Report of:

Subsurface Exploration
Bridge and MSE Retaining Walls
SR 823 Over Relocated Shumway Hollow Road
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, Ohio 43017

Ohio Department of Transportation
District 9

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DLZ Job No. 0121-3070.03

September 26, 2006

Prepared by

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

SR 823 OVER RELOCATED SHUMWAY HOLLOW ROAD

SCI-823-0.00 PORTSMOUTH BYPASS

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1.0 INTRODUCTION

This report includes the findings of evaluation of foundations and mechanically stabilized earth (MSE) retaining walls for the structure at the above-referenced location of the project. The findings included in this report pertain to the structure at the interchange of the proposed SR 823 and relocated Shumway Hollow Road only. The findings of evaluation of other structure will be submitted in separate documents.

The project consists in part of placing two structures for the proposed SR 823 over relocated Shumway Hollow Road. The two structures as planned, are one-span structures using MSE walls to hold back the roadway embankments and contain the abutments.

The purpose of this exploration was to 1) determine the subsurface conditions to the depths of the borings, 2) evaluate the engineering characteristics of the subsurface materials, and 3) provide information to assist in the design of the structure foundations, MSE walls, and the roadway embankments. The exploration presented in this report was performed essentially in accordance with DLZ Ohio, Inc.'s (DLZ) proposal for the project.

The geotechnical engineer has planned and supervised the performance of the geotechnical engineering services, considered the findings, and prepared this report in accordance with generally accepted geotechnical engineering practices. No other warranties, either expressed or implied, are made as to the professional advice included in this report.

2.0 GENERAL PROJECT INFORMATION

It is understood that the plan location of the bridge structure for the proposed SR 823 over Shumway Hollow Road has not changed from the approved location, as shown on the Plan and Profile drawing in Appendix I. It is understood that MSE walls will be placed at approximate stations 383+75 and 384+69 to contain the abutments and hold back the roadway embankment for the proposed SR 823. Furthermore, it is assumed that driven H-piles will be used to support the abutments of the proposed structure.

Based upon the cross-sections and profile information along the relocated Shumway Hollow Road, it is assumed that the maximum height of the embankment at station 384+69 (Forward Abutment) will be approximately 29.0 feet. This height is based upon the maximum difference between the proposed grade and the approximate existing grade along the relocated Shumway Hollow Road. See cross-section drawing in Appendix I.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concept is changed or differs from that assumed, DLZ should be informed of the changes so that recommendations and conclusions presented in this report may be revised as necessary.

3.0 FIELD EXPLORATION

The field exploration consisted in part of four final and three preliminary structural borings. Borings B-1 through B-4 were drilled for the final bridge plan, essentially consisting of proposed SR 823 passing over the relocated Shumway Hollow Road. The borings were drilled on June 13 and 14, 2006. Preliminary structural borings (TR-24 through TR-26) were drilled for a previous design configuration, where Shumway Hollow Road passed over the proposed SR 823. The preliminary borings were drilled on August 18 and 19, 2004. Due to the change in the design, borings B-1 through B-4 are considered most representative of soils in the area of the proposed structures. A boring plan is presented in Appendix I. Boring logs for borings TR-24 through TR-26, and B-1 through B-4 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were determined by representatives of DLZ. The surveyed locations and ground surface elevations of the borings were determined by representatives from Lockwood, Lanier, Mathias & Noland, Inc. (2LMN).

4.0 FINDINGS

4.1 Geology of the Site

The area of this structure is characterized by gently sloping to steeply sloping topography. The project area is located in the Shawnee-Mississippian Plateau of the unglaciated portion of the Appalachian Plateau Physiographic Region. The Shawnee-Mississippian Plateau is characterized by Devonian aged to Pennsylvanian aged rocks and contains residual colluvial, glacial, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Residual and colluvial soils are found on the ridge tops and the hillsides near the site. These soils are generally thin to moderately deep, covering moderate to steep slopes. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. In the area of the proposed structure, soils of the Minford Complex generally overlie a layer of sand and gravel which is directly above bedrock. In this area, the Minford Complex is characterized by clays of high plasticity and high moisture content. The clays encountered in the borings in this area are thinly laminated with silt and fine sand layers.

Bedrock within the structure area is primarily sandstone of the Logan Formation that is of Mississippian Age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to the west of the structures roughly above elevation 860. In the area of the structure, the bedrock depth varies from approximately 25 to 37 feet below the ground surface.

4.2 Subsurface Conditions

The following sections present the generalized subsurface conditions encountered by the borings. For more detailed information, refer to the boring logs presented in Appendix II. Laboratory test results are presented on the boring logs and also in Appendix III.

4.2.1 Soil Conditions

Boring B-1 generally encountered 5 inches of topsoil at the surface. Below the topsoil, very stiff silt (A-4b) was primarily encountered to a depth of 4.0 feet below ground surface. Below the depth of 4.0 feet, stiff to very stiff clay (A-7-6) was primarily encountered to a depth of approximately 21.5 feet below ground surface. Below the depth of 21.5 feet, loose fine sand (A-3) was primarily encountered to a depth of approximately 37.5 feet, the top of bedrock. Underlying the soil, the bedrock encountered was hard and slightly to moderately weathered sandstone to the completion depth of the boring, 42.5 feet.

4.2.2 Bedrock Conditions

In the area of the proposed structure, boring B-1 generally encountered medium hard to hard, slightly to moderately weathered sandstone at a depth of 42.5 feet below the ground surface. The bedrock encountered is generally moderately to highly fractured.

4.2.3 Groundwater Conditions

Seepage was encountered in all borings drilled for the structure. Seepage was first encountered from 6.0 to 34 feet below the ground surface. At the completion of drilling, the final water level, including drill water, varied from 8.4 to 31.0 feet. The water levels recorded prior to adding drill water were reported in borings B-3 and B-4. The water level was measured to be approximately 23 feet below the ground surface. For more information please refer to the Boring Logs presented in Appendix II.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 Bridge Foundation Recommendations

It is understood that pile foundations will be used to support the proposed bridge. Furthermore, it is understood that MSE walls would be used to construct the embankments and contain the abutments.

The borings for the proposed structure generally encountered up to 12 inches of topsoil at the surface. Underlying the topsoil, stiff to hard silt and clay (A-6a), clay (A-7-6), sandy silt (A-4a), silt (A-4b), loose to dense gravel with sand (A-1-b), and fine sand (A-3) were encountered to depths between 23.0 and 43.5 feet (between elevations 642.3 and 646.2), where bedrock was encountered.

Bedrock encountered at the proposed structure location composed primarily of medium hard to hard sandstone that was generally slightly fractured to intact. Recovery of the core samples ranged from 87 to 100 percent and the rock quality designation (RQD) values ranged from 42 to 90 percent with an average RQD of 65 percent.

It appears that driven H-piles will be the best-suited foundation type for support of the proposed structure. It is recommended that H-Piles be driven to refusal on bedrock. Furthermore, it is recommended that steel pile points be used to protect the pile while driving. If it is determined that other loading conditions, such as the lateral loading, are to be of concern, a rock socket may be designed upon actual loading conditions. A table summarizing the site conditions and foundation recommendations follows subsequently.

It has been determined that settlements will be significant at this location. Therefore, a waiting period after construction of the embankments and MSE walls will be required. To limit the drag down forces on the piles, a waiting period of approximately 234 days will be required after the completion of embankment construction. The recommended waiting period will allow the majority of the consolidation of the soft compressible soils to occur, and thus prevent consolidation from exerting a large down drag force on the piles. See Section 5.2.2 MSE Wall Evaluations and Recommendations for more information.

Summary of Foundation Recommendation

Boring Number	Structural Element	Existing Ground Surface Elevation (Feet)	Approximate Bearing Elevation (Feet)	Recommended Foundation Type	Allowable Bearing Capacity
B-1 B-2	Forward (north) Abutment	675.0-680.0	642*	HP 14x74 Steel Piles	95 tons
B-3 B-4	Rear (south) Abutment	668.7-672.3	642*	HP 14x74 Steel Piles	95 tons

* *Denotes Approximate Top of Rock Elevation.*

If the MSE embankment is constructed prior to driving piles, pile sleeves should be used to form a void in the MSE fill to permit pile installation.

5.2 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

5.2.1 MSE Walls: General Information

An MSE retaining wall essentially consists of good quality backfill material with layers of metal or plastic reinforcing that are attached to concrete facing panels. The MSE wall and associated backfill should be constructed in accordance with the specifications of the manufacturer of the MSE wall.

A global stability analysis and bearing capacity analysis were performed for the MSE walls at this bridge location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding, overturning, and settlement. At the time this report was prepared, it was understood that driven pile foundations would be used at this site to support the bridge abutments. If the foundation type should change, DLZ should be informed so that the analyses may be revised as necessary.

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability analyses are attached. Other external and internal stability analyses are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented below. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. However, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Soil Parameters Used in MSE Wall Stability Analyses
Relocated Shumway Hollow Road

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear and Forward Abutments) (Boring B-1)	Stiff Clay	120	2000	0	0	28

Due to similarities in the soils encountered at this location, the results of the analyses of the MSE wall at the forward abutment are considered representative of both walls at this site. In general, boring B-1 was found to have the most critical subsurface profile in the area of the proposed MSE walls.

5.2.2 MSE Wall Evaluations and Recommendations

The MSE walls at the rear and forward abutments are assumed to have a maximum height of approximately 29.0 feet. However, in the area of the most critical soil profile, boring B-1, the height of the MSE wall is approximately 27.6 feet. Based upon the stability and the AASHTO guidelines, the minimum required embedment depth for the proposed MSE wall is 3.0 feet below the finished grade. The required minimum length of the reinforcing straps was found to be 0.9 times the total height plus the embedment depth, or 28.8 feet.

The global stability of the MSE wall was analyzed at this location. The results yielded acceptable factors of safety for global stability (>1.3). See attached drawing illustrating the results of the global stability analyses. The wall was also analyzed for bearing capacity and stability (sliding and overturning). The calculations for stability yielded factors of safety above the minimum recommended values. The drained bearing capacity calculation also yielded an acceptable factor of safety. However, the factor of safety for the undrained bearing capacity was calculated to be 2.2, slightly below the required minimum value of 2.5. Consequently, additional analyses were undertaken to evaluate possible remedies to this low factor of safety for undrained bearing capacity. A five-foot undercut and replacement with compacted granular fill was considered. This analysis did not achieve the minimum recommended factor of safety.

UTEXAS3 was utilized to evaluate the bearing capacity of the MSE wall. UTEXAS3 is a computer program that can be used to evaluate several types of global stability failure modes. If the problem is modeled so the failure surface passes through or below the toe of the MSE wall volume, this analysis can be considered a global stability failure mode that is essentially a bearing capacity failure. Using this type of failure model for the MSE walls, the factor of safety for undrained bearing capacity of the full height wall was calculated to be 2.3 which is less that the required minimum value of 2.5. Therefore, additional analyses were performed to determine the maximum allowable staged construction height to achieve a minimum factor of safety for undrained bearing capacity. This analysis resulted in a maximum allowable staged height of 25 feet, with a factor of safety of 2.5. In addition, it was determined that a waiting period of approximately 44 days will be required before placing additional 4 feet of fill to complete the full height of the MSE walls. The waiting period will allow excess pore water pressures to dissipate to 11 feet above ground surface to accommodate the additional loading of the embankment fill while maintaining a minimum factor of safety of 2.5.

Due to the inherent variations of the subsurface conditions, the actual required waiting period may be shorter or longer than anticipated. It is recommended that piezometers be installed in the clay layer to monitor the excess pore water pressures that will develop during construction and ensure that a critical pore water pressure is not exceeded. Analyses have been performed to determine the critical pore water pressures. Based upon the results of the analyses, if the water level in the piezometer rises 11.0 feet above the existing ground surface, construction should halt immediately. Construction may continue after pore pressures in the clay layer have dissipated. The results of the critical pore pressure stability analyses are presented in Appendix IV.

The total maximum settlement of the MSE wall volumes at this location was estimated to be approximately 8 inches at the centerline of the wall. Of the total of 8 inches, approximately 1 inch will occur instantaneously in the sand and gravel layers. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankments. Differential settlement at this location was estimated to be approximately 0.3%. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1/100). Settlement calculations are presented in Appendix IV.

Time-rate of consolidation calculations have indicated that approximately 234 days will be required to achieve 90 percent consolidation of foundational soils without using wick drains or other methods. This calculation is based upon the coefficient of vertical consolidation. It should be noted that the clay layer was observed to be varved (containing thin silt and fine sand layers); however, without the use of wick drains, the horizontal drainage path to relieve pore pressures will be very long. The use of a higher coefficient of consolidation to model the horizontal drainage relies on the assumption that these layers are continuous throughout the embankment area. Due to variations in the soil profile under the proposed MSE wall area, this would not be a reasonable assumption. If the previously mentioned consolidation period is too long, the use of wick drains or other methods may be explored to accelerate the consolidation of foundation soils. These alternatives can be evaluated for this site upon request. Time-rate of consolidation calculations are presented in Appendix IV.

A table summarizing the MSE retaining wall parameters and results of analyses can be found on the following page.

MSE Retaining Wall Parameters and Analyses Results
Relocated Shumway Hollow Road (Rear and Forward Abutments)
Analysis Based on Boring B-1

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\phi' = 30°$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 28°(0.67) = 0.36 Use (μ)(0.67) =0.35 as a maximum value as per AASHTO, BDM,303.4.1.1
Allowable Bearing Capacity – Undrained Condition q_{all} = 4255 psf For MSE wall with minimum 28.8-foot long reinforcing
Allowable Bearing Capacity – Drained Condition q_{all} = 5,513 psf For MSE wall with minimum 28.8-foot long reinforcing
Global Stability Factor of Safety – Undrained Condition = 2.3 Factor of Safety – Drained Condition = 1.7 Factor of Safety – Seismic Condition = 1.6
Estimated Settlement of MSE volume Total settlement = 8 inches Differential settlement = 0.3% ≤ 1/100
Maximum Full Height of MSE Wall = 29.0 feet Minimum Embedment Depth = 3.0 feet Minimum Length of Reinforcement for External Stability = 28.8 feet Maximum Staged Construction Height = 25 feet Maximum Pore Pressure* = 11.0 feet above ground surface

**Maximum pore pressure as measured in piezometer installed in clay layer. See results of analyses for more information.*

5.3 Groundwater Considerations

Water seepage and final water levels were generally encountered at depths below 20.0 feet in the borings. Excavation for the pier foundation is expected to be limited to four feet or less. The abutment foundations will be on top of the MSE fill embankment. Consequently, little if any seepage is anticipated for the foundation excavations. However, the Contractor should be prepared to deal with unexpected seepage and precipitation that enters any excavations.

6.0 CLOSING REMARKS

We appreciate having the opportunity to be of service to you on this project. Please do not hesitate to call if you have any questions concerning our report.

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Arthur (Pete) Nix, P.E.
Senior Geotechnical Engineer

SJR

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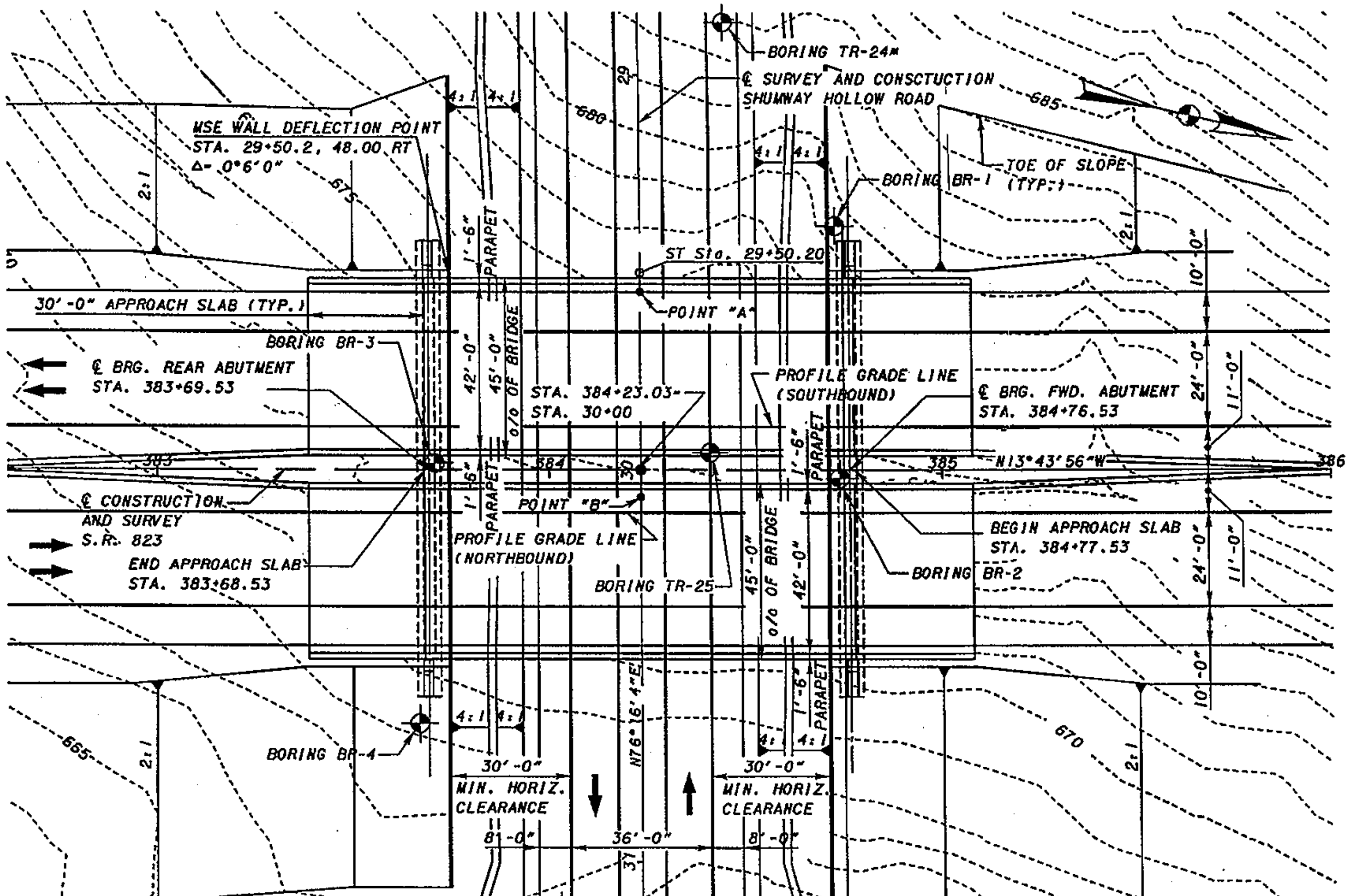
APPENDIX I

Structure Plan and Profile Drawing – 11”x17”

Cross Section Drawing at Structure Location – 8.5”x11”

Boring Plan - 11”x17”



PLAN

DENOTES BORING LOCATION

* BORING TR-24 & TR-26 NOT SHOWN TO SCALE

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-24	384+43.87	147.87' LT.
TR-25	384+41.03	4.35' LT.
TR-26	384+04.27	126.85 RT.
B-1	384+73.00	61.69' LT.
B-2	384+74.00	2.23' RT.
B-3	383+71.00	1.68' LT.
B-4	383+67.00	64.92' RT.

BENCHMARK 1	BENCHMARK 2
(TO BE PROVIDED LATER)	(TO BE PROVIDED LATER)

TRAFFIC DATA

S.R. 823

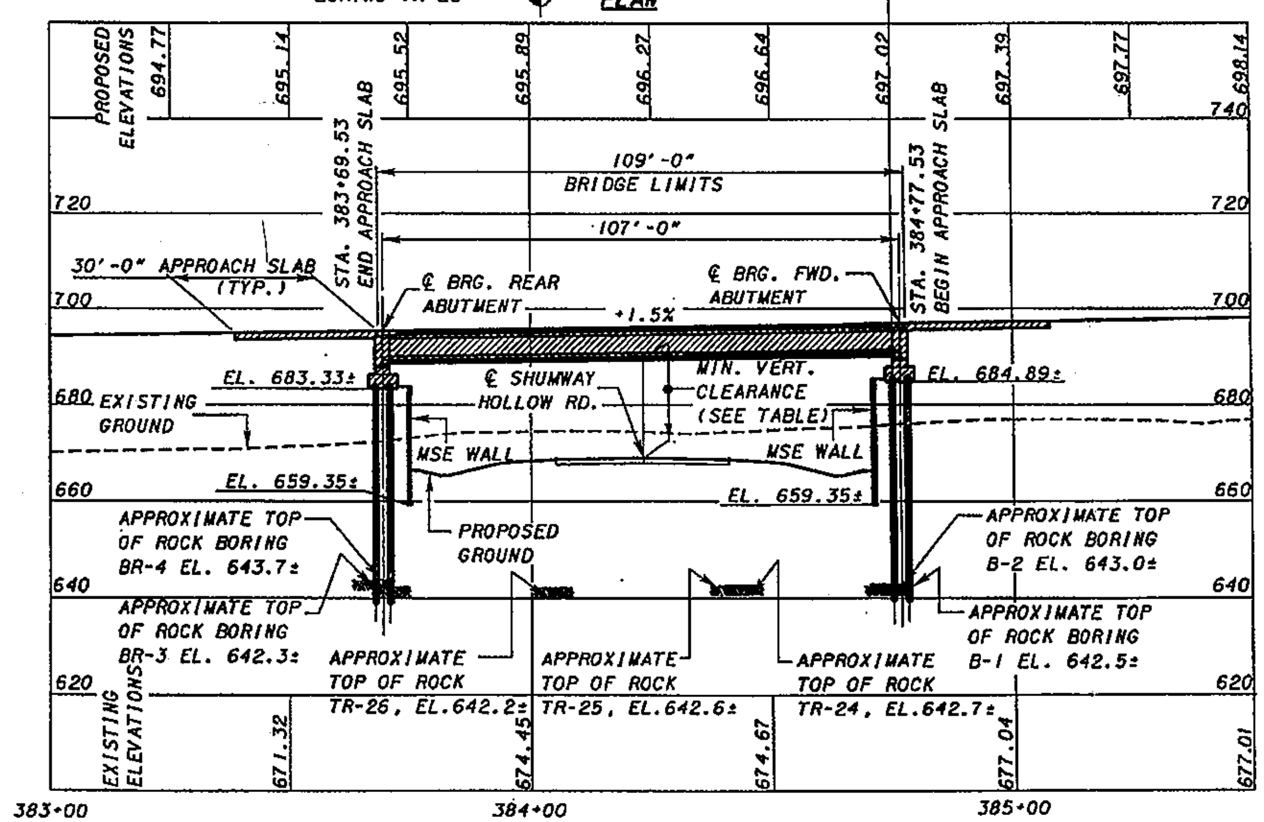
CURRENT YEAR ADT (2010) = 19,800
DESIGN YEAR ADT (2030) = 26,000
CURRENT YEAR ADTT (2010) =2,772
DESIGN YEAR ADTT (2030) = 3,640

PROPOSED STRUCTURE

TYPE: SINGLE SPAN, 72" TYPE 4 (MOD.) PRESTRESSED CONCRETE I-BEAM WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY SEMI-INTEGRAL ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS

SPANS: 107'-0" c/c BEARINGS
ROADWAY: 42' TOE TO TOE OF PARAPETS
LOADING: HS-25 AND ALTERTNATE MILITARY LOADING FWS=60 PSF
SKEW: NONE
CROWN: 0.016 FT/FT
ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)
LATITUDE:
LONGITUDE:

TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	19.13'	20.32'
PREFERRED	17.0'	17.0'



ELEVATIONS ALONG PROFILE GRADE LINE S.R. 823 LEFT BRIDGE

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

FOUNDATION DATA:

ALL NEW PILES SHALL BE HP 14x73 PILES AND HAVE A MAXIMUM CAPACITY OF 90 TONS PER PILE.

