Bridge Preliminary Design Report

Ramp B over Fairground Rd.
SCI-823-1593

SCI-823-10.13
PID No. 79977

Prepared for

Ohio Department of Transportation

November 2007

CH2MHILL

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1. Introduction

Following review and resolution of comments on the Structure Type Study resubmitted in March 2007, a single span prestressed concrete I-beam bridge with reinforced concrete deck and semi-integral abutments behind MSE walls was the structure type selected by the Department on May 30, 2007 for construction of the proposed Ramp B over Fairground Road bridge.

The proposed bridge has a span length of 96'-10" with a 10°-54'-50" LF skew. The reinforced concrete deck is 33'-0" wide. Both abutments are located behind MSE walls and are supported on piles driven to refusal on rock.

The following is a summary of major developments that have occurred on the project and evolutionary changes made to the structure design since the Structure Type Study was approved.

- *Vertical Geometry:* The vertical alignment of Ramp B has been adjusted from that shown in the type study to reduce the vertical clearance over Fairground Road and the Norfolk Southern tracks. However, excess vertical clearance still exists. The Ramp B profile grade over Fairground Road is dependent upon the SR-823 profile, gore design, ramp cross slope, and the required profile elevation needed at the railroad crossing. The end of the Fairground Road structure for Ramp B is approximately 200' from the merge point with SR-823. This short distance does not allow significant changes to the ramp profile in this area because of the need to tie into SR-823. The SR-823 profile is controlled by the following factors:

    - A culvert must be provided at SR-823 STA. 869+00 for an existing drainage swale. The profile was adjusted in this area before the interchange so that a culvert could be installed.
    - The 3% grade in the interchange area exceeds recommended critical length of grade. ODOT L&D Vol. 1, Figure 203-1a shows that over a length of 3000' on a 3% upgrade, truck speeds are reduced by approximately 15 mph. A 10 mph reduction in speed is the recommended guideline for lengths of critical grades. Providing a steeper profile grade would further reduce truck speeds.
    - A steeper grade would increase the amount of rock cut along SR-823 and project costs. Because the project has a significant amount of excess fill material (millions of cubic yards), the project team has attempted to reduce rock cut whenever possible.

  Therefore, all of these factors culminated in the Ramp B vertical clearance over Fairground Road exceeding the minimum required vertical clearance by 5'-7".

- *Horizontal Geometry:* The horizontal geometry of Ramp B has not changed since the type study.

- *Bridge Substructure:* The location of the abutments has changed. During the Structure Type Study, the abutments were located in accordance with ODOT BDM Figure 330, as there is a minimum of 3'-0" clearance between the back face of the MSE wall and the front face of the abutment footing. This location provided sufficient clearance for either a pile supported abutment or for an abutment with a spread footing. During preliminary design it was determined that the abutments will be supported by piles, thereby allowing the abutments to be moved closer to the MSE wall. The abutments are now located in accordance with ODOT BDM Figure 331, as there is a minimum of 3'-6" between the back face of the MSE wall and the centerline of the front row of piles. The distance between the back face of the MSE wall and the front face of the abutment footing is now a minimum of 2'-0".

The bottom of footing elevations for both abutments have been modified to reflect the lowered profile. The rear abutment bottom of footing elevation is now 581.50, while the forward abutment bottom of footing elevation is now 584.40.

Both abutments will be supported on steel H-piles. In the Structure Type Study submission, the possibility of using spread footings was mentioned as a way to avoid driving piles through the recommended deep soil-mixed subsurface below the MSE walls. However, since it has since been determined that deep soil mixing ground remediation is not necessary at this location, both abutments will be supported by piles.

- *Bridge Superstructure:* The number of prestressed concrete beams has changed since the Structure Type Study submission.

  During the Structure Type Study, it was proposed that the superstructure would consist of 4-AASHTO Type 4 beams spaced at 9'-0". During preliminary design, it was found that such a configuration would result in a heavily reinforced beam design. Although this structure is on a horizontal curve, the prestressed beams will be straight and parallel to one another. This results in varying overhang widths along the length of the bridge. Using a deeper beam with a wider top flange was avoided because an increased width of the top flange would have further reduced the minimum overhang dimensions. As a result, the beam spacing has been reduced and a beam line has been added in order to achieve a beam design that is in close accordance with the requirements of the ODOT BDM. During preliminary design, CH2M HILL determined that using 5-AASHTO Type 4 (54") beams spaced at 6'-10 1/2" result in a beam design requiring a concrete release strength of 5500 psi and a 28-day concrete strength equal to 7000 psi. This design is a slight deviation from the recommended concrete release strength of 5000 psi specified in the ODOT BDM. CH2M HILL contacted the Ohio Prestressers Association to confirm that such a design could be fabricated at no additional cost. They have confirmed that a release strength of 5500 psi can be obtained at no additional cost, and this correspondence can be found in Appendix E of this report. In addition, vertical clearance calculations reflecting both this adjusted beam spacing and the new Ramp B profile can be found in Appendix D of this report.

- *Aesthetics:* Aesthetic treatments for this structure and site could include concrete staining or coatings, form liners for the substructure, railing on MSE walls, landscaping, etc. At this time, it is ODOT's intent not to provide aesthetic treatments for this structure or site.

2. Design Criteria

The following design criteria apply to this structure, Ramp B over Fairground Road:

Functional Classification:	Directional Ramp	
Traffic Data:	ADT (2010)	2,700
	ADT (2030)	3,600
	ADTT (2030)	500
	Design Speed	35 mph
	Legal Speed	30 mph
Vertical Clearance:	Fairground Road = 15′-0″, minimum	
Horizontal Clearance:	Fairground Road = 30′-0″, minimum	

3. Maintenance of Traffic

The proposed Ramp B alignment will carry traffic exiting northbound US-23 onto southbound SR-823. Because the Ramp B alignment is new construction, maintenance of traffic during construction of the Ramp B bridge over Fairground Road will be limited. With the exception of limited Fairground Road closure for superstructure beam setting, as well as traffic safety precautions throughout bridge construction, no additional maintenance of traffic solutions will need to be investigated.

4. Foundation Recommendations

Subsurface investigations for the SCI-823-10.13 project have been conducted in two phases. The boring program is complete, and included all of the proposed pavement and embankment borings, borings for MSE walls, and bridge borings.

Four borings at the Ramp B bridge over Fairground Road were taken during the first phase and two borings during the second phase. Based on these borings, foundation recommendations have been made by DLZ. Geotechnical engineers at CH2M HILL performed a brief review of the MSE wall/bridge foundation recommendations contained in the final subsurface exploration report prepared by DLZ, and provided written comments in a technical memorandum. A copy of DLZ's foundation report and CH2M HILL's review comments are included with this submission in Appendix C.

The semi-integral rear and forward abutments, behind an MSE wall, will be supported by HP 14x73 H-piles driven to refusal on bedrock. Because the piles will be driven to bedrock, it is recommended that reinforced pile points be used to prevent the piles from being damaged. Although reinforced pile points are not required in shale, the predominate bedrock type in the area of the proposed structure contains interbedded sandstone, which if driven into, could damage piles. Pile sleeves should be placed from the bottom of the leveling pad to the pile cap elevation to permit pile installation through the soil reinforced zone of the MSE wall. The final pile arrangement should consider avoiding potential conflicts with typical MSE reinforcing strap patterns.

Detailed foundation recommendations for the MSE walls are in a separate report and are included in the preliminary design report for the MSE walls.

A summary of the foundation recommendations is provided in the following table.

Substructure Unit	Type	Bottom of Footing Elev.	Estimated Pile Tip Elev.	Pile Type	Max. Design Load (tons)	Distance: Top of Pile[1] to Estimated Pile Tip	Estimated Pile Length	Pile Order Length
Rear Abut.	Semi - Integral	581.50	543.00	HP 14x73	95	39.50′	40′	45′
Fwd. Abut.	Semi - Integral	584.40	550.60	HP 14x73	95	34.80′	35′	40′

[1] Assumes top of pile is one foot above bottom of footing

5. MSE Wall Recommendations

Foundation recommendations for the MSE abutment walls will be included with the Retaining Wall Preliminary Design Report submission.

6. Cost Estimate

An updated bridge cost estimate reflecting the proposed preliminary design for the Ramp B bridge over Fairground Road is included in Appendix A of this report. The estimate and all unit prices used are based upon 2006 costs. The estimated construction cost for the MSE walls will be included with the MSE retaining wall cost estimate, to be included with the separate Retaining Wall Preliminary Design Report submission.

7. Bridge and Structure File Numbers

Bridge and structure file number assignments have been requested from the Office of Structural Engineering. They are as follows:

Bridge Number: **SCI-823-1593**

Structure File Number: **7306717**

APPENDIX A

SCI-823-10.13

Ramp B Over Fairground Road

Preliminary Bridge Design Cost Estimate

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By: DGS Date: 8/3/2007
Checked: SKT Date: 9/24/2007

SUMMARY

Span Arrangement No. Spans	Span Arrangement Lengths	Total Span Length (ft.)	Framing Alternative	Proposed Stringer Section	Subtotal Superstructure Cost	Subtotal Substructure Cost	Structure Incidental Cost (16%) (Note 4)	Structure Contingency Cost (20%)	Total Initial Construction Cost (Note 1)	Superstructure Life Cycle Maintenance Cost	Total Relative Ownership Cost
1	96.83	96.83	5 ~ P.S. Concrete I-Beams	AASHTO Type 4	$261,000	$129,000	$62,000	$90,000	$542,000	$200,000	$742,000

NOTES:

1. The total initial construction costs do not include MSE Wall/ground improvement costs. If required, see Retaining Wall Preliminary Design report for those costs.

2. Use 2006 pavement cost = $46.00 /sq. yd.

   Pavement Widths:

Average Rear Approach	Average Fwd. Approach	Combined Average
33.00 ft.	33.00 ft.	33.00 ft.

3. Use 2006 Concrete Barrier, Single Slope Median, Type B1 cost = $64.00 /ft.
   Use 2006 Concrete Barrier, Single Slope, Type D cost = $81.00 /ft.

4. Structure incidental cost allowance includes provision for structure excavation, porous backfill & drainage pipe, sealing of concrete surfaces, bearings, pile driving equipment mobilization, settlement platforms, joint sealers, and joint fillers costs.

5. The estimate and all unit prices used are based upon 2006 costs.

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Checked: SKT — Date: 9/24/2007

SUPERSTRUCTURE

Span Arrangement No. Spans	Span Arrangement Lengths	Total Span Length (ft.)	Deck Length (ft.)	Deck Area (sq. ft.)	Deck Volume (cu. yd.)	Deck Concrete Cost	Deck Reinforcing Cost	Approach Slab Cost	Framing Alternative	Proposed Stringer Section	Structural Steel Weight (pounds)	Structural Steel Cost	Prestressed Beam Cost	Initial Superstructure Cost
1	96.83	96.83	98.83	3,300	125	$61,500	$28,900	$45,300	5 ~ P.S. Concrete I-Beams	AASHTO Type 4	0.0	$0	$124,900	**$261,000**

Deck Cross-Sectional Area:

Parapets:

	No.	Individual Area (sq. ft.)	Parapet Area (sq. ft.)
Parapets	2	4.26	8.52
Median	0	9.29	0.00

Slab:

T (ft.)	Ave. W (ft.)	Slab Area	Haunch & Overhang Area	Total Concrete Area (sq. ft.)
0.71	33.00	23.4	2.3	34.2

Note: Deck width measured as average width.
10% of deck area allowed for haunches and overhangs.

QC/QA Concrete, Class QSC2
Unit Cost ($/cu. yd):

	Year 2005	Annual Escalation	Year 2006
Deck	$512.91	3.0%	$528.00
Parapets	$370.36	3.0%	$381.00
Weighted Average =			$491.00

Based on parapet and slab percentages of total concrete area

Epoxy Coated Reinforcing Steel
Unit Cost ($/lb):

Assume 285 lbs of reinforcing steel per cubic yard of deck concrete for concrete beam bridges

	Year 2005	Annual Escalation	Year 2006
Deck Reinforcing	$0.79	3.0%	$0.81

Prestressed Concrete Beams
Unit Costs:

	Year 2005		Annual Escalation	Year 2006		No. Required
AASHTO Type 4 Beams						
Type 4 I-Beams (54")	$220	lf	6.0%	$233	lf	485
Intermediate Diaphragms	$930	ea.	6.0%	$986	ea.	12

Structural Steel
Unit Costs ($/lb.):

	Cost Ratio	Year 2005	Annual Escalation	Year 2006
Rolled Beams - Grade 50 (level 3)	n/a	$1.10	12.0%	$1.23
Plate Girders - Grade 50 (level 5)	n/a	$1.25	12.0%	$1.40
Hybrid Plate Girders - Grade 50/70W	1.10	$1.38	12.0%	$1.54

Note - all structural steel weight will be estimated at 65 pounds per each square foot of bridge deck area for long span curved girders.
45 pounds per each square foot of bridge deck area for short span curved girders.

Reinforced Concrete Approach Slabs (T=17")
Unit Cost ($/sq. yd.):

Length = 30 ft. Width = 33.00 ft
Area = 110 sq. yd.

	Year 2005	Annual Escalation	Year 2006
Approach Slabs	$199.78	3.0%	$206.00

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SUBSTRUCTURE

Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Proposed Stringer Section	Pier Concrete Cost	Pier Reinforcing Cost	Abutment Concrete Cost	Abutment Reinforcing Cost	Pile Foundation Cost	Initial Substructure Cost
1	96.83	5 ~ P.S. Concrete I-Beams	AASHTO Type 4	$0	$0	$72,400	$13,300	$43,200	**$129,000**

Pile Foundation Unit Cost ($/ft.): HP Steel Piles, Furnished & Driven

Abutment Piles:

Number Rear	Number Forward	Top of Pile Elevation Rear	Top of Pile Elevation Forward	Pile Tip Elevation Rear	Pile Tip Elevation Fwd.	Estimated Length Per Rear Pile	Estimated Length Per Forward Pile	Total Pile Order Length	Total Cost	
14	14	582.5	585.4	543.0	550.6	40	35	1,190	$43,200	HP14 x 73

HP10 x 42 Steel Piles, Furnished & Driven	Year 2005 Unit Cost	Annual Escalation	Year 2006
Furnished	$17.50	6.0%	$18.60
Driven	$10.69	3.0%	$11.00
Total			$29.60

HP12 x 53 Steel Piles, Furnished & Driven	Year 2005 Unit Cost	Annual Escalation	Year 2006
Furnished	$19.02	6.0%	$20.20
Driven	$9.38	3.0%	$9.70
Total			$29.90

HP14 x 73 Steel Piles, Furnished & Driven	Year 2005 Unit Cost	Annual Escalation	Year 2006
Furnished	$27.30	6.0%	$28.90
Driven	$7.19	3.0%	$7.40
Total			$36.30

Abutment QC/QA Concrete, Class QSC1 Cost:

Component	Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Abutment					
Rear	56.8	$384.26	3.0%	$396.00	$22,500
Fwd	54.6	$384.26	3.0%	$396.00	$21,600
Wingwalls					
Rear	34.7	$384.26	3.0%	$396.00	$13,700
Fwd	36.8	$384.26	3.0%	$396.00	$14,600

Reinforcing Steel Unit Cost ($/lb):

Assume 125 lbs of reinforcing steel per cubic yard of pier concrete.
Assume 90 lbs of reinforcing steel per cubic yard of abutment concrete.

	Year 2005	Annual Escalation	Year 2006
Pier	$0.79	3.0%	$0.81
Abutment	$0.79	3.0%	$0.81

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LIFE CYCLE MAINTENANCE COST

Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Structural Steel Painting (4) Cost Per Cycle	Structural Steel Painting (4) Number of Maintenance Cycles	Structural Steel Painting (4) Total Life Cycle Cost	Superstructure Sealing (4) Cost Per Cycle	Superstructure Sealing (4) Number of Maintenance Cycles	Superstructure Sealing (4) Total Life Cycle Cost
1	96.83	5 ~ P.S. Concrete I-Beams	$0	0	$0	$7,700	4	$30,800

Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Bridge Deck Overlay (4) Deck Demo & Chipping	Deck Overlay	Deck Joint Gland (2)	Number of Maintenance Cycles	Total Life Cycle Cost	Bridge Redecking (4) Deck Concrete Cost (3)	Deck Reinforcing Cost (3)	Deck Joint Cost (2)	Deck Removal Cost	Number of Maintenance Cycles	Total Life Cycle Cost	Superstructure Life Cycle Maintenance Cost (1)	Total Initial Construction Cost	Total Relative Ownership Cost
1	96.83	5 ~ P.S. Concrete I-Beams	$10,600	$12,300	$0	2	$45,800	$61,500	$28,900	$0	$33,000	1	$123,400	**$200,000**	**$542,000**	**$742,000**

Structural Steel Painting:
Structural Steel Area:

Web Depth (in.)	No. Stringers	Total Span Length (ft.)	Assumed Ave. Bot. Flange Width (in.)	Nominal Exposed Girder Area (sq. ft.)	Secondary Member Allowance	Total Exposed Steel Area (sq. ft.)
		0.00		0	20%	0

Painting Cost per sq. ft.:

	Year 2005	Annual Escalation	Year 2006
Prep.	$6.88	3.0%	$7.09
Prime	$1.62	3.0%	$1.67
Intermed.	$1.89	3.0%	$1.95
Finish	$1.86	3.0%	$1.92
Total			$12.63

Superstructure Sealing:
PS Concrete I-Beam Area:
54" AASHTO Type 4

	H	V	Diag.	No.	Total
Bot. Flange	26			1	26.00
		8		2	16.00
Lower Fillets	9	9	12.73	2	25.46
Web		23		2	46.00
Upper Fillets	6	6	8.49	2	16.97
Top Flange		8		2	16.00
Total Exposed Perimeter					146.43 in.

PS Concrete Area:

No. Stringers	Total Span Length (ft.)	Nominal Exposed Beam Area (sq. ft.)	Secondary Member Allowance	Total Exposed Concrete Area (sq. yd.)
5	96.83	5,908	10%	720

Sealing Cost per sq. yd.:

	Year 2005	Annual Escalation	Year 2006
Epoxy-Urethane Sealer	$10.44	3.0%	$10.75

Bridge Redecking:
Bridge Deck Joint Cost per foot:

	Year 2005	Annual Escalation	Year 2006
Structural Expansion Joint Including Elastomeric Strip Seal	$305.46	3.0%	$314.62

Bridge Width (ft.)	No. Joints
33.00	0

Bridge Deck Removal Cost:

Deck Area (3) (sq. ft.)	Year 2006	Deck Removal Cost
3,300	$10.00	$33,000

Bridge Deck Overlay (Item 848):
Bridge Deck MSC Overlay Cost per sq. yd.:

	Year 2005	Annual Escalation	Year 2006
Micro Silica Modified Concrete Overlay Using Hydrodemolition (1.25" thick)	$29.57	3.0%	$30.46
Surface Preparation Using Hydrodemolition	$25.93	3.0%	$26.71
Hand Chipping (10% of deck area)	$85.66	3.0%	$88.23

Bridge Deck MSC Overlay Cost per cu. yd.:

Micro Silica Modified Concrete Overlay (Variable Thickness), Material Only	$145.00	3.0%	$149.35

Deck Area (3) (sq. ft.)	Deck Area (sq. yd.)	Hand Chipping (sq. yd.)	Variable Thickness Repair (cu. yd.)
3,300	367	9	8

Assume 25% of deck area requires removal to depth of 4.5" (3.00" additional removal).

Bridge Deck Joint Gland Replacement Cost per foot:

	Year 2005	Annual Escalation	Year 2006
Elastomeric Strip Seal Gland	$76.37	3.0%	$78.66

Assume gland replacement cost equals 25% of original deck joint construction cost.

NOTES:

1. Life cycle maintenance costs assume a 75 -year structure life, and are expressed in present value (2006) dollars.
2. Bridges with straight girders are assumed to have semi-integral abutments, therefore strip seal deck joints are only included for curved girder bridges.
3. See Superstructure Cost sheet.
4. Assume bridge deck overlay at Year 20 & Year 60 and bridge deck replacement at Year 40. Assume steel superstructures are painted at Year 25, then on a 25-year recurrence interval. Assume concrete superstructures are sealed on a 15-year interval. Assume complete bridge replacement at Year 75.
5. Life cycle maintenance cost differences are assumed to be predominately a function of superstructure maintenance costs. Consequently, substructure lifecycle maintenance costs are not included in this analysis.

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By: DGS Date: 8/3/2007

Checked: SKT Date: 9/24/2007

COST SUMMARY

Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Proposed Stringer Section	Total Initial Superstructure Cost	Total Initial Substructure Cost	Total Initial Construction Cost (1)	Superstructure Life Cycle Maintenance Cost	Total Relative Ownership Cost
1	96.83	5 ~ P.S. Concrete I-Beams	AASHTO Type 4	$261,000	$129,000	$542,000	$200,000	**$742,000**

NOTE:

1. Includes contingencies and incidental costs.
2. The estimate and all unit prices used are based upon 2006 costs.

APPENDIX B

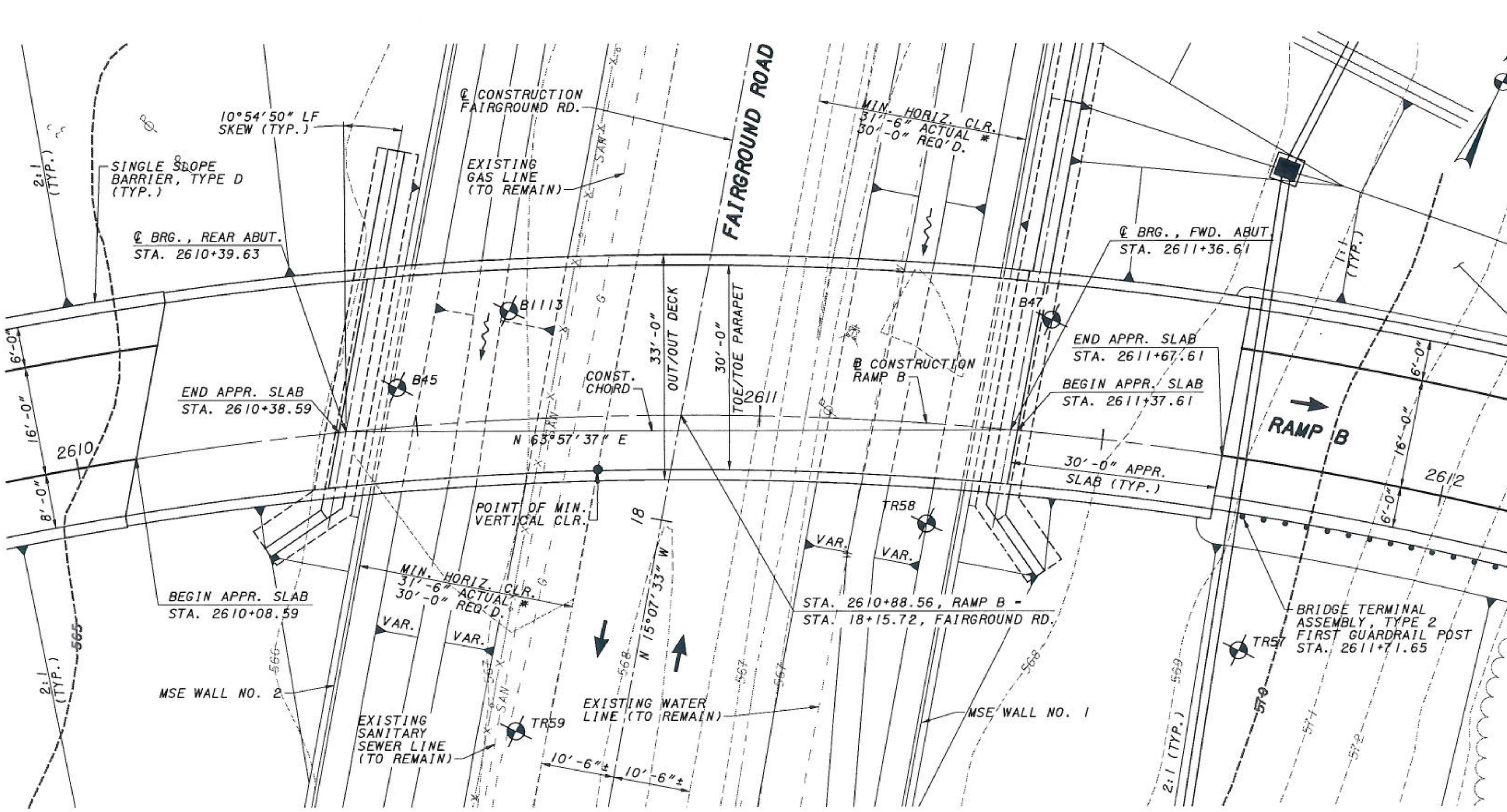
BENCHMARKS

CURVE DATA - RAMP B
P.I. Sta - 2609+99.07
Δ - 102° 45′ 15″ (RT)
Dc - 11° 15′ 00″
R - 509.30′
T - 637.46′
L - 913.37′
E - 306.63′

CONCRETE SLOPE PAVING, AS PER PLAN

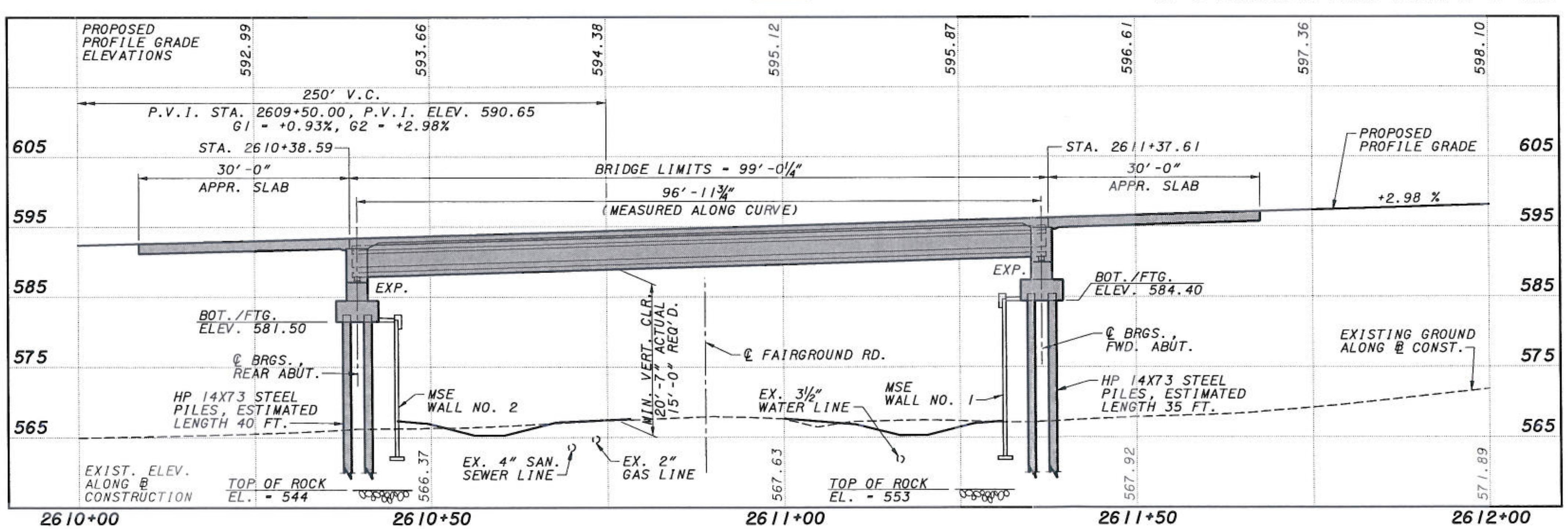
TRAFFIC DATA
CURRENT ADT (2010) - 2700
DESIGN ADT (2030) - 3600
DESIGN ADTT - 500

LEGEND
INDICATES BORING LOCATION



PLAN

* 31′-6″ PROVIDED TO PERMIT FUTURE 12′-0″ LANE



PROFILE ALONG ℄ CONSTRUCTION, RAMP B

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

POWER AND TELEPHONE LINES TO BE RELOCATED.

PROPOSED STRUCTURE

TYPE: SINGLE SPAN COMPOSITE PRESTRESSED CONCRETE I-BEAMS WITH REINFORCED CONCRETE DECK AND SEMI-INTEGRAL ABUTMENTS ON MSE WALLS

LENGTH OF SPAN: 96′-11¾″ C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30′-0″ TOE/TOE PARAPETS

SIDEWALK: NONE

DESIGN LOADING: HS25 AND THE ALTERNATE MILITARY LOADING, FWS - 60 LB/FT²

SKEW: 10°54′50″ LEFT FORWARD, MEASURED FROM THE NORMAL TO THE CONSTRUCTION CHORD

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (30′-0″ LONG)

ALIGNMENT: HORIZONTALLY CURVED (℄ RADIUS - 509.30′)

SUPERELEVATION: 0.071 FT/FT

LATITUDE: N 38°53′31″

LONGITUDE: W 82°59′51″

DESIGN AGENCY: CH2MHILL, 5775 Perimeter Drive, Suite 190, Dublin, Ohio 43017	DESIGNED DGS	DRAWN JBA	REVIEWED SCJ	DATE 10/07	SCIOTO COUNTY STA. 2610+38.59 TO STA. 2611+37.61	SITE PLAN — BRIDGE NO. SCI-823-1593 RAMP B OVER FAIRGROUND ROAD	SCI-823-10.13 PID 79977
	CHECKED SKT	REVISED	STRUCTURE FILE NUMBER 7306717				1/3 (824)

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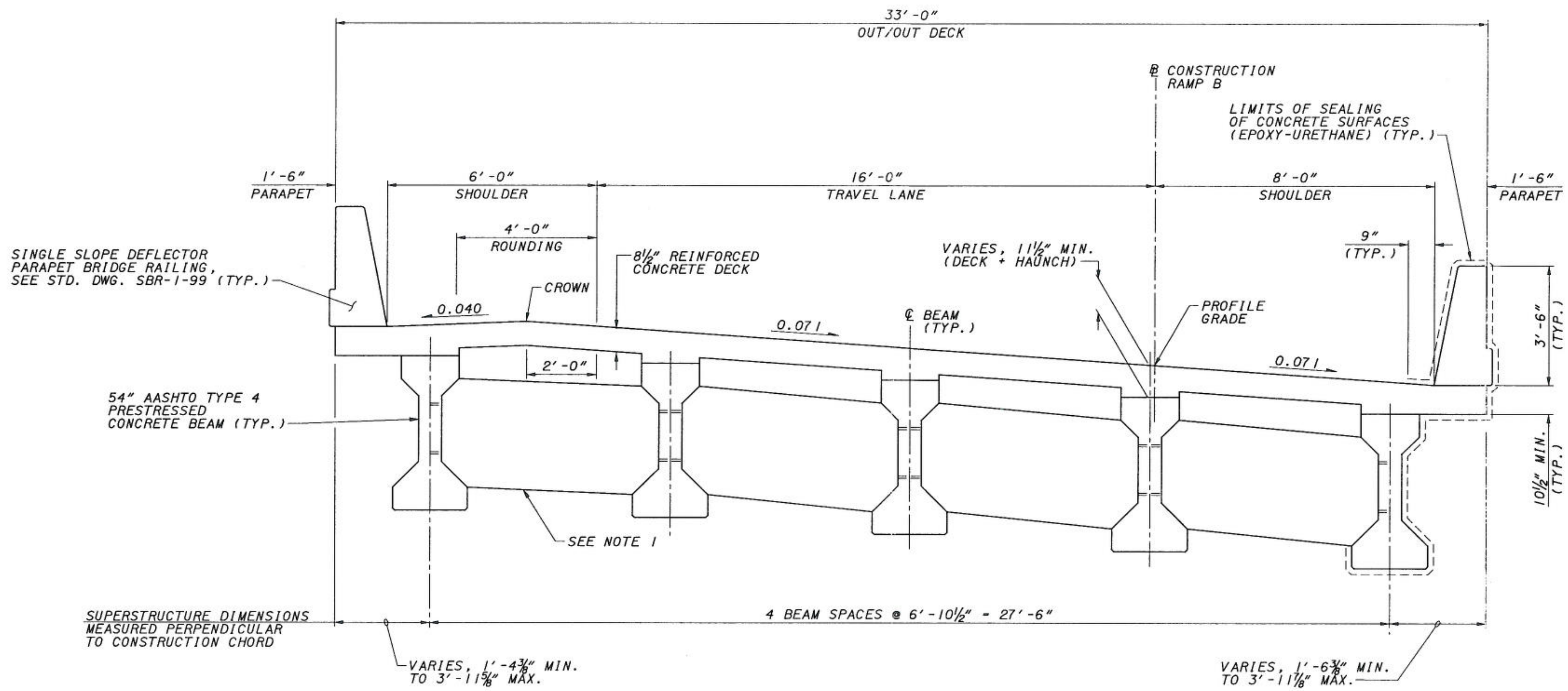


TYPICAL TRANSVERSE SECTION

NOTES:

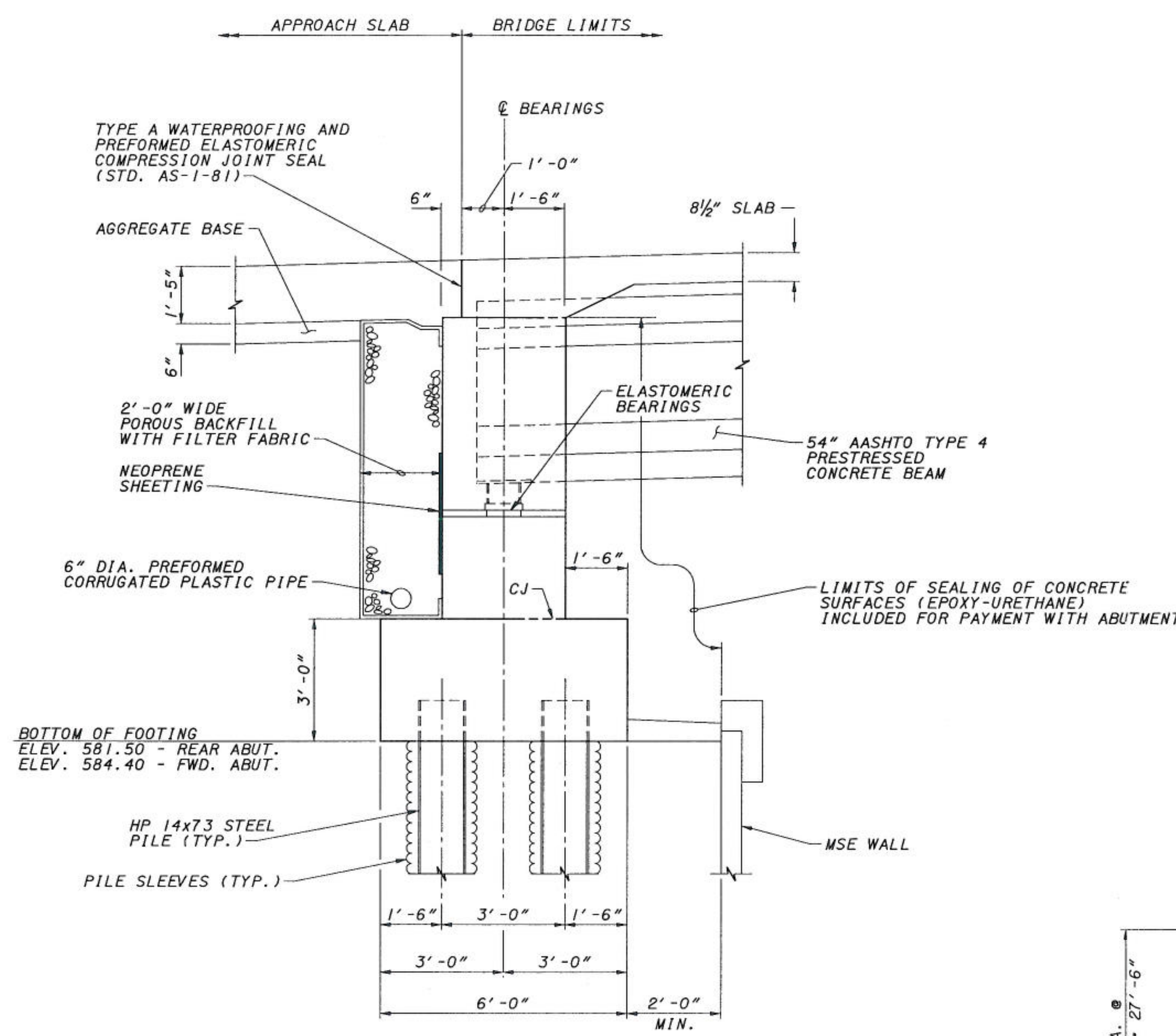
1. INTERMEDIATE DIAPHRAGMS MAY BE CAST-IN-PLACE CONCRETE OR GALVANIZED STEEL. FOR DETAILS OF BOTH DIAPHRAGM TYPES, SEE STANDARD CONSTRUCTION DRAWING PSID-1-99.

DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

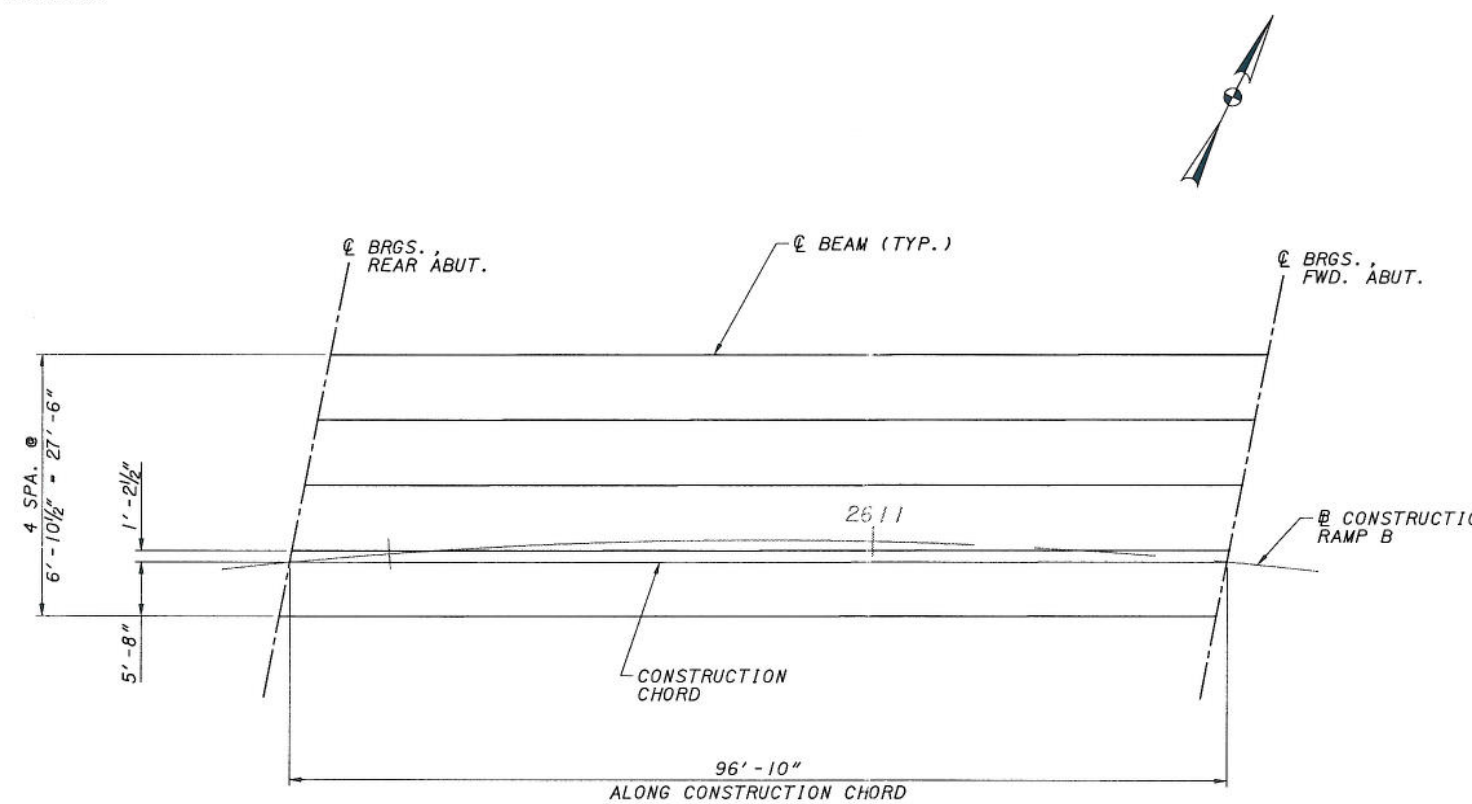
DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	10/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT		7306717	

TYPICAL TRANSVERSE SECTION
BRIDGE NO. SCI-823-1593
RAMP B OVER FAIRGROUND ROAD

SCI-823-10.13
PID 79977



REAR AND FORWARD ABUTMENT SECTION



FRAMING PLAN

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	10/07
CHECKED SKT	REVISED	STRUCTURE FILE NUMBER 7306717	

DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

ABUTMENT SECTION AND FRAMING PLAN
BRIDGE NO. SCI-823-1593
RAMP B OVER FAIRGROUND ROAD

SCI-823-10.13
PID 79977

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APPENDIX C

Review Comments to DLZ's Geotechnical Report MSE Walls 1 and 2 - US 23/SR 823 Interchange Portsmouth, Ohio

PREPARED FOR:	Rob Miller/CH2M HILL /COL
	Steve Jirschele/COL
	Shawn Thompson/COL
PREPARED BY:	Christopher Dumas/WDC
DATE:	November 2, 2007
Copy:	Emad Farouz/WDC
PROJECT NUMBER:	SCI-823-10.13

I have reviewed the subject document and provide the following comments.

