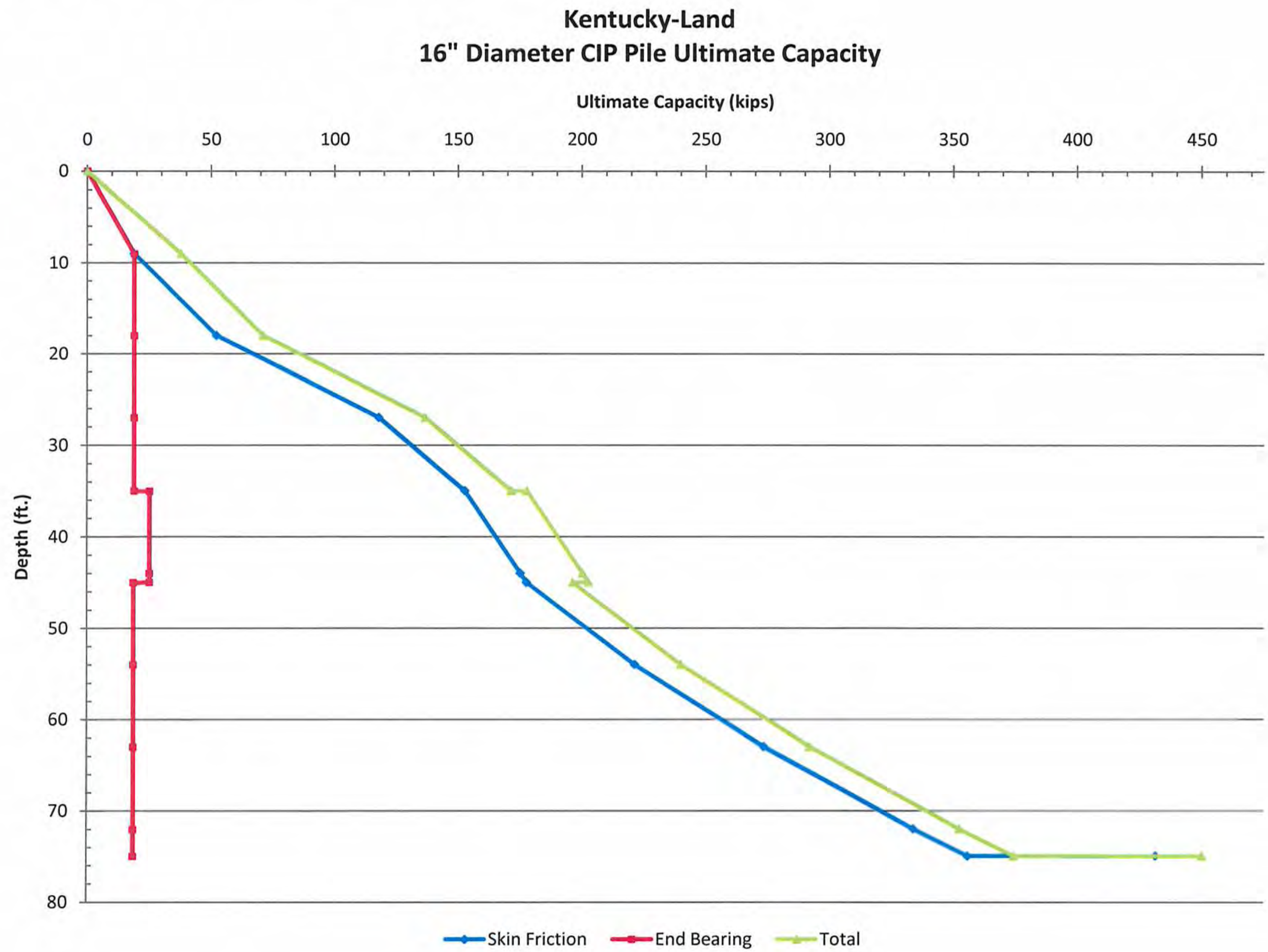
Res. Shaft = 95 %
(Proportional)

——— 1st Pile
– – – – 2nd Pile

HC Nutting Company
BSB : 12/09/2010 : DWW

15-Dec-2010
GRLWEAP(TM) Version 1998-1

Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
100.0	23.235	1.202	5.1	5.49	35.71
150.0	25.375	1.069	8.5	5.86	31.89
200.0	27.872	0.690	12.8	6.55	31.69
250.0	29.104	0.012	16.8	6.90	30.45
300.0	29.711	0.000	21.4	7.07	29.48
350.0	30.999	0.000	26.8	7.44	30.21
400.0	31.714	0.000	34.9	7.63	30.41
450.0	32.650	0.000	45.8	7.88	31.11
500.0	33.182	0.000	63.4	8.01	31.30
550.0	33.458	0.000	97.4	8.07	30.98


Kentucky-Land
16" Diameter CIP Pile Ultimate Capacity
Ultimate Capacity (kips)
0
50
100
150
200
250
300
350
400
450
Depth (ft.)
0
10
20
30
40
50
60
70
80
Skin Friction
End Bearing
Total

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\L5_16.DVN
Project Name: BSB Project Date: 12/09/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 0.00 ft
Diameter of Pile: 16.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	35.00 ft	0.00%	120.00 pcf	1500.00 psf	T-80 Clay
2	Cohesive	10.00 ft	0.00%	120.00 pcf	2000.00 psf	T-80 Clay
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	32.00 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	500.59 psf	0.02 Kips
9.01 ft	Cohesive	N/A	N/A	500.59 psf	18.89 Kips
18.01 ft	Cohesive	N/A	N/A	690.27 psf	52.07 Kips
27.01 ft	Cohesive	N/A	N/A	1041.36 psf	117.82 Kips
34.99 ft	Cohesive	N/A	N/A	1041.36 psf	152.63 Kips
35.01 ft	Cohesive	N/A	N/A	595.08 psf	152.70 Kips
44.01 ft	Cohesive	N/A	N/A	595.08 psf	175.13 Kips
44.99 ft	Cohesive	N/A	N/A	595.08 psf	177.57 Kips
45.01 ft	Cohesionless	2592.31 psf	21.97	N/A	177.64 Kips
54.01 ft	Cohesionless	2874.01 psf	21.97	N/A	221.16 Kips
63.01 ft	Cohesionless	3155.71 psf	21.97	N/A	273.22 Kips
72.01 ft	Cohesionless	3437.41 psf	21.97	N/A	333.80 Kips
74.99 ft	Cohesionless	3530.69 psf	21.97	N/A	355.74 Kips
75.01 ft	Cohesionless	4470.31 psf	24.90	N/A	355.93 Kips
84.01 ft	Cohesionless	4752.01 psf	24.90	N/A	467.96 Kips
93.01 ft	Cohesionless	5033.71 psf	24.90	N/A	593.26 Kips
102.01 ft	Cohesionless	5315.41 psf	24.90	N/A	731.85 Kips
106.99 ft	Cohesionless	5471.29 psf	24.90	N/A	814.24 Kips

ULTIMATE - END BEARING

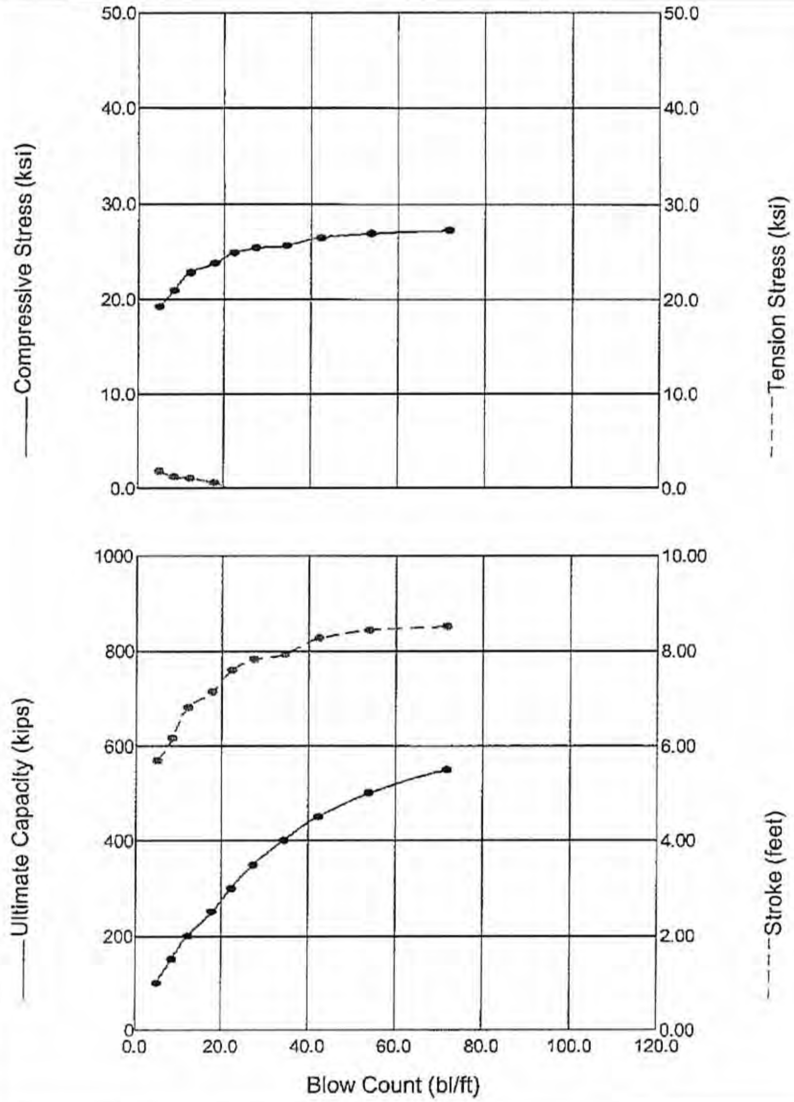
Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	18.85 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	18.85 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	18.85 Kips
27.01 ft	Cohesive	N/A	N/A	N/A	18.85 Kips
34.99 ft	Cohesive	N/A	N/A	N/A	18.85 Kips
35.01 ft	Cohesive	N/A	N/A	N/A	25.13 Kips
44.01 ft	Cohesive	N/A	N/A	N/A	25.13 Kips
44.99 ft	Cohesive	N/A	N/A	N/A	25.13 Kips
45.01 ft	Cohesionless	2592.63 psf	30.00	18.60 Kips	18.60 Kips
54.01 ft	Cohesionless	3156.03 psf	30.00	18.60 Kips	18.60 Kips
63.01 ft	Cohesionless	3719.43 psf	30.00	18.60 Kips	18.60 Kips
72.01 ft	Cohesionless	4282.83 psf	30.00	18.60 Kips	18.60 Kips
74.99 ft	Cohesionless	4469.37 psf	30.00	18.60 Kips	18.60 Kips
75.01 ft	Cohesionless	4470.63 psf	55.60	102.65 Kips	102.65 Kips
84.01 ft	Cohesionless	5034.03 psf	55.60	102.65 Kips	102.65 Kips
93.01 ft	Cohesionless	5597.43 psf	55.60	102.65 Kips	102.65 Kips
102.01 ft	Cohesionless	6160.83 psf	55.60	102.65 Kips	102.65 Kips
106.99 ft	Cohesionless	6472.57 psf	55.60	102.65 Kips	102.65 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.02 Kips	18.85 Kips	18.87 Kips
9.01 ft	18.89 Kips	18.85 Kips	37.74 Kips
18.01 ft	52.07 Kips	18.85 Kips	70.92 Kips
27.01 ft	117.82 Kips	18.85 Kips	136.67 Kips
34.99 ft	152.63 Kips	18.85 Kips	171.48 Kips
35.01 ft	152.70 Kips	25.13 Kips	177.83 Kips
44.01 ft	175.13 Kips	25.13 Kips	200.26 Kips
44.99 ft	177.57 Kips	25.13 Kips	202.71 Kips
45.01 ft	177.64 Kips	18.60 Kips	196.24 Kips
54.01 ft	221.16 Kips	18.60 Kips	239.76 Kips
63.01 ft	273.22 Kips	18.60 Kips	291.81 Kips
72.01 ft	333.80 Kips	18.60 Kips	352.40 Kips
74.99 ft	355.74 Kips	18.60 Kips	374.34 Kips
75.01 ft	355.93 Kips	102.65 Kips	458.59 Kips
84.01 ft	467.96 Kips	102.65 Kips	570.61 Kips
93.01 ft	593.26 Kips	102.65 Kips	695.92 Kips
102.01 ft	731.85 Kips	102.65 Kips	834.50 Kips
106.99 ft	814.24 Kips	102.65 Kips	916.89 Kips

