Report of:

Subsurface Exploration
SR 823 Bridge Over Lucasville-Minford Road (CR 28)
(SCI-823-1018 L&R)
SCI-823-10.13 Portsmouth Bypass
Scioto County, Ohio

STRUCTURAL ENGINEERING
FEB 29 2008
RECEIVED

Prepared for:

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Dublin, Ohio 43017

Ohio Department of Transportation
District 9

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DLZ Job No. 0121-3070.03

June 27, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

SR 823 BRIDGE OVER LUCASVILLE-MINFORD ROAD (CR 28)

(SCI-823-1018 L&R)

PROJECT SCI-823-10.13 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

For:

TranSystems Corporation
5747 Perimeter Drive, Suite 240
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SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of evaluations of foundations for the proposed bridges at the above-referenced project location. The findings included in this report pertain to the structures at proposed SR 823 over Lucasville-Minford Road only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing twin structures for the proposed SR 823 over Lucasville-Minford Road (CR 28). The two structures as planned, are both three-span structures with spill through slopes at the abutments. The proposed spill through slopes are characterized by 2H:1V slopes.

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 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 pile foundations are preferred to support the abutments of the proposed structures. It is understood that spill through slopes will be used at the abutments.

Furthermore, it is assumed that the maximum height of the embankment at stations 537+32.50 (Rear Abutment) and 539+92.25 (Forward Abutment) will be approximately 43.0 and 40.0 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of SR 823 and the existing grade, as shown on the cross section drawings provided by TranSystems Corporation.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concepts are changed or differ 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 of four structural borings. The structural borings (TR-11 through TR-14) were drilled between June 7, 2004 and March 17, 2005. The as-drilled boring locations are shown on the Structure Plan and Profile drawing presented in Appendix I. Boring logs for borings TR-11 through TR-14 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. Soils at the proposed bridge site are composed primarily of alluvial and lacustrine soils. These soils are moderately thick, 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. Bedrock within the structure area is primarily sandstone of the Cuyahoga Formation of Mississippian age. Bedrock of the Mississippian Logan and the Pennsylvanian Breathitt Formation can be found on the slopes north and east of the structures roughly above elevations 800 and 1020, respectively. In the area of the structure, the bedrock was covered by moderately thick overburden ranging in thickness between 33 and 43 feet.

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

The results of this investigation indicated that soil conditions at the site were somewhat uniform. In general, the subsoil stratigraphy consisted of shallow surficial materials consisting of topsoil underlain by native cohesive and granular soil deposits.

Boring TR-11 was drilled for the north (forward) abutment. Boring TR-14 was drilled for the south (rear) abutment, while borings TR-13 and TR-12 were drilled for Piers 1 and 2, respectively.

All borings encountered surficial material consisting of 4 to 6 inches of topsoil. The topsoil was underlain by native soil deposits. All borings encountered native cohesive and granular soil deposits below the surficial material except boring TR-12 where possible fill was encountered consisting of medium stiff to stiff sandy silt (A-4a).

The cohesive deposits generally consisted of stiff to very stiff silt and clay (A-6a), stiff sandy silt (A-4a), stiff to very stiff silty clay (A-6b), and soft to very stiff clay (A-7-6), while the granular soil deposits consisted mainly of very dense sandy silt (A-4a). The native soil deposits extended to an approximate depth ranging between 33.5 and 43.0 feet below the ground surface where bedrock was encountered.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in all borings. A layer of severely weathered rock, ranging in thickness between 3.0 to 4.0 feet, was encountered above the more competent cored bedrock in borings TR-11 and TR-12, while a 10.5-foot thick severely weathered siltstone layer was encountered in boring TR-14. The bedrock consisted of medium hard, broken to highly fractured sandstone and siltstone. The amount of rock recovered in each core run varied between 51 and 100 percent. In boring TR-13, core loss was encountered between approximate depths of 40.0 and 44.9 feet below the ground surface corresponding to elevations 673.5 and 668.9, respectively. The rock quality designation (RQD) of the bedrock ranged between 25 and 92 percent with an average of 71 percent, indicating fair rock.

4.2.3 Groundwater Conditions

Seepage was encountered in all borings between approximate depths of 10.5 and 38.5 feet. Measurable water levels in the borings prior to rock coring were encountered only in boring TR-14 at an approximate depth of 24.8 feet. Water was used during rock coring operations and masked any seepage zones that might exist in the rock. Measurable water levels, upon the completion of coring, were present in all borings between approximate depths of 8.9 and 28.6 feet.

It should be noted that groundwater levels may fluctuate with seasonal variations and following periods of heavy or prolonged precipitation, and therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be needed to obtain a more accurate estimate of the groundwater table elevation.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is understood that the proposed bridges will consist of three spans utilizing spill through abutment slopes (2H:1V). It is understood through comments from ODOT's Office of Structural Engineering (OSE) that integral abutments supported on piles are preferred to support the proposed structure. Consequently, driven piles have also been considered for the support of the piers.

In addition to piles, drilled shafts have also been considered to support the proposed structure. Given the abutment type and settlement considerations, spread footings were not considered a reasonable alternative to deep foundations, and therefore were not considered in this report.

A summary of the bridge foundation recommendations is presented in Table 1. Detailed recommendations for the bridge foundations and embankment construction are presented in the following sections, and calculations are presented in Appendix IV.

It should be noted that the bedrock surface varies across the project area. The approximate bearing elevations presented below indicate the elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 1-Summary of Foundation Recommendation

Structural Element	Structure / Boring	Existing Ground Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	Left & Right / TR-14	713.6	HP 14x73 piles	674.6	Pile Capacity+
			Drilled Shafts	664.1	40 ksf*
Pier 1	Left & Right / TR-13	713.5	HP 14x73 piles	673.8	Pile Capacity+
			Drilled Shafts	668.0	40 ksf*
Pier 2	Left & Right / TR-12	713.0	HP 14x73 piles	673.2	Pile Capacity+
			Drilled Shafts	668.0	40 ksf*
Forward Abutment	Left & Right / TR-11	722.5	HP 14x73 piles	678.6	Pile Capacity+
			Drilled Shafts	670.5	40 ksf*

* Includes 5-foot socket into competent rock, end bearing capacity only.

+ Allowable pile capacity may be reduced by downdrag forces.

See following section for additional information regarding downdrag forces.

5.1 Bridge Foundation Recommendations-Rear and Forward Abutments

It is understood through comments from OSE that pile foundations are preferred for the support of the abutments. Additionally, it is understood that integral abutments are preferred due to the anticipated downdrag forces, which would be detrimental to battered piles typically used with semi integral abutments.

Pile foundation analyses indicate that displacement type (pipe) piles cannot fully develop resistance prior to encountering bedrock. Prior to contacting bedrock, analyses indicate that 14-inch CIP piles could develop allowable capacities of 41 and 26 tons at the rear and forward abutments, respectively. Consequently, it is recommended that H-piles driven to refusal on bedrock be used to support the structures. The full structural capacity of the piles can be used in this configuration. However, it should be noted that the full allowable capacity of the piles may be reduced by downdrag forces. If piles are driven to refusal on bedrock, it is recommended that reinforced piles points be used to protect the pile while driving.

Significant consolidation of the foundation soils is expected at the abutments under the loading of the new embankment fill. In order to minimize the downdrag forces that would develop along the length of the piles during consolidation of the foundation soils, it is recommended that the embankment be fully constructed to the proposed grade elevation using staged construction and allowed to consolidate prior to driving piles. To accelerate the consolidation of the foundation soils, it is recommended that wick drains with a 2 to 3 foot granular blanket be installed prior to placing any embankment fill. The embankment should be constructed, and wick drains and instrumentation should be installed as outlined in the Report of Subsurface Investigation for the Lucasville-Minford Road Interchange, dated November 29, 2006. The details of the embankment stability are discussed further in section 5.4 of this document.

If, however, some downdrag force can be tolerated, piles can be driven after a shorter waiting period (prior to full consolidation of foundation soils). Based upon the soil profile encountered by boring TR-11, the total primary consolidation was estimated to be approximately 28 inches. Downdrag forces were estimated given various waiting periods prior to driving piles, and are presented in Table 2. Because piles are driven to refusal on bedrock at this site, piles cannot be driven to a higher capacity. Consequently, the downdrag forces will reduce the allowable structural capacity of the piles. Estimates of downdrag forces are provided for HP 14x73 piles. Estimates for other pile sizes can be provided upon request.

Table 2- Unfactored Downdrag Forces Applied to Driven Piles

Waiting Period (days)*	Percent Consolidation	Downdrag Force (kips)
0	0	152
90	78.4	132
180	93.3	92
280	98.6	0

*Waiting period after construction of approach embankment, prior to driving piles. Estimates provided for HP 14x73 piles.

Prior to installing the piles, the bridge approach embankments behind the abutments should be constructed up to subgrade elevation for a minimum distance of 200 feet behind the abutments. The foundation soils should then be allowed to consolidate a sufficient amount to reduce the downdrag forces to acceptable levels, depending on the required structural design capacity of the piles.

The ODOT construction representative may adjust the required waiting period (for a specified degree of consolidation) in the field based upon observations of instrumentation installed for the purposes of monitoring the consolidation of the foundation soils.

As an alternative to pile foundations, drilled shafts may also be considered for the support of the abutments. Based upon the subsurface conditions encountered by the borings, it is recommended that drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 1. It should be noted that drilled shaft foundations will also be subjected to downdrag forces. Negative skin friction in the consolidating soil layers will be mobilized as a downdrag force. Design values for these downdrag forces can be provided upon request.

5. 2 Bridge Foundation Recommendations-Piers 1 and 2

Based upon the subsurface conditions encountered by the borings drilled for this structure, it is recommended that deep foundations be used to support Piers 1 and 2 of the proposed structure.

Pile foundation analyses indicate that displacement type (pipe) piles cannot fully develop resistance prior to encountering bedrock. Prior to contacting bedrock, analyses indicate that 14-inch CIP piles could develop allowable capacities of 41 and 26 tons based on borings drilled for the rear and forward abutments, respectively. Similar capacities are anticipated at the piers. Consequently, it is recommended that H-piles driven to refusal on bedrock be used to support the structures. The full structural capacity of the piles can be used in this configuration. However, it should be noted that the full allowable capacity of the piles may be reduced by downdrag forces. If piles are driven to refusal on bedrock, it is recommended that reinforced piles points be used to protect the pile while driving.

Total primary consolidation at the toe of the proposed embankments (pier locations) was estimated to be approximately 2.5 inches under the loading of the new approach embankments. As per the Structure Plan and Profile Drawing, it is anticipated that battered piles will be utilized to support the piers. Battered piles are particularly sensitive to the effects of downdrag forces. To minimize the amount of negative skin friction that is mobilized and acting on the piles as a downdrag force, it is recommended that piles not be installed until at least 84 percent of primary consolidation has occurred.

Time-rate of consolidation calculations indicate that a waiting period prior to driving piles of approximately 110 days will be required to achieve 84 percent of primary consolidation if the approach embankments are constructed as described in Section 5.1 of this document. The ODOT construction representative may adjust the required waiting period in the field based upon observations of instrumentation installed for the purposes monitoring the consolidation of the foundation soils.

As an alternative to pile foundations, drilled shafts may also be considered for the support of the piers. Based upon the subsurface conditions encountered by the borings, it is recommended that drilled shafts be socketed a minimum of 5 feet into competent rock. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 40 ksf (20 tsf). Recommended bearing elevations for drilled shaft foundations are presented in Table 1. It should be noted that drilled shaft foundations will also be subjected to downdrag forces. Negative skin friction in the consolidating soil layers will be mobilized as a downdrag force. Design values for these downdrag forces can be provided upon request.

5.3 General Drilled Shaft Recommendations

For end-bearing drilled shafts, it is recommended that skin friction in the overburden soil/fill and shallow rock socket be neglected. The bearing surface should be clean and free of loose material and water prior to placement of concrete. The drilled center-to-center spacing of drilled shafts should generally be no less than 2.5 times their diameter. A qualified representative of the Geotechnical Engineer should field verify that the drilled shafts are founded on competent bearing materials and the installation procedures meet specifications.

If adequate capacity cannot be developed with reasonable shaft diameter, drilled shafts should be designed as friction-type shafts. Neglecting the overburden, upper two feet and bottom length equal to one diameter of the socket, allowable sidewall shear stress/adhesion of 5,000 pounds per square foot (psf) may be used for the rock socket. If designed as friction-type shafts, the shafts should be designed such that design loads are carried entirely by the rock socket resistance, ignoring any end bearing.

Shafts that are installed as friction-type piles must have good sidewall contact with the concrete with preferably rough sides. If any shaft is allowed to sit over 12 hours filled with fluid (water or slurry), the potential for sidewall softening develops. This is especially true with the rock sockets and granular materials. The bedrock material encountered across the site contains siltstone and argillaceous sandstone that could deteriorate quickly when exposed to water or left to desiccate, losing its strength quickly. If it is anticipated that a drilled shaft excavation will be allowed to remain open for longer than 12 hours, the shaft excavation should be drilled at least 6 inches smaller in diameter and reamed to the design diameter immediately prior to placement of concrete. If a drilled shaft excavation does not have concrete placed within 12 hours of completion of the excavation, the shaft should be oversized 6 inches in diameter.

Drilled shafts that are end bearing and are allowed to remain open for more than 12 hours should be drilled short by at least 12 inches and reamed out to the design bearing depth immediately prior to placement of concrete to prevent softening of the bearing material. If a drilled shaft excavation does not have concrete placed within 12 hours of completion of the excavation, the shaft should be extended 12 inches in depth prior to the placement of concrete.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although no significant seepage was encountered by any of the borings drilled for this project, water could flow into the drilled shaft excavations during installation, particularly within wet zones that may be present in the rock. It is anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

5.4 Embankment Stability Analysis

Based on the borings drilled for the structure, the embankment foundation was assumed to consist of 13.0 feet of stiff silty clay (A-6b) or clay (A-7-6) to a depth of 13.0 feet below the ground surface. Beneath this layer, soft to medium stiff clay (A-7-6) was encountered to a depth of 37 feet below the ground surface. Below this layer, stiff silt and clay (A-6a) was encountered to a depth of 43 feet, at the top of severely weathered bedrock.

The maximum embankment height within the limits of the structure or approach embankment is assumed to be 43.0 feet, near the rear abutment of the proposed structure. Consequently, a height of 43.0 feet is assumed for the analyses at this site.

Stability analyses were performed to determine the stability of the embankments and spill through slopes within the limits of the proposed structure. It should, however, be noted, that the analyses performed for embankments within the interchange have been found to be slightly more critical than those for the spill through slopes. The developed cross sections were characterized by 2H:1V side slopes. Consequently, it is recommended that the approach embankments and spill through slopes be constructed according to the recommendations outlined in the Report of Subsurface Investigation for the Lucasville-Minford Road Interchange, dated November 29, 2006. Some of the results contained within the cited interchange report are reproduced in this report for reference, for additional information, please refer to the cited document.

Stability analyses performed for the interchange embankments indicate that undrained stability is a concern at this site. Subsequent analyses have indicated that the embankments could be constructed with staged construction while using wick drains to accelerate consolidation times. For the purposes of the recommendations presented in this document, it is assumed that staged construction and wick drains will be used to construct the embankments contained within the interchange area. If a construction method or sequence other than that which was assumed is used, DLZ should be informed so that the analyses and recommendations presented here can be revised as necessary.

Slope stability analyses were performed for the proposed State Route 823 near station 538+00 in order to determine the height of the embankment that could be initially constructed assuming no significant removal of existing poor soils. Undrained analyses were performed for a 32-foot height first stage embankment, assuming a cross section characterized by 2H:1V side slopes. The critical factor of safety assuming end-of-construction (undrained) conditions was found to be 1.32. This critical factor of safety meets the generally recommended minimum factor of safety of 1.30 for highway embankments.

Drained analyses were then performed for the 32-foot stage embankment assuming excess pore pressures in the foundation soils. This analysis would reflect the conditions during construction with instrumentation to verify the subsurface conditions. The pore water pressure head during stage 1 and stage 2 construction should not rise above the existing ground surface elevation. If pore pressures rise above this level, fill placement should halt immediately. Construction may continue after pore pressures in the clay layers have dissipated. The waiting period time between stage 1 (32-foot embankment) and stage 2 (45-foot embankment) should be more than 60 days (assumes wick drains) to allow dissipation of the excess pore water pressures.

The excess pore water pressures will dissipate near the toe of the new embankment due to the decreasing embankment load. In the analyses, it was assumed that the excess pore pressures dissipated along the outside slope of the new embankment. The assumed excess pore pressure distribution is shown on the stability analyses results in Appendix IV. Based on the findings of these analyses, it is recommended that at least 70 percent of the excess pore water pressures be allowed to dissipate before the remainder of the embankment is constructed, this corresponds to an approximate waiting period of 60 days, as cited above. The results of the stability analyses are presented in the Appendix IV.

Settlement analyses indicate that the embankment will undergo an approximate settlement of 28 inches. It is anticipated that wick drains and staged construction will be used to expedite the consolidation process. Embankment settlements and pore water pressures should be monitored during construction using settlement platforms and piezometers. The settlement platforms should be installed at representative locations as approved by the ODOT construction representative. Copies of the instrumentation plan from the interchange report are included in Appendix V for reference. Note that the locations shown from piezometers and settlement plates 3 and 4 (P-3, P-4, S-3, S-4) have been adjusted slightly so they are under the maximum height of the embankment.

5.5 Groundwater Considerations

Water seepage was encountered in all borings between approximate depths of 10.5 and 38.5 feet below the ground surface. Measurable water levels in the borings prior to rock coring were encountered only in boring TR-14 at an approximate depth of 24.8 feet. Representative final water levels could not be obtained due to the use of water during rock coring operations. Excavations for the pier foundations are expected to be limited to ten feet or less, and will likely only encounter minor seepage in the soil layers. However, shafts extending below the top of rock may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage, water flow, or precipitation that may enter any excavations.

5.6 General Earthwork Recommendations

The proposed alignment of SR 823 over Lucasville Minford Road (CR 28) traverses a relatively flat area. Consequently, fill placement will be required to construct the approach embankments for the bridges. The maximum fill anticipated is approximately 43 feet.

Between 4 and 6 inches of topsoil were encountered at the ground surface. All topsoil and vegetation within the footprint of the new embankment and roadway should be removed prior to new fill placement. All pavement, and organic soil within 3 feet of subgrade level should also be removed prior to placing fill or pavement materials.

Organic soils were not encountered in any of the borings. However, if organic soils are encountered, it is recommended that at least the top three feet of subgrade soil be removed prior to the construction of the new embankment. Overexcavation may need to be deeper if organic soils are encountered at depths greater than three feet.

The embankments should be constructed in accordance with ODOT Items 203. It is anticipated that the embankments will be constructed with side slopes of 2H:1V or flatter. Based on the materials encountered by the borings, the foundation soils are considered adequately stable under the proposed embankment loads, if constructed as presented in the Report of Subsurface Investigation for the Lucasville-Minford Road Interchange, dated November 29, 2006.

Excavations deeper than 5.0 feet must be sloped or shored to protect workers entering the excavations. Refer to OSHA regulations (29CFR Part 1926) concerning sloping and shoring requirements for excavations.

Relative to the footing excavations, the following additional recommendations are presented:

1. All footings should be founded deep enough for frost protection, considered to be 36 inches in this area.

2. Excavation bottoms should be examined by the geotechnical engineer prior to placement of reinforcing steel and concrete in order to determine the suitability of the supporting soils.

3. If the footings are not supported by piles/drilled shafts, excavations should be undercut to suitable bearing material if such material is not encountered at the planned footing level. Such undercuts may be backfilled with a lean mix concrete (1,500 psi @ 28 days) or footing concrete.

4. All footing excavations should be cut to stable side walls and flat bottoms with the bottoms comprised of firm soil undisturbed by the method of excavation or softened by standing water. Concrete should be placed the same day that the footings are excavated.

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

Dorothy A. Adams

Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

sjr

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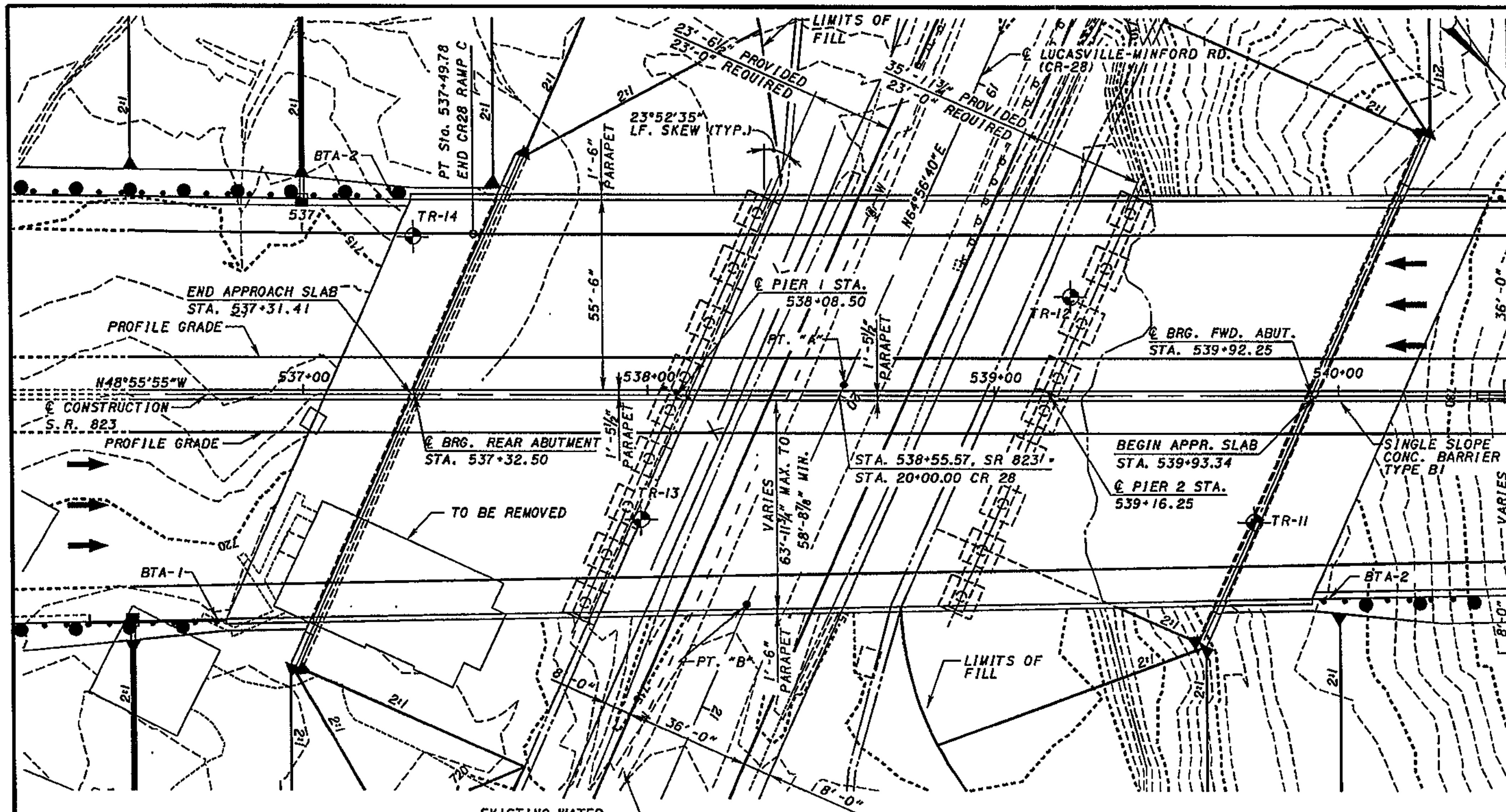
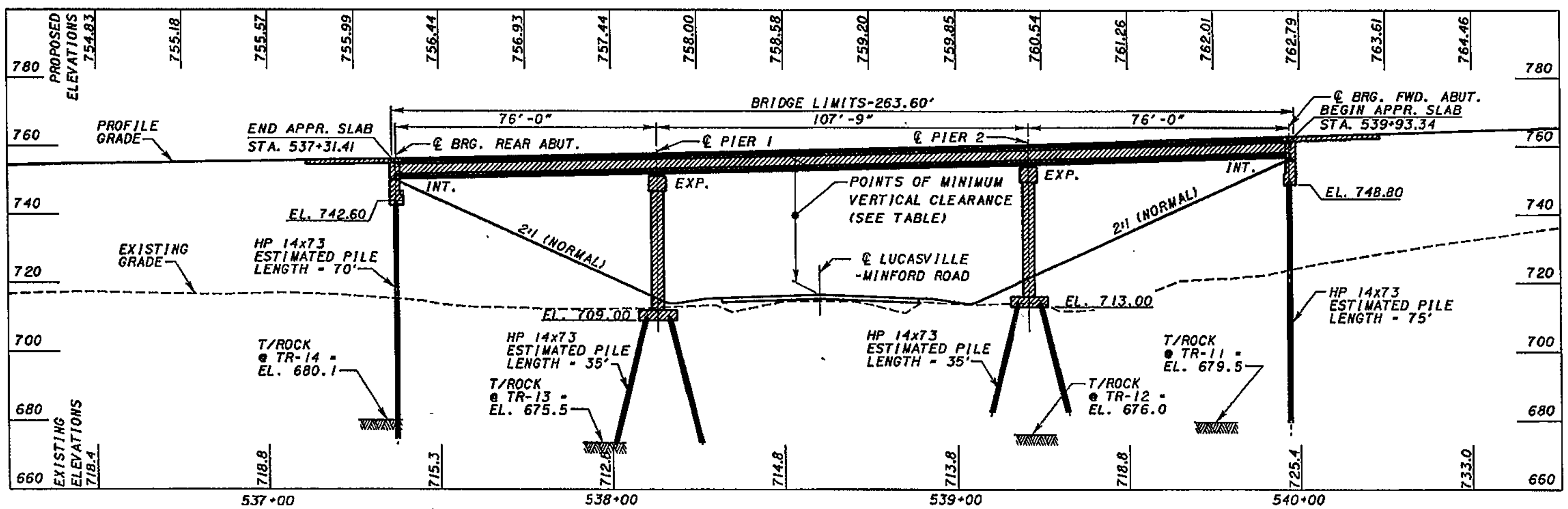
APPENDIX I

Structure Plan and Profile Drawing – 11”x17”



PLAN



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

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

TRAFFIC DATA

(SR 823)

CURRENT YEAR ADT (2010) = 19,800
CURRENT YEAR ADTT (2010) = 4752
DESIGN YEAR ADT (2030) = 26,000
DESIGN YEAR ADTT (2030) = 6240

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	36.07'	34.03'
REQUIRED	15.00'	15.00'

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS

LOCATION	STATION	OFFSET
REAR ABUT.	536+73.04	66.00' RT.
REAR ABUT.	537+27.38	57.00' LT.
FWD. ABUT.	539+96.07	59.70' RT.
FWD. ABUT.	540+47.82	57.00' LT.

