REPORT OF

SUBSURFACE EXPLORATION

TRUCK LANE IMPROVEMENTS

GATEWAY INTERNATIONAL BRIDGE

BROWNSVILLE, TEXAS

TTL

TRINITY TESTING LABORATORIES, INC.

SAN ANTONIO LAREDO LOWER RIO GRANDE VALLEY

TRINITY TESTING LABORATORIES, INC.

2020 LOOP 499, #310 HARLINGEN, TEXAS 78550 (512) 423-8231

June 7, 1992

Cameron County Engineering
964 E. Harroson
Brownsville, Texas 78550

Attention: Mr. Andy Cueto

Subject: Report of Subsurface Exploration
Truck Lane Improvements
Gateway International Bridge
Brownsville, Texas
Project No. 31103

Gentlemen:

We are pleased to submit this report of subsurface exploration for the truck lane improvements at the Gateway International Bridge in Brownsville. The findings and descriptions of the exploration and testing procedures are presented in the report along with foundation design recommendations.

We appreciate the opportunity to assist in this phase of the project. Please feel free to call us if you have any questions regarding this report or if we may be of further service.

Very truly yours,

TRINITY TESTING LABORATORIES, INC.

Gregory P. Thomas

John W. Dougherty

John W. Dougherty, P.E.

Copies submitted: Client - 2
Faraklas Engineers, Inc. - 1

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INTRODUCTION

Authorization and Scope

The subsurface exploration presented in this report was authorized by Cameron County Purchase Order Number 74879 dated May 12, 1992. The purposes of the exploration were to determine the subsurface stratification and the characteristics of the various soil encountered at the site and to develop recommendations for pile design for the extension of the bridge abutment.

Project Description

The bridge improvements will consist of the widening of the span between Pier #5 and the abutment on the U.S bound lanes of the bridge to provide a larger turning radius. The widening will extend the abutment about forty (40) feet to the southeast and widen the widen the curved portion of the span up to about twenty (20) feet. Dead loads on the extension will be 75 to 100 pounds per square foot and loads on the abutment extension will be 250 to 350 kips.

Pier #5 was reinforced in 1984 and is considered adequate to support the new loads. The structure constructed at that time around the pier can be seen in Photograph Nos. 2 and 3 in the appendix. The structure is supported on augered, pressure grouted (Augercast) piles that are fourteen inches in diameter with tip elevations of -35 feet. A 35 ton individual pile capacity was used in the design of the reinforcement. Augercast piles are the preferred foundation type for the abutment extension as well.

FIELD AND LABORATORY TESTING

Field Testing

The site was explored by drilling two soil test borings to depths of seventy-five (75) feet below the existing grades at the locations that are shown on the Soil Boring Location Plan in the appendix. The borings were located as close as possible to the locations designated by Faraklas Engineers, Inc. The ground elevations at the boring locations was estimated from the topographic information on the furnished site plan.

The borings were advanced to depths of forty-one (41) and fifty (50) feet using continuous flight, hollow stem augers. Rotary drilling with bentonite drilling fluid circulated to stabilize the bore hole and carry the cuttings to the surface was used below those depths. Samples of the materials encountered were obtained by thin wall tube sampling methods or by split barrel sampling with standard penetration testing. The field sampling and testing were performed in substantial compliance with applicable ASTM standards. The test boring logs are presented the appendix along with descriptions of the test methods.

Laboratory Testing

The soil samples were examined and visually classified by the soils engineer and samples representative of the various soil strata encountered were selected for laboratory testing. Atterberg Limits, percent fines, moisture content, and unit dry weight tests were performed to assist in classifying the soils and to provide indicators of soil behavior. Unconfined compression and hand penetrometer tests were used, along with the standard penetration test results to estimate the strength of the soil. The test results are presented on the boring logs and the Unconfined Compression Tests summary. The test procedures are described in the appendix.

SITE AND SUBSURFACE CONDITIONS

Area Geology

The "Geologic Atlas of Texas" shows that the area of this site is underlain by undivided alluvium that is composed of clay, sand, silt, gravel and organic matter. The silts and sands are mostly quartz, calcareous, and dark gray to dark brown in color. The gravel along the Rio Grande includes sedimentary rocks from the Cretaceous and Tertiary periods and a wide variety of igneous and sedimentary rocks, including agate, from Trans-Pecos Texas, Mexico and New Mexico. Gravel in side streams of the Rio Grande is mostly local Tertiary rocks and chert derived from Uvalde Gravel that caps divides.

Site Conditions

The project site is the abutment span of the U.S bound lane of the bridge. The span is about seventy feet long and thirty-six feet wide. It is supported by the abutment and Pier #5. Photograph Nos. 1 through 3 in the appendix show the project area.

Subsurface Conditions

The elevations at the test borings were +30 and +41 at borings B-1 and B-2, respectively. The subsurface profile in the appendix shows the general soil stratification and strength test information at the two borings. These results are similar to those in the borings for the Pier #5 repairs, including the softer zone in the area of -20 to -28 that is described below.

At B-2, hard, brown, fine sandy, silty clays and brown, fine sandy, clayey silts were initially penetrated to about elevation +30 feet. Hand penetrometer test values were more than the 4-½ tons per square foot capacity of the testing device. The results of one penetration test in the clayey silts were 16 blows per foot of penetration. The clays and silts are classified as CL and CL-ML, respectively under the Unified Soil Classification System.

From elevation +30 feet, which is the surface at B-1, to about -5 feet, tan, gray, and dark gray and reddish-tan, silty clays were penetrated. These soils are firm to very stiff.

Hand penetrometer test results were from ½ to 2-¼ tons per square foot. Two samples had unconfined compressive strengths of 0.91 and 1.50 tons per square foot. Unit dry weights vary from about 90 to 100 pounds per square foot. These soils are predominantly low plasticity, CL clays.

From about -5 feet to -28 feet, tan, silty clays were penetrated. These soils are very stiff to hard to about -20 feet with a softer firm to very stiff zone to about -28 feet. These clays are classified as CL or CH depending upon whether the liquid limit is less than 50 (CL) or 50 or greater (CH). Hand penetrometer test values were ¼ to 3-¼ tons per square foot and standard penetration test results were 13 to 28 blows per foot. One sample had an unconfined compressive strength of 2.62 tons per square foot.

Hard tan, silty clays were next penetrated to the boring terminations at -36 feet and -45 feet. These clays are CL at B-1 with liquid limits of 42 and 49 and plasticity indices of 21 and 29 for two tests. At B-2, the liquid limit and plasticity tests for one sample were 58 and 31, respectively. These soils are classified as CH under the Unified System. Standard penetration test results were 23 to 42 blows per foot of penetration.

The above descriptions and the subsurface profile are generalized to highlight the major subsurface stratification and soil characteristics. The boring logs should be consulted for specific information at each boring location.