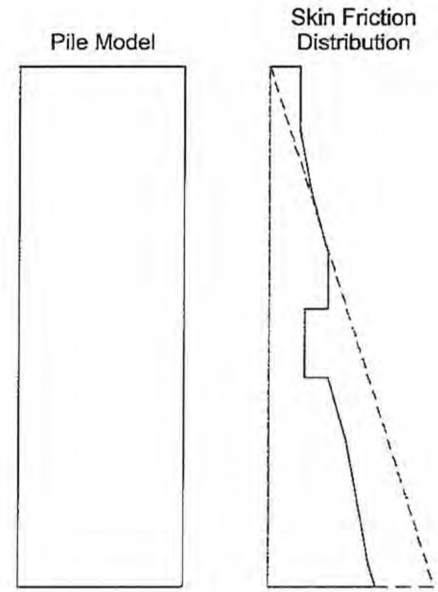
HC Nutting Company
BSB : 12/09/2010 : DWW

15-Dec-2010
GRLWEAP (TM) Version 1998-1



DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	10998 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	75.00 ft
Pile Top Area	24.35 in2



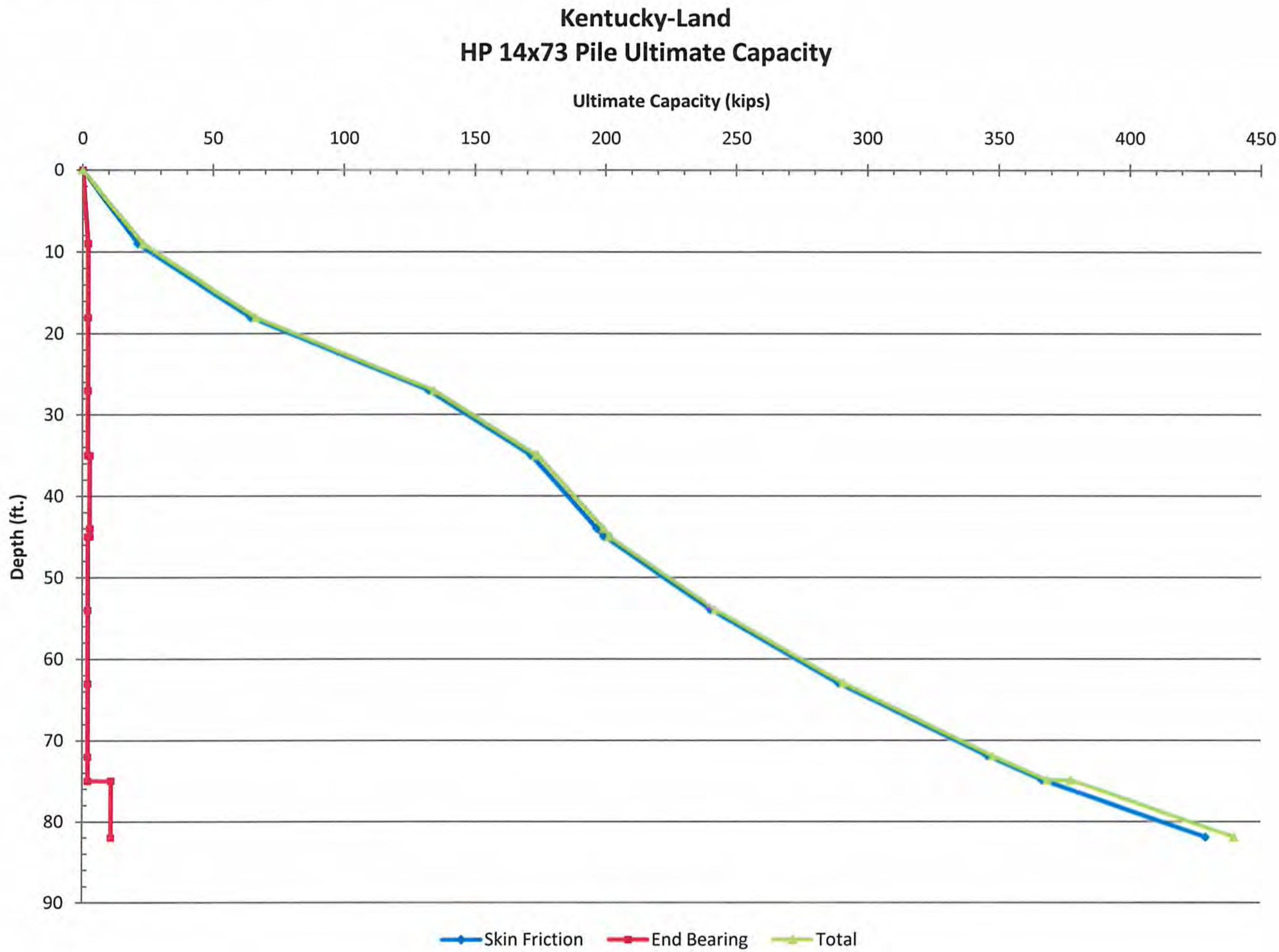
Res. Shaft = 95 %
(Proportional)

——— 1st Pile
- - - - 2nd Pile

HC Nutting Company
BSB : 12/09/2010 : DWW

15-Dec-2010
GRLWEAP(TM) Version 1998-1

Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
100.0	19.231	1.880	5.1	5.69	33.88
150.0	21.000	1.265	8.6	6.15	30.06
200.0	22.845	1.102	12.3	6.81	29.25
250.0	23.751	0.632	17.9	7.14	27.80
300.0	24.887	0.000	22.2	7.59	27.20
350.0	25.401	0.000	27.4	7.82	26.29
400.0	25.708	0.000	34.3	7.93	25.98
450.0	26.531	0.000	42.2	8.26	26.76
500.0	26.947	0.000	54.0	8.43	26.96
550.0	27.261	0.000	71.9	8.51	26.93


Kentucky-Land
HP 14x73 Pile Ultimate Capacity
Ultimate Capacity (kips)
0
50
100
150
200
250
300
350
400
450
Depth (ft.)
0
10
20
30
40
50
60
70
80
90
Skin Friction
End Bearing
Total

DRIVEN 1.2

GENERAL PROJECT INFORMATION

Filename: N:\PROJECTS\2010\N1105070\DRIVEN\HPILES\L514X73.DVN
Project Name: BSB Project Date: 12/09/2010
Project Client: PB
Computed By: DWW
Project Manager: AJM

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 0.00 ft
Perimeter Analysis: Box
Tip Analysis: Pile Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	0.00 ft
	- Driving/Restrike	0.00 ft
	- Ultimate:	0.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	35.00 ft	0.00%	120.00 pcf	1500.00 psf	T-80 Clay
2	Cohesive	10.00 ft	0.00%	120.00 pcf	2000.00 psf	T-80 Clay
3	Cohesionless	30.00 ft	0.00%	125.00 pcf	30.0/30.0	Nordlund
4	Cohesionless	32.00 ft	0.00%	125.00 pcf	34.0/34.0	Nordlund

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	500.59 psf	0.02 Kips
9.01 ft	Cohesive	N/A	N/A	500.59 psf	21.19 Kips
18.01 ft	Cohesive	N/A	N/A	761.13 psf	64.42 Kips
27.01 ft	Cohesive	N/A	N/A	1041.36 psf	132.17 Kips
34.99 ft	Cohesive	N/A	N/A	1041.36 psf	171.22 Kips
35.01 ft	Cohesive	N/A	N/A	595.08 psf	171.30 Kips
44.01 ft	Cohesive	N/A	N/A	595.08 psf	196.47 Kips
44.99 ft	Cohesive	N/A	N/A	595.08 psf	199.21 Kips
45.01 ft	Cohesionless	2592.31 psf	23.58	N/A	199.28 Kips
54.01 ft	Cohesionless	2874.01 psf	23.58	N/A	240.12 Kips
63.01 ft	Cohesionless	3155.71 psf	23.58	N/A	288.96 Kips
72.01 ft	Cohesionless	3437.41 psf	23.58	N/A	345.80 Kips
74.99 ft	Cohesionless	3530.69 psf	23.58	N/A	366.39 Kips
75.01 ft	Cohesionless	4470.31 psf	26.72	N/A	366.56 Kips
84.01 ft	Cohesionless	4752.01 psf	26.72	N/A	463.78 Kips
93.01 ft	Cohesionless	5033.71 psf	26.72	N/A	572.52 Kips
102.01 ft	Cohesionless	5315.41 psf	26.72	N/A	692.79 Kips
106.99 ft	Cohesionless	5471.29 psf	26.72	N/A	764.29 Kips

ULTIMATE - END BEARING

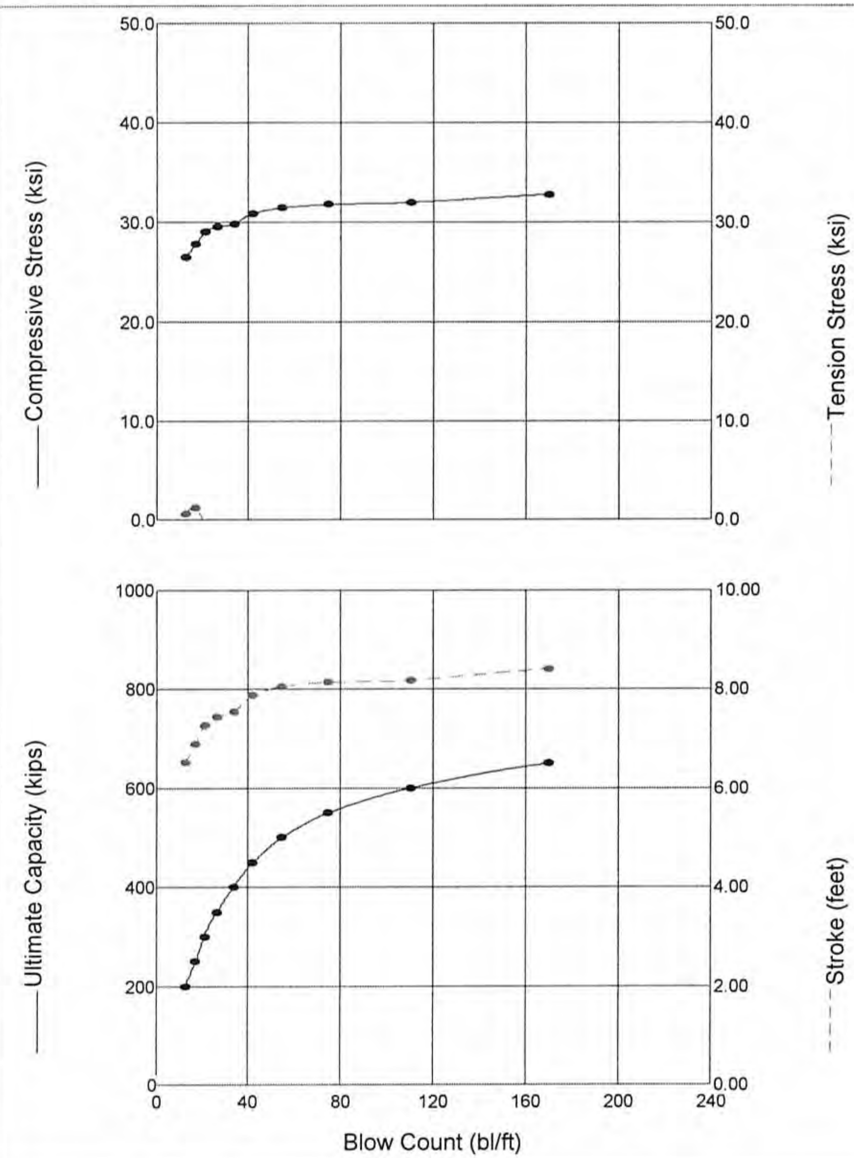
Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	2.01 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	2.01 Kips
18.01 ft	Cohesive	N/A	N/A	N/A	2.01 Kips
27.01 ft	Cohesive	N/A	N/A	N/A	2.01 Kips
34.99 ft	Cohesive	N/A	N/A	N/A	2.01 Kips
35.01 ft	Cohesive	N/A	N/A	N/A	2.67 Kips
44.01 ft	Cohesive	N/A	N/A	N/A	2.67 Kips
44.99 ft	Cohesive	N/A	N/A	N/A	2.67 Kips
45.01 ft	Cohesionless	2592.63 psf	30.00	1.98 Kips	1.98 Kips
54.01 ft	Cohesionless	3156.03 psf	30.00	1.98 Kips	1.98 Kips
63.01 ft	Cohesionless	3719.43 psf	30.00	1.98 Kips	1.98 Kips
72.01 ft	Cohesionless	4282.83 psf	30.00	1.98 Kips	1.98 Kips
74.99 ft	Cohesionless	4469.37 psf	30.00	1.98 Kips	1.98 Kips
75.01 ft	Cohesionless	4470.63 psf	55.60	10.93 Kips	10.93 Kips
84.01 ft	Cohesionless	5034.03 psf	55.60	10.93 Kips	10.93 Kips
93.01 ft	Cohesionless	5597.43 psf	55.60	10.93 Kips	10.93 Kips
102.01 ft	Cohesionless	6160.83 psf	55.60	10.93 Kips	10.93 Kips
106.99 ft	Cohesionless	6472.57 psf	55.60	10.93 Kips	10.93 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.02 Kips	2.01 Kips	2.03 Kips
9.01 ft	21.19 Kips	2.01 Kips	23.20 Kips
18.01 ft	64.42 Kips	2.01 Kips	66.42 Kips
27.01 ft	132.17 Kips	2.01 Kips	134.18 Kips
34.99 ft	171.22 Kips	2.01 Kips	173.23 Kips
35.01 ft	171.30 Kips	2.67 Kips	173.98 Kips
44.01 ft	196.47 Kips	2.67 Kips	199.14 Kips
44.99 ft	199.21 Kips	2.67 Kips	201.88 Kips
45.01 ft	199.28 Kips	1.98 Kips	201.26 Kips
54.01 ft	240.12 Kips	1.98 Kips	242.09 Kips
63.01 ft	288.96 Kips	1.98 Kips	290.94 Kips
72.01 ft	345.80 Kips	1.98 Kips	347.78 Kips
74.99 ft	366.39 Kips	1.98 Kips	368.37 Kips
75.01 ft	366.56 Kips	10.93 Kips	377.49 Kips
84.01 ft	463.78 Kips	10.93 Kips	474.70 Kips
93.01 ft	572.52 Kips	10.93 Kips	583.45 Kips
102.01 ft	692.79 Kips	10.93 Kips	703.71 Kips
106.99 ft	764.29 Kips	10.93 Kips	775.22 Kips