DESIGN AGENCY: Systems

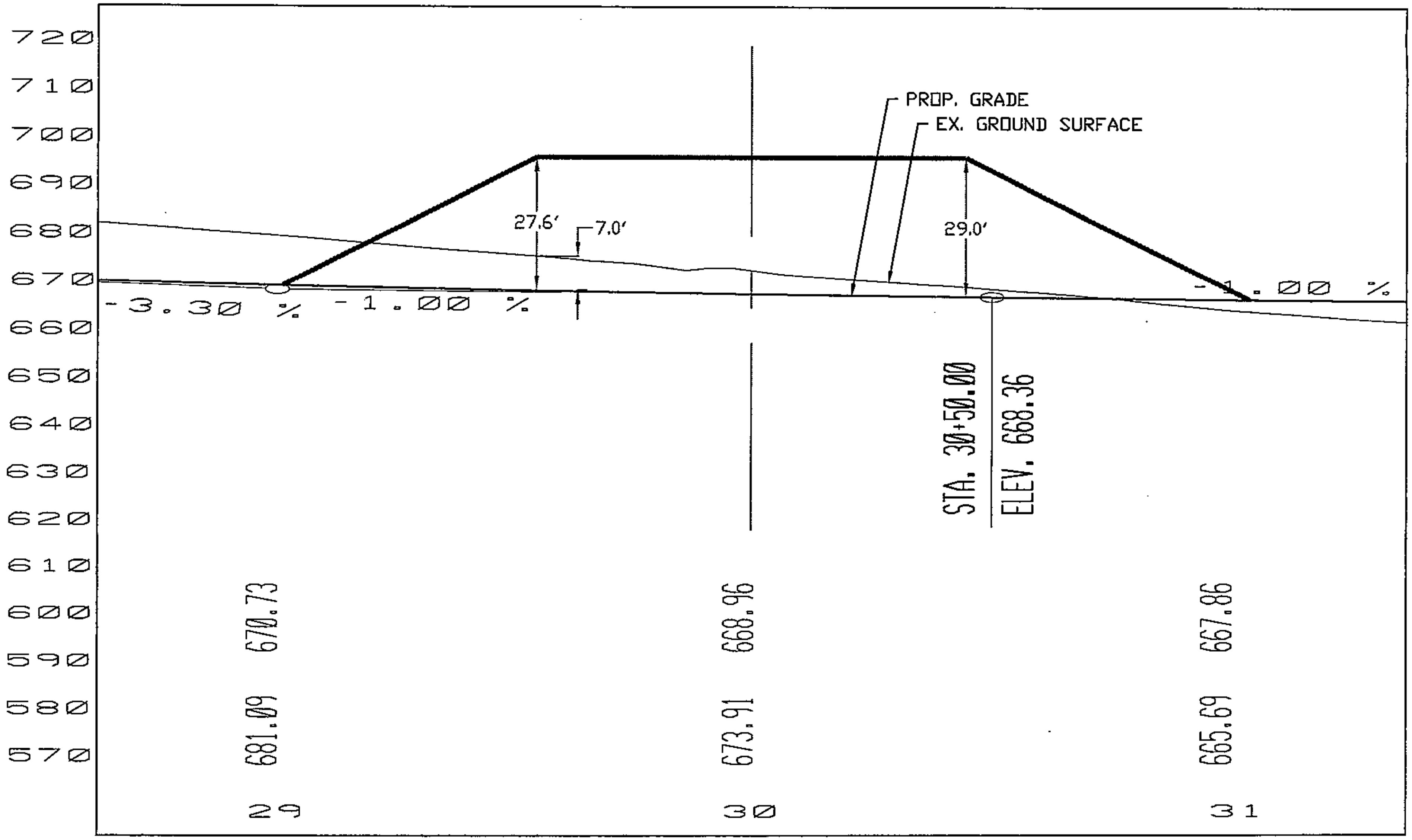
DESIGNED PJP | CHECKED MSL | DRAWN CAS | REVIEWED JRC | DATE 8/8/06 | STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 383+68.53
STA. 384+77.53

PRELIMINARY SITE PLAN - ALTERNATIVE 2
BRIDGE NO. SCI-823-XXXX
S.R. 823 OVER SHUMWAY HOLLOW ROAD (TR-234)

SCI-823-0.00
PID 19415

1/2



823 OVER SHUMWAY HOLLOW ROAD
PROFILE ALONG SHUMWAY HOLLOW ROAD
FORWARD ABUTMENT - VIEW LOOKING NORTH

1" = 30' HOR AND VERT

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 07/28/06

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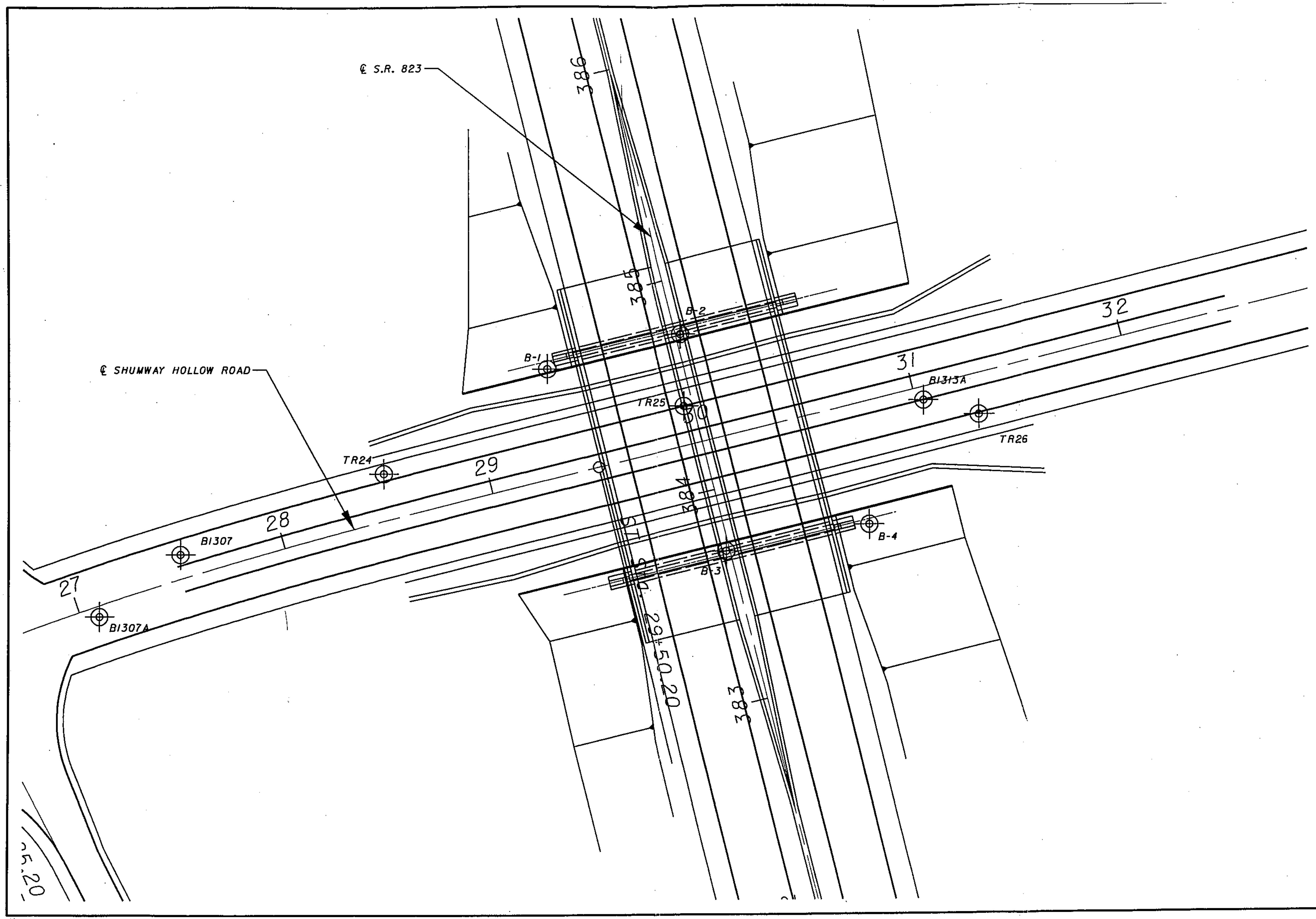


HORIZONTAL SCALE IN FEET
0 10 20 40

CALCULATED

CHECKED

SR 823 OVER SHUMWAY HOLLOW
STRUCTURE BORING PLAN

SCI-823

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Seven (7) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

Drilling and sampling were conducted in accordance with procedures generally recognized and accepted as standardized methods of investigation of subsurface conditions concerning geotechnical engineering considerations. Borings were drilled with either a truck-mounted or ATV-mounted drill rig.

Drive split-barrel sampling was performed in 1.5 foot increments at intervals not exceeding 5 feet. In the event the sampler encountered resistance to penetration of 6 inches or less after 50 blows of the drop hammer, the sampling increment was discontinued. Standard penetration data were recorded and one or more representative samples were preserved from each sampling increment.

In borings where rock was cored, NXM or NQ size diamond coring tools were used.

In the laboratory all samples were visually classified by a geotechnical engineer. Moisture contents of representative fine-grained soil samples were determined. A limited number of samples, considered representative of foundation materials present, were selected for performance of grain-size analyses and plasticity characteristics tests. The results of these tests are shown on the boring logs.

The boring logs included in the Appendix have been prepared on the basis of the field record of drilling and sampling, and the results of the laboratory examination and testing of samples. Stratification lines on the boring logs indicating changes in soil stratigraphy represent depths of changes approximated by the driller, by sampling effort and recovery, and by laboratory test results. Actual depths to changes may differ somewhat from the estimated depths, or transitions may occur gradually and not be sharply defined. The boring logs presented in this report therefore contain both factual and interpretative information and are not an exact copy of the field log.

Although it is considered that the borings have disclosed information generally representative of site conditions, it should be expected that between borings conditions may occur which are not precisely represented by any one of the borings. Soil deposition processes and natural geologic forces are such that soil and rock types and conditions may change in short vertical intervals and horizontal distances.

Soil/rock samples will be stored at our laboratory for a period of six months. After this period of time, they will be discarded, unless notified to the contrary by the client.

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

1. Depth (in feet) – refers to distance below the ground surface.

2. Elevation (in feet) – is referenced to mean sea level, unless otherwise noted.

3. Standard Penetration (N) – the number of blows required to drive a 2-inch O.D., 1-3/8 inch I.D., split-barrel sampler, using a 140-pound hammer with a 30-inch free fall. The blows are recorded in 6-inch drive increments. Standard penetration resistance is determined from the total number of blows required for one foot of penetration by summing the second and third 6-inch increments of an 18-inch drive.

   50/n – indicates number of blows (50) to drive a split-barrel sampler a certain number of inches (n) other than the normal 6-inch increment.

4. The length of the sampler drive is indicated graphically by horizontal lines across the "Standard Penetration" and "Recovery" columns.

5. Sample recovery from each drive is indicated numerically in the column headed "Recovery".

6. The drive sample location is designated by the heavy vertical bar in the "Sample No., Drive" column.

7. The length of hydraulically pressed "Undisturbed" samples is indicated graphically by horizontal lines across the "Press" column.

8. Sample numbers are designated consecutively, increasing in depth.

9. Soil Description

   a. The following terms are used to describe the relative compactness and consistency of soils:

   Granular Soils – Compactness

Term	Blows/Foot Standard Penetration
Very Loose	0 – 4
Loose	4 – 10
Medium Dense	10 – 30
Dense	30 – 50
Very Dense	over 50

   Cohesive Soils – Consistency

Term	Unconfined Compression tons/sq.ft.	Blows/Foot Standard Penetration	Hand Manipulation
Very Soft	less than 0.25	below 2	Easily penetrated by fist
Soft	0.25 – 0.50	2 – 4	Easily penetrated by thumb
Medium Stiff	0.50 – 1.0	4 – 8	Penetrated by thumb with moderate pressure
Stiff	1.0 – 2.0	8 – 15	Readily indented by thumb but not penetrated
Very Stiff	2.0 – 4.0	15 – 30	Readily indented by thumb nail
Hard	over 4.0	over 30	Indented with difficulty by thumb nail

   b. Color – If a soil is a uniform color throughout, the term is single, modified by such adjective as light and dark. If the predominant color is shaded by a secondary color, the secondary color precedes the primary color. If two major and distinct colors are swirled throughout the soil, the colors are modified by the term "mottled".

   c. Texture is based on the Ohio Department of Transportation Classification System. Soil particle size definitions are as follows:

Description	Size	Description	Size
Boulders	Larger than 8"	Sand – Coarse	2.0 mm to 0.42 mm
Cobbles	8" to 3"	– Fine	0.42 mm to 0.074 mm
Gravel – Coarse	3" to ¾"	Silt	0.074 mm to 0.005 mm
– Fine	¾" to 2.0 mm	Clay	smaller than 0.005 mm

d. The main soil component is listed first. The minor components are listed in order of decreasing percentage of particle size.

e. Modifiers to main soil descriptions are indicated as a percentage by weight of particle sizes.

trace	0 to 10%
little	10 to 20%
some	20 to 35%
"and"	35 to 50%

f. Moisture content of **cohesionless soils** (sands and gravels) is described as follows:

Term	Relative Moisture or Appearance
Dry	No moisture present
Damp	Internal moisture, but none to little surface moisture
Moist	Free water on surface
Wet	Voids filled with free water

g. The moisture content of **cohesive soils** (silts and clays) is expressed relative to plastic properties.

Term	Relative Moisture or Appearance
Dry	Powdery
Damp	Moisture content slightly below plastic limit
Moist	Moisture content above plastic limit but below liquid limit
Wet	Moisture content above liquid limit

10. Rock Hardness and Rock Quality Designation

a. The following terms are used to describe the relative hardness of the **bedrock**.

Term	Description
Very Soft	Permits denting by moderate pressure of the fingers. Resembles hard soil but has rock structure. (Crushes under pressure of fingers and/or thumb)
Soft	Resists denting by fingers, but can be abraded and pierced to shallow depth by a pencil point. (Crushes under pressure of pressed hammer)
Medium Hard	Resists pencil point, but can be scratched with a knife blade. (Breaks easily under single hammer blow, but with crumbly edges.)
Hard	Can be deformed or broken by light to moderate hammer blows. (Breaks under one or two strong hammer blow, but with resistant sharp edges.)
Very Hard	Can be broken only by heavy and in some rocks repeated hammer blows.

b. Rock Quality Designation, RQD – This value is expressed in percent and is an indirect measure of rock soundness. It is obtained by summing the total length of all core pieces which are at least four inches long, and then dividing this sum by the total length of the core run.

11. Gradation – when tests are performed, the percentage of each particle size is listed in the appropriate column (defined in Item 9c).

12. When a test is performed to determine the natural moisture content, liquid limit moisture content, or plastic limit moisture content, the moisture content is indicated graphically.

13. The standard penetration (N) value in blows per foot is indicated graphically.

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

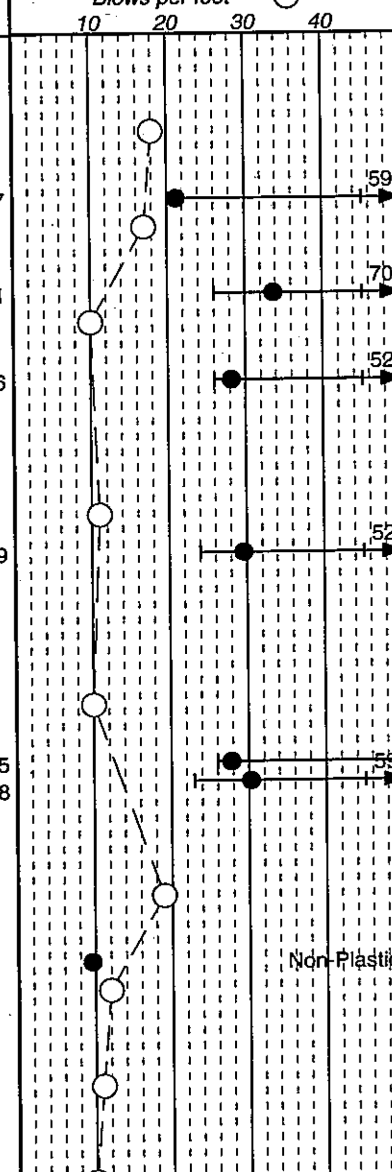
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1 | Location: Sta. 384+73.4, 61.7 ft. LT of SR 823 CL | Date Drilled: 06/13/06

WATER OBSERVATIONS: Water seepage at: 34.0' - 35.0'
Water level at completion: 31.0' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	680.0												
0.4	679.6						Topsoil - 5"						
		5 7 11	12	1		2.5	Very stiff brown SILT (A-4b), trace to little clay, trace fine sand; damp.						
4.0	676.0	6 6 11	13	2		4.5+	Very stiff to hard brown CLAY (A-7-6), little to some silt, trace fine to coarse sand, trace gravel; moist.	2	0	--	1	30	67
5													
		3 4 6	16	3		2.0		5	0	--	0	4	91
					P-1	3.25		0	0	--	1	13	86
10													
		4 5 6	18	5		2.5							
					P-2	2.75		0	0	--	2	19	79
15													
		4 4 6	18	7		1.5	@ 16.0'-17.5', stiff.						
					P-3	2.5		0	0	--	1	24	75
								0	0	--	1	21	78
20													
21.5	658.5	5 9 10	16	9			Medium dense light brown COARSE AND FINE SAND (A-3a), little to some silty clay; dry to damp.						
		4 5 7	17	10				0	0	--	80	20	
25							@ 25.0', wet.						
		4 5 6	15	11			@ 28.0', moist.						
28.5	651.5	5 5 5	13	12			Loose to medium dense brown GRAVEL WITH SAND (A-1-b)						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1 | Location: Sta. 384+73.4, 61.7 ft. LT of SR 823 CL | Date Drilled: 06/13/06

WATER OBSERVATIONS: Water seepage at: 34.0' - 35.0'
Water level at completion: 31.0' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	650.0						Loose to medium dense brown GRAVEL WITH SAND (A-1-b), trace silty clay; moist.							
35		5 6 5	14	13										
							@ 37.5', auger refusal.							
37.5	642.5	Core 60"	Rec 54"	RQD 62%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, pyritic (halos), massived bedding, slightly fractured, contains few argillaceous laminations.							
42.5	637.5						Bottom of Boring - 42.5'							

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○
10 20 30 40

Depth scale: 30, 35, 37.5, 40, 42.5, 45, 50, 55, 60

FILE: 0121-3070-03 [9/27/2006 9:25 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

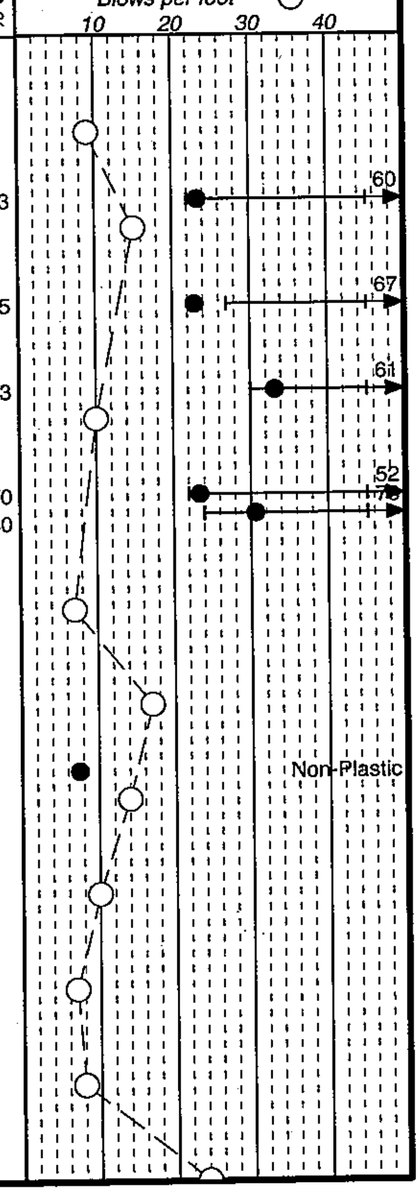
LOG OF: Boring B-2 | Location: Sta. 384+73.7, 2.2 ft. RT of SR 823 CL | Date Drilled: 06/13/06 to 06/14/06

WATER OBSERVATIONS: Water seepage at: 26.0' - 30.0'
Water level at completion: 28.0' (prior to coring)
20.0' (inside hollowster augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	675.0												
0.3	674.7						Topsoil - 4"						
		3 4 5	15	1		3.5	Very stiff brown SILT AND CLAY (A-6a), trace fine sand; damp.						
3.0	672.0						Very stiff brown CLAY (A-7-6), trace to some silt, trace fine to coarse sand, trace gravel; damp to moist.						
		6 7 8	14	2		3.5		2	1	--	3	11	83
5													
					P-1	4.0		1	0	--	0	14	85
		3 4 6	17	4		2.75		4	2	--	2	9	83
10													
					P-2	2.75		0	2	--	4	24	70
						2.50		5	1	--	2	12	80
		3 3 4	18	6		2.0							
15													
16.2	658.8	5 7 10	18	7			Loose to medium dense brown COARSE AND FINE SAND (A-3a), little silty clay; damp to wet.						
		5 7 7	17	8				0	0	--	85	15	
20													
		6 6 4	16	9									
		4 4 3	13	10									
25													
		6 4 4	15	11									
30		6 13 11	15	12									

GRADATION

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



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Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-2 | Location: Sta. 384+73.7, 2.2 ft. RT of SR 823 CL | Date Drilled: 06/13/06 to 06/14/06

WATER OBSERVATIONS: Water seepage at: 26.0' - 30.0'
Water level at completion: 28.0' (prior to coring)
20.0' (inside hollowster augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	645.0						@ 32.0', auger refusal.							
32.0–37.0	643.0–638.0	Core 60"	Rec 57"	RQD 52%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, laminated to thinly bedded, moderately fractured. @ 32.0'-34.5', highly fractured.							
37.0	638.0						Bottom of Boring - 37.0'							

STANDARD PENETRATION (N): Natural Moisture Content, % - ●; PL ├──┤ LL; Blows per foot - ○; scale 10, 20, 30, 40

Depth scale: 30, 35, 40, 45, 50, 55, 60

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

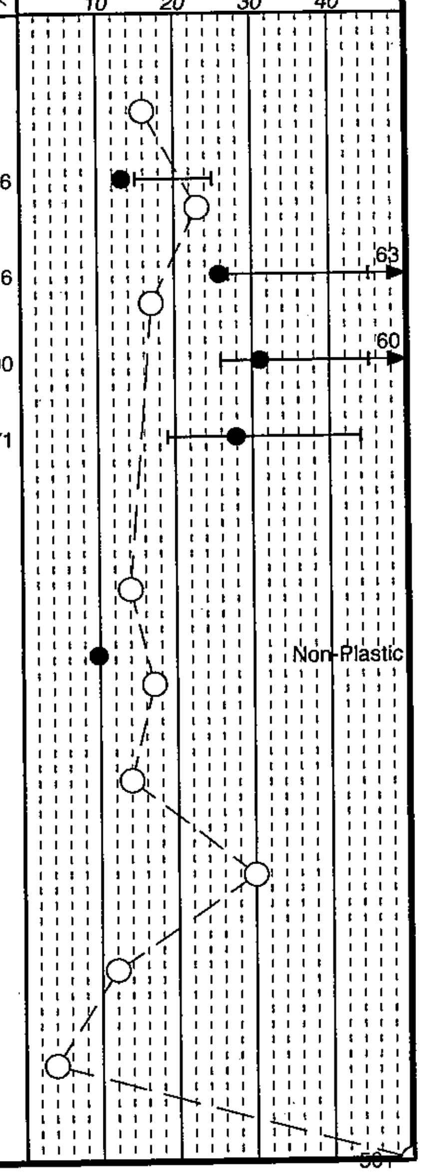
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-3 | *Location:* Sta. 383+71.2, 1.7 ft. LT of SR 823 CL | *Date Drilled:* 06/14/06

WATER OBSERVATIONS: Water seepage at: 23.0' - 28.0'
Water level at completion: 23.0' (prior to coring)
16.5' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	672.3												
0.4	671.9						Topsoil - 5"						
		6 6 10	13	1		2.5	Very stiff brown SANDY SILT (A-4a), "and" clay, trace gravel; damp.						
		8 12 11	14	2		4.0		2	5	--	9	48	36
4.5	667.8						Stiff to very stiff brown CLAY (A-7-6), trace fine to coarse sand; damp to moist.						
		5 7 10	10	3		3.5		0	1	--	2	11	86
					P-1	1.75		0	0	--	1	9	90
					P-2	2.25		0	0	--	3	26	71
14.0	658.3	4 6 8	16	6			Medium dense to dense brown COARSE AND FINE SAND (A-3a), little silty clay, trace coarse sand; damp to moist.						
		7 8 9	18	7				0	1	--	80	19	
		6 7 7	16	8									
		9 12 18	16	9									
		6 6 6	15	10									
		2 3 1	12	11									
30		8 21 50/2	12	12			@ 30.0', auger refusal.						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



FILE: 0121-3070-03 [9/27/2006 9:25 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-3 | *Location:* Sta. 383+71.2, 1.7 ft. LT of SR 823 CL | *Date Drilled:* 06/14/06

WATER OBSERVATIONS: Water seepage at: 23.0' - 28.0'
Water level at completion: 23.0' (prior to coring)
16.5' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ (10 20 30 40)
30.0	642.3	Core 60"	Rec 56"	RQD 63%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, laminated to thinly bedded, moderately fractured.							
35.0	637.3						Bottom of Boring - 35.0'							
40														
45														
50														
55														
60														

FILE: 0121-3070-03 [9/27/2006 9:25 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

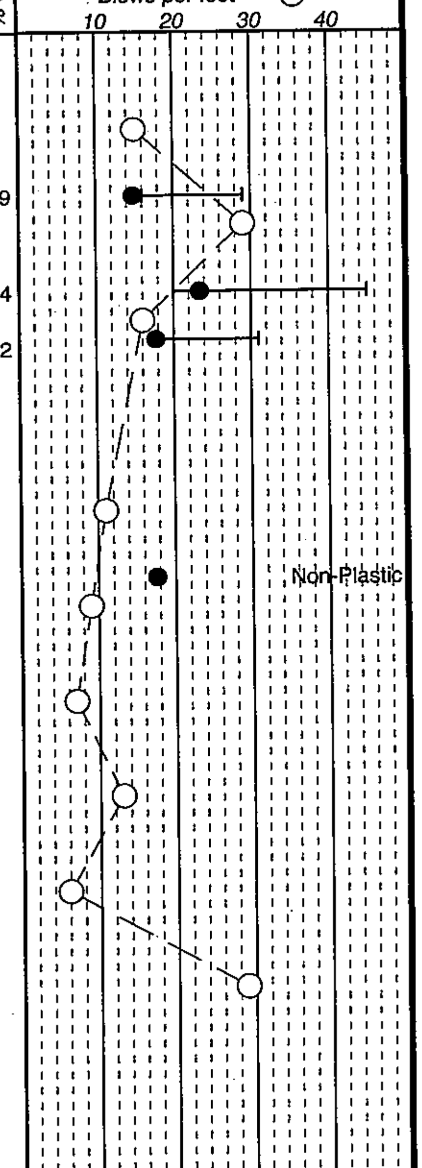
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-4 | Location: Sta. 383+66.6, 64.9 ft. RT of SR 823 CL | Date Drilled: 06/14/06

WATER OBSERVATIONS: Water seepage at: 18.0' - 24.0'
Water level at completion: 22.5' (prior to coring)
15.0' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	668.7												
0.3	668.4						Topsoil - 4"						
		5 7 8	12	1		2.5	Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp.						
		9 14 15	17	2		4.5+		3	6	--	8	44	39
5.0	663.7						Hard brown CLAY (A-7-6), some silt, trace fine to coarse sand, trace gravel; moist.						
		6 7 9	15	3		4.5+		3	2	--	4	27	64
7.5	661.2						Hard brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; damp.	2	5	--	8	43	42
8.2	660.5				P-1	2.25	Medium dense brown FINE SAND (A-3), trace silty clay; damp.	0	8	--	89	3	
10													
		4 6 5	16	5									
13.5	655.2	4 5 4	18	6			Loose to medium dense brown COARSE AND FINE SAND (A-3a), little silty clay; moist to wet.	0	0	--	83	17	
15													
		3 4 3	14	7									
		4 6 7	12	8									
20													
		3 3 3	17	9									
22.5	646.2						Medium dense brown GRAVEL WITH SAND AND SILT (A-2-4), little to some silt; moist.						
		4 8 21	18	10									
25													
25.5	643.2	Core 60"	Rec 52"	RQD 62%	R-1		@ 25.5', auger refusal. Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly weathered, laminated to thinly bedded, moderately fractured.						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



FILE: 0121-3070-03 [9/27/2006 8:19 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-4 | Location: Sta. 383+66.6, 64.9 ft. RT of SR 823 CL | Date Drilled: 06/14/06

WATER OBSERVATIONS: Water seepage at: 18.0' - 24.0'
Water level at completion: 22.5' (prior to coring)
15.0' (inside hollowstem augers, includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	638.7													
30.5	638.2						Bottom of Boring - 30.5'							
35														
40														
45														
50														
55														
60														

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

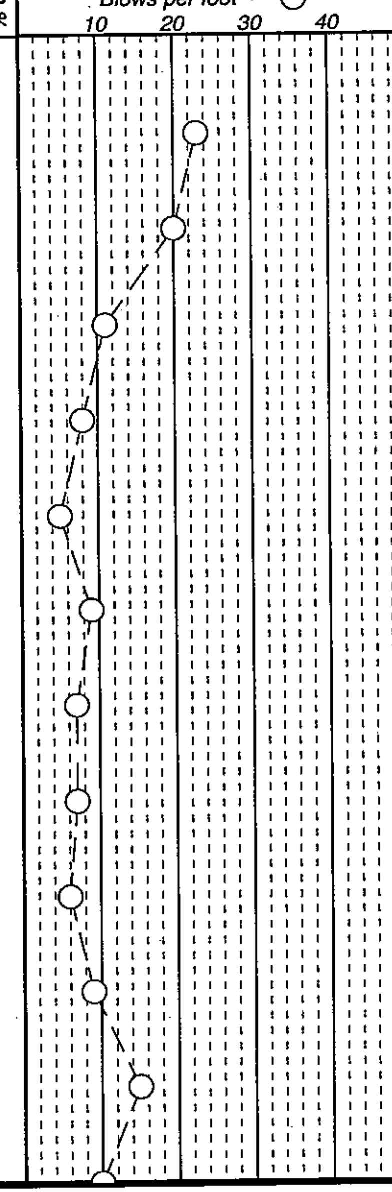
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-24 | Location: Sta. 384+43.9 147.1 ft. LT of SR 823 CL | Date Drilled: 8/20/04 to 8/23/04

WATER OBSERVATIONS: Water seepage at: 6.0'
Water level at completion: 29.8' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	686.2						Topsoil - 12"						
1.0	685.2	6 10 13	18	1		4.5+	Hard brown SILT AND CLAY (A-6a), little fine to coarse sand; damp.						
		13 10 10	18	2		4.5+							
6.0	680.2	3 3 8	12	3		2.75	Stiff to very stiff brown CLAY (A-7-6), little fine to coarse sand, little silt; varved; moist.						
		2 3 5	18	4		2.0							
		2 2 3	18	5		2.0							
		2 4 5	18	6		2.25							
		2 3 4	18	7		1.25							
		1 3 4	18	8		3.75							
		2 3 3	18	9		1.0							
		3 4 5	18	10		1.5							
27.0	659.2	4 3 12	18	11		1.75	Medium dense brown FINE SAND (A-3), trace gravel; damp.						
30		5 5 5	18	12									