Groundwater

Groundwater was not encountered in the borings during; at 31 feet below grade in B-1 and at 41 feet in B-2. These depths are about elevation +0 feet. The use of drilling fluid in the borings prevented the measurement of delayed water levels. Groundwater levels will fluctuate changes in the river level.

EVALUATION

General

The soil conditions encountered in the two borings are similar to those found in the 1984 borings and are considered to fairly represent the conditions at the abutment extension location. They consist of generally very stiff to hard silty clays with some softer layers or lenses. The soil conditions are suitable for the use of augercast piles to support the new portion of the abutment. Driven piles could also be used but large cross-sectional area members would be most appropriate since the piles will develop capacity largely by friction on the pile shaft with only a small contribution from end bearing. Driven piles would be more likely to damage the nearby streets and utilities by vibration or soil heave that the non displacement augered piles. Recommendations for augercast pile design and construction are presented below.

Potential Vertical Rise (PVR)

The soils above the groundwater level are mildly expansive soils. Potential vertical soil movements have been estimated using the Texas Department of Highways and Public Transportation method TEX-124-E, Potential Vertical Rise. This method utilizes the soils in situ moisture conditions and plasticity characteristics within the active zone. It is estimated that depth of the active zone in this area is approximately fifteen feet. The estimated potential Vertical Rise (PVR) values are less than one (1) inch for all moisture conditions. No sustained surcharge load was assumed in the PVR calculations. It is noted that the PVR estimates are provided as an indicator of the severity of potential soil movements at this site and are not intended as a prediction of actual soil and foundation movements.

RECOMMENDATIONS

Site Preparation

Site preparation should consist of the relocation of utilities, if necessary and the excavating for the new abutment section.

All excavations should be sloped or shored according to federal and local regulations. The federal regulations are contained in Section 1926.652 of Title 29, Code of Federal Regulations (29 CFR). The portion of the regulations pertaining to soil classification is reproduced in the appendix. Shoring requirements are based upon several broad soil classes. Based on the information from this site, we classify the shallow, hard, silty clay and clayey silt strata at B-2 as Type A. The classification at the time of construction, however, may be different due to changed moisture conditions and should be determined at the start of construction.

Pile Foundation

It is assumed that dredging of the river channel which could be as deep as elevation -5 feet, will not remove the soil surrounding the abutment piles. The soil above +30 feet was neglected in the pile capacity calculations to account for the abutment and the upper few feet of pile that do not fully develop the soil/pile friction.

In order to obtain reasonable pile capacity and place the pile tips below the softer soil zone just above elevation -30 feet, we recommend that the piles be drilled to a uniform depth with the pile tips at elevation -35 feet. This will avoid any potential settlement due to loading of the soft soils by the pile tops of by the transfer of pile load to the soil by shaft friction. A pile diameter of at least fourteen (14) inches is recommended.

Static pile capacity calculations indicate an individual pile capacity of 50 tons for 14-inch piles with a tip elevation of -35 feet. This includes a factor of safety of 3.0. A factor of safety of 2.5, not unreasonable for this pile type in these soil conditions, allows a pile capacity of 60 tons. The end bearing component is about 1/5 of the total pile capacity.

Guide specifications for the augercast pile materials and installation procedures are included in the appendix.

Pile Inspection

Normal dynamic driving criteria and inspection procedures for driven piles do not apply to augercast piles. Satisfactory installation depends upon the use of correct procedures and upon the skill and experience of the contractor. Observation of the installation, is therefore of critical importance. Full time monitoring assures that the pump pressures, rate of auger retraction, grout mixing, grout quantity procedures are uniformly followed. In this case, the small number of piles make a full scale load test excessively expensive. Previous experience on the site indicates that the design capacity is developed when proper equipment, materials and installation procedures are used.

Abutment Backfill

Backfill for the abutment should be a low plasticity sandy clay similar to the existing shallow soils or, preferably, a clean sand or sand and gravel. The latter soils will minimize lateral soil loads on the abutment and will facilitate drainage. Specifications for fill selection and placement are included in the appendix.

GENERAL REMARKS

Construction Services

We recommend that the soils engineer be retained to provide soil engineering services during installation of foundation piles. This is necessary to make the observations noted above and determine compliance with the design and project specifications. It also allows design or construction changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.

Limitations

The subsurface exploration presented in this report is based partly upon the information obtained from the two test borings. The nature and extent of variations in soil conditions between or beyond the borings may not be known until actual construction. The transition lines shown on the boring logs are approximate and the actual transitions may be gradual. Soil samples not altered by laboratory testing will be retained for a period of 30 days and then, unless we are directed otherwise, will be discarded.

This report has been prepared for the use of the Cameron County Engineering Department for specific application to the truck lane improvements at the Gateway International Bridge in Brownsville according to generally accepted foundation engineering practices. No other warranty, expressed or implied, is made. Additional information regarding the limitations and use of geotechnical engineering reports is included in the appendix.

APPENDIX

Photographs
Soil Boring Location Plan
Subsurface Profile
Logs of Borings
Symbols and Terms Used on Boring Logs
Unconfined Compression Tests
Field and Laboratory Testing Procedures
Recommended Specifications for Placement
  of Compacted Fill
Augercast Pile Guide Specifications
29 CFR Ch. XVII (7-1-90 Edition
  Appendix A to Subpart P
Important Information About Your
  Geotechnical Engineering Report