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
3. THE PROPOSED PROFILE GRADE IS WITHIN BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ = BORING LOCATION

1300' VERT. CURVE DATA

P.V.I. STA. = 535+00.00
P.V.I. ELEV. 742.19
G_1 = -2.90%
G_2 = +4.00%

PROPOSED STRUCTURE

TYPE: THREE SPAN, 60" MODIFIED AASTHO TYPE 4 PRESTRESSED CONCRETE I-BEAM WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY INTEGRAL ABUTMENTS AND CAP AND COLUMN PIERS

SPANS: 74'-9½", 105'-4", 74' 9½" (C/C BRG.)

ROADWAY: 55'-6" T/T OF BARRIER

LOADING: HS-25, ALTERNATE MILITARY LOADING AND FWS = 60psf

SKEW: 23°52'35" LF

CROWN: 0.016 FT/FT

ALIGNMENT: TANGENT

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (25' LONG)

LATITUDE: 38°51'48" N

LONGITUDE: 82°53'45" W

DESIGN AGENCY TranSystems

DESIGNED PJP	DRAWN PJP	REVIEWED JRC	DATE 11/30/06
CHECKED MSL	REVISED		STRUCTURE FILE NUMBER 7306547

SCIOTO COUNTY
STA. 537+31.41
STA. 539+93.34

SITE PLAN
BRIDGE NO. SCI-823-1018 L
SR 823 OVER LUCASVILLE-MINFORD ROAD (CR-28)

SCI-823-10.31
PID 79977

1 / 6

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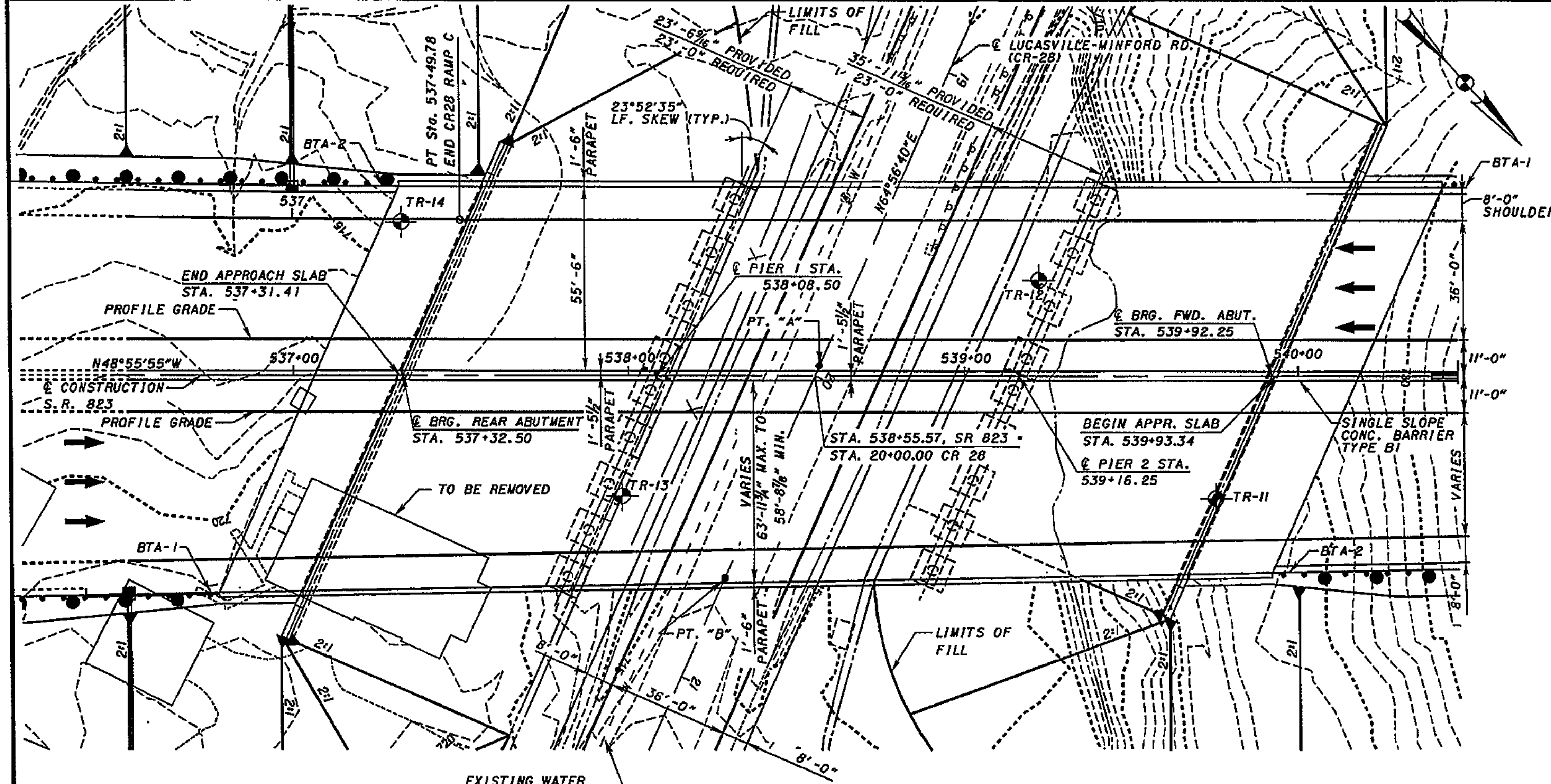


PLAN

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

TRAFFIC DATA
(SR 823)
CURRENT YEAR ADT (2010) = 19,800
CURRENT YEAR ADTT (2010) = 4752
DESIGN YEAR ADT (2030) = 26,000
DESIGN YEAR ADTT (2030) = 6240

TABLE OF VERTICAL CLEARANCES

LOCATION	"A"	"B"
PROPOSED	36.07'	34.03'
REQUIRED	15.00'	15.00'

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS

LOCATION	STATION	OFFSET
REAR ABUT.	536+73.04	66.00' RT.
REAR ABUT.	537+27.38	57.00' LT.
FWD. ABUT.	539+96.07	59.70' RT.
FWD. ABUT.	540+47.82	57.00' LT.

NOTES:

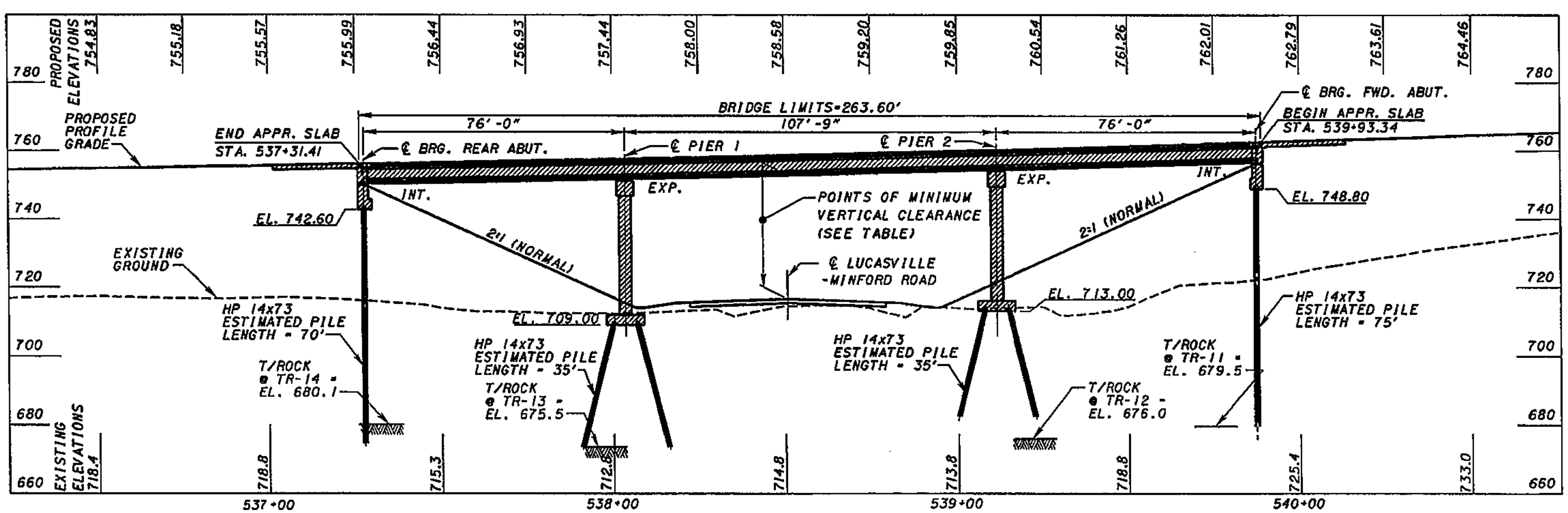
1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
3. THE PROPOSED PROFILE GRADE IS WITHIN BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

LEGEND

BTA-1 = BRIDGE TERMINAL ASSEMBLY TYPE 1
BTA-2 = BRIDGE TERMINAL ASSEMBLY TYPE 2
⊕ = BORING LOCATION

1300' VERT. CURVE DATA

P.V.I. STA. = 535+00.00
P.V.I. ELEV. 742.19
G_1 = -2.90%
G_2 = +4.00%



ELEVATION ALONG PROFILE GRADE LINE S.R. 823 RIGHT BRIDGE

PROPOSED STRUCTURE
TYPE: THREE SPAN, 60" MODIFIED AASTHO TYPE 4 PRESTRESSED CONCRETE I-BEAM WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY INTEGRAL ABUTMENTS AND CAP AND COLUMN PIERS
SPANS: 74'-9½", 105'-4", 74' 9½" (C/C BRG.)
ROADWAY: VARIES, 63'-11¾" TO 58'-8⅞" T/T OF BARRIER
LOADING: HS-25, ALTERNATE MILITARY LOADING AND FWS = 60psf
SKEW: 23°52'35" LF
CROWN: 0.016 FT/FT
ALIGNMENT: TANGENT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (25' LONG)
LATITUDE: 38°51'48" N
LONGITUDE: 82°53'45" W

DESIGN AGENCY: TranSystems

DESIGNED PJP | CHECKED MSL | DRAWN PJP | REVISED | REVIEWED JRC | DATE 11/30/06 | STRUCTURE FILE NUMBER 7306555

SCIOTO COUNTY
STA. 537+31.41
STA. 539+93.34

SITE PLAN
BRIDGE NO. SCI-823-1018 R
SR 823 OVER LUCASVILLE-MINFORD ROAD (CR-28)

SCI-823-10.31
PID 79977

2/6

APPENDIX II

General Information – Drilling Procedures and Logs of Borings

Legend – Boring Log Terminology

Boring Logs – Four (4) 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

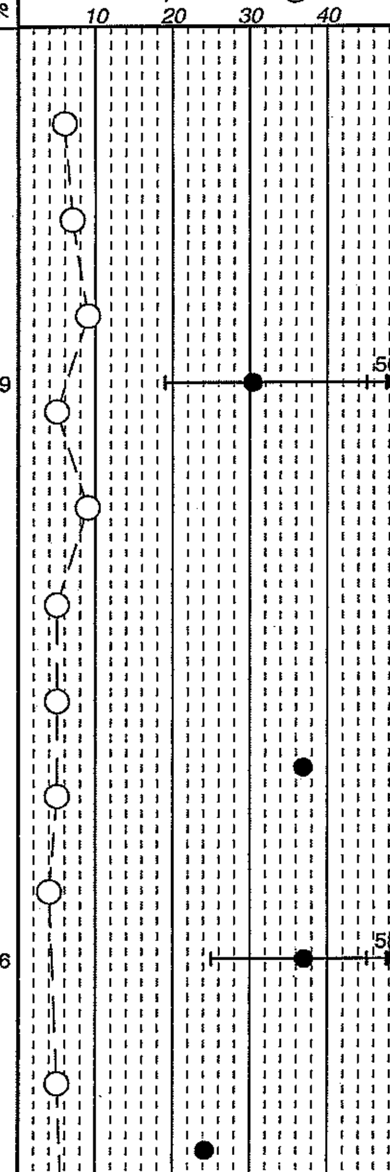
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-11 | Location: Sta. 539+75.3, 37.2 ft. RT of SR 823 CL | Date Drilled: 3/16/05 to 3/17/05

WATER OBSERVATIONS: Water seepage at: 23.5'
Water level at completion: 21.9' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	722.5												
0.3	722.2						Topsoil - 4"						
		1, 3, 3	18	1		2.5	Stiff to very stiff light brown CLAY (A-7-6), little to some silt, trace fine sand; damp.						
5		2, 3, 4	18	2		2.5							
		3, 4, 5	18	3		1.5							
10		2, 2, 3	18	4		2.0		0	0	--	1	20	79
		4, 4, 5	18	5		2.5	@ 11.0', gray.						
13.0	709.5												
15		2, 2, 3	18	6		0.75	Medium stiff gray CLAY (A-7-6); moist.						
		2, 2, 3	18	7		0.75							
20		WOH, 2, 3	18	8		0.5							
		1, 2, 2	18	9		0.5							
25		WOH, WOH, 3	18	10		0.5		0	0	--	0	14	86
		1, 2, 3	18	11		0.5							
30		WOH, WOH, 3	18	12		0.5	@ 28.5', contains sandstone fragments.						

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



FILE: 0121-3070-03 [6/18/2007 11:10 AM]

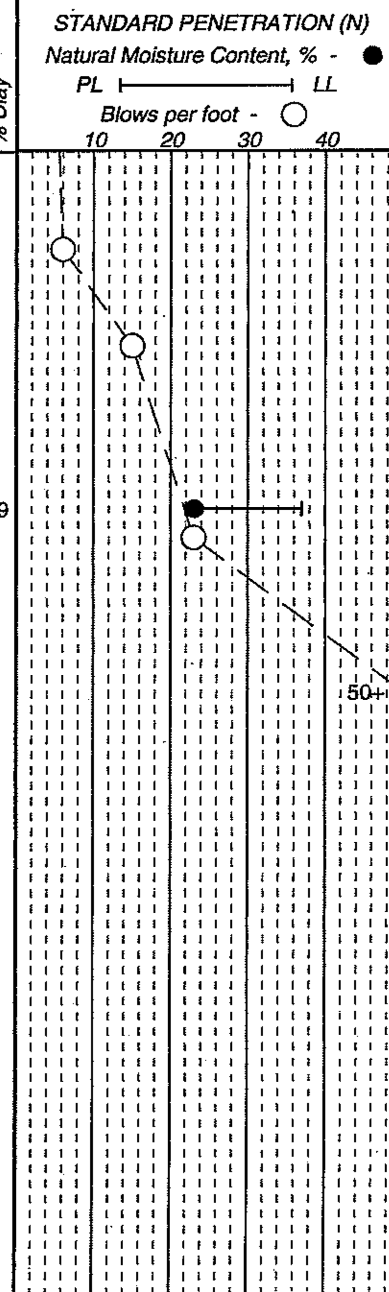
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-11 | Location: Sta. 539+75.3, 37.2 ft. RT of SR 823 CL | Date Drilled: 3/16/05 to 3/17/05

WATER OBSERVATIONS: Water seepage at: 23.5'
Water level at completion: 21.9' (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	692.5						Medium stiff light brown CLAY (A-7-6); moist.						
		1 2 4	18	13		0.5							
		8 6 9	18	14		0.75							
37.0	685.5						Very stiff gray SILT AND CLAY (A-6a), trace fine sand; damp to moist.						
		7 8 15	18	15		2.5		0	0	--	1	50	49
43.0	679.5	50/5	5	16			Severely weathered gray SHALE.						
47.0	675.5	Core 120"	Rec 120"	RQD 91%	R1		Medium hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly laminated to very thinly bedded, slightly fractured, contains abundant argillaceous laminations.						
57.0	665.5						Bottom of Boring - 57.0'						



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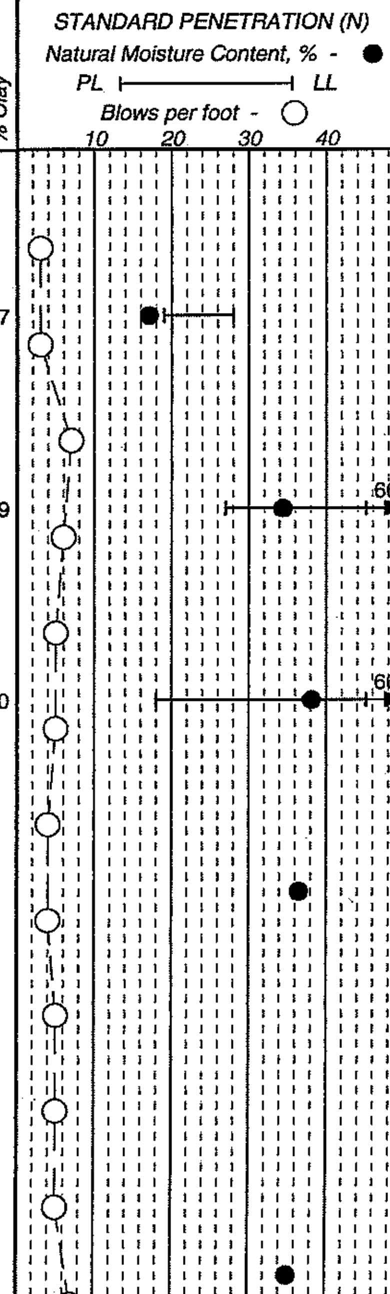
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-12 | *Location:* Sta. 539+22.5, 28.9 ft. LT of SR 823 CL | *Date Drilled:* 3/17/05

WATER OBSERVATIONS: Water seepage at: 10.5'-30.5'
Water level at completion: 10.1' (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	713.0												
0.4	712.6						Topsoil - 5"						
		1 1 2	18	1		0.75	POSSIBLE FILL: Medium stiff brown SANDY SILT (A-4a), some gravel, little clay; damp to moist.						
5		WOH 1 2	16	2		--		30	15	--	11	27	17
5.5	707.5						Very stiff brown and gray CLAY (A-7-6); varved; moist.						
		3 3 4	18	3		2.5							
10		1 2 4	18	4		2.25		0	0	--	0	11	89
		WOH 2 3	18	5		0.75	@ 11.0'-30.0', soft to medium stiff, brownish gray.						
15		WOH 2 3	18	6		0.75		0	0	--	0	10	90
		WOH 2 2	18	7		0.5							
20		WOH 2 2	18	8		0.5							
		WOH 2 3	18	9		0.5							
25		1 2 3	18	10		0.75							
		1 2 3	18	11		0.5							
30		3 3 4	18	12		0.5							



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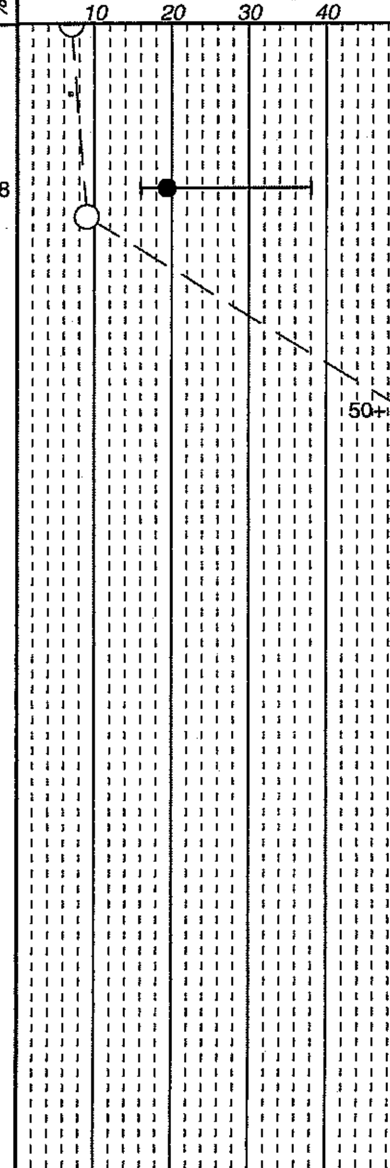
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-12 | *Location:* Sta. 539+22.5, 28.9 ft. LT of SR 823 CL | *Date Drilled:* 3/17/05

WATER OBSERVATIONS: Water seepage at: 10.5'-30.5'
Water level at completion: 10.1' (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
30.0	683.0						Stiff gray and brown SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; varved; damp to moist.						
35		2 3 6	18	13		1.5		1	5	--	8	59	28
37.0	676.0						Severely weathered gray SHALE.						
40.0	673.0	12 38 50/4	16	14									
							Medium hard gray SANDSTONE; very fine grained, highly weathered to decomposed, argillaceous, micaceous, slightly fractured, contains ferric bands and abundant argillaceous laminations, fissile after desiccation.						
45		Core 120"	Rec 120"	RQD 92%	R1		@ 45.9'-48.2', light brown siltstone layer.						
50.0	663.0						Bottom of Boring - 50.0'						
55													
60													

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



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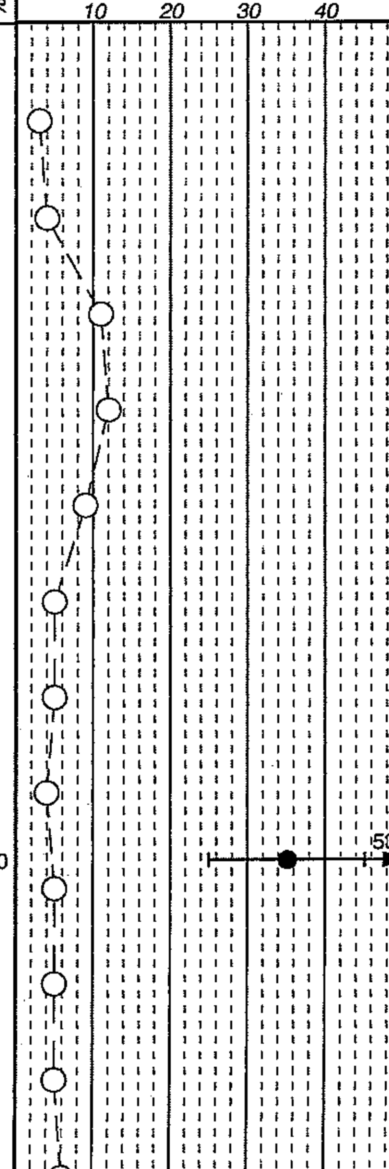
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-13 | Location: Sta. 537+97.9, 36.5 ft. RT of SR 823 CL | Date Drilled: 6/8/04

WATER OBSERVATIONS: Water seepage at: 33.5'-35.0'
Water level at completion: 28.6' (includes drilling water)

Depth (ft)	Elev. (ft) 713.5	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
0.5	713.0						Topsoil - 6"						
		1 1 2	10	1		1.75	Stiff gray SANDY SILT (A-4a), some clay, trace gravel; moist.						
3.0	710.5	1 2 2	16	2		1.25	Stiff to very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, trace to little gravel; moist.						
		2 4 7	18	3		3.5	@ 6.0'-7.5', mottled brown and gray.						
8.5	705.0	3 5 7	18	4		3.25	Stiff to very stiff mottled brown and gray CLAY (A-7-6), trace fine to coarse sand; moist.						
		2 3 6	18	5		2.25							
		1 2 3	18	6		1.25							
		1 2 3	18	7		2.0	@ 16.0'-27.5', gray.						
		1 2 2	18	8		1.5							
		1 2 3	18	9		0.75	@ 21.0'-22.5', medium stiff.	0	0	--	1	19	80
		1 2 3	18	10		1.0							
		WOH 2 3	18	11		1.5	@ 26.0', contains sand seams.						
		2 2 4	18	12		1.5							

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



Depth scale: 0, 5, 10, 15, 20, 25, 30

FILE: 0121-3070-03 [6/18/2007 11:10 AM]

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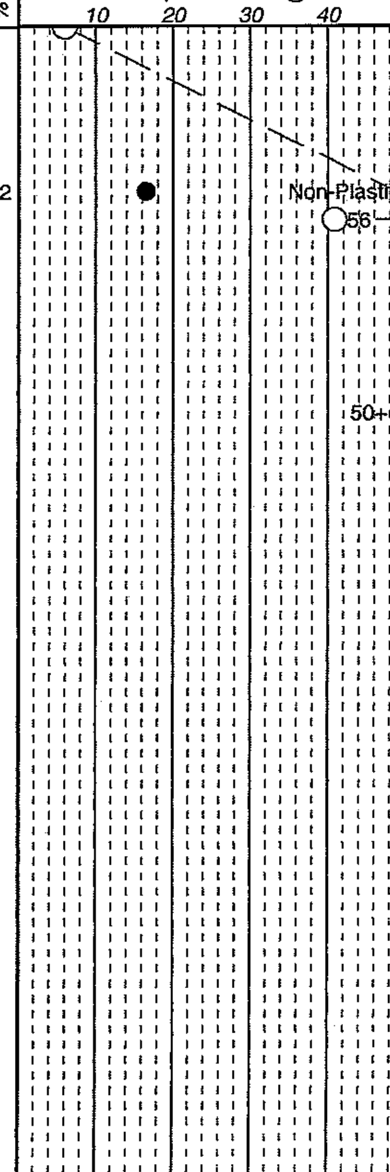
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-13 | Location: Sta. 537+97.9, 36.5 ft. RT of SR 823 CL | Date Drilled: 6/8/04

WATER OBSERVATIONS: Water seepage at: 33.5'-35.0'
Water level at completion: 28.6' (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
30.0	683.5						Very dense gray and brown SANDY SILT (A-4a), little clay, trace gravel; contains sandstone fragments; damp to moist.						
35		8 24 32	18	13				5	10	--	25	48	12
		27 49 50/2	16	14									
40.0	673.5						Medium hard gray SILTSTONE; fissile.						
							@40.0' - 44.9', core loss.						
45		Core 120"	Rec 61"	RQD 25%	R-1								
							@46.6' - 46.8', clay seam.						
							@47.8' - 50.0', broken to highly fractured with occasional clay seams.						
50.0	663.5						Bottom of Boring - 50.0'						
55													
60													

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



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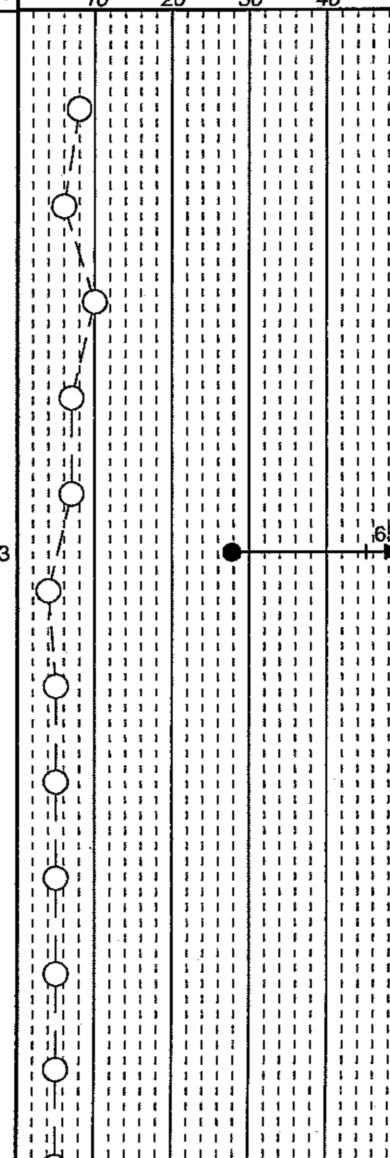
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-14 | Location: Sta. 537+32.3, 46.2 ft. LT of SR 823 CL | Date Drilled: 6/4/04 to 6/7/04

WATER OBSERVATIONS: Water seepage at: 33.5', 38.5'
Water level at completion: 24.8' (Prior to coring)
8.9' (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	713.6												
0.3	713.3						Topsoil - 4"						
		2 3 5	18	1		2.25	Very stiff brown and gray SILTY CLAY (A-6b), little fine to coarse sand; moist.						
5		2 2 4	18	2		3.75							
		2 4 6	18	3		2.75	@ 6.0'-7.5', gray.						
8.5	705.1	2 3 4	18	4		2.75	Stiff to very stiff brown CLAY (A-7-6), trace to little fine to coarse sand; varved; damp to moist.						
10		2 3 4	18	5		2.25							
		WOH 2 2	18	6	ST-1	1.0	@13.5', gray; qu=3820 psf.	0	0	--	0	7	93
15		WOH 2 3	18	7		1.5							
		1 2 3	18	8		1.25							
20		1 2 3	18	9		1.25							
		1 2 3	18	10		1.5	@ 23.5', gray and brown.						
25		2 2 3	18	11		1.25							
30		2 2 3	18	12		2.0	@ 28.5', contains sand seams.						