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○



DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-24 | Location: Sta. 384+43.9 147.1 ft. LT of SR 823 CL | Date Drilled: 8/20/04 to 8/23/04

WATER OBSERVATIONS: Water seepage at: 6.0'
Water level at completion: 29.8' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
30	656.2						Medium dense brown FINE SAND (A-3), trace gravel; damp.						
34.0	652.2	4 2 2	18	13		0.5	Soft gray SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; moist.						
37.0	649.2	10 17 22	18	14			Severely weathered gray SANDSTONE, argillaceous. @ 43.0', augers encountered difficult drilling.						
43.5	642.7	Core 120"	Rec 118"	RQD 77%	R-1		Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, moderately to highly fractured. @ 44.8' to 44.9',45.2',45.4',47.0' contains argillaceous laminations and fractures. @ 47.0', slightly weathered, unfractured to slightly fractured.						
53.5	632.7						Bottom of Boring - 53.5'						

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○
10 20 30 40

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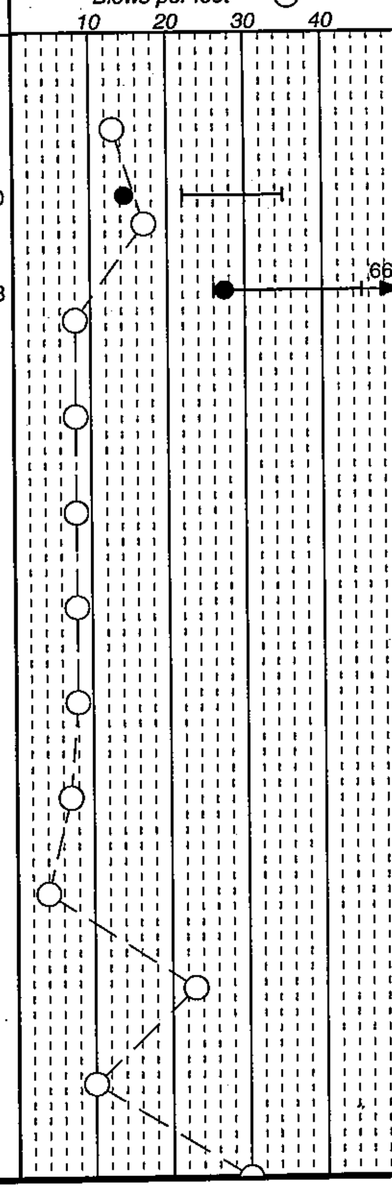
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-25 | Location: Sta. 384+41.0, 4.4 ft. LT of SR 823 CL | Date Drilled: 8/19/04 to 8/20/04

WATER OBSERVATIONS: Water seepage at: 16.0', 21.0'
Water level at completion: 16.4' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	674.6												
0.5	674.1						Topsoil - 6"						
		4 7 6	18	1		4.0	Hard brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; damp.						
		6 8 9	18	2		4.5		4	6	--	13	37	40
5.5	669.1						Stiff to very stiff brown CLAY (A-7-6), little silt, trace fine sand; damp to moist.						
		2 3 5	18	3		2.0		0	0	--	1	11	88
		2 3 5	18	4		1.25							
		2 4 4	18	5		1.75							
		3 3 5	18	6		2.5							
		2 3 5	18	7		2.25							
18.0	656.6						Loose to medium dense brown FINE SAND (A-3), trace silt; damp to wet.						
		4 3 4	18	8									
		1 1 3	18	9									
		9 11 12	18	10									
		7 3 7	18	11									
28.0	646.6						Dense brown GRAVEL WITH SAND (A-1-b); contains sandstone fragments; moist.						
		2 14 16	18	12									
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



FILE: 0121-3070-03 [9/27/2006 8:54 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-25 | Location: Sta. 384+41.0, 4.4 ft. LT of SR 823 CL | Date Drilled: 8/19/04 to 8/20/04

WATER OBSERVATIONS: Water seepage at: 16.0', 21.0'
Water level at completion: 16.4' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30.0	644.6						Severely weathered brown and gray SANDSTONE.							
		27 50/5	6	13										50+
32.0	642.6	Core 48"	Rec 46"	RQD 42%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, micaceous, argillaceous, massively bedded, slightly fractured. @ 32.0'-37.0', highly fractured.							
		Core 72"	Rec 72"	RQD 100%	R-2									
42.0	632.6						Bottom of Boring - 42.0'							

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○
10 20 30 40

Depth scale: 30, 35, 40, 45, 50, 55, 60

FILE: 0121-3070-03 [9/27/2006 8:54 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

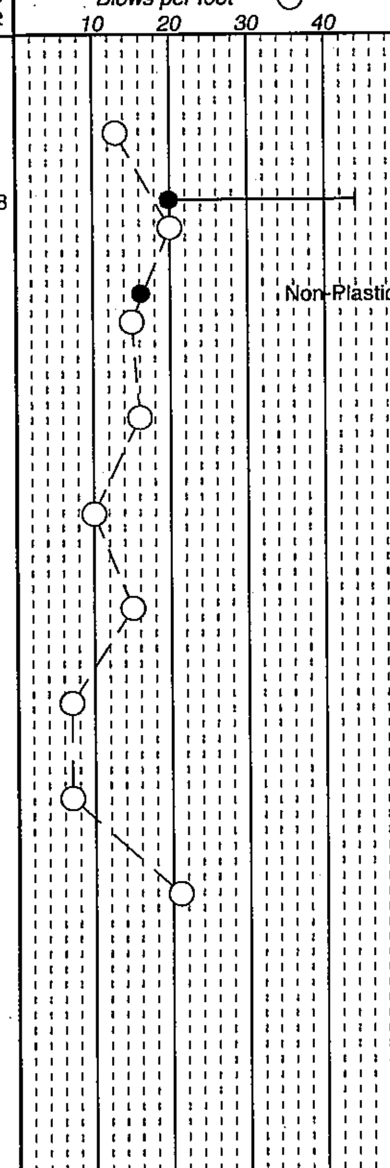
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-26 | Location: Sta. 384+04.3, 126.8 ft. RT of SR 823 CL | Date Drilled: 8/19/04

WATER OBSERVATIONS: Water seepage at: 8.5'
Water level at completion: 8.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	665.2						Topsoil - 12"						
1.0	664.2	5 5 8	18	1		4.5	Hard brown CLAY (A-7-6), "and" fine to coarse sand, trace gravel, little silt; damp to moist.						
		8 9 11	18	2		4.5+		0	2	--	38	12	48
5.5	659.7	7 7 8	18	3			Loose to medium dense brown COARSE AND FINE SAND (A-3a), little silty clay; moist to wet.	0	0	--	81	19	
		4 9 7	18	4									
		3 4 6	18	5									
		4 5 10	18	6									
		5 4 3	18	7			@ 16.0', wet.						
		2 3 4	18	8									
20.5	644.7	4 8 13	18	9			Medium dense gray GRAVEL WITH SAND (A-1-b); contains sandstone fragments; moist.						
23.0	642.2	Core 120"	Rec 111"	RQD 73%	R-1		Hard gray SANDSTONE; very fine to fine grained, argillaceous, micaceous, slightly to moderately weathered, massively bedded, slightly fractured. @ 23.0'-25.5', moderately fractured. @ 23.1', 23.5', thin clay seams.						
30													

STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ├──┤ LL
Blows per foot - ○



FILE: 0121-3070-03 [9/27/2006 9:25 AM]

DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-26 | Location: Sta. 384+04.3, 126.8 ft. RT of SR 823 CL | Date Drilled: 8/19/04

WATER OBSERVATIONS: Water seepage at: 8.5'
Water level at completion: 8.5' (includes drilling water)

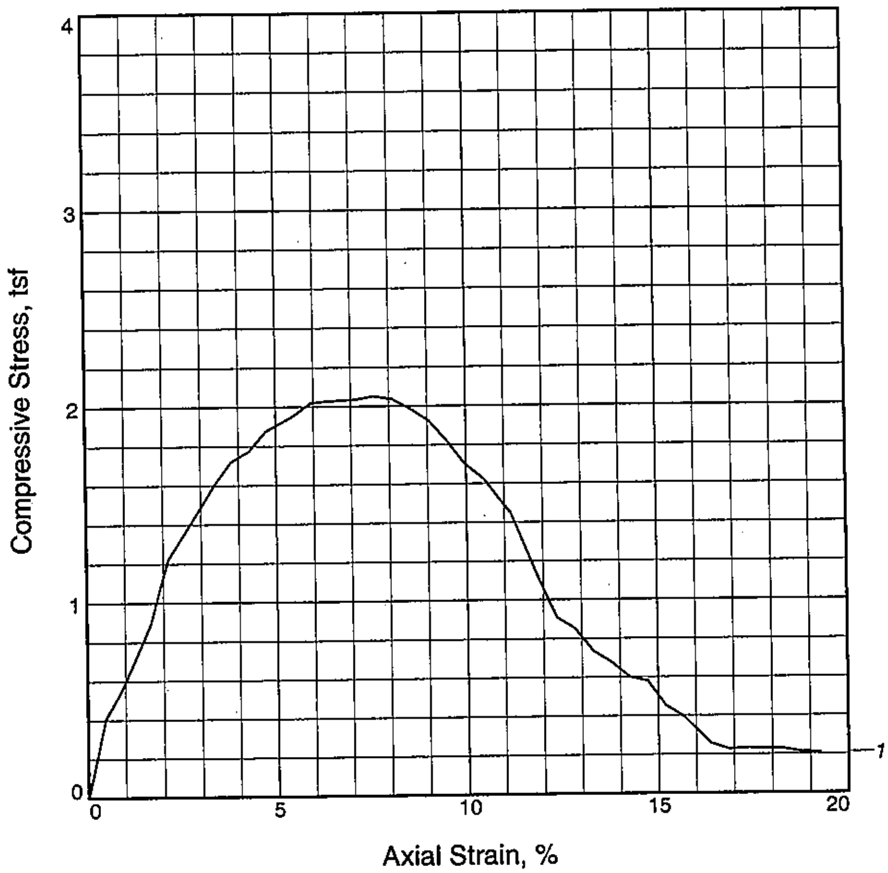
Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30.0	635.2						Hard gray SANDSTONE; very fine to fine grained, argillaceous, micaceous, slightly to moderately weathered, massively bedded, slightly fractured.							
33.0	632.2						Bottom of Boring - 33.0'							

Depth scale: 35, 40, 45, 50, 55, 60

FILE: 0121-3070-03 [9/27/2006 9:25 AM]

APPENDIX III
Laboratory Test Results

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.051			
Undrained shear strength, tsf	1.026			
Failure strain,	7.6			
Strain rate, in./min.	0.06			
Water content, %	29.2			
Wet density, pcf	122.2			
Dry density, pcf	94.6			
Saturation, %	98.6			
Void ratio	0.8151			
Specimen diameter, in.	2.83			
Specimen height, in.	4.21			
Height/diameter ratio	1.49			

Description: Fat clay

LL = 52	**PL** = 26	**PI** = 26	**Assumed GS=** 2.75	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Figure _______

Client: TranSystems, Inc.

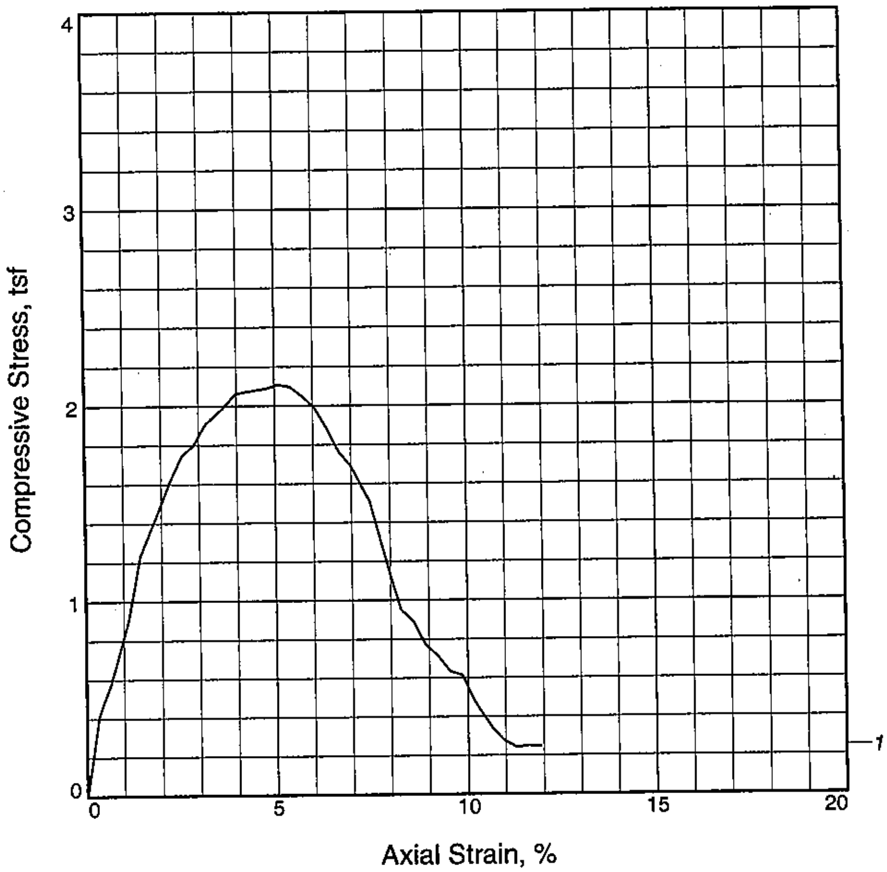
Project: SCI-823-0.00

Source of Sample: B-1 **Depth:** 8.0

Sample Number: P-1

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.107			
Undrained shear strength, tsf	1.053			
Failure strain,	5.1			
Strain rate, in./min.	0.06			
Water content, %	29.2			
Wet density, pcf	81.8			
Dry density, pcf	63.3			
Saturation, %	47.0			
Void ratio	1.7102			
Specimen diameter, in.	2.83			
Specimen height, in.	6.29			
Height/diameter ratio	2.22			

Description:

LL = | **PL =** | **PI =** | **Assumed GS=** 2.75 | **Type:** 3" Press Tube

Project No.: 0121-3070.03

Date:

Remarks:

Figure _______

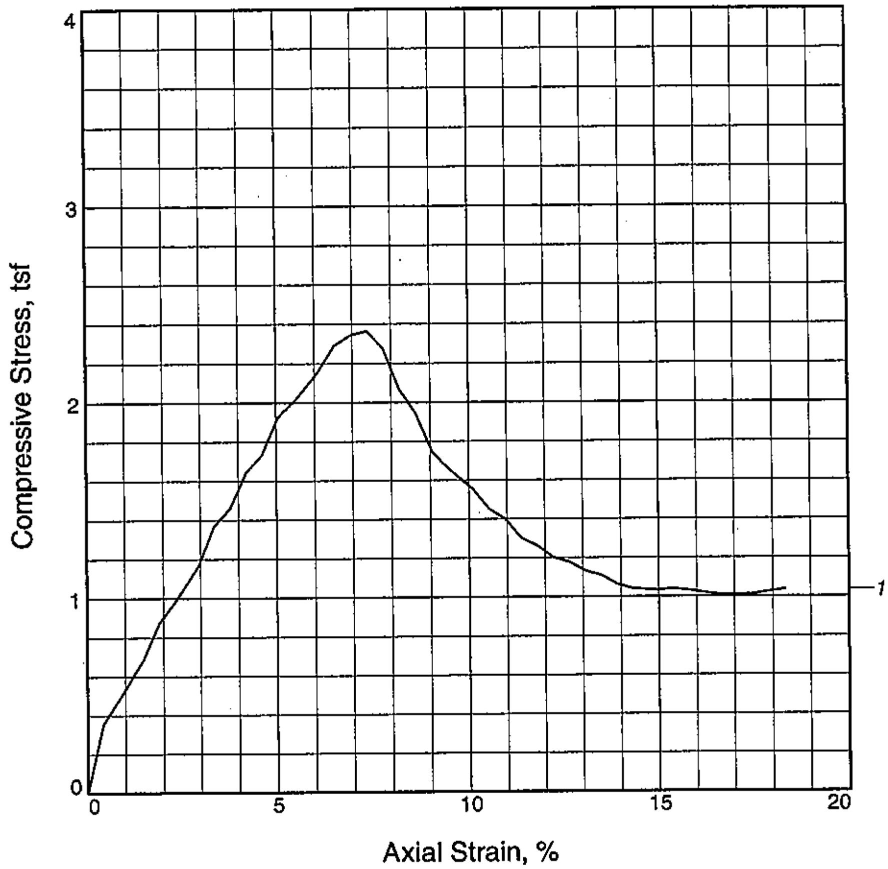
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-1 **Depth:** 8.0

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.365			
Undrained shear strength, tsf	1.182			
Failure strain,	7.4			
Strain rate, in./min.	0.06			
Water content, %	31.3			
Wet density, pcf	121.2			
Dry density, pcf	92.3			
Saturation, %	100.2			
Void ratio	0.8602			
Specimen diameter, in.	2.83			
Specimen height, in.	4.75			
Height/diameter ratio	1.68			

Description: Fat clay

LL = 52	**PL** = 24	**PI** = 28	**Assumed GS=** 2.75	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

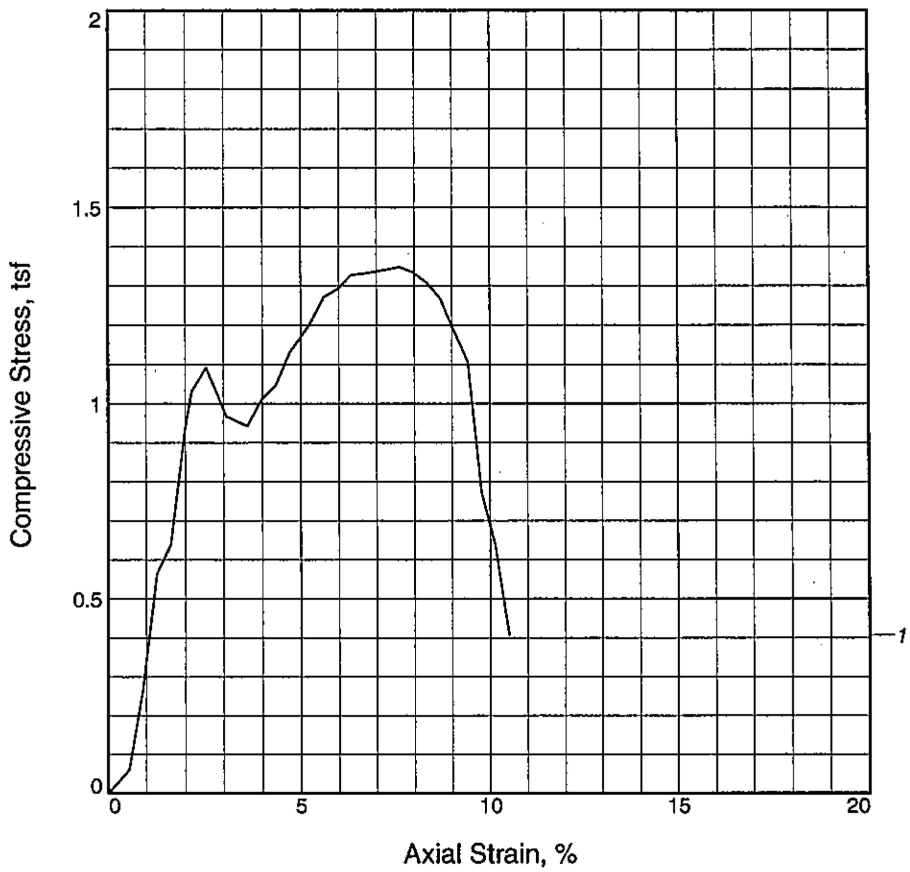
Source of Sample: B-1 **Depth:** 12.5

Sample Number: P-2

DLZ

Figure _______

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.348			
Undrained shear strength, tsf	0.674			
Failure strain,	7.6			
Strain rate, in./min.	0.06			
Water content, %	29.0			
Wet density, pcf	120.6			
Dry density, pcf	93.5			
Saturation, %	95.4			
Void ratio	0.8358			
Specimen diameter, in.	2.85			
Specimen height, in.	5.52			
Height/diameter ratio	1.94			

Description:

LL =	PL =	PI =	Assumed GS= 2.75	Type: 3" Press Tube

Project No.: 0121-3070.03

Date:

Remarks:

Figure ______

Client: TranSystems, Inc.

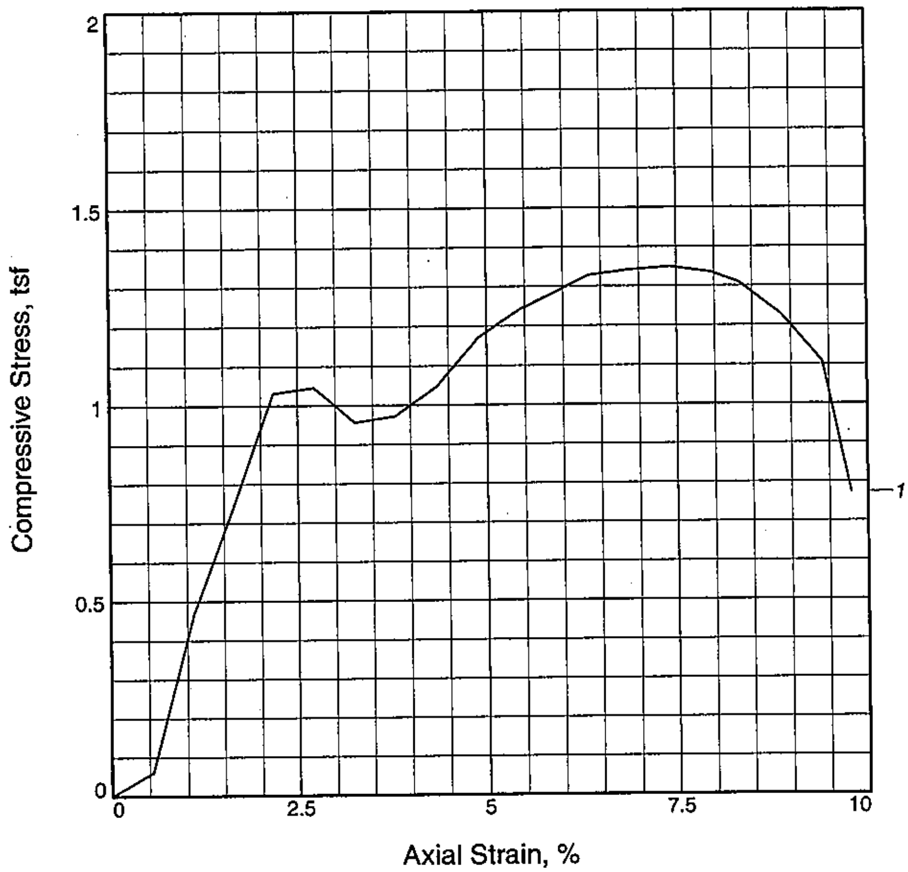
Project: SCI-823-0.00

Source of Sample: B-1 **Depth:** 19.5

Sample Number: P-3c

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.349			
Undrained shear strength, tsf	0.674			
Failure strain,	7.4			
Strain rate, in./min.	0.06			
Water content, %	29.0			
Wet density, pcf	120.6			
Dry density, pcf	93.5			
Saturation, %	97.6			
Void ratio	0.8024			
Specimen diameter, in.	2.85			
Specimen height, in.	5.52			
Height/diameter ratio	1.94			

Description:

LL = 59	**PL** = 23	**PI** = 36	**Assumed GS=** 2.70	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

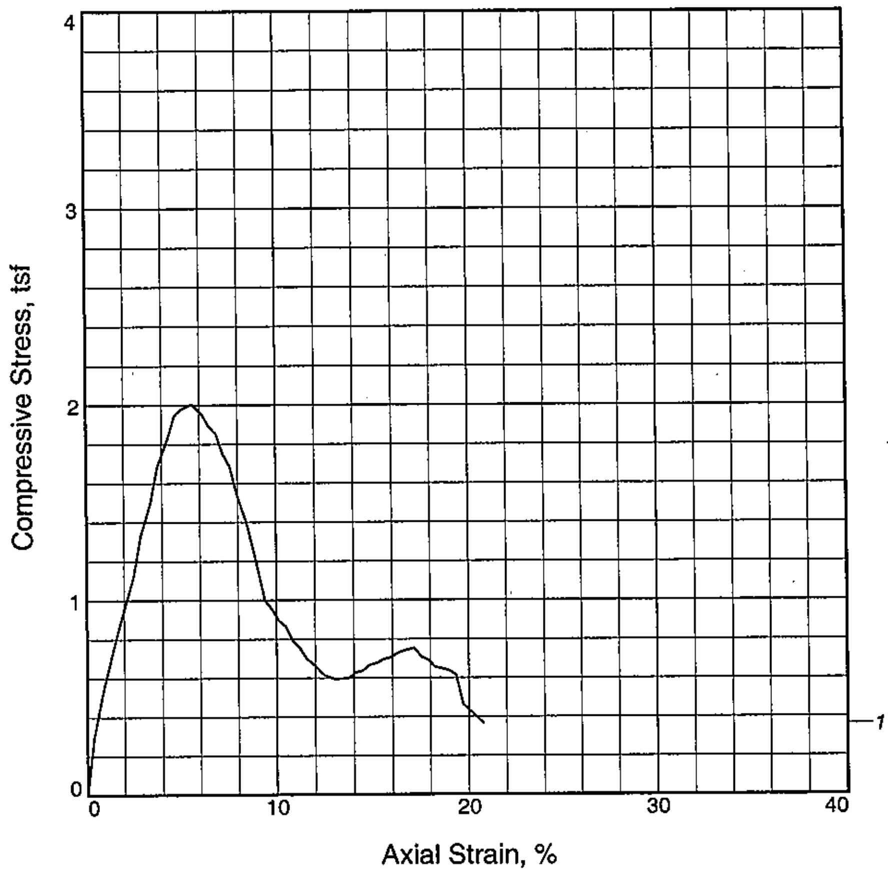
Source of Sample: B-1 **Depth:** 19.5

Sample Number: P-3C

DLZ

Figure ________

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.005			
Undrained shear strength, tsf	1.003			
Failure strain,	5.6			
Strain rate, in./min.	0.06			
Water content, %	24.3			
Wet density, pcf	125.6			
Dry density, pcf	101.0			
Saturation, %	98.2			
Void ratio	0.6683			
Specimen diameter, in.	2.83			
Specimen height, in.	5.53			
Height/diameter ratio	1.96			

Description:

LL =	PL =	PI =	Assumed GS= 2.7	Type: 3" Press Tube

Project No.: 0121-3070.03

Date:

Remarks:

Figure ________

Client: TranSystems, Inc.