1. MSE Wall 2: The DLZ design computations in Appendix IV, page 1 of 21, show a geotechnical design profile as follows:

    a. Ground surface of the boring (elevation 566-ft) to elevation 548-ft (18-ft below ground surface) is a silty clayey material with an undrained shear strength of C = 1,500 psf.
    b. From elevation 548-ft down to elevation 544-ft (22-ft below ground surface) is a sandy material.
    c. Top of Rock is encountered at elevation 544 (22-ft below the ground surface).
    d. Water Table is encountered at elevation 550-ft = 16-ft below ground surface.

    However, all the MSE Wall 2 borings show a predominately loose sand (79% sand and gravel) with an estimated friction angle of approximately 28-degree (based on N value corrected for overburden) at a depth of approximately 8-ft (elevation 558-ft). This layer extends down an additional 7-ft to elevation 551-ft. The "sand and gravel" layer encountered is a very loose to loose sandy gravel, also with an estimated friction angle of approximately 28-degress. This layer extends to the top of rock (elevation 544-ft).

    I would recommend re-evaluation of the borings for MSE Wall 2, and performing slope stability analysis using the profile depicted below in Figure 1. In particular, I recommend replacing the friction angle of the gravel sand at the rock interface with a more appropriate value of 28-degrees.

2. Bearing Capacity and Staged Construction for MSE Wall 2: The three phase staged construction concept proposed to accommodate the very low bearing capacity Factor of Safety has several risks:

a. It is time consuming, complex, and has considerable uncertainty for the contractor. The constructor will need to install instrumentation and avoid damaging the instrumentation while placing the stages. If he damages them during placement, he will have to reinstall them during which time there will be a gap in critical data. In addition, the contractor will not have a defined wait time.
b. It will require piezometers, settlement platforms, and slope inclinometers to be installed, maintained, read daily (or more) and interpreted. This will require a highly qualified Geotechnical Instrumentation engineer to be on site at all times and be in daily communication with the design engineer.
c. If the wall moves, the contractor will have to unload the wall. Not only will this create a delay and potential claim, but it will also be difficult to rapidly unload the wall. It is possible the wall could move completely out of tolerances before movement is stopped, and total reconstruction could be needed. Additionally, if the wall moves, it will be risky to try to unload the wall since the last thing we want to do is a) place additional equipment load and b) place workers in a situation that could jeopardize their safety.
d. Additionally, it was mentioned that ODOT had some challenging experience with wire faced MSE walls. It is our opinion that without the use of wire face MSE wall, the construction of the wall will be very challenging, if not infeasible.

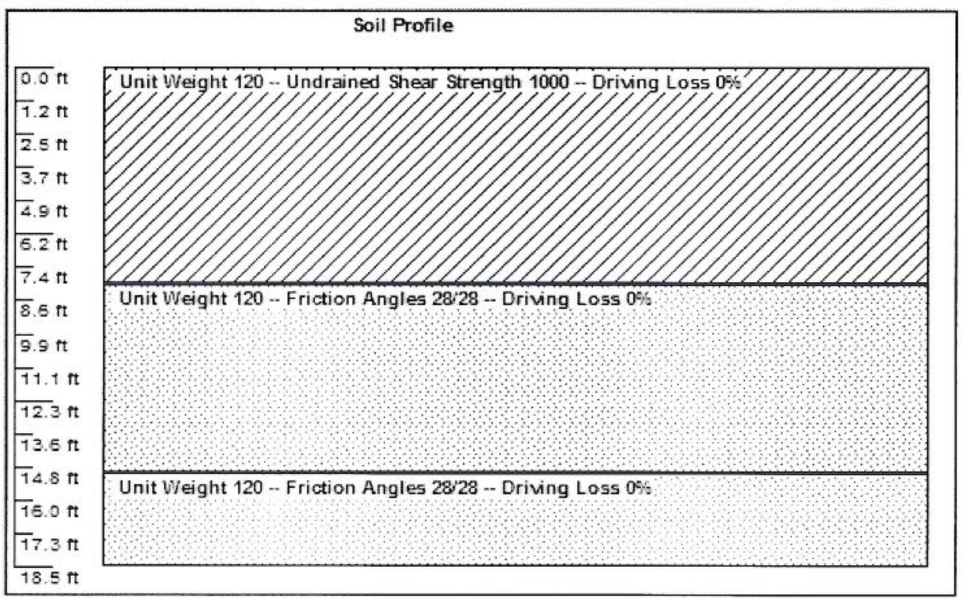


Figure 1 - Soil Profile for MSE Wall 2

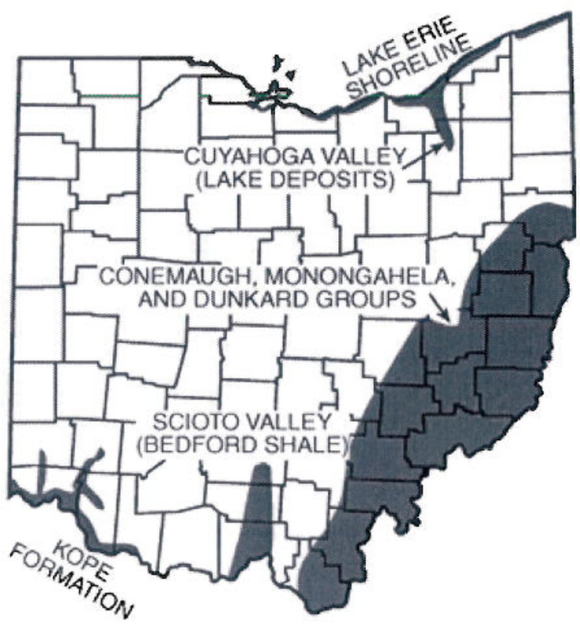
With these considerations in mind, I recommend that the MSE Wall 2 location be over-excavated an additional 2.5-ft from what has been proposed, and backfilled with MSE backfill grade material. The proposed scheme has the bottom of the leveling pad at elevation 560.5 (5.5-ft below grade). Granular soil is at approximately elevation 558-ft or an additional 2.5-ft. See comment 1. In addition, the water table is well below this excavation. Advantages include:

a. Simple and rapid.

b. The backfilled over-excavation will eliminate any bearing capacity problems, improve global stability, and may allow a reduction in the strap length. However, it may require short shoring.

3. Slope Stability: This is a major concern, specifically, a weathered shale layer a few inches thick above competent and hard shale. This is a notoriously common condition in Ohio that results in many landslides annually.

The borings, such as B-47, described severely weathered light gray shale above the competent rock. The weathered seam that causes these frequent failures is typically only a few inches thick, and as such, will not be identified by SPT borings. Typically, these materials have low effective friction angles which could be as low as 12-degrees.



[1]Figure 1 – Areas of Ohio Subject to Severe Slope Failures. *"In the lower part of the Scioto River valley, thick colluvium developed on shales of Mississippian age, particularly the Bedford Shale, is prone to failure."*

This situation applies to both MSE Walls 1 and 2, with it possibly being more acute for MSE Wall 1 where the depth to rock is only 13-ft. The consequences of this occurring on these walls during construction or after the bridge is completed and in use could include:

a. Construction delays while a new design is developed and constructed. The repair cost will likely be nearly double the cost of performing ground improvement or other alternative construction methods (see Conclusion and Recommendations).

b. Delay of improved traffic function.

[1] GeoFacts No. 8, Ohio Geological Survey, September 2003.

c. Road closure and detouring of traffic for 1-12 months, depending on the level of damage.

d. Slip surface will damage or fail the bridge abutment foundations. This could possibly lead to the girders and deck also being damaged or a span falling off the abutment bearings. Repair will require underpinning the bridge, removing the abutment foundation, abutment, MSE wall, and approach embankment, followed by installation of ground improvement, or other alternative methods, and complete reconstruction of the abutment foundations, wall, and approach embankment. If the superstructure is damaged, then the girders and deck may also need to be replaced.

e. The slip and movement could be relatively rapid and cause injury to a motorists or construction workers.

Conclusion and Recommendations

1. The use of over-excavation to improve bearing capacity is preferable to multi-phased staged construction. See comment 2.

2. The consequence of a slip failure of these walls makes avoidance of this risk an overriding priority. It is recommended that alternative construction methods be evaluated. They would include:

   i. Ground Improvement such as Controlled Modulus Columns and Vibro-Concrete Columns.

   ii. Pile supported embankment. The shallow depth to rock makes this option economical. An example could be HP 12x53 driven to rock on ten foot centers with a small cap placed on top. Approximately three layers of geogrid on 1-2 foot lifts are placed on top. Details of this can be obtained from the FHWA, Virginia Dot, Geogrid Manufacturers, and the British Standards Institute. Several have been constructed in highway applications over the last several years. Details can be provided upon request.

   iii. MSE wall supported on two geogrid layers with stone in between and bearing on timber piles driven to rock. Piles are driven on approximately 5 to 10-ft on centers and approximately 2-ft thick stone sandwiched between two layers of geogrid. The wall is then constructed on this stable platform. This has been done successfully on the VA-288 project.

   iv. MSE wall built on top of a pile supported raft foundation. Piles are driven on approximately 15-ft centers and an approximately 1-ft thick reinforced slab is poured on top. The wall is then constructed on this stable platform. This has been done successfully in Virginia on the $750-million Springfield Interchange. Key advantages include:

      a. Much more economical than extending the bridge. No superstructure girders are required.

      b. More economical than CIP walls. The lateral load is taken up by the MSE wall. There is no need to cast a large and expensive CIP vertical face with architectural form liners.

c. Eliminates the need for costly and time consuming geotechnical investigation, lab testing, interpretation, and design.

d. Eliminates the need for Geotechnical Instrumentation.

e. Eliminates the need for full time Geotechnical expertise being present at the site full time.

f. Simple to construct. No new specialized knowledge required in design or construction.

g. Eliminates risk and uncertainty in the short term and long term.

3. It would be advantageous at this stage of project development to complete a geologic report for the site which includes historical landslide information for the project geologic area.

4. Cone Penetrometer Testing (CPT) and soil sampling of the soils at the rock interface should be performed before additional time and effort is expended on the current approaches to MSE Walls 1 & 2. Without certainty regarding the presence of the very soft weathered shale soil interface, significant time and resources could be expended on a scheme that will later be shown to be non applicable. It could be more productive to pursue the alternatives listed above until such data becomes available.

5. Muti-phased staged construction. If this is selected as the preferred alternative, it is essential that:

   a. The preliminary and final design phases establish a detailed Geotechnical Instrumentation plan:

      - Instrumentation types, locations, and frequency of readings. At minimum, the site will likely require:
        - Several piezometers and settlement platforms for each wall and high fill areas. Redundancy will need to be built into the plan to accommodate instrumentation malfunction/failure/damage.
        - One to two slope inclinometers (SI) for each wall face. The walls are very tall and long. A single SI will not provide adequate coverage of the long and critical abutment MSE Walls 1 & 2.
        - Settlement Platforms.
        - Recommend instrumentation references:
          - FHWA-NHI-00-043, Mechanically Stabilized Earth Walls and Reinforced Soil Slopes
          - FHWA-NHI-132034, Ground Improvement Manual
          - FHWA-HI-98-034, Geotechnical Instrumentation
          - AASHTO Subsurface Investigation Manual

      - Construction Specifications. These should address issues such as: installation, equipment and methods, qualifications for personnel installing and monitoring the instrumentation, and contractor damaging and replacing instrumentations including liquidated damages.

b. A highly qualified Geotechnical Instrumentation engineer to oversee instrumentation installation, monitor instruments in the field, reduce data, produce data reports, and communicate (verbal or electronic) with the design and construction engineer on a nearly daily basis.

SUPPLEMENTAL: Review Comments to DLZ's Geotechnical Report MSE Walls 1 & 2 – US 23/SR 823 Interchange Portsmouth, Ohio

PREPARED FOR:	Rob Miller/CH2M HILL /COL
	Steve Jirschele/COL
	Shawn Thompson/COL
PREPARED BY:	Christopher Dumas/WDC
DATE:	November 5, 2007
Copy:	Emad Farouz/WDC
PROJECT NUMBER:	SCI-823-10.13

1. Wall 1 & 2: Sheet 12 of 30 of the Retaining Wall Plans dated 8-07 shows cross sections of the abutments. These indicate approximately 10-ft of fill from the bottom of the abutment pile cap to the bottom of the approach slab. Assuming the construction sequence is to build the MSE wall, drive piles through hollow cans, fill the cans with sand, construct the abutment, and then place the ten feet of fill to bottom of the approach slab, and no surcharge load is to be placed, then evaluations of the following should be considered:
   a. How much primary and long term secondary settlement will occur after the piles are driven? This settlement will occur in the soils below the MSE fill and cans. Therefore, downdrag will need to be considered in the portion of the piles below the cans.
   b. If straps are to be placed on the abutment backwall for lateral restraint of the backfill soils, primary and long term secondary settlement could pull the straps downward and cause possible rotation of the backwall, structural distress, and/or break the strap connections.
   c. Impact of primary and long term secondary settlement on the approach slab.

BRIDGE PLANS (June 2007)

2. Rear Abutment Section - Ramp B (Sheet 3 of 3): See comment 1 above.
3. Forward Abutment Section - Ramp B (Sheet 3 of 3): The approach embankment and end slope are approximately 30-ft in height. Since this height of fill is nearly the same as MSE Wall 2, this approach embankment and end slope will likely be constructed in stages. Assuming the construction sequence is to build the embankment in stages, drive piles through hollow cans, fill the cans with sand, construct the abutment, and then place the ten feet of fill to bottom of the approach slab, and no surcharge load is to be

placed, then evaluation of how much primary and long term secondary settlement will occur after the piles are driven should be considered:

a. Bending Stresses in the Battered Piling: The plans show a front row of battered piles. The downward movement of the soil will induce bending in the piles. The magnitude of stress and impact on the performance of these piles will need to be considered.
b. This settlement will occur in the soils below the cans. Therefore, downdrag will need to be considered in the portion of the piles below the cans.
c. If straps are to be placed on the abutment backwall for lateral restraint of the backfill soils, the primary and long term secondary settlement could pull the straps downward and cause possible rotation of the backwall, structural distress, and/or break the strap connections.
d. Impact of primary and long term secondary settlement on the approach slab.

4. <u>Ramp C</u>: Please see comments 1-3 above.

Report for:

Subsurface Exploration for Bridge and MSE Retaining Walls
US 23 Ramp B Over Fairground Road (CR 55), (Bridge No. SCI-823-1593)
US 23 Ramp C Over Fairground Road (CR 55), (Bridge No. SCI-823-1595)
SR 823 Over Fairground Road (CR 55), (Bridge No. SCI-823-1594)
Project SCI-823-10.13 Portsmouth Bypass (PID 79977)
Scioto County, Ohio

Prepared for:

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DLZ Job No. 0121-3070.03

November 5. 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

US 23 RAMP B OVER FAIRGROUND ROAD (CR 55)
(BRIDGE NO. SCI-823-1593)

US 23 RAMP C OVER FAIRGROUND ROAD (CR 55)
(BRIDGE NO. SCI-823-1595)

SR 823 OVER FAIRGROUND ROAD (CR 55)
(BRIDGE NO. SCI-823-1594)

PROJECT SCI-823-10.13 PORTSMOUTH BYPASS (PID 79977)

SCIOTO COUNTY, OHIO

For:

CH2M Hill
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Dublin, Ohio 43017

By:

DLZ OHIO, INC.
6121 Huntley Road
Columbus, OH 43229

DLZ Job. No. 0121-3070.03

November 5, 2007

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PROJECT SCI-823-10.13 PORTSMOUTH BYPASS (PID 79977)
SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of the subsurface exploration and the engineering evaluation of the foundations and mechanically stabilized earth (MSE) retaining walls for the US 23 Interchange bridges over Fairground Road of the Portsmouth bypass project. This project consists in part of constructing three bridges for proposed US 23 Ramp B and US 23 Ramp C, as well as SR 823 over Fairground Road (CR 55). Due to the close proximity and similarities of the proposed structures, recommendations for all three bridges are presented in this document. The findings of other structure evaluations for the Portsmouth bypass project will be submitted in separate documents. It should be noted that this report has been modified from the version dated September 4, 2007. Minor modifications regarding the staged construction details have been made. The information contained in this report supercedes the information in any previous versions of this report.

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, the MSE walls, and the approach 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 MSE walls will be placed at approximate SR 823 stations 891+70 (MSE Wall No. 1) and 892+58 (MSE Wall No. 2). See plan and elevation drawings for the proposed MSE walls in Appendix I. Based upon the provided drawings, it is assumed that the maximum height of MSE Wall No. 1 (east wall) is approximately 31.0 feet. Similarly, the assumed maximum height of MSE Wall No. 2 (west wall) is approximately 29.0 feet. These heights are based upon the maximum difference between the proposed grade of US 23 Ramp B and the approximate existing grade. It should be noted that these wall heights do not include the embedment depth.

The structures as planned, are all single span structures using MSE walls to hold back the roadway embankments and contain the abutments. It is assumed that deep foundations will be used to support the abutments of the proposed structures.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concept 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 drilling a total of nineteen borings in the area of the three proposed structures over Fairground Road. Ten structure borings (TR-xx borings) were drilled for previously proposed structure configurations. Six roadway borings (B-11xx borings) were drilled for the proposed roadway in the area of the three proposed Fairground Road structures. Finally, three structure borings (B-45 through B-47) were drilled for the currently proposed structures over Fairground Road. The boring logs for all borings are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were planned and staked in the field by both representatives of DLZ and representatives of Lockwood, Lanier, Mathias & Noland, Inc. (2LMN). The surveyed locations and ground surface elevations of the borings were determined by representatives of 2LMN. The surveyed locations of the borings are shown on the structure site plan presented in Appendix I, and also on the boring location plan presented in Appendix II.

4.0 FINDINGS

4.1 Geology of the Site

Generalized geological references report that the site lies on the east side of the flood plain of the Teays Stage, Portsmouth River, which is currently the east side of the Scioto River valley. This area is unglaciated, however the Scioto River valley is filled with Illinoian and Wisconsin glacial outwash to depths of up to 90 feet.

The area of these structures is characterized by gently to moderately sloping topography rising from of the floodplain of the Scioto River. 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, alluvial, and lacustrine soils.

The genesis of the soils varies across the site. Soils in the floodplain consist primarily of alluvium and alluvial terraces, generally composed of silty clay, coarse sand, gravel, and cobbles. However, some soils on the hillsides are comprised of lacustrine deposits. 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.

Generalized geologic references report that bedrock across the proposed interchange site consists of shale and sandstone of the Cuyahoga Formation, Sunbury shale, and Berea sandstone of Mississippian to Devonian age.

Toward the eastern end of the proposed interchange, shale and sandstone of the Cuyahoga Formation as well as Sunbury shale were evident in the borings drilled for the Fairground Road structures. Borings drilled west of the Fairground Road site encountered progressively thinner layers of the shale bedrock. Ultimately, the shale was no longer encountered at the top of rock, generally west of the Norfolk and Southern Railroad and immediately east of US 23. West of the Norfolk and Southern railroad, Berea Sandstone was generally encountered at the top of rock.

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. The results of index tests (grain-size and plasticity) are shown on the boring logs, presented in Appendix II. The results of strength and consolidation testing are presented in Appendix III.

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

4.2.1 Soil Conditions

Borings drilled for structure elements and MSE retaining walls on the east side of Fairground Road generally encountered 2 to 5 inches of topsoil at the surface. Below the surface material, cohesive layers consisting of soil ranging from silt (A-4b) to silty clay (A-6b) were encountered to depths ranging from 8.0 to 10.5 feet below the ground surface. Below this layer, cohesionless layers consisting of soils ranging from gravel with sand (A-1-b) to silt (A-4b) were encountered to depth ranging from 13.0 to 14.5 feet below the ground surface, at the top of weathered bedrock.

Similarly, borings drilled for structure elements and MSE retaining walls on the west side of Fairgrounds Road generally encountered 2 to 4 inches of topsoil at the surface. Below the surface material, cohesive layers consisting of soil ranging from sandy silt (A-4a) to silty clay (A-6b) were encountered to depths ranging from 8.0 to 13.0 feet below the ground surface. Below this layer, cohesionless layers consisting of soils ranging from coarse and fine sand (A-3a) to gravel with sand, silt, and clay (A-2-6) were encountered to depths ranging from 17.5 to 21.5 feet below the ground surface, at the top of weathered bedrock.

4.2.2 Bedrock Conditions

Bedrock was confirmed by coring in all borings. Along the east side of Fairground Road, the bedrock generally consisted of soft to medium hard brownish gray shale and medium hard gray, argillaceous sandstone of the Cuyahoga Formation to the termination of the borings, ranging in depth from 24.5 to 37.0 feet below the ground surface.

Borings drilled on the west side of Fairgrounds Road generally encountered bedrock consisting of soft to medium hard gray shale interbedded with sandstone of the Cuyahoga Formation. Three borings (B-45, B-46, and B-1116) were advanced deep enough to encounter medium hard black shale (Sunbury shale) at depths ranging from 33.8 to 38.0 feet below the ground surface, to the termination of the borings. In this location the contact elevation of the Sunbury shale ranges from approximately 527.8 to 531.8, as reported by the borings drilled at this site.

The recovery in each core run varied between 75 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 21 and 100 percent with an average of 74 percent, indicating "fair" to "good" quality rock.

Unconfined compressive strength of tested rock cores ranged between 1,971 and 4,011 pounds per square inch (psi). The tested rock cores were obtained at depths between 20.4 feet and 38.1 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1. The results of these tests are also presented in Appendix III.

Table 1-Rock Core Test Results

Boring	Depth (ft)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
B-45	27.2-28.2	155	2,651
B-45	37.5-38.1	147	3,757
B-46	25.2-25.6	155	4,011
B-46	35.7-36.1	146	3,030
B-47	20.4-20.7	155	1,971
B-47	26.8-27.2	155	3,110

4.2.3 Groundwater Conditions

In borings where seepage was observed, it was first observed at depths ranging from 7.0 to 23.5 feet below the ground surface. Seepage was not observed in borings B-47, TR-54, TR-56, TR-57, and TR-58. Measurable water levels were observed in borings B-1113 and B-1116 prior to rock coring at depths ranging from 17.5 to 29.8 feet below the ground surface. Measurable final water levels were present in all borings upon the completion of coring between approximate depths of 3.3 and 18.0 feet. Final water levels include water that was used during rock coring operations and consequently may not be representative of actual groundwater conditions.

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.

A piezometer was installed in boring B-46 to monitor the groundwater level in the area of the proposed Fairground Road structures. Readings indicate that the groundwater level in boring B-46 is approximately 16.0 feet below the ground surface, corresponding to an elevation of 549.6.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is understood that three bridges are proposed at the Fairground Road location. The recommendations contained in this document pertain to all three of the proposed structures over Fairground Road. For the two proposed MSE walls, separate analyses and recommendations are presented for each of the walls (MSE Wall No. 1 and MSE Wall No. 2).

It is understood through comments from ODOT's Office of Structural Engineering (OSE) that single span structures are preferred for three proposed bridges over Fairground Road. Furthermore, it is understood that driven HP 14x73 piles are preferred to support the proposed structures. In addition to driven piles, recommendations for drilled shaft foundations are also provided.

5.1 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls will be used to construct the approach embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. Based upon subsurface conditions and discussions with the client, it was assumed that deep foundations would be used to support the bridges for the purposes of performing stability analyses and settlement calculations for the proposed MSE walls.

It should be noted that MSE Wall No. 1 lies on the east side of Fairground Road. Similarly, MSE Wall No. 2 lies on the west side of Fairground Road. These walls are continuous, extending in front of the abutments for all three proposed bridges at this location. Because the walls are continuous, and due to the varied soil strength characteristics along wall locations, the most critical subsurface conditions, coupled with the greatest wall height were selected to analyze the stability and settlement of each wall.

5.1.1 MSE Walls - General Information

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

A global stability analysis and bearing capacity analysis were performed for the MSE walls at this bridge location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding, overturning, and settlement.

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability and settlement analyses are presented in Appendix IV. Other internal stability analyses (i.e. strap design) are required for the design of an MSE wall, but are considered outside the scope of this report.

Global stability analyses have indicated that the approach embankments may be built using slopes characterized by 2H:1V side slopes. It should be noted that adjacent embankment sections may be more critical than the section analyzed for this report. Consequently, the embankment side slope recommendations will be presented in a separate report for the interchange. However, for the purposes of these analyses it is assumed that 2H:1V slopes will be used.

5.1.2 Shear Strength Parameter Selection

Shear strength values for use in stability analyses were based on laboratory strength testing, in-situ vane shear testing, in-situ moisture content and hand penetrometer values, typical values, and engineering judgment. Table 2 outlines the strength parameters assumed in analyses for the respective MSE retaining walls and embankments. Also, the results of laboratory testing are included in Appendix III.

Due to the varied results of CIU testing, possibly due to varying granular content, additional test results from the interchange area were considered for the staged construction evaluation of MSE Wall No. 2. Tests run on silty clay (A-6b) samples obtained from nearby borings B-1105A and B-1108 reported the angle of shearing resistance (from total stress curve, F_{cu}) ranging from 20.4 to 22.2 degrees. Considering these test results, as well as those from borings B-45 and B-46, we conservatively selected 15.0 degrees for the angle of shearing resistance for the staged construction analyses. The results of these tests are also included in Appendix III.

In accordance with ODOT guidelines, a unit weight of 120 pounds per cubic foot (pcf) and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. Similarly, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 2- Soil Parameters Used in The MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill	Select Granular Backfill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Wall No. 1) (B-47 & B-1146)	Very Stiff Silt (A-4b) & Silt and Clay (A-6a)	120	2350	0	0	29
Foundation Soil (Wall No. 2) (B-45 & B-1113)	Stiff Silty Clay (A-6b)	120	1500	0^*	0	29

*Bearing capacity analyses required an assumed value for the angle of shearing resistance (F_{cu}) for staged construction evaluations of MSE Wall No. 2 only.

The stability analyses were performed using UTEXAS3 Version 1.204, a slope stability computer program using variations of the method of slices. UTEXAS3 was developed by Dr. Stephen Wright at the University of Texas for the U.S. Army Corps of Engineers. The results of stability analyses and settlement calculations are included in Appendix IV.

5.1.3 MSE Wall Evaluations and Recommendations – Wall No. 1

For MSE Wall No. 1, located on the east side of Fairground Road, the subsurface profile encountered by borings B-47 and B-1146 were assumed to be the most critical borings with respect to stability. Consequently a composite profile based upon these two borings was assumed in the stability analyses for this wall.

It should be noted that the maximum wall height (measured to the top of the coping) was approximately 18.0 feet. However, as per ODOT's Supplemental Specification 840 (SS 840), section 840.04 A, the full height of MSE retaining walls in front of abutments should be measured to the profile grade elevation at the face of the wall. Consequently, the maximum wall height at the MSE Wall No. 1 location was measured to be approximately 34.0 feet (including the 3-foot embedment). It is assumed that the top of leveling pad for this wall will be placed at approximate elevation 563.

Borings B-47 and B-1146 generally encountered very stiff silt (A-4b) and silt and clay (A-6a) from the bottom of the leveling pad excavation (el. 561.5) to approximate elevation 557.2. Below this layer, borings generally encountered cohesionless silt (A-4b) to approximate elevation 554.5, at the top of weathered bedrock.

Analyses were performed to determine the global stability, bearing capacity and stability (overturning and sliding) of the proposed MSE wall bearing on the existing soils. The results of the analyses indicated that the factors of safety for global stability, bearing capacity and stability (overturning and sliding) were all above the minimum recommended values.

Calculations have indicated that a minimum reinforcement length of 1.1 times the full height (H+D) or 37.4 feet is required for stability of the proposed MSE wall at the Wall No.1 location.

The maximum settlement at the face of MSE Wall No.1 was estimated to be approximately 3 inches for the full height wall section. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankment loading. Differential settlement at this location was estimated to be approximately 0.4 percent, which is less than the typically cited maximum value. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1.0 percent).

Time-rate of consolidation calculations indicate that ninety percent of the total primary consolidation should occur in approximately 7 days. Given the relatively small magnitude of consolidation, and the short estimated time to ninety percent consolidation, it will likely not be necessary to use prefabricated vertical drains (wick drains) or other means to accelerate consolidation.

Table 3 presents the MSE retaining wall parameters and results of analyses for MSE Wall No. 1.

Table 3 - MSE Retaining Wall Parameters and Analyses Results
MSE Wall No. 1, East of Fairground Road

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on Φ' = 30°)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37
Allowable Bearing Capacity – Undrained Condition q_{all} = 4,901 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 8,627 psf
Global Stability Factor of Safety – Undrained Condition = 2.6 Factor of Safety – Drained Condition = 2.1 Factor of Safety – Drained Seismic Condition = 1.9
Estimated Settlement of MSE Volume Maximum Total Settlement = 3 inches Differential Settlement = 0.4% (maximum allowable is 1.0% ODOT BDM 204.6.2.1)
Full Height of MSE Wall = 34.0 feet (including embedment depth) Minimum Embedment Depth = 3.0 feet* Minimum Length of Reinforcement for External Stability, 1.1(H+D) = 37.4 feet

* Assumed top of leveling pad elevation is 563. Embedment Depth may vary depending on actual top of leveling pad elevation. Minimum embedment depth of 3.0 feet.

5.1.4 MSE Wall Evaluations and Recommendations – Wall No. 2

For MSE Wall No. 2, located on the west side of Fairground Road, the subsurface profile encountered by borings B-45 and B-1113 were assumed to be the most critical with respect to stability. Consequently a composite profile based upon these two borings was assumed in the stability analyses for this wall.

It should be noted that the maximum wall height (measured to the top of the coping) was approximately 16.5 feet. However, as per ODOT's Supplemental Specification 840 (SS 840), section 840.04 A, the full height of MSE retaining walls in front of abutments should be measured to the profile grade elevation at the face of the wall. Consequently, the maximum wall height was measured to be approximately 32.0 feet (including the 3-foot embedment). It is assumed that the top of leveling pad for this wall will be placed at approximate elevation 562.

Borings B-45 and B-1113 generally encountered stiff silty clay (A-6b) and gravel with sand, silt, and clay (A-2-6) from the bottom of the leveling pad excavation (el. 560.5) to approximate elevation 548.0. Below this layer, borings generally encountered gravel with sand and silt (A-2-4) to approximate elevation 544.5, at the top of weathered bedrock.

Analyses were performed to determine the global stability, bearing capacity and stability (overturning and sliding) of the proposed MSE walls bearing on the existing soils. The results of the analyses indicated that the factors of safety for global stability, drained bearing capacity and stability (overturning and sliding) were all above the minimum recommended values. However, the factor of safety for undrained bearing capacity of MSE Wall No. 2 was found to be 1.7, which is below the minimum required value of 2.5.

In order to construct the wall while maintaining the minimum factor of safety against undrained bearing capacity, the use of staged construction was explored.

Additional analyses were performed which assume that an increase in the undrained shear strength of the foundation soils will occur via consolidation under the loading of each stage. These analyses indicate that MSE Wall No. 2 could be built in three stages while monitoring the pore water pressures in clay layers. In order to maintain the minimum required factor of safety against undrained bearing capacity failure, it is recommended that the proposed MSE wall be constructed in stages.

Based upon additional analyses, the first stage of 19.0 feet plus the embedment depth may be constructed while maintaining a factor of safety of 2.5 against undrained bearing capacity failure. At least ninety percent of excess pore pressures should be allowed to dissipate prior to placing the next stage. Correspondingly, excess pore water pressures measured in the foundation clay layers during construction should fall below 1.6 psi prior to placing the next stage. After excess pore pressures have sufficiently dissipated, the second stage of 8.0 feet may be constructed. At least ninety percent of excess pore pressures should be allowed to dissipate prior to placing the final stage. Correspondingly, excess pore water pressures measured in the foundation clay layers during construction should fall below 0.7 psi prior to placing the final stage. After excess pore pressures have sufficiently dissipated, the final stage may be constructed up to the proposed grade.

Time-rate of consolidation calculations indicate that an estimated consolidation period of 18 days after both the first stage and the second stage of construction would be required to allow the excess pore water pressures to dissipate in the foundation soils. It is anticipated that a significant portion of the pressures will dissipate during the construction of the MSE walls. The ODOT construction representative may modify the waiting periods observed during construction based upon pore pressure measurements in the field. Given the relatively short estimated time to ninety percent consolidation, it will likely not be necessary to use prefabricated vertical drains (wick drains) or other means to accelerate dissipation of pore water pressures for staged construction.

As stated previously, it is recommended that pore water pressures be monitored in the clay layers of the foundation soils. Recommendations and placement instructions for the piezometers will be included in the Final Report of Subsurface Exploration and MSE Wall and Embankment Evaluations for Proposed US 23/SR 823 Interchange.

Calculations indicated that a minimum reinforcement length of 1.1 times the full height (H+D) or 35.2 feet is required for stability of the proposed MSE wall at the Wall No. 2 location.

The maximum settlement at the face of MSE Wall No. 2 was estimated to be approximately 5 inches for the full height wall section. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankment loading. Differential settlement at this location was estimated to be approximately 0.6 percent, which is less than the typically cited maximum value. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1.0 percent).

Table 4 presents the MSE retaining wall parameters and results of analyses for MSE Wall No. 2.

Table 4 - MSE Retaining Wall Parameters and Analyses Results
MSE Wall No. 2, West of Fairground Road

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi' = 30°$)
Sliding along base of MSE wall Sliding Coefficient (m)(0.67) = tan 29°(0.67) = 0.37
Allowable Bearing Capacity – Undrained Condition *(Staged Construction)*+ q_{all} *Stg.1=3,153 psf* q_{all} *Stg.2=4,284 psf* q_{all} *Stg.3=4,761 psf*
Allowable Bearing Capacity – Drained Condition q_{all} = 8,181 psf
Global Stability Factor of Safety – Undrained Condition = 2.0 Factor of Safety – Drained Condition = 2.2 Factor of Safety – Drained Seismic Condition = 2.0
Estimated Settlement of MSE Volume Maximum Total Settlement = 5 inches Differential Settlement = 0.6% (maximum allowable is 1.0% ODOT BDM 204.6.2.1)
Full Height of MSE Wall = 32.0 feet (including embedment depth) Minimum Embedment Depth = 3.0 feet* Minimum Length of Reinforcement for External Stability, 1.1(H+D) = 35.2 feet

* Assumed top of leveling pad elevation is 562. Embedment depth may vary depending on actual top of leveling pad. Minimum embedment depth of 3.0 feet.

+ See Section 5.1.4 for staged construction details.

5.2 Bridge Foundation Recommendations

It is understood that driven HP 14x73 piles are preferred to support the proposed structures. In addition, recommendations for drilled shaft foundations are also provided. Additionally, due to the nature of single span structures, uplift is not anticipated at either of the abutment locations for the three structures. Due to the height of the proposed embankments and the poor soil conditions encountered, it is assumed that spread footing foundations will not be considered. Consequently, foundation recommendations for spread footings will not be provided at this time. However, recommendations for spread footings or alternative foundations can be provided upon request.

5.2.1 Pile Foundations

It is recommended that HP 14x73 piles, driven to refusal on the top of rock be used to support the proposed abutments. Table 5 summarizes the site conditions and foundation recommendations for the three proposed Fairground Road structures. It should be noted that the bedrock surface varies across the project area. The approximate pile tip elevations presented in Table 5 indicate the approximate elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 5-Summary of Driven Pile Tip Elevations, HP 14x73*

Structure	Substructure	Boring Number	Existing Ground Surface Elevation (Ft)	Estimated Pile Tip Elevation (Ft)
SR 823 (Right) over Fairgrounds Road	Rear Abutment	B-1146	567.7	553.2
	Forward Abutment	B-1144	565.2	542.2
SR 823 (Left) over Fairgrounds Road	Rear Abutment	B-1145	567.3	552.0
	Forward Abutment	TR-55A	565.4	545.4
Ramp B over Fairgrounds Road	Rear Abutment	B-45	566.0	543.0
	Forward Abutment	TR-58	567.1	550.6
Ramp C over Fairgrounds Road	Rear Abutment	TR-54	566.9	551.9
	Forward Abutment	B-46	565.6	545.6

* *Cited pile tip elevations are also considered representative of HP 12x53 piles.*

It is anticipated that piles will encounter refusal at a depth of approximately 14.5 to 16.5 feet below the ground surface for foundations on the east side of Fairground Road. Similarly, it is anticipated that piles will encounter refusal at a depth of approximately 20.0 to 23.0 feet below the existing ground surface for foundations on the west side of Fairgrounds Road. Based upon the degree of weathering and the strength characteristics of the shale bedrock, it is anticipated that the piles will penetrate approximately one to two feet beyond the top of rock elevation cited on the boring logs.

If driven to refusal, the maximum allowable capacity of the pile can be used. Because the piles will be driven to bedrock, it is recommended that reinforced pile points be used to prevent the piles from being damaged. Although reinforced pile points are not required in shale, the predominate bedrock type in the area of the proposed structure contains interbedded sandstone, which if driven into could damage piles. Pile sleeves should be placed from the bottom of the leveling pad to the pile cap elevation to permit pile installation through the soil reinforced zone of the MSE wall.

To prevent downdrag forces from reducing the allowable capacity of the piles, the piles should not be driven until at least ninety percent (U=90%) of the total primary consolidation has occurred. Fill should be placed to the proposed roadway grade level and allowed to consolidate prior to driving piles. It is estimated that a waiting period of 18 and 7 days after completing fill placement and prior to driving piles at the west and east abutments, respectively, will be required to achieve ninety percent (U=90%) consolidation. Downdrag calculations are presented in Appendix IV.

Due to the tendency of certain shales to "relax", it is recommended that the contractor restrike the piles seven days after installation to ensure that the allowable bearing capacity of the pile is met.

5.2.2 Drilled Shaft Foundations

As an alternative to pile foundations, drilled shafts could also be considered for the support of the proposed abutments. It is recommended that the 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). Calculations for drilled shaft foundations are presented in Appendix IV.

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 a reasonable shaft diameter, drilled shafts may be designed as friction-type shafts. Neglecting the overburden, upper two feet and bottom length equal to one diameter of the socket, an allowable sidewall shear stress/adhesion of 3,750 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 argillaceous sandstone and shales 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 prior to the placement of concrete.

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. Significant seepage was encountered by several of the borings. In addition, groundwater levels indicate that groundwater would flow into excavations into rock through granular layers overlying bedrock. It should be 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 to 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.

When using drilled shaft foundations in conjunction with MSE retaining walls, it is necessary to consider the placement of the drilled shafts with respect to the MSE wall and soil reinforcing straps. Drilled shafts should be installed at a sufficient distance from the back of the MSE wall such that the soil reinforcement can be splayed around the shafts with splay angles of 15 degrees or less. From the center of the drilled shafts to the back of the MSE wall, this dimension is approximately two times the shaft diameter.

5.3 General Earthwork Recommendations

The proposed alignment traverses a gently to moderately sloping area. Consequently, the placement of fill will be required to construct the approach embankments at the abutments. The maximum fill anticipated is approximately 31 feet at MSE Wall No. 2.

Generally between 2 to 5 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.

Soils with significant organic content were not encountered in any of the borings drilled for the structures. However, organic or very soft soils may be encountered at locations other than where the borings were drilled. Consequently, the contractor should be prepared to perform overexcavation of any poor soils at other locations and replace the overexcavated soil with compacted engineered fill as needed.

The embankments should be constructed in accordance with ODOT Item 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.

Excavations should be prepared in accordance with ODOT Item 503, "Excavation for Structures." 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.

It is recommended that earthwork be performed under continuous observation and testing by a soils technician with the general guidance of a geotechnical engineer.

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. 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.

5.4 Groundwater Considerations

In borings where seepage was observed, it was first observed at depths ranging from 7.0 to 23.5 feet below the ground surface. Seepage was not observed in borings B-47, TR-54, TR-56, TR-57, and TR-58. Measurable water levels were observed in borings B-1113 and B-1116 prior to rock coring at depths ranging from 17.5 to 29.8 feet below the ground surface. Measurable final water levels were present in all borings upon the completion of coring between approximate depths of 3.3 and 18.0 feet. Final water levels include water that was used during rock coring operations and consequently may not be representative of actual groundwater conditions.

Excavations for the leveling pad of the proposed MSE retaining walls are anticipated to encounter only minor seepage. However, for deeper excavations, groundwater and significant seepage should be anticipated in the granular layers overlying bedrock. Excavations or shafts extending below the top of rock may encounter more significant seepage through fractured zones in the bedrock. The contractor should be prepared to deal with seepage, water flow, and precipitation that may enter any excavations.