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



FILE: 0121-3070-03 [6/18/2007 11:10 AM]

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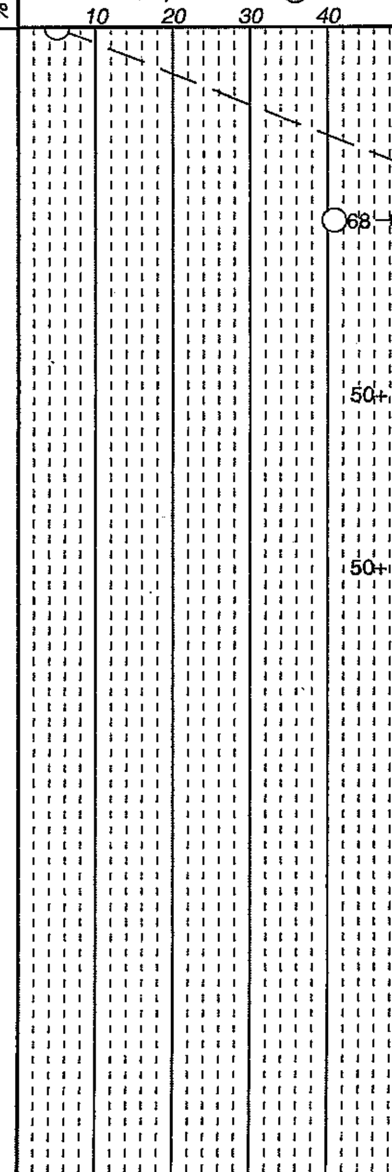
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-14 | Location: Sta. 537+32.3, 46.2 ft. LT of SR 823 CL | Date Drilled: 6/4/04 to 6/7/04

WATER OBSERVATIONS: Water seepage at: 33.5', 38.5'
Water level at completion: 24.8' (Prior to coring)
8.9' (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	683.6						Stiff to very stiff gray and brown CLAY (A-7-6), trace to little fine to coarse sand; varved; contains sand seams; damp to moist.						
33.5	680.1	14 28 40	18	13			Severely weathered gray SILTSTONE.						
		40 50/4	12	14									
44.0	669.6	50/4	4	15			Medium hard gray SILTSTONE; fissile. @ 45.7', 46.4', 49.3', 50.7', 53.0', clay seams. @ 46.1'-46.7', 49.0'-49.3', broken to highly fractured.						
		Core 60"	Rec 54"	RQD 72%	R-1								
		Core 60"	Rec 56"	RQD 77%	R-2		@ 53.5'-53.7', vertical fracture.						
54.0	659.6						Bottom of Boring - 54.0'						

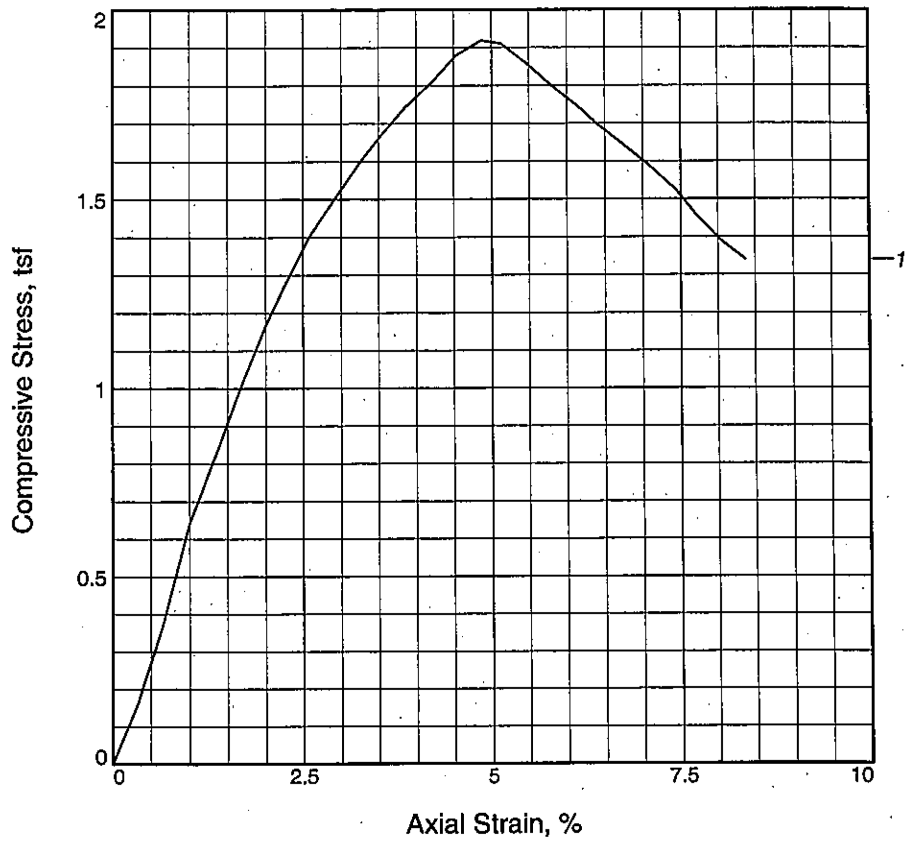
STANDARD PENETRATION (N)
Natural Moisture Content, % - ●
PL ⊢—⊣ LL
Blows per foot - ○



FILE: 0121-3070-03 [6/18/2007 11:10 AM]

APPENDIX III
Laboratory Test Results

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.911			
Undrained shear strength, tsf	0.955			
Failure strain,	5.1			
Strain rate, in./min.	0.06			
Water content, %	38.5			
Wet density, pcf	116.9			
Dry density, pcf	84.4			
Saturation, %	100.0			
Void ratio	1.0865			
Specimen diameter, in.	2.85			
Specimen height, in.	6.26			
Height/diameter ratio	2.20			

Description: Fat clay

LL = 65	**PL** = 28	**PI** = 37	**Assumed GS=** 2.82	**Type:** 2.8" press tube

Project No.: 0121-3070.03

Date: 7/26/04

Remarks:

Figure ________

Client: TranSystems, Inc.

Project: SCI-823-0.00

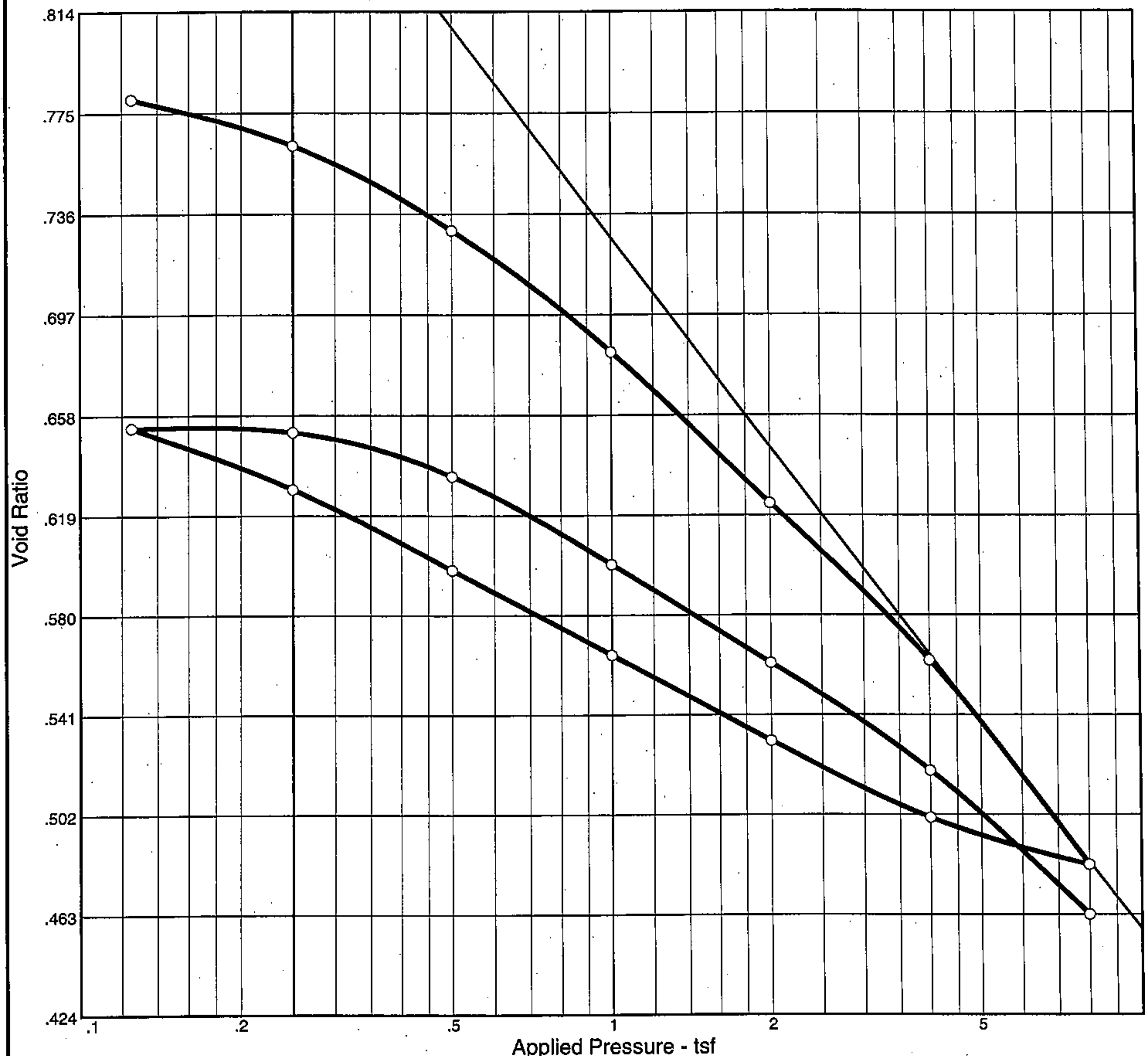
Source of Sample: TR-14 **Depth:** 13.0

Sample Number: ST-1

DLZ

Tested By: Gary Bowen

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
100.0 %	28.7 %	95.9	57	31	2.75	CH	A-7-6(36	0.790

MATERIAL DESCRIPTION

Fat clay

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

Project: SCI-823-0.00

Source: B-1223 **Sample No.:** P1 **Elev./Depth:** 8.0

DLZ

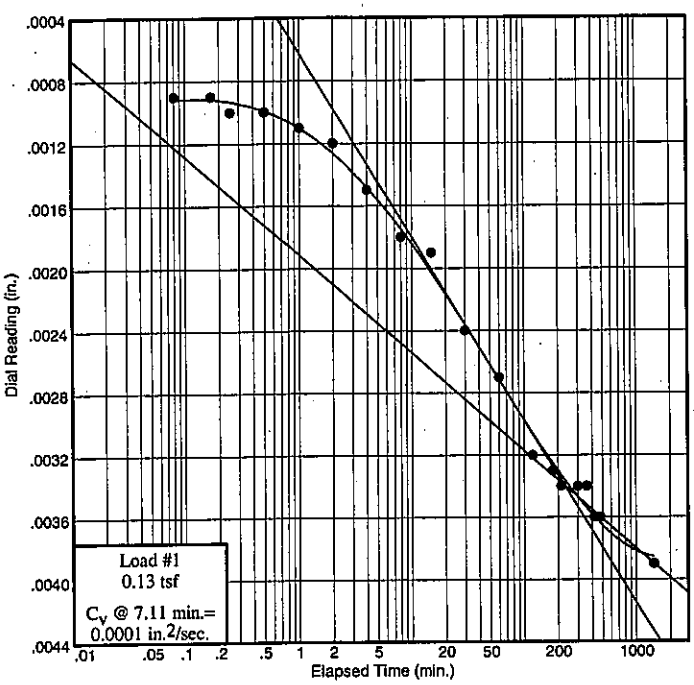
Remarks:

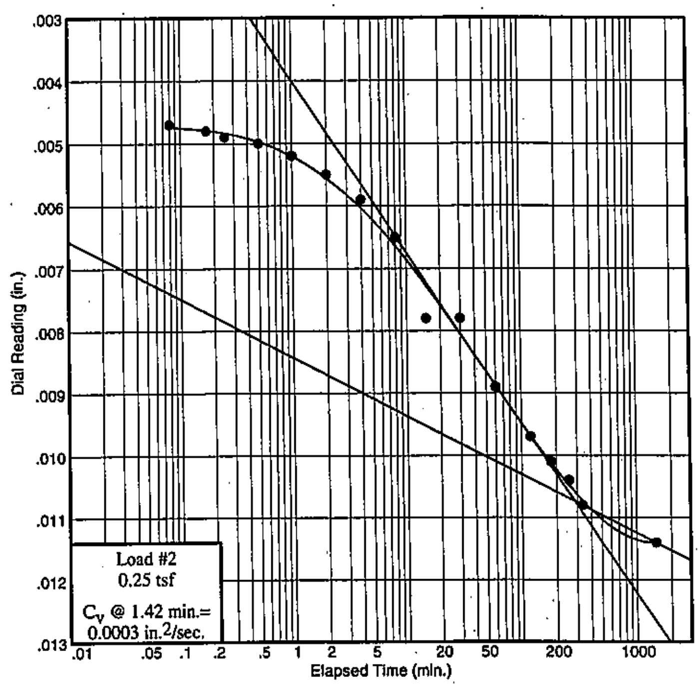
Figure

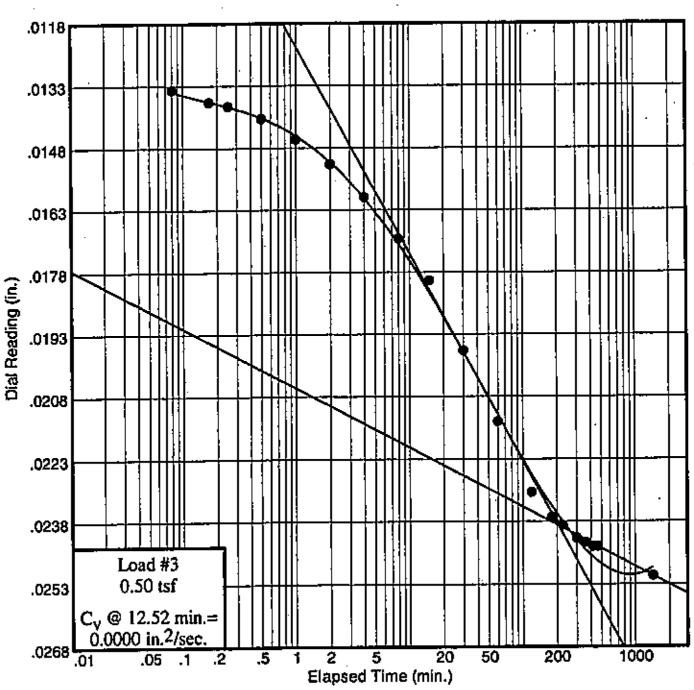
Dial Reading vs. Time

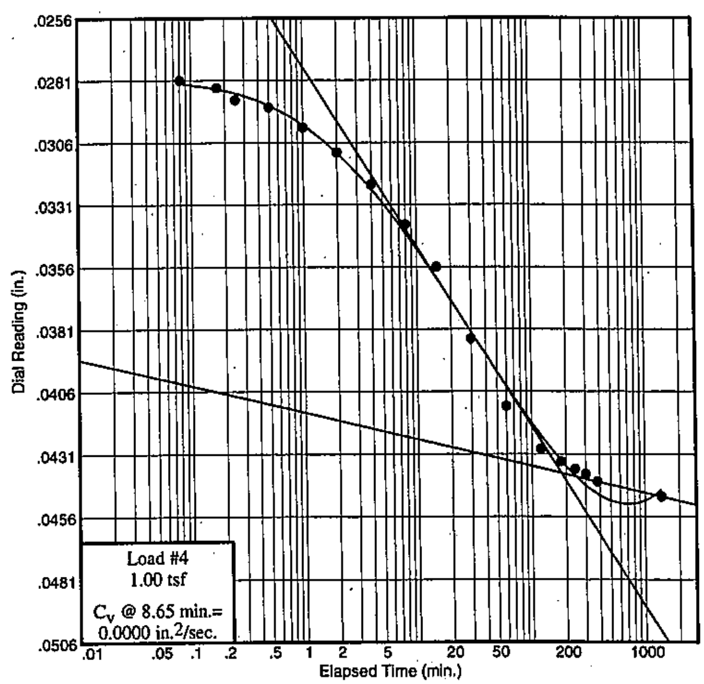
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1223 Sample No.: P1 Elev./Depth: 8.0









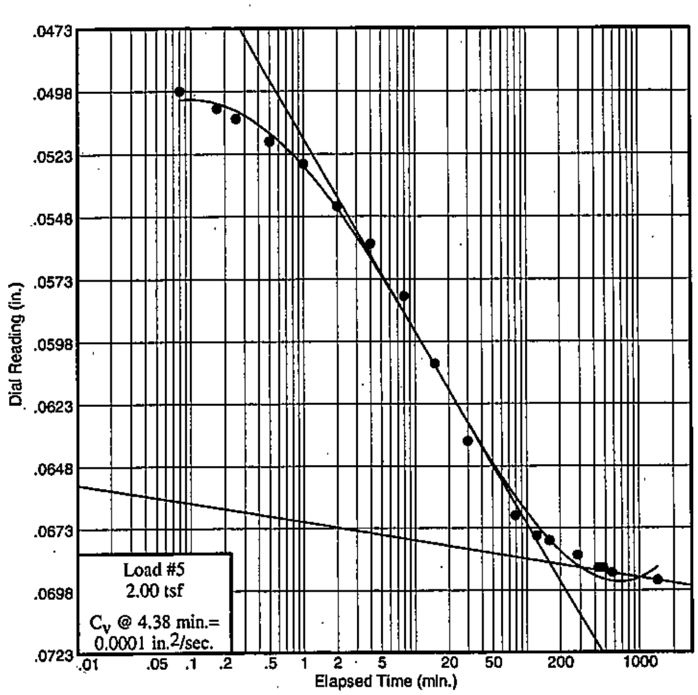
DLZ

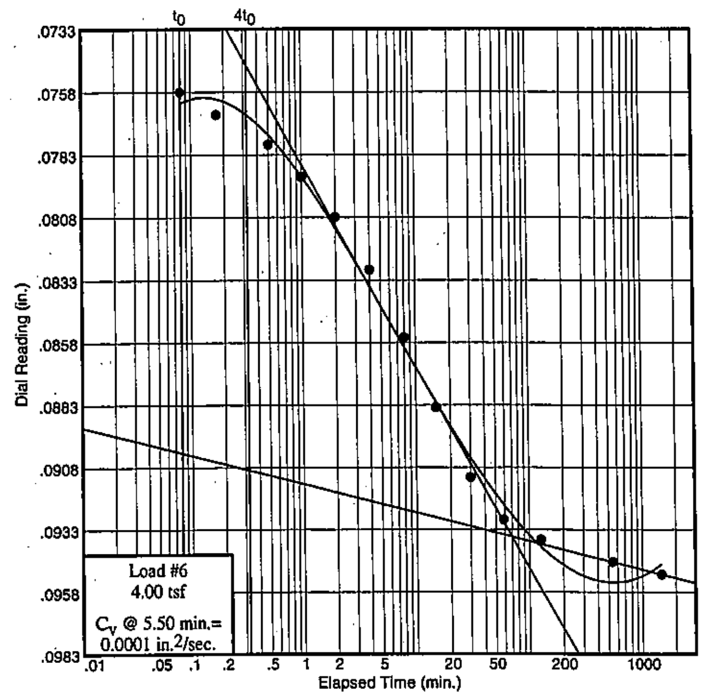
Figure

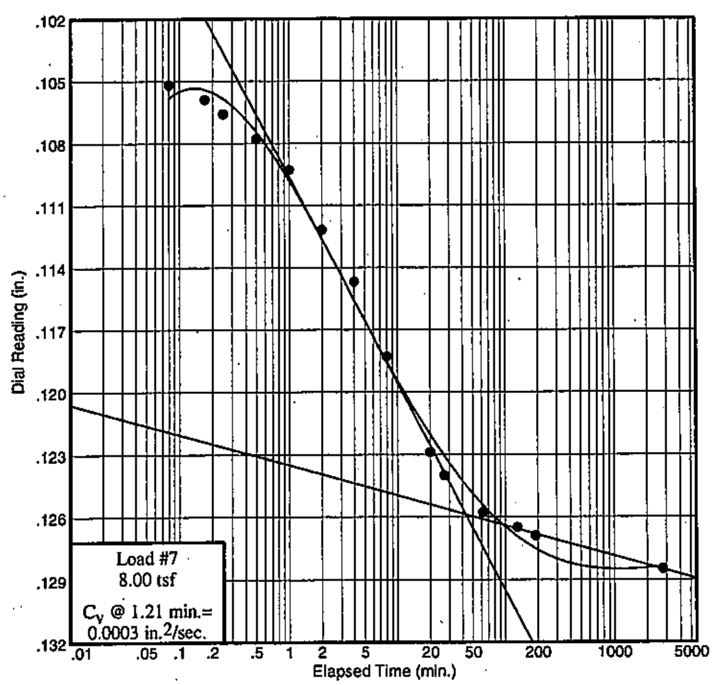
Dial Reading vs. Time

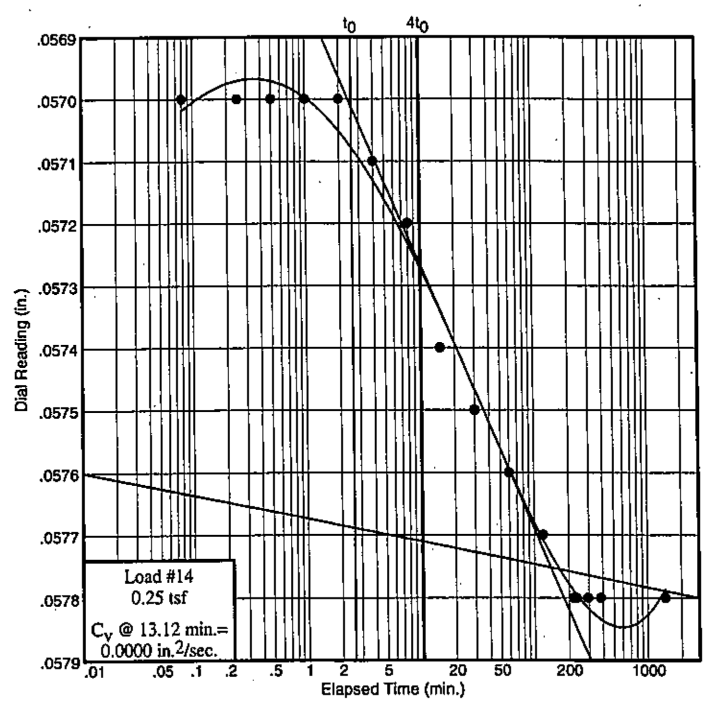
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1223 | Sample No.: P1 | Elev./Depth: 8.0