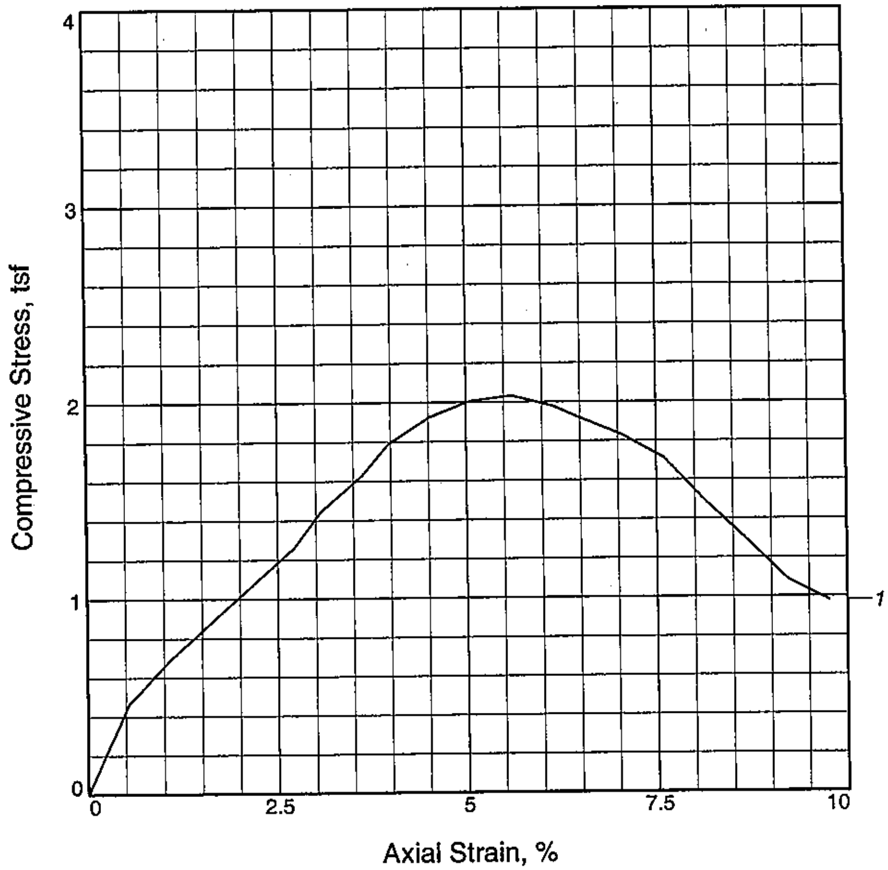
Project: SCI-823-0.00

Source of Sample: B-2 **Depth:** 6.0

Sample Number: P1

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.031			
Undrained shear strength, tsf	1.016			
Failure strain,	5.6			
Strain rate, in./min.	0.06			
Water content, %	24.3			
Wet density, pcf	125.6			
Dry density, pcf	101.0			
Saturation, %	97.2			
Void ratio	0.6807			
Specimen diameter, in.	2.83			
Specimen height, in.	5.53			
Height/diameter ratio	1.96			

Description: Fat clay

LL = 67	**PL** = 27	**PI** = 40	**Assumed GS=** 2.72	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 7/5/06

Remarks:
Specific Gravity= 2.72

Figure _______

Client: TranSystems, Inc.

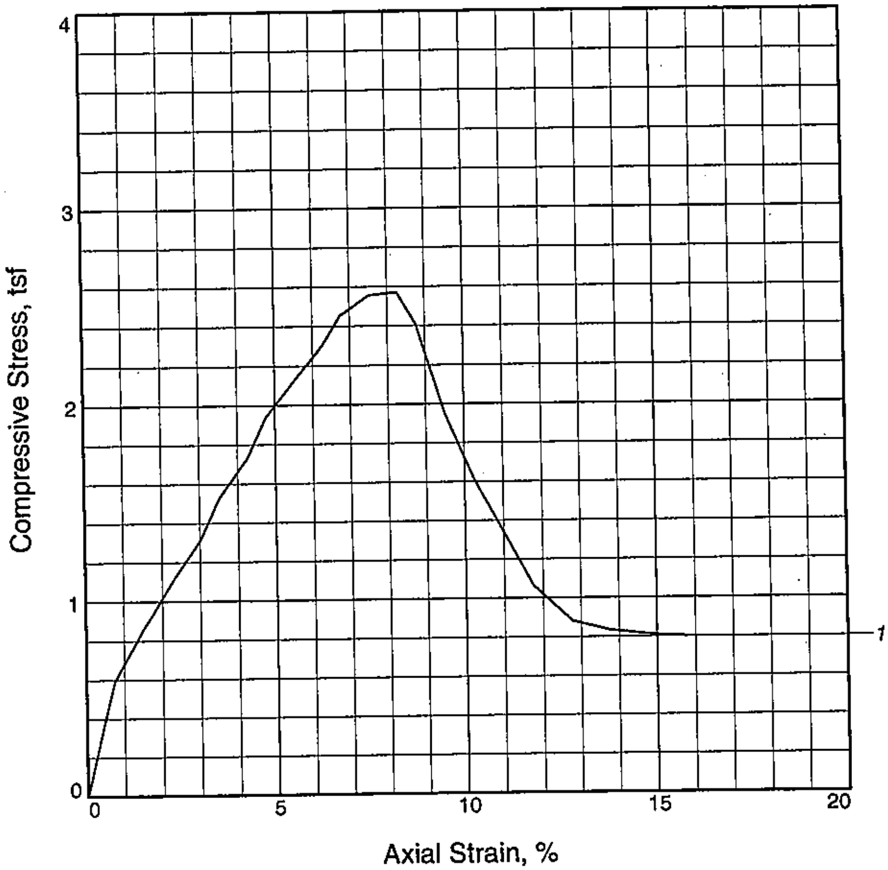
Project: SCI-823-0.00

Source of Sample: B-2 **Depth:** 6.0

Sample Number: P-1

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.572			
Undrained shear strength, tsf	1.286			
Failure strain,	8.3			
Strain rate, in./min.	0.06			
Water content, %	31.1			
Wet density, pcf	120.2			
Dry density, pcf	91.7			
Saturation, %	100.1			
Void ratio	0.8382			
Specimen diameter, in.	2.84			
Specimen height, in.	4.00			
Height/diameter ratio	1.41			

Description: Fat clay

LL = 52	**PL** = 22	**PI** = 30	**Assumed GS=** 2.70	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

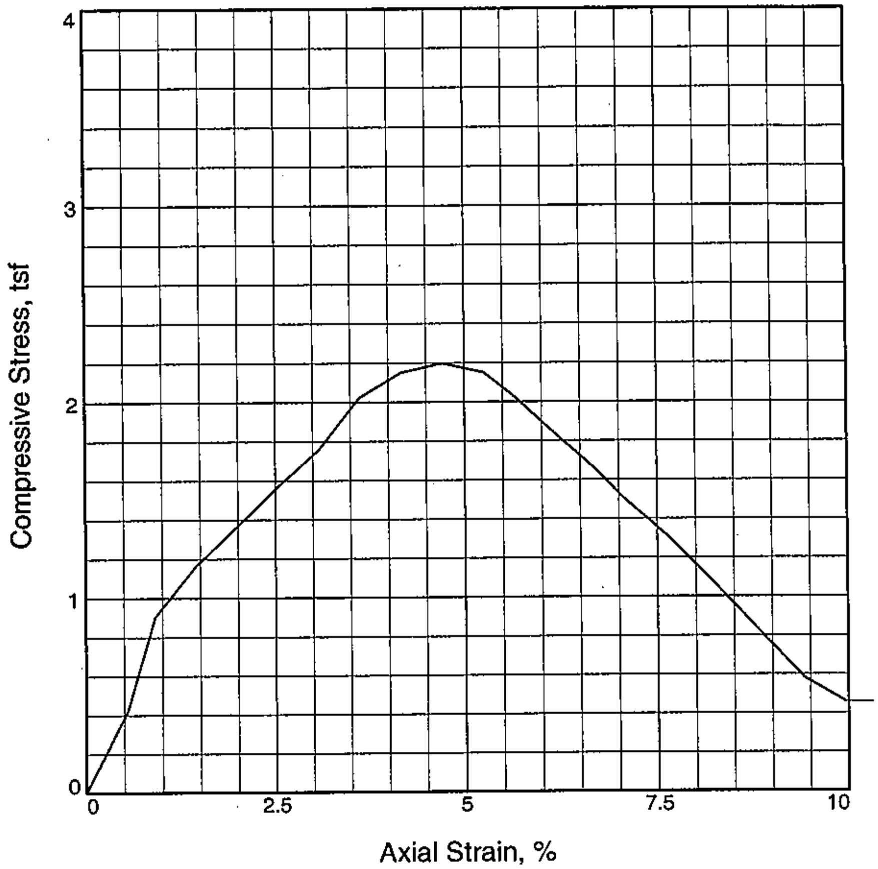
Source of Sample: B-2 **Depth:** 11.0

Sample Number: P-2

DLZ

Figure _______

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.198			
Undrained shear strength, tsf	1.099			
Failure strain,	4.7			
Strain rate, in./min.	0.06			
Water content, %	30.2			
Wet density, pcf	121.4			
Dry density, pcf	93.3			
Saturation, %	96.2			
Void ratio	0.8811			
Specimen diameter, in.	2.84			
Specimen height, in.	5.52			
Height/diameter ratio	1.94			

Description:

LL = 78	**PL =** 24	**PI =** 54	**Assumed GS=** 2.81	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:
Specific Gravity= 2.81

Figure ______

Client: TranSystems, Inc.

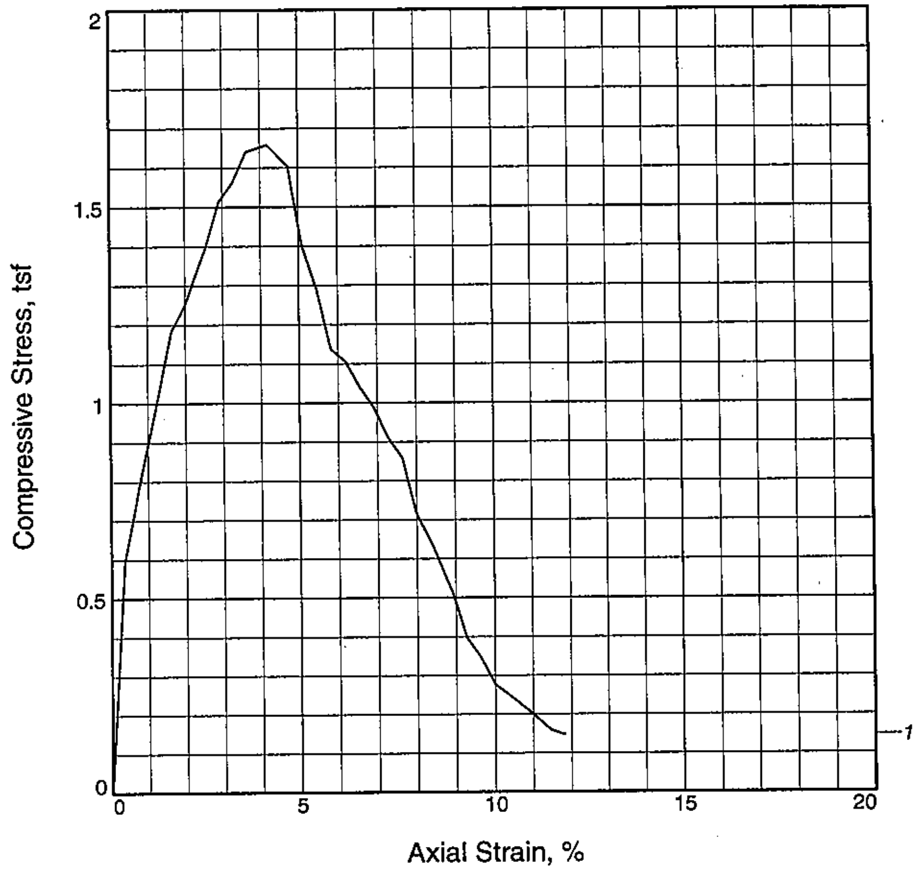
Project: SCI-823-0.00

Source of Sample: B-2 **Depth:** 12.5

Sample Number: P-2B

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.658			
Undrained shear strength, tsf	0.829			
Failure strain,	4.2			
Strain rate, in./min.	0.06			
Water content, %	29.1			
Wet density, pcf	122.6			
Dry density, pcf	95.0			
Saturation, %	98.9			
Void ratio	0.8078			
Specimen diameter, in.	2.85			
Specimen height, in.	5.50			
Height/diameter ratio	1.93			

Description: Fat clay

LL = 60	**PL** = 26	**PI** = 34	**Assumed GS=** 2.75	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

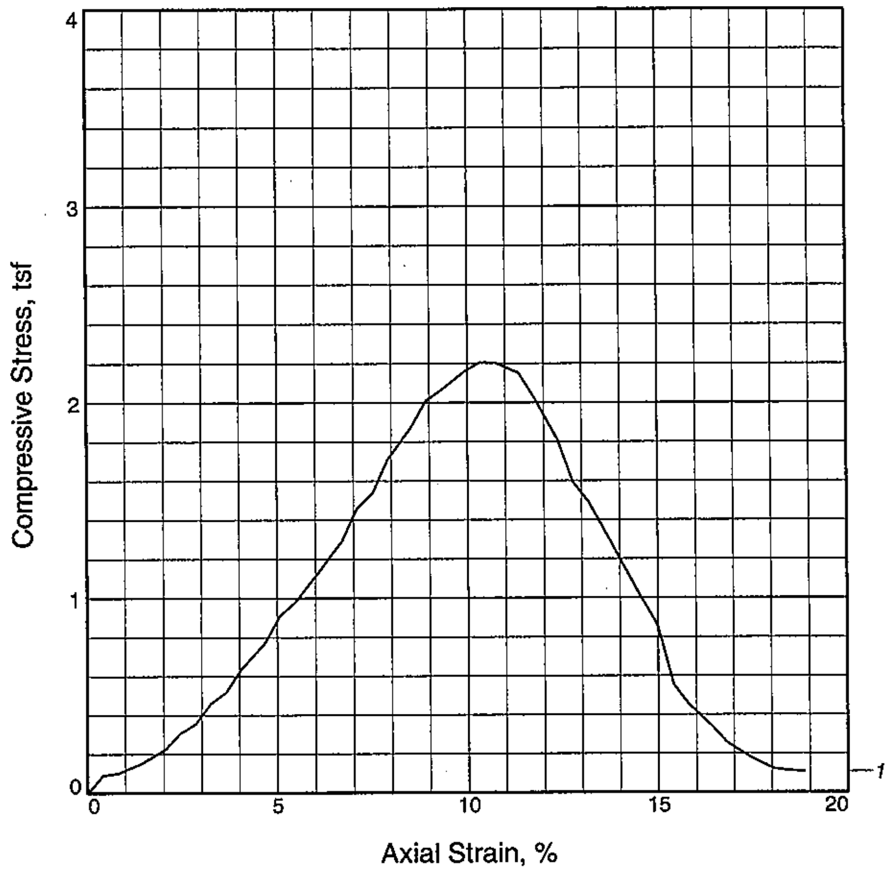
Source of Sample: B-3 **Depth:** 8.0

Sample Number: P-1

DLZ

Figure _______

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.204			
Undrained shear strength, tsf	1.102			
Failure strain,	10.3			
Strain rate, in./min.	0.06			
Water content, %	29.5			
Wet density, pcf	119.9			
Dry density, pcf	92.6			
Saturation, %	94.9			
Void ratio	0.8544			
Specimen diameter, in.	2.83			
Specimen height, in.	4.93			
Height/diameter ratio	1.74			

Description: Lean clay

LL = 44	**PL =** 19	**PI =** 25	**Assumed GS=** 2.75	**Type:** 3" Press Tube

Project No.: 0121-3070.03

Date: 07/05/06

Remarks:

Figure _______

Client: TranSystems, Inc.

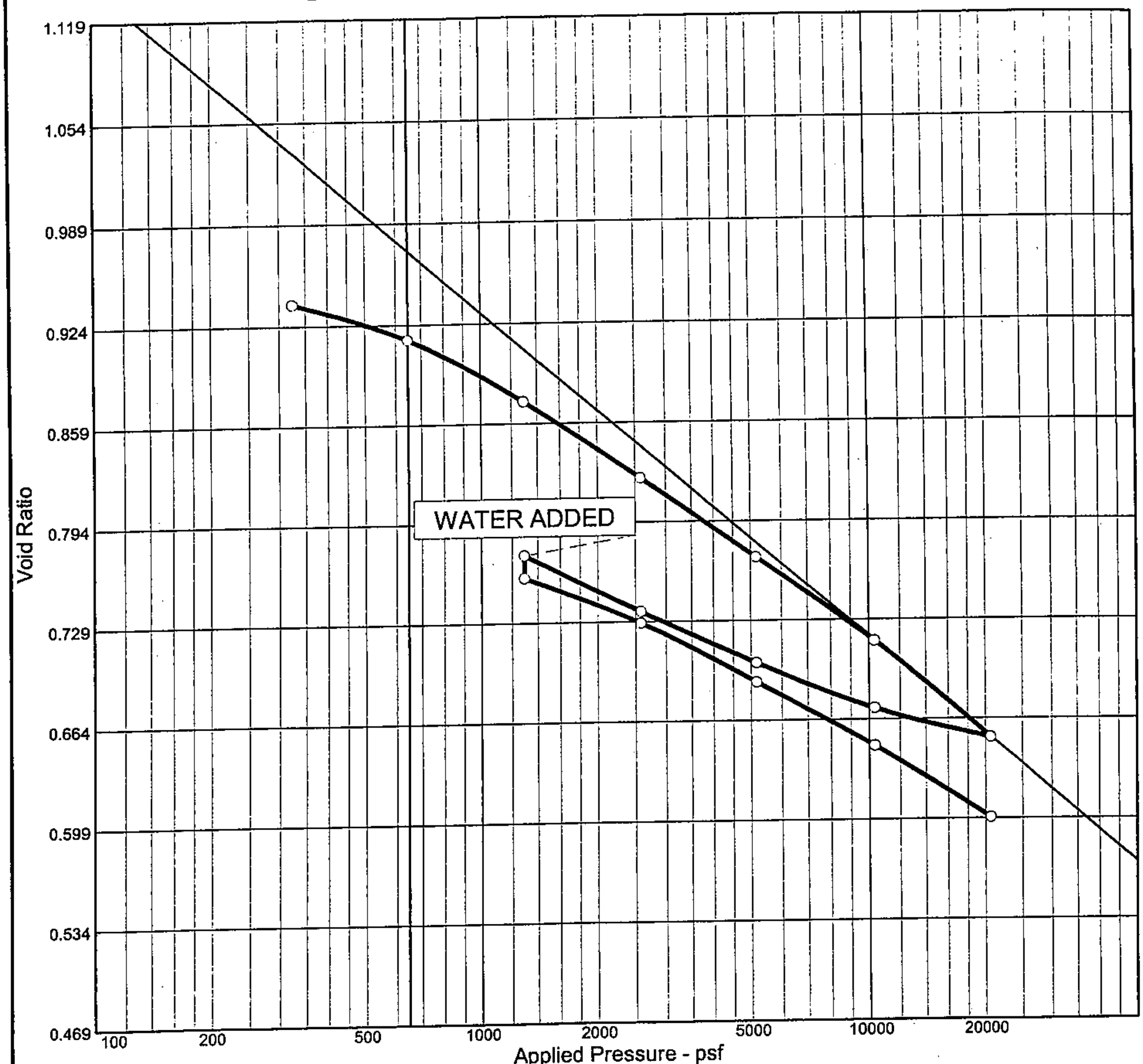
Project: SCI-823-0.00

Source of Sample: B-3 **Depth:** 10.0

Sample Number: P-2

DLZ

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
104.8 %	36.5 %	87.5	49	23	2.74	CL	A-7-6(27	0.955

MATERIAL DESCRIPTION

Lean clay

Project No. 0121- **Client:** TranSystems, Inc.

Project: SCI-823-0.00

Source: B-1 **Sample No.:** P-3 **Elev./Depth:** 18.0

DLZ

Remarks:

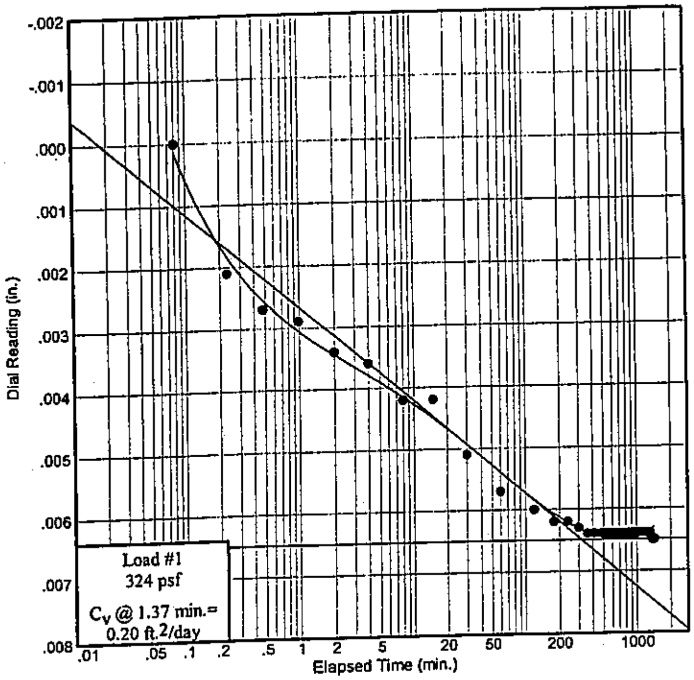
Specific Gravity= 2.74

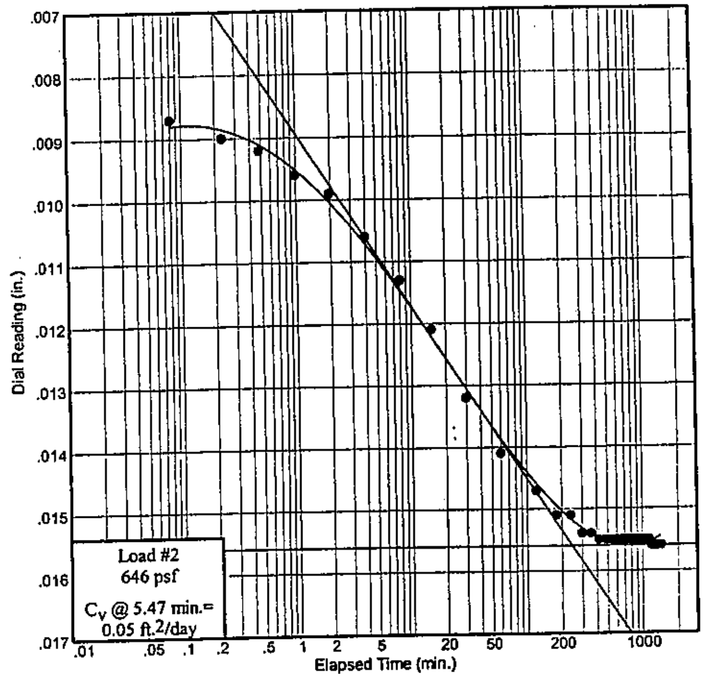
Figure

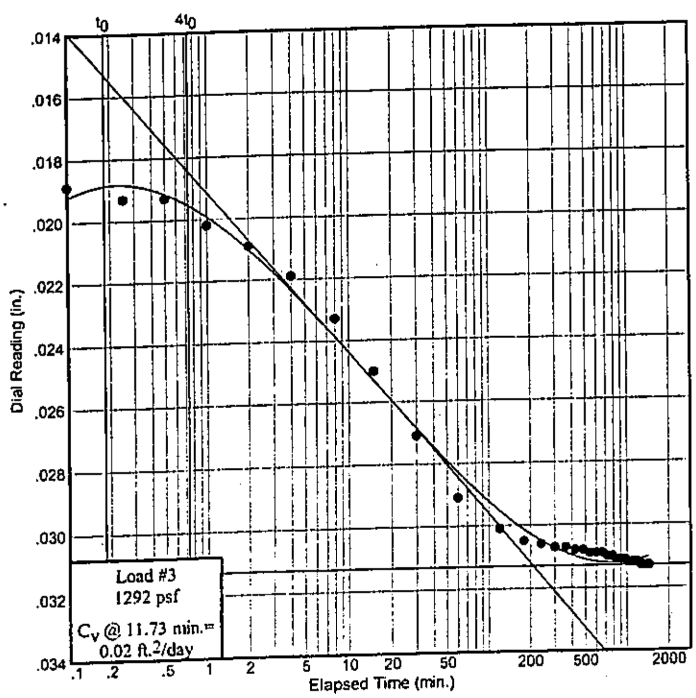
Dial Reading vs. Time

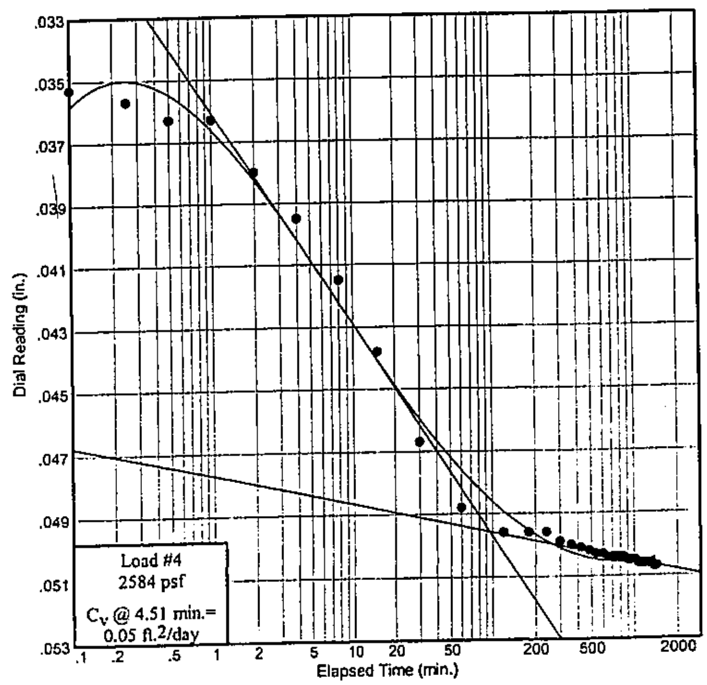
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1 Sample No.: P-3 Elev./Depth: 18.0