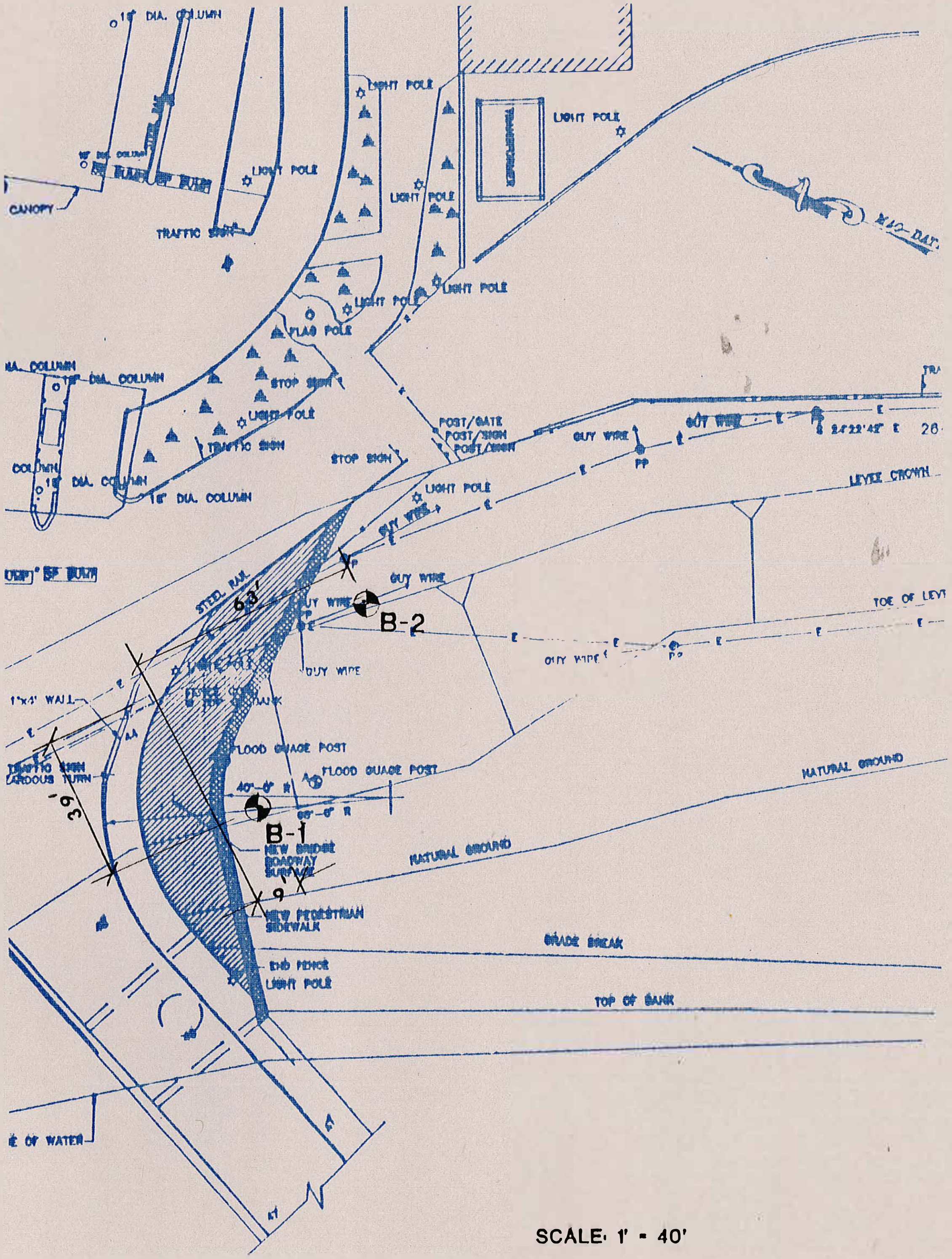
#1 - Abutment area viewed from boring location B-2.

#2 - Pier #5 and abutment span seen from boring B-2.

Gateway International Bridge
Truck Lane Improvements, Brownsville, Texas
Project Number: 31103

#3 Pier #5 and abutment span viewed from south of boring location B-1.

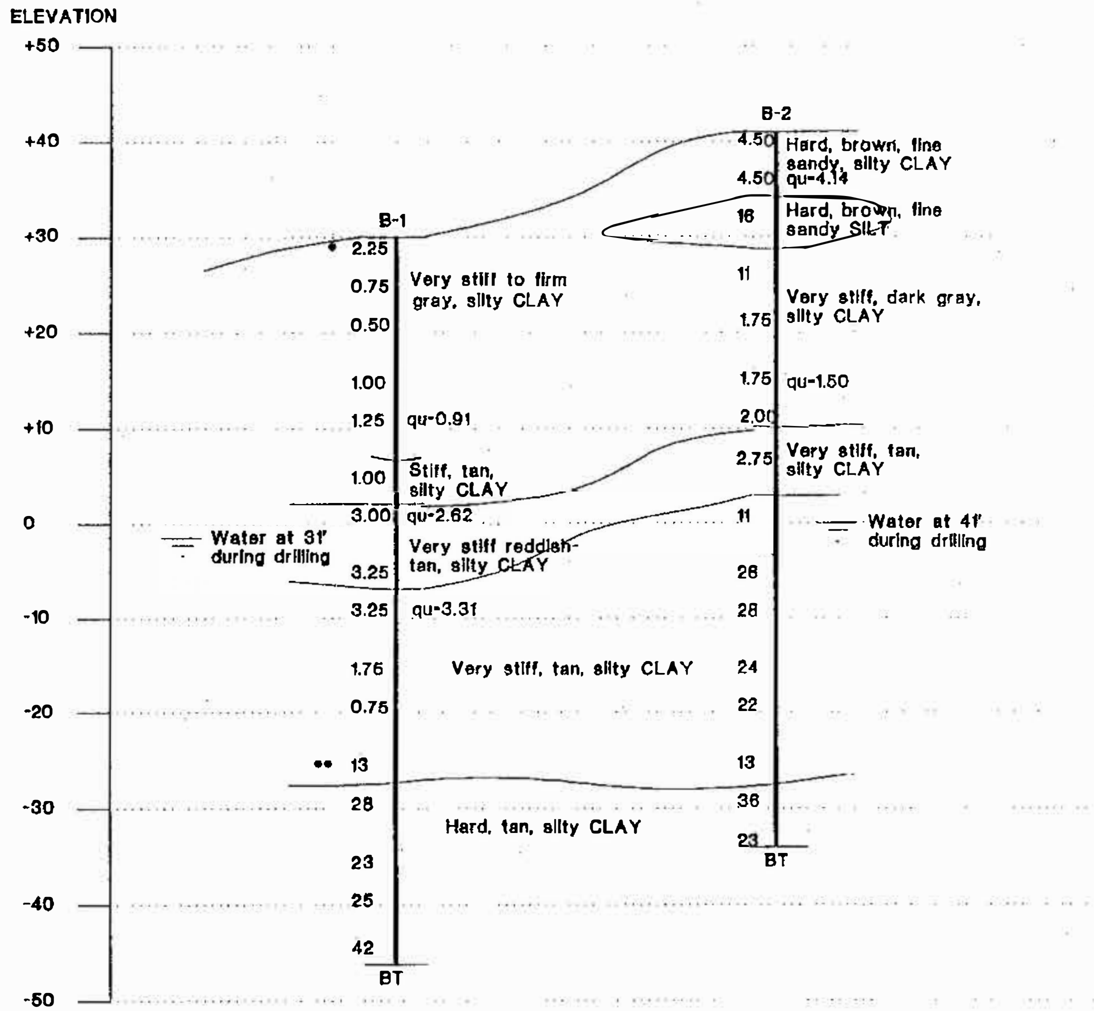
Gateway International Bridge
Truck Lane Improvements, Brownsville, Texas
Project Number: 31103



SCALE: 1' = 40'

- SOIL TEST BORING LOCATION (DRILLED 5/27-28/92)

TTL — **SOIL BORING LOCATION PLAN**	PROJECT: TRUCK LANE IMPROVEMENTS, GATEWAY INTERNATIONAL BRIDGE, BROWNSVILLE, TEXAS
TRINITY TESTING LABORATORIES, INC., 310 BREESPORT, SUITE B, SAN ANTONIO, TEXAS 78216	PROJECT NUMBER: 31103