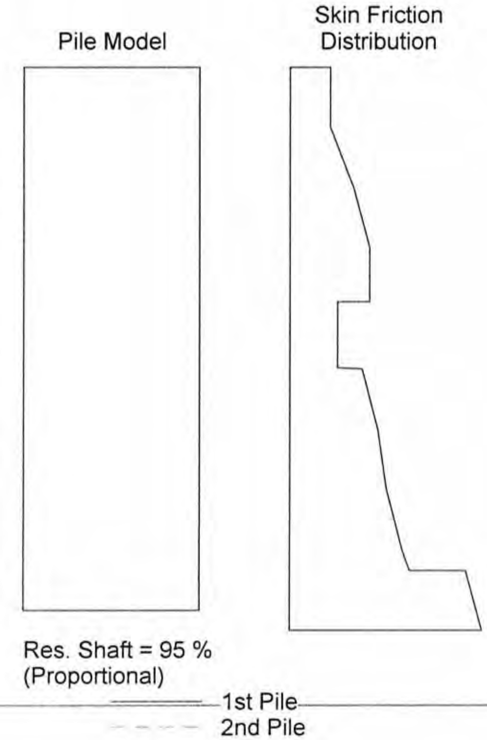
HC Nutting Company
BSB : 12/09/2010 : DWW

28-Feb-2011
GRLWEAP (TM) Version 1998-1



DELMAG D 30-32

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	81.00 ft
Pile Top Area	21.40 in2

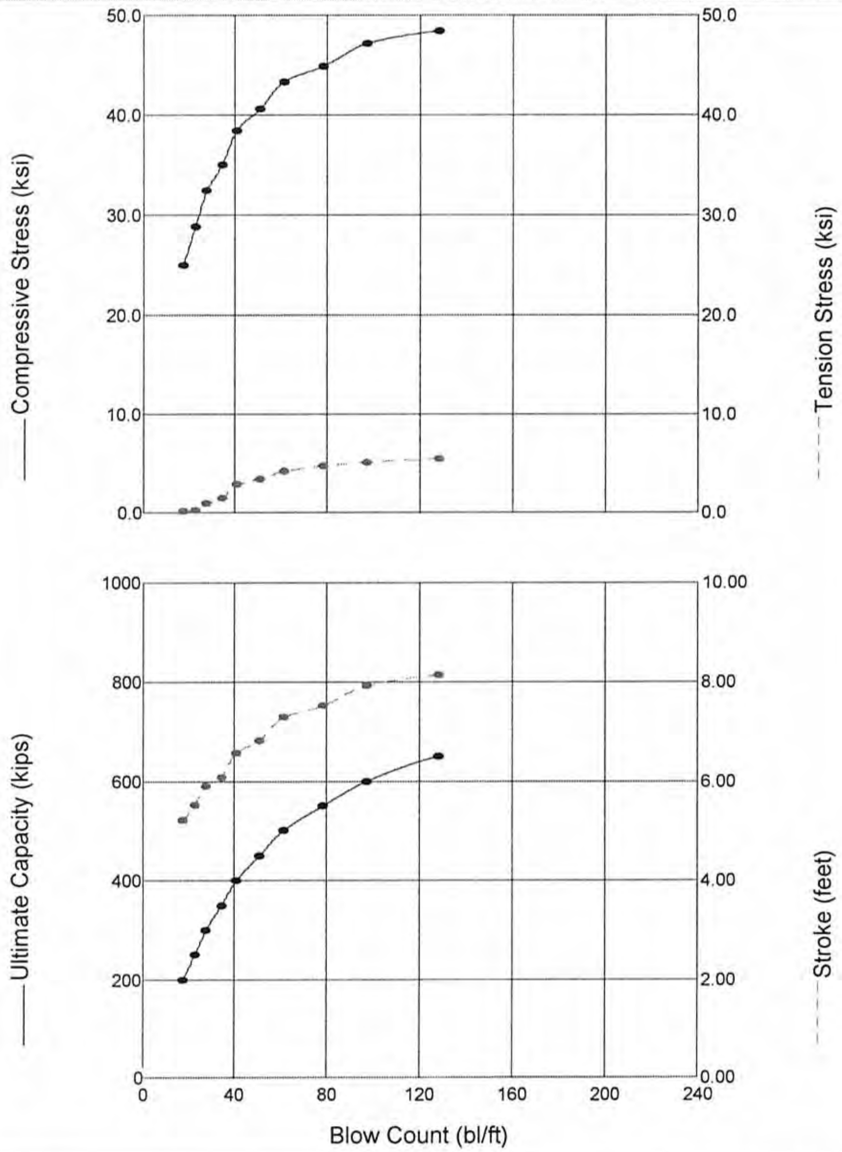


Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
200.0	26.492	0.629	12.7	6.52	31.00
250.0	27.794	1.298	17.3	6.89	30.19
300.0	28.999	0.000	21.2	7.26	29.64
350.0	29.559	0.000	26.6	7.44	29.34
400.0	29.865	0.000	33.9	7.54	29.43
450.0	30.931	0.000	42.1	7.88	30.44
500.0	31.484	0.000	54.6	8.05	30.74
550.0	31.825	0.000	74.9	8.14	30.74
600.0	32.044	0.000	110.6	8.18	30.73
650.0	32.834	0.000	170.3	8.41	31.47

HC Nutting Company
BSB : 12/09/2010 : DWW

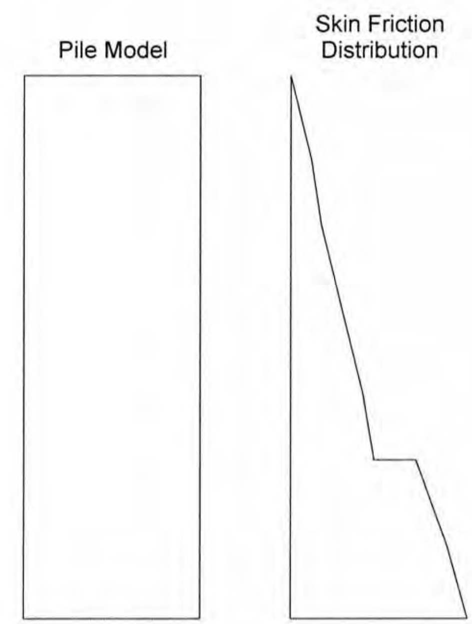
Ohio River - HP 14x73 to rock

28-Feb-2011
GRLWEAP (TM) Version 1998-1



DELMAG D 30

Efficiency	0.800
Helmet	2.70 kips
Hammer Cushion	109975 kips/in
Skin Quake	0.100 in
Toe Quake	0.100 in
Skin Damping	0.100 sec/ft
Toe Damping	0.150 sec/ft
Pile Length	57.98 ft
Pile Top Area	21.40 in2



Res. Shaft = 5 %
(Proportional)

1st Pile
2nd Pile

Ohio River
HP14x73 to rock

HC Nutting Company
BSB : 12/09/2010 : DWW
Ohio River HP14x73 to rock

28-Feb-2011
GRLWEAP(TM) Version 1998-1

Ulitmate Capacity kips	Maximum Compression Stress ksi	Maximum Tension Stress ksi	Blow Count bl/ft	Stroke feet	Energy kips-ft
200.0	25.032	0.215	17.7	5.22	24.51
250.0	28.832	0.344	22.9	5.52	24.55
300.0	32.532	1.001	27.8	5.90	25.71
350.0	35.052	1.543	34.6	6.09	26.05
400.0	38.483	2.929	40.9	6.58	27.81
450.0	40.642	3.482	50.9	6.83	28.55
500.0	43.390	4.238	61.3	7.29	30.25
550.0	44.975	4.779	78.5	7.52	30.93
600.0	47.248	5.149	97.4	7.93	32.37
650.0	48.492	5.524	128.2	8.14	33.25

$R_{R\,max}$ = 530 kips

$Comp_{Max}$ = 44.3 ksi

Ten_{Max} = 4.56 ksi

Blow Count = 71.6 bl/ft

Appendix 6D
Preliminary
Construction
Cost Estimates

2017 COSTS
Cost Estimate Date: 10/18/2010
Construction Start Date: 1/1/2016
Mid-Point Date (Alt 1): 6/1/2017
Inflation Rate (Alt 1): 37.6 %
Mid-Point Date (Alts 3&6): 1/1/2018
Inflation Rate (Alts 3&6): 41.0 %
Design Contigency: 20.0 %

Table 1. Main Bridge/Approach Spans Cost Breakdown

	Main Bridge				Approaches				
Alternative	Quanitity	Units	Unit Cost	Total Cost	Quanitity	Units	Unit Cost	Total Cost	**TOTAL**
1: Simply supported tied-arch	144,000	SF	$2,488	$358,276,000	172,800	SF	$1,229	$212,400,000	**$570,676,000**
3: 2 Vertical towers, 3 legs/tower	288,000	SF	$2,195	$632,295,000	28,800	SF	$1,260	$36,275,000	**$668,570,000**
6: 1 Vertical towers, 2 legs/tower	243,996	SF	$2,299	$561,015,000	72,804	SF	$1,172	$85,294,000	**$646,309,000**

Table 2. Main Bridge/Approach Spans Cost Breakdown by State

	Main Bridge					Approaches					TOTAL COST W/ INFLATION				
	Total Cost	KY Cost		OH Cost		**Total Cost**	KY Cost		OH Cost		**Total Cost**	KY Cost		OH Cost	
Alternative	**($M)**	($M)	%	($M)	%	**($M)**	($M)	%	($M)	%	**($M)**	($M)	%	($M)	%
1: Simply supported tied-arch	**$358.3**	$358.3	100.0	$0.0	0.0	**$212.4**	$126.3	59.5	$86.1	40.5	**$570.7**	$484.6	84.9	$86.1	15.1
3: 2 Vertical towers, 3 legs/tower	**$632.3**	$532.8	84.3	$99.5	15.7	**$36.3**	$5.2	14.2	$31.1	85.7	**$668.6**	$538.0	80.5	$130.6	19.5
6: 1 Vertical towers, 2 legs/tower	**$561.0**	$393.3	70.1	$167.8	29.9	**$85.3**	$85.3	100.0	$0.0	0.0	**$646.3**	$478.6	74.0	$167.8	26.0

Appendix 6E
Wind Analysis Reports

Tel: 519.823.1311
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Rowan Williams Davies & Irwin Inc.
650 Woodlawn Road West
Guelph, Ontario, Canada N1K 1B8

CONSULTING ENGINEERS
& SCIENTISTS

December 7, 2010

Ruchu Hsu
Parsons Brinckerhoff (PB)
One Penn Plaza, 250 W. 34th Street
New York, NY 10119
USA

Re: Brent Spence Bridge – stability assessment and design review
RWDI Reference Number: 0940582

Dear Ruchu,

We have assessed the likely aerodynamic performance of the 3 proposed alternates of the Brent Spence Bridge, which spans the Ohio River between Covington, Kentucky and Cincinnati, Ohio. This letter expresses opinions regarding the three alternates, based on our experience with wind tunnel testing and analysis of similar bridge designs.