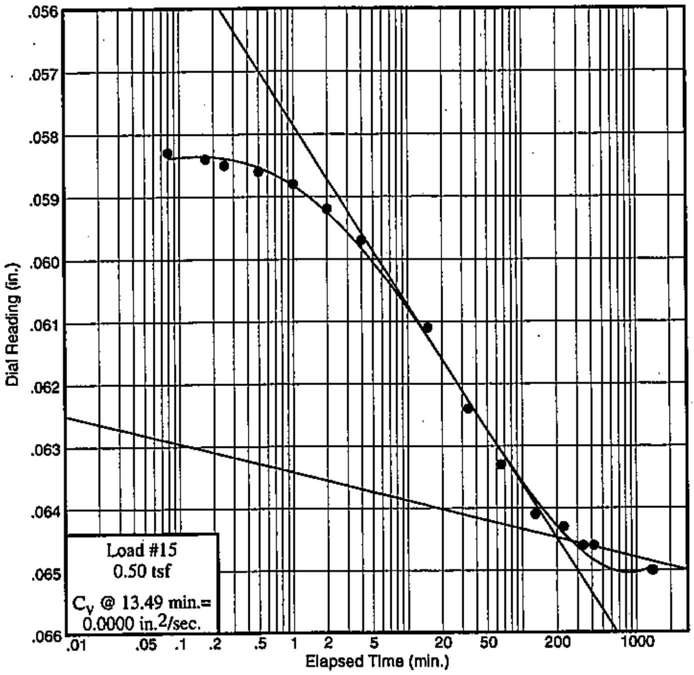
DLZ

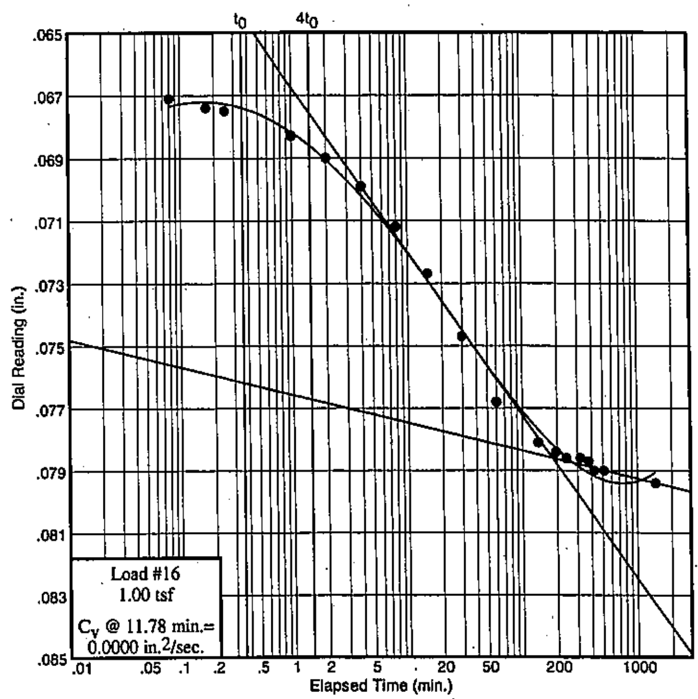
Figure

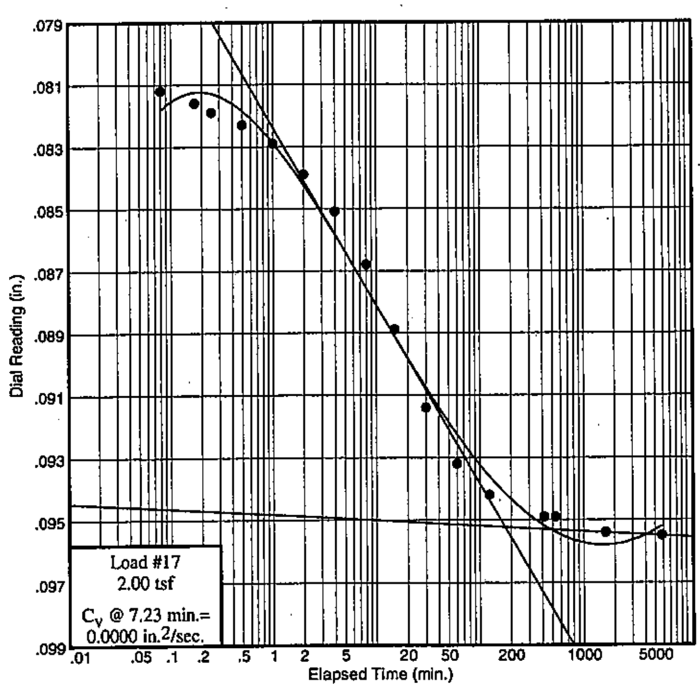
Dial Reading vs. Time

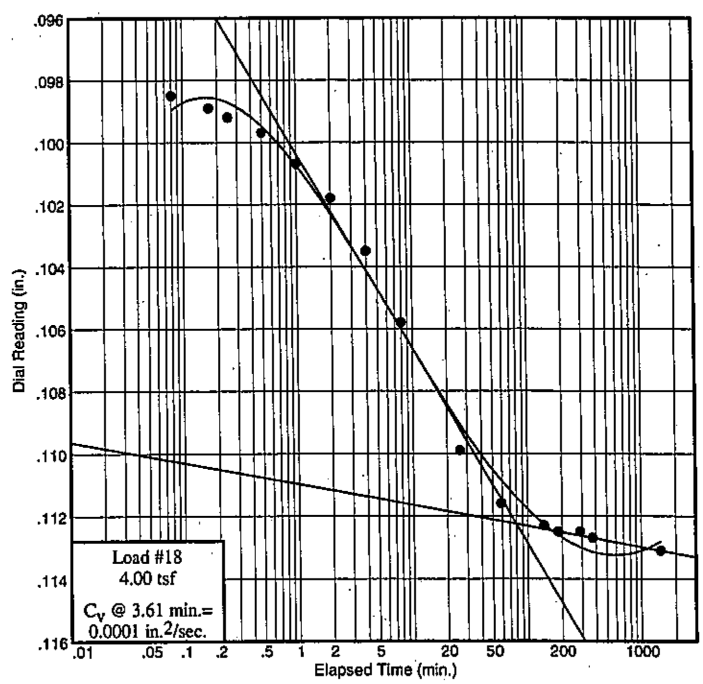
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1223 | Sample No.: P1 | Elev./Depth: 8.0









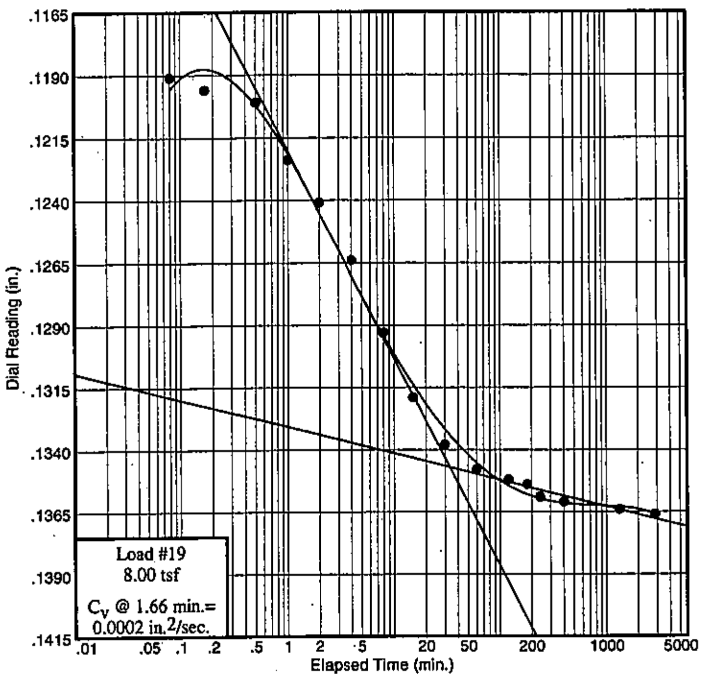
DLZ

Figure

Dial Reading vs. Time

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

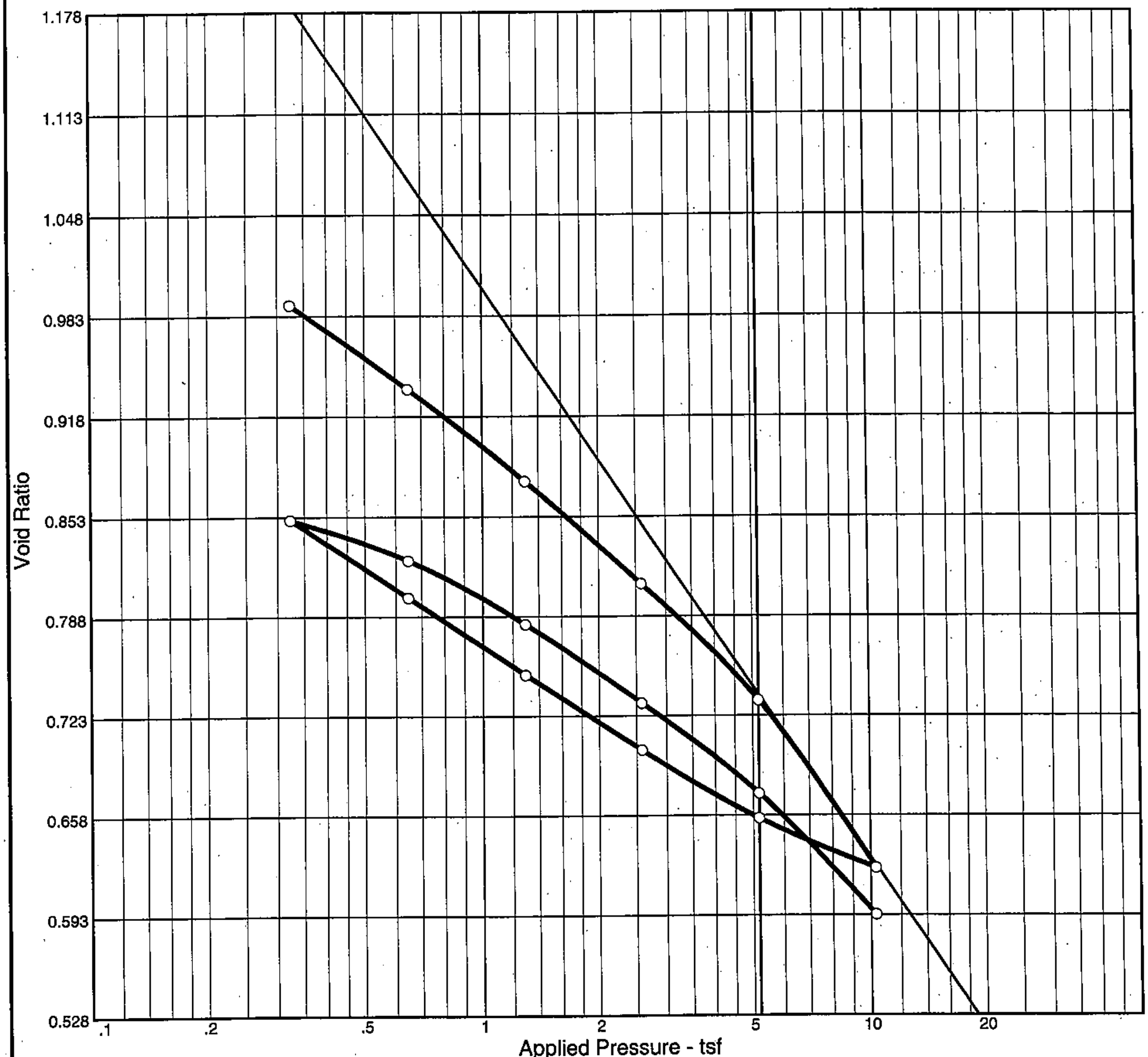
Source: B-1223 Sample No.: P1 Elev./Depth: 8.0



DLZ

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
100.3 %	40.4 %	82.0	67	41	2.79	CH	A-7-6(48	1.124

MATERIAL DESCRIPTION

Fat clay

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

Project: SCI-823-0.00

Source: B-1223 **Sample No.:** P2 **Elev./Depth:** 18.0

DLZ

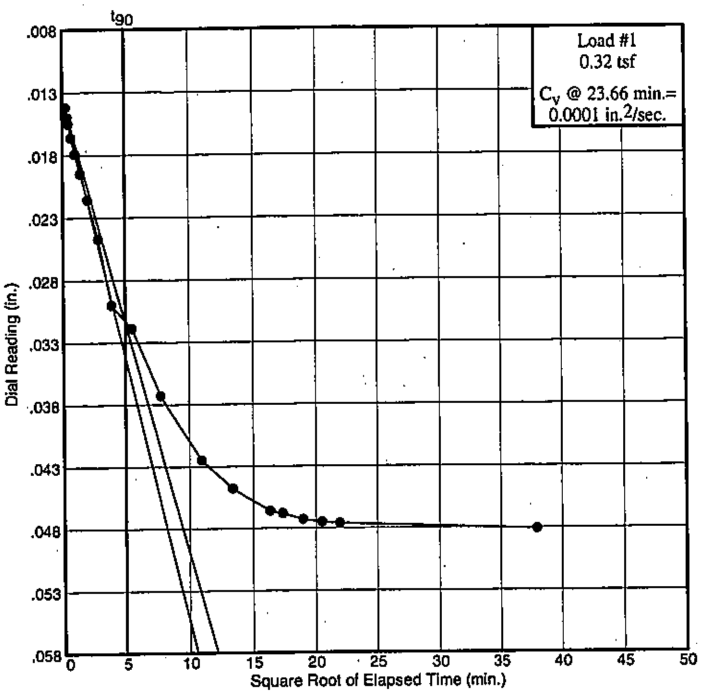
Remarks:

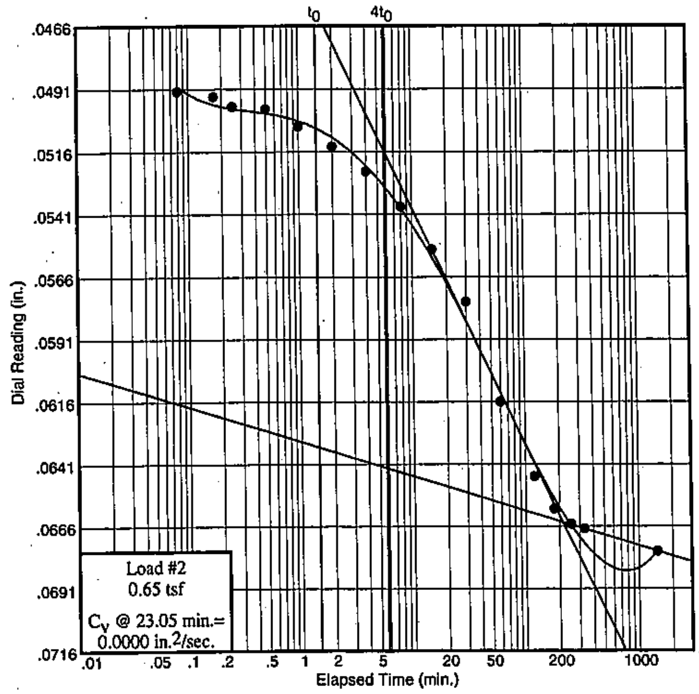
Figure

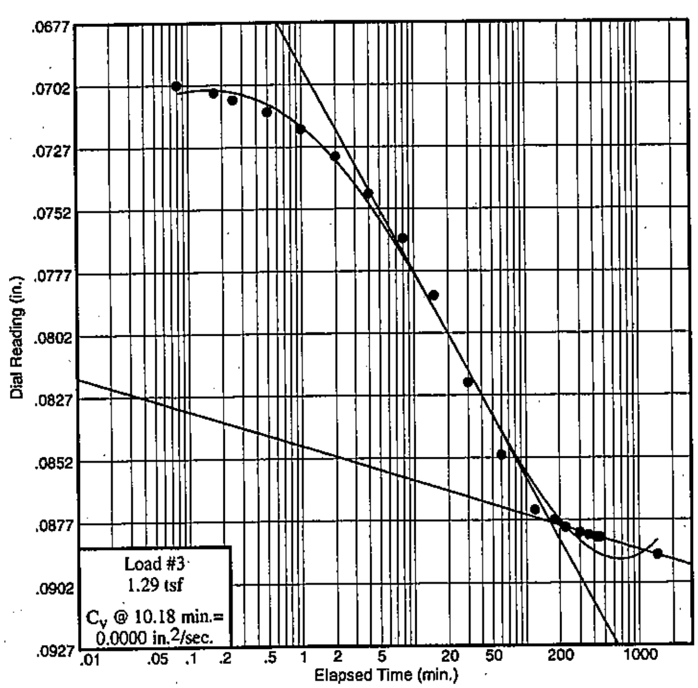
Dial Reading vs. Time

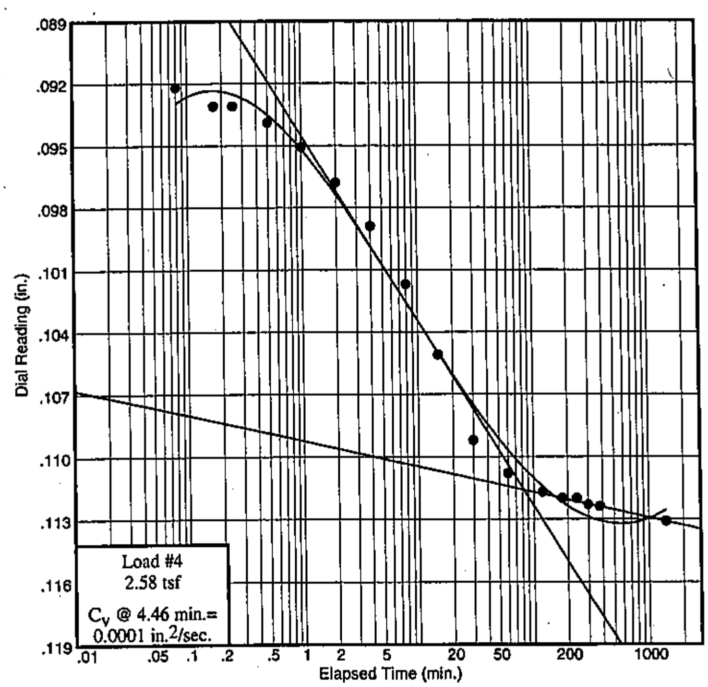
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1223 Sample No.: P2 Elev./Depth: 18.0









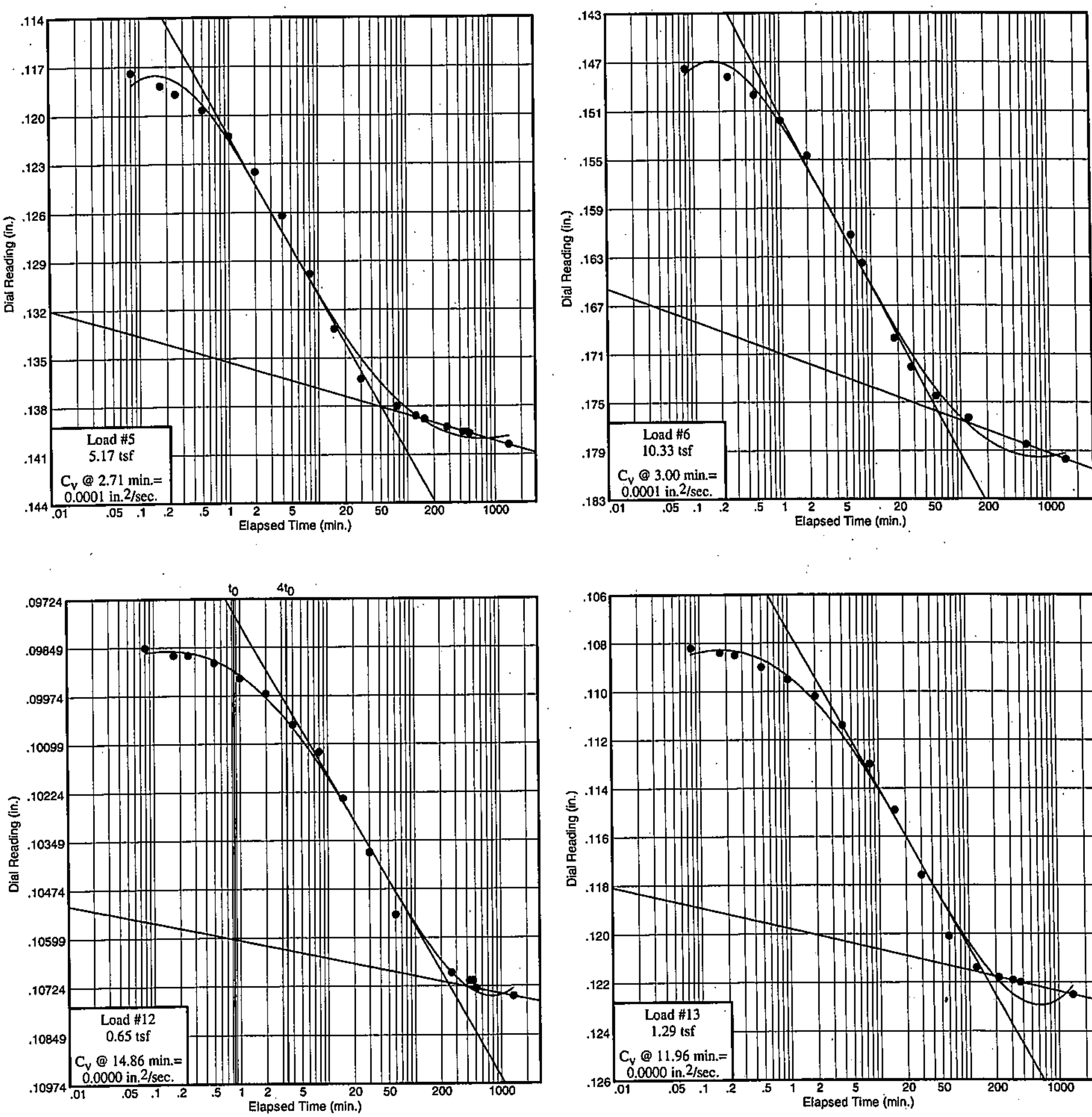
DLZ

Figure

Dial Reading vs. Time

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

Source: B-1223 | Sample No.: P2 | Elev./Depth: 18.0



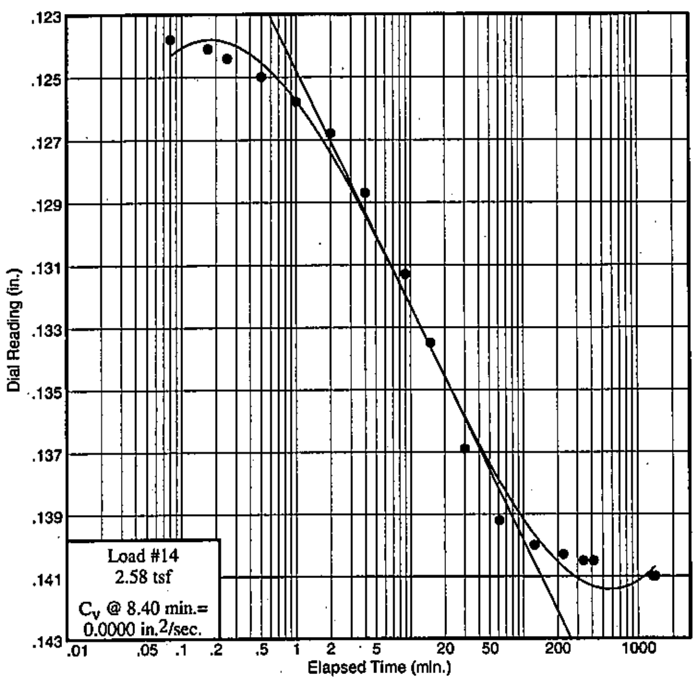
DLZ

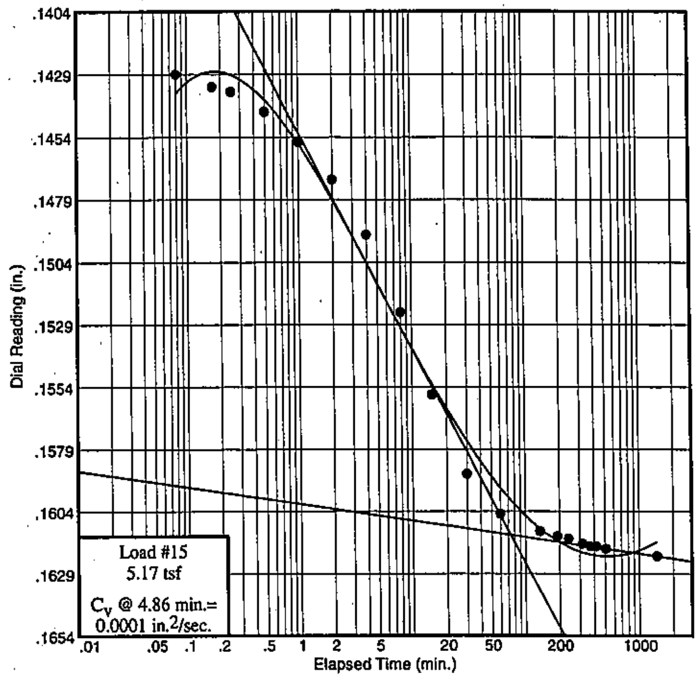
Figure

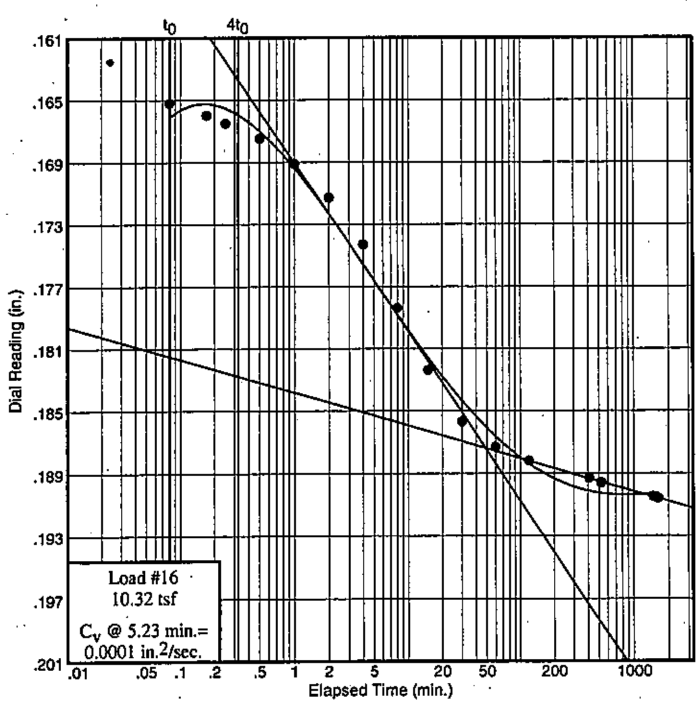
Dial Reading vs. Time

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

Source: B-1223 Sample No.: P2 Elev./Depth: 18.0







DLZ

Figure

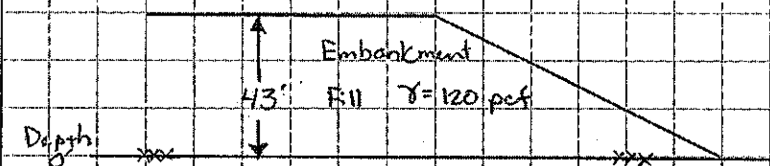
APPENDIX IV
Slope Stability Analysis
Settlement Analysis
Downdrag Forces
Drilled Shaft – End Bearing and Side Resistance Calculations

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT	TranSystems Corp. / ODOT D-9	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	1 OF 33
SUBJECT	Consolidation Parameters	COMP. BY	SMK DATE 6-13-07
	SR 823 over Lucasville-Minford Rd	CHECKED BY	DAA DATE 6-25-07

* Maximum embankment height = 43.0'
→ At station 537+60, 60' Lt.