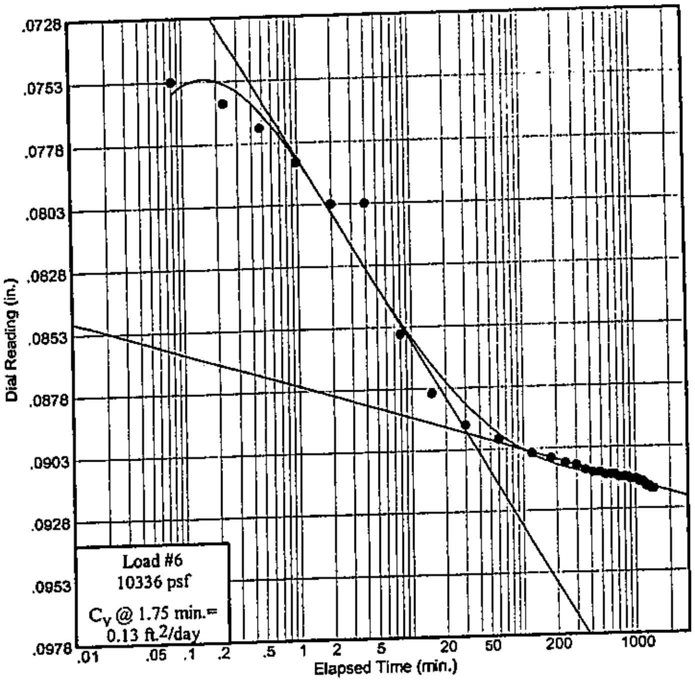
DLZ

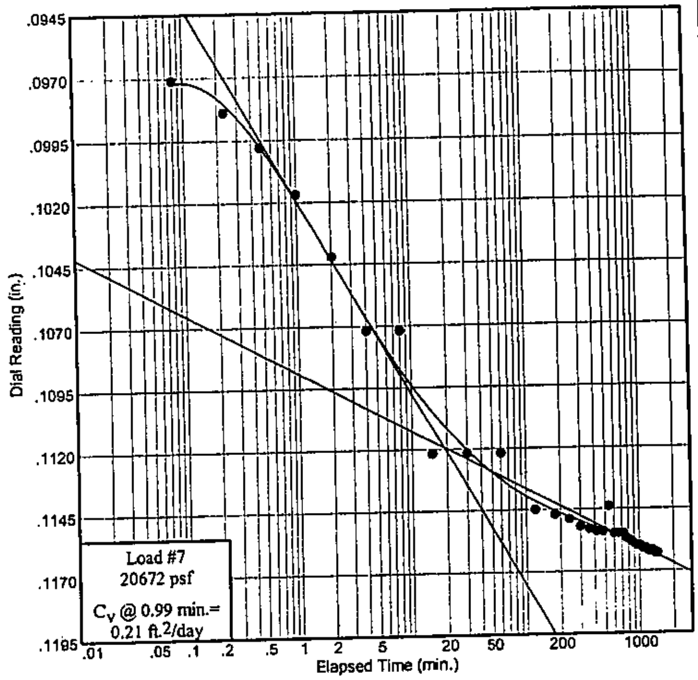
Figure

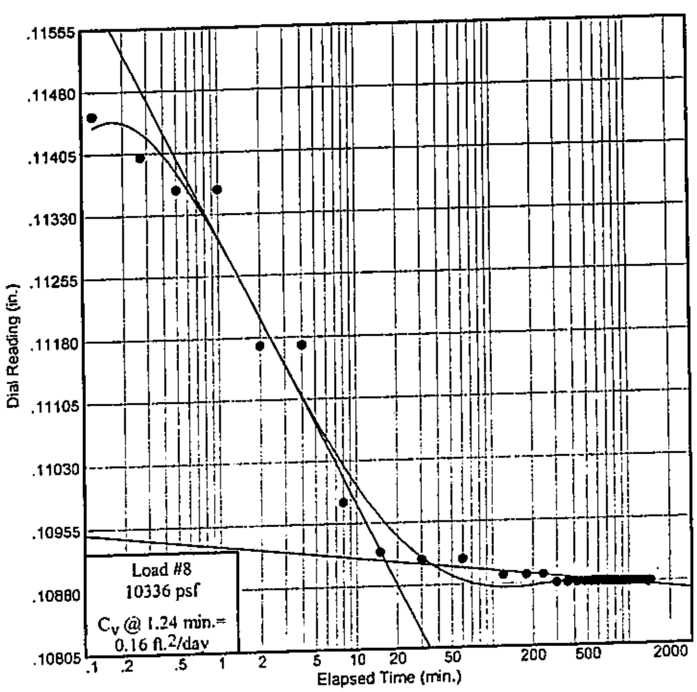
Dial Reading vs. Time

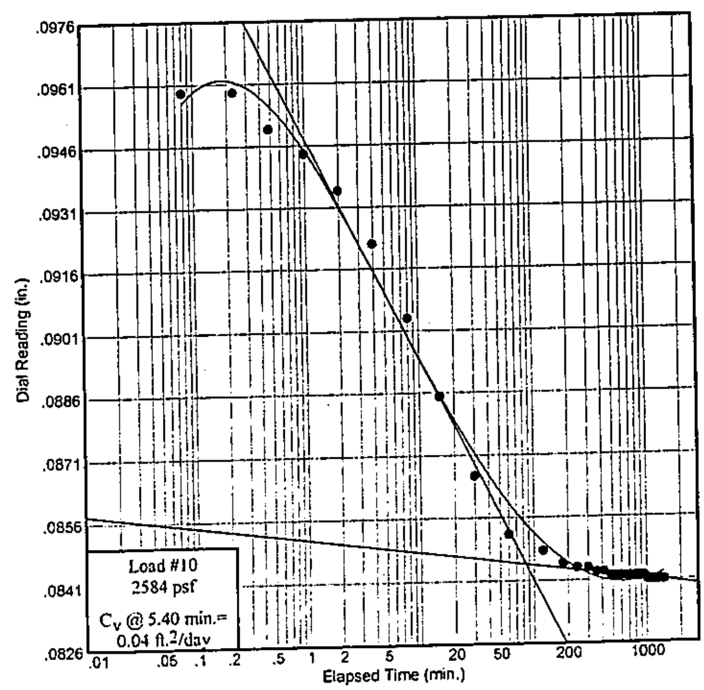
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1 | Sample No.: P-3 | Elev./Depth: 18.0









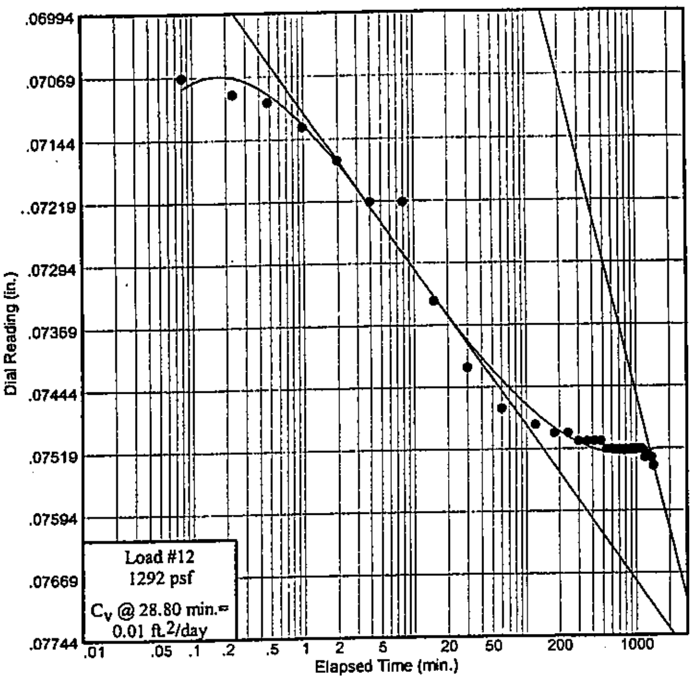
DLZ

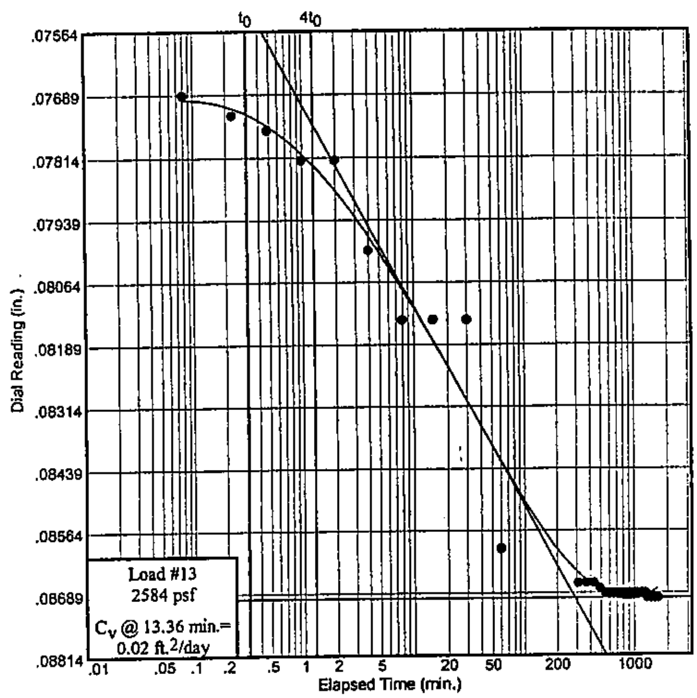
Figure

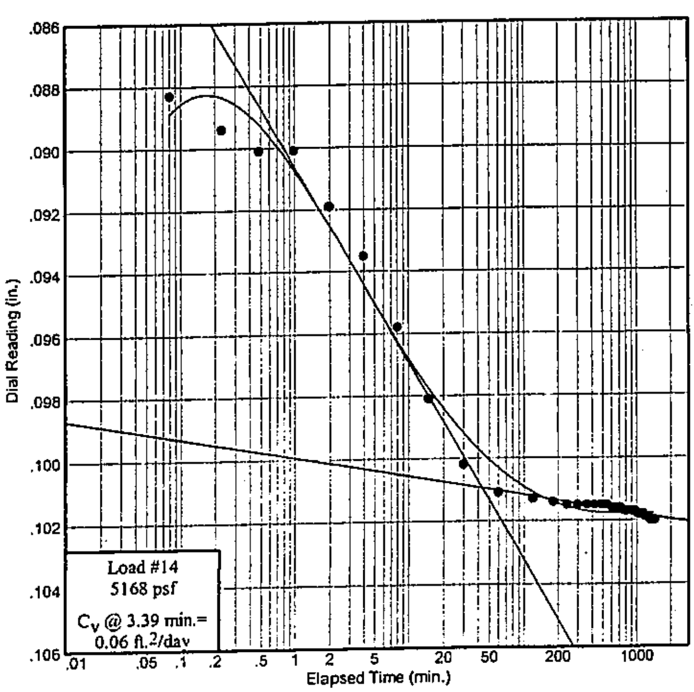
Dial Reading vs. Time

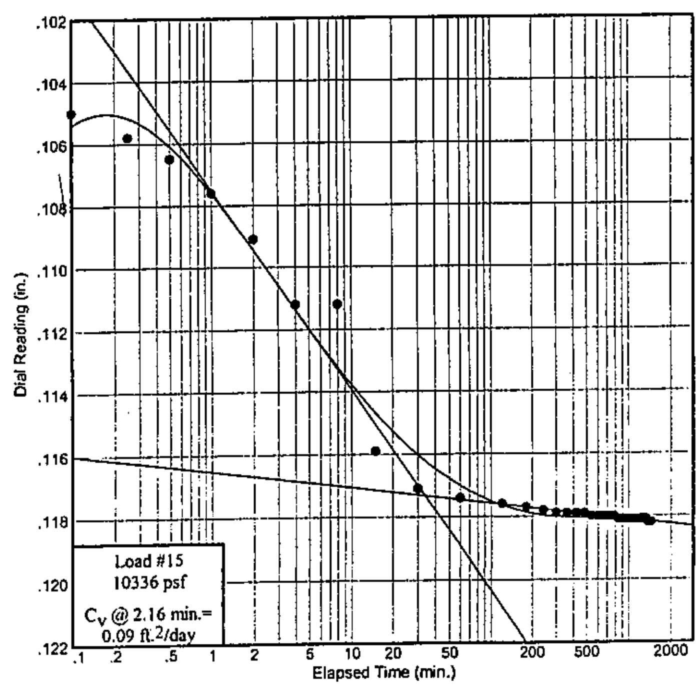
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1 | Sample No.: P-3 | Elev./Depth: 18.0









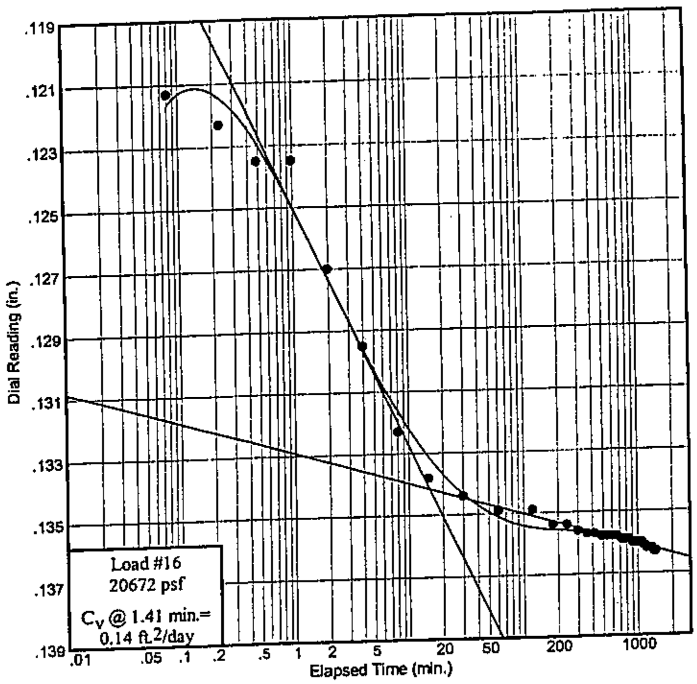
DLZ

Figure

Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

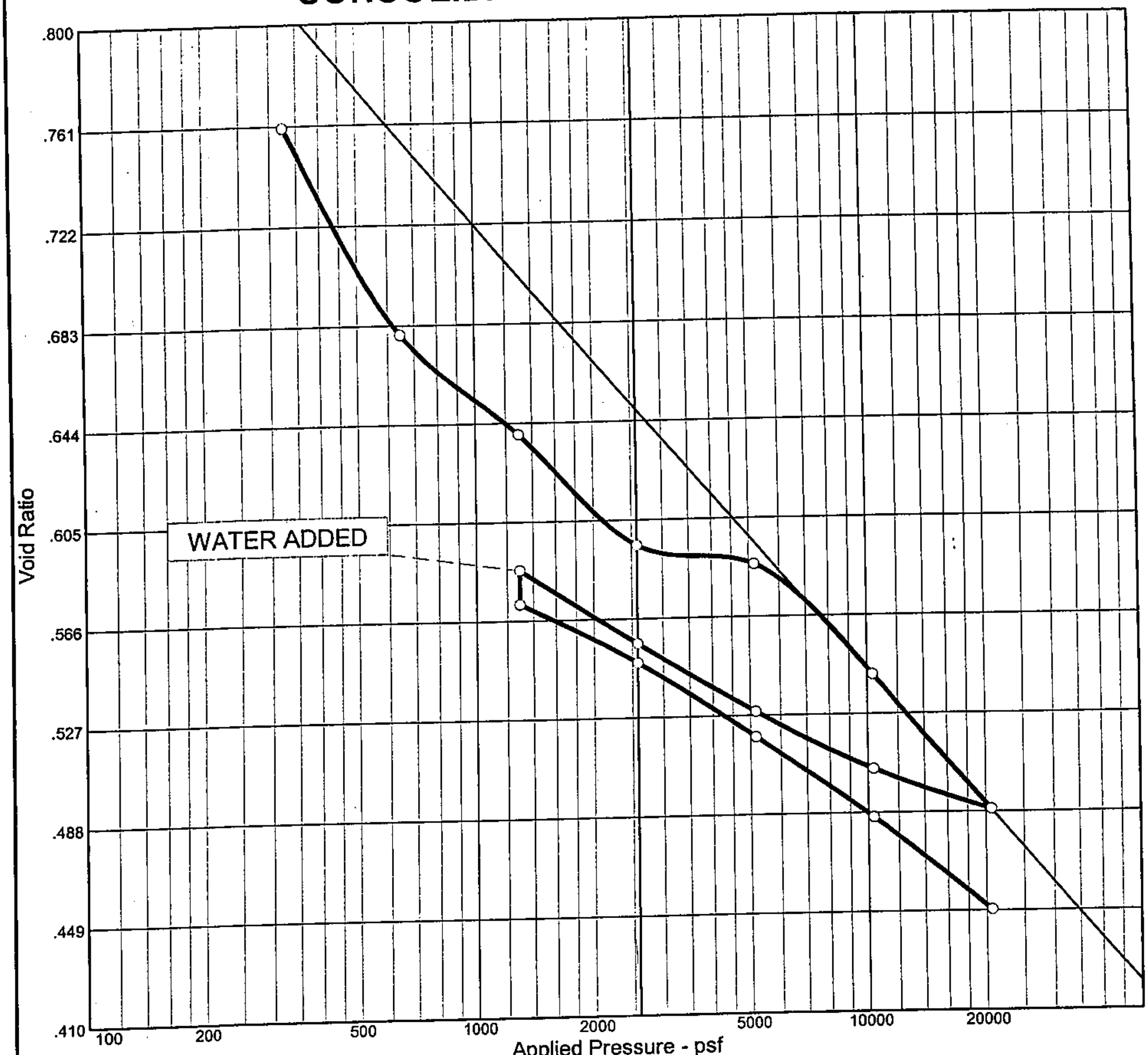
Source: B-1 | Sample No.: P-3 | Elev./Depth: 18.0



DLZ

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
95.1 %	28.2 %	95.7	78	54	2.81	CH	A-7-6(56	0.833

MATERIAL DESCRIPTION

Project No. 0121-

Client: TranSystems, Inc.

Project: SCI-823-0.00

Source: B-2

Sample No.: P-2B

Elev./Depth: 12.5

DLZ

Remarks:

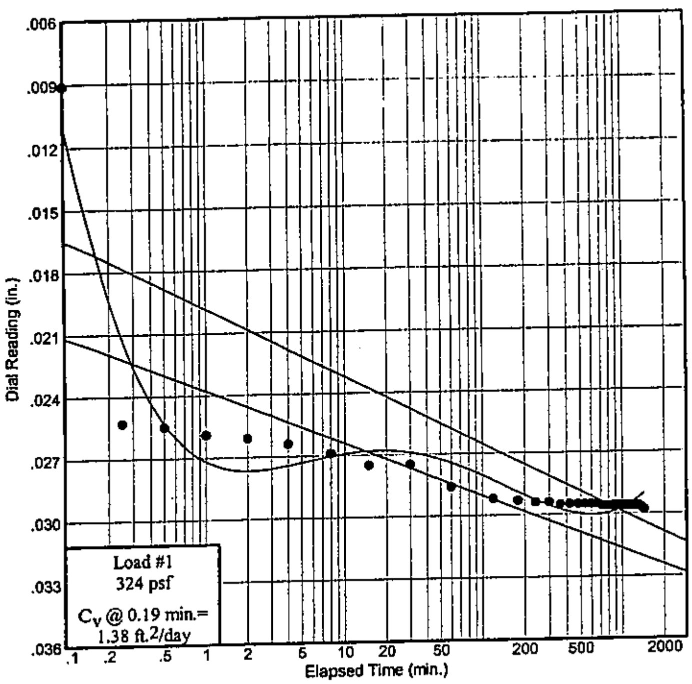
Specific Gravity= 2.81

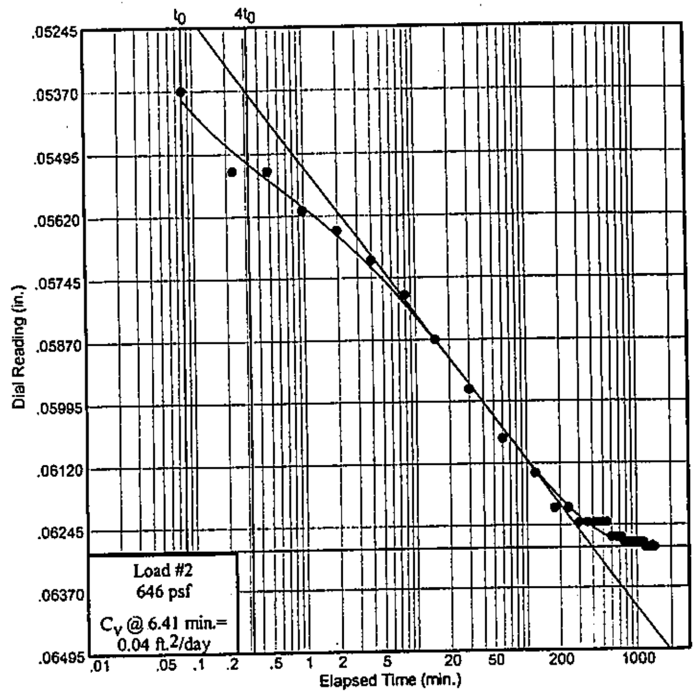
Figure

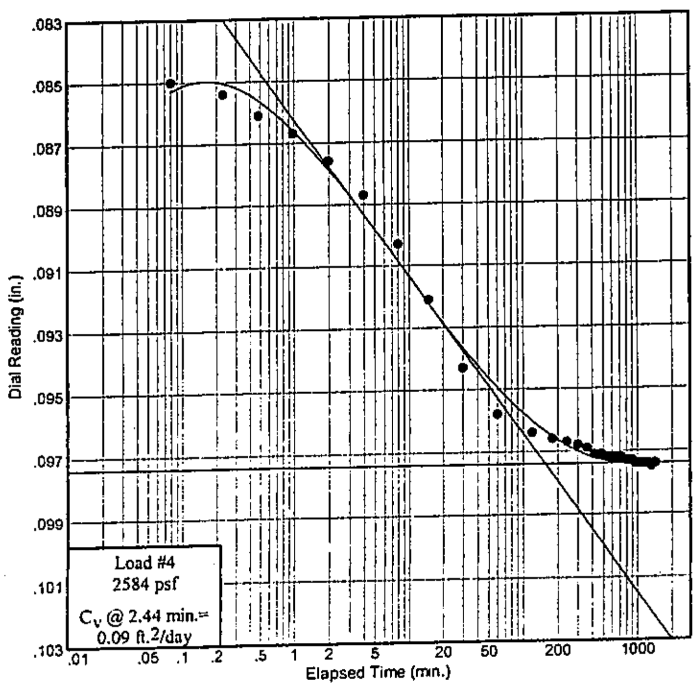
Dial Reading vs. Time

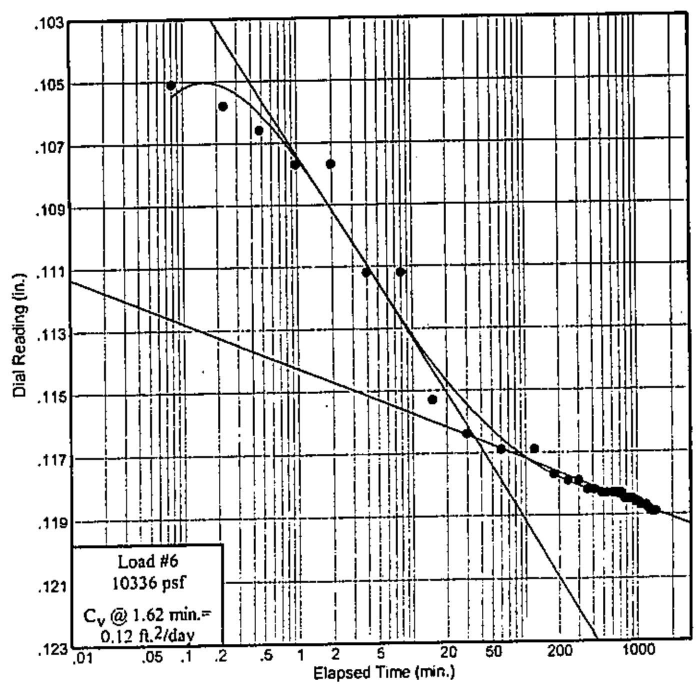
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5









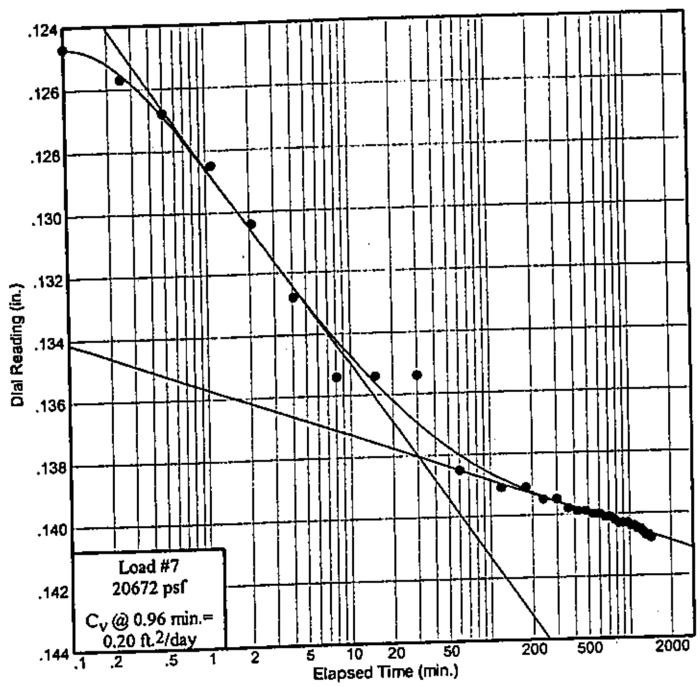
DLZ

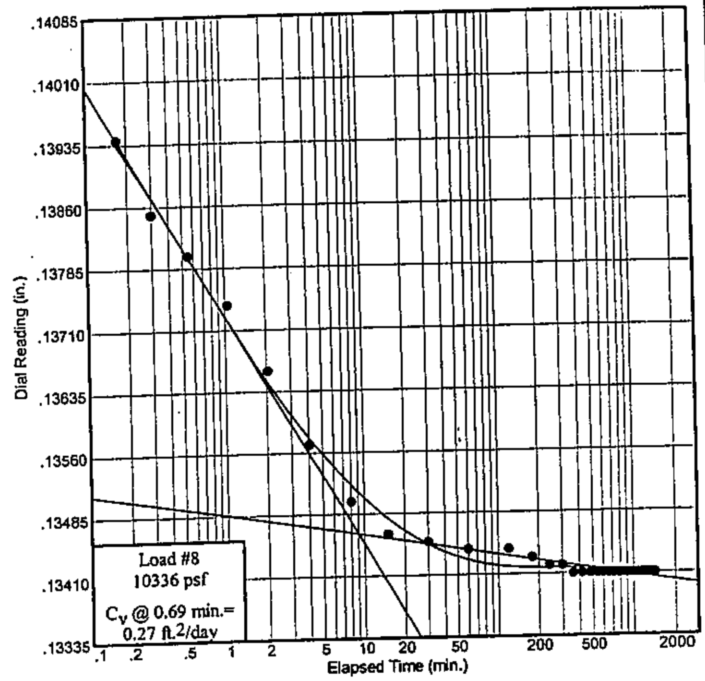
Figure

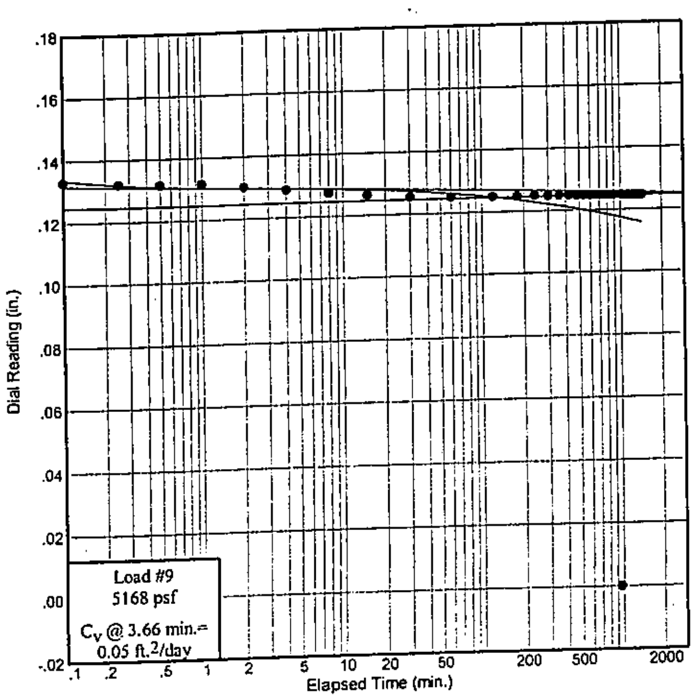
Dial Reading vs. Time

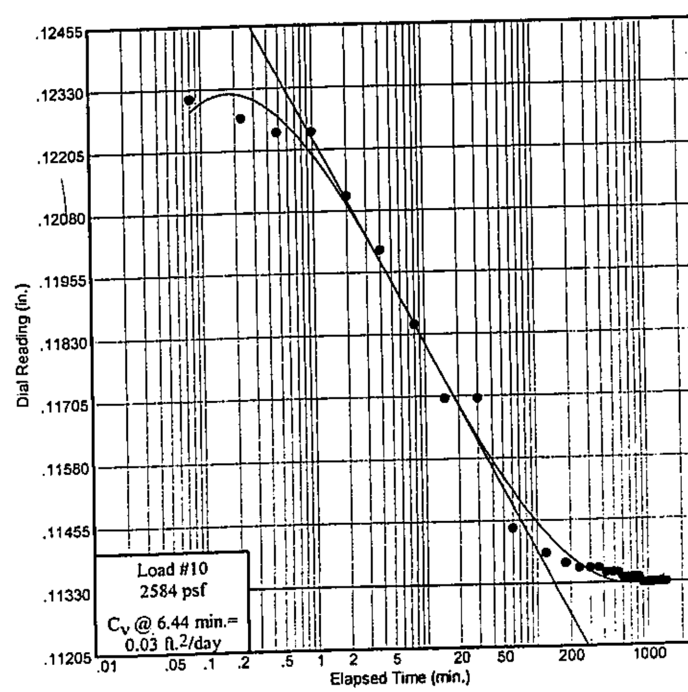
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5