6.0 CLOSING REMARKS

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

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams

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

sjr

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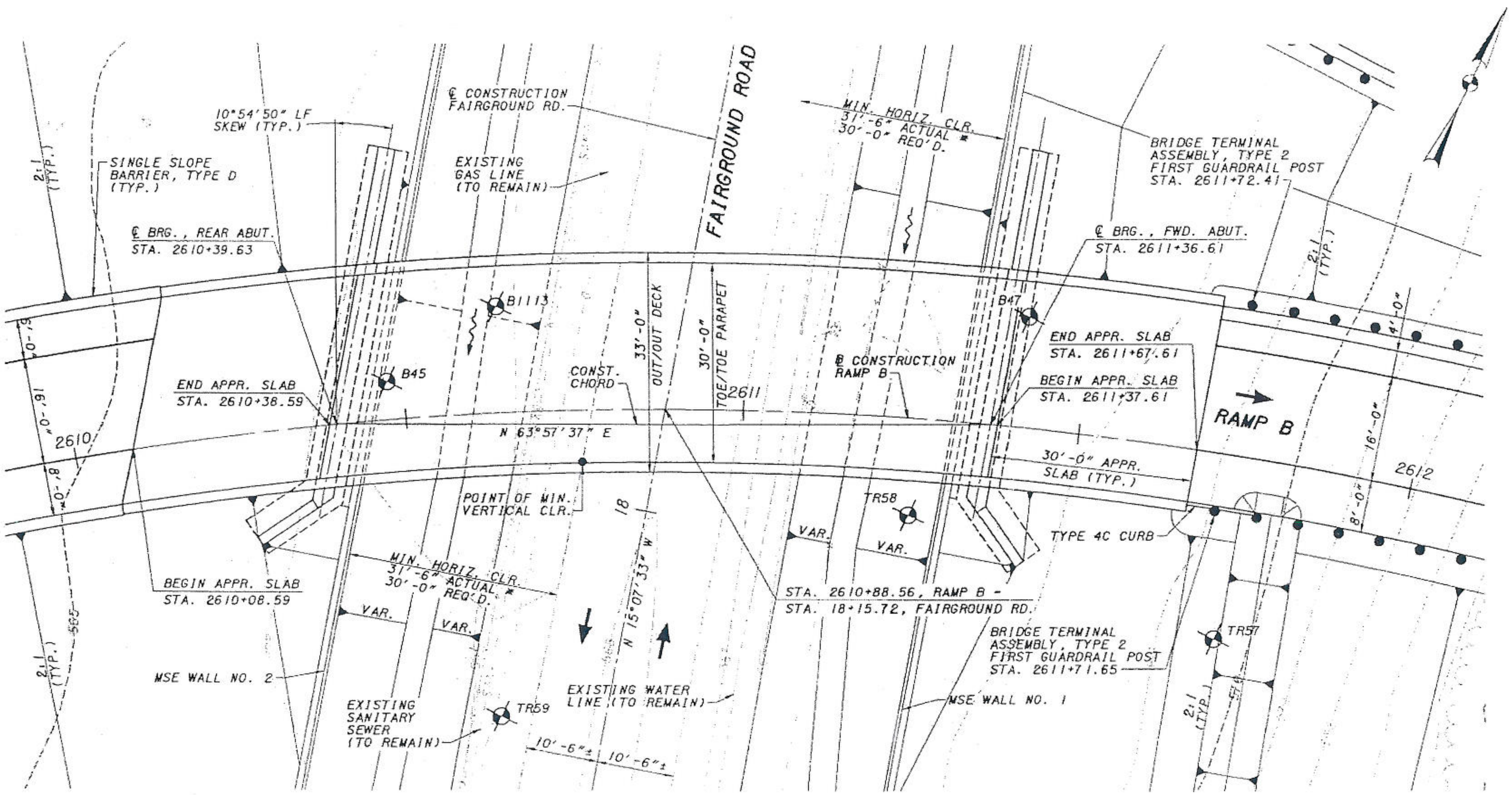
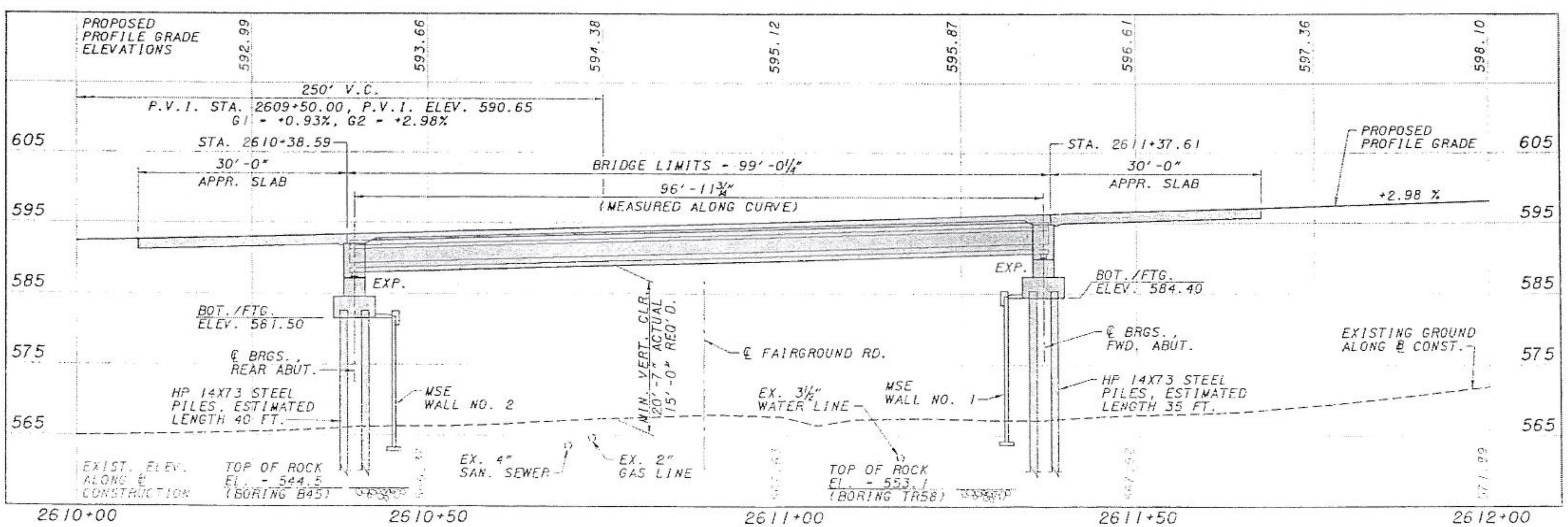
APPENDIX I

Structure Plan and Profile Drawings - 11"x17"
MSE Wall Plan and Elevation Drawings - 11"x17"



PLAN

* 31'-6" PROVIDED TO PERMIT FUTURE 12'-0" LANE



PROFILE ALONG ℄ CONSTRUCTION, RAMP B

BENCHMARKS

CURVE DATA - RAMP B

P.I. Sta - 2609+99.07
Δ = 102° 45' 15" (RT)
Dc = 11° 15' 00"
R = 509.30'
T = 637.46'
L = 913.37'
E = 306.83'

TRAFFIC DATA

CURRENT ADT (2010) - 2700
DESIGN ADT (2030) - 3600
DESIGN ADTT - 500

LEGEND

INDICATES BORING LOCATION

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

POWER AND TELEPHONE LINES TO BE RELOCATED

PROPOSED STRUCTURE

TYPE: SINGLE SPAN COMPOSITE PRESTRESSED CONCRETE I-BEAMS WITH REINFORCED CONCRETE DECK AND SEMI-INTEGRAL ABUTMENTS ON MSE WALLS

LENGTH OF SPAN: 96'-11¾" C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-0" TOE/TOE PARAPETS

SIDEWALK: NONE

DESIGN LOADING: HS25 AND THE ALTERNATE MILITARY LOADING, FWS - 60 LB/FT²

SKEW: 10°54'50" LEFT FORWARD, MEASURED FROM THE NORMAL TO THE CONSTRUCTION CHORD

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (30'-0" LONG)

ALIGNMENT: HORIZONTALLY CURVED (℄ RADIUS - 509.30')

SUPERELEVATION: 0.071 FT/FT

LATITUDE: N 38°53'31"

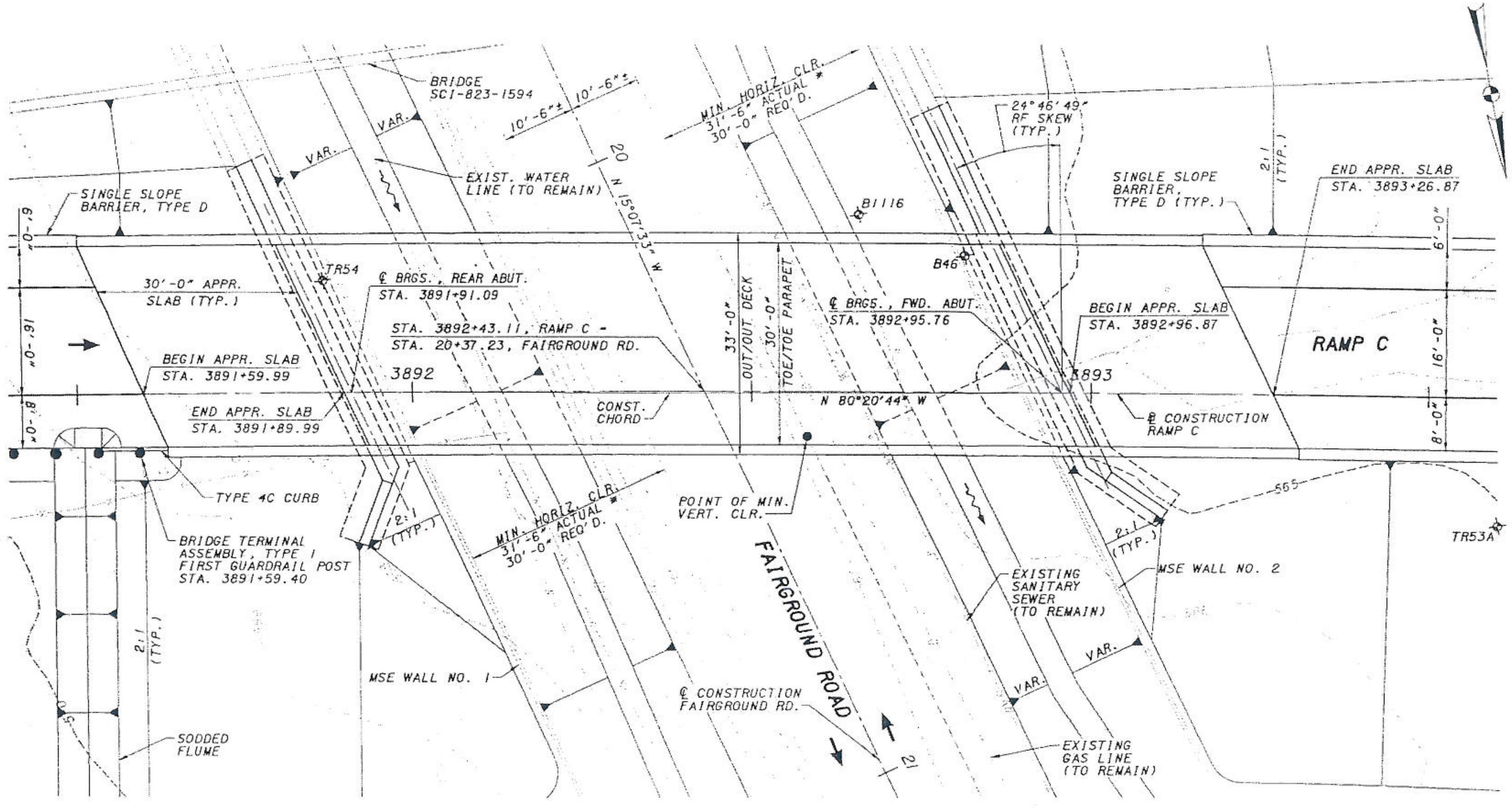
LONGITUDE: W 82°59'51"

DESIGN AGENCY	CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017
DESIGNED	DGS
CHECKED	SKT
DRAWN	JBA
REVISED	
REVIEWED	SCJ
DATE	08/07
STRUCTURE FILE NUMBER	7306717

SCIOTO COUNTY
STA. 2610+38.59
TO STA. 2611+37.61

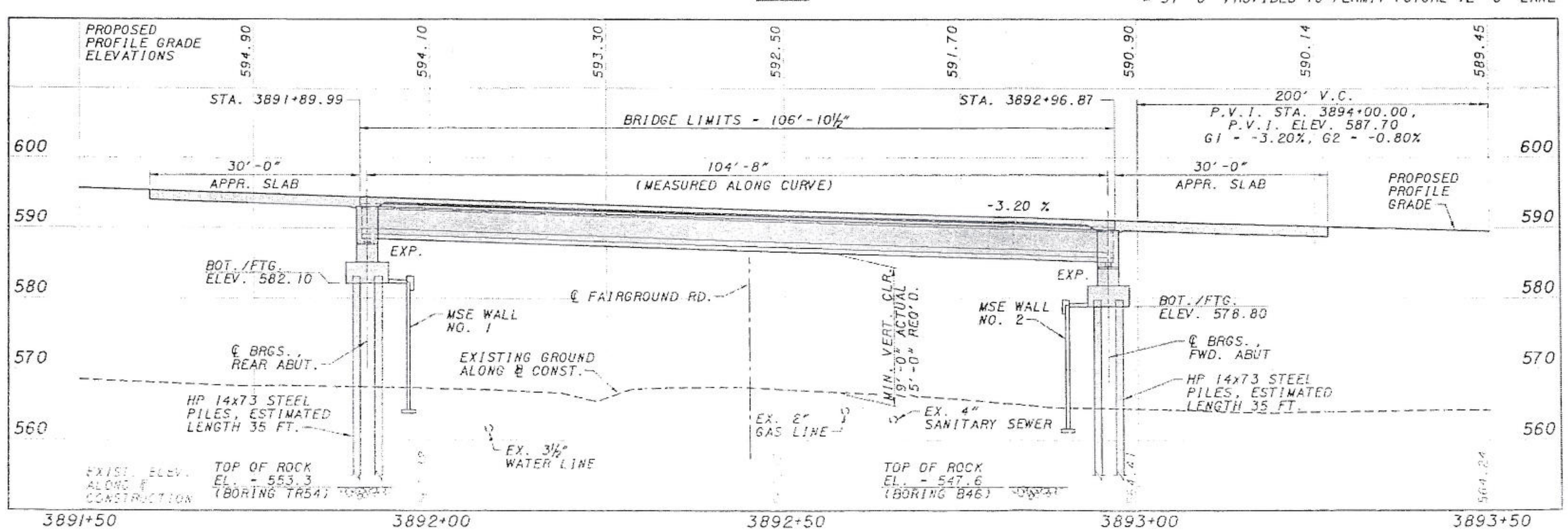
SITE PLAN
BRIDGE NO. SCI-823-1593
RAMP B OVER FAIRGROUND ROAD

SCI-823-10.13
PID 79977



PLAN

* 31'-6" PROVIDED TO PERMIT FUTURE 12'-0" LANE



PROFILE ALONG ℄ CONSTRUCTION, RAMP C

BENCHMARKS

CURVE DATA - RAMP C
P.I. Sta - 3889+21.16
Δ - 9° 37' 49" (RT)
Dc - 1° 00' 00"
R - 5,729.58'
T - 482.65'
L - 963.03'
E - 20.29'

TRAFFIC DATA

CURRENT ADT (2010) - 6200
DESIGN ADT (2030) - 9400
DESIGN ADTT - 1320

LEGEND

✧ INDICATES BORING LOCATION

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

POWER AND TELEPHONE LINES TO BE RELOCATED

PROPOSED STRUCTURE

TYPE: SINGLE SPAN COMPOSITE PRESTRESSED CONCRETE I-BEAMS WITH REINFORCED CONCRETE DECK AND SEMI-INTEGRAL ABUTMENTS ON MSE WALLS

LENGTH OF SPAN: 104'-8" C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-0" TOE/TOE PARAPETS

SIDEWALK: NONE

DESIGN LOADING: HS25 AND THE ALTERNATE MILITARY LOADING, FWS - 60 LB/FT²

SKEW: 24°46'49" RIGHT FORWARD, MEASURED FROM THE NORMAL TO THE CONSTRUCTION CHORD

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (30'-0" LONG)

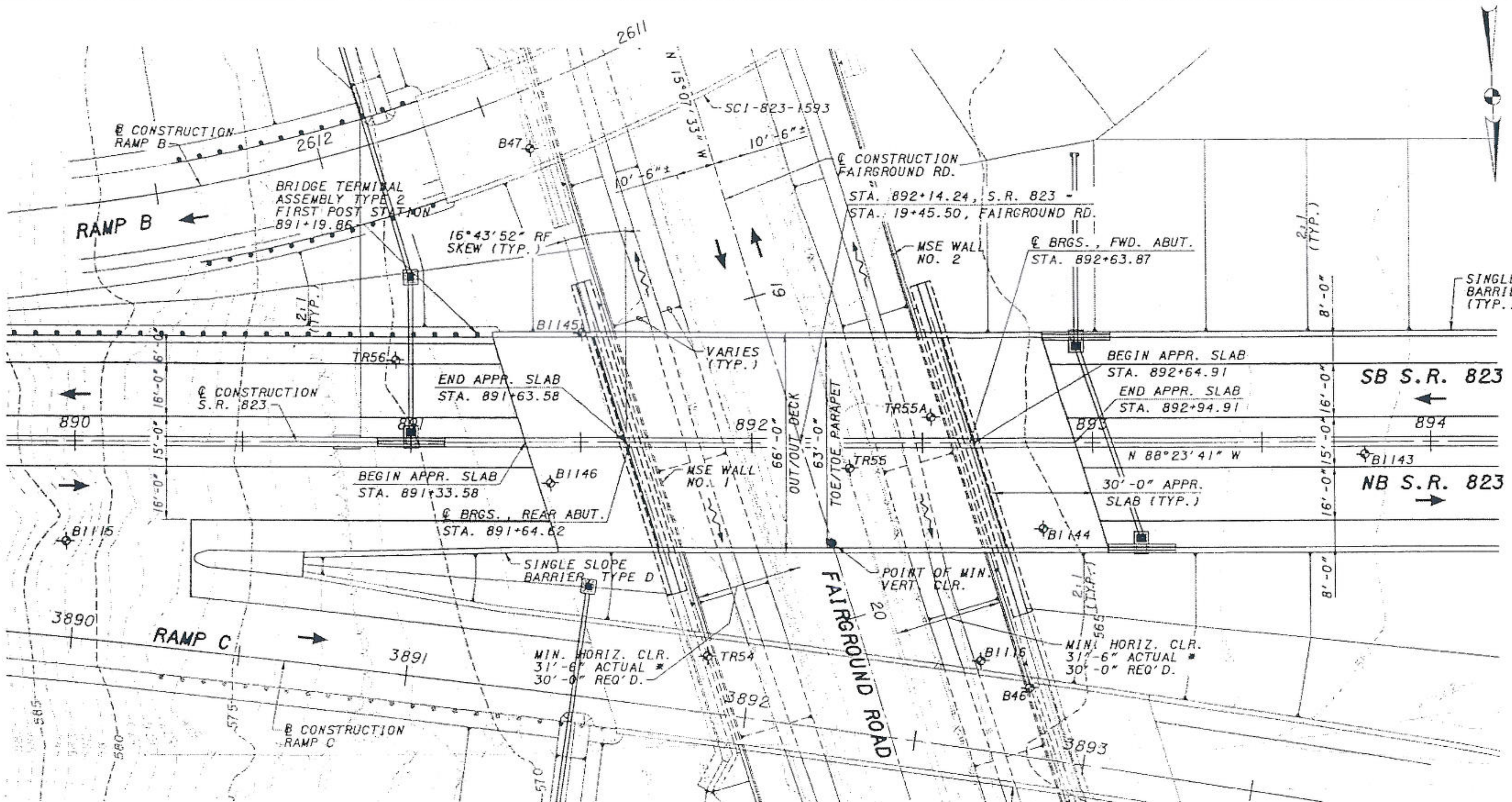
ALIGNMENT: HORIZONTALLY CURVED (℄ RADIUS - 5729.58')

SUPERELEVATION: 0.029 FT/FT

LATITUDE: N 38°53'33"

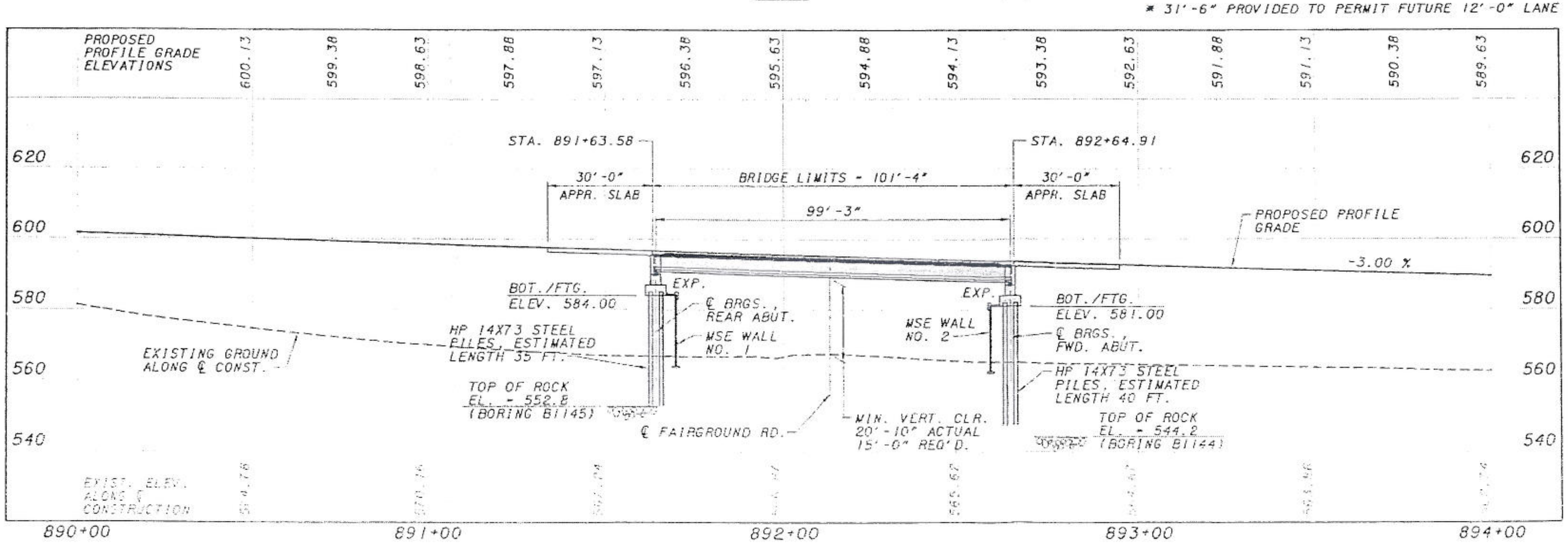
LONGITUDE: W 82°59'52"

DESIGN AGENCY CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	REVIEWED SCJ / DATE 08/07 / STRUCTURE FILE NUMBER 7306733	DRAWN JBA / REVISED	DESIGNED DGS / CHECKED SKT	SCIOTO COUNTY STA. 3891+89.99 TO STA. 3892+96.87	SITE PLAN BRIDGE NO. SCI-823-1595 RAMP C OVER FAIRGROUND ROAD	SCI-823-10.13 PID 79977	1 / 3



PLAN

* 31'-6" PROVIDED TO PERMIT FUTURE 12'-0" LANE



PROFILE ALONG PROFILE GRADE LINE, S.R. 823

BENCHMARKS

TRAFFIC DATA

CURRENT ADT (2010) - 8900
DESIGN ADT (2030) - 13000
DESIGN ADTT - 1820

LEGEND

✧ INDICATES BORING LOCATION

NOTES

EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.

POWER AND TELEPHONE LINES TO BE RELOCATED.

PROPOSED STRUCTURE

TYPE: SINGLE SPAN COMPOSITE PRESTRESSED CONCRETE I-BEAMS WITH REINFORCED CONCRETE DECK AND SEMI-INTEGRAL ABUTMENTS ON MSE WALLS

LENGTH OF SPAN: 99'-3" C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-1⅛" TOE/TOE PARAPETS (RB)
30'-1⅛" TOE/TOE PARAPETS (LB)

SIDEWALK: NONE

DESIGN LOADING: HS25 AND THE ALTERNATE MILITARY LOADING, FWS - 60 LB/FT²

SKEW: 16°43'52" RIGHT FORWARD

WEARING SURFACE: MONOLITHIC CONCRETE

APPROACH SLABS: AS-1-81 (30'-0" LONG)

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

LATITUDE: N 38°53'32"

LONGITUDE: W 82°59'52"

DESIGN AGENCY	DESIGNED	DRAWN	REVIEWED	DATE		
CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	JBA	JBA	SCJ	07/07	SCIOTO COUNTY STA. 891+63.58 TO STA. 892+64.91	SITE PLAN BRIDGE NO. SCI-823-1594 S.R. 823 OVER FAIRGROUND ROAD
	CHECKED SKT	REVISED	STRUCTURE FILE NUMBER 7306725			

SCI-823-10.13
PID 79977

1 / 3

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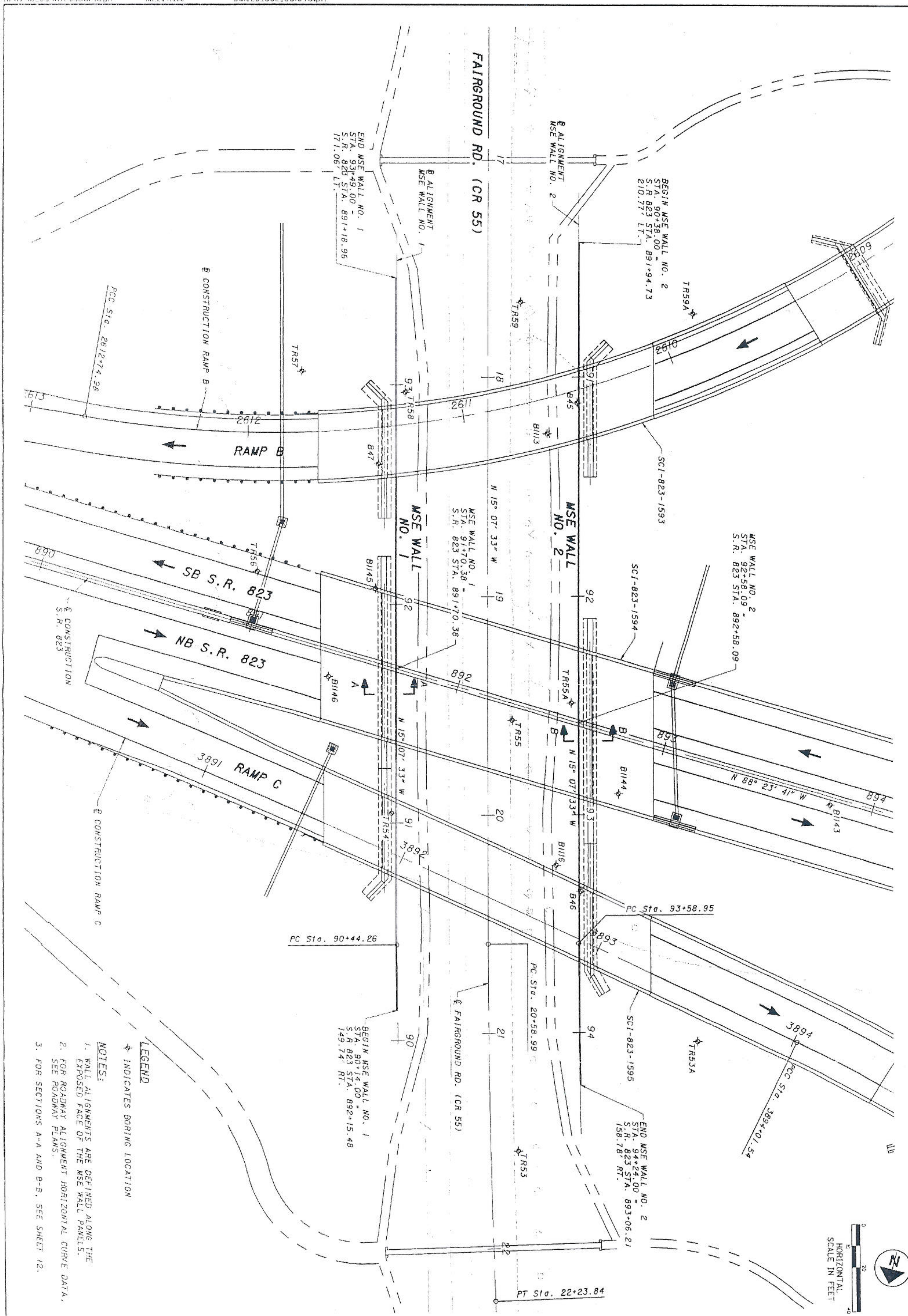
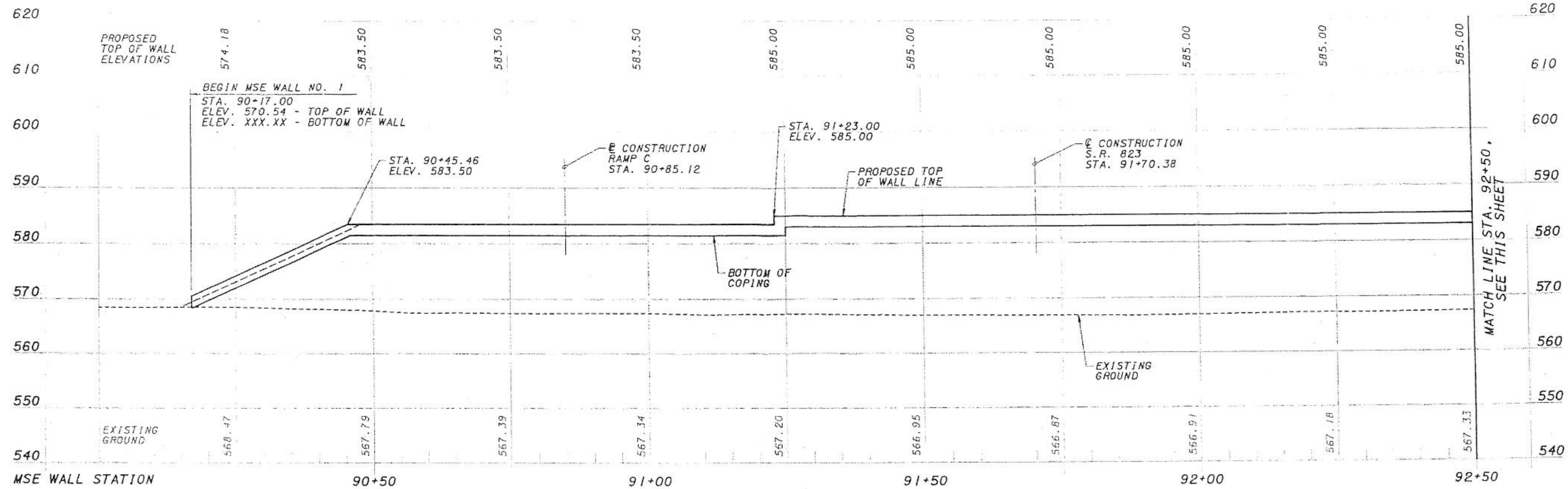
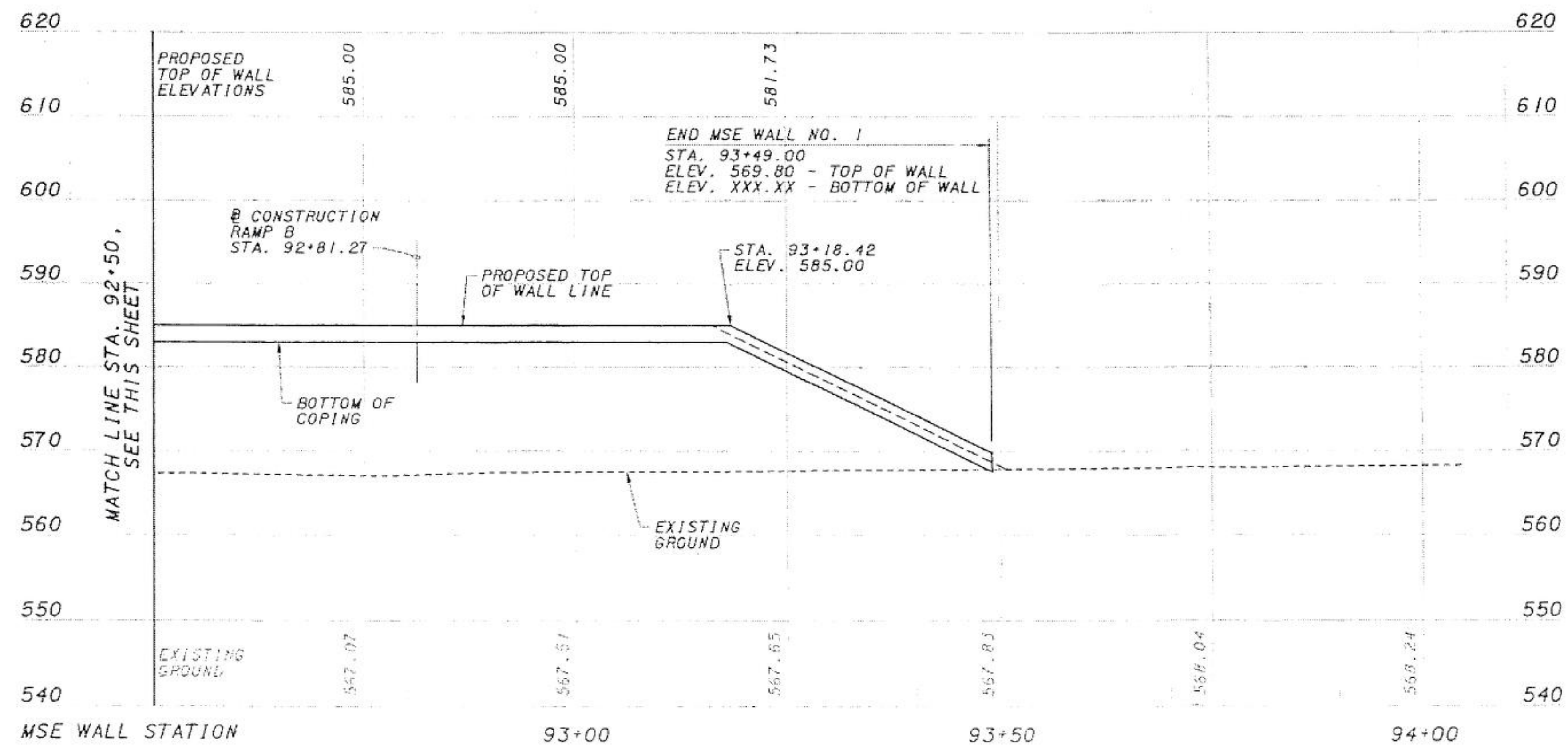


9/30	SCI-823-10.13 PID 79977	MSE WALLS 1 & 2 PLAN VIEW	DESIGNED JBA	DRAWN JBA	REVIEWED	DATE 08/07	DESIGN AGENCY CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017
			CHECKED JBA	REVISED	STRUCTURE FILE NUMBER		





NOTES:

1. MSE WALL ELEVATIONS ARE SHOWN VIEWED FROM THE EXPOSED FACE OF THE WALL.
2. FOR DEFINITION OF MSE WALL TOP AND BOTTOM ELEVATIONS, SEE WALL SECTION ON SHEET X.

DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA		08/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	

WALL ELEVATION - MSE WALL NO. 1

SCI-823-10.13
PID 79977

X / X

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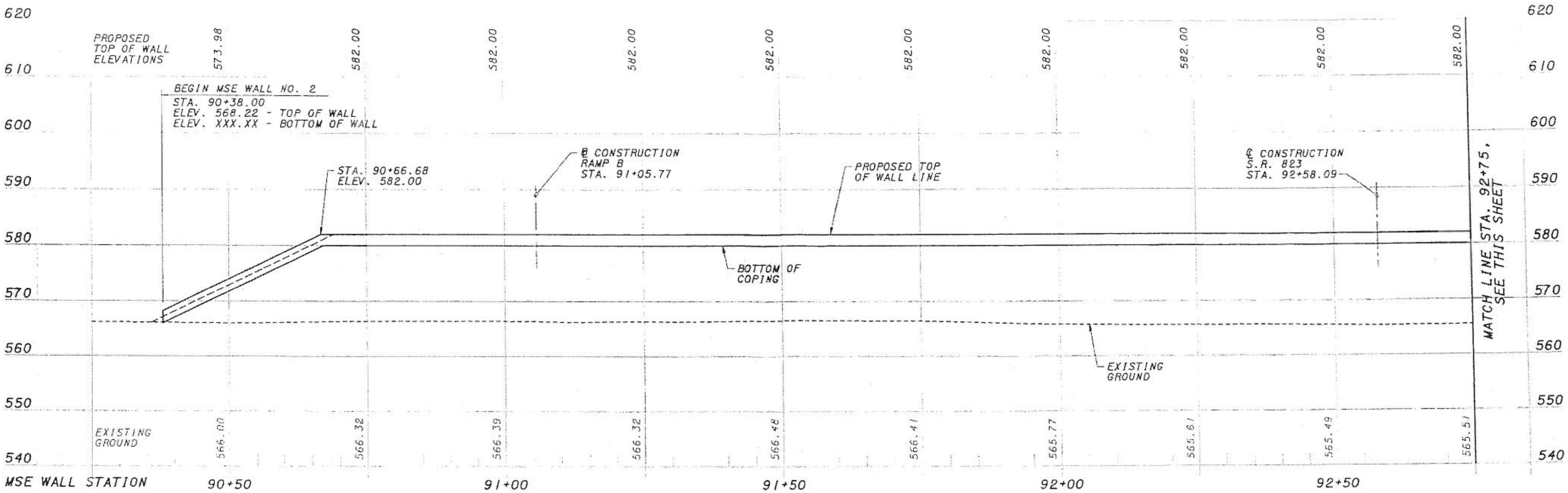

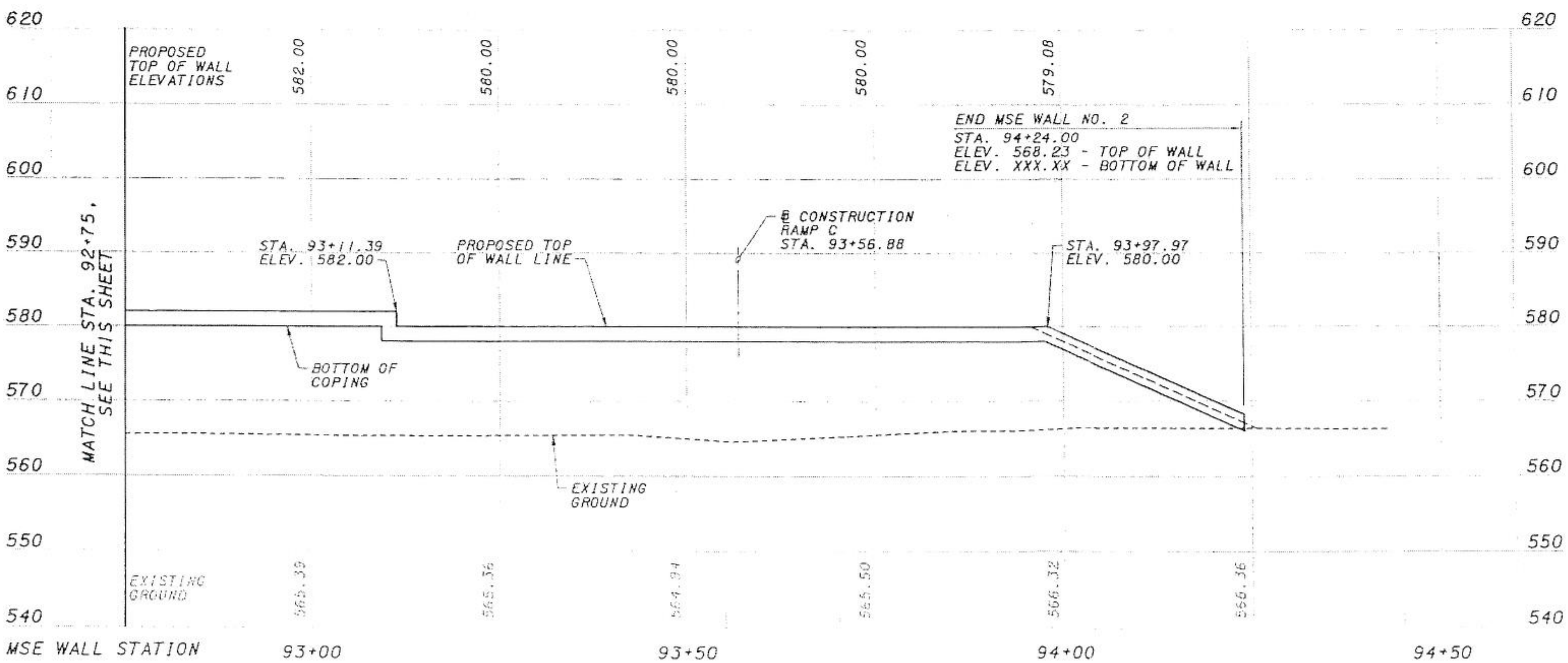

NOTES:

1. MSE WALL ELEVATIONS ARE SHOWN VIEWED FROM THE EXPOSED FACE OF THE WALL.
2. FOR DEFINITION OF MSE WALL TOP AND BOTTOM ELEVATIONS, SEE WALL SECTION ON SHEET X.