Soil Profile based upon boring TR-11 (most critical at this site)
Consolidation Parameters taken from boring B-1223 (sta. 535+28, 71' Lt)



Depth				
0–13'	① Clay (A-7-6)	LL = 56, PL = 19, ω = 30%		From B-1223, P-1; γ = 122 pcf, P_c = 0.89 tsf, C_c = 0.270, C_r = 0.10, e_o = 0.790
13'–37'	② Clay (A-7-6)	LL = 58, PL = 25, ω = 37%		From B-1223, P-2; γ = 116 pcf, P_c = 5.4 tsf, C_c = 0.370, C_r = 0.160, e_o = 1.124
37'–43'	③ Silt and Clay (A-6a)	LL = 37, PL = 22, ω = 23%		*See Sample Calculation Below. Estimated from FHWA NHI-00-045
43'	Bed Rock			

***Sample Calculation**

1) Check Preconsolidation Pressure:

$$\frac{\omega - PL}{LL - PL} = \frac{23-22}{37-22} = \frac{1}{15} = 0.067 < 0.7$$

∴ We may assume soil is preconsolidated beyond σ_f.

2) Initial Void Ratio: (Assumes fully saturated)

$$e_o = \frac{(2.75)(23)}{100} = 0.63$$

3) C_c and C_r

$$C_c = \frac{\omega}{100} = \frac{23}{100} = 0.23 \qquad C_r = \frac{\omega}{1000} = 0.02$$

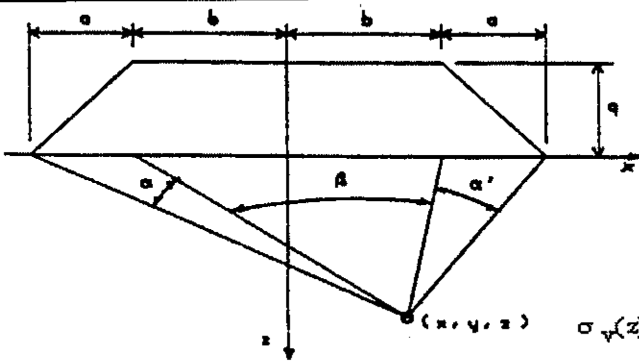
SMK

DLZ	SUBJECT	Client TranSystems Inc.	JOB NUMBER	0121-3070-03
		Project SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 33
		Item Settlement Analysis	COMP. BY	SAK DATE 6-13-07
		Lucasville-Minford Road	CHECKED BY	DAA DATE 6-25-07

Profile based upon TR-11, Settlement parameters taken from similar soil layers encountered in B-1223

SETTLEMENT ANALYSIS - EMBANKMENT

Embankment Informaiton:



Groundwater Table:	D=	10.0	ft			
Embankment Height:	H =	43	ft			
Fill Unit Weight:	γ_{emb} =	120	pcf	q =	5,160	psf
Width of Slope:	a =	86				
Top half-width of Emb:	b =	60				
Distance from CL:	x =	0				
Output Range:	z =	0	to	43	ft	

**See Data output Attached*

$$\sigma_{v}(z) := \left(\frac{q}{\pi\, a}\right)\left(a\cdot(\alpha(z)+\beta(z)+\alpha'(z)) + b\cdot(\alpha(z)+\alpha'(z)) + x\cdot(\alpha(z)-\alpha'(z))\right)$$

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right] + \operatorname{atan}\left[\frac{(b+x)}{z}\right] \qquad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right] - \operatorname{atan}\left[\frac{(b-x)}{z}\right] \qquad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right] - \operatorname{atan}\left[\frac{(b+x)}{z}\right]$$

Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

Soil Properites: *Settlement is calculated at mid-point of layer*

No.	Bot. of Laye		Soil Type	γ_{soil} (pcf)	σ'_c (psf)	σ'_o (psf)	$\Delta\sigma z$ (psf)	σ'_f (psf)	Cohesionless Soils C'	Cohesive Soils C_r	C_c	e_o
1	13.0	ft	Clay	122	1,780	793	5,159	5,952	0.0	0.10	0.27	0.790
2	23.0	ft	Clay	116	10,800	1,667	5,145	6,812	0.0	0.16	0.37	1.124
3	33.0	ft	Clay	116	10,800	2,203	5,104	7,307	0.0	0.16	0.37	1.124
4	37.0	ft	Clay	116	10,800	2,578	5,059	7,637	0.0	0.16	0.37	1.124
5	43.0	ft	Clay	120	10,800	2,858	5,016	7,874	0.0	0.02	0.23	0.630
6	0.0			0	0							
7	0.0			0	0							
8	0.0			0	0							
9	0.0			0	0							
10	0.0			0	0							

No.	Settlement:		Total Settlement
1	1.283	ft	
2	0.461	ft	**2.315 ft**
3	0.392	ft	
4	0.142	ft	
5	0.037	ft	**27.8 in**
6			
7			
8			
9			
10			

Reference: Geotechnical Engineering Principles and Practices; Coduto, 1999

Overconsolidated Soils - Case I ($\sigma'_0<\sigma'_c$) Eqn:11.24

$$(\delta_c)_{ult} = \Sigma \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Overconsolidated Soils - Case II ($\sigma'_0<\sigma'_c<\sigma_f$) Eqn:11.25

$$(\delta_c)_{ult} = \Sigma\left[\frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_c}{\sigma'_0}\right) + \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_c}\right)\right]$$

Normally Consolidated Soils ($\sigma'_0=\sigma'_c$) Eqn: 11.23

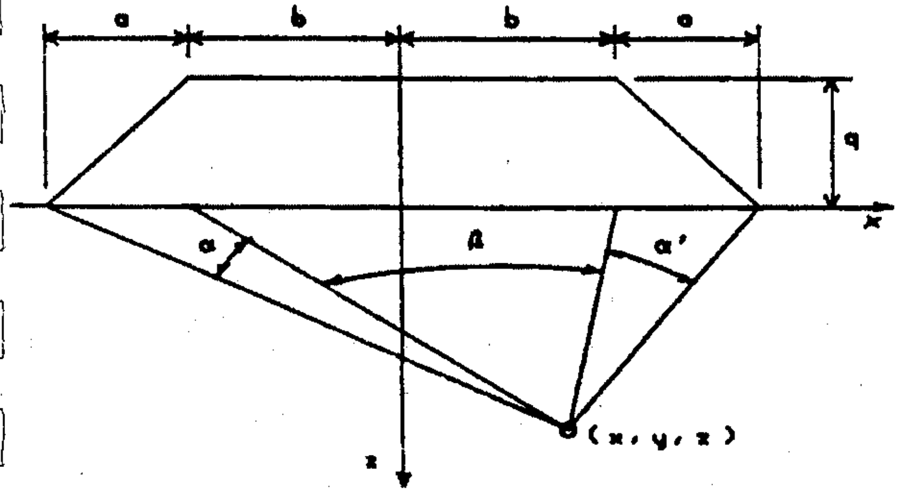
$$(\delta_c)_{ult} = \Sigma \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Cohesionless Soils ($\sigma'_0=\sigma'_c$)

$$(\delta_c)_{ult} = \Sigma \frac{1}{C'} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

SUBJECT		JOB NUMBER	0121-3070.03
Client	TranSystems Inc.	SHEET NO.	3 OF 33
Project	SCI-823 Portsmouth Bypass	COMP. BY	SJK DATE 6-13-07
Item	Settlement Analysis	CHECKED BY	DAA DATE 6-25-07
Lucasville-Minford Road			

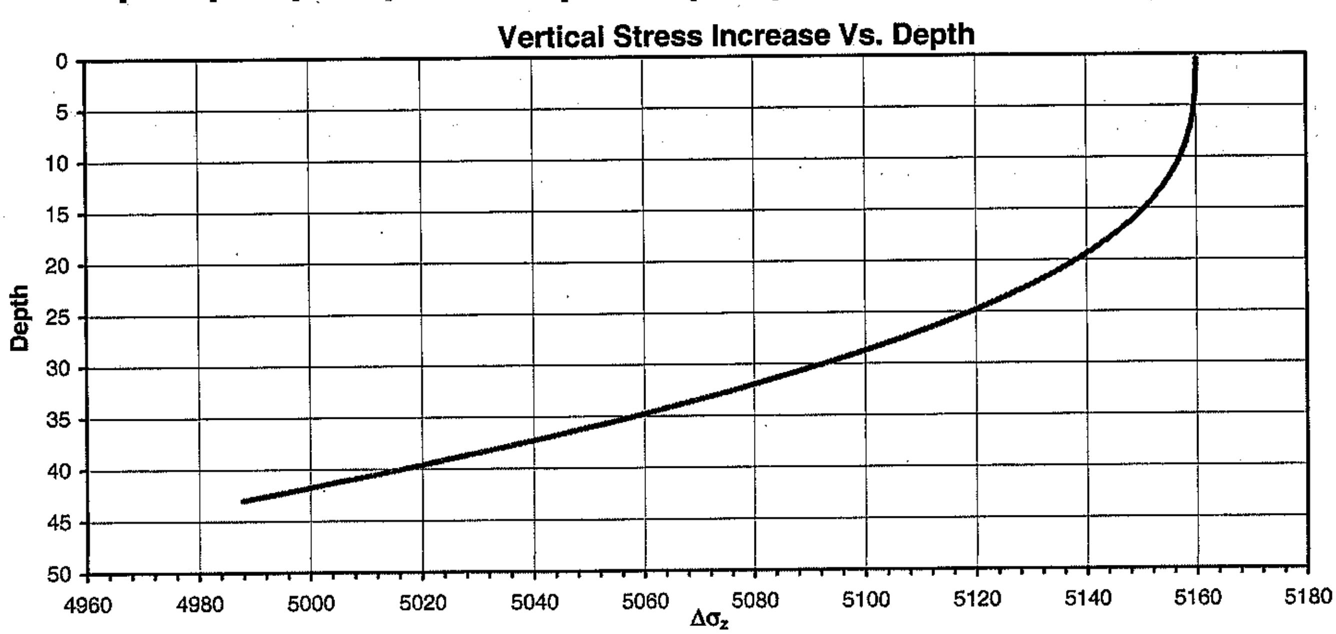
INCREASE IN VERTICAL STRESS DUE TO EMBANKMENT LOADING



q =	5160	load
a =	86	width of slope
b =	60	top half-width of embankment
x =	0	distance from CL
z =	0 to 43	depth range

$$\sigma_V(z) := \left(\frac{q}{\pi\, a}\right)\left(a\cdot(\alpha(z)+\beta(z)+\alpha'(z)) + b\cdot(\alpha(z)+\alpha'(z)) + x\cdot(\alpha(z)-\alpha'(z))\right)$$

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right] + \operatorname{atan}\left[\frac{(b+x)}{z}\right] ; \quad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right] - \operatorname{atan}\left[\frac{(b-x)}{z}\right] \quad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right] - \operatorname{atan}\left[\frac{(b+x)}{z}\right]$$



Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

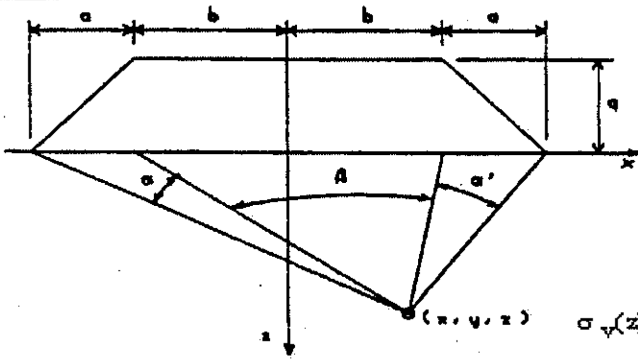
DLZ SUBJECT

Client TranSystems Inc.	JOB NUMBER	0121-3070-03	
Project SCI-823 Portsmouth Bypass	SHEET NO.	4	OF 33
Item Settlement Analysis	COMP. BY	SJK	DATE 6-13-07
Lucasville-Minford Road **Embankment TOE**	CHECKED BY	DAA	DATE 6-25-07

Profile based upon TR-11, Settlement parameters taken from similar soil layers encountered in B-1223

SETTLEMENT ANALYSIS - EMBANKMENT

Embankment Informaiton:



Groundwater Table:	D=	10.0	ft		
Embankment Height:	H =	43	ft		
Fill Unit Weight:	γ_{emb} =	120	pcf	q =	5,160 psf
Width of Slope:	a =	86			
Top half-width of Emb:	b =	100			
Distance from CL:	x =	186			
Output Range:	z =	0	to	43	ft

**See Data output Attached*

$$\sigma_v(z) := \left(\frac{q}{\pi\, a}\right)\left(a\cdot(\alpha(z)+\beta(z)+\alpha'(z))+b\cdot(\alpha(z)+\alpha'(z))+x\cdot(\alpha(z)-\alpha'(z))\right)$$

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right]+\operatorname{atan}\left[\frac{(b+x)}{z}\right] \qquad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right]-\operatorname{atan}\left[\frac{(b-x)}{z}\right] \qquad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right]-\operatorname{atan}\left[\frac{(b+x)}{z}\right]$$

Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

Soil Properites: *Settlement is calculated at mid-point of layer*

								Cohesionless Soils	Cohesive Soils		
No.	Bot. of Laye	Soil Type	γ_{soil} (pcf)	σ'_c (psf)	σ'_0 (psf)	$\Delta\sigma z$ (psf)	σ'_f (psf)	C'	C_r	C_c	e_0
1	13.0 ft	Clay	122	1,780	793	123	916	0.0	0.10	0.27	0.790
2	23.0 ft	Clay	116	10,800	1,667	332	1,999	0.0	0.16	0.37	1.124
3	33.0 ft	Clay	116	10,800	2,203	515	2,718	0.0	0.16	0.37	1.124
4	37.0 ft	Clay	116	10,800	2,578	631	3,209	0.0	0.16	0.37	1.124
5	43.0 ft	Clay	120	10,800	2,858	713	3,571	0.0	0.02	0.23	0.630
6	0.0		0	0							
7	0.0		0	0							
8	0.0		0	0							
9	0.0		0	0							
10	0.0		0	0							

No.	Settlement:	
1	0.045	ft
2	0.059	ft
3	0.069	ft
4	0.029	ft
5	0.008	ft
6		
7		
8		
9		
10		

Total Settlement

0.210 ft

2.5 in

Reference: Geotechnical Engineering Principles and Practices; Coduto, 1999

Overconsolidated Soils - Case I ($\sigma'_0<\sigma'_c$) Eqn:11.24

$$(\delta_c)_{ult} = \sum \frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

Overconsolidated Soils - Case II ($\sigma'_0<\sigma'_c<\sigma_f$) Eqn:11.25

$$(\delta_c)_{ult} = \sum\left[\frac{C_r}{1+e_0} H \log\left(\frac{\sigma'_c}{\sigma'_0}\right)+\frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_c}\right)\right]$$

Normally Consolidated Soils ($\sigma'_0=\sigma'_c$) Eqn: 11.23

$$(\delta_c)_{ult} = \sum \frac{C_c}{1+e_0} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

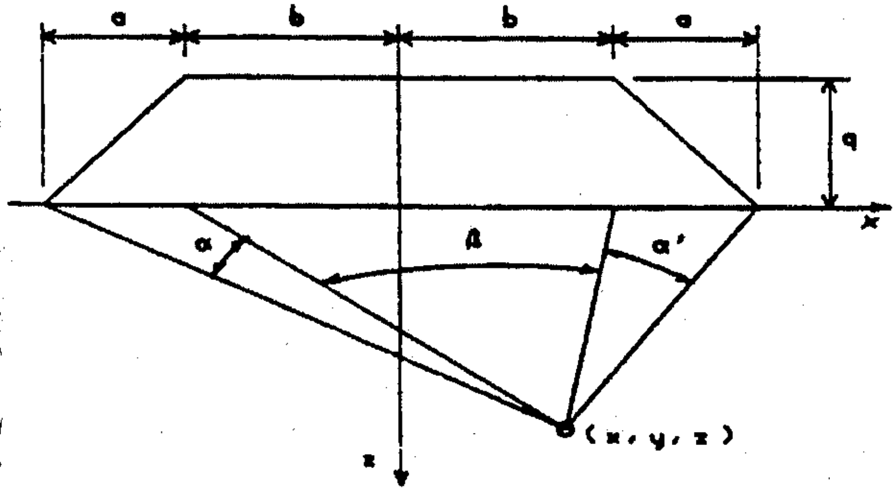
Cohesionless Soils ($\sigma'_0=\sigma'_c$)

$$(\delta_c)_{ult} = \sum \frac{1}{C'} H \log\left(\frac{\sigma'_f}{\sigma'_0}\right)$$

DLZ

SUBJECT			
Client	TranSystems Inc.	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	5 OF 33
Item	Settlement Analysis	COMP. BY	SJR DATE 6-13-07
Lucasville-Minford Road Embankment TOE		CHECKED BY	DAA DATE 6-25-07

INCREASE IN VERTICAL STRESS DUE TO EMBANKMENT LOADING



q = 5160 load

a = 86 width of slope

b = 100 top half-width of embankment

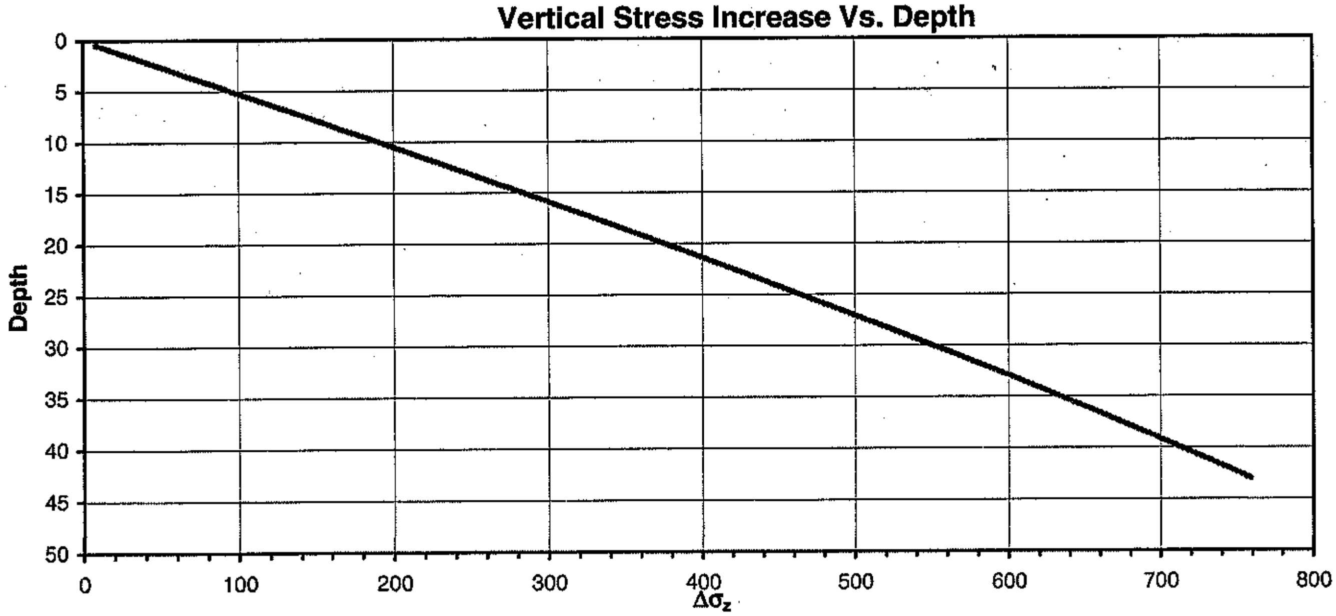
x = 186 distance from CL

z = 0 to 43 depth range

$$\sigma_v(z) := \left(\frac{q}{\pi a}\right)\left(a\cdot(\alpha(z)+\beta(z)+\alpha'(z)) + b\cdot(\alpha(z)+\alpha'(z)) + x\cdot(\alpha(z)-\alpha'(z))\right)$$

$$\beta(z) := \operatorname{atan}\left[\frac{(b-x)}{z}\right] + \operatorname{atan}\left[\frac{(b+x)}{z}\right] ; \quad \alpha'(z) := \operatorname{atan}\left[\frac{(a+b-x)}{z}\right] - \operatorname{atan}\left[\frac{(b-x)}{z}\right] \quad \alpha(z) := \operatorname{atan}\left[\frac{(a+b+x)}{z}\right] - \operatorname{atan}\left[\frac{(b+x)}{z}\right]$$

Vertical Stress Increase Vs. Depth



Reference: US Army Corps of Engineers EM 1110-1-1904 "Settlement Analysis", Table C-1

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT Tran Systems Corp / ODOT D-9 — PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth Bypass — SHEET NO. 6 OF 33
SUBJECT Downdrag Force on piles — COMP. BY SAK DATE 6-14-07
S.R. 728, Abutment Piles — CHECKED BY DAA DATE 6-25-07

Assumptions:
1) Soil profile encountered by boring TR-11 is assumed to be the most critical with respect to settlement.
2) Analysis for TR-11 is deemed to be representative of rear and forward abutment driven piles.
3) Analysis assumes no waiting period prior to driving piles.
4) HP 14X73 piles, driven to refusal on bedrock are assumed.

* From Settlement analysis of foundation soils under embankment loading, determine depth which settlements are limited to 0.4" or less.

Depth = 38.0' @t=0, Settlement from 38.0' to 43.0' ≤ 0.4"

* From Driven analysis we can estimate the skin friction resistance component from a depth of 0' to 38.0'.

From 0-38.0', skin friction = 152.4 K

* If piles are istalled prior to a consolidation period, the skin friction would be mobilimed as a downdrag force. DD = 152 K (unfactored)

To eleminate downdrag forces from reducing the allowable capacity of the piles, a waiting period of 280 days (U=98.6%) should be observed. (Assumes Wick Drains @ 3' Spacing)

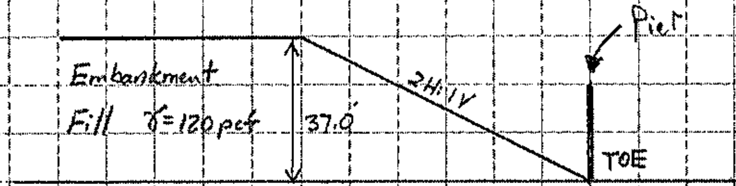
*Alternate waiting periods and associated downdrag forces

Percent Consolidation	Time (Days)	Downdrag* Force (Kips)
0	0	152 K
78.4	90	132 K
93.3	180	92 K
98.6	280	0

* Note: Unfactored Downdrag Force

SAK

Piers will utilize battered piles



* Total Primary Consolidation; $\delta_c = 2.5''$ (At TOE)

To prevent downdrag forces from effecting battered pile for the piers, we want to limit the remaining primary consolidation to 0.4" or less.

$$\frac{0.4''}{2.5''} = 0.16$$

* To limit $\delta_{c_{Rem}} \leq 0.4''$ we want $U = 1 - 0.16 = 84\%$

* Piles should not be driven until $U = 84\%$

↳ As per time-rate calculations, $t = 110$ days.
Assumes wick drain spacing = 3 ft.