LEGEND

- • Hand penetrometer test (TSF)
- •• Standard penetration test (Blows per Foot)
- qu Unconfined compressive strength (TSF)
- BT Boring Terminated

SUBSURFACE PROFILE	PROJECT: TRUCK LANE IMPROVEMENTS, GATEWAY INTERNATIONAL BRIDGE, BROWNSVILLE, TEXAS
TRINITY TESTING LABORATORIES, INC. 2020 LOOP 499, #310, HARLINGEN, TEXAS 78550	PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-1

PROJECT: Truck Lane Improvements to Gateway International Bridge
LOCATION: Brownsville, Texas
GROUNDWATER OBSERVATIONS: 31' DURING DRILLING

DATE: 5-27-92
WEATHER: Partly Cloudy, Showers
TYPE BORING: HSA/Rotary
ELEVATION: 30.0

DEPTH (FT.)	SAMPLES	SOIL TYPE Qp	DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
0-1	S	(2-¼)	Very stiff to firm, gray, silty CLAY (CL)	22.3					97.7
4-5	S	(¾)		29.9					92.6
9-10	S	(½)	Same with black organic stains	33.0					88.7
14-15	S	(1)		30.8					88.2
19-20	S	(1-¼)		33.0	41	22	99.9	0.91	82.3
24-25	S	(1)	Stiff, tan, silty CLAY (CL)	25.4					93.3

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w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-1

PROJECT: Truck Lane Improvements to Gateway Imternational Bridge

DATE: 5-27-92

LOCATION: Brownsville, Texas

DEPTH (FT.)	SAMPLES	SOIL TYPE Qp	DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
26–28			Stiff, tan, silty CLAY (CL)						
28–37			Very stiff, reddish-tan, silty CLAY (CH)						
29–30	S	(3)		27.4				2.62	92.7
31			▼ Groundwater encountered at 31 feet during drilling						
34–35	S	(3-¼)		28.2					90.8
37–50			Very stiff to firm, tan, silty CLAY (CH)						
39–40	S	(3-¼)		21.8	57	37	99.5	3.31	99.7
44–45	S	(1-¾)		25.4					
49–50	S	(¾)		27.8					

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w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-1

PROJECT: Truck Lane Improvements to Gateway Imternational Bridge
DATE: 5-27-92
LOCATION: Brownsville, Texas

DEPTH (FT.)	SAMPLES	SOIL TYPE Qp		DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
51			///	Firm to hard, tan, silty CLAY with thin sand lenses from 50 to 52 feet						
52			///							
53			///	(Rotary drilling with bent-tonite drilling fluid used below 50 feet)						
54	P	13	///		29.0	49	29	100		
55			///							
56			///							
57			///							
58			///							
59	P	28	///		31.3					
60			///							
61			///							
62			///							
63			///							
64	P	23	///	Same with rust stains	26.9					
65			///							
66			///							
67			///							
68			///							
69	P	25	///	Same with rust stains	23.4	43	21	99.9		
70			///							
71			///							
72			///							
72			///							
74	P	42	///		21.9					
75			///	Boring Terminated						

w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-2

PROJECT: Truck Lane Improvements to Gateway International Bridge
LOCATION: Brownsville, Texas
GROUNDWATER OBSERVATIONS: 41' DURING DRILLING

DATE: 5-28-92
WEATHER: Partly Cloudy, Showers
TYPE BORING: HSA/Rotary
ELEVATION: 41.0

DEPTH (FT.)	SAMPLES	Qp	SOIL TYPE	DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
0-1	S	(4-½+)	///	Hard, brown, fine sandy, silty clay with traces of gravel and calcareous deposits (CL)	8.0					
4-5	S	(4-½+)	///		15.3				4.14	99.1
9-10	P	16	|||	Hard, brown, fine sandy, clayey SILT (CL-ML)	21.7	22	6	73.4		
14-15	P	11	\\\\\\	Very stiff, dark gray, silty CLAY (CH)	32.7					
19-20	S	(1-¾)	///	Very stiff, tan, silty CLAY with rust stains (CL)	38.0					
24-25	S	(1-¾)	///		27.3				1.50	91.6

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w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-2

PROJECT: Truck Lane Improvements to Gateway Imternational Bridge

DATE: 5-27-92

LOCATION: Brownsville, Texas

DEPTH (FT.)	SAMPLES	SOIL TYPE Qp		DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
26–30			///	Very stiff, tan, slightly fine sandy, silty CLAY (CL)						
29–30	S	(2)	///		26.7	42	28	93.3		91.6
31–38			\\\	Very stiff to hard, reddish-tan, silty CLAY (CH)						
34–35	S	(2-¾)	\\\		27.8					
38–50			\\\	Very stiff to hard, tan, silty CLAY (CL)						
39–40	P	11	\\\		30.0					
41			\\\	▼ **Groundwater encountered at 41 feet during drilling**						
			\\\	(Rotary drilling with bentonite drilling fluid used below 41 feet)						
44–45	P	26	\\\		26.4	52	34	98.8		
49–50	P	28	\\\	Same with calcareous deposits	28.3					

----- Continued next page -----

w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

LOG OF BORING NUMBER B-2

PROJECT: Truck Lane Improvements to Gateway Imternational Bridge

DATE: 5-27-92

LOCATION: Brownsville, Texas

DEPTH (FT.)	SAMPLES	SOIL TYPE Qp	DESCRIPTION AND COMMENTS	w (%)	LL	PI	-200 (%)	Qu TSF	UNIT DRY WT. (PCF)
51			Hard, tan, silty CLAY (CH)						
52									
53									
54	P	24		29.7	58	34	100		
55									
56									
57									
58									
59	P	22		25.7					
60									
61									
62			Sandy seams between 61 and 63'						
63									
64	P	13		25.7					
65									
66									
67									
68									
69	P	36		27.9					
70									
71									
72									
72									
74	P	23		30.7	58	31	99.9		
75			Boring Terminated						

w - Moisture Content(%) LL - Liquid Limit PI - Plasticity Index -200 - Silt and Clay Fraction(%)
N - Standard Penetration Resistance (BL/FT) Q_p - Hand Penetrometer Test (TSF)

PROJECT NUMBER: 31103

SYMBOLS AND TERMS USED ON BORING LOGS

UNIFIED SOIL CLASSIFICATION SYSTEM			
GRAVELS More than half of coarse fraction larger than No. 4 sieve size		GW	Well graded gravels or sand and gravel mixtures, little or no fines.
		GP	Poorly graded gravels or sand and gravel mixtures, little or no fines.
		GM	Silty gravels, poorly graded gravel-sand-silt mixtures.
		GP	Clayey gravels, poorly graded gravel-sand-clay mixtures.
SANDS More than half of coarse fraction smaller than No. 4 sieve size		SW	Well graded sands or gravelly sands, little or no fines.
		SP	Poorly graded sands or gravelly sands, little or no fines.
		SM	Silty sands, poorly graded sand-silt mixtures.
		SC	Clayey sands, poorly graded sand-clay mixtures.
SILTS AND CLAYS Liquid Limit less than 50%	|||||	ML	Inorganic silts and very fine sands of low to medium plasticity.
	/////	CL	Inorganic clays of low to medium plasticity.
	.....	OL	Organic silts and organic silty clays of low plasticity.
SILTS AND CLAYS Liquid Limit more than 50%	| | |	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts.
	\\\\\\\\\\	CH	Inorganic clays of high plasticity.
	::::::	OH	Organic clays of medium to high plasticity.
HIGHLY ORGANIC SOILS	-----	Pt	Peats and other highly organic soils.