DESIGN AGENCY	DESIGNED	DRAWN	REVIEWED	DATE
CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	JBA	JBA		08/07
	CHECKED	REVISED	STRUCTURE FILE NUMBER	

WALL ELEVATION - MSE WALL NO. 2

SCI-823-10.13
PID 79977

X / X

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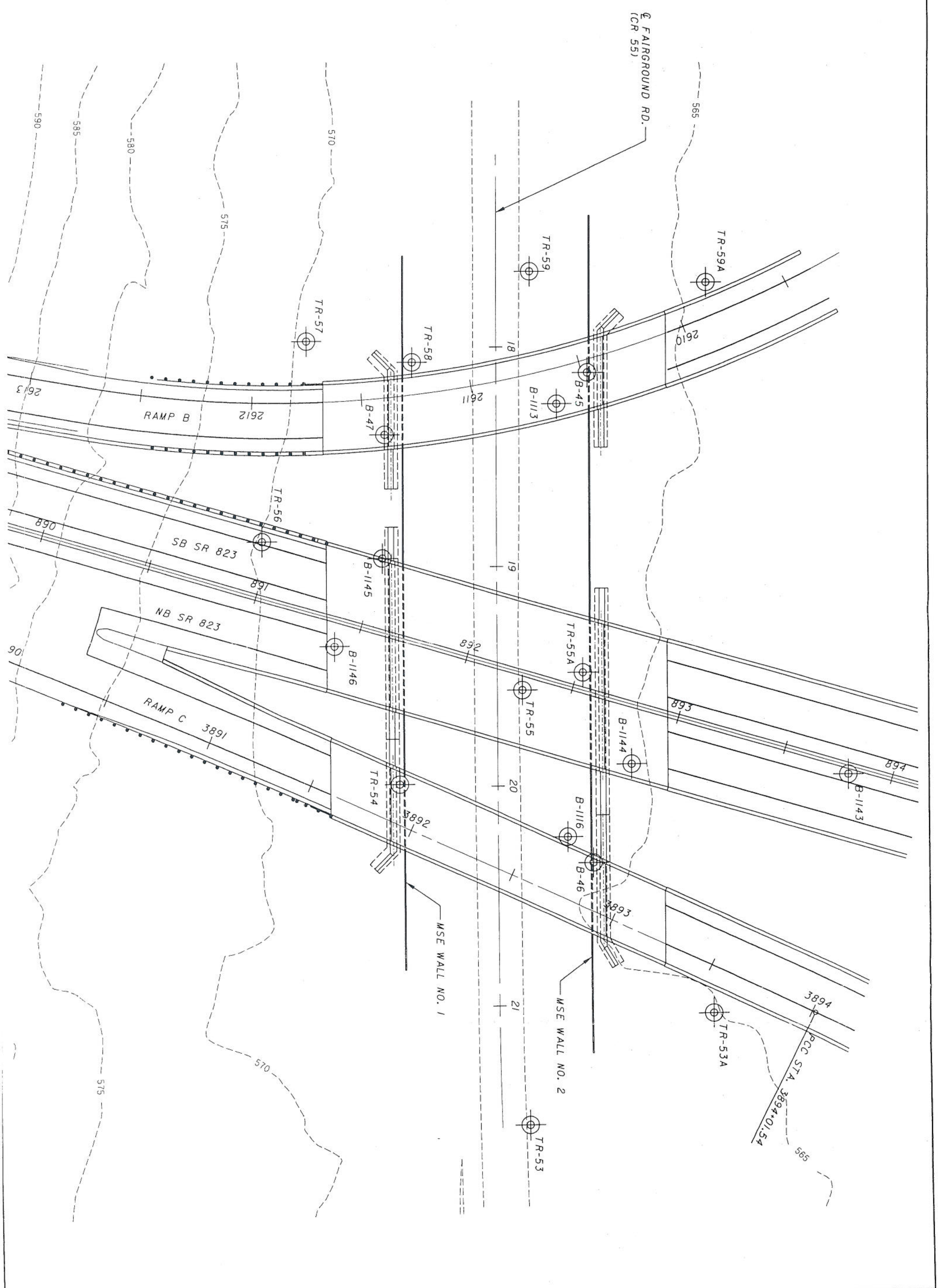

SCI-823-10.13
PID 79977

BORING LOCATION PLAN
US 23 INTERCHANGE - FAIRGROUND ROAD

HORIZONTAL SCALE IN FEET
0 10 20 40

APPENDIX II

Boring Location Plan
General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Nineteen (19) 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.

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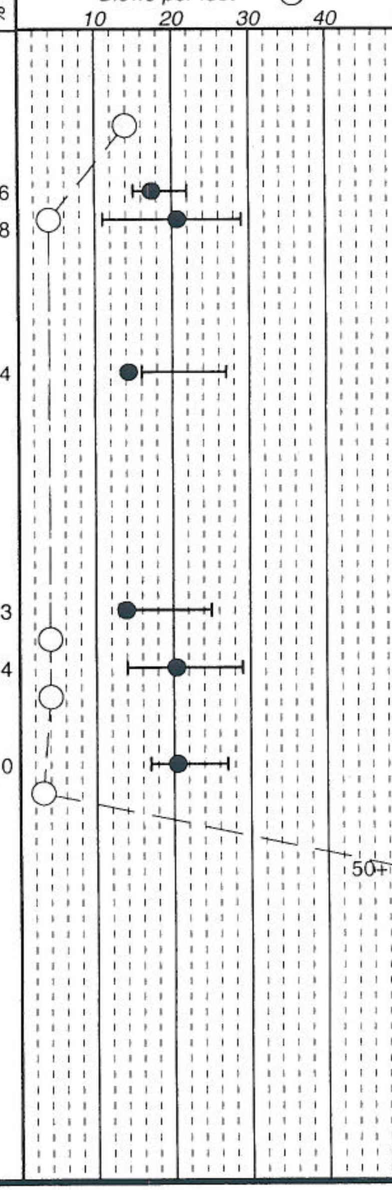
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-45 | Location: Sta. 892+15.3, 139.8 ft LT of SR 823 CL | Date Drilled: 6/14/07

WATER OBSERVATIONS: Water seepage at: 18.5'
Water level at completion: 8.3' (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	566.0												
0.3	565.7						Topsoil with gravel fill - 4"						
		5 7 7	18	1		-	Stiff dark brown SILT (A-4b), little fine sand, trace coarse sand, some clay; damp to moist.						
		3 2 2	15	2		1.0	@ 3.0', brown, trace gravel.	0	5	--	18	51	26
5.0	561.0				P-1	1.25	Stiff brown SILTY CLAY (A-6b), little to some fine sand, trace coarse sand; moist.	0	3	--	20	49	28
8.5	557.5				P-2		Loose to medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6), some silty clay; damp to moist.	30	21	--	18	17	14
					P-3								
15		3 2 2	18	3				13	43	--	23	8	13
		1 2 2	17	4			@ 16.0', moist to wet.	30	26	--	17	13	14
18.0	548.0	WOH 1 2	8	5			Loose brown GRAVEL WITH SAND AND SILT (A-2-4), some silty clay; moist to wet.	47	20	--	7	17	10
21.0	545.0	31 50/3	8	6			Severely weathered to decomposed gray SHALE.						
22.0	544.0	Core 18"	Rec 18"	RQD 33%	R-1		Soft to medium hard gray SHALE; highly weathered, thinly laminated, moderately fractured, with typical low angle clay-filled fractures; contains sandstone beds.						
25		Core 60"	Rec 60"	RQD 92%	R-2		@ 23.8'-24.0', high angle iron stained fracture. @ 27.2', qu=2,651 psi.						
30													

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



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

LOG OF: Boring B-45 | Location: Sta. 892+15.3, 139.8 ft LT of SR 823 CL | Date Drilled: 6/14/07

WATER OBSERVATIONS: Water seepage at: 18.5'
Water level at completion: 8.3' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	536.0	Core 120"	Rec 120"	RQD 79%	R-3		Soft to medium hard gray SHALE; moderately weathered, argillaceous, micaceous, thinly laminated, moderately fractured, with typical low angle clay-filled fractures. @ 31.9'-33.0', 33.5'-34.9', decomposed.							
34.9	531.1						Medium hard blue SHALE; moderately weathered, carbonaceous, thinly laminated, slightly fractured. @ 37.5', qu=3,757 psi.							
40		Core 42"	Rec 42"	RQD 100%	R-4		@ 36.2'-36.8', high angle fracture.							
42.0	524.0						Bottom of Boring - 42.0'							
45														
50														
55														
60														

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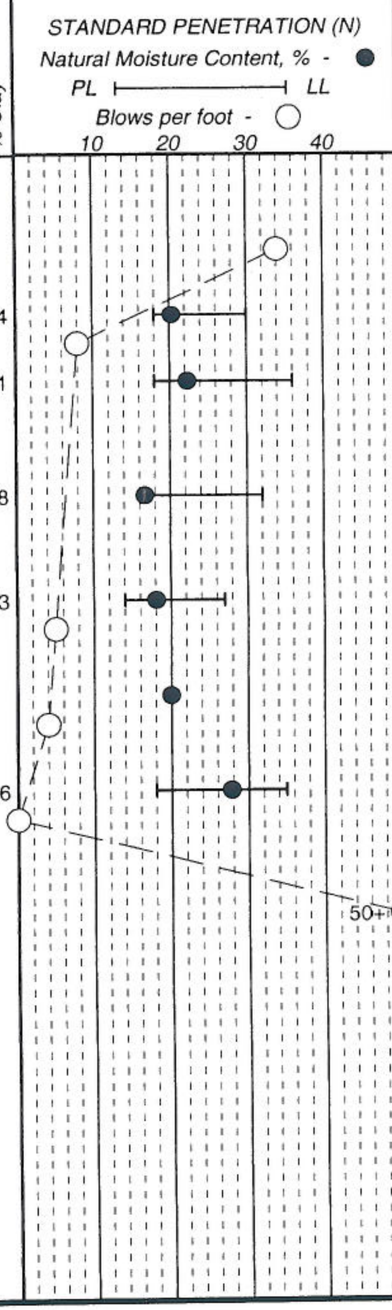
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-46 | Location: Sta. 892+81.1, 73.3 ft RT of SR 823 CL | Date Drilled: 6/15/07

WATER OBSERVATIONS: Water seepage at: 15.5'
Water level at completion: 4.3' (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	565.6												
0.3	565.3						Topsoil-3"						
		6 20 14	11	1		4.5+	Hard brown SILT AND CLAY (A-6a), trace fine to coarse sand; damp to moist.						
		3 3 5	17	2		1.75	@ 3.5'-5.0', stiff.	0	1	--	3	62	34
5.0	560.6				P-1	1.5	Stiff brown SILTY CLAY (A-6b), trace fine to coarse sand; moist.	0	2	--	4	53	41
8.0	557.6				P-2	1.5	Stiff brown SILT AND CLAY (A 6a), some fine to coarse sand, some gravel, damp to moist.	23	19	--	12	28	18
10.5	555.1	2 3 2	18	3			Loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6), little to some silty clay; moist.	17	36	--	27	7	13
		2 2 2	18	4									
15		W O H	18	5			@ 15.5', very loose, wet.	28	47	--	5	4	16
18.0	547.6	19 42 50/4	16	6			Severely weathered to decomposed gray shale.						
20.0	545.6	Core 60"	Rec 57"	RQD 80%	R-1		Soft to medium hard gray SHALE; highly weathered, thinly laminated, moderately fractured, contains occasional thin sandstone beds.						
25							@ 25.2', qu=4,011 psi.						
30		Core	Rec	RQD	R-2								



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

LOG OF: Boring B-46 | Location: Sta. 892+81.1, 73.3 ft RT of SR 823 CL | Date Drilled: 6/15/07

WATER OBSERVATIONS: Water seepage at: 15.5'
Water level at completion: 4.3' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N): Natural Moisture Content, % - ●; PL ⊢—⊣ LL; Blows per foot - ○ (10, 20, 30, 40)
30	535.6	108"	108"	79%			Soft to medium hard gray SHALE; moderately to highly weathered, thinly laminated, moderately fractured; contains calcareous, thin sandstone beds. @ 31.2', highly weathered. @ 33.3', decomposed.							
33.8	531.8	Core 72"	Rec 72"	RQD 97%	R-3		Medium hard black SHALE; slightly weathered, carbonaceous, thinly laminated, slightly fractured to unfractured. @ 35.7', qu=3,030 psi. @ 33.8'-34.0', high angle fracture.							
40.0	525.6						Bottom of Boring - 40.0'							

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

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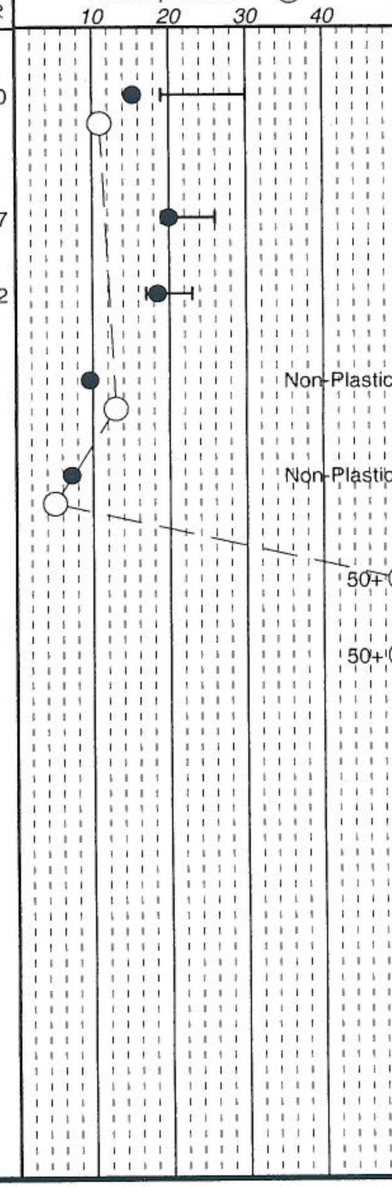
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-47 | Location: Sta. 891+35.2, 86.9 ft LT of SR 823 CL | Date Drilled: 06/18/07 to 06/19/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 8.3' (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	567.5												
0.4	567.1						Topsoil - 5"						
		4 6 5	16	1		4.5+	Hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, dry to damp.	0	1	--	3	56	40
3.5	564.0				P-1	4.5+	Medium stiff brown SILT (A-4b), some clay, trace fine sand; moist.	0	0	--	4	69	27
					P-2	1.25		0	1	--	14	63	22
8.5	559.0	4 6 7	18	2			Medium dense brown SILT (A-4b), trace to little fine to coarse sand; moist.	0	6	--	4	90	
11.0	556.5	3 2 3	18	3			Loose brown SANDY SILT (A-4a), little to some fine to coarse sand, some gravel; moist.	33	9	--	11	47	
13.0	554.5	19 50/5	11	4			Severely weathered light gray SHALE.						
		50/6	6	5									
17.0	550.5	Core 60"	Rec 60"	RQD 95%	R-1		Soft to medium hard gray SHALE; highly weathered to decomposed, thinly laminated, highly fractured, contains occasional thin sandstone beds; sphalerite. @ 20.4', qu=1,971 psi.						
		Core 24"	Rec 24"	RQD 93%	R-2								
		Core 120"	Rec 120"	RQD 71%	R-3		@ 26.8', qu=3,110 psi.						
30													

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



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

LOG OF: Boring B-47 *Location:* Sta. 891+35.2, 86.9 ft LT of SR 823 CL *Date Drilled:* 06/18/07 to 06/19/07

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: 8.3' (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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ 10 20 30 40
30	537.5						Soft to medium hard gray SHALE; highly weathered to decomposed, thinly laminated, highly fractured; contains occasional sandstone beds.							
35		Core 36"	Rec 36"	RQD 51%	R-4		@ 36.7'-36.9', slickenside fracture.							
37.0	530.5						Bottom of Boring - 37.0'							
40														
45														
50														
55														
60														

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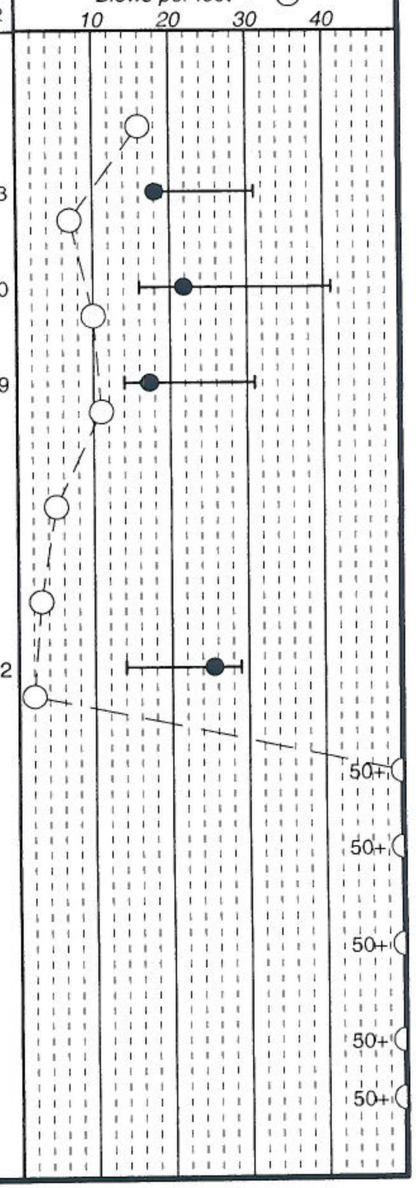
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1113 | Location: Sta. 892+06.0, 122.3 ft LT of SR 823 CL | Date Drilled: 9/28/05

WATER OBSERVATIONS: Water seepage at: 15'
Water level at completion: 29.8' (prior to coring)
18.0' (15 hours after completion)

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	566.8												
0.4	566.4						Topsoil - 5"						
		8 8 8	13	1		3.5	Very stiff dark brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; contains roots; damp.						
		4 3 4	15	2		4.0		1	3	--	11	52	33
5.5	561.3						Very stiff to hard brown and gray CLAY (A-7-6), "and" silt, little fine to coarse sand; moist.						
		3 4 6	16	3		4.0		0	1	--	12	47	40
8.0	558.8						Stiff brown and gray SILTY CLAY (A-6b), "and" fine to coarse sand, little gravel; moist.						
		5 4 7	18	4		1.5		15	15	--	28	23	19
		2 3 2	17	5		1.5							
		2 1 2	16	6		1.0							
15.5	551.3						Very loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.						
		2 1 1	18	7				18	37	--	25	8	12
18.0	548.8						Severely weathered gray SHALE, micaceous.						
		34 50/5	11	8									
		50/5	5	9									
		50/4	4	10									
		50/4	4	11									
		50/4	4	12									
30													

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



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Water level at completion: 29.8' (prior to coring)
18.0' (15 hours after completion)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	536.8						Severely weathered gray SHALE, micaceous.							
		Core 48"	Rec 33"	RQD 0%	R1									
		50/5	5	13										50+
35														
		50/4	4	14										50+
40														
		50/2	2	15										50+
44.0	522.8						Medium hard black SHALE; slightly to moderately weathered, carbonaceous, thinly laminated, slightly fractured. @ 45.1', 47.2', 48.9', decomposed fractures.							
45														
		Core 60"	Rec 56"	RQD 58%	R2									
49.0	517.8						Bottom of Boring - 49.0'							
50														
55														
60														

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

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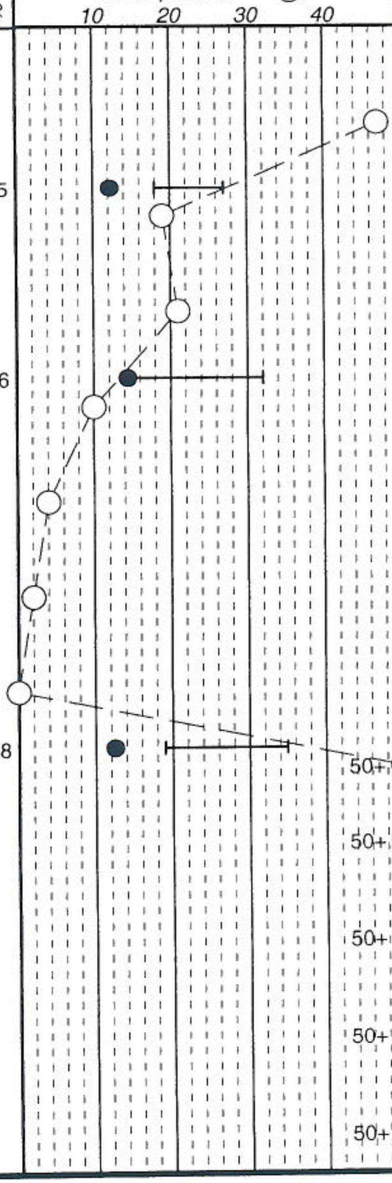
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1116 | Location: Sta. 892+66.6, 65.2 ft RT of SR 823 CL | Date Drilled: 9/27/05

WATER OBSERVATIONS: Water seepage at: 16.0'
Water level at completion: 17.5' (prior to coring)
17.5' (inside hollowstem augers)

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	565.8												
0.3	565.5						Topsoil - 4"						
		18 21 26	9	1			Dense grayish brown SANDY SILT (A-4a), trace clay; possible boulder; dry.						
3.0	562.8	8 9 10	8	2			Medium dense brown SILT (A-4b), some clay, trace fine to coarse sand; damp.	0	2	--	7	66	25
5		6 8 13	13	3									
8.0	557.8	4 4 6	13	4			Loose to medium dense reddish brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); damp to moist.	11	37	--	24	12	16
10		3 2 2	13	5			@ 11.0', moist to wet.						
		2 1 1	13	6			@ 13.5'-17.5', very loose, wet.						
15		1 WOH WOH	0	7									
18.0	547.8	12 50/3	9	8		--	Hard gray SILTY CLAY (A-6b), trace fine to coarse sand, trace gravel; dry to damp.	1	5	--	3	53	38
20.0	545.8	50/4	4	9			Severely weathered gray and black SHALE.						
		50/4	4	10									
25		50/4	4	11									
		50/4	4	12									
30													

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



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Client: TranSystems, Inc.	Project: SCI-823-0.00	Job No. 0121-3070.03
LOG OF: Boring B-1116	Location: Sta. 892+66.6, 65.2 ft RT of SR 823 CL	Date Drilled: 9/27/05

WATER OBSERVATIONS: Water seepage at: 16.0'
Water level at completion: 17.5' (prior to coring)
17.5' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Blows per foot
30	535.8						Severely weathered gray and black SHALE.							
		50/5	5	13										50+
38.0	527.8	Core 120"	Rec 104"	RQD 21%	R1		Medium hard black SHALE; slightly to moderately weathered, laminated, slightly fractured.							
48.0	517.8						Bottom of Boring - 48.0'							

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

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

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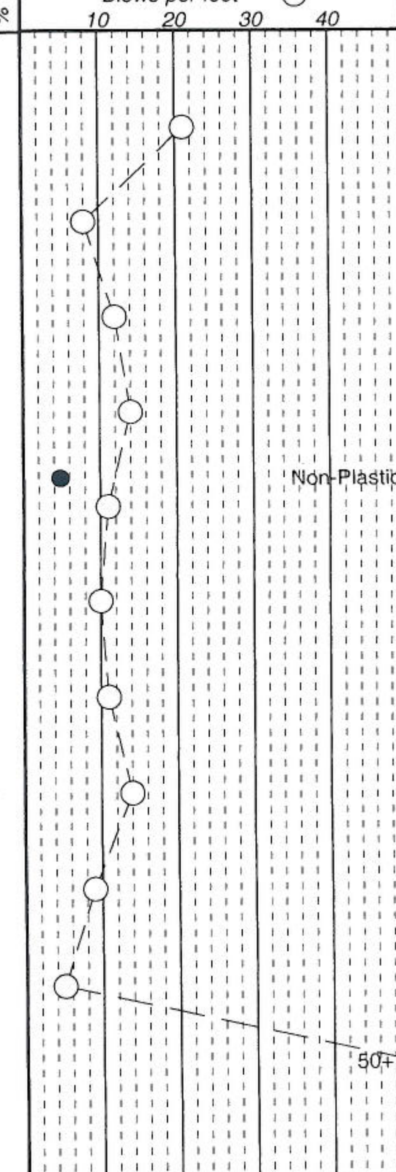
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1143 | Location: Sta. 893+80.3, 3.7 ft. RT of SR 823 CL | Date Drilled: 10/13/05

WATER OBSERVATIONS: Water seepage at: 23.5'-26.3'
Water level at completion: None (prior to coring)
3.5' (inside hollowstem augers)

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
	563.2												
0.2	563.0						Topsoil - 2"						
		7 10 11	18	1		--	FILL: Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp.						
5		4 5 3	14	2		4.5+							
		5 6 6	18	3		3.0							
10		4 5 9	9	4		3.0							
10.5	552.7												
		5 5 6	8	5			POSSIBLE FILL: Loose to medium dense brown GRAVEL WITH SAND (A-1-b), trace to little silt, trace to little clay; dry to damp.	21	44	--	19		16
15		4 5 5	9	6									
		3 4 7	10	7									
20		4 6 8	14	8									
		3 4 5	18	9									
25		3 2 3	3	10			@ 23.5', wet.						
		25 50/4	8	11A 11B									
27.0	536.2						Soft greenish gray SHALE; decomposed, micaceous, thinly laminated, highly fractured. @ 28.8'-29.3', loss of recovery from washed out clay.						
30													

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



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

LOG OF: Boring B-1143 | Location: Sta. 893+80.3, 3.7 ft. RT of SR 823 CL | Date Drilled: 10/13/05

WATER OBSERVATIONS: Water seepage at: 23.5'-26.3'
Water level at completion: None (prior to coring)
3.5' (inside hollowstem augers)

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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	533.2						@ 29.3'-29.4', 29.6'-29.7', high angle fractures.							
30.5	532.7	Core 120"	Rec 120"	RQD 81%	R1		Medium hard black SHALE; moderately weathered, carbonaceous, thinly laminated, moderately fractured. @ 30.5', 31.2', 31.7', 33.6', 34.7', 35.3', 36.7', low angle fractures.							
37.0	526.2						Bottom of Boring - 37.0'							

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

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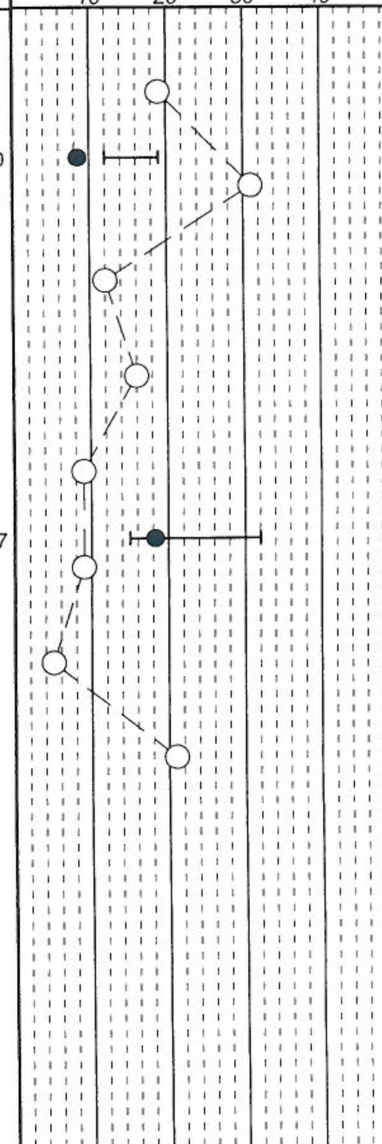
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1144 | Location: Sta. 892+85.3, 25.8 ft. RT of SR 823 CL | Date Drilled: 9/22/05

WATER OBSERVATIONS: Water seepage at: 16.0'
Water level at completion: Not Reported (used mud rotary)

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
	564.9	5 8 11	6	1			FILL: Medium dense gray and brown SANDY SILT (A-4a), little clay, little gravel; contains organic material; dry to damp.						
3.0 / 03.0	562.2 / 562.2	12 14 17	12	2		--	Stiff to very stiff reddish brown SANDY SILT (A-4a), little clay, trace gravel; damp to moist. Topsoil - 3" @ 5.0', began using mud rotary to advance hole.	4	14	--	32	30	20
5		9 11 1	13	3		--							
10		6 7 9	14	4		--							
		5 4 5	12	5		--							
13.0	552.2	3 4 5	8	6			Loose to medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6), little clay; moist to wet.	13	31	--	30	9	17
15		2 3 2	14	7									
20		1 1 20	8	8									
21.0	544.2	Core 60"	Rec 45"	RQD 14%	R1		Soft to medium hard gray SHALE interbedded with SANDSTONE; very fine grained, highly weathered to decomposed, thinly laminated to thinly bedded, moderately fractured. @ 21.0'-21.1', chert layer.						
25 / 26.0	539.2						Bottom of Boring - 26.0'						
30													

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



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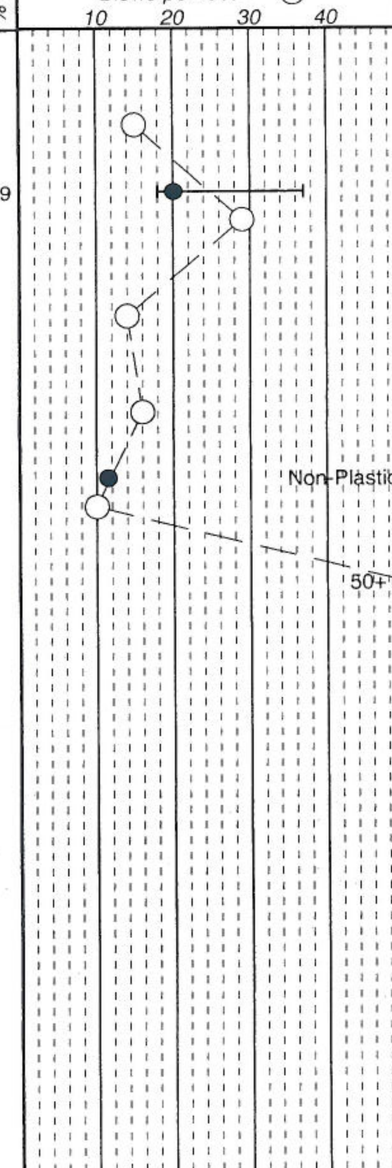
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1145 | Location: Sta. 891+50.3, 32.7 ft. LT of SR 823 CL | Date Drilled: 10/13/05

WATER OBSERVATIONS: Water seepage at: 10.5'-13.0'
Water level at completion: None (prior to coring)
3.5' (inside hollowstem augers)

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
0.2	567.3 / 567.1						Topsoil - 2"						
		5 7 8	18	1		4.5+	Very stiff to hard brown SILTY CLAY (A-6b), trace fine to coarse sand, trace gravel; moist.						
		9 13 16	18	2		4.0		0	1	--	2	58	39
		3 5 9	18	3		2.0							
		4 7 9	18	4		2.5	@ 8.5', some fine to coarse sand.						
10.5	556.8	4 3 7	18	5			Medium dense dark brown GRAVEL WITH SAND (A-1-b), trace clay, trace silt; moist to wet.	4	47	--	33	16	
		8 50/4	10	6									
14.5	552.8						Soft gray SHALE; highly weathered, micaceous, medium bedded, highly fractured. @ 14.5', 14.7', 15.6', 16.1', 16.5', 17.1', 17.2', 18.0', low angle fractures. @ 14.9'-15.2', loss of recovery.						
19.0	548.3	Core 120"	Rec 116"	RQD 87%	R1		Medium hard gray SANDSTONE; very fine grained, moderately weathered, argillaceous, micaceous, laminated to thinly bedded, moderately fractured, contains abundant argillaceous laminations. @ 21.2', 21.3', 21.7', 22.2', low angle fractures.						
24.5	542.8						Bottom of Boring - 24.5'						

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



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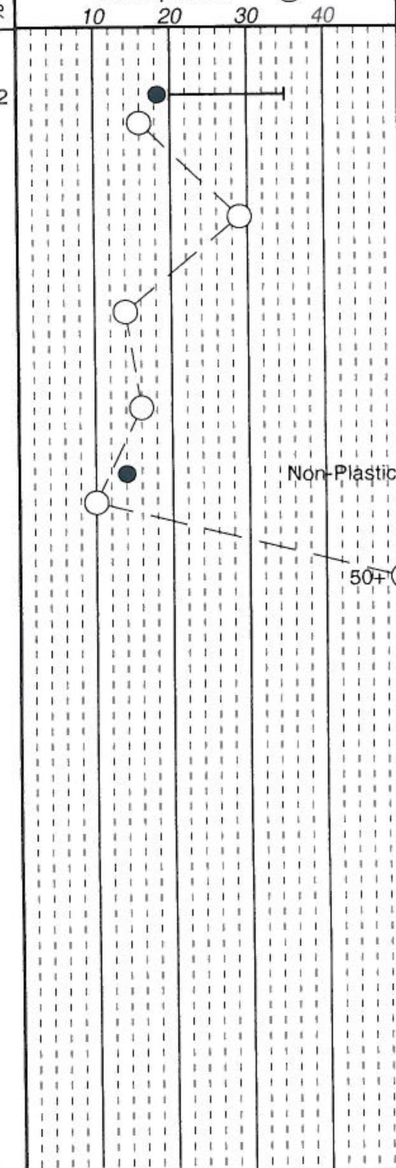
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-1146 | *Location:* Sta. 891+40.7, 12.2 ft. RT of SR 823 CL | *Date Drilled:* 10/13/05

WATER OBSERVATIONS: Water seepage at: 10.5'-13.0'
Water level at completion: None (prior to coring)
3.3' (inside hollowstem augers)

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
0.2	567.7 / 567.5						Topsoil - 2"						
		6 7 9	18	1		4.5+	Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp to moist.	0	3	--	5	50	42
		7 12 17	18	2		4.5+							
		2 6 8	18	3		3.0							
		5 8 8	18	4		1.75	@ 8.5', stiff, contains sand seam.						
10.5	557.2	2 4 6	14	5			Medium dense dark brown GRAVEL WITH SAND (A-1-b), trace clay, trace silt; moist.	30	29	--	25	16	
13.0	554.7	35 50/2	8	6			Severely weathered gray SHALE.						
14.5	553.2						Soft brownish gray SHALE; highly weathered to decomposed, micaceous, medium bedded, highly fractured, many broken areas from low angle fractures. @ 14.8'-14.9', high angle fracture.						
19.0	548.7	Core 120"	Rec 120"	RQD 77%	R1		Medium hard gray SANDSTONE; very fine grained, moderately to highly weathered, micaceous, laminated to thinly bedded, moderately fractured, contains abundant argillaceous laminations. @ 21.3', 22.4', low angle fractures.						
24.5	543.2						Bottom of Boring - 24.5'						

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



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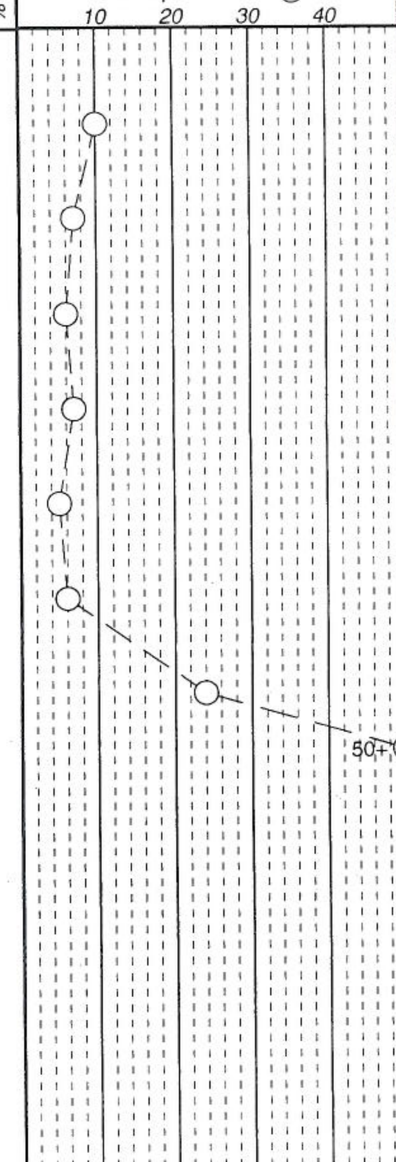
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-53 | Location: Sta. 892+87.2, 196.6 ft. RT of SR 823 CL | Date Drilled: 7/8/04

WATER OBSERVATIONS: Water seepage at: 7.0', 8.5', 11.0', 15.5'
Water level at completion: 17.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	566.7												
0.2	566.5						Topsoil - 2"						
		9 6 4	18	1		4.0	Stiff to very stiff brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; damp to moist.						
		2 3 4	16	2		1.25							
		2 2 4	16	3		1.75	@ 6.0'-7.5', contains interbedded sand seams.						
8.5	558.2	6 4 3	17	4			Loose to medium dense brown COARSE AND FINE SAND (A-3a), little to some silty clay, trace gravel; moist to wet.						
		3 2 3	15	5									
		3 3 3	18	6									
		2 4 20	14	7			@ 16.0', wet.						
17.5	549.2						Severely weathered gray SILTSTONE.						
		50/5	5	8									
19.0	547.7	Core 12"	Rec 11"	RQD 92%	R-1		Soft, gray, SANDSTONE; decomposed.						
20.0	546.7	Core 108"	Rec 104"	RQD 67%	R-2		Medium hard to hard gray SANDSTONE; very fine to fine grained, argillaceous, micaceous, laminated to thinly bedded, moderately to highly fractured. @ 22.0'-22.3',22.6'-22.8',23.2'- 23.3', 24.4'-24.6', 26.3'-26.5', vertical fractures. @ 22.0'-22.2',23.0'-23.1',26.3'- 26.5',28.6'-28.8', ferric bands.						
29.0	537.7						Bottom of Boring - 29.0'						

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



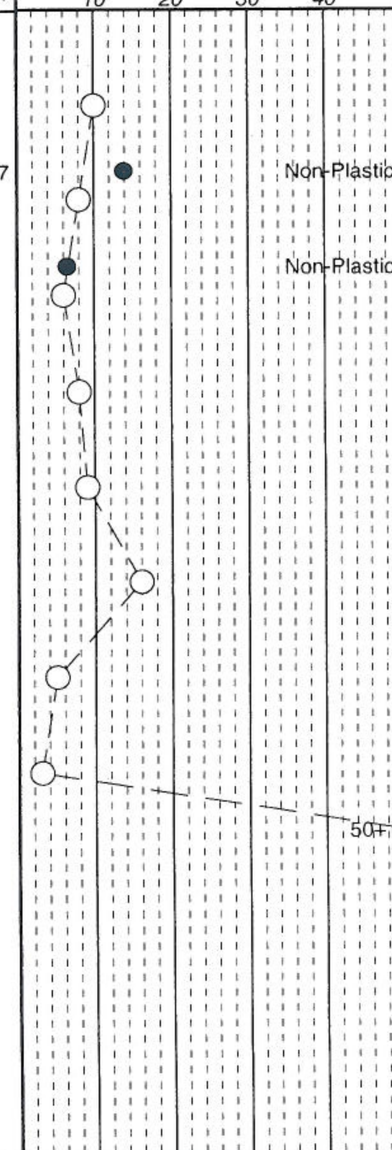
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-53A | *Location:* Sta. 893+53.1, 124.5 ft. RT of SR 823 CL | *Date Drilled:* 3/15/05

WATER OBSERVATIONS: Water seepage at: 18.5'
Water level at completion: None (Prior to coring)
11.0' (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	565.3						Topsoil - 7"						
0.6	564.7	5 5 5	4	1		--	Stiff gray SILTY CLAY (A-6b), trace gravel; contains shale fragments; damp.						
3.0	562.3	3 4 4	11	2		4.5+	Hard brown SANDY SILT (A-4a), some gravel, little clay; damp.	28	16	--	15	24	17
5.5	559.8	5 3 3	12	3			Loose brown COARSE AND FINE SAND (A-3a), some gravel, some silty clay; dry to damp.	21	20	--	36	23	
		3 3 5	13	4									
		4 4 5	12	5									
		4 7 9	10	6			@ 13.5'-15.0', medium dense.						
		5 3 2	8	7									
18.0	547.3	1 2 1	8	8			Very loose dark brown GRAVEL WITH SAND (A-1-b), trace clay; wet.						
20.5	544.8	50/4	4	9			Severely weathered gray SHALE.						
22.5	542.8	Core 120"	Rec 120"	RQD 78%	R-1		Medium hard gray SHALE; moderately weathered, thinly laminated, arenaceous, slightly fractured, contains ferric sandstone bands, fissile after desiccation. @ 22.5'-28.0', highly fractured. @ 28.8'-28.9', high angle fracture. @ 23.5',27.8',31.3', clay seams. @ 29.2'-30.0', very fine sandstone.						
30													

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-53A | Location: Sta. 893+53.1, 124.5 ft. RT of SR 823 CL | Date Drilled: 3/15/05