Information on the proposed bridge layouts, with preliminary dynamic structural properties for each, was provided to RWDI on October 8, 2010. Mass properties were provided in subsequent correspondence on November 17, 2010.

Bridge Descriptions

RWDI were asked to review the aerodynamic performance of the following three alternates, all of which are double-decked with a main span of at least 1000ft:

i. Alternate 1: tied arch
ii. Alternate 3: Two tower cable-stayed
iii. Alternate 6: Single tower cable-stayed

Elevation and sectional views of each bridge are provided in Figures 1 through 3. Mass information used in the assessment is provided in Table 1. Frequencies of vibration for each alternate are provided in Tables 2a through 2c for at least the first 10 modes of vibration. Vertical and torsional modes involving significant deck motions are identified in each table.

Stability Considerations for the Completed Bridge

For the stability assessment of the deck and the towers, there are three types of wind-induced oscillations that need to be considered:

i. Flutter. A self-excited aerodynamic instability that can grow to very large amplitudes in torsion only or coupled torsion and vertical motion, that is to be avoided at all costs.
ii. Galloping. An instability involving across-wind motions similar to flutter that can theoretically grow to unlimited amplitude and is thus to be avoided.
iii. Vortex-induced oscillations. Limited amplitude vibrations caused by alternate and regular vortices shed from both sides of a bluff body, such as the decks. It occurs over limited wind speed ranges. This vibration can be tolerated if the amplitudes are not excessive.

Flutter

Flutter is an instability caused by the deflection of a structure, modifying the aerodynamics in such as way as to alter (increase) the wind loads. Typically, flutter occurs above a threshold wind speed. It is important to ensure that, should a bridge deck cross section exhibit a tendency towards flutter or divergence, the threshold wind speed be well beyond the wind speeds being considered for the ultimate strength design of the bridge.

At this stage in the design process, preliminary screening tools were applied to assess the aerodynamic stability of the bridge deck alternates. In 1961, Selberg[1] introduced simple empirical formulae for the estimation of the onset velocity of flutter. Using Selberg's formulations and the mass, modal and geometric properties of each of the decks, the critical wind speed for the onset of flutter has been estimated for each alternate. Recall from RWDI Wind Climate Analysis Report No. 0940582 that the recommended wind speed at deck height for the 10,000 year return period was equal to a 10-minute mean speed of 86.3 mph.

The flutter speeds estimated using the method of Selberg are well in excess of 86.3 mph for each of the three alternates reviewed by RWDI.

An alternate approach was used to confirm Selberg's method. Using aerodynamic derivatives measured on the Tacoma Narrows bridge deck section (which failed due to torsional flutter response caused by low torsional stiffness and a vortex shedding wind speed near the flutter velocity), torsional flutter velocities were estimated using the mass, modal and geometric properties of the Brent Spence Bridge alternates. This approach, which should yield conservative results, also suggested that the critical wind speeds for the onset of torsional flutter are well beyond the 10-minute mean speed of 86.3 mph for each of the three alternates.

Galloping

Galloping is a self-induced vibration of a flexible structure in an across-wind bending mode. Galloping has been frequently seen in iced transmission line cables, however many non-circular cross sections are prone to gallop. Galloping starts at an onset wind velocity, and normally increases rapidly with increasing wind velocity. The onset wind velocity may be approximately estimated using the Eurocode EN 1991-1-4 standard, as follows:

$$v_{CG} = 2\ Sc \div a_G\ x\ n_{1y}\ x\ b$$

where Sc is the Scruton number, a_G is a factor of galloping instability, n_{1y} is the first vertical mode frequency of vibration, and b is the across-wind deck dimension. The Scruton number is defined in the Eurocode EN 1991-1-4 as

$$Sc = 2\ x\ \delta_s\ x\ m_{i,e} \div \rho_{air} \div b^2$$

where δ_s is the logarithmic decrement structural damping, $m_{i,e}$ is the equivalen mass per unit length of deck in mode i, ρ_{air} is the air density (taken as 1.225 kg/m^3) and b is defined as above.

In the absence of measured data for a_G a value of 10 may be used, and is considered conservative. Assuming a structural damping ratio of δ_s =0.063 (1% of critical), and substituting in the mass, modal and geometric properties of each bridge alternate indicates the following:

[1] Selberg, A., Oscillations and Aerodynamic Stability of Suspension Bridges, Acta Pol. Scandina., Ci 13, 1961

i. Alternate 1 – Tied Arch: v_{CG} >> 86.3 mph
ii. Alternate 3 – Two Tower Cable-stayed: v_{CG} >> 86.3 mph
iii. Alternate 6 – Single Tower Cable-stayed: v_{CG} ~ 80 mph

Although admittedly conservative, the Eurocode approach suggests that Alternate 6, the single tower cable-stayed double-deck bridge, may be susceptible to galloping excitations at a wind speed near the once-in-10,000 year recurrence. This finding suggests that further detailed investigation of the tendency towards galloping of Alternate 6 is warranted, should this be a preferred alternate.

Vortex-Induced Oscillations

The phenomenon of vortex shedding occurs frequently on bluff engineering structures. Based on RWDI's experience, and research publications available in the literature, it is our view that vortex-shedding vibrations in both the vertical and torsional directions may occur for each of the alternates reviewed. However, the magnitude of the vibrations is unlikely to be severe and we are confident that appropriate aerodynamic modifications to the deck cross-sections will mitigate the vibrations.

Early model tests on open-truss suspended bridge decks undertaken for the Firth of Forth bridge indicated excellent performance with regards to vertical vibrations, i.e. fairly benign response. Depending on the aspect ratio of the truss depth and deck width, open truss bridge decks can also exhibit good torsional behaviour. However, it is known that both the torsional and vertical response of truss stiffened suspended decks is sensitive to the number and size of openings between running surfaces on the deck, and studies undertaken for the Tsing-Ma bridge indicated the placement of the openings in the deck surface was critical for mitigating vibrations. Note that this particular bridge incorporated edge fairings into its design to further enhance the wind-induced behaviour.

The magnitudes of vortex-induced vibrations are difficult to estimate precisely at this stage without wind tunnel testing. It would be prudent at this stage to consider countermeasures to mitigate vortex-induced vibrations, in the event that subsequent wind tunnel tests indicate they are necessary. These countermeasures could take the form of:

i. Edge fairings.
ii. Vents in the top and bottom deck surfaces.
iii. Open traffic barriers.
iv. Aerodynamic Damper Plates
v. Turbulence Generators

RWDI can provide sketches of the proposed solutions prior to any wind tunnel tests, to enable the design team to evaluate and rank order the solutions, to facilitate possible trials during the model studies.

Construction Stage Considerations

There are unique construction stage considerations for each alternate reviewed. A brief summary of our conclusions follows.

Tied-Arch Alternate

During construction of a tied-arch bridge type, depending on the selected erection scheme, the following may deserve attention with regards to aerodynamic instability:

i. the free standing arch structures
ii. the suspended double-deck cantilever before closure at mid-span (depends on erection scheme)

The free standing arches may be subject to galloping and/or vortex shedding instability, particularly before they are linked to adjacent arches. While fabrication of the arches may be undertaken off-site and the erection window can be narrow – thereby reducing the risk of an aerodynamic instability – the risk of instability remains. The following stabilizing schemes have been applied in practice:

a) install temporary tie-downs for the arches
b) install temporary link-beams to connect the arches and providing additional stiffness

During erection, should the deck be suspended from the hangers (beginning at the main-span piers and joined at mid-span), the “free” decks may have a low flutter onset speed due to the reduced stiffness and low frequency of vibration. There are erection sequences that are known to have improved performance, which RWDI and the design team will be familiar with. If these scheme are not suitable then similar measures as recommended for the free-standing arch may be applied to eleviate this problem.

<u>Cable-stayed Alternates</u>

During construction of the cable-stayed bridge alternates, there are typically two primary concerns:

i. the free standing tower; and
ii. the suspended double- deck cantilever before closure at mid-span (and/or closure at the main and back spans)

The free standing tower legs may be subject to galloping and/or vortex shedding instabilities themselves. Though some early estimates of instability may be carried out numerically, the best tool for assessing stability and verifying the wind loads and deflections during construction stages is an aero-elastic model test. If any type of instability turns out to be a problem for the towers, the following stabilizing schemes may be applied (and have been used successfully on other bridge developments):

a) install temporary tie-downs to the critical tower elevation
b) install temporary cross-beams to connect tower legs (which will require both legs to be build at the same time)
c) install temporary dampers

Considering the cantilevered double- deck during construction, the principal problem typically is not stability but load demands at the base of the towers and at the deck to tower connections. Although lower wind speeds are normally considered for design during construction, there may be a critical cantilever length where the peak loads during construction could become higher compared to the completed bridge. To reduce wind loads during construction temporary frame supports or temporary ties and/or guides are normally used by the contractors.

Serviceability Considerations

Each of the three bridge alternates have serviceability considerations which are affected by wind loading and aerodynamics. Common to all bridges are issues involving the wind-induced vibration of the stay and hanger cables.

Cables may vibrate due to:

i. Vortex shedding
ii. Rain/wind induced vibrations
iii. Wake galloping of groups of cables

iv. Galloping of cables with ice accumulations
v. Galloping of isolated cables inclined to the wind
vi. Excitations induced from the stay anchors
vii. Motions due to wind buffeting on cables

Vibrations of cables occur due to their low mass and low damping. The expected damping ratio of a stay cable or hanger cable would typically be less than 0.1%, without the use of supplementary damping or energy absorbing bushes. The excitation mechanisms noted above are considered instabilities.

It is well documented that cable-stayed bridges have experienced galloping of dry inclined cables and/or rain/wind-induced vibrations, which have led to peak vibration amplitudes as high as 5 times the diameter of the very longest stay cables. This is significant since these deflections are visible to users of the bridge, and are sufficient to cause alarm - not to mention potential damage due to fatigue of connections. Vibrations of this sort should therefore be suppressed. An effective method for controlling rain/wind induced vibrations would be through the use of helical fillets which spiral along the length of the cable. The pitch of a typical helical fillet is about 2 to 3 times the diameter of the cable. However, in colder climates these may lead to excessive ice accretion which in turn may cause galloping in its classical form. The installation of secondary cross-cables, often referred to as cross-ties or aiguilles, has also been used to suppress rain/wind vibrations. Examples of where this approach has been adopted are the William H. Harsha Bridge in Maysville, Kentucky, and the Second Severn Bridge crossing between Wales and England, to name two.

Vortex shedding is typically not a problem of stay cables, in that the critical wind speeds causing vortex shedding are low, and the magnitudes of vortex-induced vibrations are minimal. However, vortex shedding is common problem on hangers. Countermeasures such as Stockbridge dampers have been applied in such cases.

Vibration induced through the stay anchors, or parametric excitation as it is sometimes referred, occurs when the cables have similar frequencies of vibration to the decks, towers, and/or arches. Any dynamic load such as wind, vehicular or pedestrian traffic could be the origin of the vibration. Small motions of the deck, towers, or arches could result in significant cable vibrations. The most common method for suppressing motion-induced vibrations is through the use of cross-ties, which effectively detune cables' frequencies off the modal frequencies associated with the anchorage motions.