TYPE OF TEST OR SAMPLE	
A - Auger sample	C - Rotary Coring Sample
P - Split Barrell Sample with Standard Penetration	S - Thin Wall Tube (Shelby Tube) Sample
	T - THD Cone Penetrometer Test

CONSISTENCY OF COHESIVE SOILS

Descriptive Term	Unconfined Compressive Strength (TSF)	Standard Penetration Resistance (BL/FT)
Very Soft	Less than 0.25	Less than 1
Soft	0.25 - 0.50	1 - 2
Firm	0.50 - 1.00	2 - 4
Stiff	1.00 - 2.00	4 - 8
Very Stiff	2.00 - 4.00	8 - 16
Hard	Greater than 4.00	Greater than 16

RELATIVE DENSITY OF COHESIONLESS SOILS

Descriptive Term	"N" Value (BL/FT)
Very Loose	0 - 4
Loose	4 - 10
Medium Dense	10 - 30
Dense	30 - 50
Very Dense	Over 50

RELATIVE PROPORTIONS

Proportional Term	Percentage by Weight
No Term	Less than 5
Slightly	5 - 12
---ly	12 - 35
and	35 - 50

TRINITY TESTING LABORATORIES, INC.

UNCONFINED COMPRESSION TESTS

Truck Lane Imprivements
Gateway International Bridge
Brownsville, Texas
Project Number: 31103

UNCONFINED COMPRESSION TESTS						
BORING NUMBER	DEPTH (FT) FROM TO	MOISTURE CONTENT (%)	UNIT DRY WEIGHT (PCF)	UNCONF. COMP. STRENGTH (TSF)	STRAIN AT FAILURE	SOIL DESCRIPTION
B-1	19-20	34.8	82.3	0.91	21.2	Gray, silty clay
B-1	29-30	27.4	92.7	2.62	7.5	Reddish-tan, silty clay
B-1	39-40	21.8	99.7	3.31	17.5	Tan, silty clay
B-2	4-5	15.3	99.1	4.14	5.0	Brown, fine sandy, silty clay
B-2	24-25	27.3	91.6	1.50	24.2	Dark gray, silty clay

FIELD AND LABORATORY TESTING PROCEDURES

FIELD TESTING

A. Boring Procedure Between Samples

The borehole is extended downward, between samples, by continuous flight, hollow or solid stem augers or by rotary drilling techniques using bentonite drilling fluid or water.

B. Standard Penetration Test and Split-Barrel Sampling of Soils (ASTM D-1586)

This sampling method consists of driving a 2-inch outside diameter split barrel sampler using a 140 pound hammer freely falling through a distance of 30 inches. The sampler is first seated 6 inches into the material to be sampled and then driven an additional 12 inches. The number of blows required to drive the sampler the final 12 inches is known as the Standard Penetration Resistance. Recovered samples are first classified as to color and texture by the driller. Later, in the laboratory, the driller's field classification is reviewed by the soils engineer who examines each sample.

C. Thin-walled Tube Sampling of Soils (ASTM D-1587)

This method consists of pushing thin walled steel tubes, usually 3 inches in diameter, into the soils to be sampled using hydraulic or other means. Cohesive soils are usually sampled in this manner and relatively undisturbed samples are recovered.

D. Soil Investigation and Sampling by Auger Borings (ASTM D-1452)

This method consists of augering a hole and removing soil samples from the auger flight or bit at 5 foot intervals or with each change in the substrata. Disturbed samples are obtained and this method is, therefore, limited to situations where it is satisfactory to determine the approximate subsurface profile.

E. Diamond Core Drilling for Site Investigation (ASTM D-2113)

This method consists of advancing a hole into hard strata by rotating a single or double tube core barrel equipped with a cutting bit. Diamond, tungsten carbide, or other cutting agents may be used for the bit. Wash water is used to remove the cuttings and to cool the bit. Normally, a 2 inch outside diameter by 1-3/8 inch inside diameter (NX) coring bit is used unless otherwise noted. The rock or hard material recovered within the core barrel is examined in the field and in the laboratory and the cores are stored in partitioned boxes. The core recovery is the length of material recovered and is expressed as a percentage of the total distance penetrated.

LABORATORY TESTING

A. Atterberg Limits - Plasticity Tests (ASTM D-423, D-424)

Atterberg Limits determine the soil's plasticity characteristics. The soil's Plasticity Index (PI) is representative of this characteristic and is the difference between the Liquid Limit (LL) and the Plastic Limit (PL). The LL is the moisture content at which the soil will flow as a heavy viscous fluid. The PL is the moisture content at which the soil begins to lose its plasticity. The test results are presented on the boring logs adjacent to the appropriate sampling information.

B. Particle Size Analysis (ASTM D-421, D-422)

Grain size analysis tests are performed to determine the particle size and distribution of the samples tested. The grain size distribution of the soils coarser than the Standard Number 200 sieve was determined by passing the sample through a standard set of nested sieves. The results are given on the gradation sheets in the appendix.

C. Moisture Content (ASTM D-2216)

The moisture content of soil is the ratio, expressed as a percentage, of the weight of water in a given soil mass to the weight of solid particles. It is determined by measuring the wet and oven dry weights of a soil sample. This test procedure is outlined by ASTM Designation D-2216. The test results are presented on the boring logs.

D. Unconfined Compression Test (ASTM D-2166)

The unconfined compressive strength of soil is determined by placing a section of an undisturbed sample into a loading frame and applying an axial load until the sample fails in shear. The test results are presented on the boring logs adjacent to the appropriate sampling information or on separate sheets.

E. California Bearing Ratio (CBR) (ASTM D-1883)

The CBR test is performed by compacting soil in a six inch diameter mold at the desired density, soaking the sample for four days under a surcharge load approximating the pavement weight and then testing the soil in punching shear. A two inch diameter piston is forced into the soil to determine the resistance to penetration. The CBR is the ratio of the actual load required to produce 0.1 inches of penetration to that producing the same penetration in a standard crushed stone.