WATER OBSERVATIONS: Water seepage at: 18.5'
Water level at completion: None (Prior to coring)
11.0' (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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	535.3						Medium hard gray SHALE; moderately weathered, thinly laminated, arenaceous, slightly fractured, contains ferric sandstone bands, fissile after desiccation.							
32.5	532.8						Bottom of Boring - 32.5'							
35														
40														
45														
50														
55														
60														

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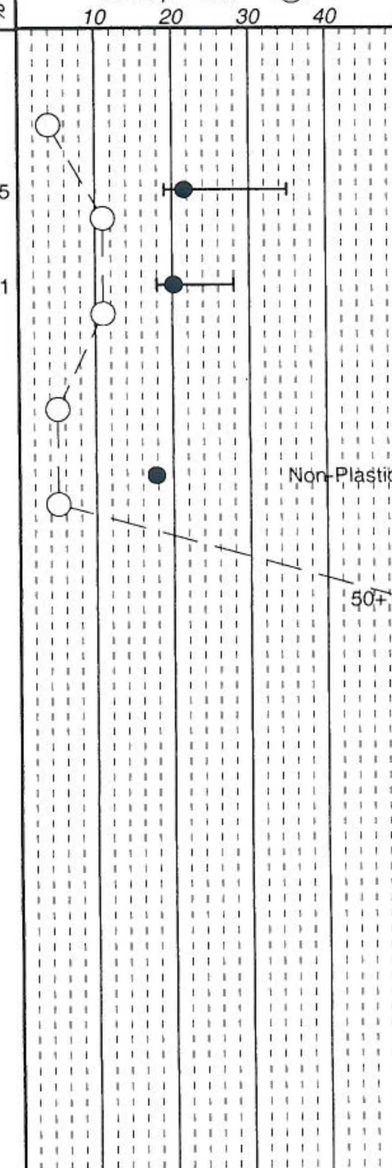
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-54 | Location: Sta. 893+86.9, 63.8 ft. RT of SR 823 CL | Date Drilled: 3/16/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
11.0' (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
	566.9												
0.2	566.7						Topsoil - 3"						
		2 2 2	14	1		1.0	Stiff to very stiff brown SILTY CLAY (A-6b), trace fine sand; damp. @ 0.0'-2.5', contains roots.						
		2 5 6	17	2		3.5		0	0	--	4	61	35
5.5	561.4	3 5 6	18	3		2.25	Very stiff brown SILT (A-4b), some clay, little fine sand; damp.	0	0	--	12	67	21
8.0	558.9	1 3 2	11	4			Loose dark brown COARSE AND FINE SAND (A-3a), trace to little clay, trace gravel; damp.						
		1 2 3	13	5				7	38	--	37	18	
13.6	553.3	7 35 50/4	14	6			Severely weathered gray SHALE.						
15.0	551.9	Core 120"	Rec 120"	RQD 83%	R-1		Medium hard gray SHALE; arenaceous, decomposed to highly weathered, laminated, moderately fractured. @ 15.0'-17.3', broken with high angles fractures and thin clay seams. @ 18.9'-19.0', 20.6'-20.9', high angle fractures.						
22.6	544.3						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, medium bedded, slightly fractured.						
23.5	543.4						Hard gray SHALE; highly weathered, arenaceous, very thinly bedded, slightly fractured.						
25.0	541.9						Bottom of Boring - 25.0'						

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



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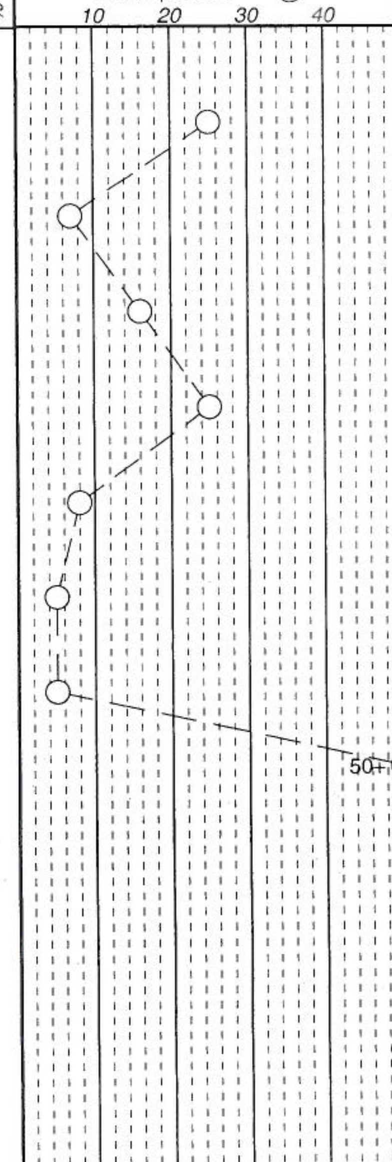
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-55 | Location: Sta. 892+28.2, 7.5 ft. RT of SR 823 CL | Date Drilled: 7/8/04

WATER OBSERVATIONS: Water seepage at: 13.5', 16.0'
Water level at completion: None (prior to coring)
16.0' (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
0.3	567.1 / 566.8						Topsoil - 3"						
		20 14 11	15	1			FILL: Medium dense brown SANDY SILT (A-4a), little gravel; dry.						
3.0	564.1	5 3 4	13	2		2.0	Very stiff to hard brown SILT AND CLAY (A-6a), some fine to coarse sand, trace to little gravel; damp to moist.						
5		5 7 9	16	3		4.25							
10		10 12 13	18	4		4.5+							
11.0	556.1	3 4 4	17	5			Loose brown COARSE AND FINE SAND (A-3a), little to some silty clay, trace gravel; moist to wet.						
15		2 2 3	13	6									
16.0	551.1	1 2 3	14	7		<0.25	Very soft brownish gray SILT AND CLAY (A-6a), some fine to coarse sand, some gravel; contains rock fragments; wet.						
18.5	548.6	30 50/5	11	8			Severely weathered gray SILTSTONE.						
19.5 / 20 / 25	547.6	Core 120"	Rec 120"	RQD 94%	R-1		Medium hard to hard gray SANDSTONE; slightly weathered, argillaceous, micaceous, moderately to highly fractured, contains abundant argillaceous laminations. @ 22.8'-23.1',23.6'- 23.7',25.6' ferric sandstone seams.						
29.5	537.6												

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-55 | Location: Sta. 892+28.2, 7.5 ft. RT of SR 823 CL | Date Drilled: 7/8/04

WATER OBSERVATIONS:
Water seepage at: 13.5', 16.0'
Water level at completion: None (prior to coring)
16.0' (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	GRADATION % Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢—⊣ LL Blows per foot - ○ 10 20 30 40
30	537.1						Bottom of Boring - 29.5'							
35														
40														
45														
50														
55														
60														

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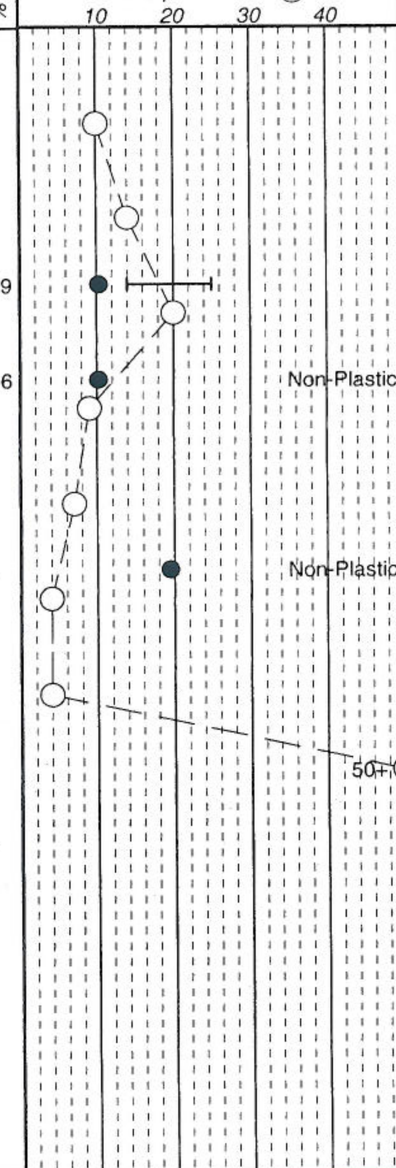
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-55A | *Location:* Sta. 892+52.3, 7.8 ft. LT of SR 823 CL | *Date Drilled:* 3/15/05

WATER OBSERVATIONS: Water seepage at: 13.5'-18.0'
Water level at completion: 18.0' (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	565.4						No Topsoil Hard gray SILTY CLAY (A-6b); damp.						
		3 5 5	10	1		4.5+							
3.0	562.4						Hard brown SILT AND CLAY (A-6a), "and" fine to coarse sand, some gravel; damp.						
5		6 7 7	9	2		4.5+							
		11 11 9	12	3		4.5+		22	15	--	23	21	19
8.0	557.4						Loose brown COARSE AND FINE SAND (A-3a), trace to little clay, trace gravel; damp.						
10		4 5 4	14	4				4	25	--	39	16	16
		5 4 3	12	5									
		2 2 2	15	6			@ 13.5', becomes wet.	9	38	--	40	13	
15		1 2 2	7	7									
18.0	547.4						Severely weathered gray SHALE.						
		35 50/5	11	8									
20.0	545.4						Medium hard gray SHALE interbedded with SANDSTONE; highly weathered, very thinly bedded, highly fractured. @ 20.0'-22.0', 26.7'-27.5', 28.3'-28.5', 29.3'-29.6', highly fractured with clay seams. @ 21.0'-21.3', 21.7'-21.9', 26.5'-26.7', 26.9'-22.0', hard brown ferric sandstone, slightly weathered, laminated.						
25		Core 120"	Rec 120"	RQD 64%	R-1								
30.0	535.4						Bottom of Boring - 30.0'						

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-55A | Location: Sta. 892+52.3, 7.8 ft. LT of SR 823 CL | Date Drilled: 3/15/05

WATER OBSERVATIONS: Water seepage at: 13.5'-18.0'
Water level at completion: 18.0' (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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	535.4													
35														
40														
45														
50														
55														
60														

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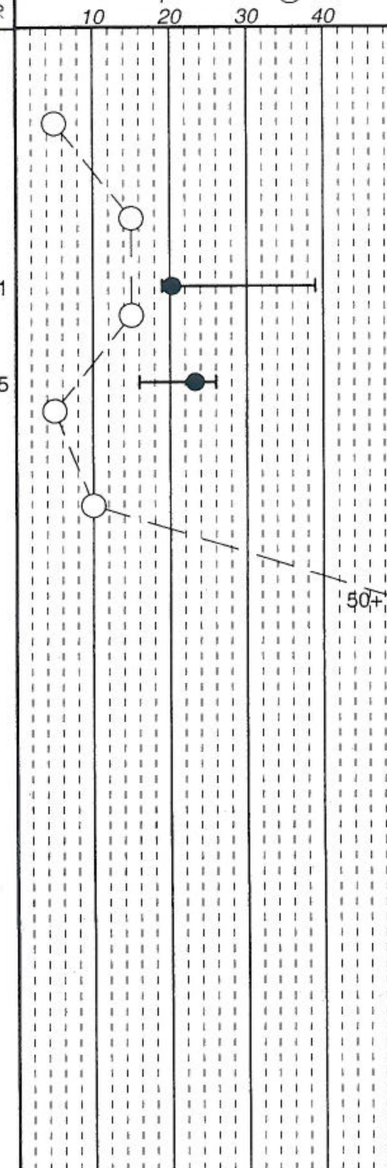
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-56 | *Location:* Sta. 890+95.4, 24.6 ft. LT of SR 823 CL | *Date Drilled:* 3/16/05

WATER OBSERVATIONS: Water seepage at: None
Water level at completion: None (prior to coring)
7.5' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.2	570.0 / 569.8						Topsoil -3"						
		2 2 3	15	1		2.5	Very stiff to hard brown SILTY CLAY (A-6b), trace fine sand; damp.						
5		4 6 9	17	2		4.5+							
		4 6 9	16	3		4.25		0	0	--	2	57	41
8.0	562.0	2 2 3	18	4			Loose brown and gray SILT (A-4b), some fine to coarse sand, some clay; damp to moist.	0	1	--	19	55	25
10		2 6 4	9	5									
14.1	555.9	8 23 50/4	15	6			Severely weathered grayish brown SILTSTONE.						
15.0	555.0	Core 120"	Rec 120"	RQD 68%	R-1		Medium hard grayish brown SANDSTONE interbedded with SHALE; very fine to fine grained, slightly weathered, argillaceous, laminated to thinly bedded, highly fractured. @ 16.4'-17.2', high angle fracture and clay seam. @ 17.2', gray. @ 19.2'-19.7', clay seam. @ 20.4'-20.8', broken, clay seam.						
20													
25.0	545.0						Bottom of Boring - 25.0'						
30													

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



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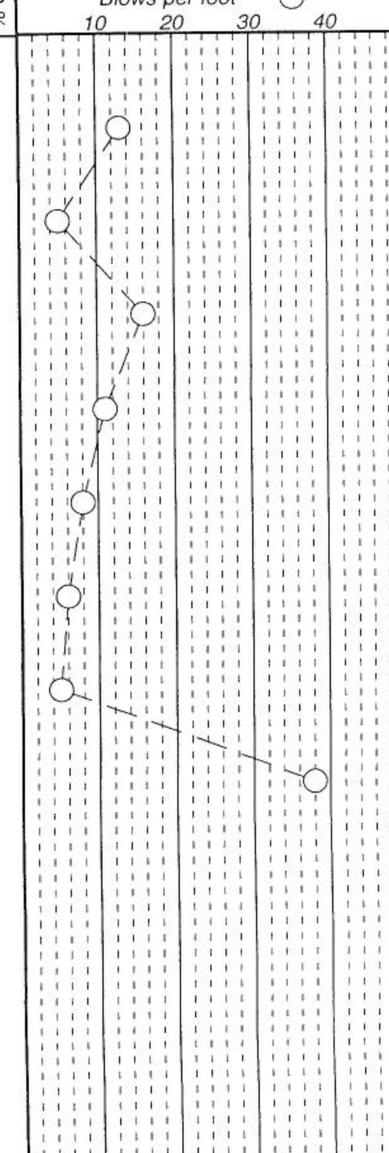
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-59 | Location: Sta. 891+76.9, 176.6 ft. LT of SR 823 CL | Date Drilled: 7/8/04

WATER OBSERVATIONS: Water seepage at: 13.5'-18.5'
Water level at completion: 17.0' (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
0.2	567.3 / 567.1						Topsoil - 2"						
		8 8 5	14	1		--	FILL: Stiff brown SANDY SILT (A-4a), little gravel, trace clay; damp.						
3.0	564.3	2 2 3	12	2		1.0	Stiff to very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; moist.						
		5 7 9	18	3		2.5							
		5 5 6	16	4		2.0							
11.0	556.3	3 4 4	15	5		1.25	Stiff brown SANDY SILT (A-4a), "and" fine to coarse sand, trace gravel; contains thin seam of organic material; moist.						
13.5	553.8	3 3 3	14	6			Loose brown COARSE AND FINE SAND (A-3a), some silty clay, trace gravel; wet.						
		2 2 3	15	7									
18.5	548.8	5 8 30	16	8			Severely weathered gray and brown SANDSTONE fragments, argillaceous.						
21.0	546.3	Core 120"	Rec 115"	RQD 90%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, thinly laminated to thinly bedded, broken to highly fractured, contains moderate argillaceous laminations and few ferric bands. @ 21.0'-21.2',22.0'- 22.2',23.3'-23.4', 23.8'- 23.9', 25.9'-26.2', vertical fractures. @ 26.0'-26.2', argillaceous sandstone.						

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



FILE: 0121-3070-03 [9/4/2007 3:41 AM]

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

LOG OF: Boring TR-59 | Location: Sta. 891+76.9, 176.6 ft. LT of SR 823 CL | Date Drilled: 7/8/04

WATER OBSERVATIONS: Water seepage at: 13.5'-18.5'
Water level at completion: 17.0' (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	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├──┤ LL Blows per foot - ○ 10 20 30 40
30	537.3						Medium hard gray SANDSTONE.							
31.0	536.3						Bottom of Boring - 31.0'							

35 40 45 50 55 60

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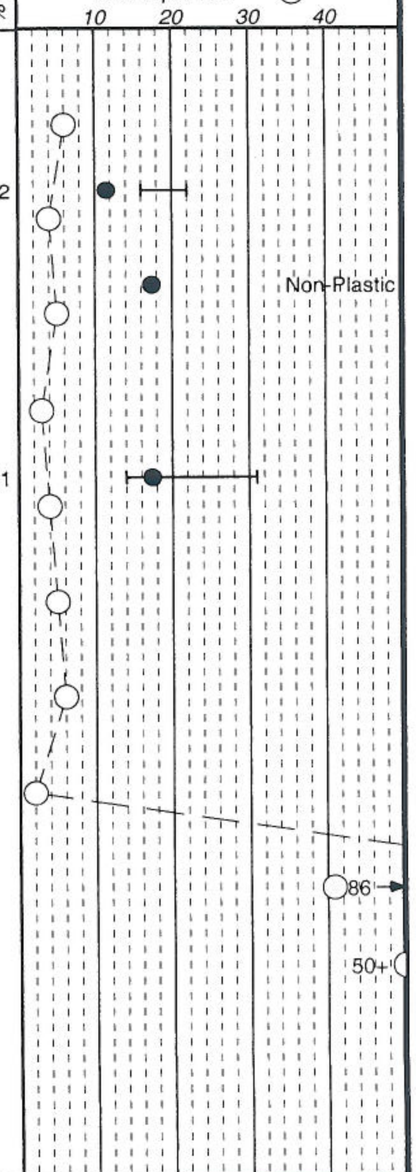
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-59A | Location: Sta. 892+55.2, 194.2 ft. LT of SR 823 CL | Date Drilled: 3/14/05

WATER OBSERVATIONS: Water seepage at: 19.0'-21.5'
Water level at completion: None (prior to coring)
17.0' (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	563.9												
0.3	563.6						Topsoil - 3"						
		3 3 3	14	1		--	Medium stiff dark gray SANDY SILT (A-4a), some clay, trace gravel; damp to moist.						
		2 2 2	12	2		--	@ 3.5', brown.	7	13	--	26	32	22
5													
5.5	558.4	2 2 3	15	3			Very loose to loose brown GRAVEL WITH SAND (A-1-b), little clay; moist.	15	36	--	37	12	
		2 2 1	13	4									
10													
10.5	553.4	2 2 2	16	5			Loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); damp to moist.	14	25	--	31	9	21
		1 2 3	15	6									
15													
		2 3 3	12	7									
		0 1 1	14	8			@ 19.0'-21.5', very loose; wet.						
20													
21.5	542.4	1 36 50	12	9			Severely weathered gray SHALE.						
		32 50/3	9	10									
25.0	538.9	Core 120"	Rec 119"	RQD 65%	R-1		Medium hard to hard gray SANDSTONE interbedded with SHALE; very fine to fine grained, highly weathered to decomposed, laminated to thinly bedded, slightly fractured. @ 25.4'-25.7', 28.5', 29.6', clay seams. @ 25.9', 26.5-26.7', 27.8', high angle fractures. @ 28.6'-29.6', moderately weathered SHALE.						
30													

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-59A | Location: Sta. 892+55.2, 194.2 ft. LT of SR 823 CL | Date Drilled: 3/14/05

WATER OBSERVATIONS: Water seepage at: 19.0'-21.5'
Water level at completion: None (prior to coring)
17.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ (10 20 30 40)
30	533.9						Medium hard to hard gray SANDSTONE interbedded with SHALE; very fine to fine grained, highly weathered to decomposed, laminated to thinly bedded, slightly fractured. @ 31.4'-31.7', clay seams with high angle fractures.							
33.0	530.9						Hard black SHALE; carbonaceous, moderately weathered, laminated, slightly fractured. @ 33.8'-34.0', broken, high angle fracture.							
35.0	528.9						Bottom of Boring - 35.0'							
40														
45														
50														
55														
60														

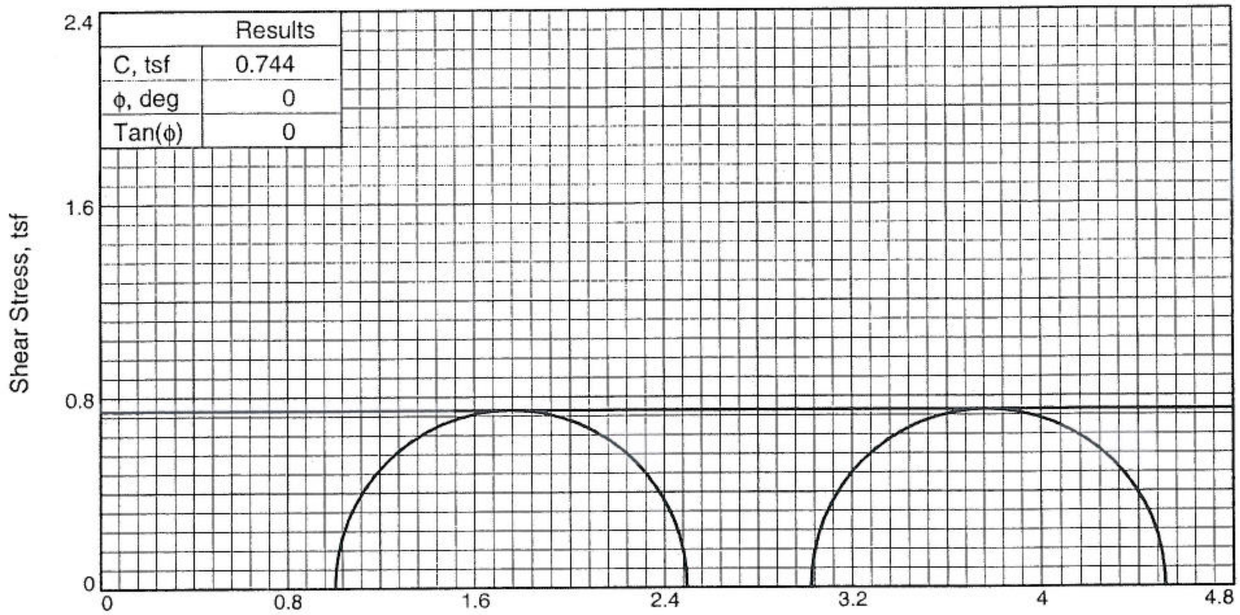
APPENDIX III

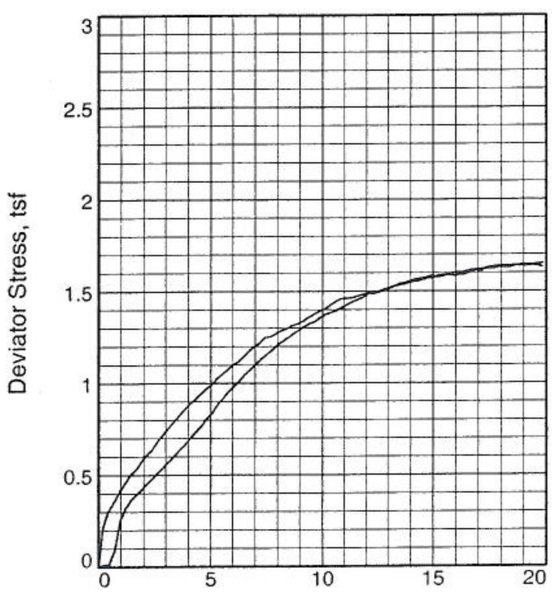
Summary of Strength and Consolidation Test Results
Strength and Consolidation Test Results

PROJECT SCI-823-0.00
Fairgrounds Road Bridge and MSE Wall Structure Borings
SUMMARY OF IN-SITU AND LABORATORY TESTING

Boring	Sample	Depth (ft.)	Test Performed	Results											
				ODOT Classification	γ_D (pcf)	WC (%)	e_o	Cc	Cr	p_c (tsf)	c (psf)	c' (psf)	φ (deg)	φ' (deg)	q_u (tsf)
B-45	P-1	5.0	UU	A-6b	103.5	22.1					1488				
B-45	P-1	5.0	CONS	A-6b	105.2	20.0	0.632	0.090	0.010	0.300					
B-45	In-situ	6.0	FVS TEST	A-6b							1116*				
B-45	P-2	8.0	CIU	A-6b/A-2-6	114.1	18.0					1490	720	6.9	24.5	
B-46	P-1	5.0	UU	A-6b	99.7	23.6					3036				
B-46	P-1	5.0	CONS	A-6b	100.0	23.1	0.692	0.240	0.040	1.900					
B-46	P-2	8.0	CIU	A-6b	108.2	18.9					256	0	23.8	35.8	
B-47	ST-1	4.0	UU	A-4b	97.6	22.7					2238				
B-47	In-situ	6.0	FVS TEST	A-4b							1306*				
B-47	ST-2	6.0	UU	A-4b	102.7	18.4					3616				

* Raw field data, values used for geotechnical analyses require the application of the appropriate correction factor.





Sample No.		1	2
Initial	Water Content,	21.6	21.6
	Dry Density, pcf	103.5	102.3
	Saturation,	89.8	87.1
	Void Ratio	0.6646	0.6850
	Diameter, in.	2.79	2.82
	Height, in.	5.58	5.58
At Test	Water Content,	22.1	22.1
	Dry Density, pcf	103.5	102.3
	Saturation,	91.6	88.9
	Void Ratio	0.6646	0.6850
	Diameter, in.	2.79	2.82
	Height, in.	5.58	5.58
Strain rate, in./min.		0.06	0.06
Back Pressure, tsf		0.00	0.00
Cell Pressure, tsf		1.01	3.02
Fail. Stress, tsf		1.49	1.49
Ult. Stress, tsf		1.65	1.63
σ_1 Failure, tsf		2.50	4.51
σ_3 Failure, tsf		1.01	3.02

Type of Test:
Unconsolidated Undrained

Sample Type: 3" press tube

Description:

Assumed Specific Gravity= 2.76

Remarks:

Figure _______

Client: TranSystems, Inc.

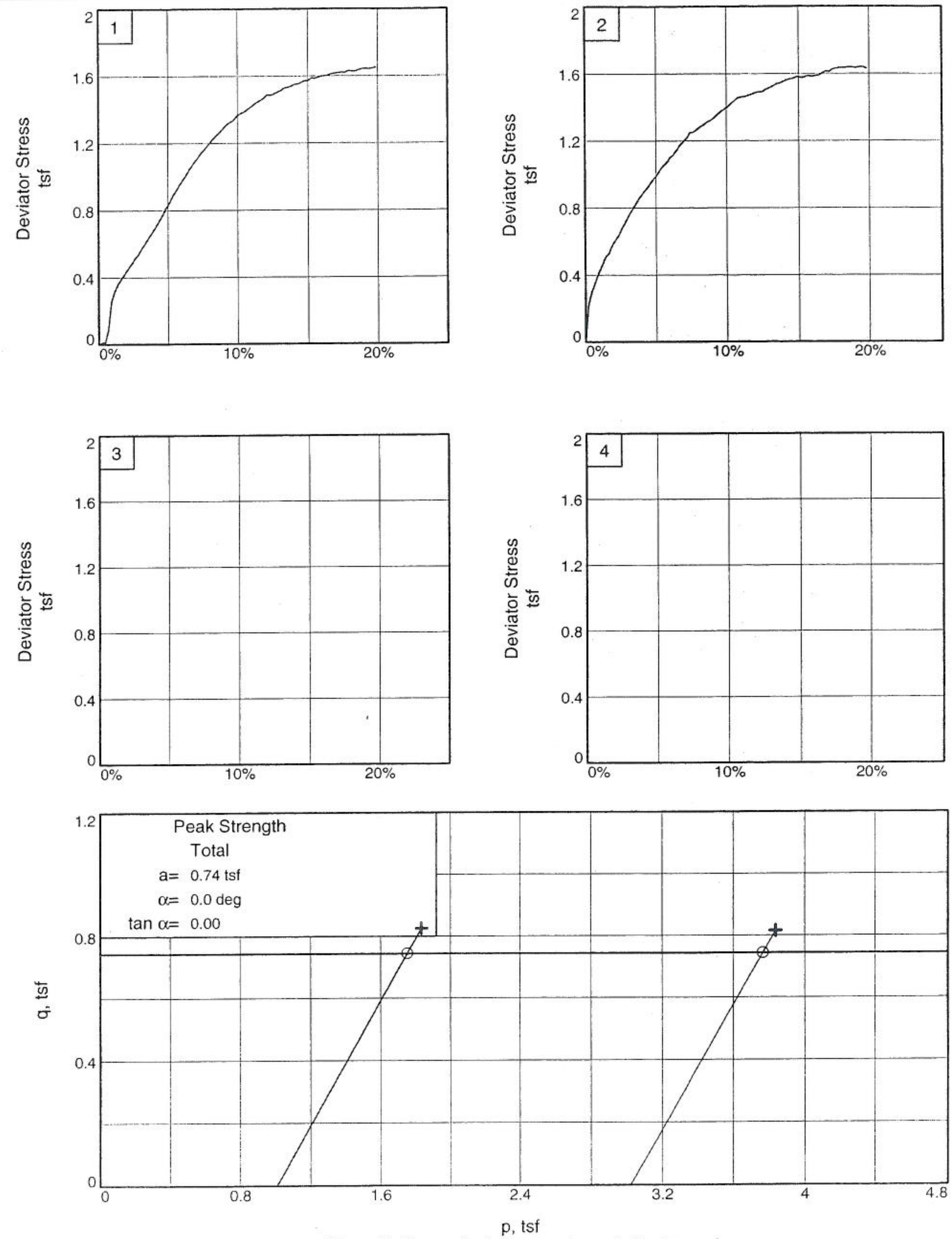
Project: SCI-823-0.00

Source of Sample: B-45 **Depth:** 5.0

Sample Number: P-1

Proj. No.: 0121-3070.03 **Date:**

DLZ



Stress Paths: o indicates peak + indicates end

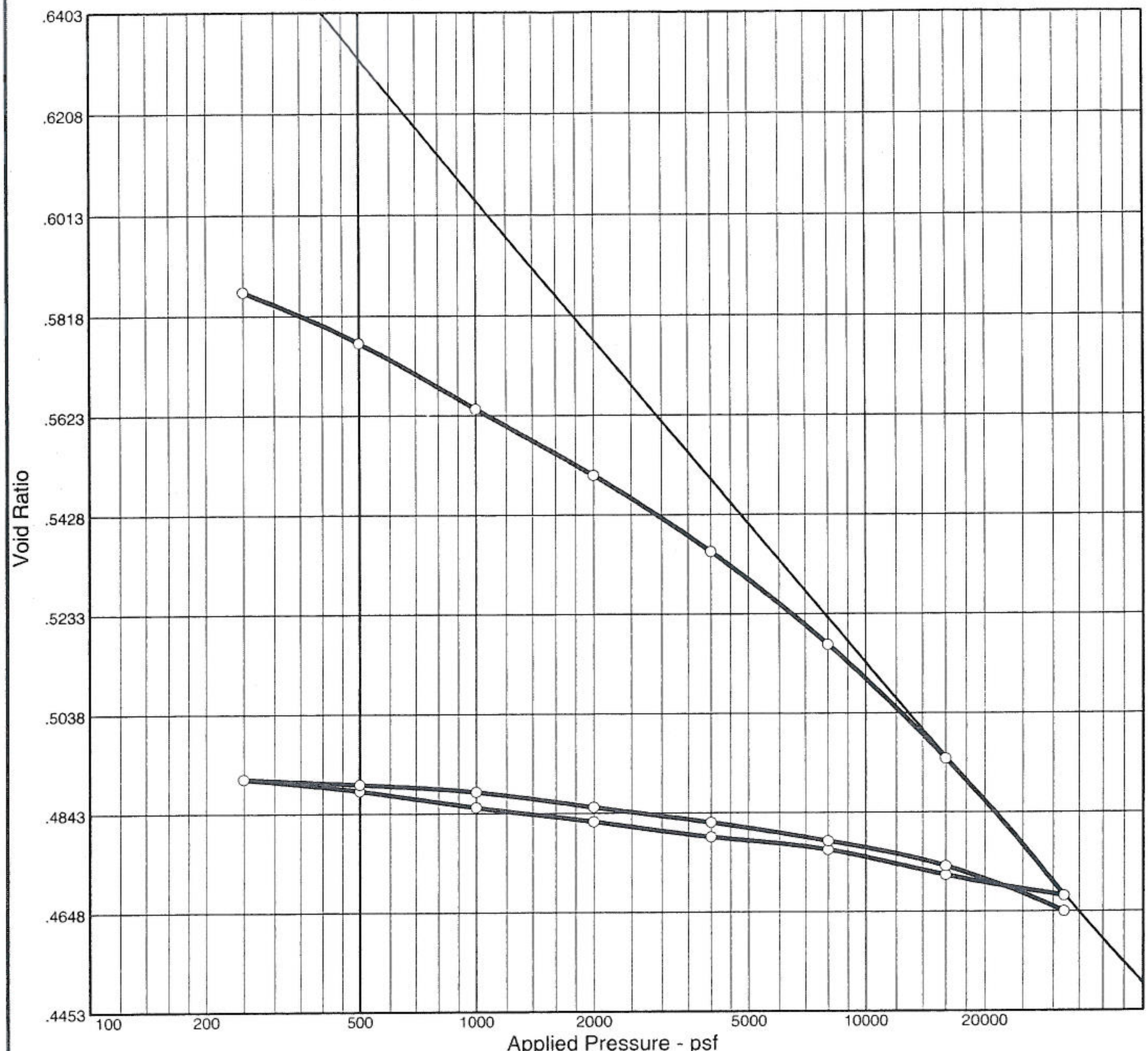
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-45 **Depth:** 5.0 **Sample Number:** P-1

Project No.: 0121-3070.03 **Figure** ________ **DLZ, INC.**

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
87.0 %	20.0 %	105.2	29	18	2.75	CL	A-6(11)	0.632

MATERIAL DESCRIPTION

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

Project: SCI-823-0.00

Source: B-45 **Sample No.:** P-1 **Elev./Depth:** 5.0

DLZ

Remarks:

Figure

Dial Reading vs. Time

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

Source: B-45 Sample No.: P-1 Elev./Depth: 5.0

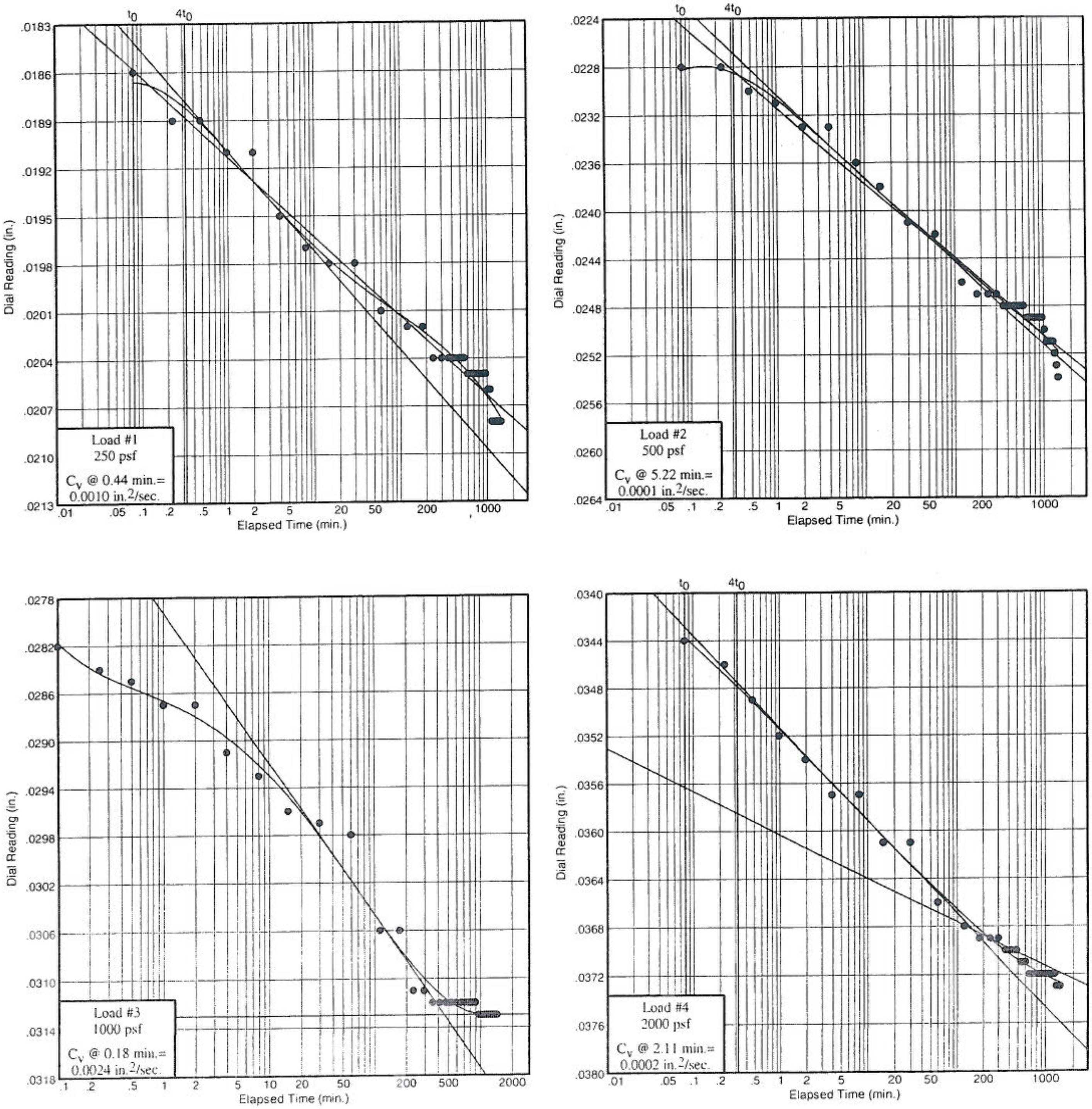


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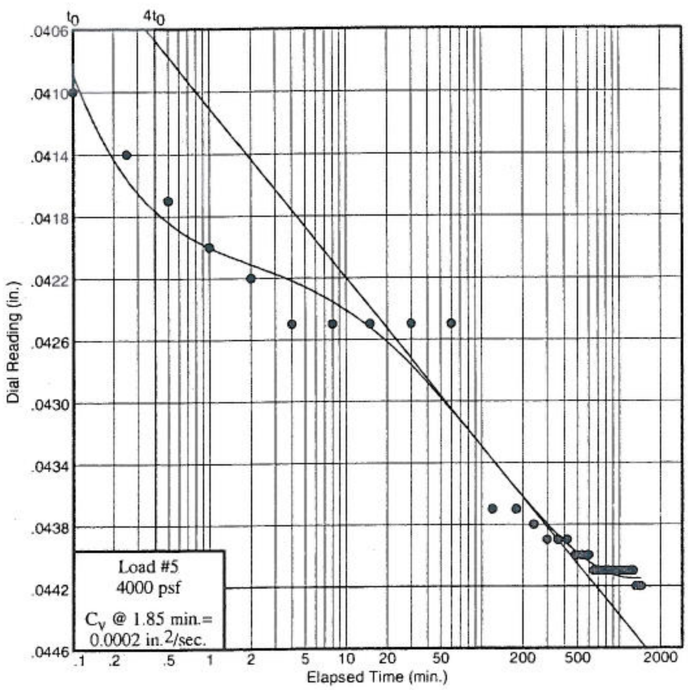
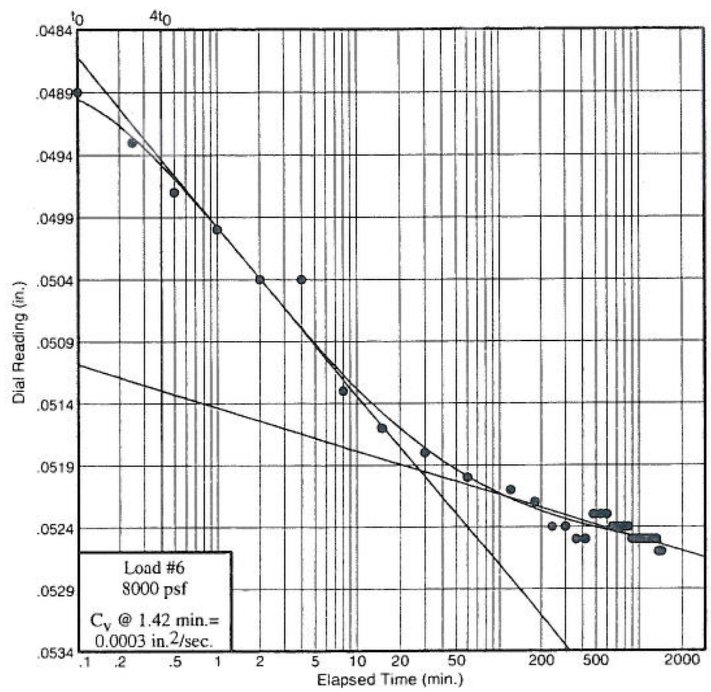
Figure