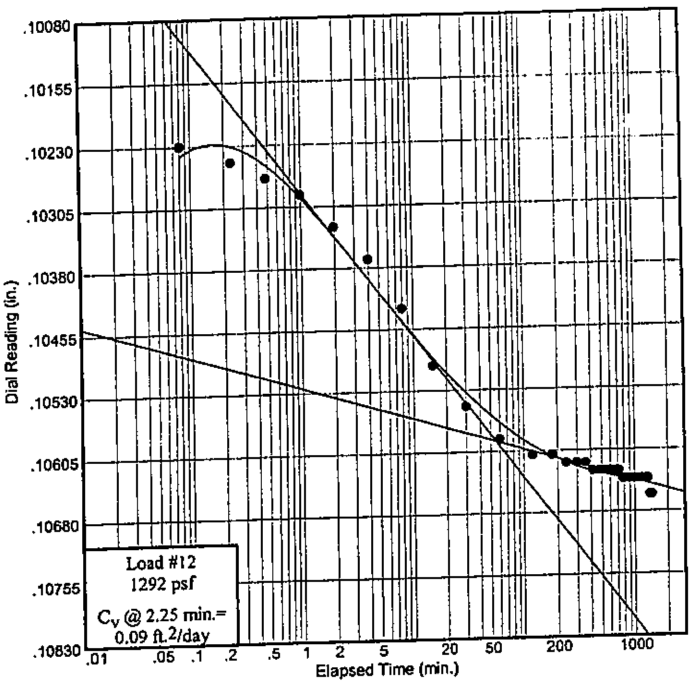
DLZ

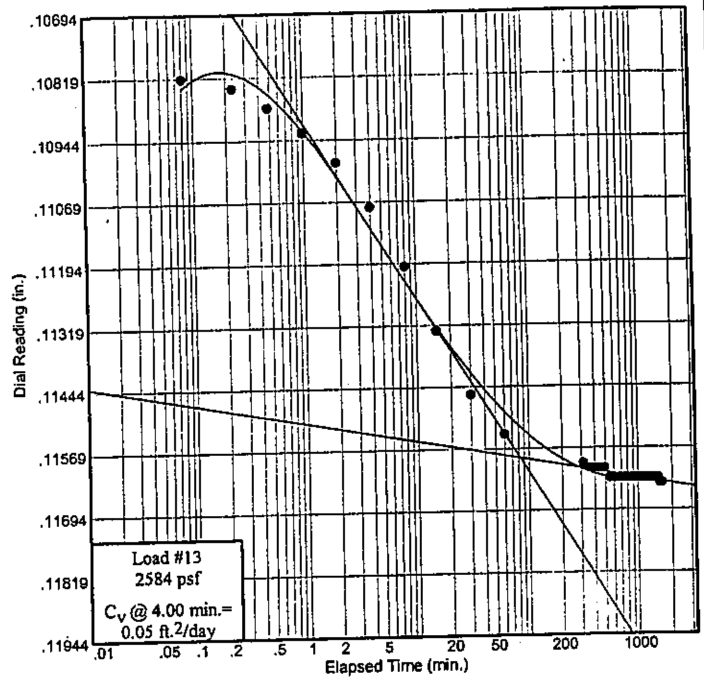
Figure

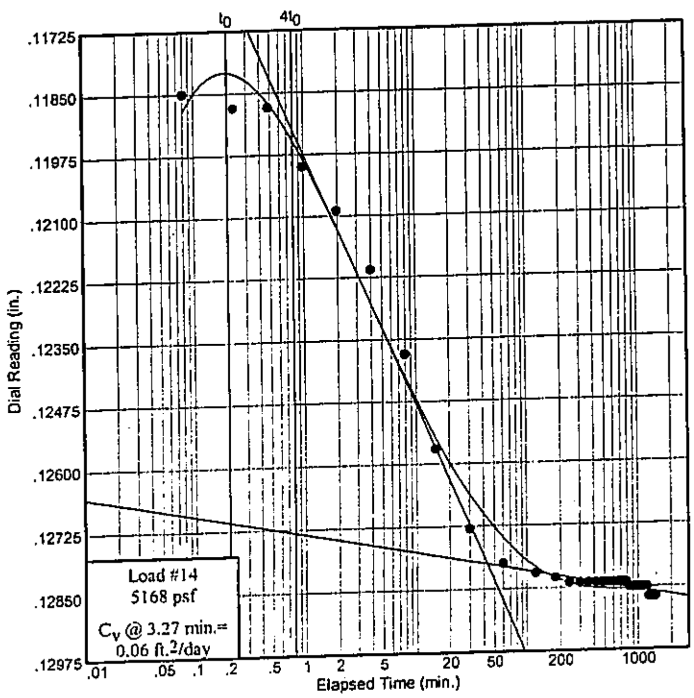
Dial Reading vs. Time

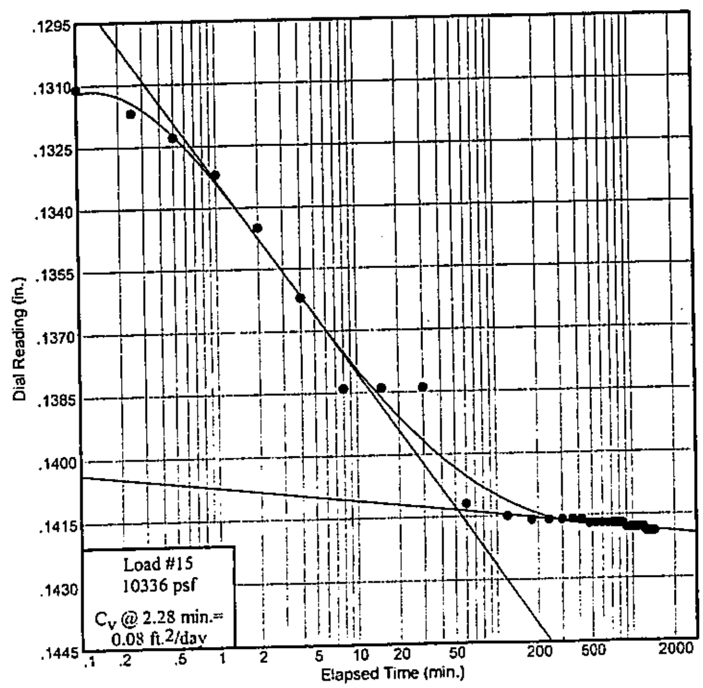
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5









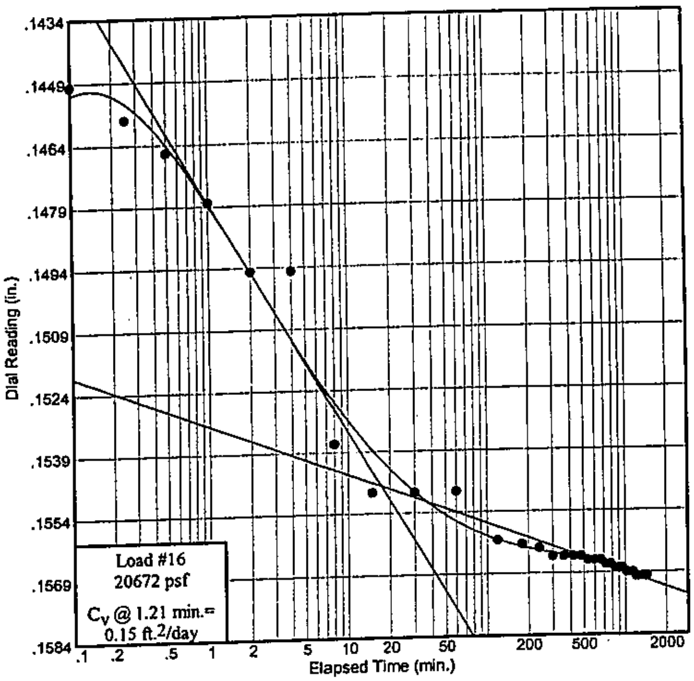
DLZ

Figure

Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5



DLZ

Figure

SWELL/CONSOLIDATION TEST DATA

ent: TranSystems, Inc.
ject: SCI-823-0.00
ject Number: 0121-3070.03

Sample Data

ırce: B-2
ıple No.: P-1
ıv. or Depth: 6.0
ıation:
ıcription: Fat clay
ıuid Limit: 67
:S: CH
ıting Remarks: Specific Gravity= 2.72

Sample Length(in./cm.): 24

Plasticity Index: 40

AASHTO: A-7-6(47

Figure No.:

Test Specimen Data

TOTAL SAMPLE

: w+t = 299.62 g.
ı w+t = 251.16 g.
:e Wt. = 62.40 g.
ıght = .75 in.
ımeter = 2.50 in.
ıght = 120.64 g.

ısture = 25.7 %
t Den. = 124.8 pcf
ı Den. = 99.3 pcf

BEFORE TEST

Consolidometer # = 5

Spec. Gravity = 2.72
Height = .75 in.
Diameter = 2.50 in.
Defl. Table = n/a

Ht. Solids = 0.4387 in.
Dry Wt. = 96.00 g.*
Void Ratio = 0.709
Saturation = 98.4 %

AFTER TEST

Wet w+t = 180.72 g.
Dry w+t = 156.53 g.
Tare Wt. = 60.26 g.

Moisture = 25.1 %
Dry Wt. = 96.27 g.
Void Ratio = 0.568

Initial dry weight used in calculations

End-of-Load Summary

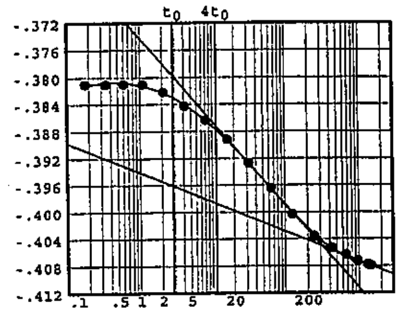
ressure (psf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
start	-0.38112				0.709	
00000	-0.40796	0.00000	0.01		0.771	3.6 Swell
00000	-0.40290	0.00000	0.48		0.759	2.9 Swell
00000	-0.39167	0.00000	0.02	0.002	0.733	1.4 Swell
00000	-0.37832	0.00000	0.05		0.703	0.4 Comprs.
00000	-0.36112	0.00000	0.04	0.001	0.664	2.7 Comprs.
00000	-0.34233	0.00000	0.09	0.002	0.621	5.2 Comprs.
00000	-0.32098	0.00000	0.08	0.002	0.572	8.0 Comprs.
00000	-0.33042	0.00000			0.594	6.8 Comprs.
00000	-0.34194	0.00000	0.06		0.620	5.2 Comprs.
00000	-0.35433	0.00000	0.02		0.648	3.6 Comprs.
00000	-0.36705	0.00000	0.01	0.000	0.677	1.9 Comprs.
00000	-0.37930	0.00000	2.61		0.705	0.2 Comprs.
00000	-0.39222	0.00000	0.00		0.735	1.5 Swell

:essure (psf)	Final Dial (in.)	Machine Defl. (in.)	C_v (ft.^{2}/day)	C_α	Void Ratio	% Compression /Swell
00000	-0.38786	0.00000	0.01		0.725	0.9 Swell
00000	-0.37784	0.00000		0.004	0.702	0.4 Comprs.
00000	-0.36724	0.00000	0.01		0.678	1.9 Comprs.
00000	-0.35240	0.00000	0.05	0.002	0.644	3.8 Comprs.
00000	-0.33675	0.00000	0.08	0.002	0.608	5.9 Comprs.
00000	-0.31900	0.00000	0.09	0.001	0.568	8.3 Comprs.

= 0.16 P_c = 2792612 psf C_r = 0.09

'essure: 500000 psf **TEST READINGS** **Load No. 1**

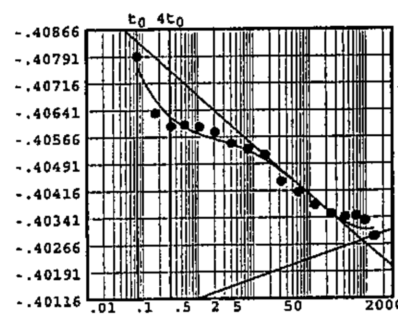
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.38112	11	64.12	-0.39645
2	0.18	-0.38106	12	124.12	-0.40034
3	0.35	-0.38105	13	244.12	-0.40361
4	0.60	-0.38099	14	424.12	-0.40531
5	1.10	-0.38106	15	664.12	-0.40634
6	2.10	-0.38218	16	964.12	-0.40727
7	4.10	-0.38432	17	1324.12	-0.40780
8	8.10	-0.38637	18	1440.07	-0.40796
9	16.10	-0.38924			
0	32.12	-0.39273			



Void Ratio = 0.771 **Swell** = 3.6 %
D_0 = -0.37822 D_{50} = -0.39165 D_{100} = -0.40508
C_v **at 25.5 min.** = 0.01 ft.^{2}/day

:essure: 1000000 psf **TEST READINGS** **Load No. 2**

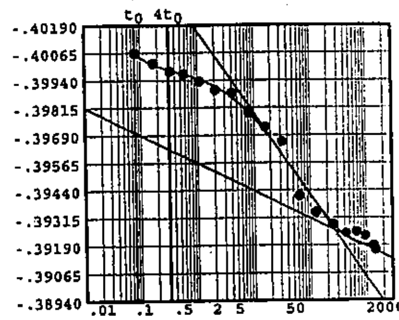
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.40795	11	32.08	-0.40441
2	0.08	-0.40791	12	64.10	-0.40411
3	0.17	-0.40633	13	124.10	-0.40376
4	0.33	-0.40596	14	244.10	-0.40351
5	0.58	-0.40600	15	424.10	-0.40344
6	1.08	-0.40594	16	664.10	-0.40346
7	2.08	-0.40580	17	964.10	-0.40332
8	4.08	-0.40548	18	1324.10	-0.40288
9	8.08	-0.40532	19	1440.20	-0.40290
0	16.08	-0.40516			



Void Ratio = 0.759 **Swell** = 2.9 %
D_0 = -0.40906 D_{50} = -0.40591 D_{100} = -0.40277
C_v **at 0.6 min.** = 0.48 ft.^{2}/day

essure: 2000000 psf **TEST READINGS** **Load No. 3**

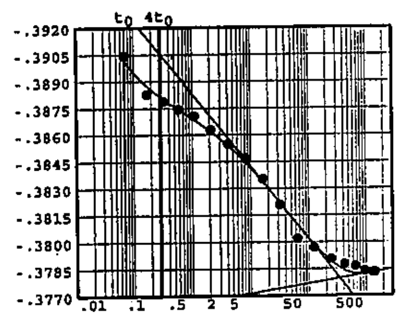
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.40290	11	32.10	-0.39660
2	0.08	-0.40064	12	64.10	-0.39413
3	0.17	-0.40016	13	124.10	-0.39339
4	0.33	-0.39981	14	244.10	-0.39285
5	0.58	-0.39967	15	424.10	-0.39243
6	1.10	-0.39933	16	664.12	-0.39250
7	2.10	-0.39894	17	964.12	-0.39232
8	4.10	-0.39880	18	1324.12	-0.39187
9	8.10	-0.39789	19	1440.27	-0.39167
0	16.10	-0.39727			



Void Ratio = 0.733 Swell = 1.4 %
D_0 = -0.40136 D_{50} = -0.39697 D_{100} = -0.39257
C_v at 16.8 min. = 0.02 ft.^{2}/day C_α = 0.002

essure: 4000000 psf **TEST READINGS** **Load No. 4**

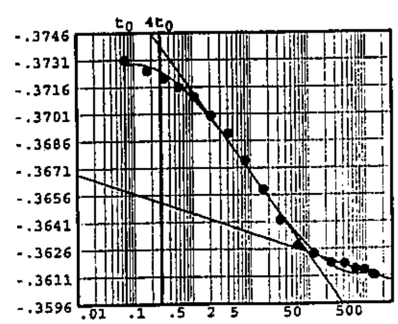
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.39168	11	32.12	-0.38208
2	0.07	-0.39043	12	64.12	-0.38020
3	0.17	-0.38827	13	124.12	-0.37968
4	0.33	-0.38787	14	244.12	-0.37906
5	0.58	-0.38742	15	424.12	-0.37876
6	1.10	-0.38705	16	664.13	-0.37865
7	2.10	-0.38627	17	964.13	-0.37839
8	4.10	-0.38550	18	1324.13	-0.37830
9	8.12	-0.38467	19	1440.40	-0.37832
0	16.12	-0.38350			



Void Ratio = 0.703 Compression = 0.4 %
D_0 = -0.39214 D_{50} = -0.38506 D_{100} = -0.37798
C_v at 5.2 min. = 0.05 ft.^{2}/day

essure: 8000000 psf **TEST READINGS** **Load No. 5**

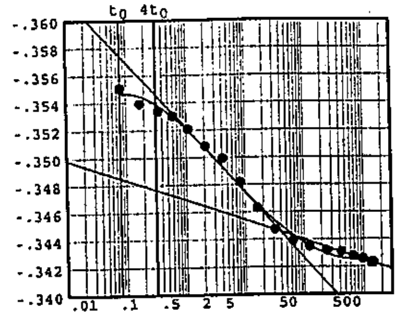
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.37831	11	32.12	-0.36418
2	0.07	-0.37308	12	64.12	-0.36279
3	0.17	-0.37249	13	124.12	-0.36232
4	0.33	-0.37205	14	244.12	-0.36183
5	0.58	-0.37158	15	424.12	-0.36178
6	1.10	-0.37103	16	664.13	-0.36148
7	2.10	-0.36998	17	964.13	-0.36142
8	4.10	-0.36899	18	1324.13	-0.36116
9	8.10	-0.36749	19	1440.30	-0.36112
0	16.10	-0.36588			



Void Ratio = 0.664 Compression = 2.7 %
D_0 = -0.37318 D_{50} = -0.36780 D_{100} = -0.36242
C_v at 6.1 min. = 0.04 ft.^{2}/day C_α = 0.001

essure: 16000000 psf | TEST READINGS | Load No. 6

).	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.36113	11	32.12	-0.34471
2	0.08	-0.35506	12	64.13	-0.34387
3	0.17	-0.35394	13	124.13	-0.34346
4	0.35	-0.35337	14	244.15	-0.34317
5	0.60	-0.35302	15	424.15	-0.34308
6	1.10	-0.35207	16	664.15	-0.34277
7	2.10	-0.35081	17	964.15	-0.34256
8	4.10	-0.34992	18	1324.17	-0.34231
9	8.12	-0.34816	19	1440.08	-0.34233
0	16.12	-0.34631			



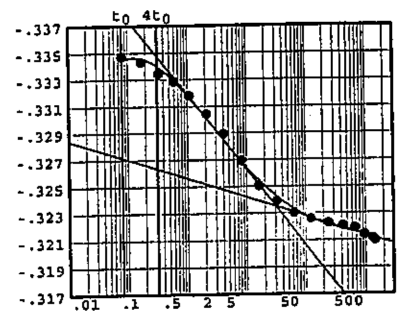
Void Ratio = 0.621 Compression = 5.2 %

D_0 = -0.35542 D_{50} = -0.35000 D_{100} = -0.34457

C_v at 2.9 min. = 0.09 ft.^{2}/day C_α = 0.002

'essure: 32000000 psf | TEST READINGS | Load No. 7

'o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.34232	11	32.12	-0.32390
2	0.08	-0.33472	12	64.12	-0.32300
3	0.17	-0.33428	13	124.12	-0.32259
4	0.33	-0.33342	14	244.13	-0.32228
5	0.60	-0.33284	15	424.13	-0.32205
6	1.10	-0.33181	16	664.13	-0.32189
7	2.10	-0.33042	17	964.15	-0.32140
8	4.10	-0.32893	18	1324.15	-0.32114
9	8.12	-0.32693	19	1440.45	-0.32098
0	16.12	-0.32503			



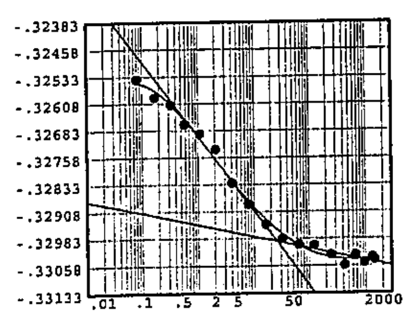
Void Ratio = 0.572 Compression = 8.0 %

D_0 = -0.33486 D_{50} = -0.32919 D_{100} = -0.32352

C_v at 3.0 min. = 0.08 ft.^{2}/day C_α = 0.002

:essure: 16000000 psf | TEST READINGS | Load No. 8

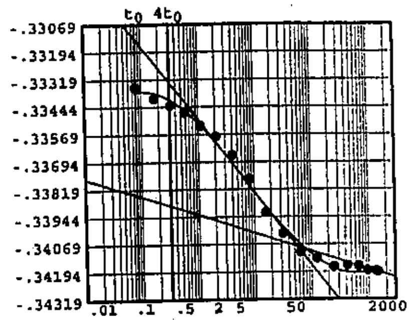
'o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.32098	11	32.10	-0.32983
2	0.08	-0.32541	12	64.10	-0.33000
3	0.17	-0.32592	13	124.10	-0.33002
4	0.33	-0.32612	14	244.12	-0.33028
5	0.58	-0.32666	15	424.12	-0.33057
6	1.08	-0.32693	16	664.13	-0.33030
7	2.10	-0.32736	17	964.13	-0.33050
8	4.10	-0.32829	18	1324.13	-0.33035
9	8.10	-0.32888	19	1440.33	-0.33042
0	16.10	-0.32944			



Void Ratio = 0.594 Compression = 6.8 %

essure: 8000000 psf — TEST READINGS — Load No. 9

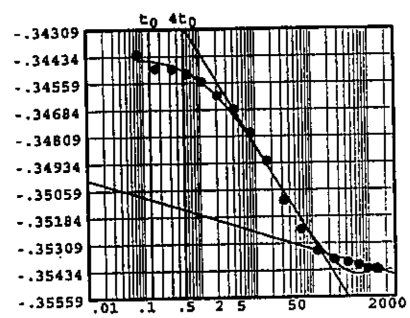
.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
	0.00	-0.33042	11	32.13	-0.34020
	0.08	-0.33356	12	64.13	-0.34106
	0.17	-0.33406	13	124.13	-0.34133
	0.33	-0.33437	14	244.13	-0.34171
	0.58	-0.33472	15	424.13	-0.34166
	1.08	-0.33530	16	664.13	-0.34169
	2.10	-0.33580	17	964.13	-0.34188
	4.10	-0.33663	18	1324.13	-0.34191
	8.12	-0.33775	19	1440.05	-0.34194
	16.12	-0.33925			



Void Ratio = 0.620   Compression = 5.2 %
D_0 = -0.33347 D_{50} = -0.33712 D_{100} = -0.34078
C_v at 4.4 min. = 0.06 ft.^{2}/day

essure: 4000000 psf — TEST READINGS — Load No. 10

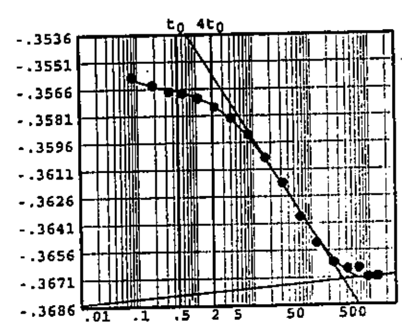
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.34194	11	32.13	-0.35109
2	0.08	-0.34426	12	64.13	-0.35244
3	0.17	-0.34492	13	124.15	-0.35344
4	0.33	-0.34495	14	244.15	-0.35384
5	0.60	-0.34518	15	424.15	-0.35400
6	1.10	-0.34554	16	664.15	-0.35411
7	2.10	-0.34619	17	964.17	-0.35429
8	4.12	-0.34683	18	1324.17	-0.35430
9	8.12	-0.34794	19	1440.38	-0.35433
0	16.12	-0.34923			



Void Ratio = 0.648   Compression = 3.6 %
D_0 = -0.34409 D_{50} = -0.34880 D_{100} = -0.35350
C_v at 11.4 min. = 0.02 ft.^{2}/day

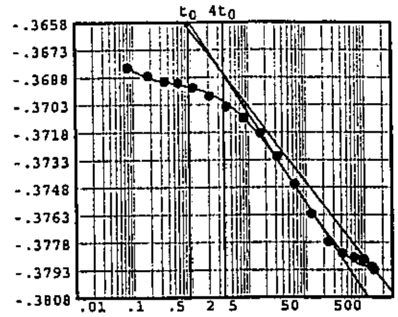
essure: 2000000 psf — TEST READINGS — Load No. 11

o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.35433	11	32.13	-0.36185
2	0.08	-0.35597	12	64.13	-0.36374
3	0.18	-0.35641	13	124.13	-0.36519
4	0.35	-0.35676	14	244.13	-0.36626
5	0.60	-0.35685	15	424.13	-0.36663
6	1.10	-0.35713	16	664.13	-0.36659
7	2.12	-0.35763	17	964.15	-0.36709
8	4.12	-0.35823	18	1324.15	-0.36706
9	8.12	-0.35915	19	1440.28	-0.36705
0	16.12	-0.36046			



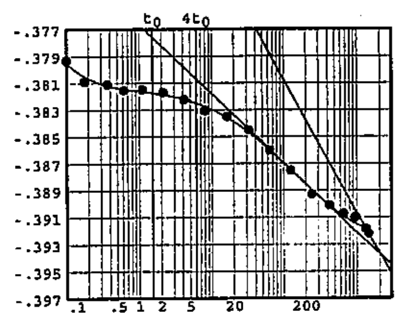
Void Ratio = 0.677   Compression = 1.9 %
D_0 = -0.35596 D_{50} = -0.36159 D_{100} = -0.36722
C_v at 26.9 min. = 0.01 ft.^{2}/day C_α = 0.000

essure: 1000000 psf		TEST READINGS			Load No. 12
ɔ.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.36706	11	32.12	-0.37312
2	0.08	-0.36827	12	64.13	-0.37463
3	0.18	-0.36873	13	124.13	-0.37625
4	0.35	-0.36905	14	244.13	-0.37778
5	0.60	-0.36913	15	424.13	-0.37841
6	1.10	-0.36938	16	664.15	-0.37862
7	2.10	-0.36983	17	964.15	-0.37874
8	4.12	-0.37041	18	1324.15	-0.37915
9	8.12	-0.37102	19	1440.07	-0.37930
0	16.12	-0.37185			