F. Swell Test (ASTM D-3788, Modified)

The well test is performed by confining a one inch thick specimen in a 2-1/2 inch diameter stainless steel ring and loading the specimen to the approximate overburden pressure. The test specimen is then inundated with distilled water and allowed to swell for forty-eight hours. The volumetric swell is measured as a percentage of the total volume and is converted mathematically to linear swell.

G. Compaction Test (ASTM D-698 or ASTM D-1557)

The compaction test is performed by compacting soil in a steel mold at varying moisture contents. Either three or five layers are compacted using a hammer weight and number of blows per layer which vary with the different test procedures. The Standard Proctor (ASTM D-698) is used for cohesive soils and the Modified Proctor (ASTM D-1557) is used for granular soils. The TEX-113-E method is applicable to both soil types with the procedure varying with the soil's plasticity. The data is plotted and the maximum unit weight and optimum moisture content determined. The test results are given in the appendix with a notation of the test method used.

RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF COMPACTED FILL

1. General

The soils engineer shall be the owners representative to control the placement of compacted fill. The soils engineer shall approve the subgrade preparation, the fill materials, the method of placement and compaction, and shall give written approval of the completed fill.

2. Preparation of Existing Ground

All topsoil, plants and other organic material shall be removed. The exposed surface shall be scarified, moistened if necessary, and compacted in the manner specified for subsequent layers of fill.

3. Fill Material

Fill shall have a liquid limit of 37 or less and a Plasticity Index of less than 18. The fill shall contain no organic or other perishable material, and no stones larger than six (6) inches. Fill material shall be approved by the soils engineer.

4. Placing Fill

Fill materials shall be placed in horizontal layers not exceeding eight (8) inches thickness after compaction. Successive loads of material shall be dumped so as to secure even distribution, avoiding the formation of layers or lenses of dissimilar materials. The contractor shall route his hauling equipment to distribute travel evenly over the fill area.

5. Compaction of Fill

a. *Moisture Control*: The moisture content of the fill material shall be distributed uniformly throughout each layer of the material. The allowable range of moisture content during compaction shall be within plus two (+2) and minus two (-2) percentage points of the optimum moisture content. The contractor may be directed to add necessary moisture to the material either in the borrow area or upon the fill surface or to dry the material, as directed by the soils engineer. The drying of cohesive soils between lifts to moisture contents less than seventy percent (70%) of optimum before the placement of subsequent lifts shall be avoided or the fill reworked at the proper moisture content.

b. *Compaction*: The material in each layer shall be compacted to obtain proper densities. Compaction by the hauling equipment alone will not be considered sufficient. Structural fills, including pavement subgrade, subbase and base, shall be compacted to densities equivalent to the percentages of the Standard Proctor (ASTM D-698) or the Modified Proctor (ASTM D-1557) maximum dry density listed in Table I. The Texas Department of Highways and Public Transportation Method TEX-113-E compaction test, which varies the compactive effort with soil type, may be substituted for the Standard or Modified Proctor methods and the percentages listed in Table I used.

TABLE I		
AREA	PERCENT COMPACTION	
	Fine Grained Soils ASTM D-698 Standard Proctor	Coarse Grained Soils ASTM D-1557 Modified Proctor
Within five (5) feet of building lines, under footings, floor slabs, slab-on-grade foundations and structures attached to buildings (i.e walls, patios, steps), abutment backfill	95	95+
More than five (5) feet beyond building lines, under walks, and fill areas to be landscaped	90	90
Pavement subgrade and subbase, including lime treated soils	95	95+
Flexible Base	N/A	98

Soils classified as coarse grained soils are those with more than fifty (50) percent, by weight, retained on the No. 200 Standard Sieve and with plasticity indices of less than 4.

6. Compaction Testing

Field density tests for the determination of the compaction of the fill shall be performed by a qualified testing laboratory in accordance with recognized procedures for making such tests. A representative number of tests shall be made in each compacted lift at locations selected by the soils engineer or his representative. For general structural and paving fills, we suggest one test per 3,000 square feet per lift with a minimum of three tests per lift.

DRILLED AND POURED CONCRETE PILES

1. GENERAL REQUIREMENTS

A. Scope of Work: The work covered by these specifications consists of furnishing all labor, equipment and materials for the placing of 14" minimum diameter AUGERCAST Piles in foundations composed of augerable materials.

B. Experience: The drilling contractor shall have an experienced superintendent in charge of the work and shall have in his employ an engineer specializing in soil mechanics and foundations, who is available for consultation as may be required for any unusual conditions encountered in the field.

C. Procedure:

1. Augercast Piles shall be made by the following method:

   A continuous-flight hollow-shaft auger shall be rotated in the ground to the specified pile depth. High-strength mortar shall be pumped as the auger is withdrawn to fill the hole, preventing hole collapse and to cause the lateral penetration of the mortar into soft or porous zones of the surrounding soil. Reinforcement shall be placed while the mortar is still fluid.

   Adjacent piles shall not be placed closer than 3.5' center to center until mortar in piles has set for 24 hours.

E. Inspection: The owner shall employ at his expense an approved testing laboratory to inspect and approve the work under this Section of the Specification. No piles shall be completed until approved by the Laboratory.

1. Records shall be kept daily by the laboratory and will contain the following information regarding piles placed:

   a. Number of the pile (location shown on plans).
   b. Depth of the pile (cut-off elevation to pile tip).
   c. Diameter of pile.
   d. Volume of concrete material placed in each pile.
   e. Pressure used in placing pile.
   f. Estimated penetration into rock.
   g. Weather conditions.
   h. Any unusual conditions encountered.

MATERIALS AND PROCEDURE

B. Augering Equipment: The hole through which the high-strength mortar is pumped during the placement of the pile shall be located at the bottom of the auger head below the bar containing the cutting teeth.

The auger flighting shall be continuous from the auger head to the top of the auger with no gaps.

The AUGERCAST Piling leads should be prevented from twisting by a stabilizing arm.

Auger shall be a minimum diameter of 14".

The mortar mix shall be tested by making two sets of 2" x 2" cubes for each day during which AUGERCAST Piles are placed. A set of cubes shall consist of 2 cubes to be tested at 7 days and 2 cubes to be tested at 28 days. Test cubes shall be made and tested in accordance with ASTM C109, with the exception that the mortar should be restrained from expansion by a top plate. Test shall be a part of testing laboratory duties.

E. Mixing and Pumping of High-Strength Cement Mortar: Only approved pumping and continuous mixing equipment shall be used in the preparation and handling of the mortar. All oil or other rust inhibitors shall be removed from mixing drums and pressure mortar pumps.

The mortar pump shall be a positive displacement piston type pump capable of developing pressures at the pump up to 350 psi.

The minimum volume of mortar placed in the hole shall at least equal the volume of the augered hole. All materials shall be recirculated through the pump.

A minimum of two pressure gauges shall be used. One gauge shall be placed at pump and one gauge shall be placed in plain view of Auger Operator.

Lift of Auger when placing mortar shall not exceed 6" at a time and should be continuous as possible.