It should be noted that cross-ties are only effective for suppressing motions in the cable plane that are due to vertical deck and along the bridge tower motions. Out-of- plane cable motions are more difficult to control, and can be excited by motions of the towers or arches normal to the plane of the cables where the structure modes of vibration are close to the cable frequencies of vibration. In cases where the modal properties of the bridge tower or arch structures are not sufficiently separated from the cable frequencies, an alternative measure for vibration control could be external supplementary damping.

Conclusions

RWDI have reviewed the three bridge alternates proposed for the Brent Spence Bridge between Covington, Kentucky and Cincinnati, Ohio, and identified any potential aerodynamic instabilities which may affect the strength and safety of the bridge, and any aerodynamic issues that may affect the serviceability.

Regarding aerodynamic instability, it appears that Alternate 1 (Tied Arch) and Alternate 3 (Two-Tower Cable-stayed) will have excellent aerodynamic performance. RWDI estimates that the onset speeds for flutter and galloping are well beyond the recommended wind speed at deck height for the 10,000 year return period (equal to a 10-minute mean speed of 86.3 mph). Preliminary review suggests that Alternate 6 (Single Tower Cable-stayed) may have a galloping onset velocity which almost equal to the 10,000 year return period wind. Although RWDI's estimates are conservative, this is worth noting at this early stage.

With regards to vortex-induced vibrations of the bridge decks, the performance of the bridge decks for each alternate may be enhanced through the use of open vents in the deck surfaces, aerodynamic fairings, or open traffic barriers. Wind tunnel testing is critical to determine which of the above is most impactful.

Regarding the serviceability of these bridge decks, RWDI have identified a number of sources of wind induced cable and hanger vibrations, and suggested possible mitigations. As the designs progress and additional dynamic structural properties and information become available, we suggest that a more detailed review of the issues involving cable vibration be reviewed.

Please do not hesitate to contact us if you have any questions or comments.

Yours very truly,

ROWAN WILLIAMS DAVIES & IRWIN Inc.

John Kilpatrick, PhD, PEng
Technical Director (UK), Senior Associate

Stoyan Stoyanoff, Ph.D., P.Eng., ing.
Project Director/Principal

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Table 1. Preliminary Mass Information

Alternate	Mass/Unit Length
1 – Tied Arch	10.5 kip/ft/rib (Arches)
	42.5 kip/ft/deck (Deck)
2 – Two Tower Cable-stayed	33 kip/ft/deck (Main span and Back span)
3 – Single Tower Cable-stayed	14 kip/ft/deck (Main span)
	27 kip/ft/deck (Back span)

Table 2a. Modal Frequencies of Vibration – Tied Arch

Mode	Frequency (Hz)
1	0.379293
2	0.690965
3[a]	0.692145
4	0.822813
5[a]	0.894064
6[a]	1.000086
7	1.200667
8[b]	1.320892
9[b]	1.427788
10	1.453115

a: vertical deck mode
b: torsional deck mode

Table 2b. Modal Frequencies of Vibration – Two Tower Cable-stayed

Mode	Frequency (Hz)
1	0.309716
2[a]	0.319934
3	0.375362
4	0.431133
5	0.431138
6[a]	0.433314
7	0.437484
8	0.438539
9	0.438556
10[b]	0.439909
11	0.440171
12	0.440171
13	0.44024
14	0.44024
15	0.466702
16[b]	0.737653

a: vertical deck mode
b: torsional deck mode

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Table 2c. Modal Frequencies of Vibration – Single Tower Cable-stayed

Mode	Frequency (Hz)
1	0.297868
2	0.360090
3[a]	0.564058
4[a]	0.724985
5	0.901072
6	1.159462
7[b]	1.259159
8[a]	1.306169
9	1.393606
10	1.424260

a: vertical deck mode
b: torsional deck mode

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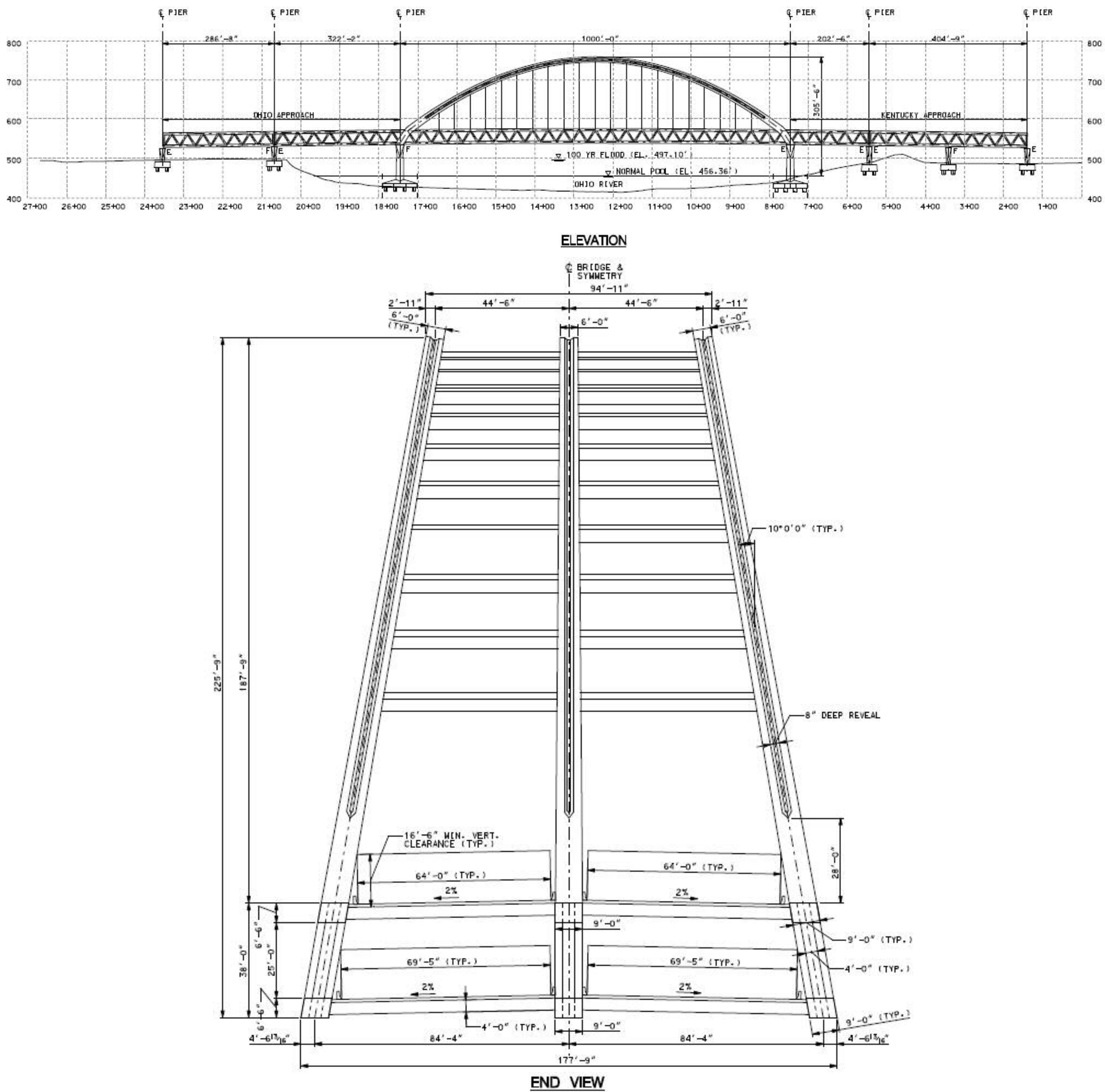

Figure 1. Alternate 1 - Tied Arch Bridge

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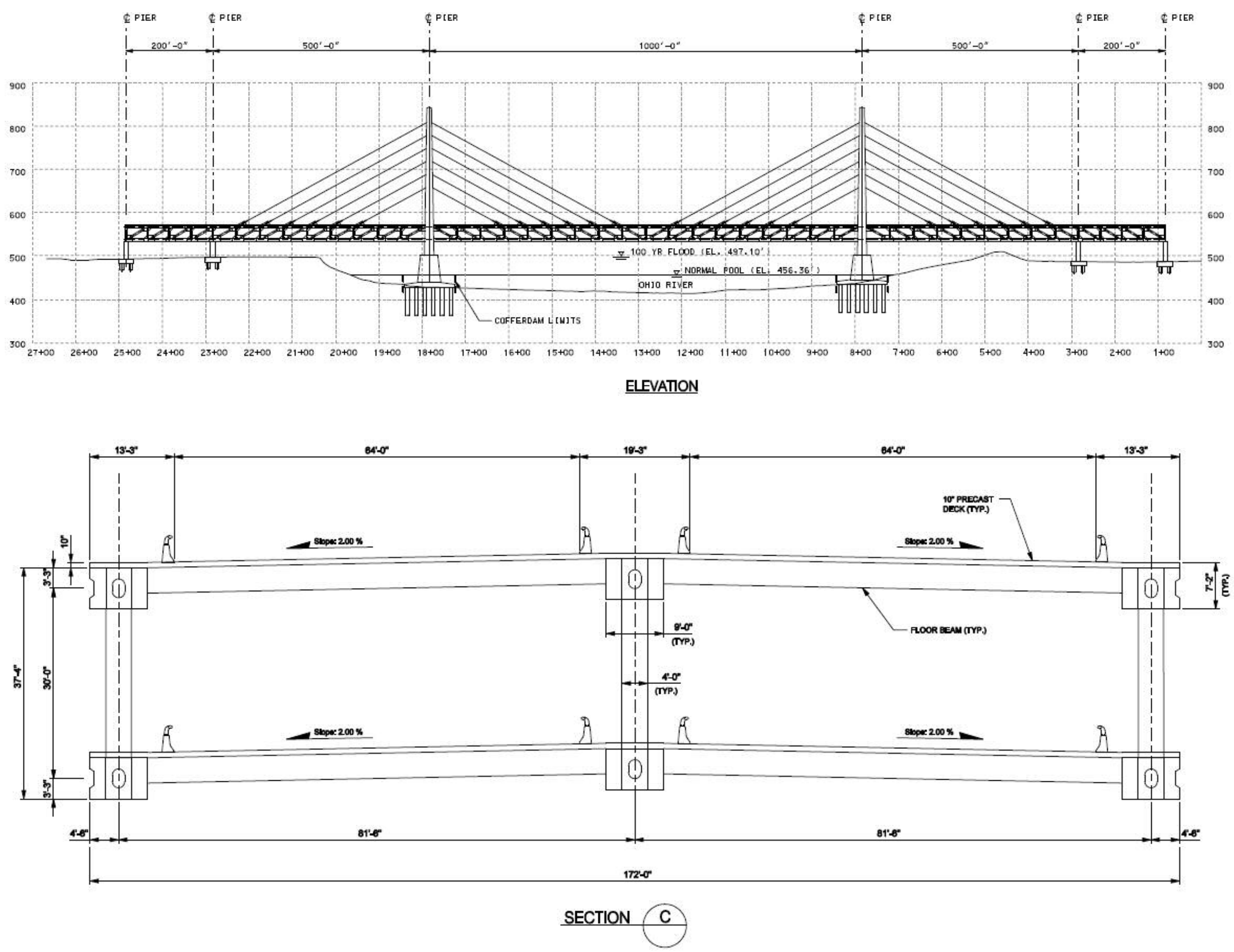