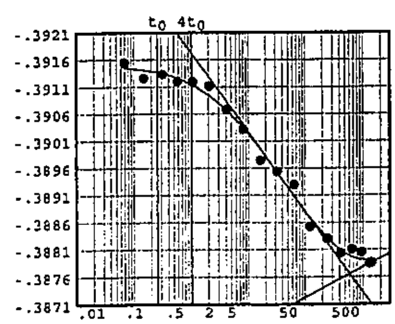
Void Ratio = 0.705 Compression = 0.2 %
D_0 = -0.36851 D_{50} = -0.36846 D_{100} = -0.36842
C_v at 0.1 min. = 2.61 ft.^{2}/day

·essure: 500000 psf		TEST READINGS			Load No. 13
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.37927	11	32.17	-0.38452
2	0.10	-0.37932	12	64.17	-0.38603
3	0.18	-0.38088	13	124.17	-0.38753
4	0.37	-0.38111	14	244.18	-0.38932
5	0.62	-0.38154	15	424.18	-0.39013
6	1.12	-0.38150	16	664.18	-0.39074
7	2.13	-0.38171	17	964.20	-0.39100
8	4.13	-0.38223	18	1324.20	-0.39186
9	8.15	-0.38305	19	1440.13	-0.39222
0	16.15	-0.38350			



Void Ratio = 0.735 Swell = 1.5 %
D_0 = -0.38100 D_{50} = -0.38739 D_{100} = -0.39379
C_v at 120.1 min. = 0.00 ft.^{2}/day

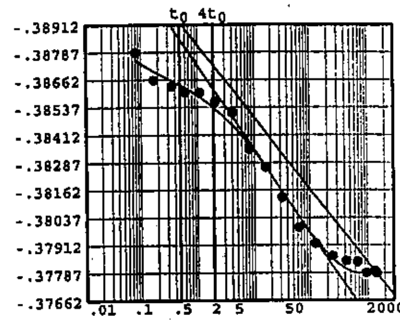
·essure: 1000000 psf		TEST READINGS			Load No. 14
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.39223	11	32.12	-0.38953
2	0.07	-0.39154	12	64.12	-0.38929
3	0.15	-0.39126	13	124.12	-0.38852
4	0.32	-0.39133	14	244.12	-0.38830
5	0.58	-0.39120	15	424.13	-0.38803
6	1.08	-0.39119	16	664.13	-0.38810
7	2.08	-0.39111	17	964.13	-0.38805
8	4.10	-0.39068	18	1324.15	-0.38784
9	8.10	-0.39031	19	1440.18	-0.38786
0	16.10	-0.38974			



Void Ratio = 0.725 Swell = 0.9 %
D_0 = -0.39159 D_{50} = -0.38962 D_{100} = -0.38765
C_v at 27.5 min. = 0.01 ft.^{2}/day

essure: 2000000 psf **TEST READINGS** **Load No. 15**

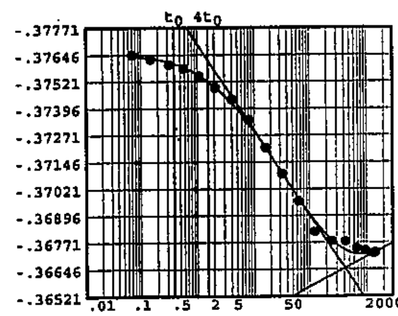
).	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.38786	11	32.12	-0.38128
2	0.08	-0.38786	12	64.12	-0.37992
3	0.17	-0.38660	13	124.12	-0.37920
4	0.35	-0.38630	14	244.13	-0.37862
5	0.60	-0.38604	15	424.13	-0.37840
6	1.10	-0.38602	16	664.13	-0.37836
7	2.10	-0.38562	17	964.13	-0.37786
8	4.10	-0.38512	18	1324.15	-0.37796
9	8.12	-0.38350	19	1440.48	-0.37784
10	16.12	-0.38266			



Void Ratio = 0.702 Compression = 0.4 %

essure: 4000000 psf **TEST READINGS** **Load No. 16**

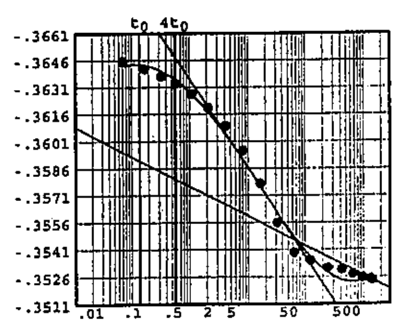
).	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.37784	11	32.10	-0.37092
2	0.07	-0.37646	12	64.10	-0.36963
3	0.15	-0.37625	13	124.12	-0.36820
4	0.32	-0.37600	14	244.12	-0.36776
5	0.58	-0.37580	15	424.12	-0.36774
6	1.08	-0.37546	16	664.13	-0.36743
7	2.08	-0.37492	17	964.13	-0.36734
8	4.10	-0.37437	18	1324.15	-0.36719
9	8.10	-0.37341	19	1440.22	-0.36724
10	16.10	-0.37212			



Void Ratio = 0.678 Compression = 1.9 %
D_0 = -0.37689 D_{50} = -0.37170 D_{100} = -0.36650
C_v at 20.8 min. = 0.01 ft.^{2}/day

essure: 8000000 psf **TEST READINGS** **Load No. 17**

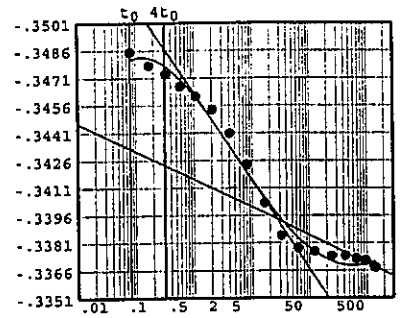
).	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.36725	11	32.10	-0.35558
2	0.07	-0.36454	12	64.10	-0.35393
3	0.17	-0.36410	13	124.10	-0.35348
4	0.33	-0.36368	14	244.10	-0.35308
5	0.58	-0.36331	15	424.12	-0.35299
6	1.10	-0.36270	16	664.12	-0.35274
7	2.10	-0.36194	17	964.12	-0.35255
8	4.10	-0.36094	18	1324.12	-0.35246
9	8.10	-0.35956	19	1440.25	-0.35240
10	16.10	-0.35775			



Void Ratio = 0.644 Compression = 3.8 %
D_0 = -0.36518 D_{50} = -0.35989 D_{100} = -0.35460
C_v at 5.5 min. = 0.05 ft.^{2}/day C_α = 0.002

essure: 16000000 psf | TEST READINGS | Load No. 18

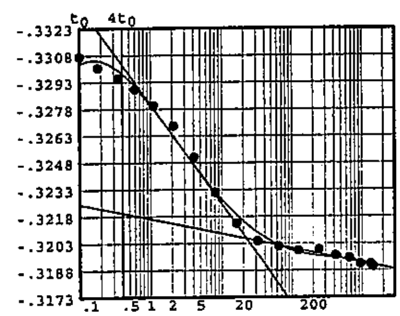
o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.35241	11	32.10	-0.33849
2	0.08	-0.34852	12	64.10	-0.33781
3	0.17	-0.34777	13	124.10	-0.33762
4	0.33	-0.34729	14	244.10	-0.33738
5	0.58	-0.34663	15	424.10	-0.33739
6	1.08	-0.34608	16	664.10	-0.33724
7	2.08	-0.34536	17	964.10	-0.33714
8	4.08	-0.34408	18	1324.12	-0.33681
9	8.10	-0.34237	19	1440.40	-0.33675
0	16.10	-0.34027			



Void Ratio = 0.608 Compression = 5.9 %
D_0 = -0.34841 D_{50} = -0.34390 D_{100} = -0.33939
C_v at 3.2 min. = 0.08 ft.^{2}/day C_α = 0.002

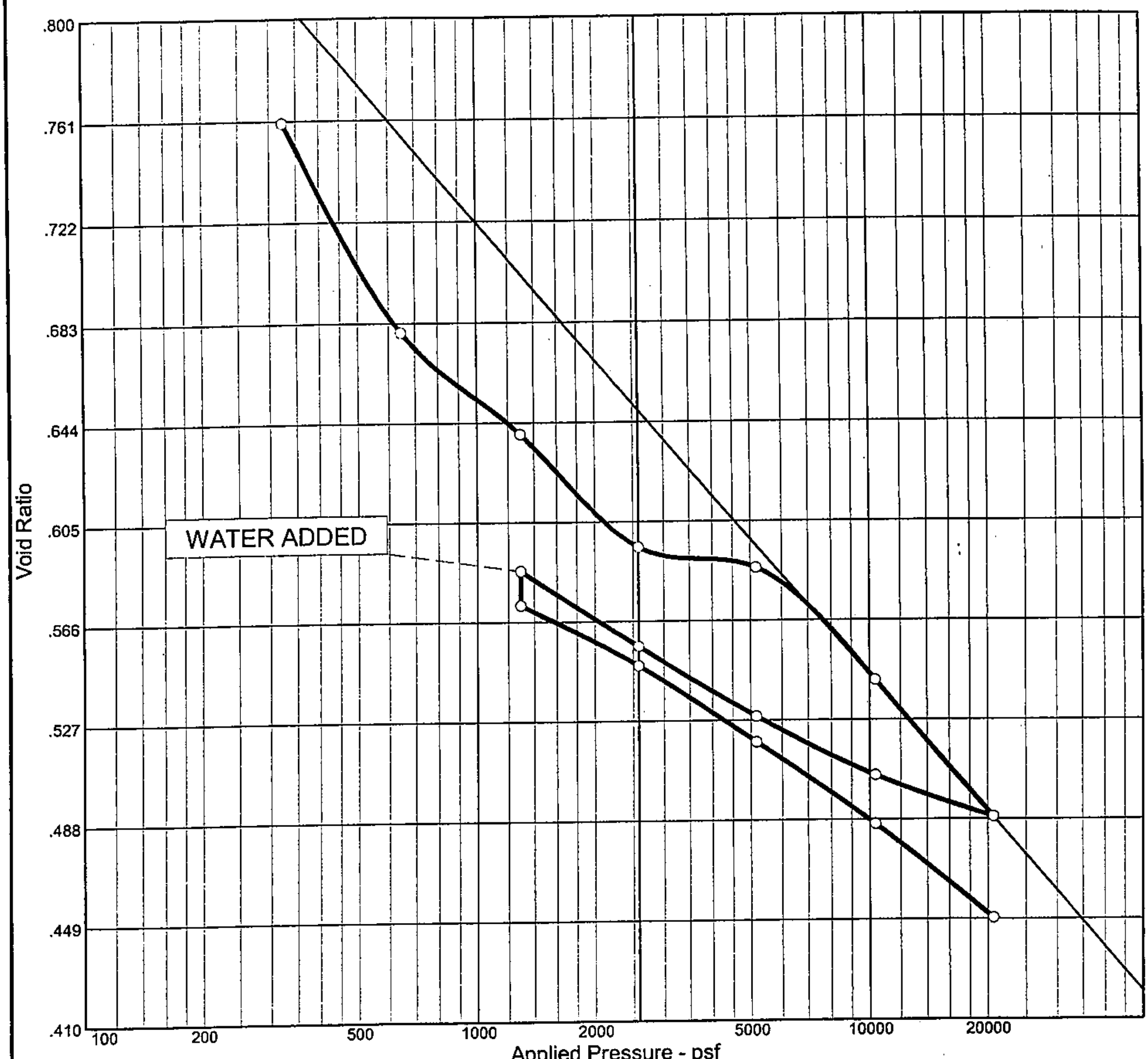
essure: 32000000 psf | TEST READINGS | Load No. 19

o.	Elapsed Time	Dial Reading	No.	Elapsed Time	Dial Reading
1	0.00	-0.33675	11	32.12	-0.32043
2	0.10	-0.33072	12	64.12	-0.32013
3	0.18	-0.33010	13	124.12	-0.31990
4	0.35	-0.32949	14	244.12	-0.31996
5	0.60	-0.32889	15	424.12	-0.31962
6	1.10	-0.32800	16	664.13	-0.31949
7	2.10	-0.32685	17	964.13	-0.31915
8	4.10	-0.32511	18	1324.13	-0.31914
9	8.10	-0.32313	19	1440.42	-0.31900
0	16.12	-0.32142			



Void Ratio = 0.568 Compression = 8.3 %
D_0 = -0.33084 D_{50} = -0.32571 D_{100} = -0.32058
C_v at 2.6 min. = 0.09 ft.^{2}/day C_α = 0.001

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
95.1 %	28.2 %	95.7	78	54	2.81	CH	A-7-6(56	0.833

MATERIAL DESCRIPTION

Project No. 0121- **Client:** TranSystems, Inc.

Project: SCI-823-0.00

Source: B-2 **Sample No.:** P-2B **Elev./Depth:** 12.5

DLZ

Remarks:

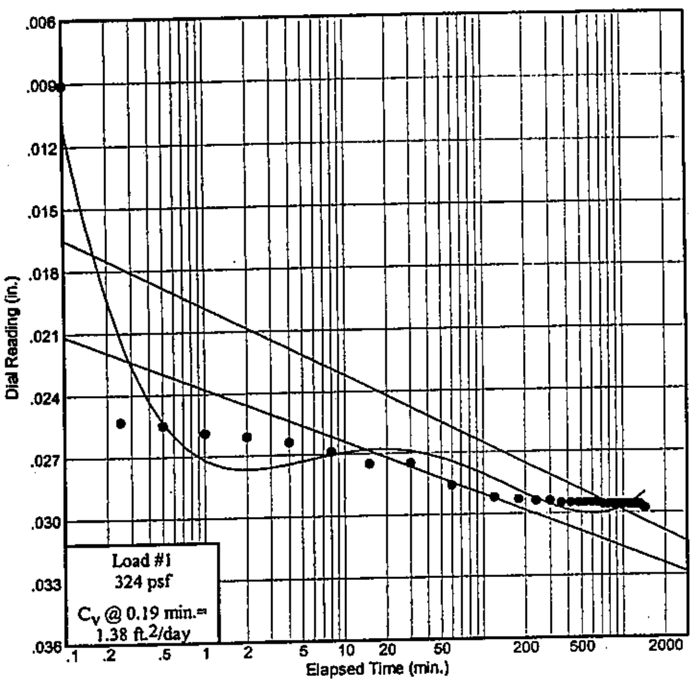
Specific Gravity= 2.81

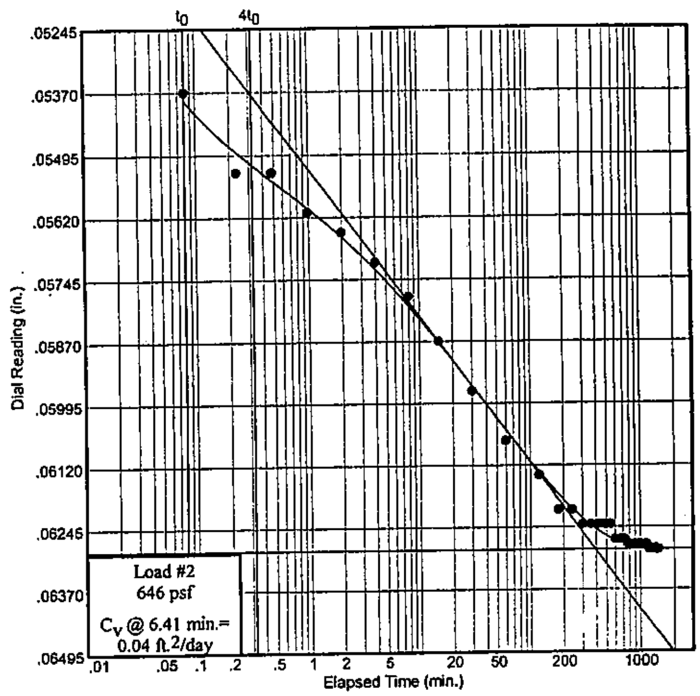
Figure

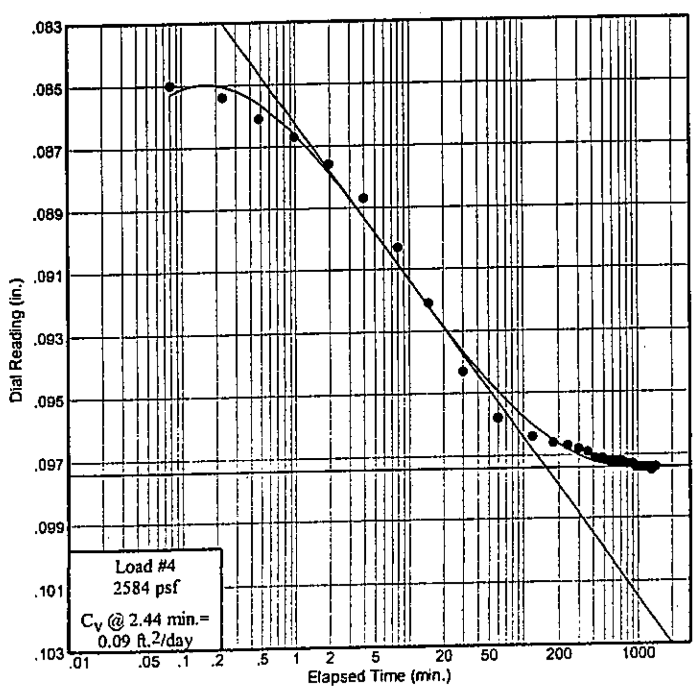
Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5







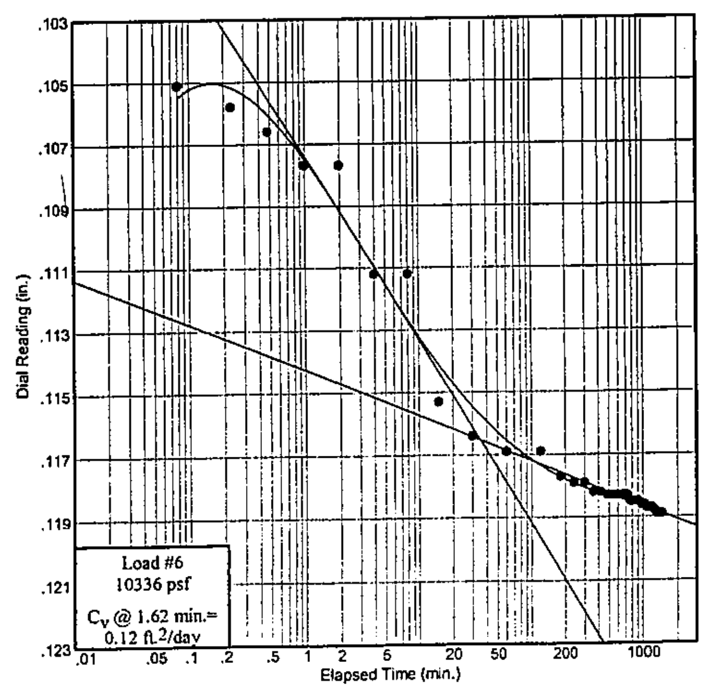


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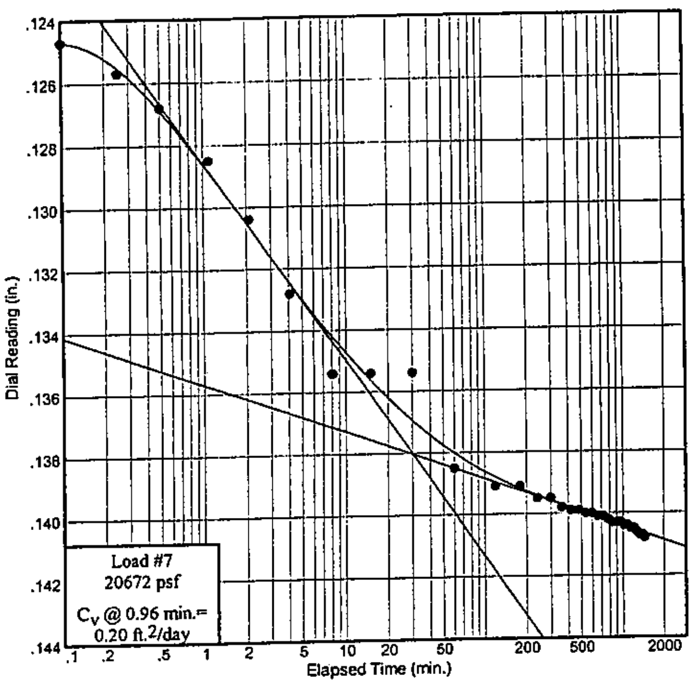
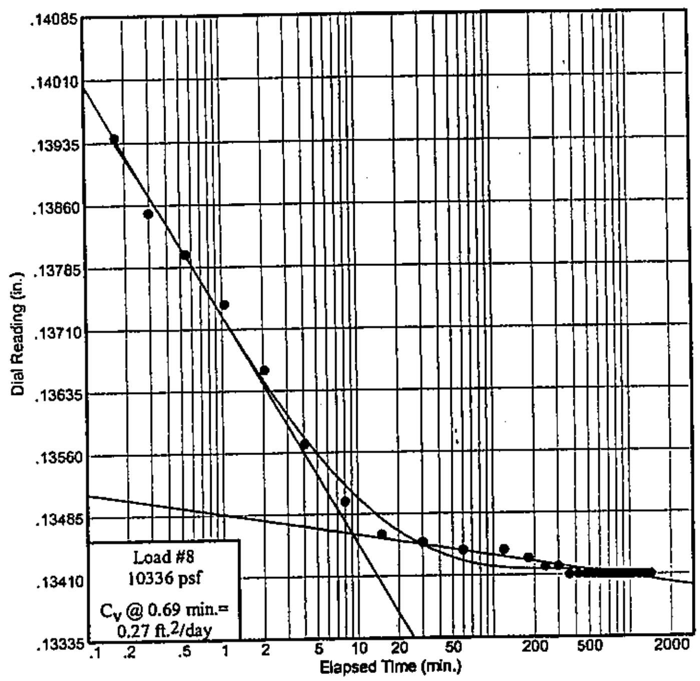
Figure

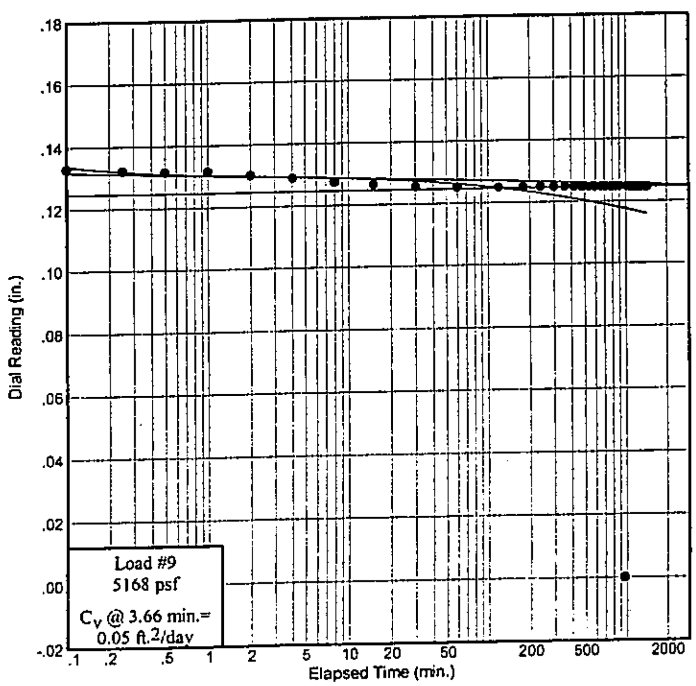
Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 Sample No.: P-2B Elev./Depth: 12.5







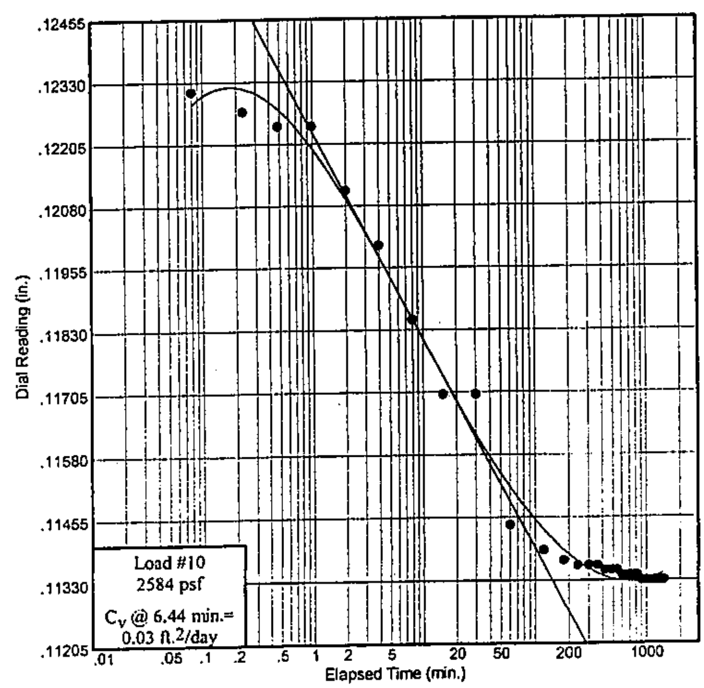


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Figure

Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5

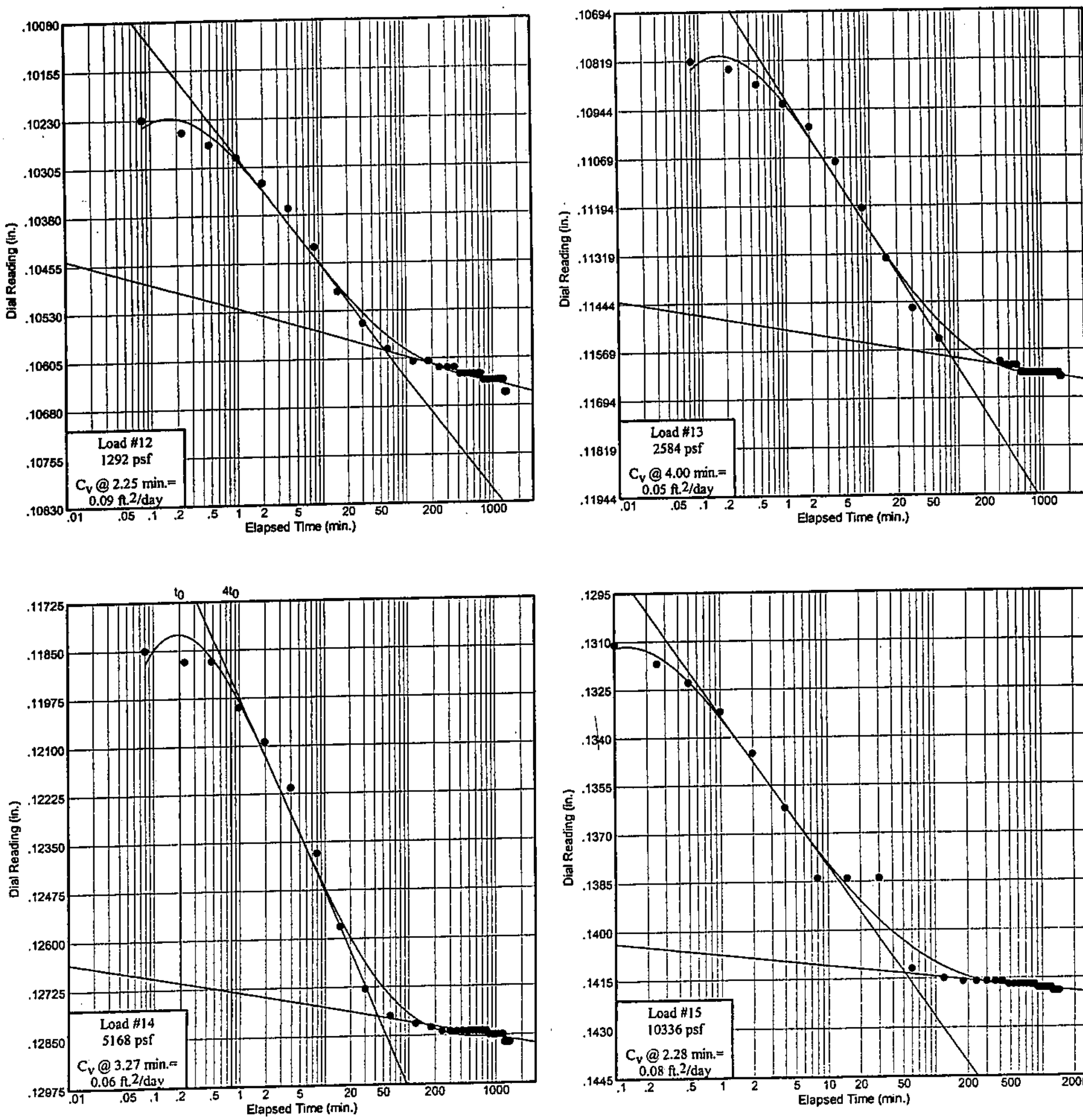

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Figure

Dial Reading vs. Time

Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-2 | Sample No.: P-2B | Elev./Depth: 12.5