G. Pile Tops: Where the pile cut-off is near the surface or above the bottom of the excavation, metal sleeves of the proper diameter shall be placed around the pile tops.

Soil Classification

(a) *Scope and Application* - (1) *Scope.* This appendix describes a method of classifying soil and rock deposits based on site and environmental conditions, and on the structure and composition of the earth deposits. The appendix contains definitions, sets forth requirements, and describes acceptable visual and manual tests for classifying soils.

(2) *Application.* This appendix applies when a sloping or benching system is designed in accordance with the requirements set forth in § 1926.652(b)(2) as a method of protection for employees from cave-ins. This appendix also applies when timber shoring for excavations is designed as a method of protection from cave-ins in accordance with appendix C in subpart P of part 1926 and when aluminum hydraulic shoring is designed in accordance with appendix D. This appendix also applies if other protective systems are designed and selected for use from data prepared in accordance with the requirements set forth in §1926.652(c), and the use of the data is predicated on the use of the soil classification system set forth in this appendix.

(b) *Definitions.* The definitions and examples given below are based on, in whole or in part, the following: American Society for Testing Materials (ASTM) Standards D653-85 and D2488; The Unified Soil Classification System, The U.S. department of Agriculture (USDA) Textural Classification Scheme; and the National Bureau of Standards Report BSS-121.

Cemented soil means a soil in which the particles are held together by a chemical agent, such as calcium carbonate, such that a hand-size sample cannot be crushed into powder of individual particles by finger pressure.

Cohesive soil means clay (fine grained soil), or soil with a high clay content, which has cohesive strength. Cohesive soil does no crumble, can be excavated with vertical sideslopes, and is plastic when moist. Cohesive soil is hard to breakup when dry, and exhibits significant cohesion when submerged. Cohesive soils include clayey silt, sandy clay, silty clay, clay and organic clay.

Dry soil means soil that does not exhibit visible signs of moisture content.

Fissured means a soil material that has a tendency to break along definite planes of fracture with little resistance, or a material that exhibits open cracks, such as tension cracks, in an open surface.

Granular soil means gravel, sand or silt (coarse grained soil) with little or no clay content. Granular soil has no cohesive strength. Some granular soils exhibit apparent cohesion. Granular soil cannot be molded when wet and crumbles easily when dry.

Layered system means two or more distinctly different soil or rock types arranged in layers. Micaceous seams or weakened planes in rock or shale are considered layered.

Moist soil means a condition in which a soil looks and feels damp. Moist cohesive soil can be easily shaped into a ball and rolled into small diameter threads before crumbling. Most granular soil that contains some cohesive material will exhibit signs of cohesion between particles.

Plastic means a property of a soil which allows the soils to be deformed or molded without cracking, or appreciable volume change. Saturated soil means a soil in which the voids are filled with water. Saturation does not require flow. Saturation, or near saturation, is necessary for the proper use of instruments, such as a pocket penetrometer or shear vane.

Soil classification system means, for the purpose of this subpart, a method of categorizing soil and rock deposits in a hierarchy of Stable Rock, Type A, Type B, and Type C, in descending order of stability.

Stable rock means natural, solid mineral matter that can be excavated with vertical sides and remain intact when exposed.

Submerged soil means soil that is under water or freely seeping.

Type A means cohesive soil with an unconfined compressive strength of 1.5 tons per square foot (tsf) (144 kPa) or greater. Examples of cohesive soils are clay, silty clay, sandy clay, clay loam and, in some cases, silty clay loam and sandy clay loam. Cemented soils, such as caliche and hardpan, are also considered Type A. However, no soil is Type A if:

(i) The soil is fissured; or

(ii) The soil is subject to vibration from heavy traffic, pile driving, or similar effects; or

(iii) The soil has been previously disturbed; or

(iv) The soil is part of a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or greater; or

(v) The material is subject to other factors that would require it to be classified as a less stable material.

Type B means:

(i) Cohesive soil with an unconfined compressive strength greater than 0.5 tsf (48 kPa) but less than 1.5 tsf (144 kPa); or

(ii) Granular cohesionless soils including angular gravel (similar to crushed rock) silt, silt loam, sandy loam, and, in some cases, silty clay loam and sandy clay loam.

(iii) Previously disturbed soils except those which would otherwise be classified as Type C soil.

(iv) Soil that meets the unconfined compressive strength or cementation requirements for Type A, but that is fissured or subject to vibration; or

(v) Dry rock that is not stable; or

(vi) Material that is part of a sloped, layered system where the layers dip into the excavation on a slope less steep that four horizontal to one vertical (4H:1V), but only if the material would otherwise be classified as Type B.

Type C means;

(i) Cohesive soil with an unconfined compressive strength of 0.5 tsf (48 kPa) or less; or

(ii) Granular soil, including gravel, sand and loamy sand; or

(iii) Submerged soil or soil from which water is freely seeping; or

(iv) Submerged rock that is not stable; or

(v) Material in a sloped, layered system where the layers dip into the excavation on a slope of four horizontal to one vertical (4H:1V) or steeper,

Unconfined compressive strength means the load per unit area at which the soil will fail in compression. It can be determined by laboratory testing or estimated in the field using a pocket penetrometer, by thumb penetration tests, and other methods.

Wet soil means soil that contains significantly more moisture than moist soil, but in such a range of values that cohesive materials will slump or begin to flow when vibrated. Granular material that would exhibit cohesive properties when moist will loose those cohesive properties when wet.

(c) *Requirements* - (1) *Classification of soil and rock deposits.* Each soil and rock deposit shall be classified by a competent person as Stable Rock, Type A, Type B, or Type C in accordance with the definitions set forth in paragraph (b) of this appendix.

(2) *Basis of Classification.* The classification of the deposits shall be made on the based on the results of at least one visual and at least one manual analysis. Such analyses shall be conducted by a competent person using tests described in paragraph (d) below, or in other recognized methods of soil classification and testing such as those adopted by the American Society of Testing Materials, or the U.S. Department of Agriculture textural classification system.

(3) *Visual and manual analyses.* The visual and manual analyses, such as those noted as being acceptable in paragraph (d) of this appendix, shall be designed and conducted to provide sufficient quantitative and qualitative information as may be necessary to identify properly the properties, factors, and conditions affecting the classification of the deposits.

(4) *Layered systems.* In a layered system, the system shall be classified in accordance with its weakest layer. However, each layer shall be classified individually where a more stable layer lies under a less stable layer.

(5) *Reclassification.* If, after classifying a deposit, the properties, factors, or conditions affecting its classification change in any way, the changes shall be evaluated by a competent person. The deposit shall be reclassified as necessary to reflect the changed circumstances.

(d) *Acceptable visual and manual tests* - (1) *Visual tests.* Visual analysis is conducted to determine qualitative information regarding the excavation site in general, the soil adjacent to the excavation, the soil forming the sides of the open excavation, and the soil taken as samples from the excavated material.