SJK

Analyses based upon Driven pile analysis for HP 14x73 piles, assuming soil profile based on TR-11
Assumes 3 foot wick drain spacing for time-rate of consolidation calculations

Depth of Layer (ft)	Total Primary Consolidation	Settlement Remaining in Each Layer (inches) Percent Consolidation % 78.4	93.3	98.6
0.0-13.0	15.40	3.33	1.03	0.22
13.0-23.0	5.53	1.19	0.37	0.08
23.0-33.0	4.70	1.02	0.32	0.07
33.0-37.0	1.70	0.37	0.11	0.02
37.0-43.0	0.45	0.10	0.03	0.01

Depth Below Ground Surface (ft)	Cummulative Remaining Settlement (inches) Percent Consolidation % 78.4	93.3	98.6
0.00	6.00	1.86	0.39
13.00	2.68	0.83	0.17
23.00	1.48	0.46	0.10
33.00	0.46	0.14	0.03
37.00	0.10	0.03	0.01
43.00	0.00	0.00	0.00

1) At 90 days (U=78.4%), Remaining settlements greater than 0.4" will occur from depths of 0 to 34' below the existing ground surface.

Skin friction at these depths will be mobilized as a downdrag force on the piles
DD=132 kips

2) At 180 days (U=93.3%), Remaining settlements greater than 0.4" will occur from depths of 0 to 25' below the existing ground surface.

Skin friction at these depths will be mobilized as a downdrag force on the piles
DD=92 kips

2) At 280 days (U=98.6%), Remaining settlement is approximately equal to 0.4"

It is assumed that no skin friction will be mobilized as a downdrag force.
DD=0 kips

Actual waiting periods may be modified in the field by the Engineer based upon readings from settlement platforms and piezometers.

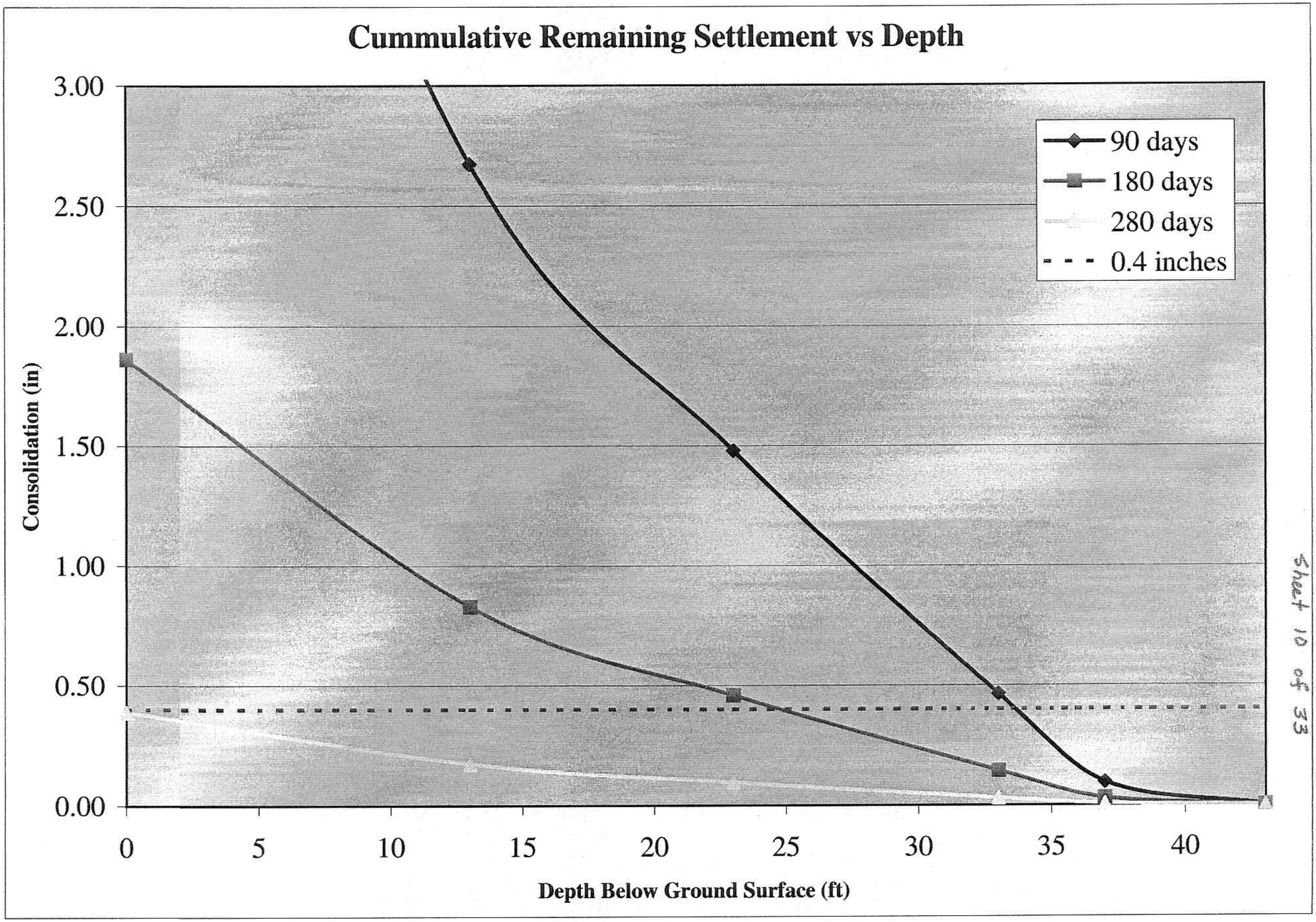
Sheet 9 of 33
SJK 6-14-07
DAA 6-25-07

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
Lucasville-Minford Rd **Based upon boring TR-11**
Reference: FHWA-RD-86-168

Wick Drain Spacing **3.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	*Remaining* δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	27.8	3.15	**0.03**	**21.5**	**27.8**
5	0.0151	0.0003	0.09	0.08	16.8	4.7	23.1				
10	0.0302	0.0006	0.16	0.08	22.9	6.4	21.4				
15	0.0454	0.0010	0.22	0.08	28.6	7.9	19.9	*Assumes double drainage*			
20	0.0605	0.0013	0.28	0.09	33.9	9.4	18.4	*Spacing = 3 ft (triangular)*			
25	0.0756	0.0016	0.33	0.09	38.8	10.8	17.0				
30	0.0907	0.0019	0.38	0.09	43.4	12.1	15.7				
35	0.1058	0.0023	0.43	0.09	47.7	13.3	14.5				
40	0.1209	0.0026	0.47	0.09	51.7	14.4	13.4				
45	0.1361	0.0029	0.51	0.09	55.5	15.4	12.4				
50	0.1512	0.0032	0.55	0.09	58.9	16.4	11.4				
55	0.1663	0.0036	0.58	0.09	62.1	17.3	10.5				
60	0.1814	0.0039	0.61	0.09	65.1	18.1	9.7				
65	0.1965	0.0042	0.64	0.10	67.8	18.8	9.0				
70	0.2116	0.0045	0.67	0.10	70.3	19.5	8.3				
75	0.2268	0.0049	0.70	0.10	72.6	20.2	7.6				
80	0.2419	0.0052	0.72	0.10	74.7	20.8	7.0				
85	0.2570	0.0055	0.74	0.10	76.7	21.3	6.5				
90	0.2721	0.0058	0.76	0.10	78.4	21.8	6.0				
95	0.2872	0.0062	0.78	0.10	80.1	22.3	5.5				
100	0.3023	0.0065	0.79	0.10	81.5	22.7	5.1				
105	0.3175	0.0068	0.81	0.10	82.9	23.0	4.8				
110	0.3326	0.0071	0.82	0.11	84.1	23.4	4.4				
115	0.3477	0.0075	0.83	0.11	85.3	23.7	4.1				
120	0.3628	0.0078	0.85	0.11	86.3	24.0	3.8				
125	0.3779	0.0081	0.86	0.11	87.2	24.2	3.6				
130	0.3930	0.0084	0.87	0.11	88.0	24.5	3.3				
135	0.4082	0.0088	0.87	0.11	88.8	24.7	3.1				
140	0.4233	0.0091	0.88	0.11	89.5	24.9	2.9				
145	0.4384	0.0094	0.89	0.11	90.1	25.1	2.7				
150	0.4535	0.0097	0.89	0.11	90.7	25.2	2.6				
155	0.4686	0.0101	0.90	0.11	91.2	25.4	2.4				
160	0.4837	0.0104	0.91	0.12	91.7	25.5	2.3				
165	0.4989	0.0107	0.91	0.12	92.1	25.6	2.2				
170	0.5140	0.0110	0.92	0.12	92.6	25.7	2.1				
175	0.5291	0.0114	0.92	0.12	92.9	25.8	2.0				
180	0.5442	0.0117	0.92	0.12	93.3	25.9	1.9				
185	0.5593	0.0120	0.93	0.12	93.6	26.0	1.8				
190	0.5745	0.0123	0.93	0.12	93.9	26.1	1.7				
195	0.5896	0.0127	0.93	0.12	94.3	26.2	1.6				
200	0.6047	0.0130	0.94	0.12	94.5	26.3	1.5				
205	0.6198	0.0133	0.94	0.13	94.8	26.4	1.4				
210	0.6349	0.0136	0.94	0.13	95.1	26.4	1.4				
215	0.6500	0.0140	0.95	0.13	95.4	26.5	1.3				
220	0.6652	0.0143	0.95	0.13	95.7	26.6	1.2				
225	0.6803	0.0146	0.95	0.13	95.9	26.7	1.1				
230	0.6954	0.0149	0.96	0.13	96.2	26.7	1.1				
235	0.7105	0.0153	0.96	0.13	96.5	26.8	1.0				
240	0.7256	0.0156	0.96	0.13	96.7	26.9	0.9				
245	0.7407	0.0159	0.97	0.13	97.0	27.0	0.8				
250	0.7559	0.0162	0.97	0.13	97.2	27.0	0.8				
255	0.7710	0.0165	0.97	0.14	97.5	27.1	0.7				
260	0.7861	0.0169	0.97	0.14	97.7	27.2	0.6				
265	0.8012	0.0172	0.98	0.14	98.0	27.2	0.6				
270	0.8163	0.0175	0.98	0.14	98.2	27.3	0.5				
275	0.8314	0.0178	0.98	0.14	98.4	27.4	0.4				
280	0.8466	0.0182	0.98	0.14	98.6	27.4	0.4				
285	0.8617	0.0185	0.99	0.14	98.8	27.5	0.3				
290	0.8768	0.0188	0.99	0.14	98.9	27.5	0.3				


Cummulative Remaining Settlement vs Depth
90 days
180 days
280 days
0.4 inches
3.00
2.50
2.00
1.50
1.00
0.50
0.00
Consolidation (in)
0
5
10
15
20
25
30
35
40
Depth Below Ground Surface (ft)
Sheet 10 of 33

DRIVEN 1.0

Sheet 11 of 33

GENERAL PROJECT INFORMATION

SJK 6-14-07
DAA 6-25-07

Filename: C:\DRIVEN\728TR11.DVN
Project Name: SCI-823 Project Date: 06/15/2007
Project Client: TranSystems Corp / ODOT
Computed By: sjr
Project Manager: Nix

Based upon TR-11

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 5.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	13.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel
2	Cohesive	24.00 ft	0.00%	120.00 pcf	1000.00 psf	T-79 Steel
3	Cohesive	6.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel

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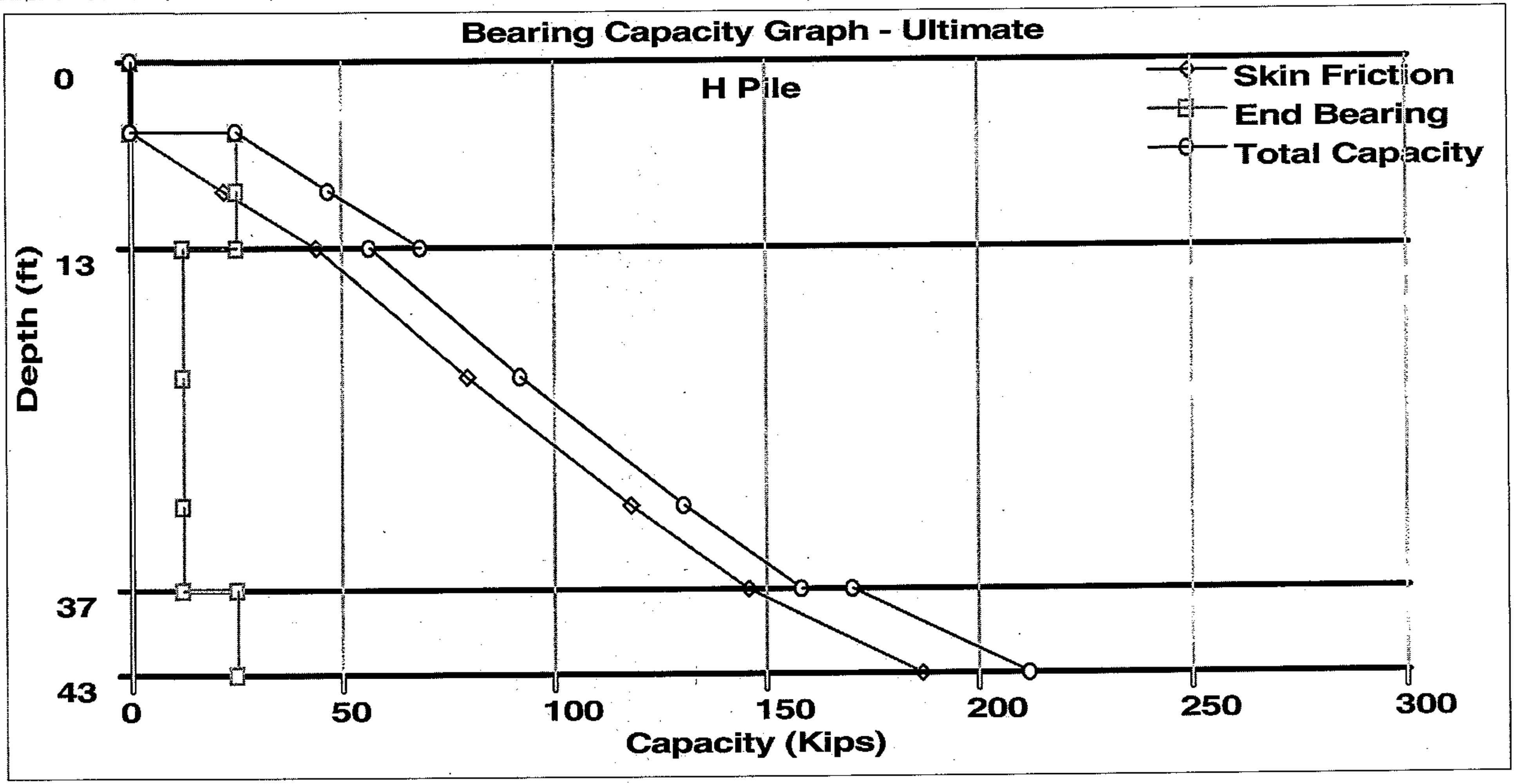

Analysis HP-14x73, 95 ton piles

Sheet 12 of 33

ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	1165.00 psf	21.95 Kips
12.99 ft	Cohesive	N/A	N/A	1165.00 psf	43.74 Kips
13.01 ft	Cohesive	N/A	N/A	803.52 psf	43.83 Kips
22.01 ft	Cohesive	N/A	N/A	840.55 psf	79.38 Kips 22
31.01 ft	Cohesive	N/A	N/A	877.57 psf	118.07 Kips 31
36.99 ft	Cohesive	N/A	N/A	902.17 psf	145.50 Kips
37.01 ft	Cohesive	N/A	N/A	1403.59 psf	145.61 Kips
42.99 ft	Cohesive	N/A	N/A	1460.99 psf	186.67 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
12.99 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
13.01 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
22.01 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
31.01 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
36.99 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
37.01 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
42.99 ft	Cohesive	N/A	N/A	N/A	24.81 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	24.81 Kips	24.81 Kips
9.01 ft	21.95 Kips	24.81 Kips	46.77 Kips
12.99 ft	43.74 Kips	24.81 Kips	68.55 Kips
13.01 ft	43.83 Kips	12.41 Kips	56.24 Kips
22.01 ft	79.38 Kips	12.41 Kips	91.79 Kips
31.01 ft	118.07 Kips	12.41 Kips	130.47 Kips
36.99 ft	145.50 Kips	12.41 Kips	157.91 Kips
37.01 ft	145.61 Kips	24.81 Kips	170.43 Kips
42.99 ft	186.67 Kips	24.81 Kips	211.48 Kips

DRIVEN 1.0

GENERAL PROJECT INFORMATION

SJK 6-14-07
DAA 6-25-07

Filename: C:\DRIVEN\728TR14.DVN
Project Name: SCI-823
Project Date: 06/15/2007
Based upon TR-14
Project Client: TranSystems Corp
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: H Pile - HP14X73
Top of Pile: 5.00 ft
Perimeter Analysis: Box
Tip Analysis: Box Area

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	8.50 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel
2	Cohesive	25.00 ft	0.00%	120.00 pcf	1000.00 psf	T-79 Steel

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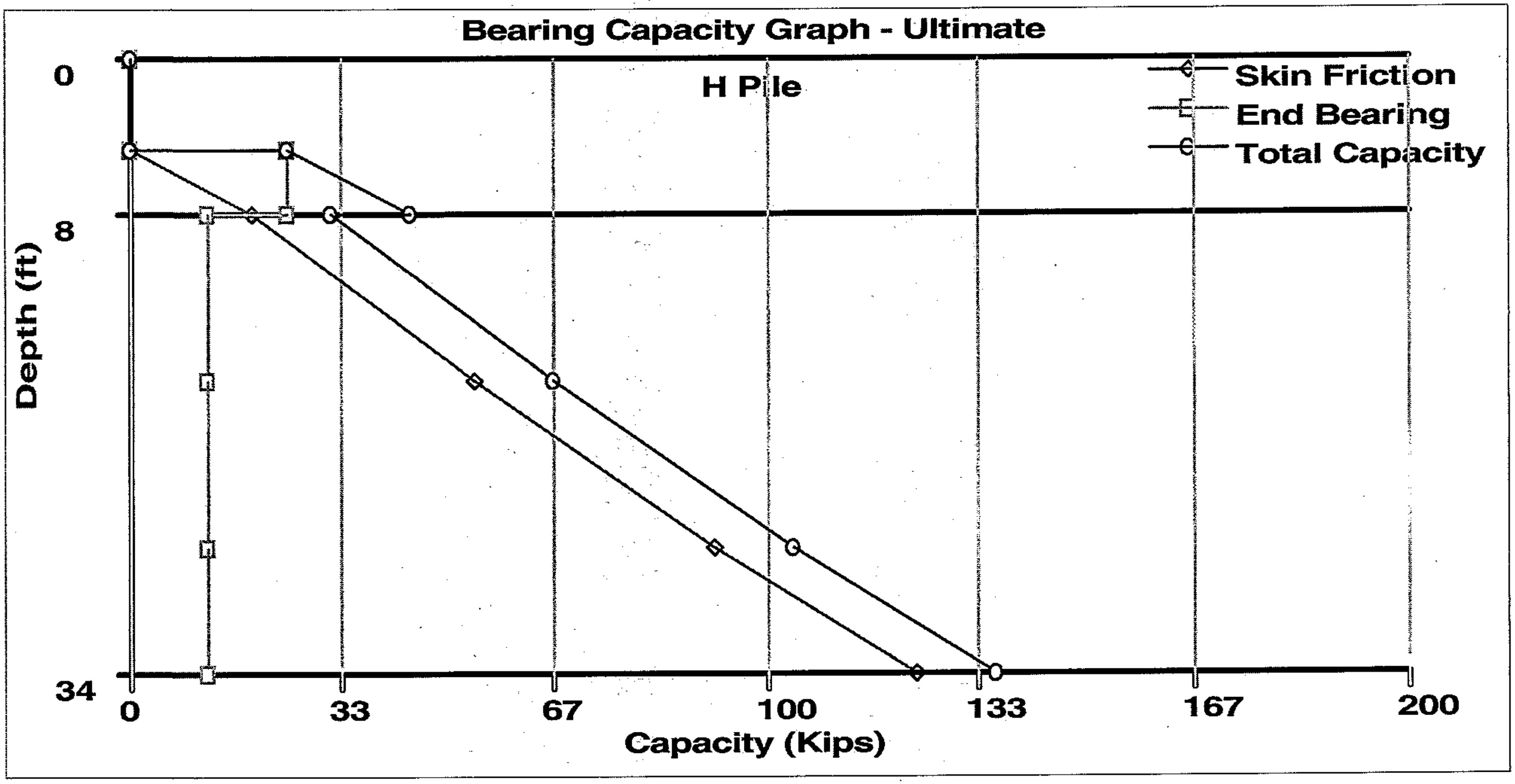


ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
8.49 ft	Cohesive	N/A	N/A	1165.00 psf	19.11 Kips
8.51 ft	Cohesive	N/A	N/A	800.00 psf	19.20 Kips
17.51 ft	Cohesive	N/A	N/A	822.03 psf	53.97 Kips
26.51 ft	Cohesive	N/A	N/A	859.06 psf	91.86 Kips
33.49 ft	Cohesive	N/A	N/A	887.77 psf	123.41 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
8.49 ft	Cohesive	N/A	N/A	N/A	24.81 Kips
8.51 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
17.51 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
26.51 ft	Cohesive	N/A	N/A	N/A	12.41 Kips
33.49 ft	Cohesive	N/A	N/A	N/A	12.41 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	24.81 Kips	24.81 Kips
8.49 ft	19.11 Kips	24.81 Kips	43.92 Kips
8.51 ft	19.20 Kips	12.41 Kips	31.60 Kips
17.51 ft	53.97 Kips	12.41 Kips	66.37 Kips
26.51 ft	91.86 Kips	12.41 Kips	104.27 Kips
33.49 ft	123.41 Kips	12.41 Kips	135.82 Kips

DRIVEN 1.0

Sheet 19 of 33
SJR 6-14-07
DAA 6-25-07

GENERAL PROJECT INFORMATION

Based upon boring TR-11

Filename: C:\DRIVEN\728TR11P.DVN
Project Name: SCI-823 Project Date: 06/15/2007
Project Client: TranSystems Corp / ODOT
Computed By: sjr
Project Manager: Nix

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 5.00 ft
Diameter of Pile: 14.00 in

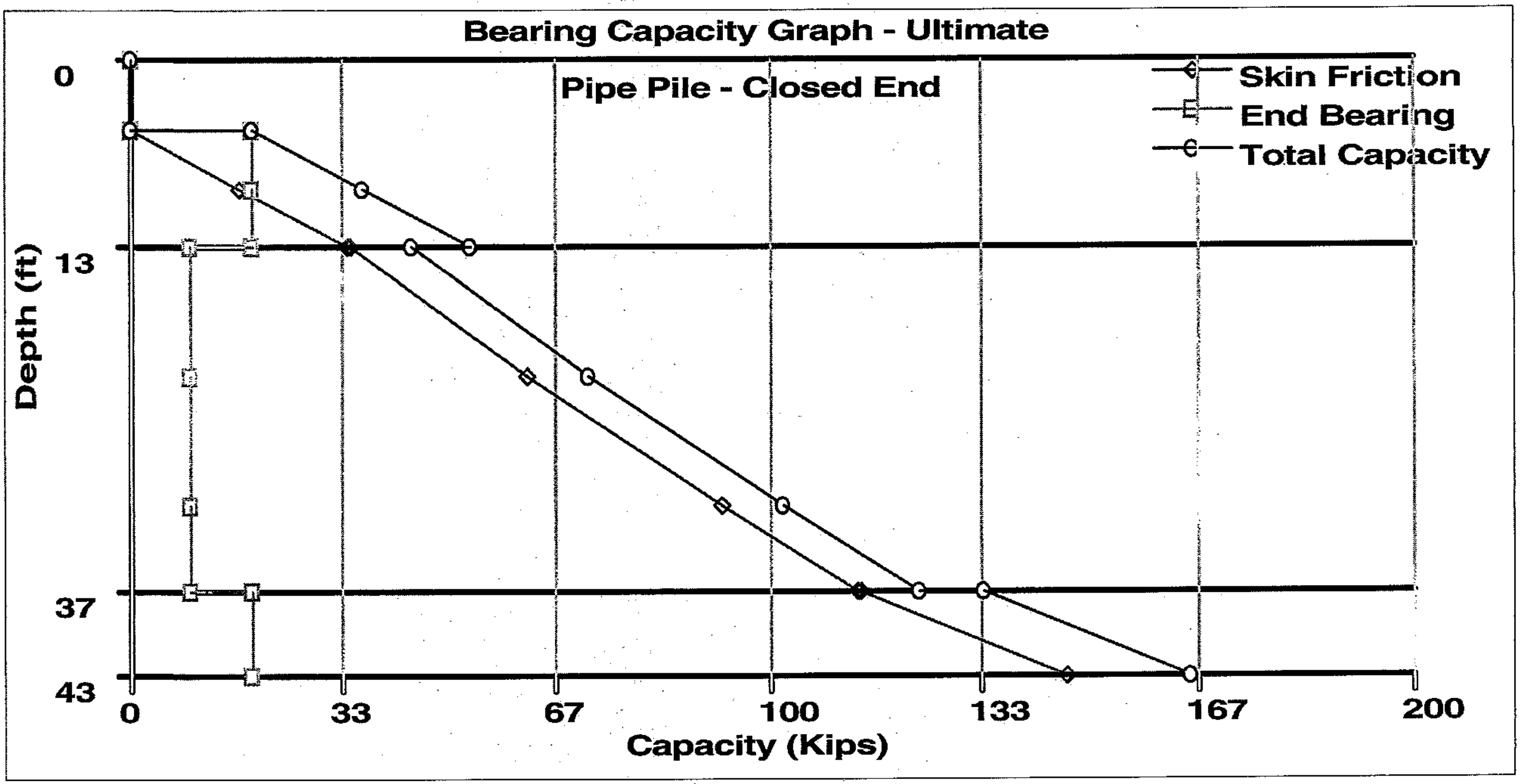
ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	13.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel
2	Cohesive	24.00 ft	0.00%	120.00 pcf	1000.00 psf	T-79 Steel
3	Cohesive	6.00 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel

Filename: C:\DRIVEN\[illegible]TR1[illegible].DVN



ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
9.01 ft	Cohesive	N/A	N/A	1165.00 psf	17.12 Kips
12.99 ft	Cohesive	N/A	N/A	1165.00 psf	34.12 Kips
13.01 ft	Cohesive	N/A	N/A	805.76 psf	34.19 Kips
22.01 ft	Cohesive	N/A	N/A	844.33 psf	62.04 Kips
31.01 ft	Cohesive	N/A	N/A	882.90 psf	92.44 Kips
36.99 ft	Cohesive	N/A	N/A	908.53 psf	114.04 Kips
37.01 ft	Cohesive	N/A	N/A	1418.43 psf	114.13 Kips
42.99 ft	Cohesive	N/A	N/A	1478.23 psf	146.54 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
9.01 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
12.99 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
13.01 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
22.01 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
31.01 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
36.99 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
37.01 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
42.99 ft	Cohesive	N/A	N/A	N/A	19.24 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	19.24 Kips	19.24 Kips
9.01 ft	17.12 Kips	19.24 Kips	36.36 Kips
12.99 ft	34.12 Kips	19.24 Kips	53.36 Kips
13.01 ft	34.19 Kips	9.62 Kips	43.81 Kips
22.01 ft	62.04 Kips	9.62 Kips	71.66 Kips
31.01 ft	92.44 Kips	9.62 Kips	102.06 Kips
36.99 ft	114.04 Kips	9.62 Kips	123.67 Kips
37.01 ft	114.13 Kips	19.24 Kips	133.38 Kips
42.99 ft	146.54 Kips	19.24 Kips	165.78 Kips

$$Q_{allow} = \frac{165.8\ k}{2} = 82.9\ k$$

$$Q_{allow} = 41.45 \text{ tons}$$

DRIVEN 1.0

GENERAL PROJECT INFORMATION

Filename: C:\DRIVEN\728TR14P.DVN
Project Name: SCI-823 Project Date: 06/15/2007
Project Client: TranSystems Corp
Computed By: sjr
Project Manager: Nix

Based upon TR-14

PILE INFORMATION

Pile Type: Pipe Pile - Closed End
Top of Pile: 5.00 ft
Diameter of Pile: 14.00 in

ULTIMATE CONSIDERATIONS

Water Table Depth At Time Of:	- Drilling:	10.00 ft
	- Driving/Restrike	10.00 ft
	- Ultimate:	10.00 ft
Ultimate Considerations:	- Local Scour:	0.00 ft
	- Long Term Scour:	0.00 ft
	- Soft Soil:	0.00 ft

ULTIMATE PROFILE

Layer	Type	Thickness	Driving Loss	Unit Weight	Strength	Ultimate Curve
1	Cohesive	8.50 ft	0.00%	120.00 pcf	2000.00 psf	T-79 Steel
2	Cohesive	25.00 ft	0.00%	120.00 pcf	1000.00 psf	T-79 Steel

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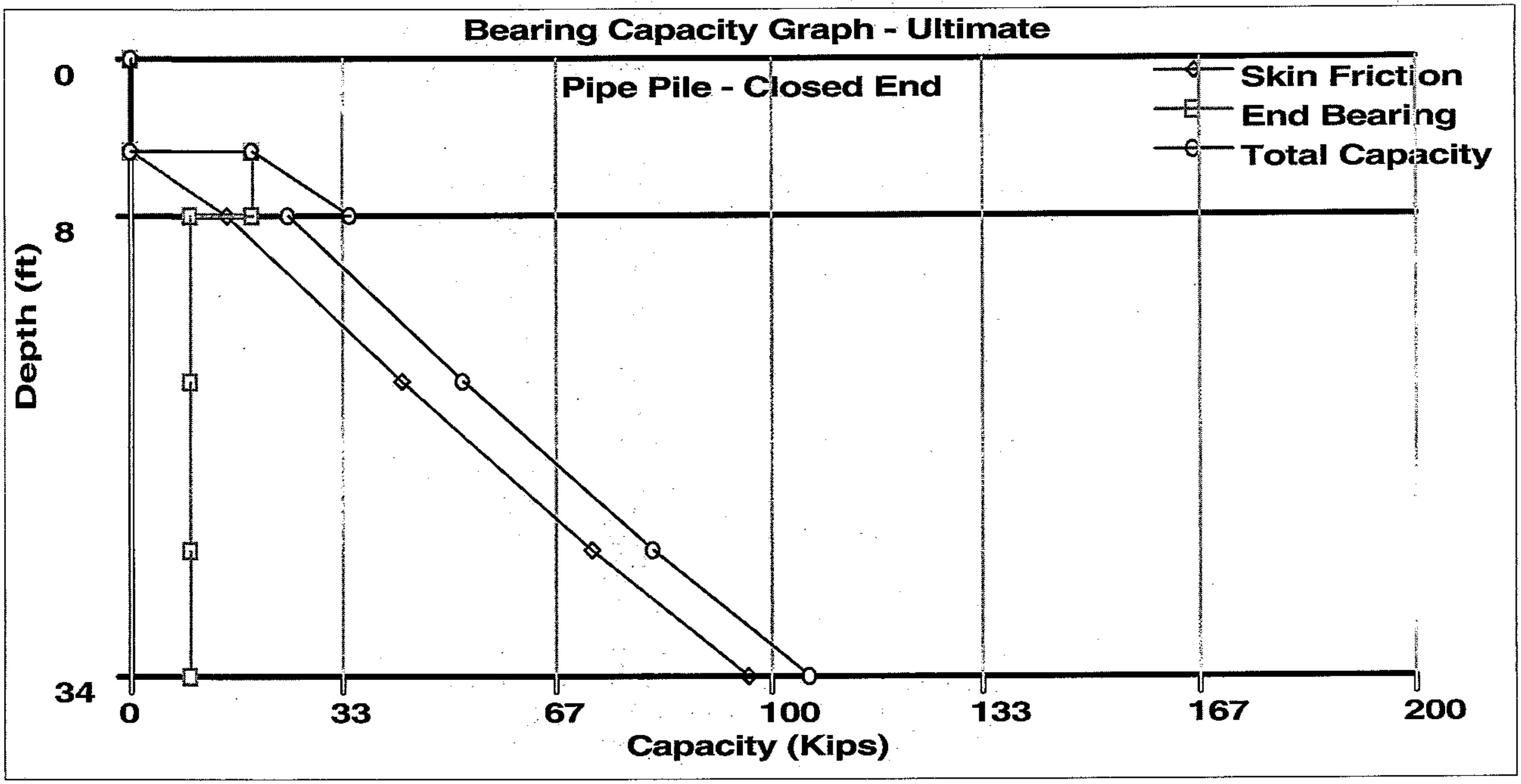


ULTIMATE - SKIN FRICTION

Depth	Soil Type	Effective Stress At Midpoint	Sliding Friction Angle	Adhesion	Skin Friction
0.01 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	0.00 psf	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	1165.00 psf	0.00 Kips
8.49 ft	Cohesive	N/A	N/A	1165.00 psf	14.90 Kips
8.51 ft	Cohesive	N/A	N/A	800.00 psf	14.97 Kips
17.51 ft	Cohesive	N/A	N/A	825.04 psf	42.19 Kips
26.51 ft	Cohesive	N/A	N/A	863.61 psf	71.95 Kips
33.49 ft	Cohesive	N/A	N/A	893.53 psf	96.79 Kips

ULTIMATE - END BEARING

Depth	Soil Type	Effective Stress At Tip	Bearing Cap. Factor	Limiting End Bearing	End Bearing
0.01 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
4.99 ft	Cohesive	N/A	N/A	N/A	0.00 Kips
5.00 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
8.49 ft	Cohesive	N/A	N/A	N/A	19.24 Kips
8.51 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
17.51 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
26.51 ft	Cohesive	N/A	N/A	N/A	9.62 Kips
33.49 ft	Cohesive	N/A	N/A	N/A	9.62 Kips

ULTIMATE - SUMMARY OF CAPACITIES

Depth	Skin Friction	End Bearing	Total Capacity
0.01 ft	0.00 Kips	0.00 Kips	0.00 Kips
4.99 ft	0.00 Kips	0.00 Kips	0.00 Kips
5.00 ft	0.00 Kips	19.24 Kips	19.24 Kips
8.49 ft	14.90 Kips	19.24 Kips	34.14 Kips
8.51 ft	14.97 Kips	9.62 Kips	24.60 Kips
17.51 ft	42.19 Kips	9.62 Kips	51.81 Kips
26.51 ft	71.95 Kips	9.62 Kips	81.57 Kips
33.49 ft	96.79 Kips	9.62 Kips	106.41 Kips

$$Q_{allow} = \frac{106.4^{K}}{2} = 53.2^{K}$$

$$Q_{allow} = 26.2 \text{ tons}$$

DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT Tran Systems Corp / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT SCI-823 Portsmouth Bypass SHEET NO. 27 OF 33
SUBJECT Drilled Shaft End bearing and side friction SR 728 COMP. BY SJK DATE 6-15-07
CHECKED BY DAA DATE 6-25-07

*For similar Rock Cores tested on SCI-823 project, q_u ranges from 2,000 to 7,000 psi.
Soft to Medium Hard Sandstone / Siltstone argillaceous, highly weathered

*Assume q_u = 2000 psi

RQD: 25% to 92%

Typically q_{allow} = 30 ksf to 40 ksf for this type of rock.

Use q_a = 40 ksf for End Bearing Drilled Shafts

Side Friction *Assumes Smooth Rock Socket

$$f_{max} = 0.65\, p_a \left[q_u / p_a \right]^{0.5} \leq 0.65\, p_a \left[f_c' / p_a \right]^{0.5}$$

Ref: FHWA-IF-99-025 E_q 11.6

p_a = 14.7 psi f_c' = 4500 psi q_u = 2000 psi

$$f_{max} = 0.65\,(14.7\,\text{psi}) \left[\frac{2000\,\text{psi}}{14.7\,\text{psi}} \right]^{0.5} = 111\ \text{psi}$$

f_{max} = 111 psi = 16,049 psf

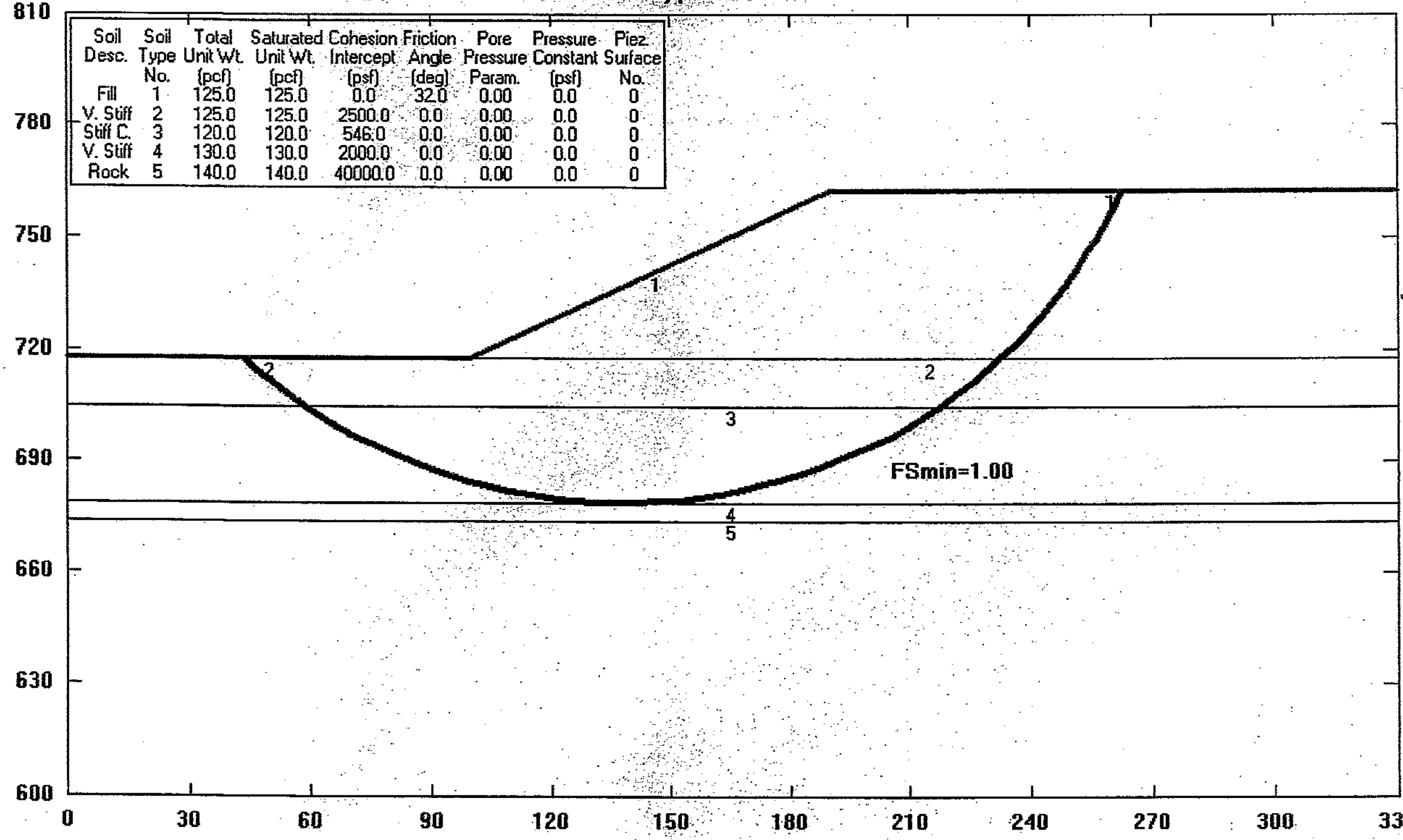
f_{allow} = 5,350 psf

Use f_{allow} = 5,000 psf

SJK

* From Interchange Report

Lucasville Main Alignment-No Stage Const
Portsmouth Bypass



Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	0
V. Stiff	2	125.0	125.0	2500.0	0.0	0.00	0.0	0
Stiff C.	3	120.0	120.0	546.0	0.0	0.00	0.0	0
V. Stiff	4	130.0	130.0	2000.0	0.0	0.00	0.0	0
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	0

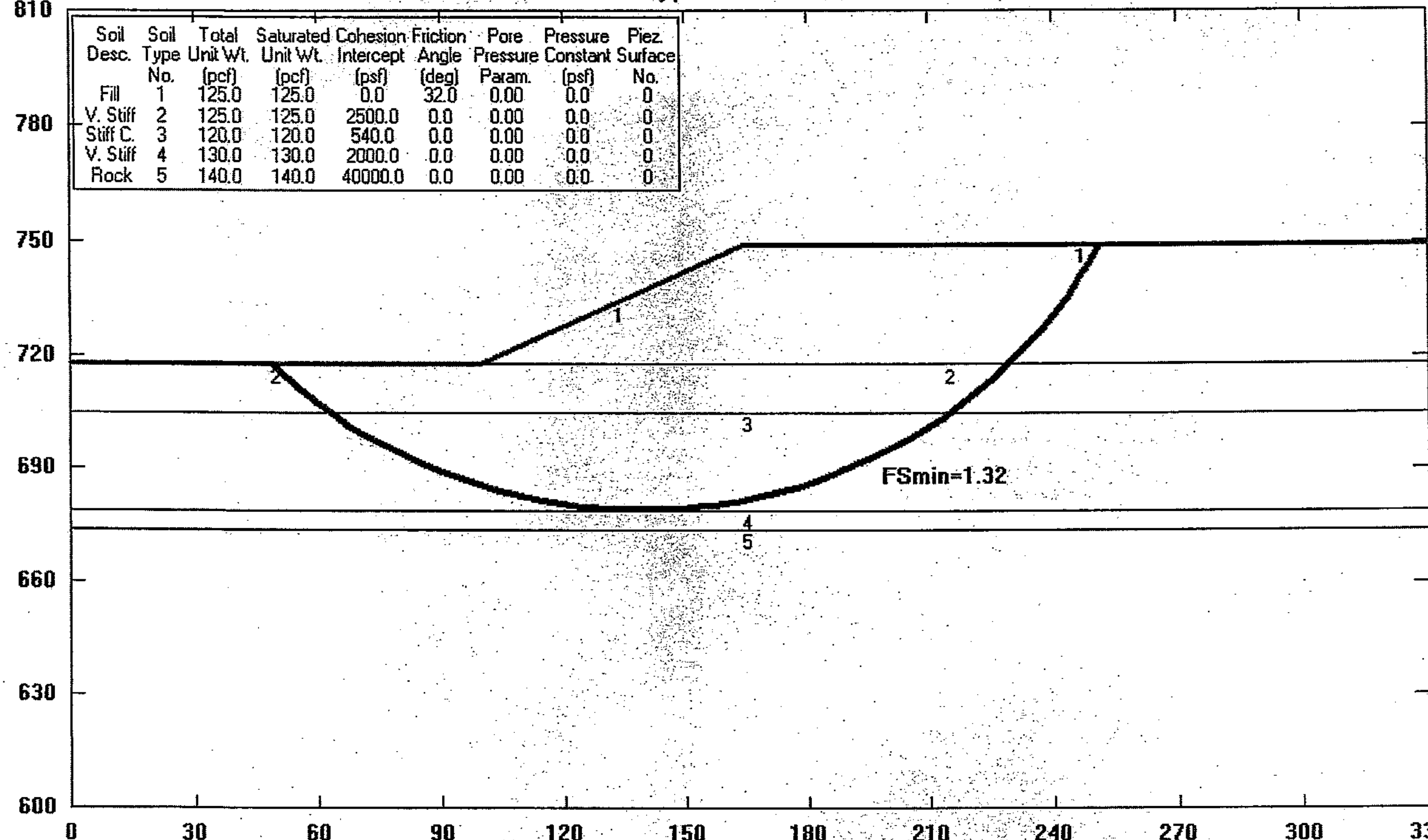
Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Sheet 29 of 33

Lucasville Main Alignment-Und Stage 1
Portsmouth Bypass

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	0
V. Stiff	2	125.0	125.0	2500.0	0.0	0.00	0.0	0
Stiff C.	3	120.0	120.0	540.0	0.0	0.00	0.0	0
V. Stiff	4	130.0	130.0	2000.0	0.0	0.00	0.0	0
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	0

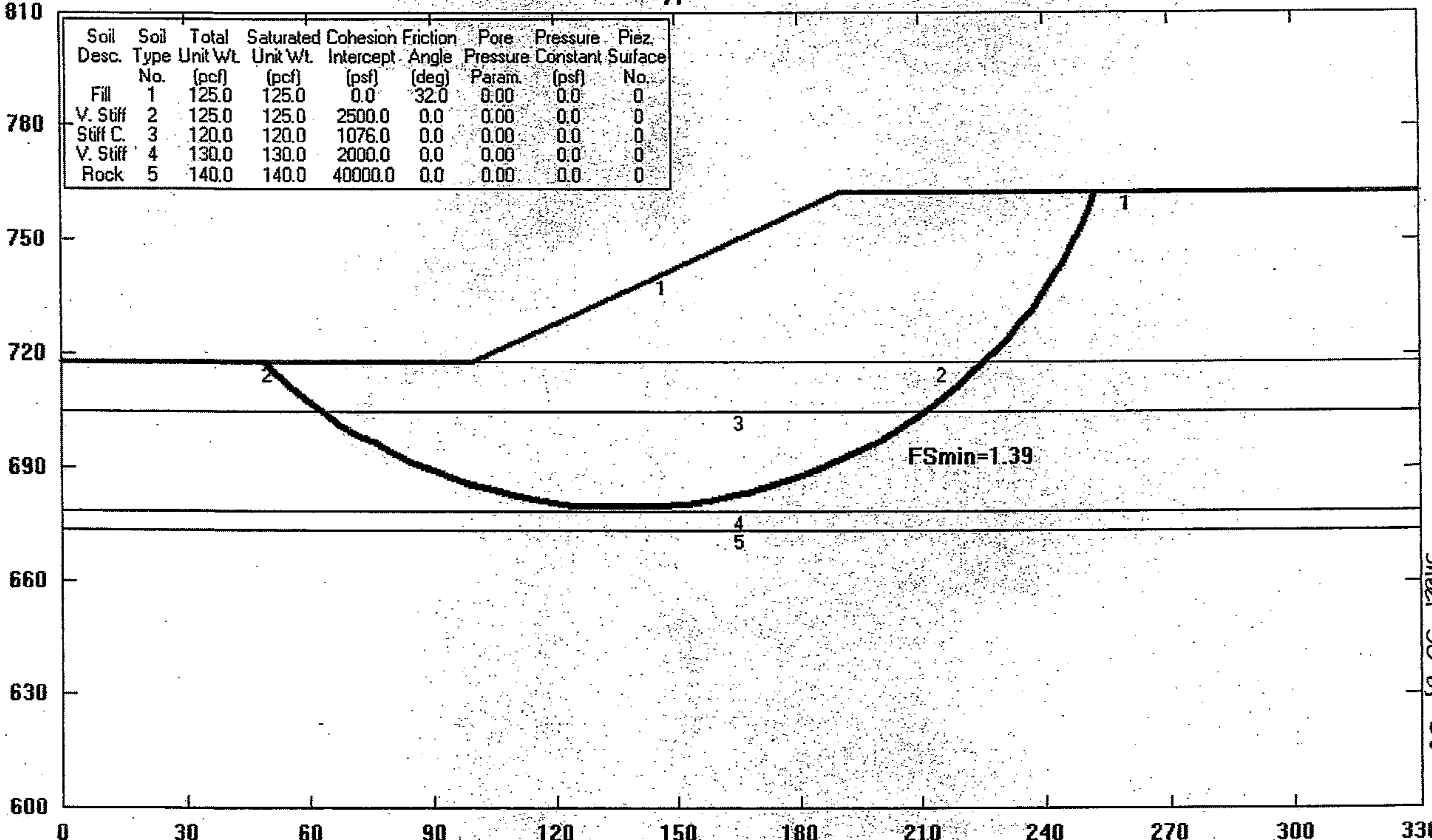


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Lucasville Main Alignment-Und Stage 2
Portsmouth Bypass

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	0
V. Stiff	2	125.0	125.0	2500.0	0.0	0.00	0.0	0
Stiff C.	3	120.0	120.0	1076.0	0.0	0.00	0.0	0
V. Stiff	4	130.0	130.0	2000.0	0.0	0.00	0.0	0
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	0

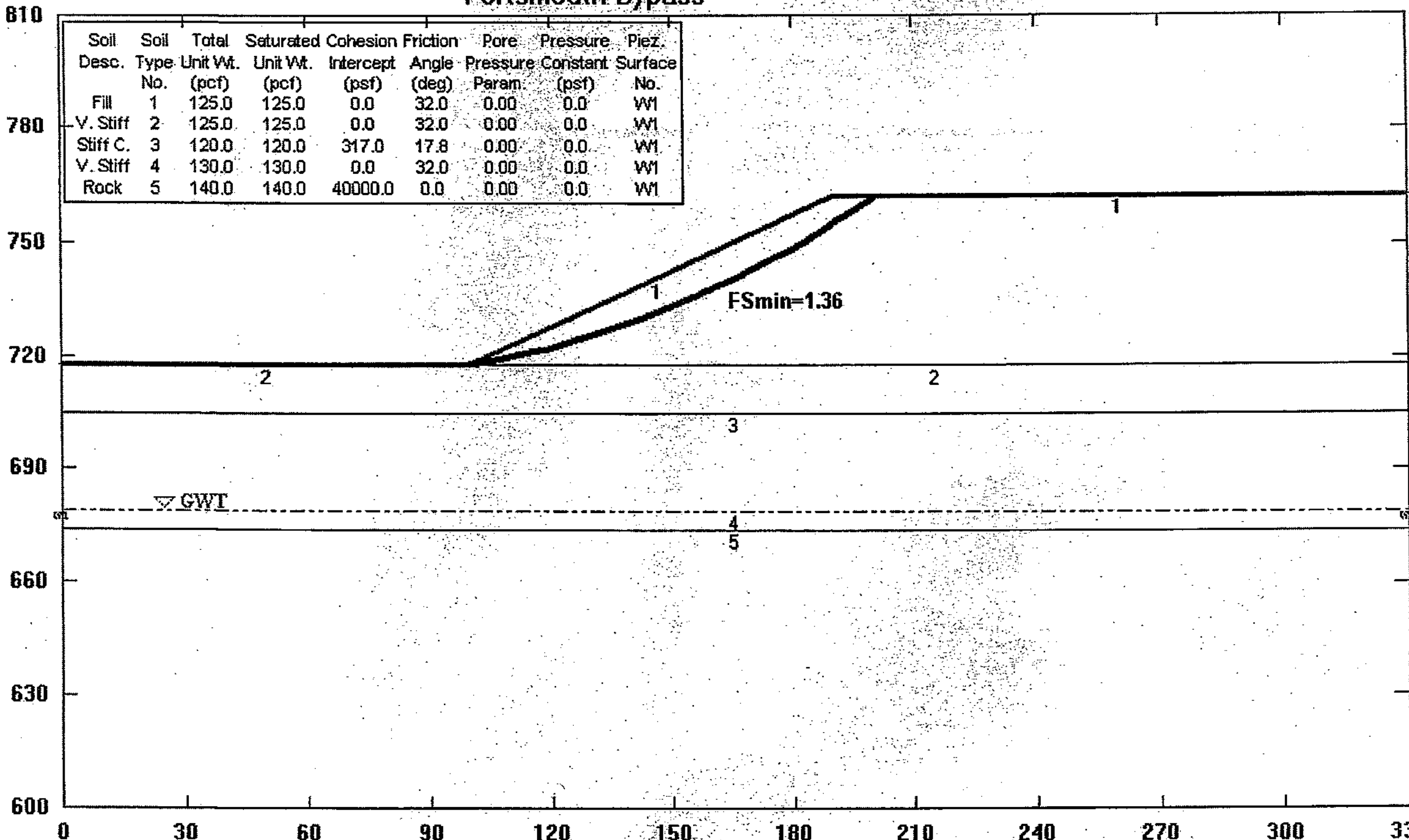


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Lucasville Main Alignment-Dra. No Stage
Portsmouth Bypass