Figure 2. Alternate 3 – Two Tower Cable-stayed Bridge

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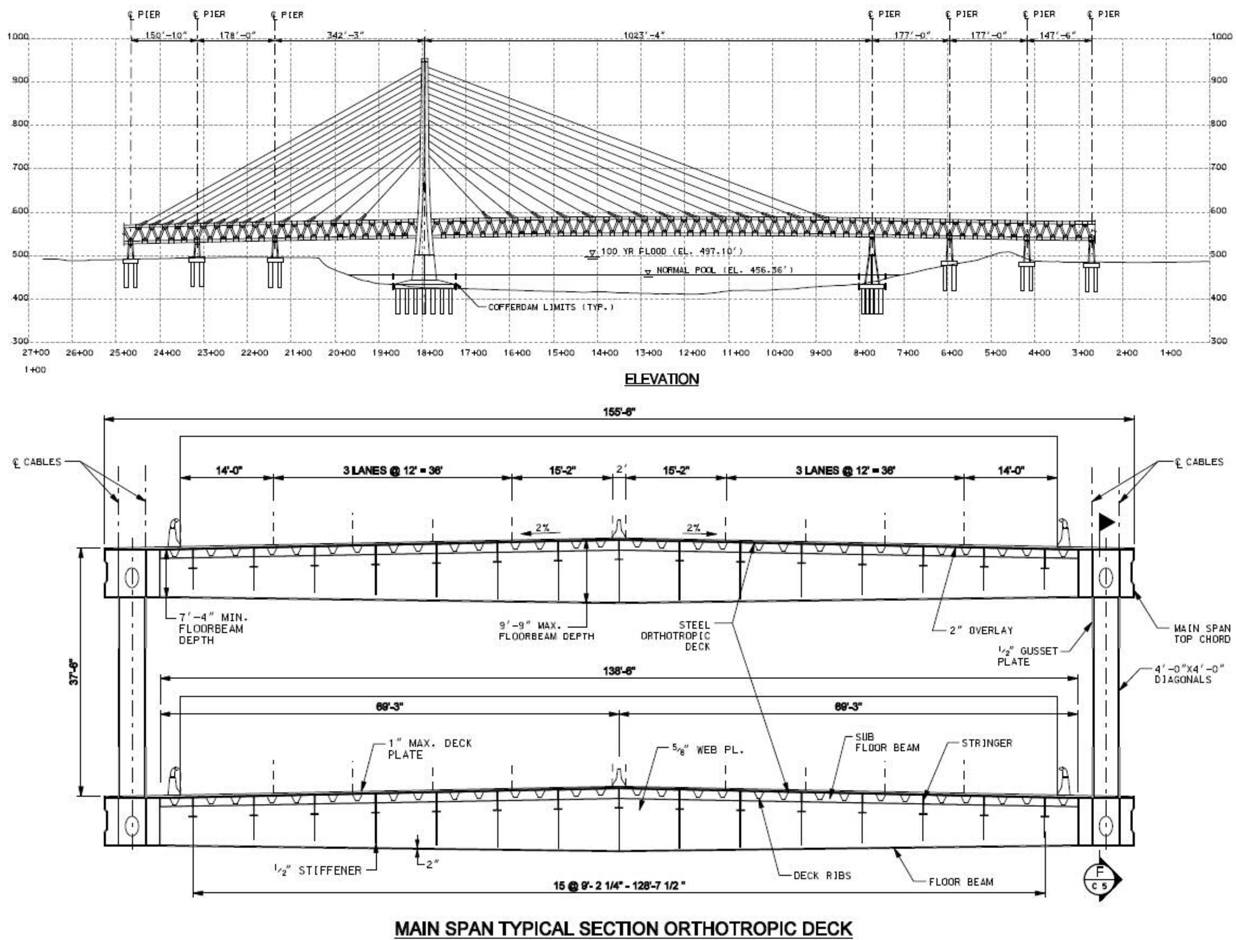

Figure 3. Alternate 6 – Single Tower Cable-stayed Bridge

FINAL REPORT

WIND CLIMATE ANALYSIS
BRENT SPENCE BRIDGE
CINCINNATI, OH

CONSULTING ENGINEERS & SCIENTISTS

Project Number: #0940582 **September 28, 2010**

SUBMITTED TO: **Duane F. Phelps, PE**
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TABLE OF CONTENTS

TABLES

FIGURES

1. INTRODUCTION

RWDI was retained by Parsons Brinckerhoff (PB) to conduct wind engineering studies for the proposed new renovation of the Brent Spence Bridge (BSB), which is located in Cincinnati, Ohio. The new bridge is on the west side of the existing BSB. Three bridge options are currently being developed by the designers Parsons Brinckerhoff. The following reports RWDI's wind engineering studies performed for the new bridge.

These include:

- **Local Wind Climatology Analysis:** The objective of this analysis was to determine the design wind speeds for wind loading.

2. WIND CLIMATE ANALYSIS

2.1 INTRODUCTION

This section of the report presents the analysis of the wind climate and wind turbulence properties undertaken for the bridge site in Cincinnati, Ohio. The results presented in this section will be used in the subsequent analyses to determine the aerodynamic stability of the bridge. Figure 2-1 provides a site plan showing the location of the bridge and local meteorological stations used in this analysis. Photographs of the site taken during our visit on July 7, 2010 are shown in Figures 2-2 and 2-3.

2.2 WIND CLIMATE AND SITE ANALYSIS

2.2.1 Source of Data

The wind statistics used to determine the design wind speeds and directionality at the bridge site were based primarily on the surface wind measurements taken between 1948 and 2008 at the Cincinnati-Northern-Kentucky International Airport, located about 8 miles west-southwest from the bridge site. Wind data for 1973 to 2009 from the Cincinnati Municipal Airport, located 5.6 miles east of the site were also used to provide additional insight into winds in the area. However, since this airport is located in a valley and sheltered for almost all wind directions, it was considered more prudent not to use this data for the final interpretation and the design wind speeds.

2.2.2 Local Terrain

The terrain surrounding the airport anemometer and the bridge site were reviewed based on satellite images, topological maps and site photographs. Adjustments were made, where necessary, for the terrain roughness upwind of the anemometer and for its height above the ground. On July 7, 2010, a RWDI engineer went to the bridge site to take photographs and to confirm the terrain information used in the analysis (see Figures 2-2 and 2-3).

2.2.3 Analysis

The design wind speeds and directionality for the bridge site were determined using the following steps:

i. Extreme value analyses using a Fisher-Tippet Type I distribution were conducted based on the wind records collected at the Cincinnati-Northern-Kentucky International Airport.

ii. The joint probability of wind speed and direction for the site was determined based on the available meteorological data. The analyzed wind data were then expressed in the form of a mathematical model for the airport.

iii. The mathematical model developed in (ii) was used to evaluate wind speed as a function of return period and also to evaluate the component of the wind velocity normal to the bridge span as a function of return period. The procedure called "Upcrossing Analysis" was used in this step.

All results contained in this report are discussed as mean-hourly (i.e., 1-hour mean) speeds, which are applicable for structural design, or as 10-minute mean speeds. In this study, 10-minute mean speeds are given since this is the typical time for an aerodynamic instability to develop on a bridge sensitive to wind. According to the wind map of the ASCE 7-05 Standard, a 90 mph basic design wind speed for the Cincinnati area is recommended, this being a 3-sec gust speed in open terrain at 33ft height. To relate the mean-hourly wind speed to the 3-second gust or 10-minute mean, the relationship shown in Figure C6-4 of the ASCE 7-05 was assumed. According to this curve, 1-hour mean wind speeds can be converted to 3-second gust speeds, and to 10-minute mean speeds multiplying by the factors 1.524 and 1.067, respectively. Using the factor 1.524 to convert from a 3-second gust speed to a mean hourly wind speed, the basic design wind speed for Cincinnati becomes 59.2 mph. Adjustments for other terrain conditions were made using ESDU methodology[1].

2.2.4 Extreme Value Analysis to Determine Design Winds

Meteorological data from the Northern-Kentucky International Airport were used to calculate extreme wind speed return periods. The maximum mean-hourly wind speeds occurring each month were extracted for the period of record, and the velocities fitted to a Fisher-Tippet Type I distribution. Various fitting methods were used which included fitting velocities as well as velocity pressures, using both a least-squares fitting method and the method of moments. A comparison of the various fitting methods was used to evaluate the best fit to the data. The resulting distributions were then employed to predict wind speeds for a range of return periods (i.e., from 1 to 10,000 years).

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1 Engineering Sciences Data Unit, Characteristics of the Atmospheric Turbulence Data Near the Ground: Part III, Variations in Space and Time for Strong Winds, ESDU 86010, London ,UK, 1986.

2.2.5 Joint Probability of Wind Speeds and Directions

A mathematical model of the joint probability of wind speed and direction was fitted to the meteorological wind data assuming a Weibull type distribution. This distribution expresses the probability of the wind speed at a given elevation exceeding a value U as

$$P_\theta(U) = A_\theta \exp\left[-\left(\frac{U}{C_\theta}\right)^{K_\theta}\right], \qquad (2\text{-}1)$$

where P_θ is the probability of exceeding the wind speed U in the angle sector θ;

θ is the central angle of an angle sector, measured clockwise from true North; and

A_θ, C_θ, K_θ are coefficients selected to give best fit to the data.

Note that A_θ is the fraction of time the wind blows from within the angle sector θ. The size of angle sectors used in this analysis was 10 degrees. To provide additional flexibility in curve fitting for normal winds, two Weibull curves were fitted, one to lower velocities and one to higher velocities, with blending expressions being used to provide a smooth transition.

The probability distributions given by Equation (2-1) may be used to obtain the overall probability of wind speed by summing over all wind directions.

$$P(U) = P_N(U) = \sum_\theta \left[P_{\theta N}(U)\right], \qquad (2\text{-}2)$$

where the subscript N refers to normal winds.

At the gradient height the wind speeds are well above the earth's surface roughness effects. The height used for determining gradient speed was 2000 ft. Since the anemometer is near ground level at the bottom of the planetary boundary layer, it is affected by ground roughness. These ground roughness effects were assessed using the methods given in ESDU[2] combined with information on the local terrain roughness gathered from topographic maps and other site information. Factors were developed to convert the anemometer records to wind speeds at gradient height and then to the bridge site.

2 ESDU International, Computer program for wind speeds and turbulence properties: flat or hilly sites in terrain with roughness changes, ESDU 01008, 2001.

2.2.6 Upcrossing Method to Determine Directionality Effects on Design Winds

By adapting random noise theory to meteorological data (Rice[3]), it can be shown that the return period, R, in years of a given gradient wind speed, U_G, is related to $P(U_G)$ by

$$R = -\left[\frac{\overline{|\dot{U}_N|}}{2} \frac{dP_N(U_G)}{dU_N} (T_A) \right]^{-1}, \tag{2-3}$$

where $\overline{|\dot{U}_N|}$ is the average of the absolute rate of change of the hourly values of U for normal winds with time; T_A is the total number of hours in a year, i.e., $T_A \approx 8766$.

Equation (2-3), together with an empirical relationship for $\overline{|\dot{U}_N|}$, can be used to determine the return periods for a series of selected wind speeds. The wind speed corresponding to a required return period (e.g., 10, 100, 1000 years etc.) can then be determined by interpolation. This method, which here uses the Weibull distribution for P_N, is called the Upcrossing Method and is one way of obtaining the variation of wind speed with return period. The other way is direct extreme value analysis as in Section 2.2.4. The direct method uses fewer assumptions. Therefore, the Weibull model was scaled to match the direct extreme value results exactly at each return period of interest. This approach allows directionality effects to be systematically accounted for by a model that is also consistent with extreme value analysis.

Since there is evidence[4] that for flutter instability the important component of wind velocity is that normal to the span, it is of interest to evaluate this normal component as a function of its return period. It can be shown[5,6] that if U_B denotes the wind velocity on the boundary of instability (in this case, the flutter velocity as defined for wind normal to the span, divided by the cosine of the actual angle between the wind direction and the normal to the span), then the return period *R* is given by

$$R = -\left[\sum_{\theta} \left(\frac{\overline{|\dot{U}_{NB}|}}{2} \frac{dP_{\theta N}}{dU_{NB}} \sqrt{1 + \left(\frac{\overline{|\dot{\theta}_{NB}|}}{\overline{|\dot{U}_{NB}|}} \frac{dU_{NB}}{d\theta_N} \right)^2} (T_A) \right) \right]^{-1}, \tag{2-4}$$

3 Rice, S.O., Mathematical Analysis of Random Noise, *The Bell System Technical Journal*, Vol. 23, 1944.

4 Irwin, P.A. and Schuyler, G.D., Experiments on a Full Aeroelastic Model of Lions' Gate Bridge in Smooth and Turbulent Flow. National Research Council of Canada, *NAE Report LTR-LA-20*6, 1977.

5 Lepage, M.F., and Irwin, P.A., A Technique for Combining Historic Wind Data with Wind Loads, *Proc. 5th U.S. National Conference on Wind Engineering*, Lubbock, Texas, 1985.

6 Irwin, P.A., Prediction and Control of the Wind Response of Long Span Bridges with Plate Girder Decks, *Proc. Structures Congress '87/ST Div/ASCE*, Orlando, Florida, August 17-20, 1987.

where $\left|\overline{\dot{U}}_{NB}\right|$ and $\left|\overline{\dot{\theta}}_{NB}\right|$ are the averages of the absolute rates of changes of wind speed and wind direction for normal winds.