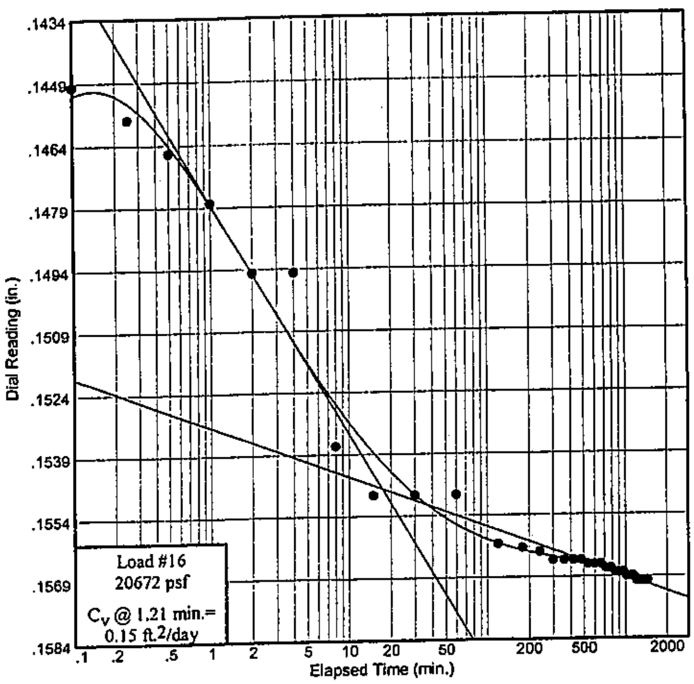

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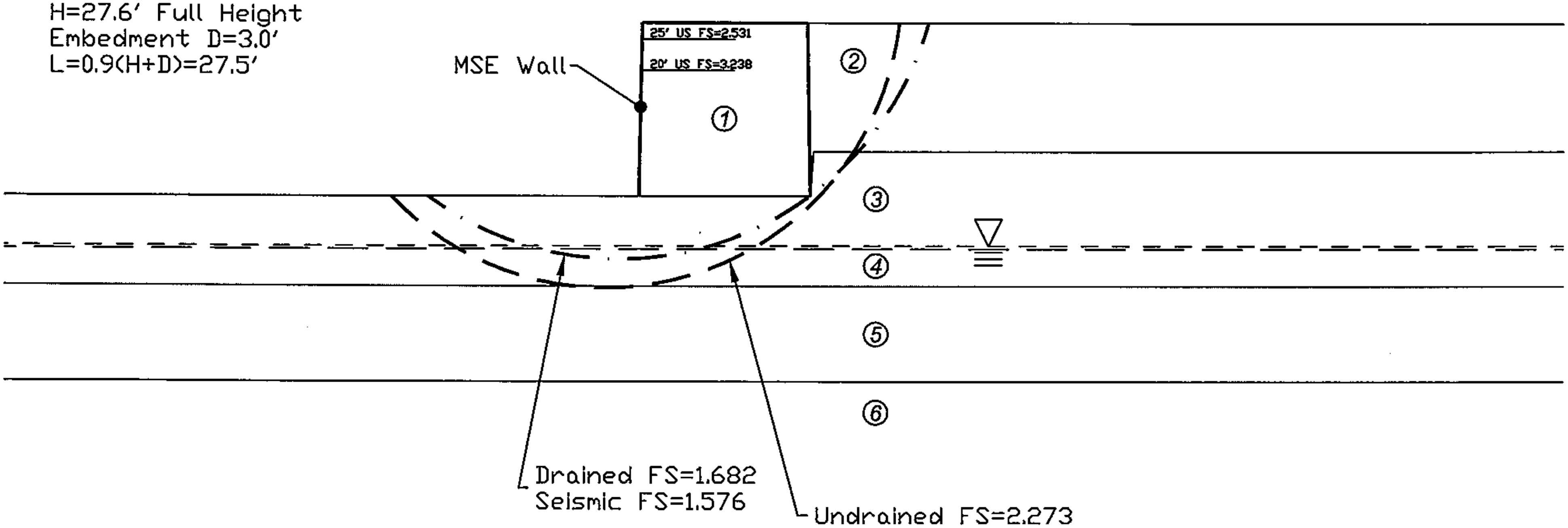
Figure

APPENDIX IV

MSE Wall Stability Analysis Results
MSE Wall Bearing Capacity and Stability Calculations
Settlement Calculations
MSE Wall Critical Pore Pressure Analysis

MSE Stability Analysis
B-1 Profile (7' Cut)
823 over Shumway Hollow RD
(Forward Abutment)
Sta. 384+75
Based on B-1
H=27.6' Full Height
Embedment D=3.0'
L=0.9(H+D)=27.5'

Material	Consistency	Soil Type	Undrained C (psf)	Undrained φ (deg)	Drained C' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Very Stiff	Clay	2000	0	0	28	125
Material 4	Stiff	Clay	1350	0	0	28	119
Material 5	M. Dense	Fine Sand	0	32	0	32	115
Material 6		Bedrock	10000	45	10000	45	145



823 OVER SHUMWAY HOLLOW ROAD
B-1 PROFILE 10' CUT - FORWARD ABUTMENT

MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 07/28/06

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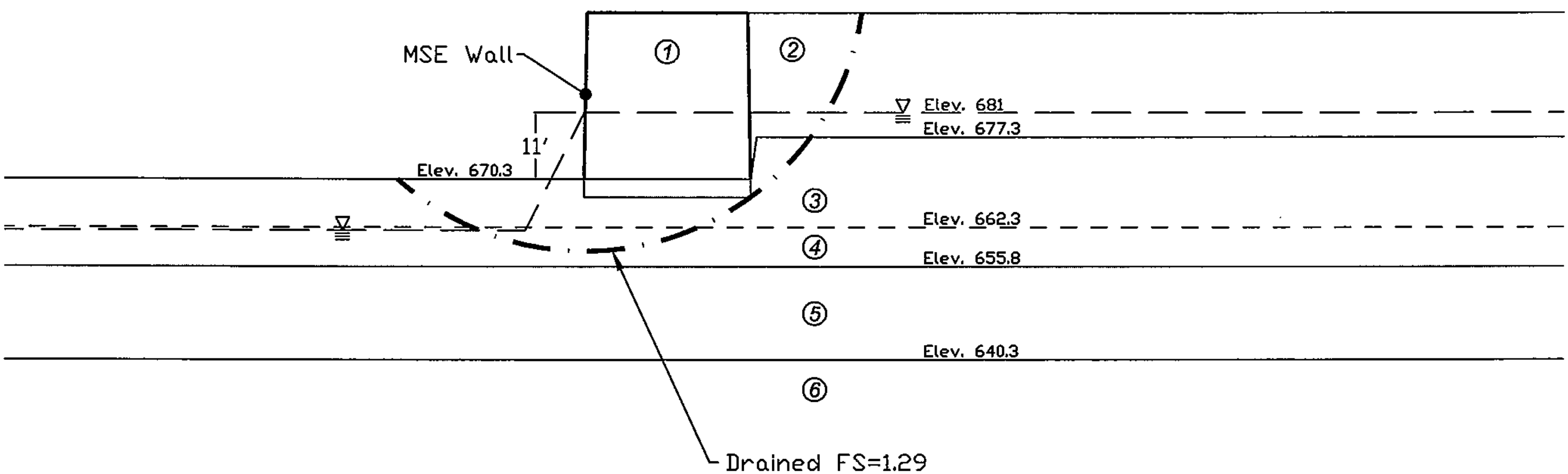
Stability Analysis - Drained Case
823 over Shumway Hollow RD
H=27.6' Full Height
3.0' EMBEDMENT
L=0.8(H+D)=27.5'
B-1 Profile (7' Cut)
(Forward Abutment)
Sta. 384+75

Determine Critical Pore Pressures
(Represented by Phreatic Surface)

			Undrained		Drained		
Material	Consistency	Soil Type	C (psf)	Φ (deg)	C' (psf)	Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Very Stiff	Clay	2000	0	0	28	125
Material 4	Stiff	Clay	1350	0	0	28	119
Material 5	M. Dense	Fine Sand	0	32	0	32	115
Material 6		Bedrock	10000	45	10000	45	145
Material 2	Compacted	Gran. Fill	0	34	0	34	120



823 OVER SHUMWAY HOLLOW ROAD
Determine Critical Pore Pressure

MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 8/22/06

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SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI 823-0.00 Portsmouth Bypass	SHEET NO.	1 OF 8
Item	MSE Wall Stability - Forward Abutment	COMP. BY	SJR DATE 07/27/06
823 over Shumway Hollow Road		CHECKED BY	DAA DATE 8/1/06
Based on Boring B-1			

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=30.6' (Full Height)
2 It is assumed that the bridge is supported on piles
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5

Wall Properties

H+D	=	32	feet
γ_{mse}	=	120	pcf
L	=	28.8	feet
L factor	=	0.90	Length factor-range (0.7 - 1.0)
ϕ	=	30	deg — Friction Angle of Embankment Fill

Foundational Soil Properties

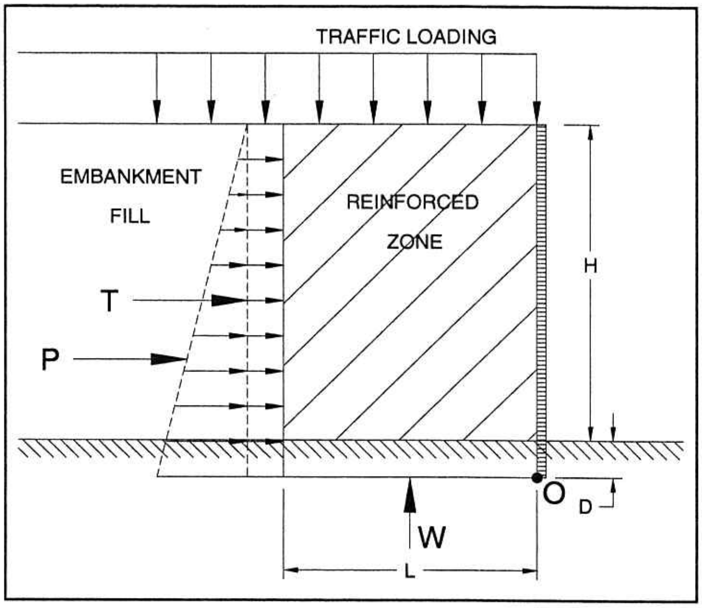
c	=	2000	psf	Cohesion
ϕ'	=	28	deg	Friction angle
ω_T	=	240	psf	Traffic loading

RESISTANCE AGAINST SLIDING ALONG BASE

Thrust: $P_a = K_a\left[\frac{1}{2}\gamma H^2 + \omega_T H\right]$

where; $K_a = \tan^2(45 - \frac{\phi}{2})$ K_a = 0.33

P_a = 22,810 lbs per foot of wall



Resistance: $P_r = W(0.67)(\mu)$ (Drained)

where; $\mu = \tan(\phi)$ 0.67μ = 0.36

0.67μ Max. = 0.35 {AASHTO, Bridge Design Manual, 303.4.1.1}

P_r = 38,707 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 57,600 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$ Calculated FS = **1.70** Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).

* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 1,592,525 lb-ft $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

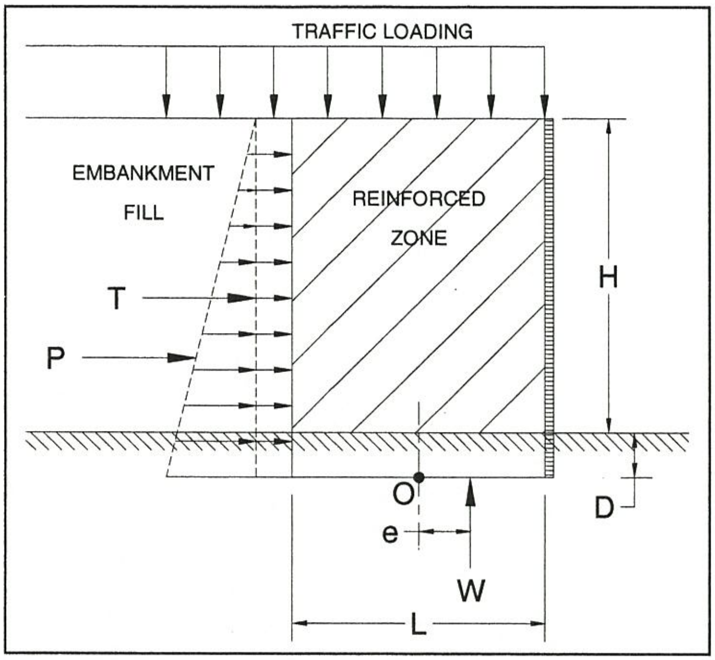
$\Sigma M_{overturning}$ = 256,819 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated FS = **6.20** Required FS = 2.00

Resistance Against Overturning is **OK**

BEARING CAPACITY OF A MSE WALL

Ref: {AASHTO; STANDARD SPECIFICATIONS FOR HIGHWAY BRIDGES, 17th Edition, 2002}



Effective Bearing Pressure

$$\sigma_v = \frac{W_t + W_{MSE}}{L - 2e}$$

σ_v = 4,812 psf

Ultimate undrained bearing capacity, q_{ult}

$$q_{ULT} = cN_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 10,453 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 4,181 psf

Factor of Safety = **2.17** **No Good**

Ultimate drained bearing capacity, q_{ult}

$$q_{ULT} = c'N_c + \sigma'_D N_q + \frac{1}{2}\gamma' B N_\gamma$$

q_{ULT} = 14,303 psf

$$q_{ALL} = \frac{q_{ULT}}{FS}$$

q_{ALL} = 5,721 psf

Factor of Safety = **2.97** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2000	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	28.8	ft	Length of MSE reinforcement
L factor	=	0.9		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	32	ft	
H	=	29	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	10.667	ft	Moment arm
Γ Wt	=	16	ft	Moment arm
B'	=	24.42	ft	
γ'	=	57.6	pcf	
W_t		6,912	lb/ft of wall	Weight from traffic
W_{mse}	=	110,592	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	25.80
N_q	1.00	N_q	14.72
N_γ	0.00	N_γ	16.72

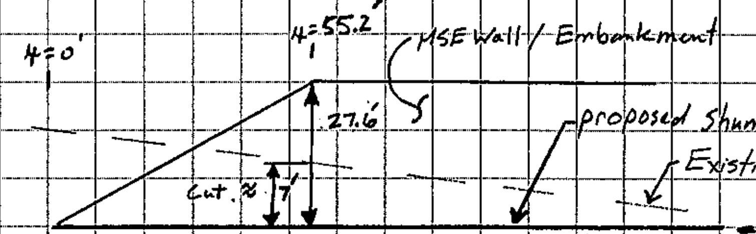
Eccentricity of Resultant Force

e = 2.19 ft

Kern

e < L/6 = 4.80 ft

Most Critical Soil Profile is at boring B-1 location.
At B-1, the maximum embankment height is approximately 27.6'
(See attached Cross-Section Drawing.)



Layer	Description	Depth	Notes
①	γ=120 pcf Compacted Granular Fill	-3.0'	Assume Incompressible
②	Clay! γ=119 pcf C_c=0.21 C_r=0.10 e_o=0.955 *Normally Consolidated	-14.5'	Consolidation Parameters taken from B-1, P-3
③	Fine Sand γ=115 pcf $\bar{N}'$≅10 C'=45	-21.5'	C' estimated from FHWA NHI-00-045
④	Gravel & Sand γ=120 pcf $\bar{N}'$≅10 C'=45	-30.7'	C' estimated from FHWA NHI-00-045

* The computer program EMBANK requires inputs for C_c, C_r, and e_o.
To evaluate the settlement of the granular layers we must calculate equivalent Consolidation parameters from C'.

$$\frac{1}{C'} = \frac{C_c}{1+e_o}$$ Say $e_o = 1.0$ in this case.

$$\frac{1}{C'} = \frac{C_c}{1+1.0} \rightarrow C' = \frac{2.0}{C_c} \rightarrow \boxed{C_c = \frac{2}{C'}}$$

When $C' = 45$, $C_c = \frac{2}{45} = 0.044$

For layers 3 and 4, Use $e_o = 1.0$ and $C_c = 0.044$

* From Embank, $\delta_c \simeq 8''$ - Maximum Settlement for MSE Wall using "End of Fill" Condition.

Differential Settlement: $\delta_{max} = 8''$ Total Wall length = 196'

$$\text{Differential Settlement} = \frac{8''\left(\frac{1'}{12''}\right)}{200.4'} = 0.003 = 0.3\% \ \checkmark \text{OKAY}$$

SH MSE

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿
INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name : SCI-823 Client : TranSystems
File Name : SH MSE Project Manager : Nix
Date : 7/27/10 Computed by : SJR checked DAA

Settlement for X-Direction

Embank. slope, x direc. = 55.20 (ft)	Height of fill H = 27.60 (ft)
y direc. = 55.20 (ft)	Unit weight of fill = 120.00 (pcf)
Embankment top width = 90.00 (ft)	p load/unit area = 3312.00 (psf)
Embankment bottom width = 200.40 (ft)	Foundation Elev. = 670.00 (ft)
Ground Surface Elev. = 673.00 (ft)	
Water table Elev. = 665.00 (ft)	Unit weight of Wat. = 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	3.0	-----	-----	-----	120.00	----	----
2	COMP.	11.5	0.210	0.100	0.000	119.00	2.65	0.95
3	COMP.	7.0	0.044	0.044	0.000	115.00	2.65	1.00
4	COMP.	9.0	0.044	0.044	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	11.50	664.25	997.45	997.45
3	7.00	655.00	1507.00	1507.00
4	9.00	647.00	1950.30	1950.30

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	10.00 Sett. (in.)	X = Stress (psf)	20.00 Sett. (in.)	X = Stress (psf)	30.00 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	50.61	0.32	311.04	1.75	612.10	3.09	913.88	4.20
3	138.79	0.07	345.73	0.17	615.98	0.28	901.33	0.38
4	206.97	0.10	392.19	0.19	632.28	0.29	892.53	0.39
		0.49		2.11		3.65		4.96

Layer	X = Stress (psf)	40.00 Sett. (in.)	X = Stress (psf)	50.00 Sett. (in.)	X = Stress (psf)	60.00 Sett. (in.)	X = Stress (psf)	70.00 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	1214.24	5.14	1505.79	5.94	1660.96	6.33	1671.92	6.35
3	1182.09	0.46	1429.63	0.54	1580.83	0.58	1633.03	0.59
4	1146.38	0.48	1363.89	0.55	1511.08	0.59	1585.79	0.61
		6.08		7.02		7.49		7.56

SH MSE

Layer	X = Stress (psf)	80.00 Sett. (in.)	X = Stress (psf)	90.00 Sett. (in.)	X = Stress (psf)	100.00 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.			
2	1673.38	6.36	1673.77	6.36	1673.86	6.36
3	1648.65	0.59	1653.87	0.59	1655.21	0.59
4	1618.14	0.62	1631.29	0.63	1634.95	0.63
		------		------		------
		7.57		7.58		7.58

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$\delta_{c_{max}} = 7.58''$

$\delta_{INSTANT} = 1.22''$

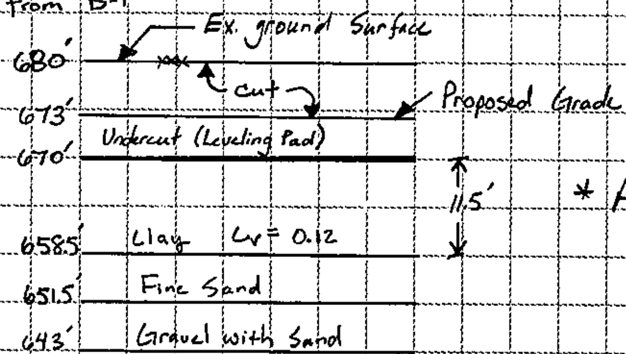
DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: Tran Systems / ODOT D-9
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Time-Rate of Consolidation
823 over Shumway Hollow - MSE

PROJECT NO. 0121-3070.03
SHEET NO. 6 OF 8
COMP. BY SJR DATE 7-28-06
CHECKED BY DAA DATE 8/1/06

* Based upon soil profile at boring B-1 (Most Critical).
Consolidation Testing performed on sample P-3 in boring B-1.

Profile from B-1



* Assume Double Drainage

Time-Rate of Consolidation: $t_{90} = \dfrac{T \cdot H_v^2}{C_v}$

for U = 90 (90% Consolidation) → T = 0.848

$$t_{90} = \frac{(0.848)(11.5'/2)^2}{0.12\ \text{ft}^2/\text{day}} = \underline{234\ \text{days}}$$ ← U=90% without Wick Drains.

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: Tran Systems
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Staged Construction Excess Pore Water Pressures
PROJECT NO.: 0121-3070.03
SHEET NO.: 7 OF 8
COMP. BY: SJK DATE: 8-1-06
CHECKED BY: DAA DATE: 8-1-06

*As per UTEXAS, the maximum staged embankment height is 25'.

Based upon this height, the excess pore water pressure is equal to the applied load at t=0 for a saturated clay.

at $t=0$ $u_e = 25'(120\ pcf) = 3{,}000\ psf$ *This will be the critical pore water pressure.

additional applied load 4.0' ; → $\Delta u_e = 4(120) = 480\ psf$

* Determine the amount of time to dissipate 480 psf of u_e

$$\frac{u_e}{\Delta\sigma_z} = \frac{3000-480}{3000} = 0.84$$

From Graph (attached) $T_v \simeq 0.16$ when $\frac{u_e}{\Delta\sigma_z} = 0.84$ $\frac{Z_{dr}}{H_{dr}} = 1.0$ (Center of clay layer)

$$T_v = \frac{C_v \cdot t}{H_{dr}^2}$$

where from B-1: $C_v = 0.12\ ft^2/day$
(Double Drainage) $H_{dr}^2 = 11.5'/2 = 5.75'$

$$t = \frac{T_v \cdot H_{dr}^2}{C_v}$$

$$t = \frac{(0.16)(5.75')^2}{0.12\ ft^2/day} = \boxed{44\ days}$$

* At 44 days the additional 4.0' of embankment may be constructed while maintaining FS=2.5.

Time (days)	u_e (psf)	
t=0	3000	- End of Construction of 25' Stage
t=44	2520	- Excess Pore Water Pressures have Dissipated.
t=44	3000	- Add additional 4.0' to Completion height of 29.0'.

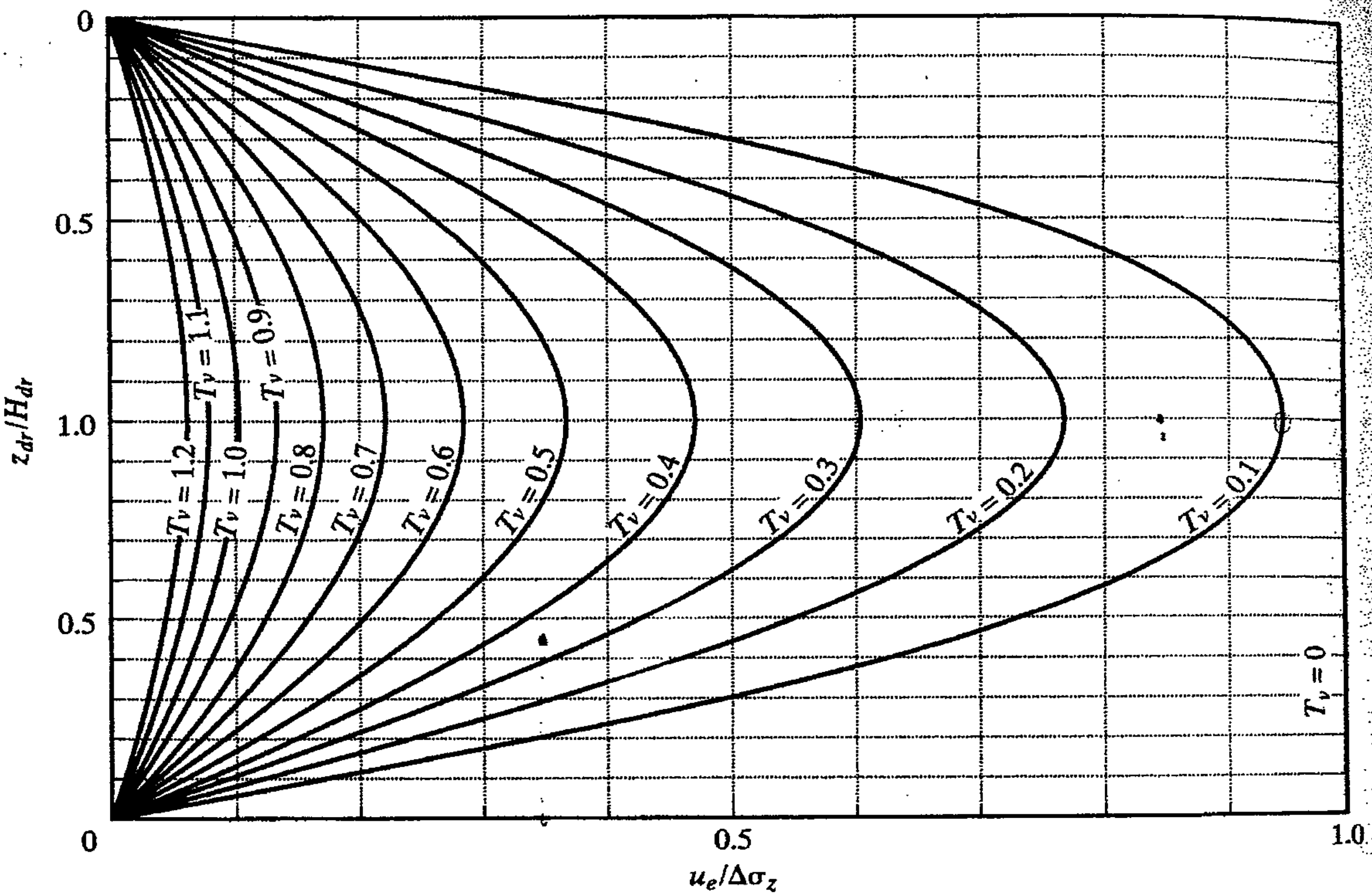


Figure 12.4 $u_e/\Delta\sigma_z$ for various values of T_v with double drainage. For the single drainage case, use only the upper half of this diagram.