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	W1
V. Stiff	2	125.0	125.0	0.0	32.0	0.00	0.0	W1
Stiff C.	3	120.0	120.0	317.0	17.8	0.00	0.0	W1
V. Stiff	4	130.0	130.0	0.0	32.0	0.00	0.0	W1
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	W1

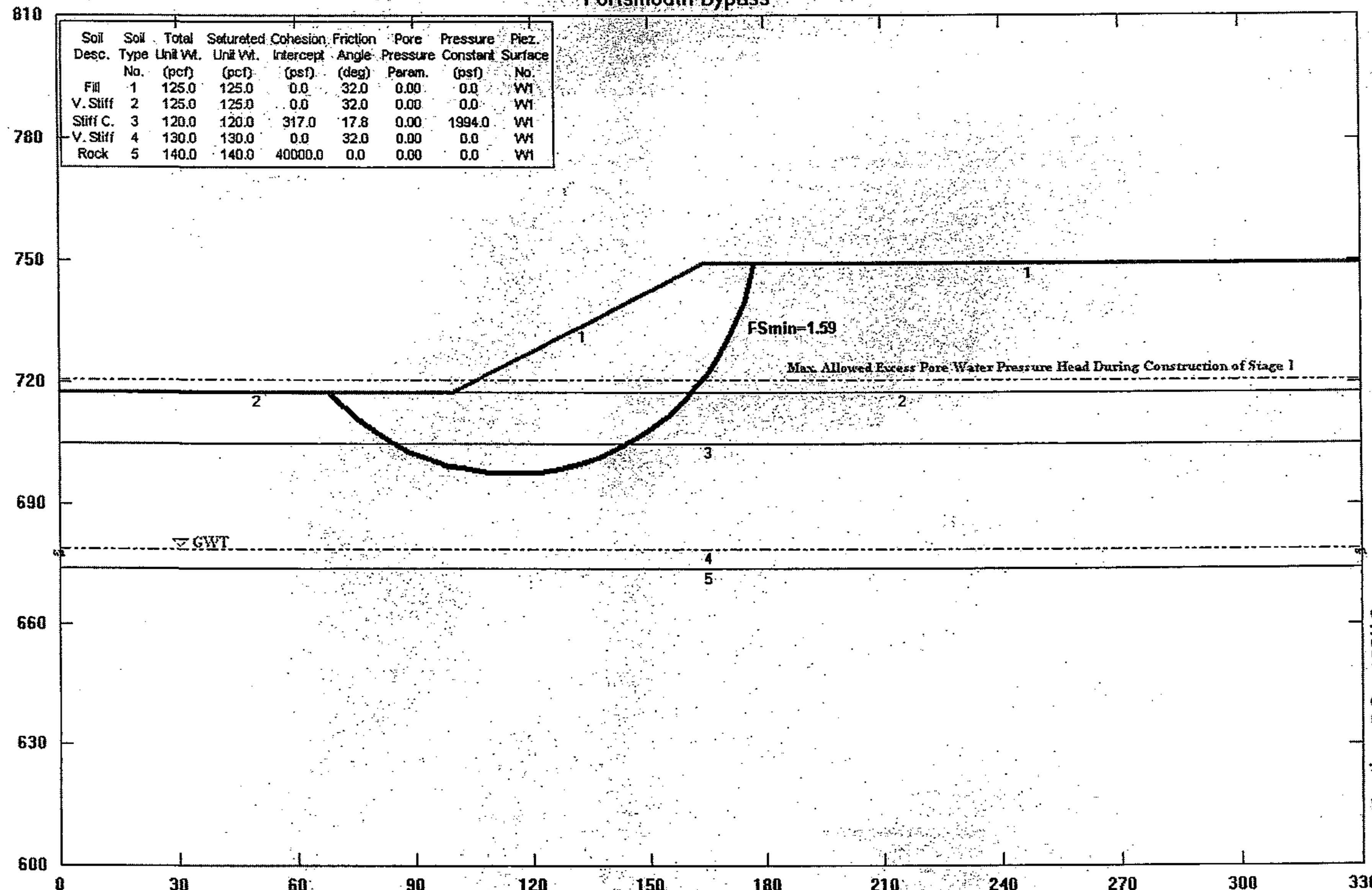


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Lucasville Main Alignment-Dra. Stage 1
Portsmouth Bypass

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	W1
V. Stiff	2	125.0	125.0	0.0	32.0	0.00	0.0	W1
Stiff C.	3	120.0	120.0	317.0	17.8	0.00	1994.0	W1
V. Stiff	4	130.0	130.0	0.0	32.0	0.00	0.0	W1
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	W1

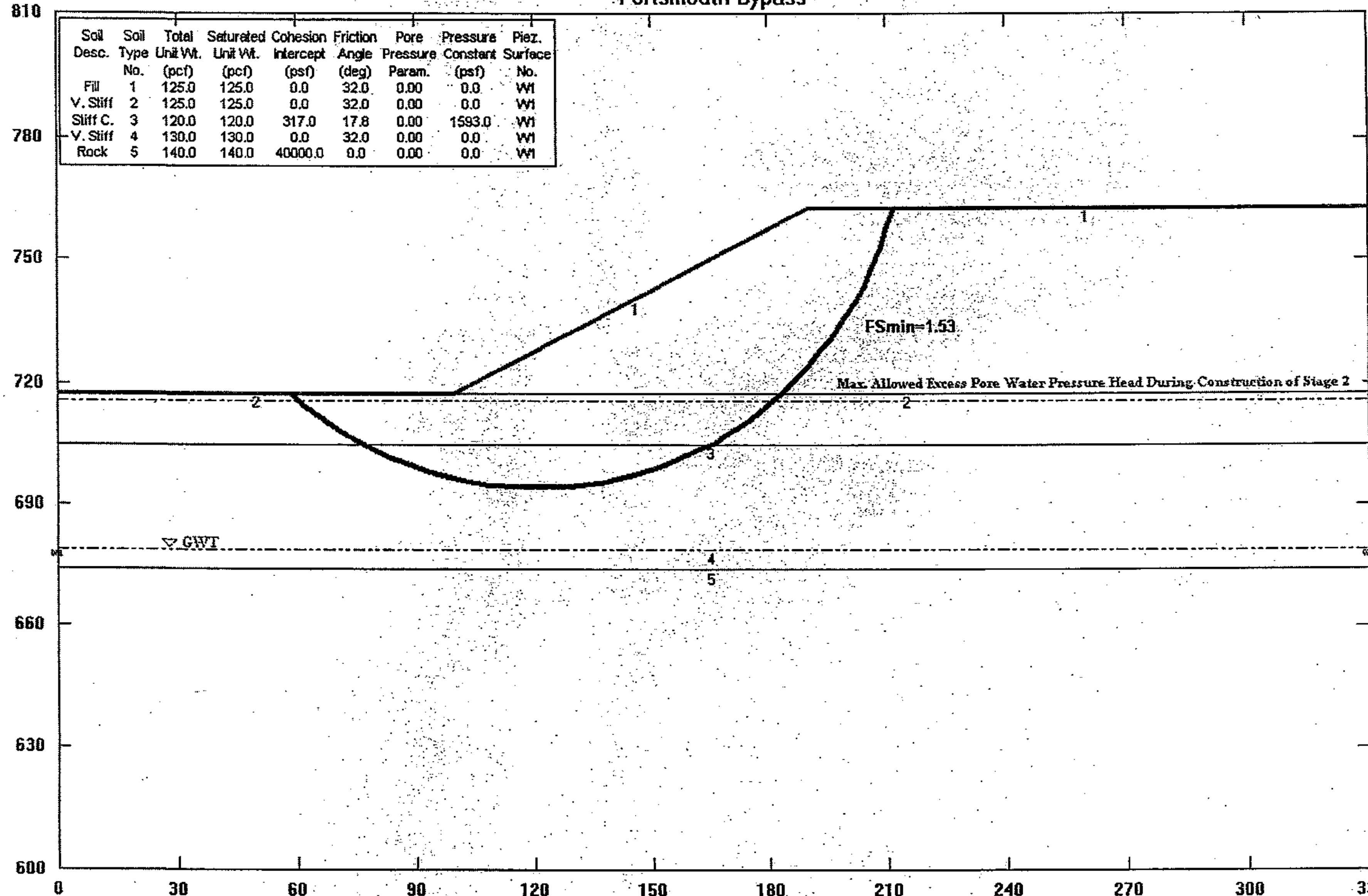


Safety Factors Are Calculated By The Modified Bishop Method

DLZ

Lucasville Main Alignment-Dra. Stage 2
Portsmouth Bypass

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Fill	1	125.0	125.0	0.0	32.0	0.00	0.0	W1
V. Stiff	2	125.0	125.0	0.0	32.0	0.00	0.0	W1
Stiff C.	3	120.0	120.0	317.0	17.8	0.00	1593.0	W1
V. Stiff	4	130.0	130.0	0.0	32.0	0.00	0.0	W1
Rock	5	140.0	140.0	40000.0	0.0	0.00	0.0	W1



Safety Factors Are Calculated By The Modified Bishop Method

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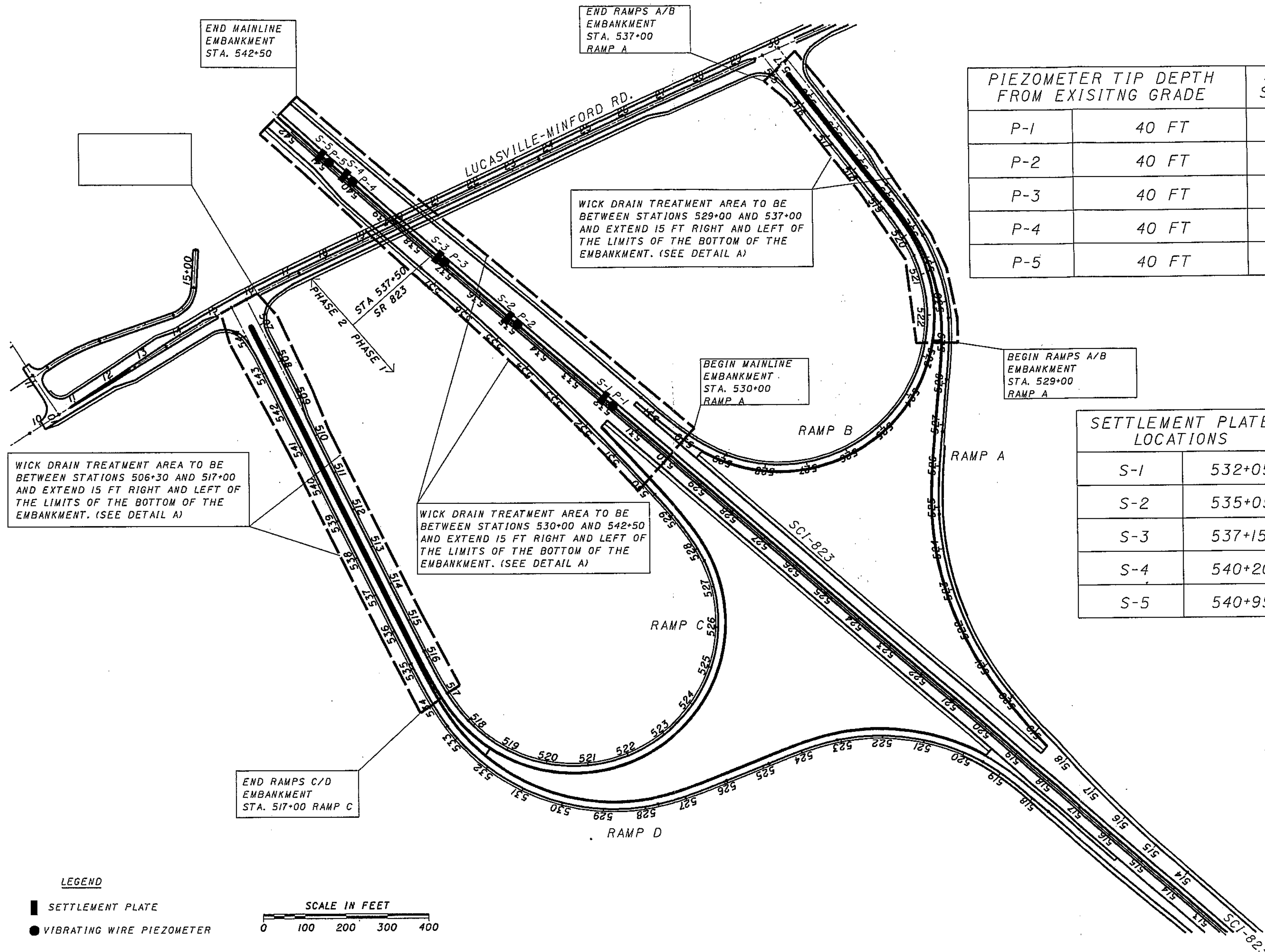
APPENDIX V

Prefabricated Vertical (Wick) Drain Instrumentation Plan
Instrumentation Details



PIEZOMETER TIP DEPTH FROM EXISITNG GRADE		SR 823 STATION
P-1	40 FT	531+95
P-2	40 FT	534+95
P-3	40 FT	537+10
P-4	40 FT	540+15
P-5	40 FT	540+90

SETTLEMENT PLATE LOCATIONS	
S-1	532+05
S-2	535+05
S-3	537+15
S-4	540+20
S-5	540+95

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CALCULATED WMA CHECKED AMJ

WICK DRAIN AND INSTRUMENTATION PLAN
LUCASVILLE-MINFORD INTERCHANGE

SCI-823-6.81

1/2

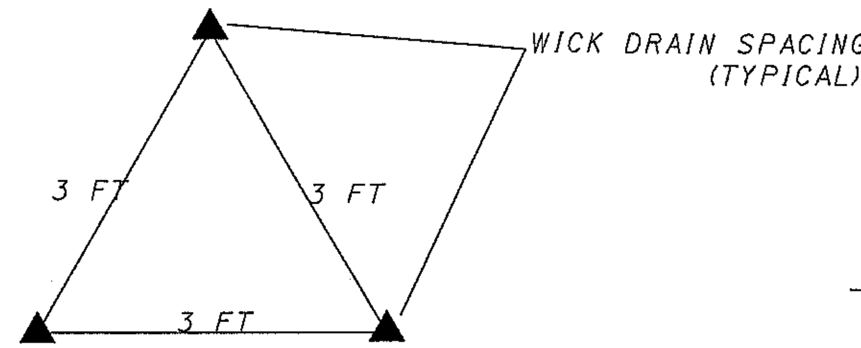
6/26/2007 12:26:53 PM M:\proj\0121\3070.03\Final Design Plans - PHASE I\Wick Drains\Lucasville-Minford\lmwickplanREV.dgn

NOTES

1. PLACE 3 FEET OF ODOT ITEM 703.02 EMBANKMENT BEFORE THE INSTALLATION OF THE WICK DRAIN WICK DRAINS TO BE INSTALLED PRIOR TO EMBANKMENT CONSTRUCTION.
2. THE SAND SHALL CONSIST OF CLEAN, FREE-DRAINING, COARSE NATURAL SAND, OR SAND AND PEA GRAVEL, SHALL BE GRADED UNIFORMLY FROM COARSE TO FINE, AND SHALL BE OF SUCH SIZE THAT, WHEN TESTING ON U.S. STANDARD SIEVES IN ACCORDANCE WITH AASHTO T27 AND WASHING THE SAMPLE IN ACCORDANCE WITH AASHTO T11, SHALL CONFORM TO THE GRADING REQUIREMENTS OF ODOT CMS 703.02.
3. THE SAND SHALL NOT CONTAIN ANY ORGANIC OR OTHER DELETERIOUS MATERIALS AND SHALL NOT BE FROZEN WHEN PLACED.
4. IF DENSE SAND, GRAVEL OR HARD SOIL LAYERS ARE ENCOUNTERED BELOW THE GROUND SURFACE AND CANNOT BE PENETRATED WITH REASONABLE EFFORT, THE CONTRACTOR SHALL BE REQUIRED TO PRE-DRILL THE WICK DRAIN LOCATIONS.
5. WICK DRAINS SHALL BE INSTALLED FROM THE WORKING SURFACE TO THE DEPTH SHOWN IN THE PLANS, OR TO COMPLETELY PENETRATE THE COMPRESSIBLE FOUNDATION SOILS AT SUCH A DEPTH EITHER SHALLOWER OR DEEPER THAN PLAN DEPTH WHERE THE SOIL RESISTS A REASONABLE EFFORT AT FURTHER PENETRATION.
6. SETTLEMENT PLATES SHALL BE GEOKON MODEL 4600 OR EQUIVALENT.
7. VIBRATING WIRE PIEZOMETERS SHALL BE SLOPE INDICATOR MODEL 52611099 OR EQUIVALENT.
8. THE NUMBER OF WICKS IS ESTIMATED TO BE 93,000 AND THE AVERAGE WICK DRAIN DEPTH IS ESTIMATED TO BE 43 FEET FOR THE ENTIRE INTERCHANGE. THE TOTAL WICK DRAIN FOOTAGE AT THIS INTERSECTION IS ESTIMATED TO BE 3,999,999 LINEAR FEET.
9. THE MAINLINE EMBANKMENT AT THE LUCASVILLE MINFORD INTERCHANGE BETWEEN STATIONS 530+00 AND 542+50 SHALL BE CONSTRUCTED IN STAGES AND THE FOUNDATION PORE PRESSURES SHALL BE MONITORED. THE MAXIMUM HEIGHT OF THE INITIAL STAGE SHALL BE 32 FEET. IF AT ANY TIME, FOUNDATION PORE WATER PRESSURE HEAD IS AT OR HIGHER THAN THE EXISTING GROUND SURFACE IN THE INITIAL STAGE, THEN EMBANKMENT CONSTRUCTION SHALL STOP. IT IS ESTIMATED THAT THE INITIAL STAGE OF EMBANKMENT CONSTRUCTION WILL NEED TO CONSOLIDATE THE FOUNDATION SOILS FOR SIXTY DAYS BEFORE THE SUBSEQUENT STAGE OF THE EMBANKMENT IS PLACED.
10. THE ACTUAL WICK DRAIN TREATMENT AREA AND DEPTH MIGHT DIFFER FROM THE PROPOSED LIMITS DUE TO SOIL VARIATIONS AT THE SITE AND THEREFORE SHOULD BE CONFIRMED IN THE FIELD BY THE CONTRACTOR

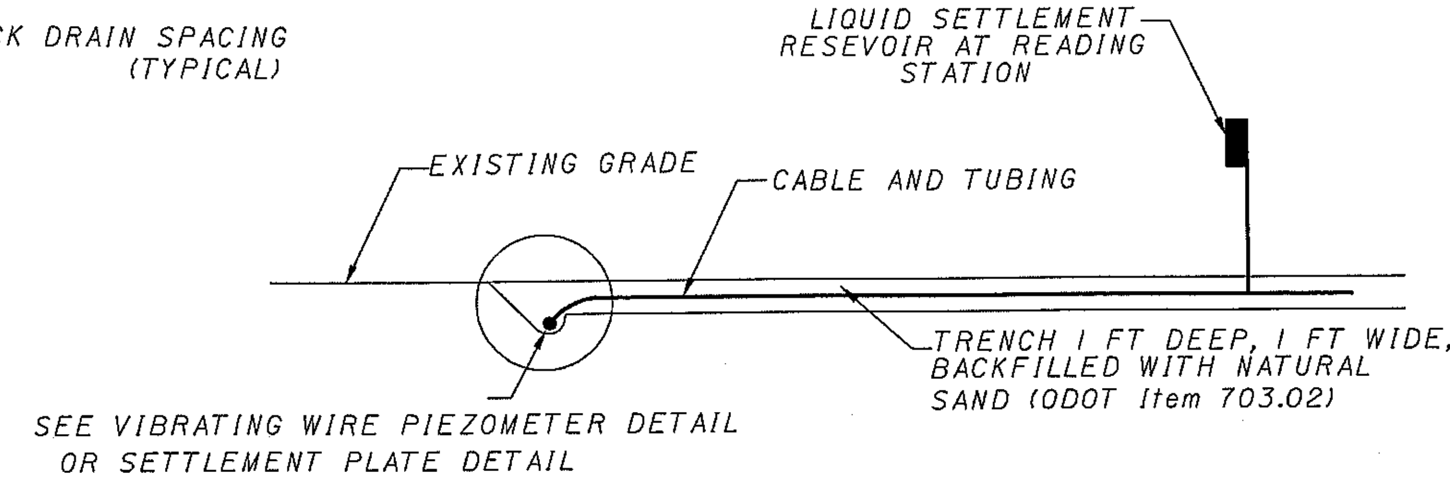
DETAIL "A"

WICK DRAIN TYPICAL LAYOUT-PLAN VIEW
(NOT TO SCALE)



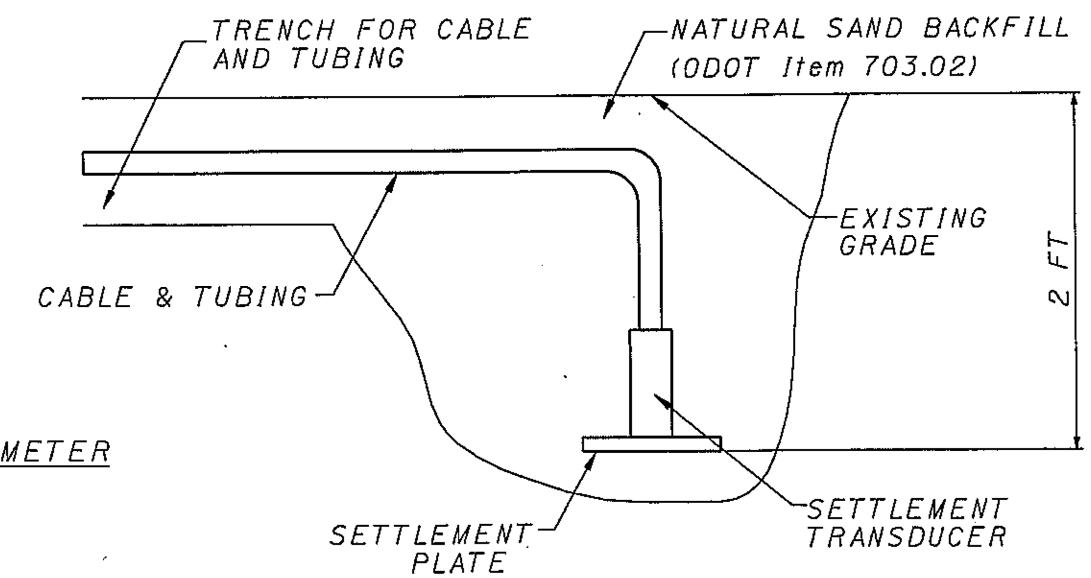
DETAIL "B"

INSTRUMENTATION DETAILS
(NOT TO SCALE)



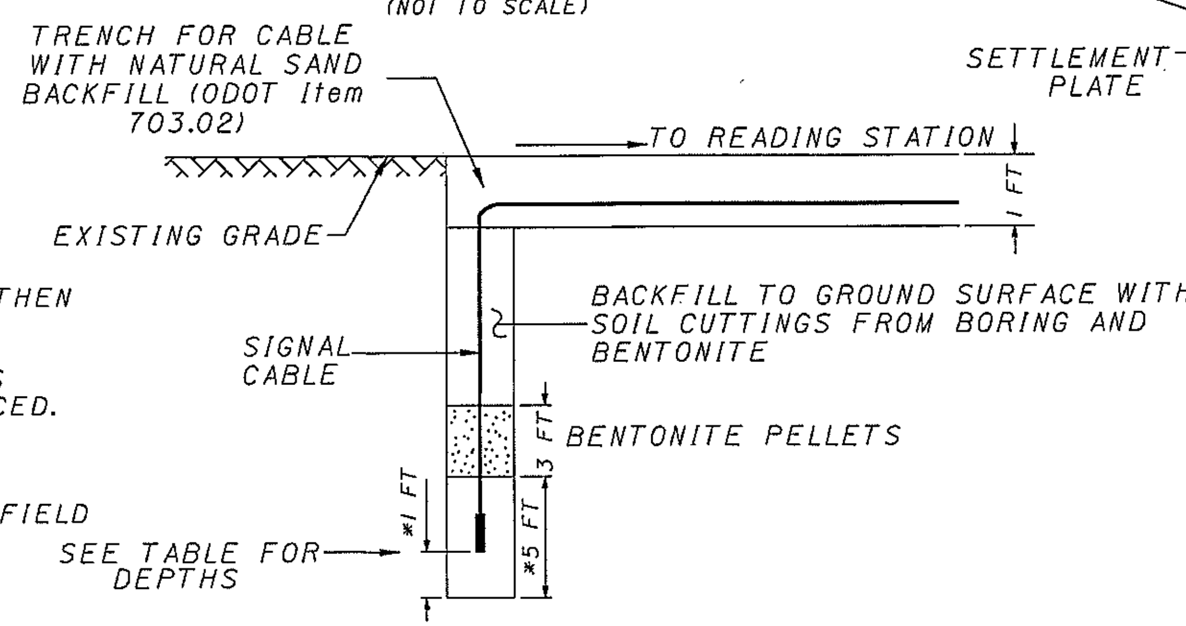
SETTLEMENT PLATE DETAIL

(NOT TO SCALE)



VIBRATING WIRE PIEZOMETER

(NOT TO SCALE)



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CALCULATED WMA CHECKED AMJ

INSTRUMENTATION DETAILS
LUCASVILLE-MINFORD INTERCHANGE

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