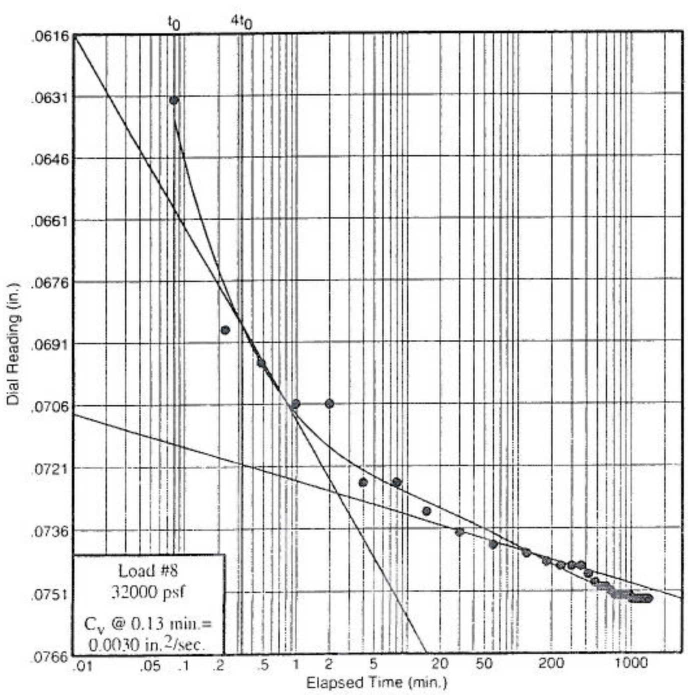
Dial Reading vs. Time

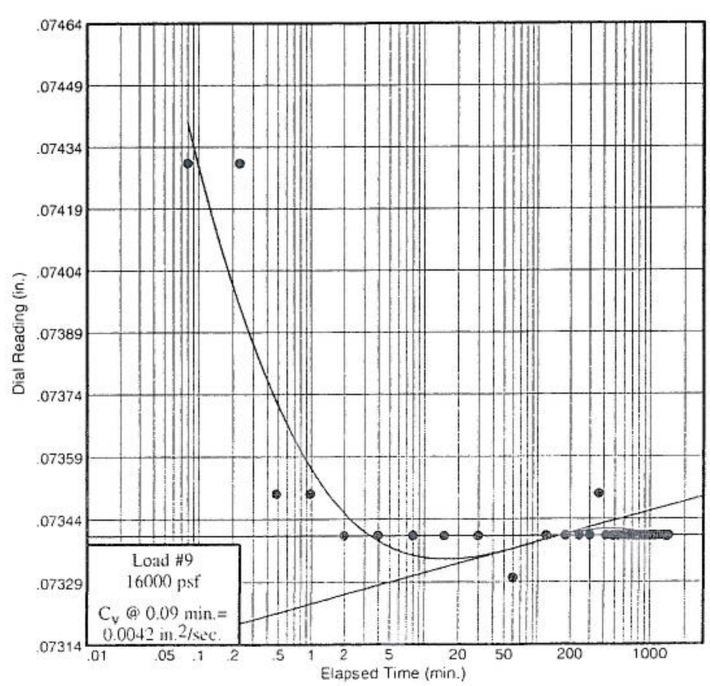
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-45 | Sample No.: P-1 | Elev./Depth: 5.0









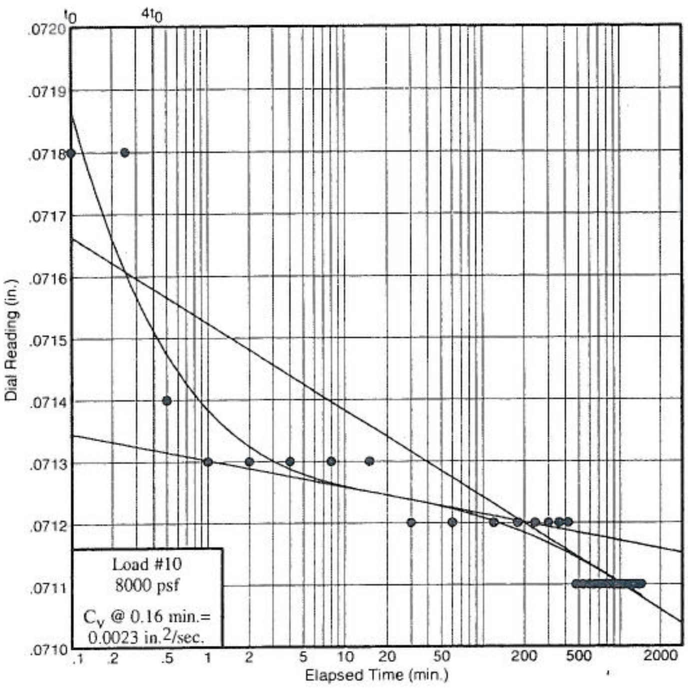
DLZ

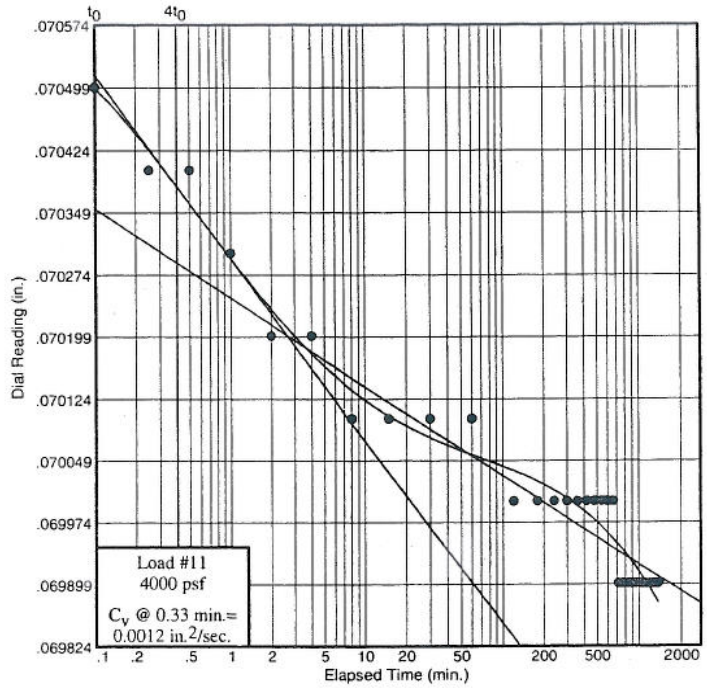
Figure

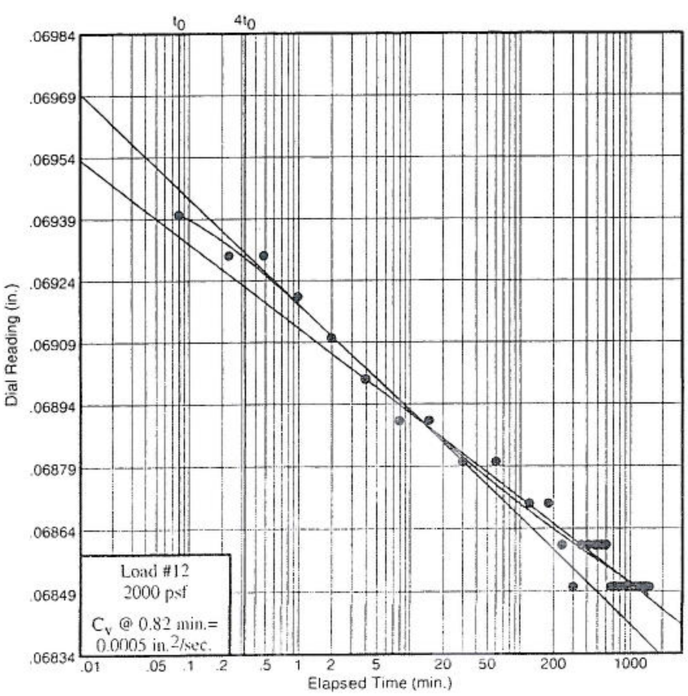
Dial Reading vs. Time

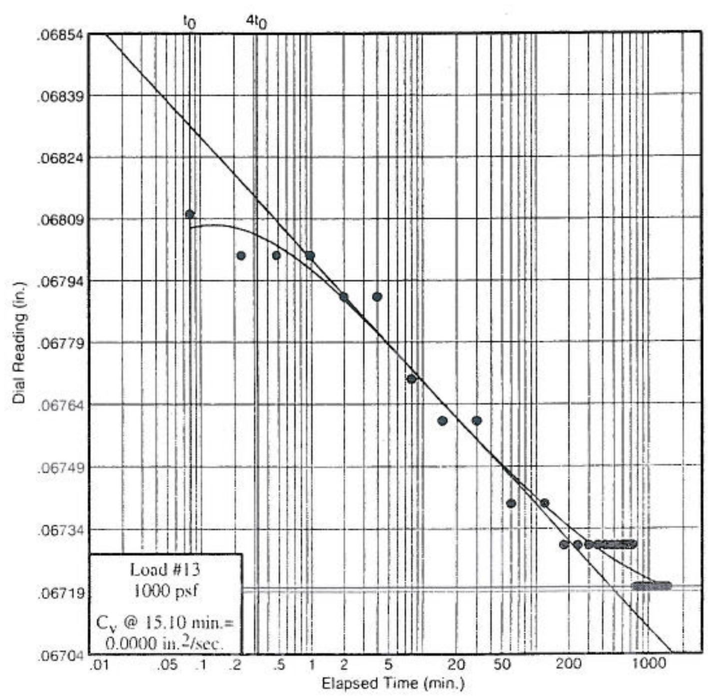
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-45 Sample No.: P-1 Elev./Depth: 5.0









DLZ

Figure

Vane Shear Test Report

Project	SCI-823 Portsmouth Bypass	Date and Time	6/14/2007
Project No.	0121-3070.03	Boring Number	B-45 Depth 6'
Client	TranSystems Corp		
Drill Rig & Crew	Doug W. on CME 850		
Tested By	Riedy / Mott		
Weather / Temp.	Sun / 85 deg		
Soil Type	Silty Clay (A-6b)		

DRILLING

Hollowstem augers to depth	D_a	5'
Vane Depth below bottom of augers	D_s	1'
Augers above ground surface	H_a	2'
Depth to vane tip	D_t	6'

SHEAR VANE

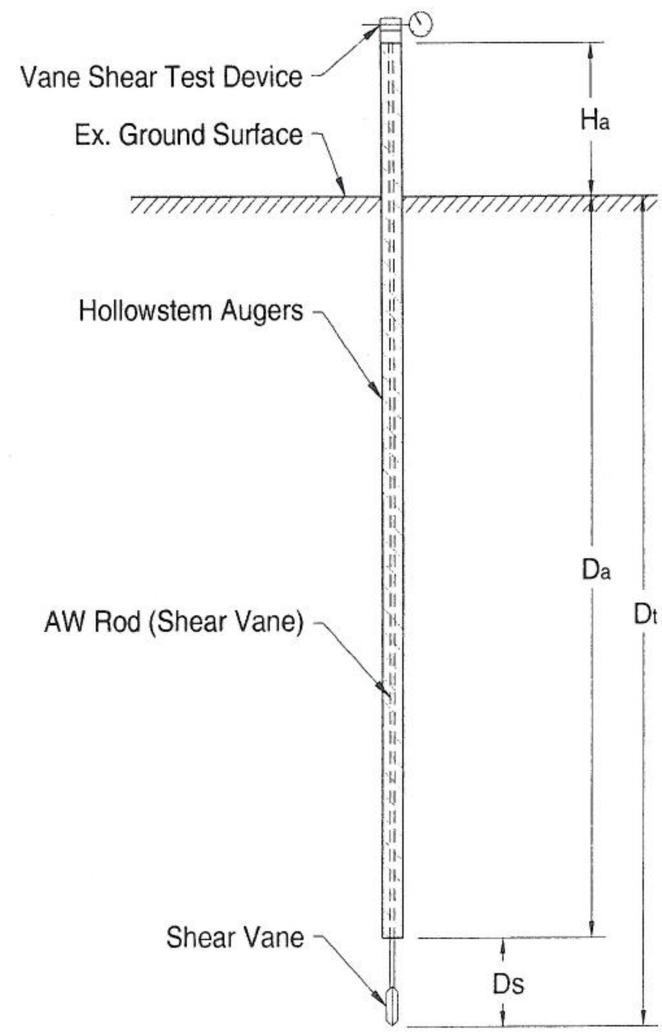
Vane Used	$\boxed{2.0"}$	2.5"	3.625"
Vane constant, k (lb-in to psf)	$\boxed{5.17}$	2.59	0.905

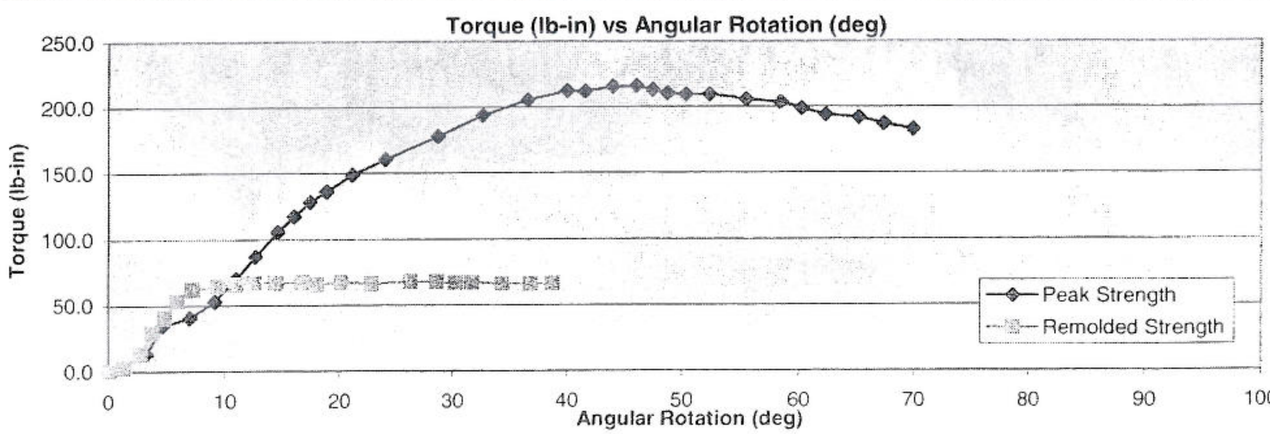
Masurement by: Automatic Drive / Torque Cell

Max Torque: 216 lb-in

Max UD Shear Strength 1116 psf

Test Setup





Vane Shear Test Report

Peak Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
15:32:01	0:00:00	0	0.0
15:32:28	0:00:27	3.2	13.4
15:32:40	0:00:39	4.7	34.9
15:32:59	0:00:58	7.0	40.4
15:33:17	0:01:16	9.1	52.9
15:33:32	0:01:31	10.9	69.9
15:33:46	0:01:45	12.6	87.0
15:34:02	0:02:01	14.52	105.6
15:34:14	0:02:13	15.96	117.2
15:34:26	0:02:25	17.4	128.3
15:34:38	0:02:37	18.84	136.2
15:34:57	0:02:56	21.12	148.8
15:35:21	0:03:20	24	160.3
15:36:00	0:03:59	28.68	177.9
15:36:33	0:04:32	32.64	193.7
15:37:06	0:05:05	36.6	205.4
15:37:34	0:05:33	39.96	212.2
15:37:47	0:05:46	41.52	211.7
15:38:07	0:06:06	43.92	215.7
15:38:24	0:06:23	45.96	215.8
15:38:36	0:06:35	47.4	213.0
15:38:47	0:06:46	48.72	210.6
15:39:00	0:06:59	50.28	209.2
15:39:17	0:07:16	52.32	209.6
15:39:43	0:07:42	55.44	205.7
15:40:08	0:08:07	58.44	203.5
15:40:23	0:08:22	60.24	198.6
15:40:41	0:08:40	62.4	193.7
15:41:05	0:09:04	65.28	191.7
15:41:23	0:09:22	67.44	187.1
15:41:44	0:09:43	69.96	182.8

Remolded Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
15:48:20	0:00:00	0	0
15:48:31	0:00:11	1.3	2.6519108
15:48:43	0:00:23	2.8	13.496029
15:48:51	0:00:31	3.7	28.912601
15:49:00	0:00:40	4.8	40.501919
15:49:09	0:00:49	5.9	53.374943
15:49:20	0:01:00	7.2	61.857727
15:49:38	0:01:18	9.4	64.540031
15:49:51	0:01:31	10.92	65.733902
15:50:04	0:01:44	12.48	66.995094
15:50:20	0:02:00	14.4	66.730019
15:50:39	0:02:19	16.68	67.381081
15:50:50	0:02:30	18	65.608727
15:51:08	0:02:48	20.16	67.27784
15:51:30	0:03:10	22.8	66.025963
15:51:59	0:03:39	26.28	67.752907
15:52:18	0:03:58	28.56	67.584648
15:52:32	0:04:12	30.24	66.863716
15:52:44	0:04:24	31.68	67.083794
15:53:06	0:04:46	34.32	66.254555
15:53:27	0:05:07	36.84	66.090385
15:53:43	0:05:23	38.76	66.317818

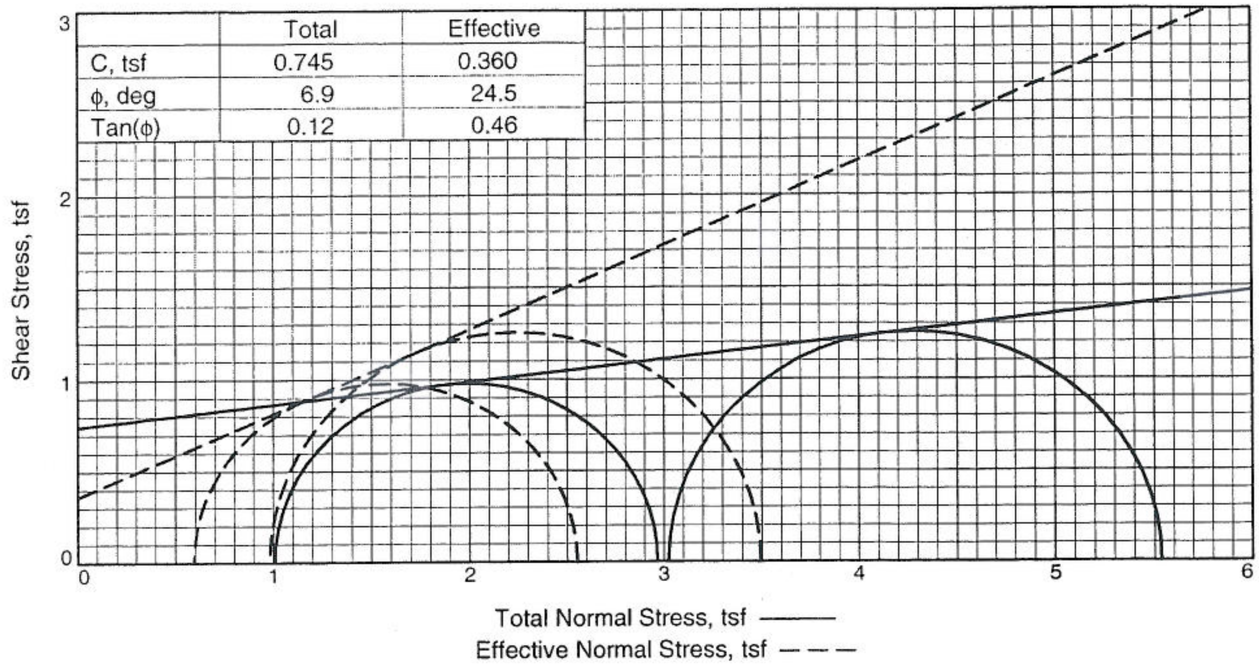
Peak Torque	*215.7953*	(lb-in)
Vane Constant	*5.17*	
Peak Shear Strength	*1116*	*psf*

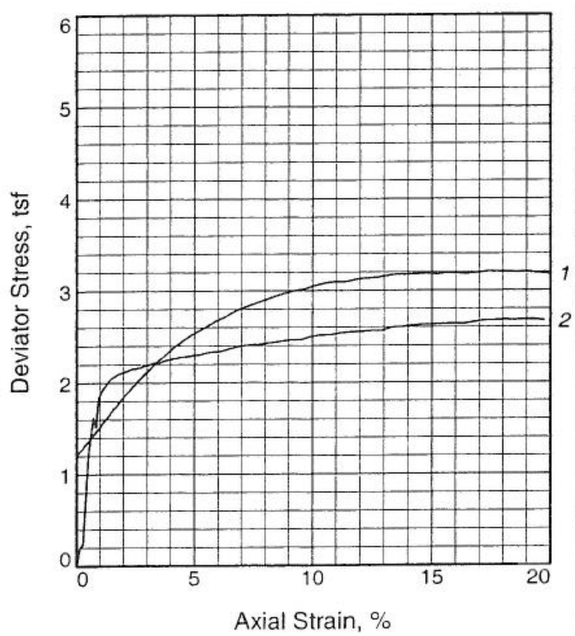
Remolded Torque	*67.75291*	(lb-in)
Vane Constant	*5.17*	
Remolded Shear Strength	*350*	*psf*
Sensitivity	*3.2*	

DLZ

DLZ Ohio, Inc.

ENGINEERS · ARCHITECTS · SCIENTISTS
PLANNERS · SURVEYORS





	Sample No.	1	2
Initial	Water Content,	14.3	14.3
	Dry Density, pcf	114.1	114.6
	Saturation,	77.9	79.0
	Void Ratio	0.5045	0.4976
	Diameter, in.	2.82	2.83
	Height, in.	5.59	5.58
At Test	Water Content,	18.0	17.0
	Dry Density, pcf	114.9	117.1
	Saturation,	100.0	100.0
	Void Ratio	0.4940	0.4663
	Diameter, in.	2.82	2.81
	Height, in.	5.55	5.51
	Strain rate, in./min.	0.01	0.01
	Back Pressure, tsf	3.31	3.31
	Cell Pressure, tsf	4.32	6.34
	Fail. Stress, tsf	1.96	2.51
	Total Pore Pr., tsf	3.73	5.35
	Ult. Stress, tsf	1.96	2.51
	Total Pore Pr., tsf	3.73	5.35
	$\bar{\sigma}_1$ Failure, tsf	2.55	3.49
	$\bar{\sigma}_3$ Failure, tsf	0.59	0.98

Type of Test:
CU with Pore Pressures

Sample Type: 3" press tube

Description: Clayey sand with gravel

LL= 27 **PL=** 16 **PI=** 11

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

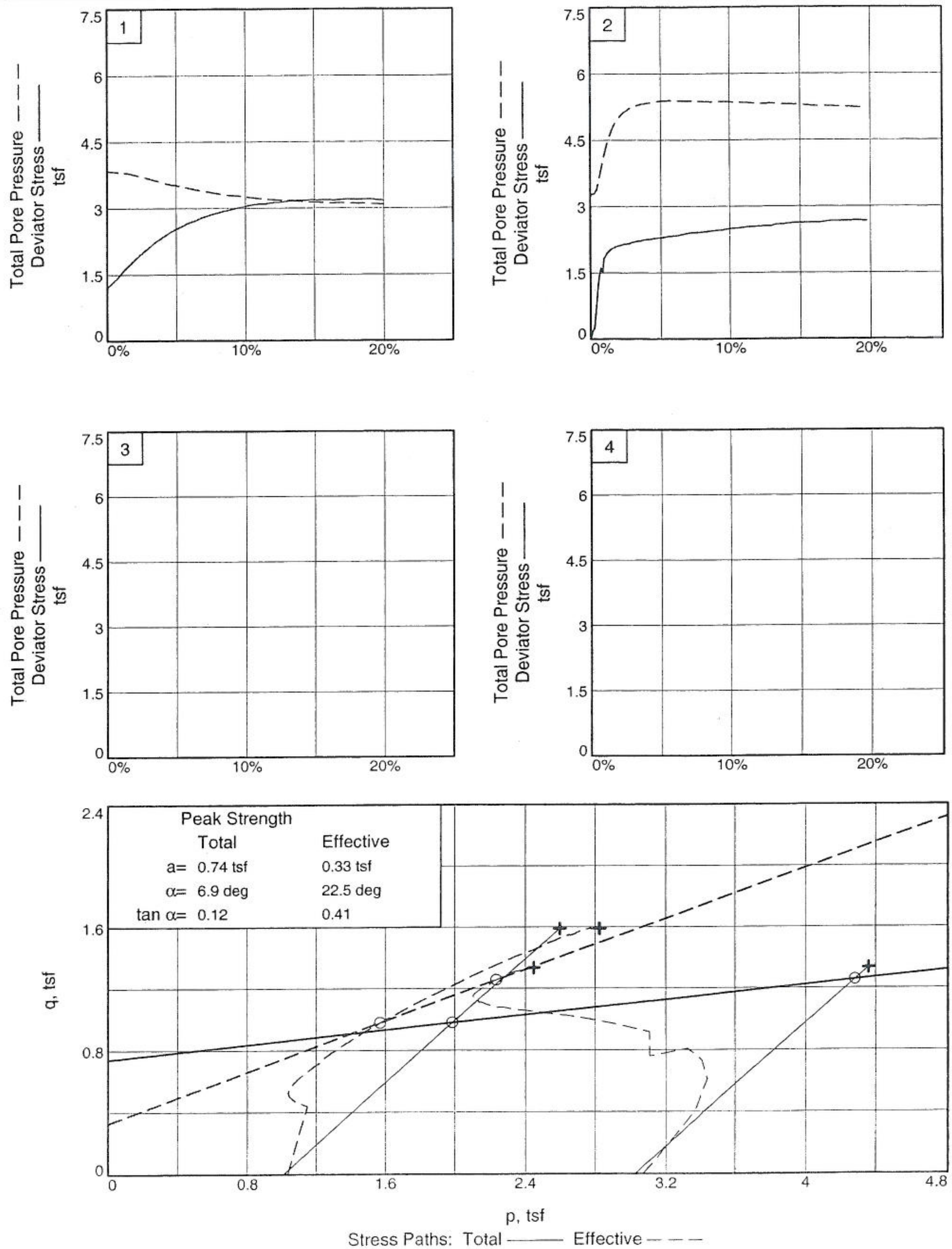
Project: SCI-823-0.00

Source of Sample: B-45 **Depth:** 8.0

Sample Number: P-2

Proj. No.: 0121-3070.03 **Date:** 7/20/07

DLZ

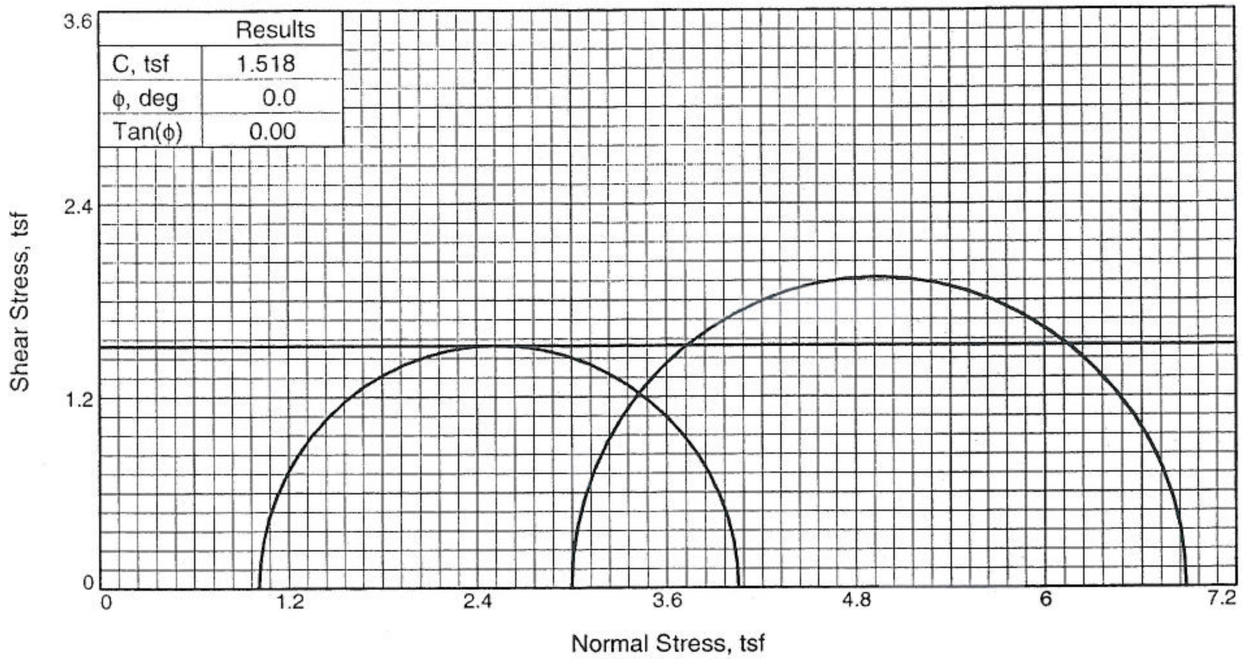


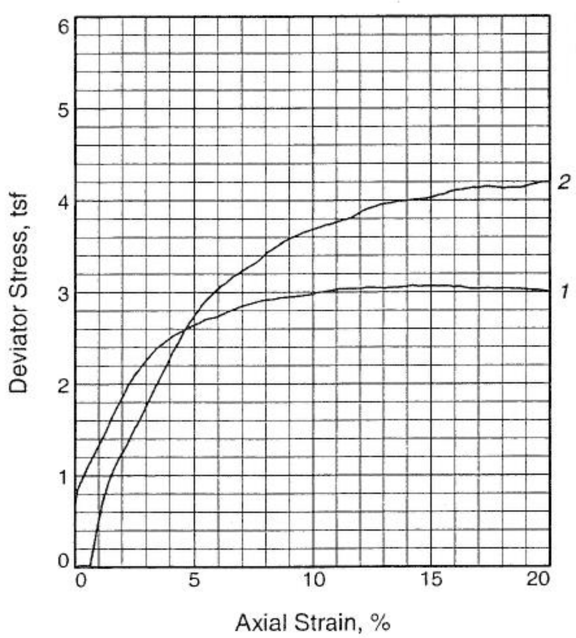
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-45 **Depth:** 8.0 **Sample Number:** P-2

Project No.: 0121-3070.03 **Figure** ________ **DLZ, INC.**





	Sample No.	1	2
Initial	Water Content,	23.7	23.2
	Dry Density, pcf	99.7	102.6
	Saturation,	90.3	94.7
	Void Ratio	0.7215	0.6731
	Diameter, in.	2.84	2.85
	Height, in.	5.56	5.55
At Test	Water Content,	23.6	23.1
	Dry Density, pcf	99.7	102.6
	Saturation,	90.0	94.3
	Void Ratio	0.7215	0.6731
	Diameter, in.	2.84	2.85
	Height, in.	5.56	5.55
	Strain rate, in./min.	0.06	0.06
	Back Pressure, tsf	0.00	0.00
	Cell Pressure, tsf	1.01	3.00
	Fail. Stress, tsf	3.04	3.88
	Ult. Stress, tsf	3.04	3.88
	σ_1 Failure, tsf	4.05	6.88
	σ_3 Failure, tsf	1.01	3.00

Type of Test:
Unconsolidated Undrained

Sample Type: 3 " press tube

Description:

Assumed Specific Gravity= 2.75

Remarks:

Figure ________

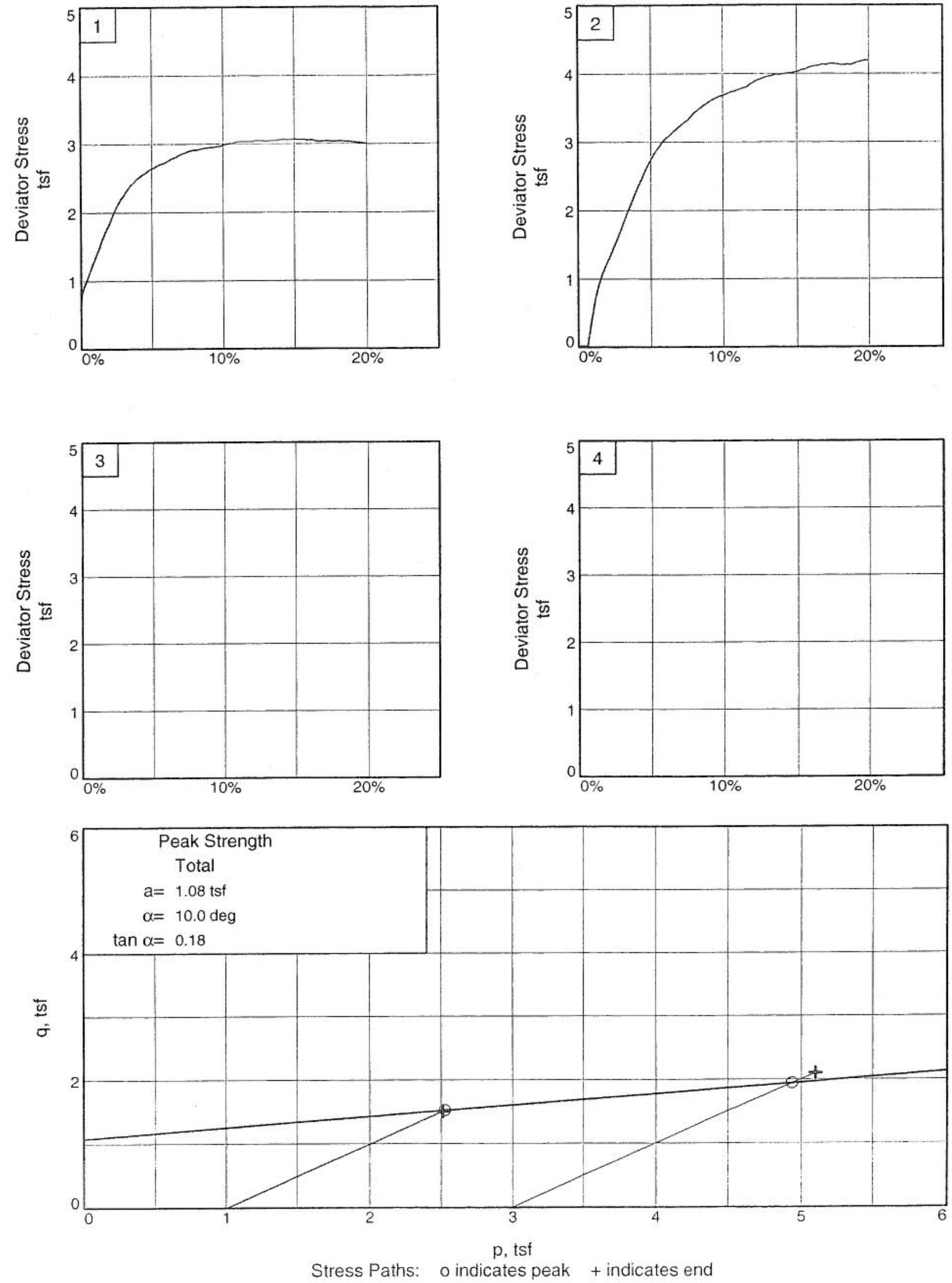
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-46 **Depth:** 5.0

Proj. No.: 0121-3070.03 **Date:**

DLZ



Client: TranSystems, Inc.

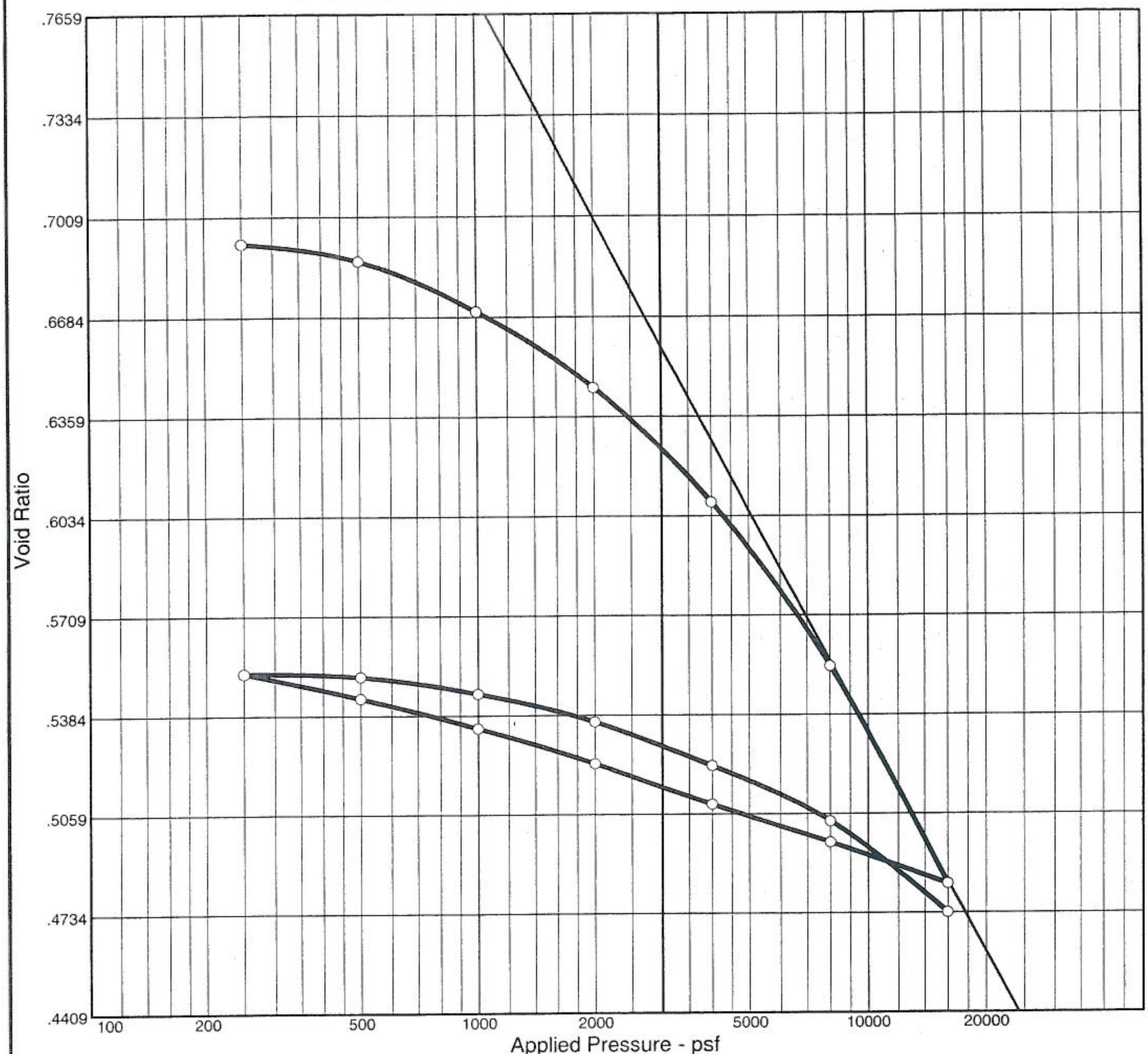
Project: SCI-823-0.00

Source of Sample: B-46 **Depth:** 5.0

Project No.: 0121-3070.03 **Figure** _______

DLZ, INC.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
90.3 %	23.1 %	100.0	36	18	2.71	CL	A-6(17)	0.692

MATERIAL DESCRIPTION

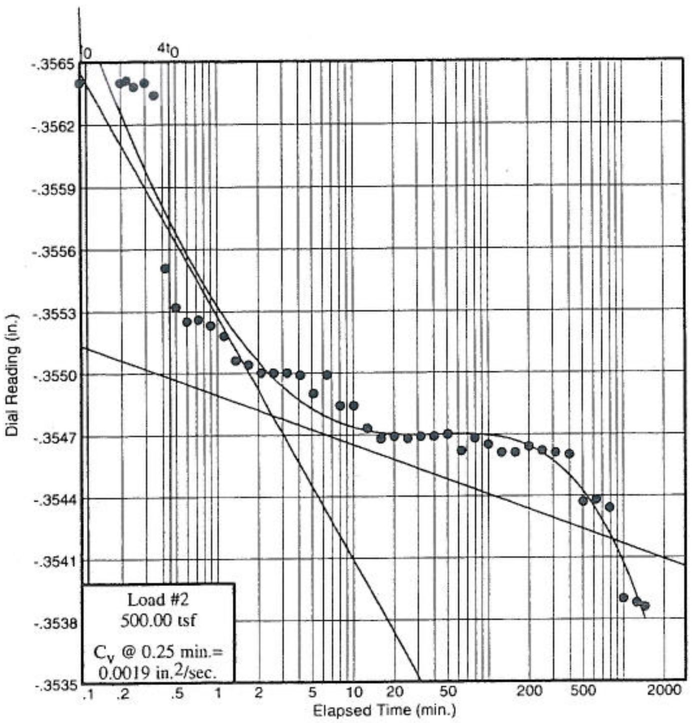
Lean clay
Specific Gravity= 2.71

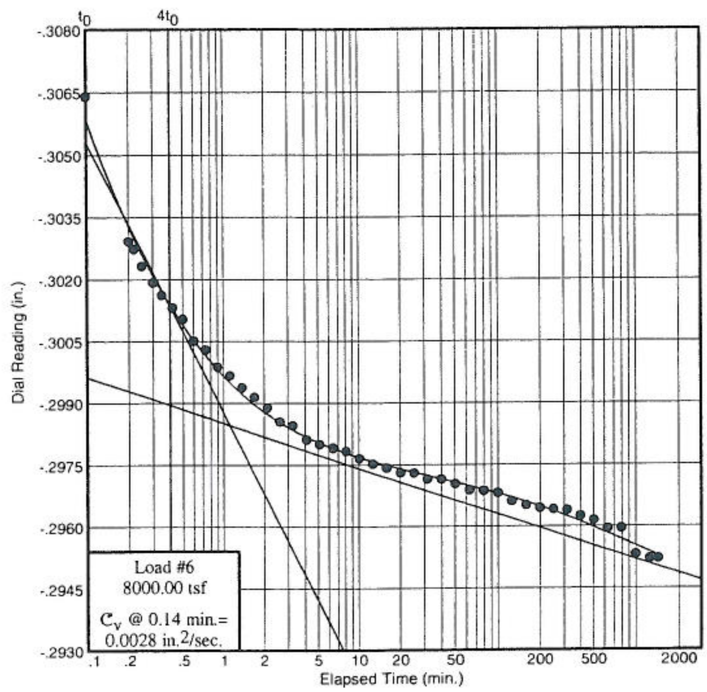
Project No. 0121-	**Client:** TranSystems, Inc.		**Remarks:**
Project: SCI-823-0.00			
Source: B-46	**Sample No.:** P-1	**Elev./Depth:** 5.0	
DLZ			**Figure**