(i) *Visual tests.* Observe samples of soil that are excavated and soil in the sides of the excavation. Estimate the range of particle sizes and the relative amounts of the particle sizes. Soil that is primarily composed of fine-grained material is cohesive material. Soil composed primarily of coarse-grained sand or gravel is granular material.

(ii) Observe soil as it is excavated. Soil that remains in clumps when excavated is cohesive.

Soil that breaks up easily and does not stay in clumps is granular.

(iii) Observe the side of the opened excavation and the surface area adjacent to the excavation. Crack-like openings such as tension cracks could indicate fissured material. If chucks of soil spall off a vertical side, the soil could be fissured. Small spalls are evidence of moving ground and are indications of potentially hazardous situations.

(iv) Observe the area adjacent to the excavation and the excavation itself for evidence of existing utility and other underground structures, and to identify previously disturbed soil.

(v) Observe the opened side of the excavation to identify layered systems. Examine layered systems to identify if the layers slope toward the excavation. Estimate the degree of slope of the layers.

(vi) Observe the area adjacent to the excavation and the sides of the open excavation for evidence of surface water, water seeping from the sides of the excavation, or the location of the level of the water table.

(vii) Observe the area adjacent to the excavation and the area within the excavation for sources of vibration that may affect the stability of the excavation face.

(2) *Manual tests*. Manual analysis of soil samples is conducted to determine the quantitative as well as qualitative properties of soil and to provide more information in order to classify soil properly.

(i) *Plasticity*. Mold a moist or wet sample of soil into a ball and attempt to roll it into threads as this as 1/8-inch in diameter. Cohesive materials can be successfully rolled into threads without crumbling. For example, if at least a two inch (50 mm) length of 1/8-inch thread can be held on one end without tearing, the soil is cohesive.

(ii) *Dry strength*. If the soil is dry and crumbles on its own or with moderate pressure into individual grains or fine powder, it is granular(any combination of gravel, sand, or silt). If the soil is dry and falls into clumps which do not break up into small clumps and which can only be broken with difficulty, it may be clay in any combination with gravel, sand or silt. If the dry soil breaks into clumps which do not break up into small clumps and which can only be broken with difficulty, and there is no visual indication that the soil is fissured, the soil may be considered unfissured.

(iii) *Thumb Penetration*. The thumb penetration test can be used to estimate the unconfined compressive strength of cohesive soils, (This test is based on the thumb penetration test described in American Society for Testing Materials (ASTM) Standard designation D2488 - "Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). Type A soils with an unconfined compressive strength of 1.5 tsf can be readily indented by the thumb; however, they can be penetrated by the thumb only with great effort. Type C soils with an unconfined compressive strength of 0.5 tsf can easily be penetrated several inches by the thumb, and can be molded by light finger pressure. This test should be conducted on an undisturbed soil sample, such as a large clump of spoil, as soon as practicable after excavation to keep to a minimum the effects of exposure to drying influences. If the excavation is later exposed to wetting influences (rain, flooding), the classification of the soil must be changed accordingly.

(iv) *Other strength tests*. Estimates of unconfined compressive strength of soils can also be obtained by use of a pocket penetrometer or by using a hand-operated shear vane.

(v) *Drying test*. The basic purpose of the drying test is to differentiate between cohesive material with fissures, unfissured cohesive material, and granular material. The procedure for the drying test involves drying a sample of soil that is approximately one inch thick (2.54 cm) and six inches (15.24 cm) in diameter until it is thoroughly dry:

(A) If the sample develops cracks as it dries, significant fissures are indicated.

(B) Samples that dry without cracking are to be broken by hand. If considerable force is necessary to break a sample, the soil has significant cohesive material content. The soil can be classified as unfissured cohesive material and the unconfined compressive strength should be determined.

(C) If a sample breaks easily by hand, it is either a fissured cohesive material or a granular material. To distinguish between the two, pulverize clumps of the sample by hand or by stepping on them. If the clumps do not pulverize easily, the material is cohesive with fissures. If they pulverize easily into very small fragments, the material is granular.

IMPORTANT INFORMATION
ABOUT YOUR
GEOTECHNICAL ENGINEERING REPORT

The following observations and suggestions are provides to help you better utilize your geotechnical engineering report and to reduce construction problems and delays related to the soil and groundwater conditions.

REPORT IS BASED UPON SPECIFIC SITE AND PROJECT

A geotechnical report is based on a subsurface exploration conducted on a specific site and planned using specific project information. The project information typically includes structure size and configuration, type of construction, and general location on the site. Limitations, such as existing buildings or utilities, specific foundation requirements for the structures, budget limitations, and the level of risk assumed by the client may affect the scope of the exploration.

Since the report applies to a specific structure and site, the geotechnical report should not be used in the following circumstances unless the geotechnical engineer has reveiewed the changes and concurs in the use of the report.

- When the nature of the proposed structure is changed, such as an office building instead of a warehouse or parking garage, or a refrigerated warehouse instead of one which is not refrigerated.
- When the size, configuration, or floor elevation is changed.
- When the location of the structure on the site is changed.
- When there is a change of ownership.

FINDINGS ARE PROFESSIONAL ESTIMATES

The actual subsurface conditions are determined only at the boring locations and only at the time the samples are taken. The information is extrapolated by the geotechnical engineer who then renders professional opinions regarding the characteristics of the subsurface materials, the behavior of the soils during construction, and appropriate foundation designs. No exploration, however complete, can be assured of sampling the entire range of soil conditions. The soils may vary between or beyond the borings and stratum transitions may be more gradual or more abrupt, and all the types of soil and rock existing on the site may not be found in the borings. The geotechnical engineer is often retained during construction to evaluate variances and recommend solutions to problems encountered on the site

SUBSURFACE CONDITIONS CAN CHANGE

Grading operations on or close to the site, floods, groundwater fluctuations, utility construction, and utility leaks are among the events which can change the subsurface conditions. The geotechnical engineer should be kept apprised of such events.

GEOTECHNICAL SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND PERSONS

A geotechnical report may have been made to evaluate foundation alternatives only, for preliminary site evaluation, or for other limited purposes. The exploration may also have been limited by the direction of the client, budget limitations, or the level of risk assumed by the client. Therefore, no one other than the original client should use the report for its intended purpose or other purposes without conferring with the geotechnical engineer.

GEOTECHNICAL REPORTS ARE SUBJECT TO MISINTERPRETATION

Geotechnical reports are based on the project information available at the time the report was made and the judgement and opinions of the geotechnical engineer. This specialized information is subject to misinterpretation by other design professionals, contractors and owners. The geotechnical engineer should be retained during the design process to interpret the recommendations and review the adequacy of the plans and specifications relative to geotechnical issues. The boring logs should not be separated from the geotechnical report but, rather, the entire report should be made available to the contractors and others needing this information.