2.3 RESULTS

2.3.1 Mean-Hourly Speeds at 33 ft Height in Open Terrain

Our analysis of the Cincinnati-Northern-Kentucky International Airport wind data indicated a 50-year return period speed of 52 mph, mean hourly in comparison with the recommended ASCE 7-05 wind speed of 59.2 mph, which implies some conservatism in the code speed for this location. Considering the complexity of the local terrain however, the obtained results were scaled to comply with code recommended speed. It should be noted that this wind study recommends wind speeds applicable for design and stability following the currently accepted practice for bridge design in North America.

Figure 2-4 shows various wind speeds at 33 ft elevation for an open terrain as a function of return period. This figure present the following information:

- mean hourly speeds at 33 ft elevation for return periods from 1 to 10,000 years derived from the available meteorological data from the Cincinnati-Northern-Kentucky International Airport;
- mean hourly speeds at 33 ft elevation for open terrain derived from the ASCE 7-05 recommended 3-sec gust speed for the Cincinnati area; and
- the 10-min mean speed for 1,000 and 10,000-year return periods.

Mean-hourly speeds are to be used for derivation of design loads whereas 10-min speeds are to be applied for stability assessments.

2.3.2 Wind Directionality Effects

Figure 2-5 shows probability of exceeding various mean-hourly wind speeds at a 105' deck height as a function of wind direction. The curves show the probability of exceeding wind speeds with 10, 100, 1000 and 10,000 year return periods as a function of wind direction. Also the probability of all winds, based on entire wind record data set is shown. The proposed bridge main span axis is oriented at approximately 2 degrees from the north-south alignment. Therefore, winds normal to the span would blow from approximately east and west. Figure 2-5 shows that the most probable directions for strong winds (e.g., once in 100 years) would likely be rotated slightly toward north and south of the main west direction (i.e., from about 250 and 290 degrees). Since the loading of individual structural components varies differently with wind direction, it is difficult to develop a generally applicable directionality reduction factor for all structural components. Some structural elements reach peak loading in quartering winds. This, combined with the above-mentioned alignment of strong winds, indicated to us that for this stage no directionality reduction should be applied to the wind loads for design winds. There is evidence (Irwin and Schuyler[4]) that flutter instability is essentially a function of the wind velocity component normal to the span. However, based on the directionality of the meteorological models near the bridge site and the orientation of the span, a significant directionality reduction is not expected. Therefore, no directionality reduction factors have been applied to the wind speeds for stability assessment or design wind loading.

From the information available for the bridge site (satellite images, topological maps and site photographs), it appears that large hills located on the south side of the Ohio River could shelter and deviate the wind flow. Bearing in mind the strong winds coming from southwest (as presented in Figure 2-5), an investigation was undertaken to determine if the hills to the southwest were significant enough to be diverting the southwest winds at the bridge site.

2.3.3 Wind Directionality Effects – Investigation of the southwest hills impact

RWDI used software called MS-Micro by Zephyr North[7] for estimating the directional deviation at the bridge site. This program uses a digital terrain information to estimate localized effects of complex terrain on wind. A numerical simulation was carried out on a domain of 9.3 miles by 9.3 miles at a grid resolution of approximately 394 ft. The simulation entailed 36 wind directions in 10 degree increments. Directional deviations were extracted at the bridge location at deck height for all 36 wind directions. The results showed the winds from the south deviating slightly to the east (counter-clockwise), and winds from the southwest and west deviate slightly more to the north (clockwise), which indicates that the winds are being diverted around the hills to the southwest of the bridge. The maximum directional deviation over all wind directions was however less than 4 degrees. Since the directional resolution of the historical data is 10 degrees, i.e. with precision lower than the expected flow deviations, no adjustment to the historical wind direction data was applied.

2.3.4 Terrain at the Bridge Site

The terrain surrounding the existing bridge is generally a combination of open water, urban and suburban areas and wooded countryside. To assess the terrain effects, the ESDU method was used. The wooded countryside and suburban areas were taken as having roughness lengths in the range of z_0 = 0.3 ft to 2.3 ft. The roughness lengths of the water fetches were classified following the ESDU recommendations being in the range of 0.003 ft to 0.008 ft. In terms of the traditional power law, in which mean velocity varies with height to the power of an exponent α, where this value ranges from 0.14 to 0.19.

2.3.5 Wind Speeds at Deck Height

The ratio of the mean velocity at a deck height of 105 ft to the mean velocity in standard open terrain at 33 ft (from Section 2.3.1) was found to be 1.08. The 100-year mean-hourly velocity at a height 105 ft was predicted to be 66.3 mph. Figure 2-6 also shows the 10-minute mean wind speeds at the deck height as a function of return period relevant for this study.

[7] http://zephyrnorth.com/index.html

2.3.5.1 Structural Design Wind Speed

For structural design of major bridges, a return period of 100 years is typically used. As described in the previous section, the 100-year mean-hourly speed was estimated to be 66.3 mph at a height of 105 ft (Table 2-1). For the construction phase, return period 20 years is typically recommended giving mean-hourly speed of 60.4 mph.

2.3.5.2 Design Wind Speed for Aerodynamic Stability

For flutter instability of the completed bridge, a very long return period needs to be considered because, if flutter occurs, there is a very high probability of structural failure. The recommended return period is 10,000 years. Since directionality reduction effects are not available (see section 2.3.2), the mean-hourly velocity for 10,000-year return period was determined as 81 mph. As previously discussed, flutter oscillations can build up over shorter periods than 1 hour; therefore, normally 10-minute mean value is applied. Using the ratio of 1.067 to scale mean hourly speeds to 10-minute mean speeds, the design speed for flutter is thus calculated to be 1.067 × 80.9 mph = 86.3 mph. For construction, a shorter return period is justifiable due to the shorter length of exposure during the construction period, and 1,000 years is recommended. The 1,000-year design flutter speed, arrived at by a similar approach, is 1.067 × 74.0 m/s = 79 mph.

2.3.6 Turbulence Properties at the Bridge Site

The same ESDU methodology used in determining the wind speeds at a height of 105 ft was also applied for the estimation of turbulence intensities and length scales at the site. The turbulence intensities (I_u, I_w, I_v and length scales (${}^xL_u$, ${}^xL_w$, ${}^yL_u$, ${}^yL_w$, and ${}^zL_w$), which are most important for the buffeting response of long-span bridges to strong winds, are given in Table 2-2.

2.4 WIND CLIMATE ANALYSIS: SUMMARY

The design wind speeds resulting from the wind climate and site analysis for the Brent Spence Bridge are summarized in Table 2.1. The resulting turbulence properties are shown in Table 2-2. The mean-hourly speeds are recommended for bridge design, and the 10-minute mean speeds are suggested for stability evaluations both during construction and for the completed bridge. The long-term wind records from the Cincinnati-Northern-Kentucky International Airport were the primary source of data used, although the data for Cincinnati Municipal Airport were also considered. Open water and the wooded/suburban/urban terrain around Cincinnati affect the exposure of the bridge site. These terrain effects have been accounted for arriving at the recommended speed values given in Table 2-1 and the turbulence properties in Table 2.2. The impact of the southwest hills was also investigated where the numerical assessment demonstrated that the wind deviation resulting from the interference of the proximity hills with the wind flow is negligible.

TABLES

Table 2-1: Recommended wind speeds at the site

Wind Speed Applicable for	Return Period (years)	Mean Wind Speed (mph) at Deck Level 105 ft and Averaging Time		Corresponding Mean Hourly Wind Speed (mph) at 33 ft Open Terrain
Design during construction	20	60.4	1 h	56.1
Design of completed bridge	100	66.3	1 h	61.5
Stability during construction	1,000	79	10 min	68.6
Stability of completed bridge	10,000	86.3	10 min	75.0

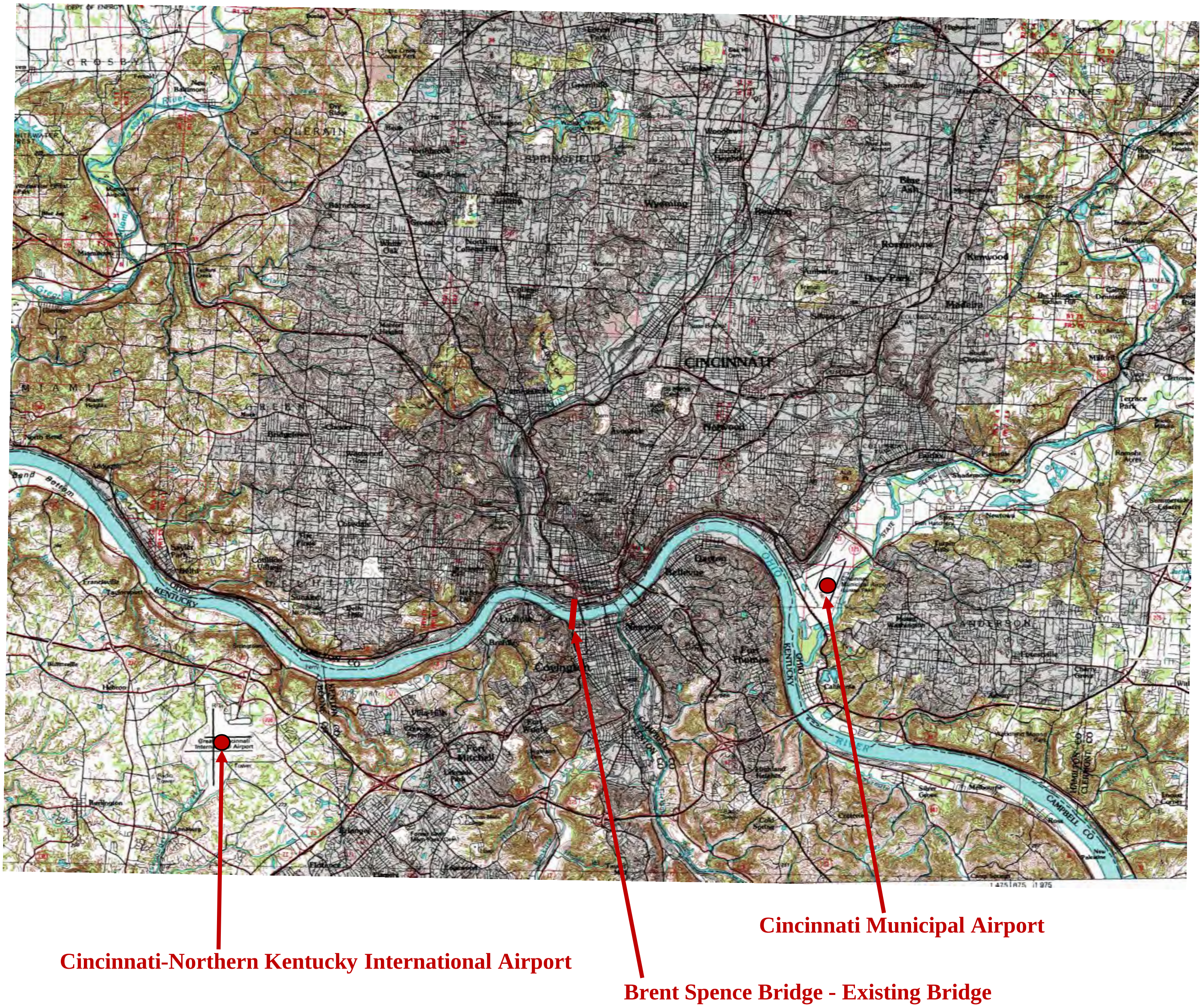
Notes: 1. Given elevation is the approximate average of the two deck elevations at mid-span, above the mean water level

Table 2-2: Turbulence Properties at Deck Level (105 ft above mean water level)

α	I_u (%)	I_v (%)	I_w (%)	$^{x}L_u$ (ft)	$^{x}L_w$ (ft)	$^{y}L_u$ (ft)	$^{y}L_w$ (ft)	$^{z}L_u$ (ft)
0.17	19.1	15	10.5	1406	117	383	64	232

Notes:
1. α - power law constant of wind profile
2. $I_{u,v,w}$ - longitudinal, horizontal-across-wind, and vertical turbulence intensities
3. $^{x,y,z}L_{u,v,w}$ - turbulence length scales