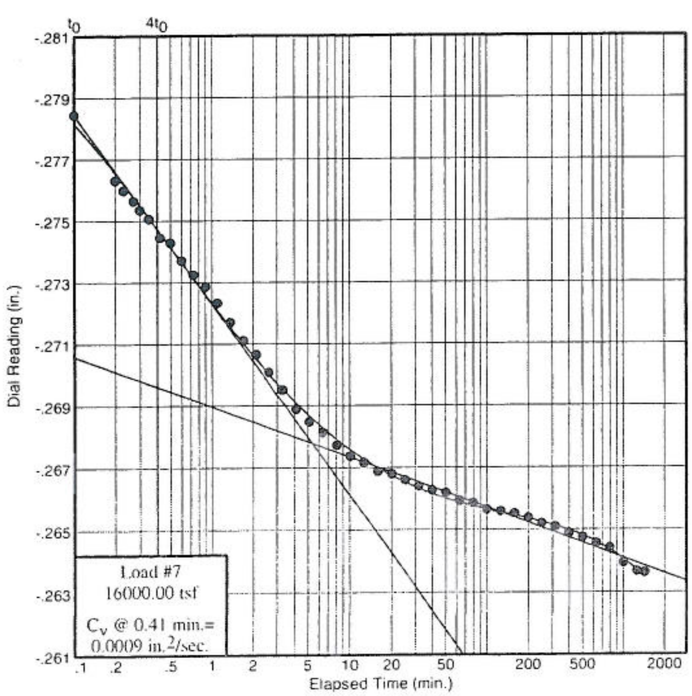
Dial Reading vs. Time

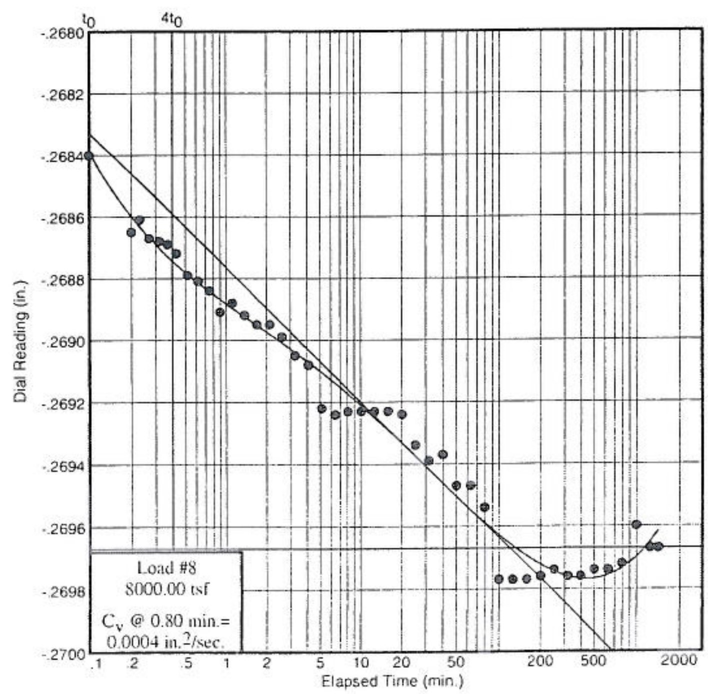
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-46 Sample No.: P-1 Elev./Depth: 5.0









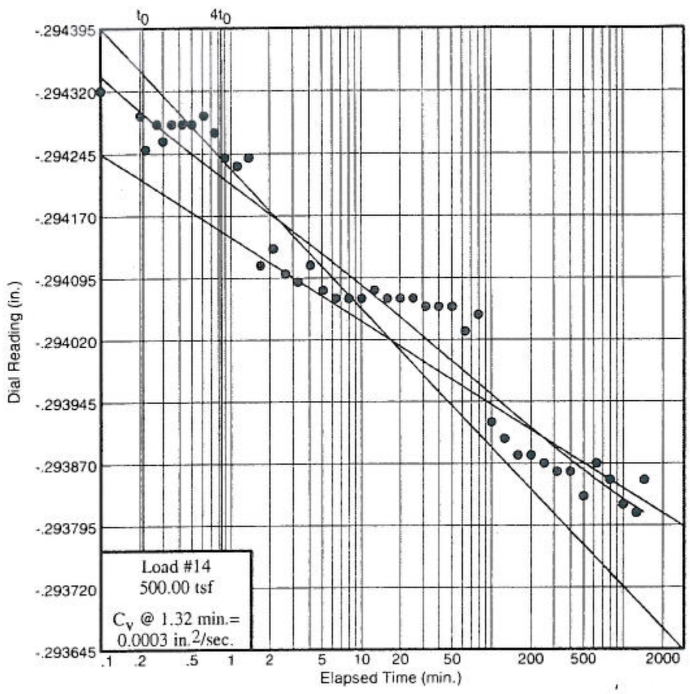
DLZ

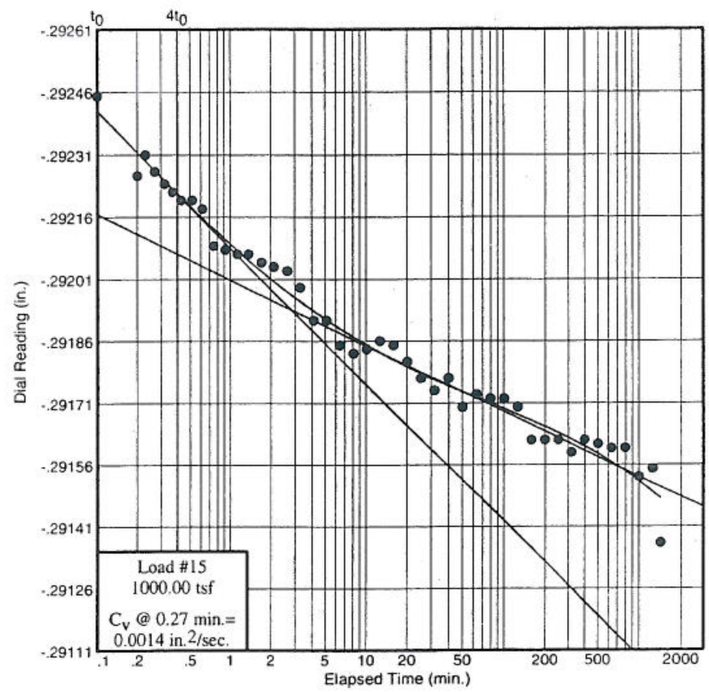
Figure

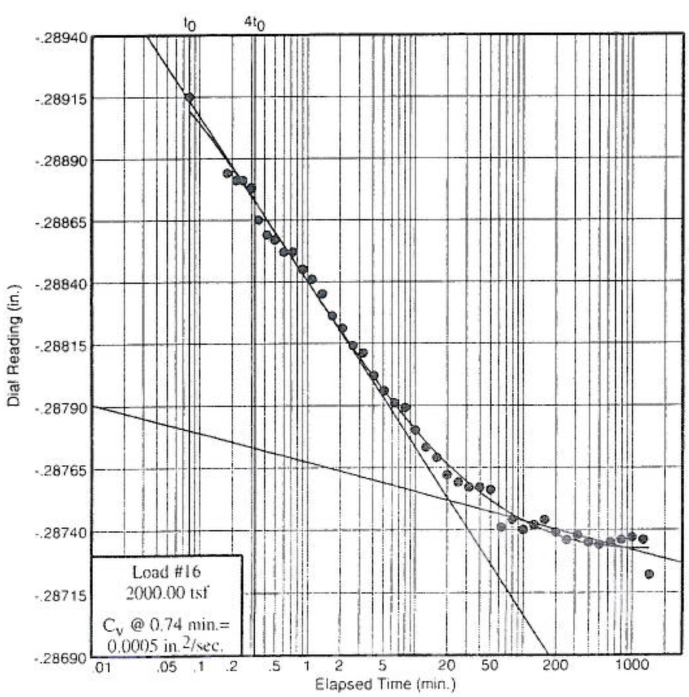
Dial Reading vs. Time

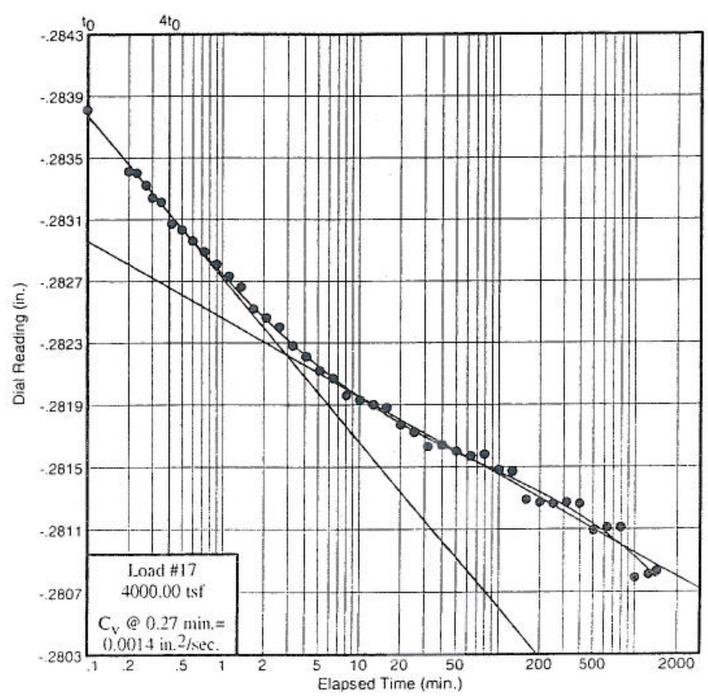
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-46 Sample No.: P-1 Elev./Depth: 5.0









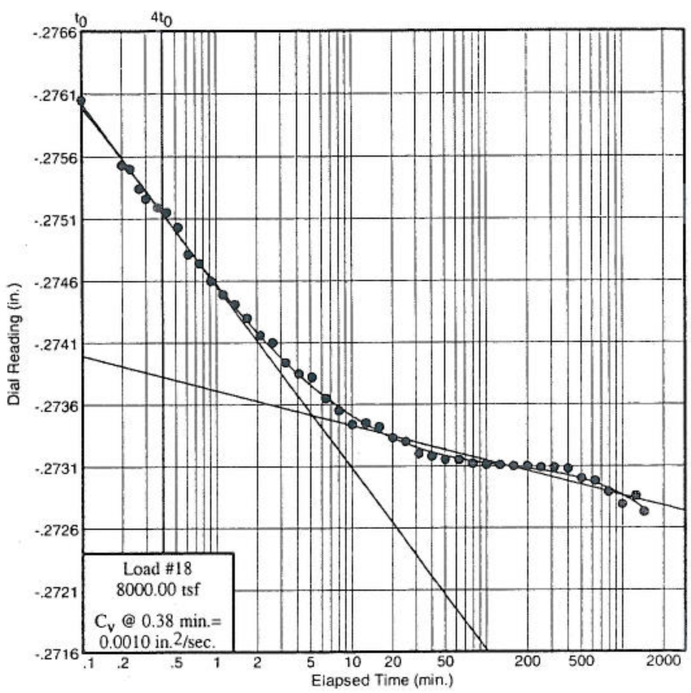
DLZ

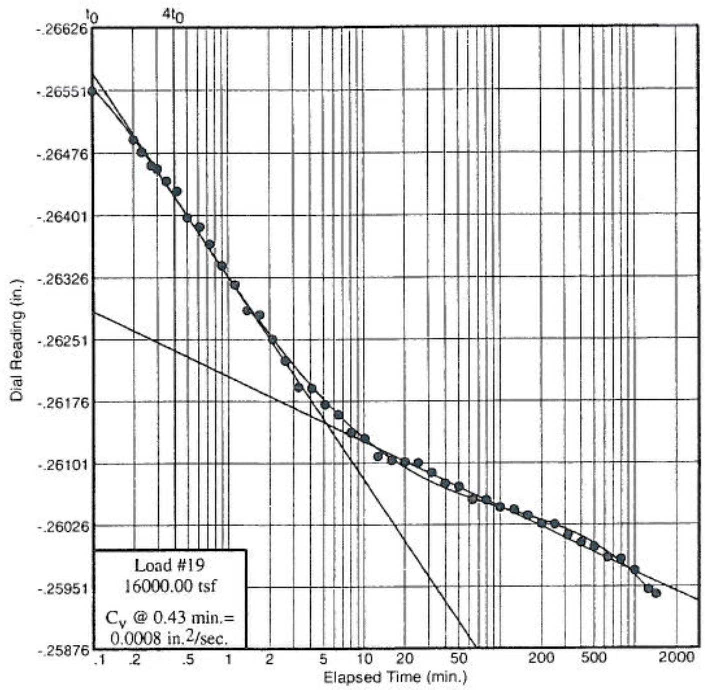
Figure

Dial Reading vs. Time

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

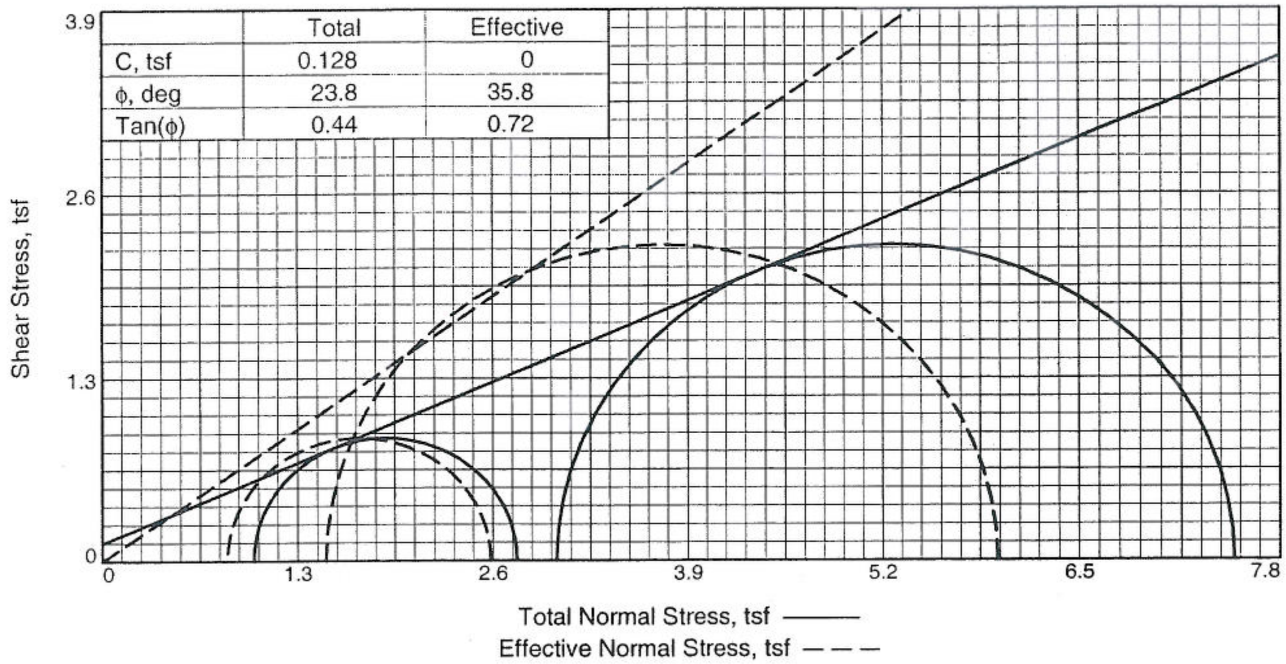
Source: B-46 | Sample No.: P-1 | Elev./Depth: 5.0

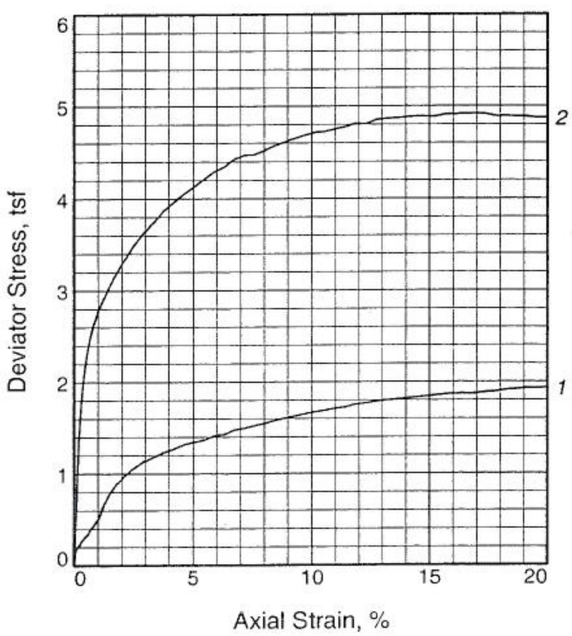




DLZ

Figure





	Sample No.	1	2
Initial	Water Content,	16.7	16.7
	Dry Density, pcf	108.2	110.2
	Saturation,	78.2	82.3
	Void Ratio	0.5861	0.5573
	Diameter, in.	2.86	2.85
	Height, in.	5.61	5.59
At Test	Water Content,	18.9	16.3
	Dry Density, pcf	112.9	118.5
	Saturation,	100.0	100.0
	Void Ratio	0.5208	0.4484
	Diameter, in.	2.80	2.76
	Height, in.	5.58	5.56
Strain rate, in./min.		0.01	0.01
Back Pressure, tsf		3.31	3.31
Cell Pressure, tsf		4.32	6.34
Fail. Stress, tsf		1.75	4.48
Total Pore Pr., tsf		3.49	4.85
Ult. Stress, tsf		1.75	4.48
Total Pore Pr., tsf		3.49	4.85
$\bar{\sigma}_1$ Failure, tsf		2.58	5.96
$\bar{\sigma}_3$ Failure, tsf		0.83	1.49

Type of Test:
CU with Pore Pressures

Sample Type: 3" press tube

Description: Clayey sand

LL= 32 **PL=** 17 **PI=** 15

Assumed Specific Gravity= 2.75

Remarks:

Figure ________

Client: TranSystems, Inc.

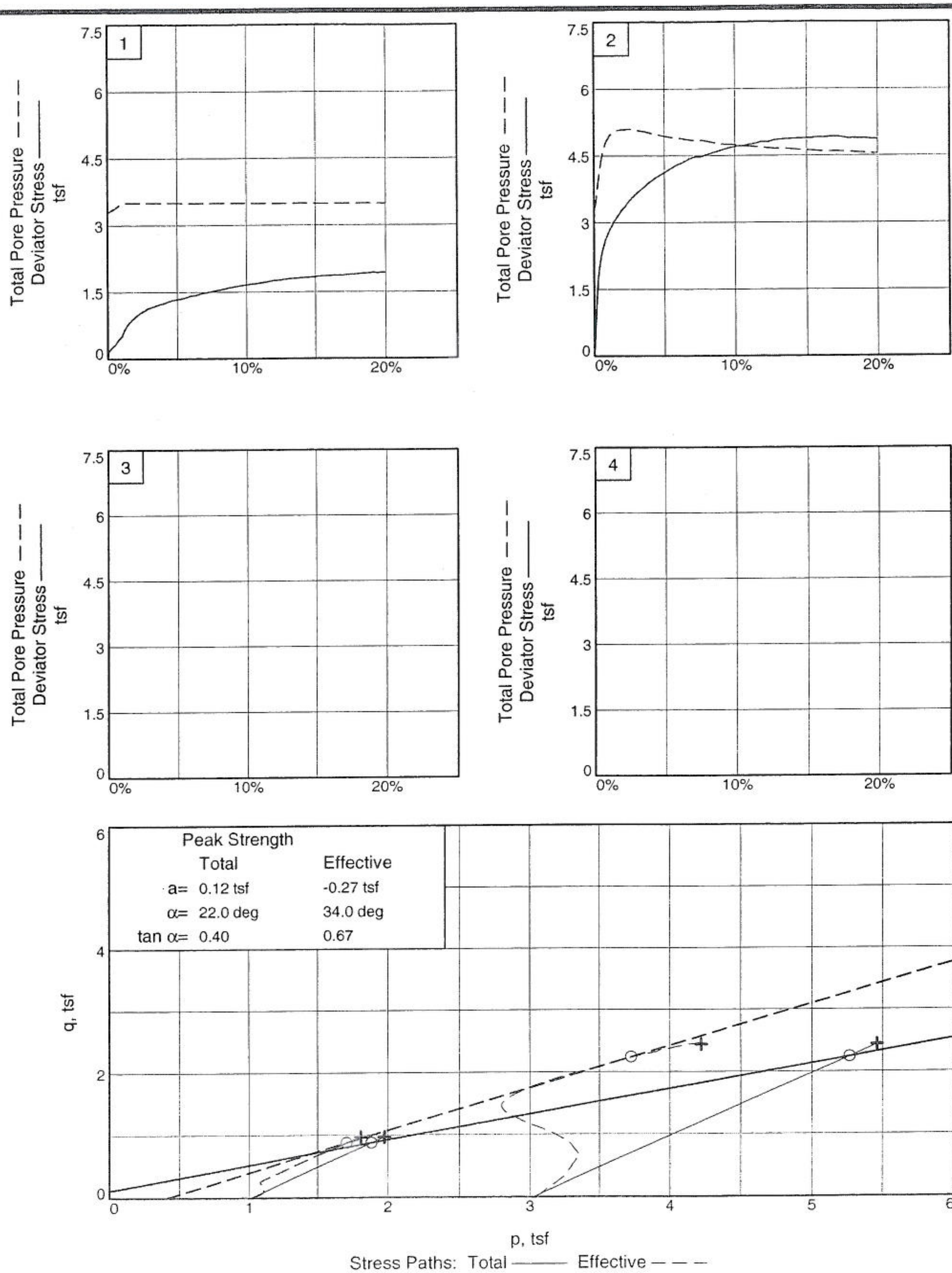
Project: SCI-823-0.00

Source of Sample: B-46 **Depth:** 8.0

Sample Number: P-2

Proj. No.: 0121-3070.03 **Date:** 7/20/07

DLZ

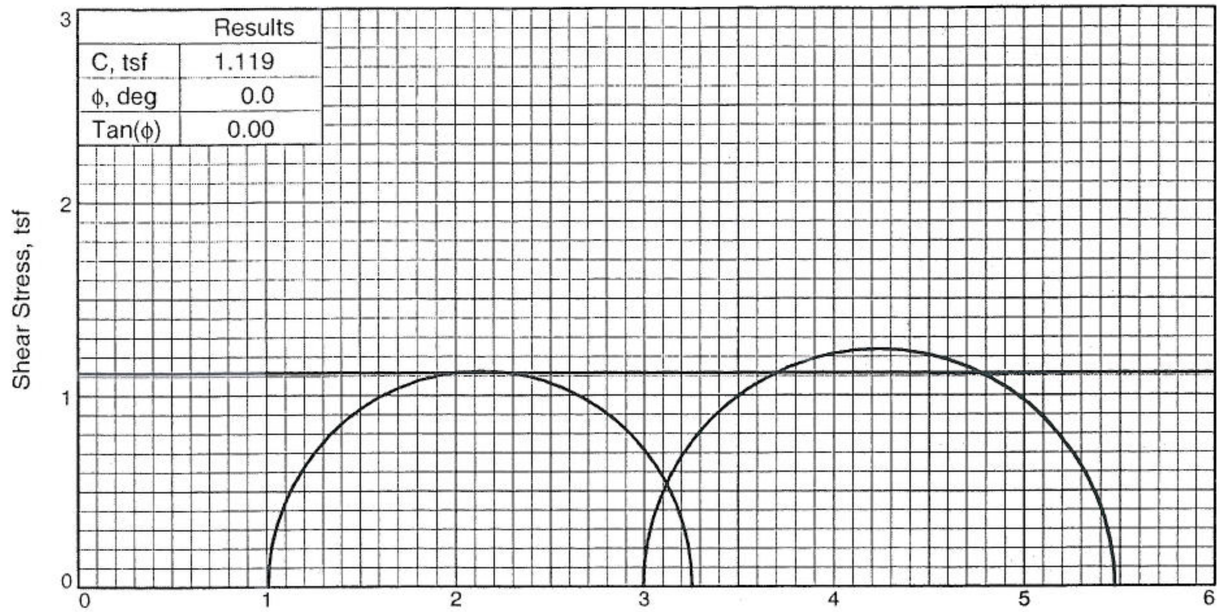


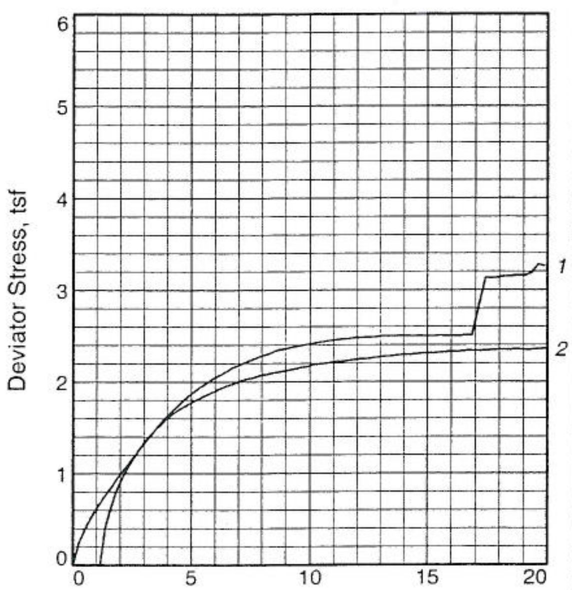
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-46 **Depth:** 8.0 **Sample Number:** P-2

Project No.: 0121-3070.03 **Figure** ________ **DLZ, INC.**





Sample No.		1	2
Initial	Water Content,	23.0	24.2
	Dry Density, pcf	97.6	97.8
	Saturation,	83.5	88.1
	Void Ratio	0.7585	0.7561
	Diameter, in.	2.84	2.84
	Height, in.	4.95	5.53
At Test	Water Content,	22.7	22.9
	Dry Density, pcf	97.6	97.8
	Saturation,	82.4	83.1
	Void Ratio	0.7585	0.7561
	Diameter, in.	2.84	2.84
	Height, in.	4.95	5.53
Strain rate, in./min.		0.06	0.06
Back Pressure, tsf		0.00	0.00
Cell Pressure, tsf		3.00	1.01
Fail. Stress, tsf		2.48	2.25
Ult. Stress, tsf		2.48	2.25
σ_1 Failure, tsf		5.47	3.26
σ_3 Failure, tsf		3.00	1.01

Type of Test:
Unconsolidated Undrained

Sample Type: 3 " press tube

Description:

LL= 26 **PL=** 19 **PI=** 7

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

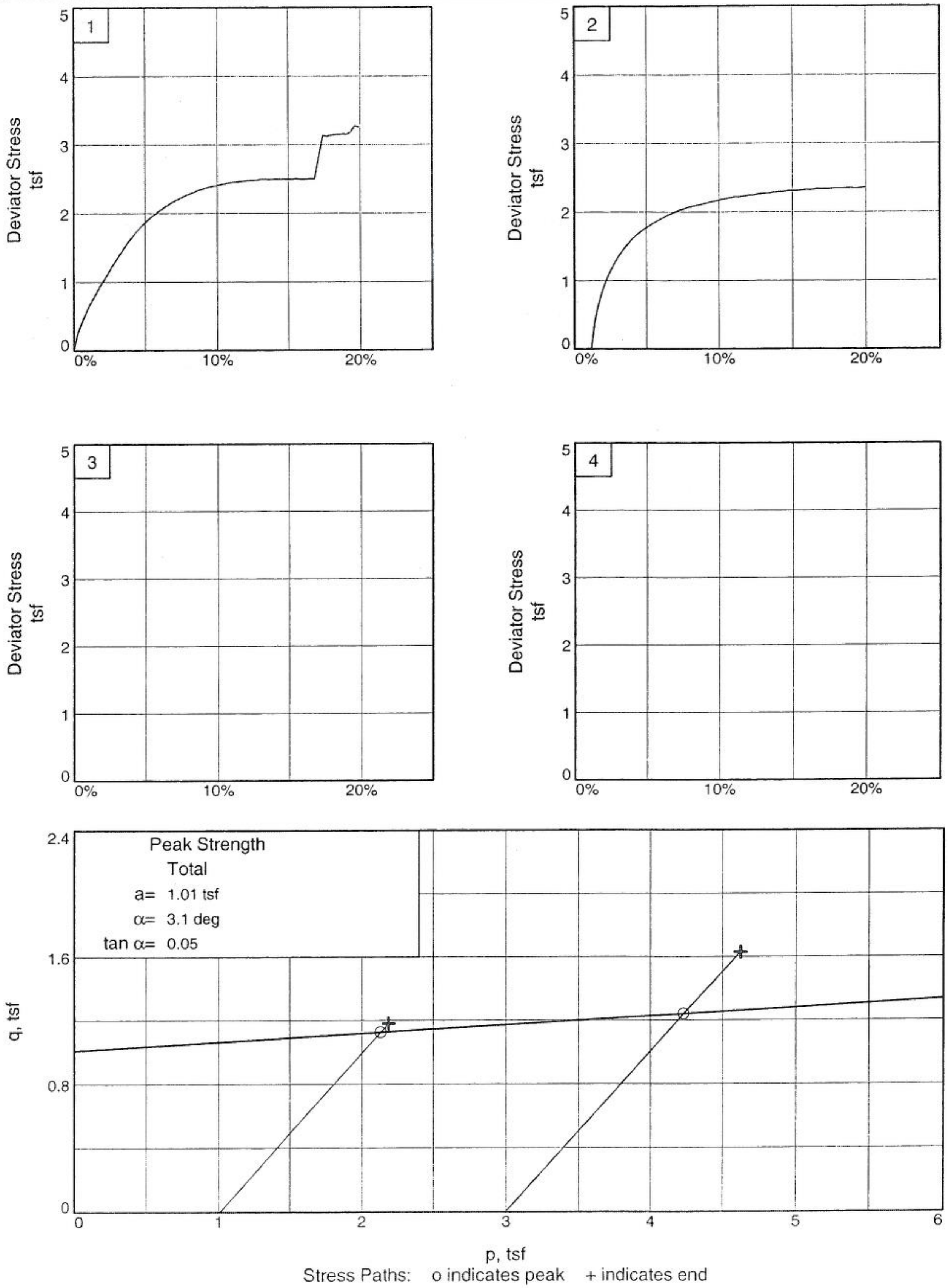
Project: SCI-823-0.00

Source of Sample: B-47 **Depth:** 4.0

Sample Number: ST-1

Proj. No.: 0121-3070.03 **Date:** 7/21/07

DLZ



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-47 **Depth:** 4.0 **Sample Number:** ST-1
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

Vane Shear Test Report

Project	SCI-823-0.00	Date and Time	6/18/2007	Begin 2:30pm End 3:00pm
Project No.	0121-3070-03	Boring Number	B-47	Depth 6.0

Client	ODOT
Drill Rig & Crew	CME 850 / D. Wamsley
Tested By	B Mott
Weather / Temp.	sunny 96
Soil Type	SILTY CLAY with sand

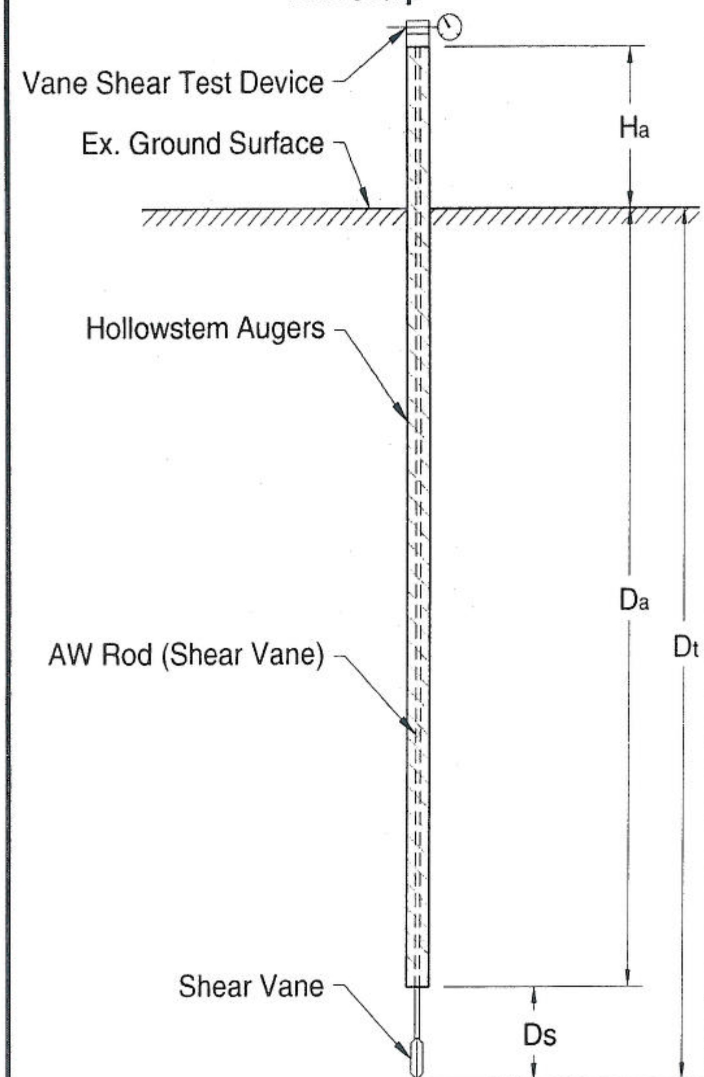
DRILLING

Hollowstem augers to depth	D_a	5
Vane Depth below bottom of augers	D_s	1.25
Augers above ground surface	H_a	7
Depth to vane tip	D_t	6

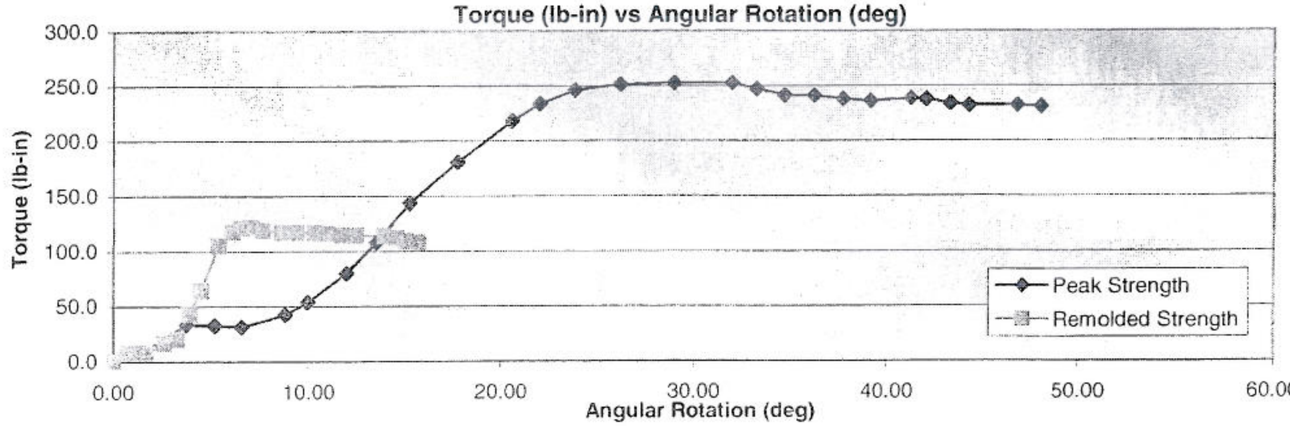
SHEAR VANE

Vane Used	$\boxed{2.0"}$	2.5"	3.625"
Vane constant, k (lb-in to psf)	$\boxed{5.17}$	2.59	0.905
Masurement by	Automatic/torque cell		
Max Torque		253	lb-in
Max UD Shear Strength		**1306**	**psf**

Test Setup



Torque (lb-in) vs Angular Rotation (deg)



Vane Shear Test Report

Peak Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
14:38:34	0:00:00	0.00	0.0
14:38:42	0:00:08	0.77	7.6
14:38:48	0:00:14	1.35	7.4
14:39:13	0:00:39	3.77	33.6
14:39:28	0:00:54	5.22	32.6
14:39:42	0:01:08	6.57	31.4
14:40:05	0:01:31	8.80	42.4
14:40:17	0:01:43	9.96	53.9
14:40:38	0:02:04	11.99	80.2
14:40:54	0:02:20	13.53	107.9
14:41:12	0:02:38	15.27	143.7
14:41:38	0:03:04	17.79	180.7
14:42:07	0:03:33	20.59	218.2
14:42:22	0:03:48	22.04	233.7
14:42:41	0:04:07	23.88	246.0
14:43:05	0:04:31	26.20	251.3
14:43:34	0:05:00	29.00	252.5
14:44:05	0:05:31	32.00	252.5
14:44:18	0:05:44	33.25	247.0
14:44:33	0:05:59	34.70	241.1
14:44:49	0:06:15	36.25	241.0
14:45:04	0:06:30	37.70	237.7
14:45:19	0:06:45	39.15	236.0
14:45:41	0:07:07	41.28	238.1
14:45:49	0:07:15	42.05	237.6
14:46:02	0:07:28	43.31	233.8
14:46:12	0:07:38	44.27	232.2
14:46:38	0:08:04	46.79	232.0
14:46:51	0:08:17	48.04	230.9

Remolded Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
14:56:26	0:00:00	0.00	0.0
14:56:34	0:00:08	0.77	8.1
14:56:38	0:00:12	1.16	8.0
14:56:44	0:00:18	1.74	7.9
14:56:53	0:00:27	2.61	16.7
14:57:00	0:00:34	3.29	20.1
14:57:07	0:00:41	3.96	43.2
14:57:13	0:00:47	4.54	64.2
14:57:22	0:00:56	5.41	104.7
14:57:29	0:01:03	6.09	118.0
14:57:34	0:01:08	6.57	121.0
14:57:39	0:01:13	7.06	121.9
14:57:45	0:01:19	7.64	119.2
14:57:55	0:01:29	8.60	117.2
14:58:03	0:01:37	9.38	117.3
14:58:13	0:01:47	10.34	116.8
14:58:20	0:01:54	11.02	116.3
14:58:28	0:02:02	11.79	114.8
14:58:35	0:02:09	12.47	114.4
14:58:50	0:02:24	13.92	114.3
14:58:57	0:02:31	14.60	112.3
14:59:02	0:02:36	15.08	109.3
14:59:09	0:02:43	15.76	107.9

Peak Torque	*252.5498*	(lb-in)
Vane Constant	*5.17*	
Peak Shear Strength	*1306*	*psf*

Remolded Torque	*121.93*	(lb-in)
Vane Constant	*5.17*	
Remolded Shear Strength	*630.40*	*psf*
Sensitivity	*2.1*	

DLZ

DLZ Ohio, Inc.