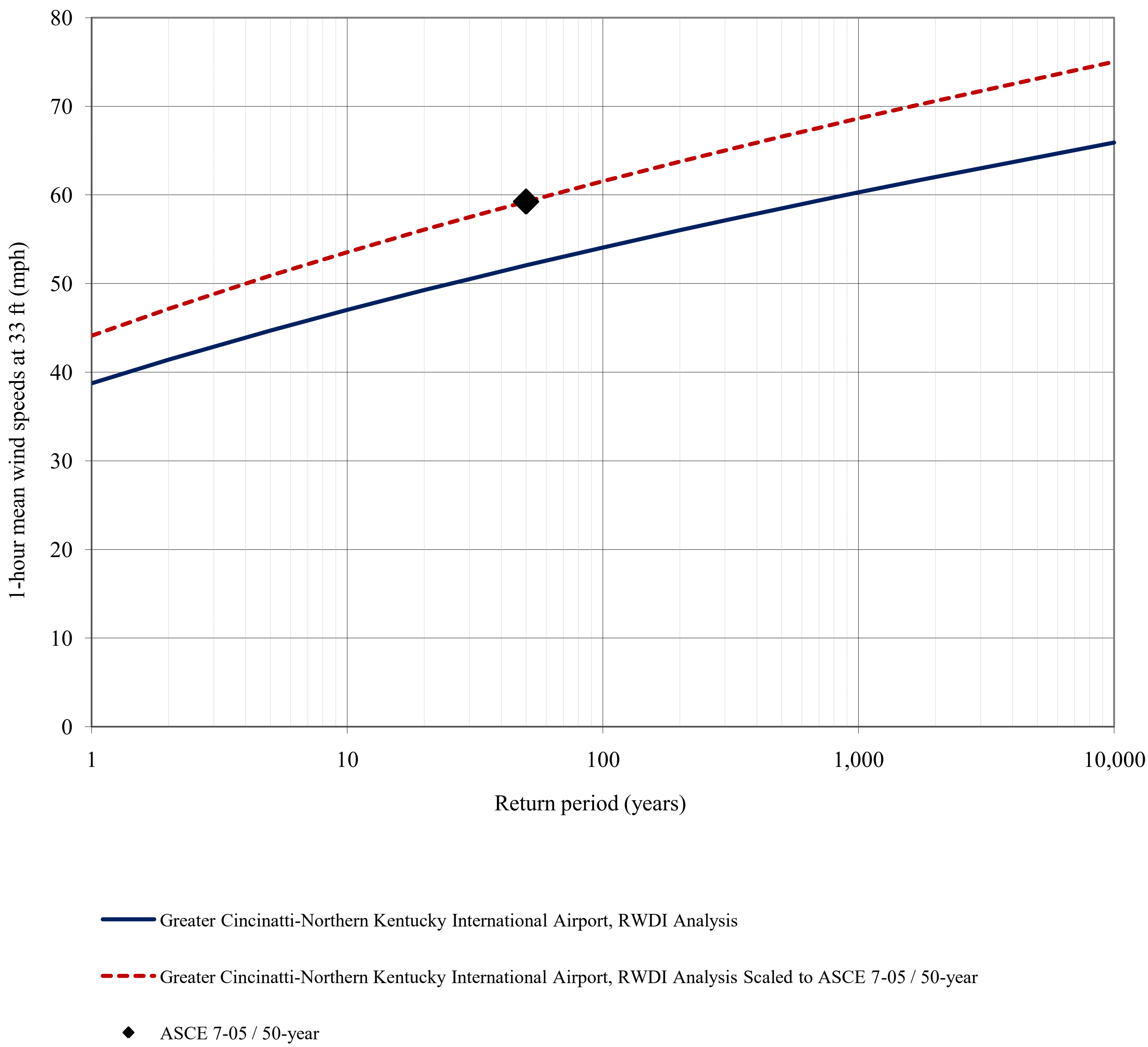
FIGURES



Plan of the Brent Spence Bridge Site Wind Engineering Study Brent Spence Bridge, Cincinnati, OH	Project 0940582	Figure 2-1 August 31, 2010	RWDI

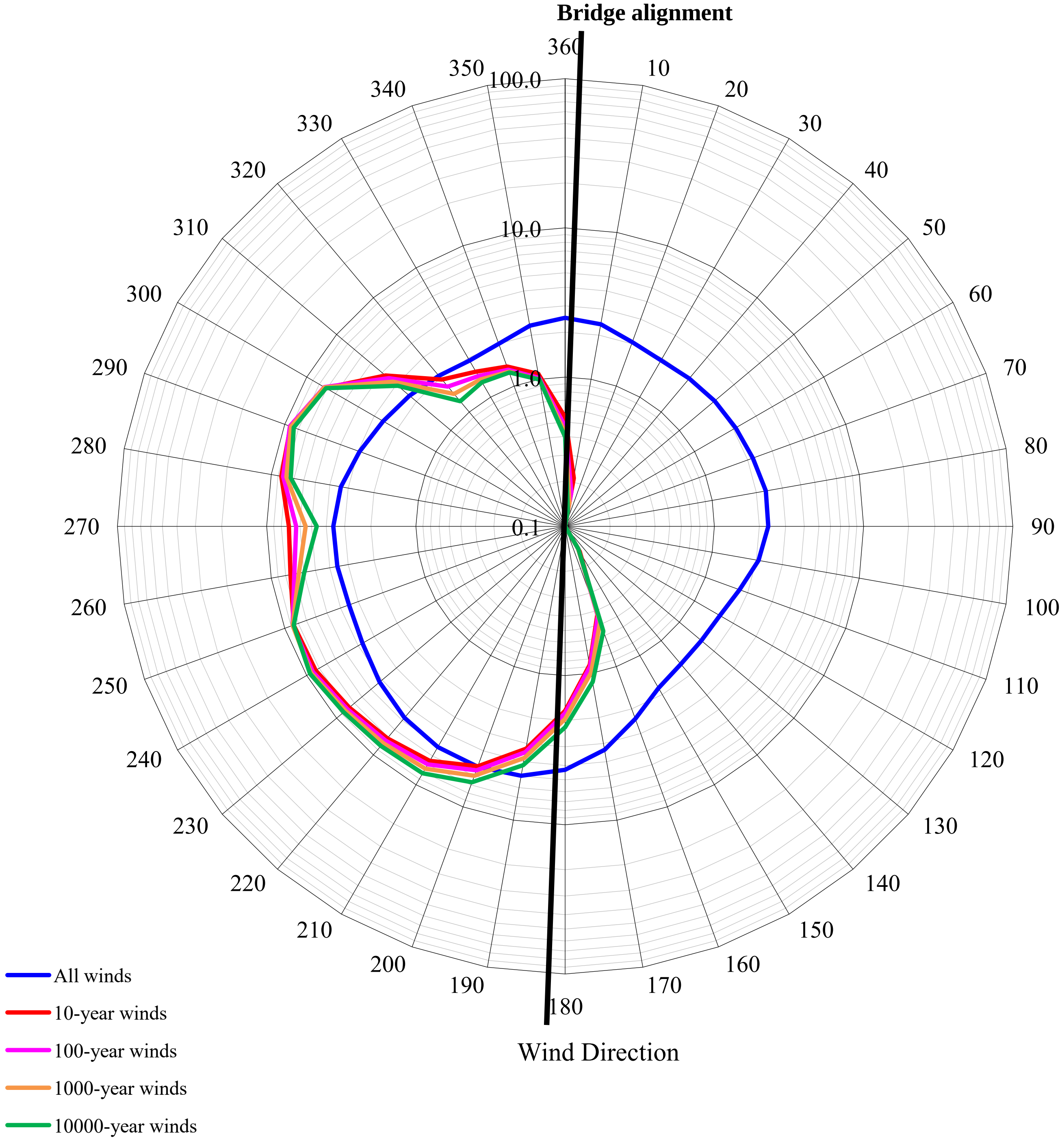
Photographs of the Brent Spence Bridge Site Wind Engineering Study Brent Spence Bridge, Cincinnati, OH Project 0940582	Figure 2-2	RWDI
	July 7, 2010	

Photographs of the Brent Spence Bridge Site	Figure 2-3	RWDI
Wind Engineering Study Brent Spence Bridge, Cincinnati, OH Project 0940582	July 7, 2010	

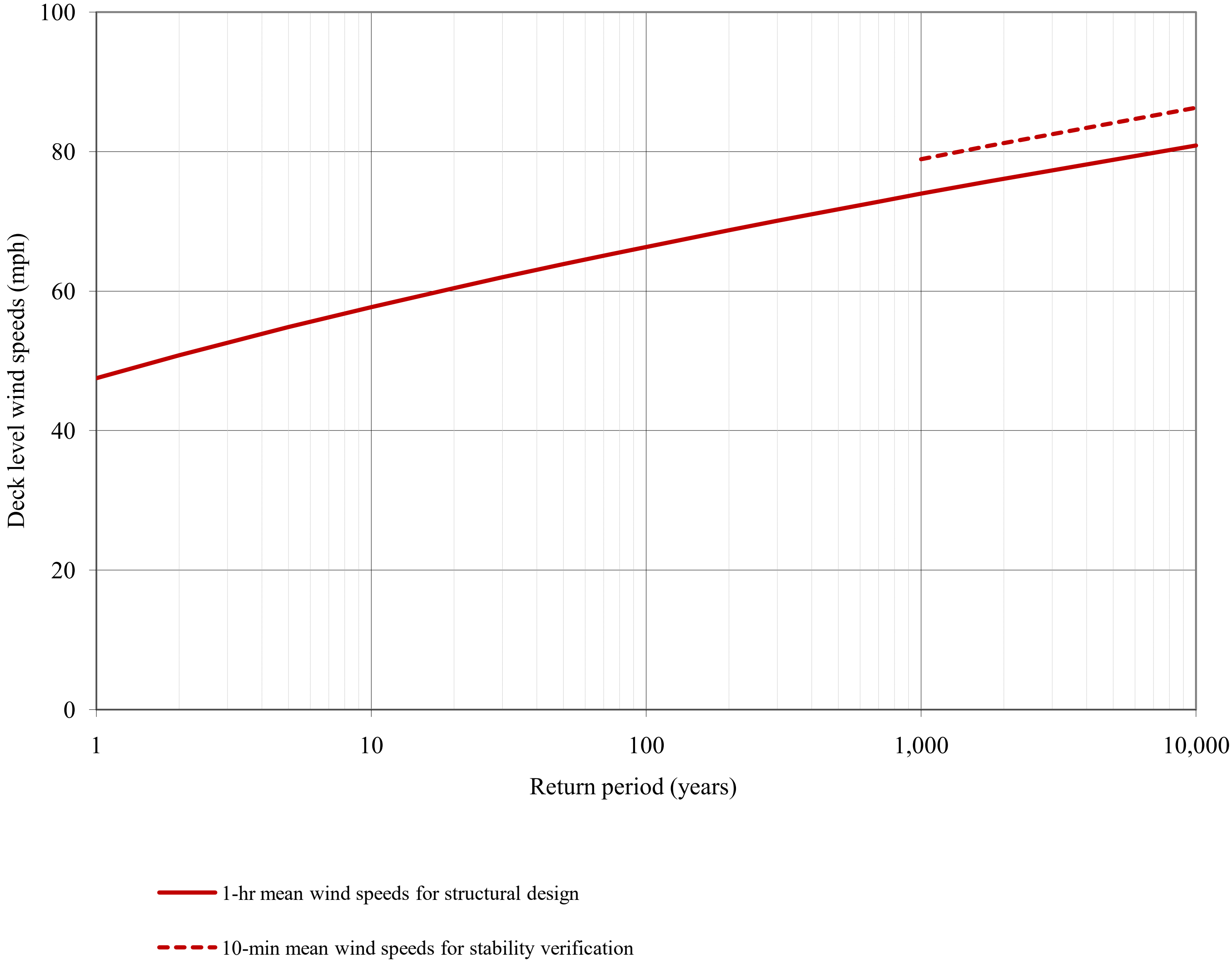


Mean-hourly wind speed at 33 ft for an open terrain Wind Engineering Study Brent Spence Bridge, Cincinnati, OH Project 0940582	Figure 2-4 July 16, 2010	RWDI

NORTH



Directional distribution of mean-hourly winds at the bridge site Probability (%) of the wind direction for certain return periods Wind Engineering Study Brent Spence Bridge, Cincinnati, OH Project 0940582	Figure 2-5 July 16, 2010	RWDI



Mean wind speed for various return periods **Wind speeds at deck level (105 ft above water level)** Wind Engineering Study Brent Spence Bridge, Cincinnati, OH Project 0940582	Figure 2-6 July 16, 2010	RWDI