ENGINEERS · ARCHITECTS · SCIENTISTS
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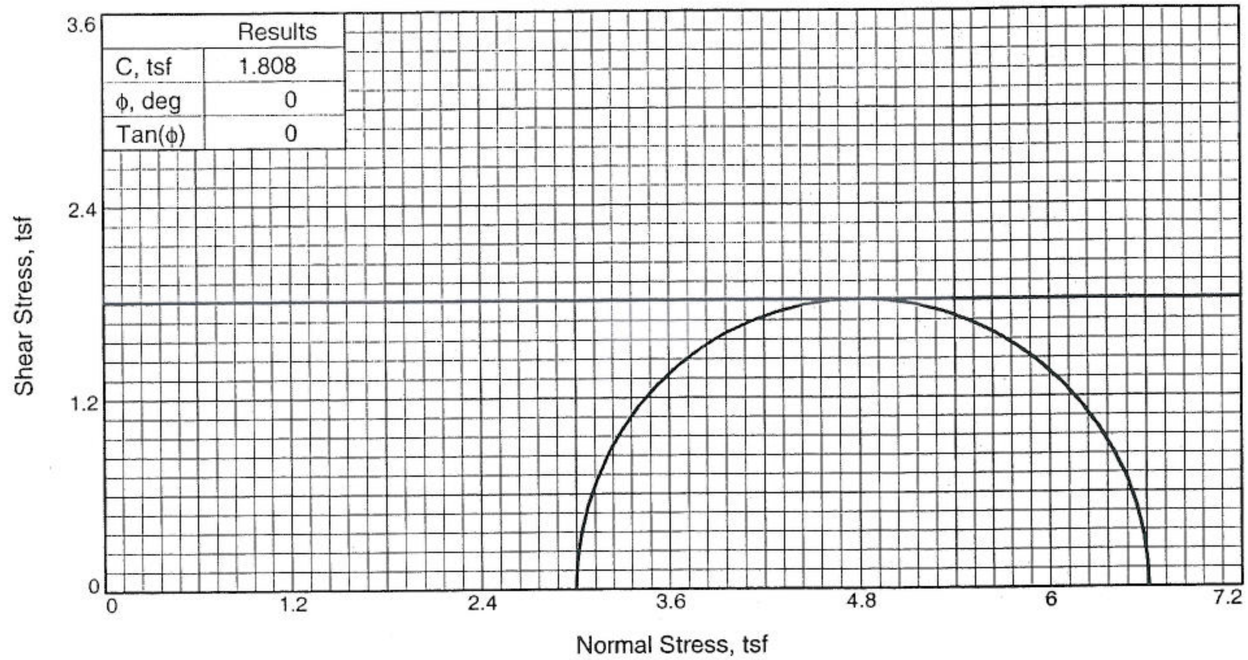
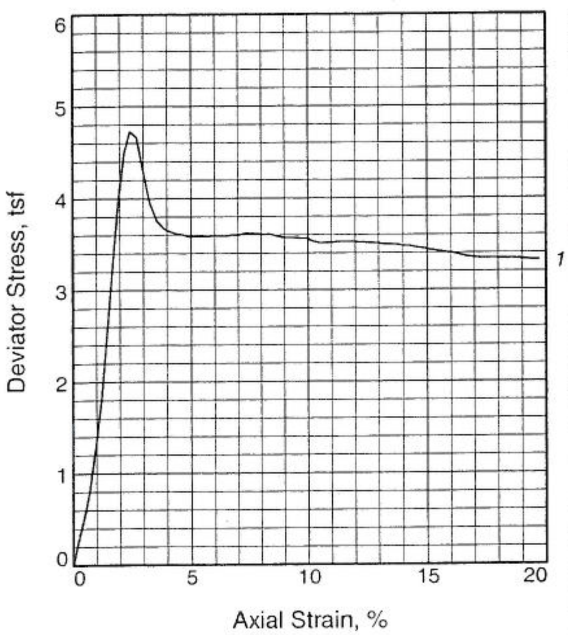




	Sample No.	1
Initial	Water Content,	19.8
	Dry Density, pcf	102.7
	Saturation,	81.0
	Void Ratio	0.6713
	Diameter, in.	2.85
	Height, in.	5.03
At Test	Water Content,	18.4
	Dry Density, pcf	102.7
	Saturation,	75.4
	Void Ratio	0.6713
	Diameter, in.	2.85
	Height, in.	5.03
	Strain rate, in./min.	0.06
	Back Pressure, tsf	0.00
	Cell Pressure, tsf	3.00
	Fail. Stress, tsf	3.62
	Ult. Stress, tsf	3.62
	σ_1 Failure, tsf	6.61
	σ_3 Failure, tsf	3.00

Type of Test:
Unconsolidated Undrained

Sample Type:

Description: Silty clay

LL= 23 **PL=** 17 **PI=** 6

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

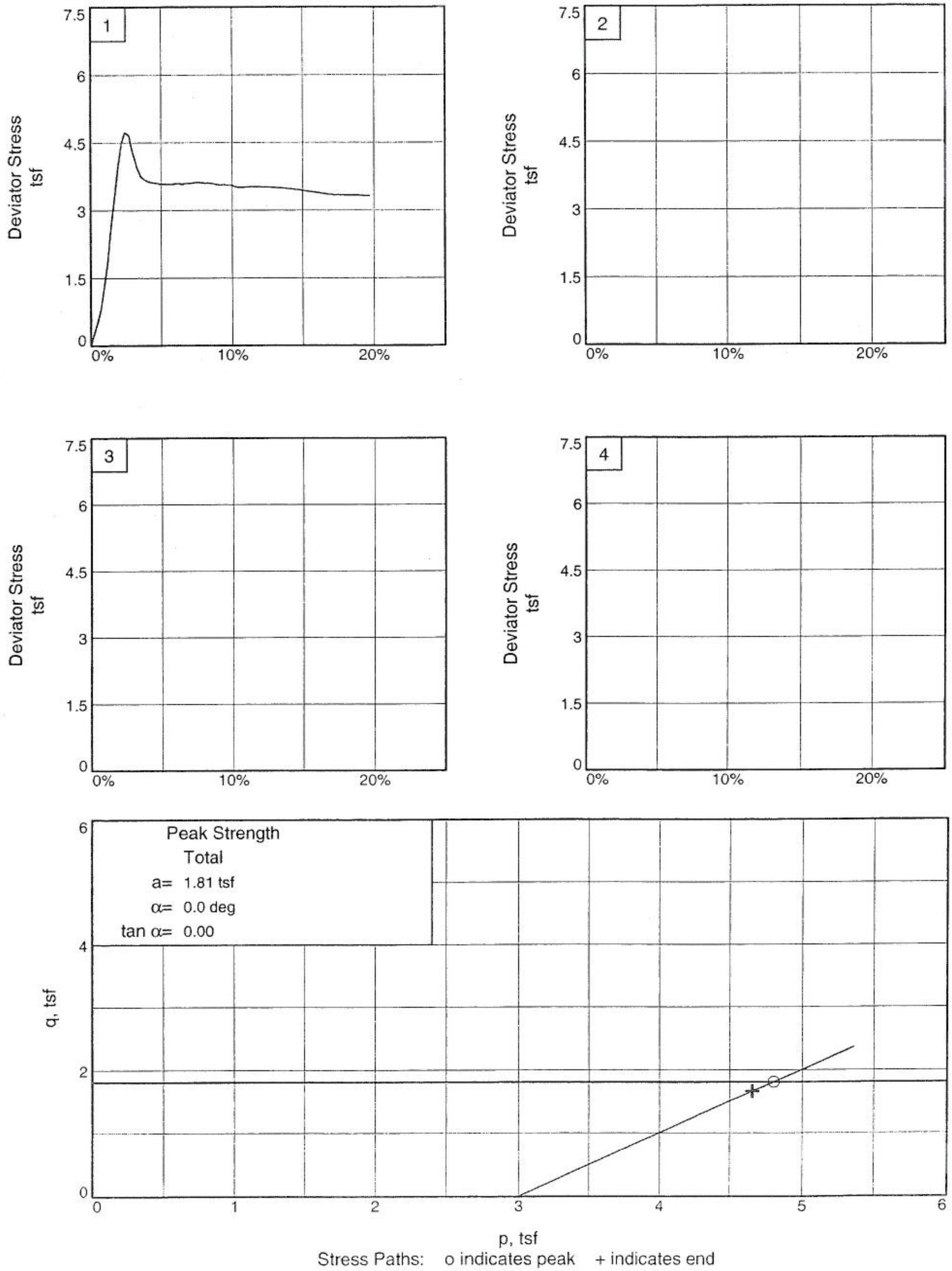
Project: SCI-823-0.00

Source of Sample: B-47 **Depth:** 6.0

Sample Number: ST-2

Proj. No.: 0121-3070.03 **Date:** 7/21/07

DLZ



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-47 **Depth:** 6.0 **Sample Number:** ST-2
Project No.: 0121-3070.03 **Figure** ________

DLZ, INC.

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

DLZ Project No.: 0121-3070.03 **Client:** CH2M Hill

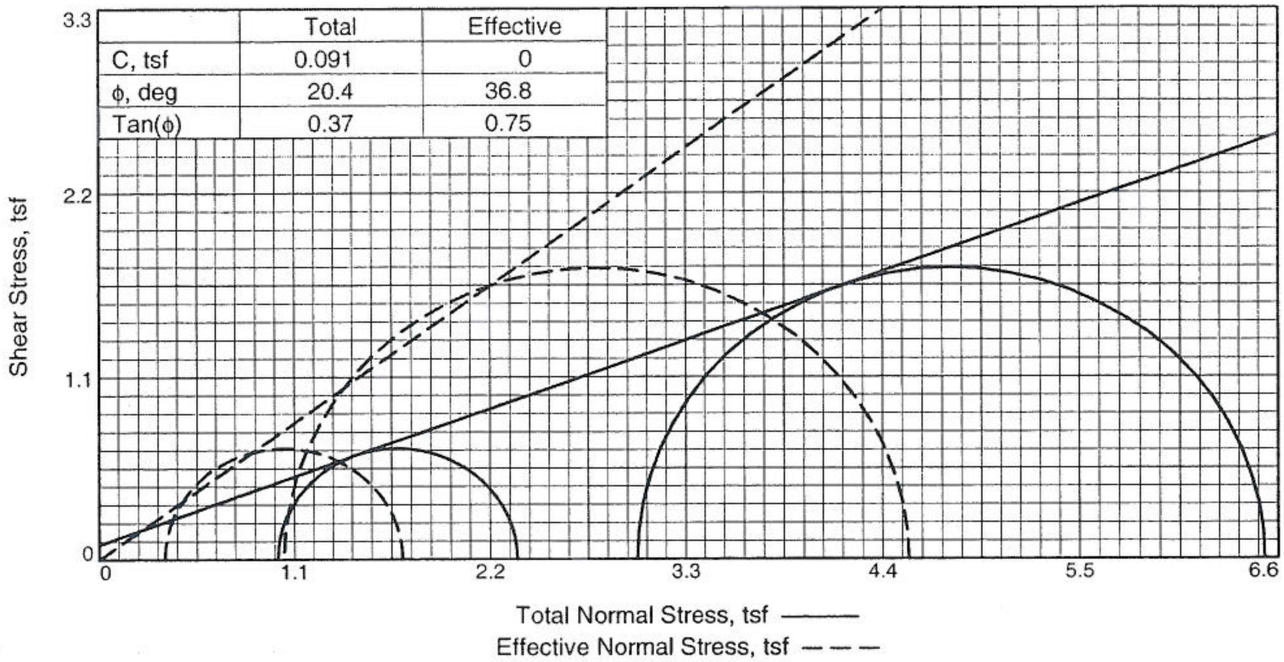
Project Name: SCI-823-0.00 **Date:** 8/21/07

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
B-45	R-2	27.2-28.2	1.966	1.988	1.977	1.975	4.313	4.297	4.274	4.295	2.175	0.0076078	533.78	154.68	8,120	**2,651**
			1.963	1.981	1.973											
B-45	R-3	37.5-38.1	1.965	1.957	1.968	1.965	3.932	3.930	3.934	3.932	2.001	0.006895	460.37	147.20	11,390	**3,757**
			1.967	1.960	1.971											
B-46	R-2	25.2-25.6	1.908	1.938	1.940	1.929	3.756	3.771	3.769	3.765	1.952	0.0063619	447.89	155.21	11,750	**4,011**
			1.908	1.936	1.941											
B-46	R-3	35.7-36.1	1.967	1.976	1.985	1.978	4.631	4.630	4.622	4.628	2.340	0.008224	543.40	145.67	9,310	**3,030**
			1.971	1.979	1.989											
B-47	R-1	20.4-20.7	1.955	1.972	1.975	1.965	3.690	3.680	3.690	3.687	1.877	0.0064637	453.65	154.73	6,020	**1,971**
			1.953	1.972	1.960											
B-47	R-3	26.8-27.2	1.952	1.969	1.981	1.968	4.261	4.252	4.243	4.252	2.161	0.0074802	525.11	154.77	9,460	**3,110**
			1.953	1.977	1.975											

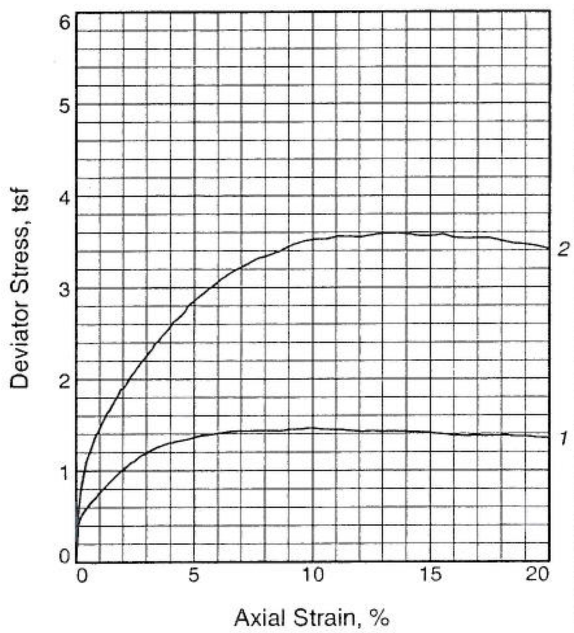
DLZ

Engineers * Architects * Scientists

6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415



	Total	Effective
C, tsf	0.091	0
ϕ, deg	20.4	36.8
Tan(ϕ)	0.37	0.75



	Sample No.	1	2
Initial	Water Content,	27.6	24.7
	Dry Density, pcf	91.6	96.3
	Saturation,	86.8	86.9
	Void Ratio	0.8742	0.7830
	Diameter, in.	2.82	2.79
	Height, in.	5.54	4.95
At Test	Water Content,	28.5	24.2
	Dry Density, pcf	96.3	103.1
	Saturation,	100.0	100.0
	Void Ratio	0.7834	0.6657
	Diameter, in.	2.78	2.74
	Height, in.	5.44	4.79
	Strain rate, in./min.	0.01	0.01
	Back Pressure, tsf	3.31	3.31
	Cell Pressure, tsf	4.32	6.34
	Fail. Stress, tsf	1.34	3.50
	Total Pore Pr., tsf	3.95	5.29
	Ult. Stress, tsf	1.34	3.50
	Total Pore Pr., tsf	3.95	5.29
	$\bar{\sigma}_1$ Failure, tsf	1.71	4.55
	$\bar{\sigma}_3$ Failure, tsf	0.37	1.04

Type of Test:
CU with Pore Pressures

Sample Type: Press tube

Description:

LL= 38 **PL=** 21 **PI=** 17

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

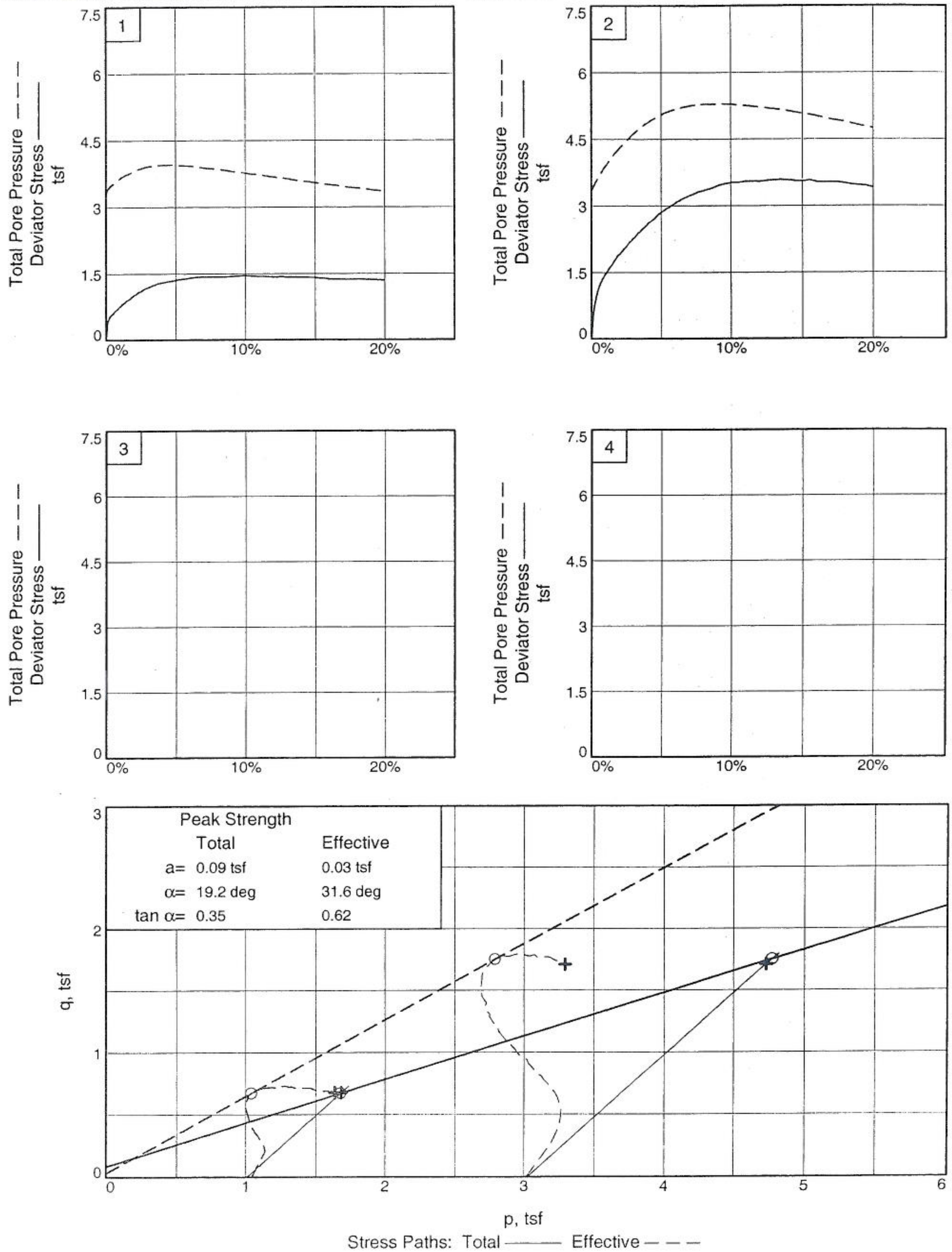
Project: SCI-823-0.00

Source of Sample: B-1105A **Depth:** 16.0

Sample Number: ST4

Proj. No.: 0121-3070.03 **Date:** 8/24/07

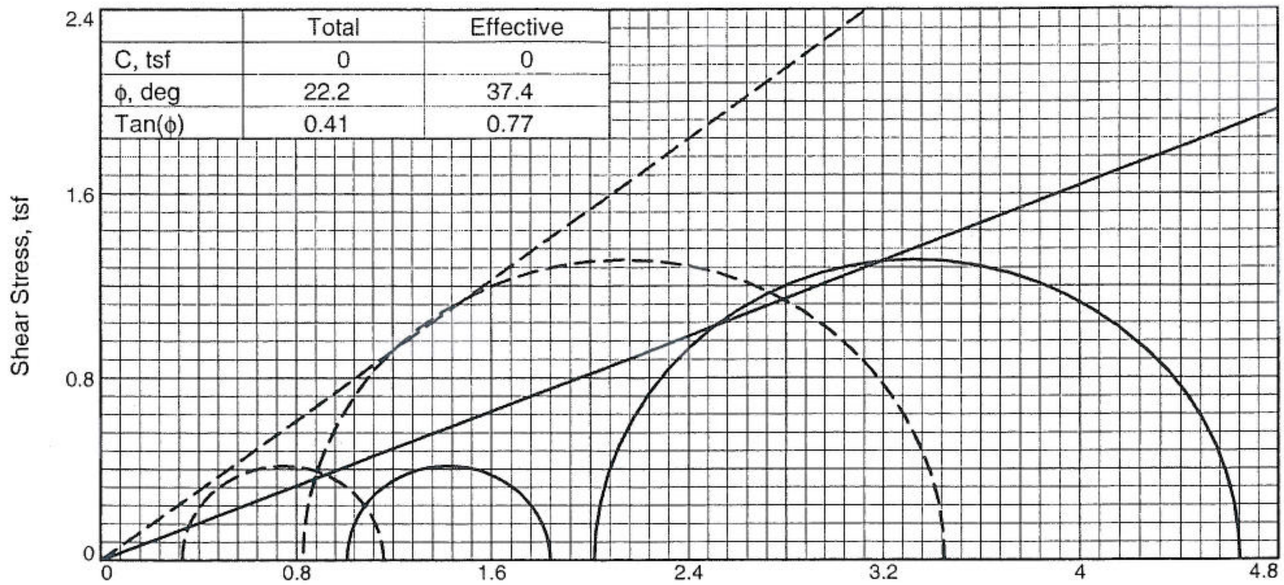
DLZ

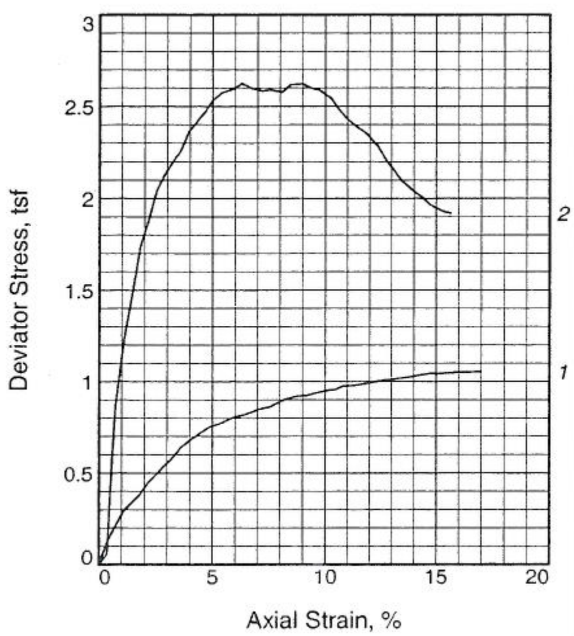


Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1105A **Depth:** 16.0 **Sample Number:** ST4
Project No.: 0121-3070.03 **Figure** ________

DLZ, INC.

	Total	Effective
C, tsf	0	0
ϕ, deg	22.2	37.4
Tan(ϕ)	0.41	0.77





Sample No.		1	2
Initial	Water Content,	28.4	29.1
	Dry Density, pcf	95.8	95.6
	Saturation,	98.7	100.4
	Void Ratio	0.7914	0.7964
	Diameter, in.	2.84	2.83
	Height, in.	5.56	5.56
At Test	Water Content,	26.3	25.7
	Dry Density, pcf	99.7	100.6
	Saturation,	100.0	100.0
	Void Ratio	0.7223	0.7068
	Diameter, in.	2.79	2.76
	Height, in.	5.56	5.56
Strain rate, in./min.		0.06	0.06
Back Pressure, tsf		4.03	4.03
Cell Pressure, tsf		5.04	6.05
Fail. Stress, tsf		0.83	2.63
Total Pore Pr., tsf		4.71	5.22
Ult. Stress, tsf		0.83	2.63
Total Pore Pr., tsf		4.71	5.22
$\bar{\sigma}_1$ Failure, tsf		1.16	3.45
$\bar{\sigma}_3$ Failure, tsf		0.33	0.82

Type of Test:
CU with Pore Pressures

Sample Type: 3" Press TUbe

Description: Lean clay

LL= 38 **PL=** 19 **PI=** 19

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-1108 **Depth:** 18.0

Sample Number: P3

Proj. No.: 0121-3070.03 **Date:** 08/16/06

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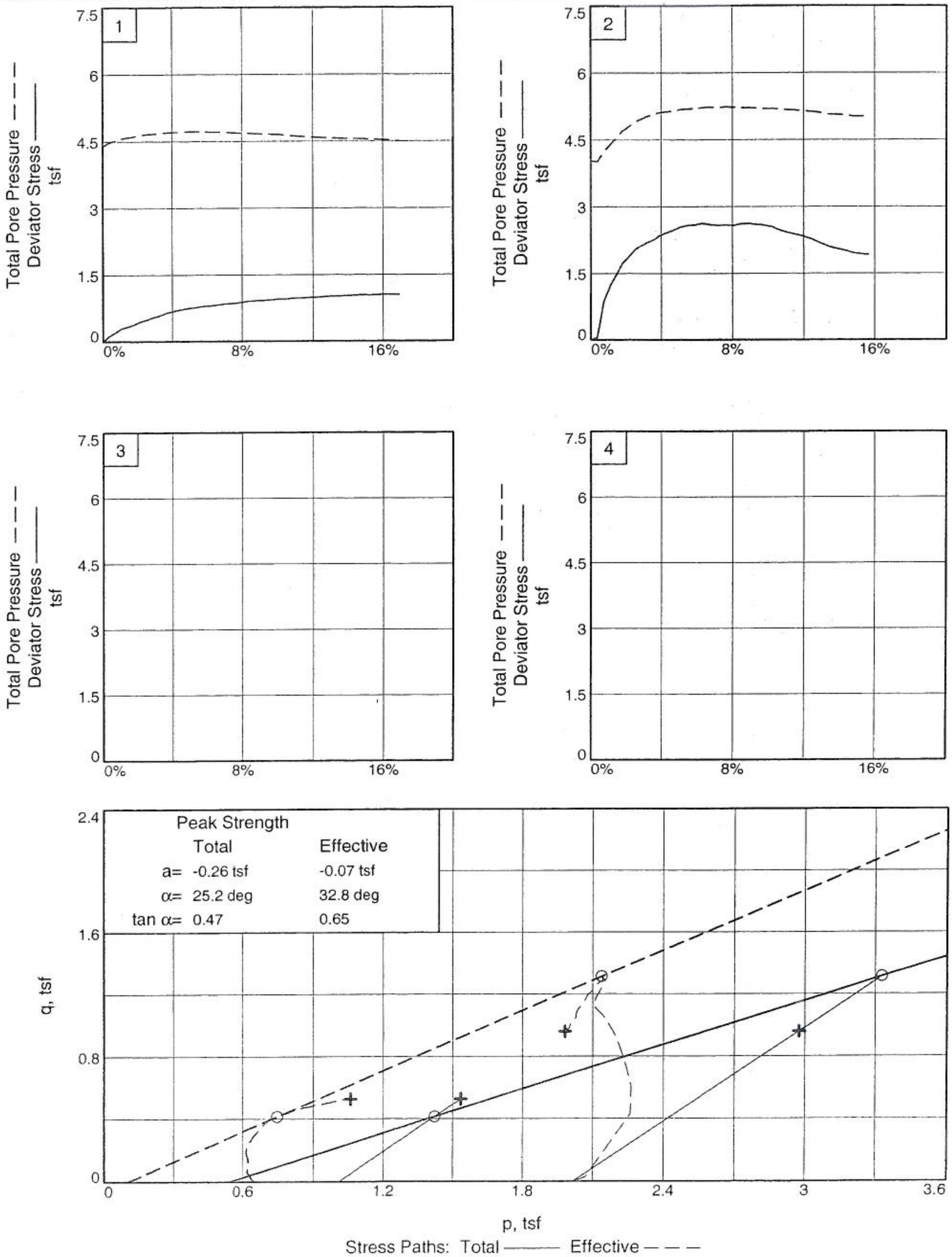

Peak Strength	Total	Effective
a=	-0.26 tsf	-0.07 tsf
α=	25.2 deg	32.8 deg
tan α=	0.47	0.65

Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1108 **Depth:** 18.0 **Sample Number:** P3
Project No.: 0121-3070.03 **Figure** _______

DLZ, INC.

APPENDIX IV

MSE Wall Bearing Capacity and Stability Calculations
MSE Wall Global Stability Analysis Results
MSE Wall Settlement Calculations
Downdrag Calculations
Drilled Shaft – Side Friction and End Bearing Calculations

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CLIENT CH2M Hill
PROJECT SCI - 823 Portsmouth Bypass
SUBJECT Fairgrounds Rd. Structures
Wall Properties / Soil Properties

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 21
COMP. BY [illegible] DATE 8-13-0[illegible]
CHECKED BY DAA DATE 8-31-07

Wall No 1 East Wall - Fairgrounds Rd

*Assumed Leveling Pad Elevation = 563'

- Ramp C Proposed Gr = 594.5' H = 31.5' (full height)
- S.R. 823 Proposed Gr = 597.0' H = 34.0' (Includes embedment)
- Ramp B Proposed Gr = 596.5' H = 33.5'

Wall No 2 West Wall - Fairgrounds Rd.

*Assumed Leveling Pad Elevation = 562'

- Ramp C Proposed Gr = 591.0' H = 29.0' full height
- SR 823 Proposed Gr = 594.0' H = 32.0' (Includes embedment)
- Ramp B Proposed Gr = 593.5' H = 31.5'

* For stability analyses:

* Wall # 1 profile based upon borings B-47 & B-1146

LP = 563'
Bot of leveling Pad excavat
el. 561.5'
el. 559.2 ∇
A-4b / A-6a C = 2350 $\phi' = 29°$
el. 557.2'
A-4b $c = 0$ $\phi' = 29°$
el. 554.5'
TOP OF ROCK

* Wall # 2 profile based upon borings B-45 & B-1113

LP = 562'
Bot of leveling Pad exca.
el. 560.5'
A-6b / A-2-6 C = 1500 $\phi = 0$
el. 550* ∇ $c' = 0$ $\phi' = 29$
el 548.0'
A-2-4 $c = 0$ $\phi = 30°$
el. 544.5'
Top of Rock

* from piezometer reading in B-46

DLZ

SUBJECT	Client CH2M Hill	JOB NUMBER	0121-3070.03
	Project SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 21
	Item MSE Wall Stability	COMP. BY	SJR DATE 8-13-07
	Wall No. 1, Fairground Road	CHECKED BY	DAA DATE 8-31-07
	Full Height MSE wall, based on B-47 & B-1146		

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=31.0'
2 Sssume bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5

Wall Properties

H+D	=	34	feet
γ_{mse}	=	120	pcf
L	=	37.4	feet
L factor	=	1.10	
ϕ	=	30	deg

Foundational Soil Properties

c	=	2350	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

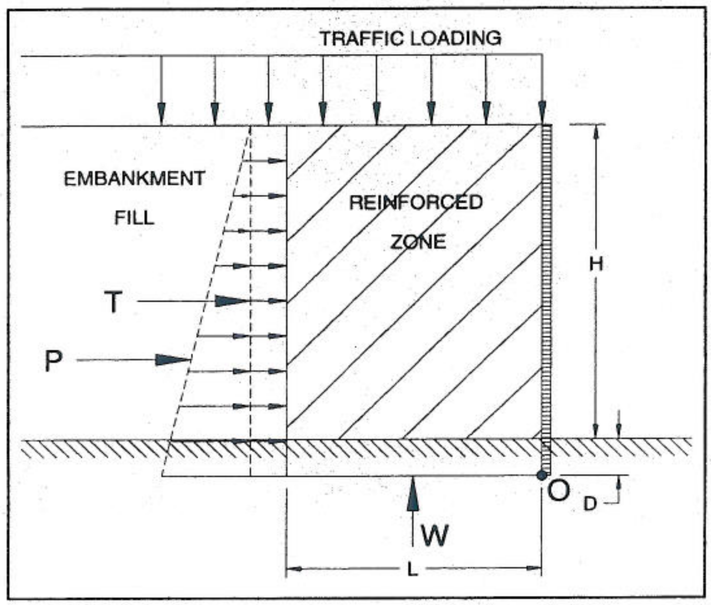
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 25,582 lbs per foot of wall



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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.37

P_r = 56,459 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 87,890 lbs per foot of wall

Use Drained Value

$FS = \frac{P_r}{P_a}$

Calculated	Required
FS = **2.21**	FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,853,470 lb-ft

$\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 305,184 lb-ft

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

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturnin\ g}}$

Calculated	Required
FS = **9.35**	FS = 2.00

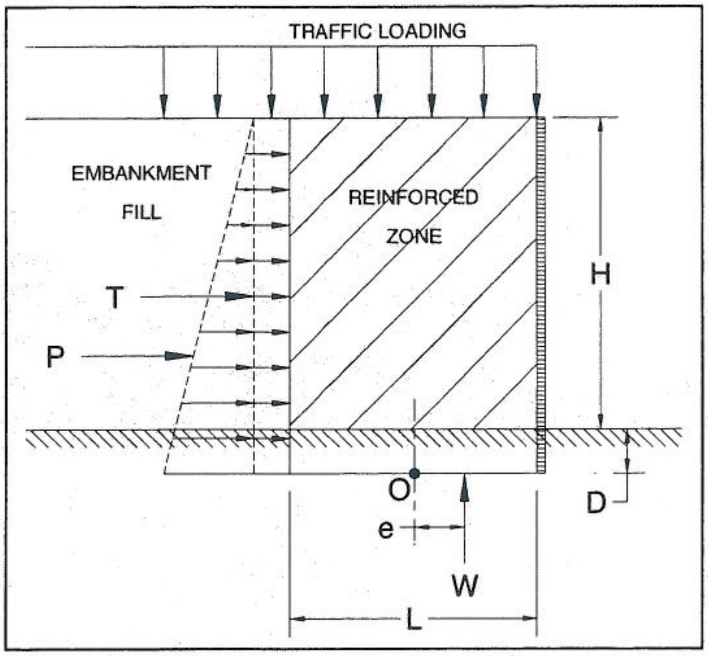
Resistance Against Overturning is **OK**

DLZ	SUBJECT	Client CH2M Hill	JOB NUMBER	0121-3070.03
		Project SCI-823 Portsmouth Bypass	SHEET NO.	3 OF 21
		Item MSE Wall Bearing Capacity	COMP. BY	SAR DATE 8-13-07
		Wall No. 1, Fairground Road	CHECKED BY	DAA DATE 8-31-07

Full Height MSE wall, based on B-47 & B-1146

BEARING CAPACITY OF A MSE WALL

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



Effective Bearing Pressure

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

σ_v = 4,806 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 12,252 psf

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

q_{ALL} = 4,901 psf

Factor of Safety = **2.55** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 21,567 psf

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

q_{ALL} = 8,627 psf

Factor of Safety = **4.49** **OK**

Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2350	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	37.4	ft	Length of MSE reinforcement
L factor	=	1.1		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	34	ft	
H	=	31	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	11.333	ft	Moment arm
Γ Wt	=	17	ft	Moment arm
B'	=	33.62	ft	
γ'	=	57.6	pcf	
W_t		8,976	lb/ft of wall	Weight from traffic
W_{mse}	=	152,592	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.89 ft

Kern

e < L/6 = 6.23 ft

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=29.0'
2 Sssume bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5

Wall Properties

H+D	=	32	feet
γ_{mse}	=	120	pcf
L	=	35.2	feet
L factor	=	1.10	
ϕ	=	30	deg

Foundational Soil Properties

c	=	1500	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.1)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 22,810 lbs per foot of wall

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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.37

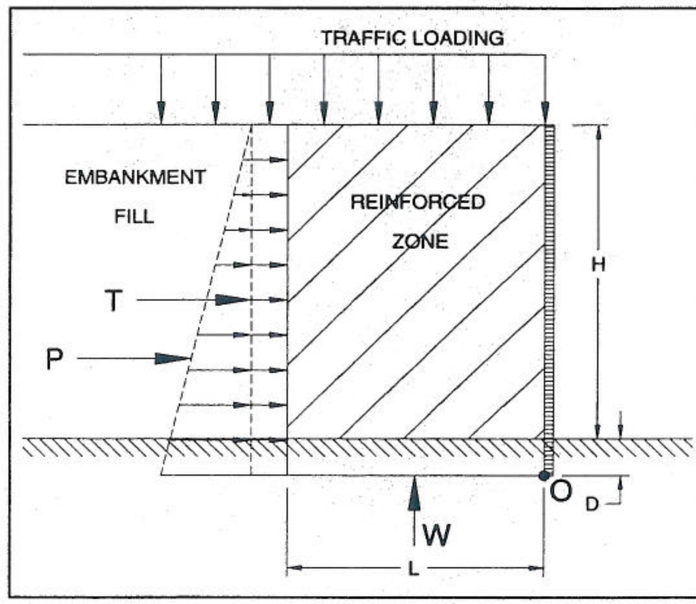
P_r = 50,012 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 52,800 lbs per foot of wall

Use Drained Value



$FS = \frac{P_r}{P_a}$

	Calculated	Required
	FS = **2.19**	FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

* Summation of Moments about point "O" (base of wall).
* Traffic loading is neglected in resisting forces

$\Sigma M_{resisting}$ = 2,378,957 lb-ft

$$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 256,819 lb-ft

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

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$

	Calculated	Required
	FS = **9.26**	FS = 2.00

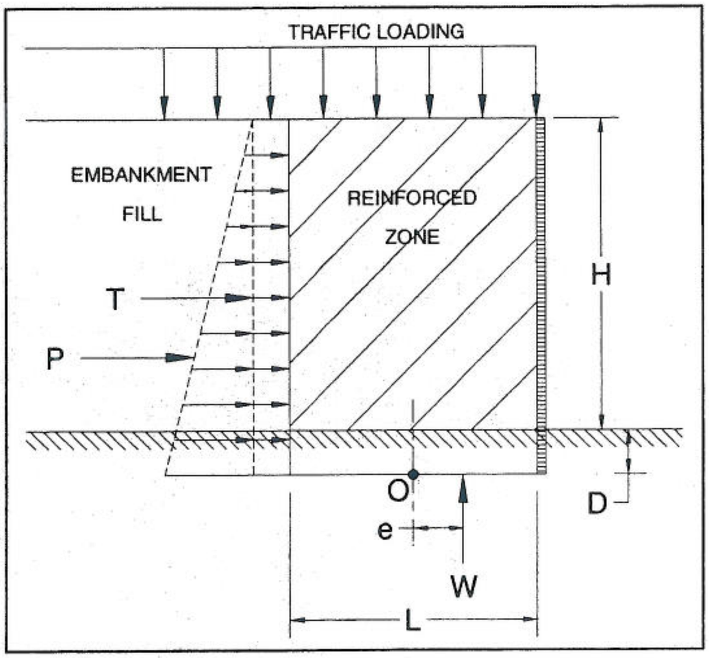
Resistance Against Overturning is **OK**

DLZ	SUBJECT			
	Client	CH2M Hill	JOB NUMBER	0121-3070.03
	Project	SCI-823 Portsmouth Bypass	SHEET NO.	5 OF 21
	Item	MSE Wall Bearing Capacity	COMP. BY	SJK DATE 8-13-07
	Wall No. 2, Fairground Road		CHECKED BY	DAA DATE 8-31-07

Full Height Wall

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1500	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

***L based on H+D=32'**

ω_t	=	240	psf	Traffic loading
L=B	=	35.2	ft	Length of MSE reinforcement
L factor	=	1.1		Length factor-range (0.7 - 1.1)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	32	ft	
H	=	29	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	10.667	ft	Moment arm
Γ Wt	=	16	ft	Moment arm
B'	=	31.62	ft	
γ'	=	57.6	pcf	
W_t		8,448	lb/ft of wall	Weight from traffic
W_{mse}	=	135,168	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,542 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 7,883 psf

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

q_{ALL} = 3,153 psf

Factor of Safety = **1.74** **No Good**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 20,453 psf

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

q_{ALL} = 8,181 psf

Factor of Safety = **4.50** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.79 ft

Kern

e < L/6 = 5.87 ft

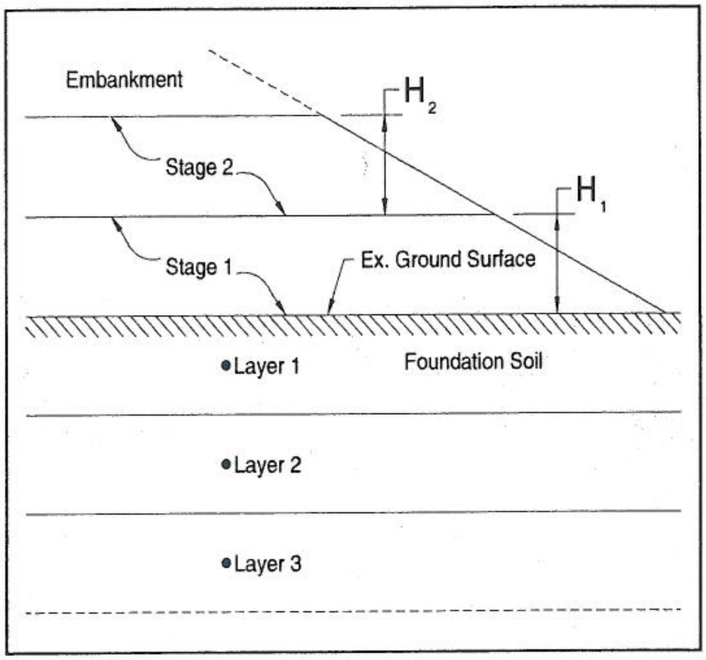
DLZ

SUBJECT			
Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	6 OF 21
Item	Undrained Strength Analysis - Staged Const.	COMP. BY	SAK DATE 8-13-07
MSE wall No. 2, Fairground Road		CHECKED BY	DAA DATE 8-30-07

Determine Increase in Undrained Shear Strength Due to Consolidation

Undrained Strength Analysis - Staged Construction

Ref: Ladd, Charles C. (1991). "Stability Evaluation During Staged Construction." *The Twenty-Second Karl Terzaghi Lecture.*, Journal of Geotechnical Engineering, ASCE, 117(4), 540-615



Increase in Undrained Shear Strength from consolidation

$$\boxed{c_u = c_{ui} + \Delta\sigma' \cdot \tan(\phi_{cu})}$$

Where: c_{ui} Initial undrained shear strength, UU or q_u testing

ϕ_{cu} Determined from CIU testing

$\Delta\sigma'$ Effective stress increase due to embankment loading

$$\boxed{\Delta\sigma' = (H_n \cdot \gamma_{emb}) \cdot U}$$

Where: U Average degree of consolidation (%)

H_n Height of Embankment, Stage n (ft)

Embankment Fill

γ_{fill} **120** pcf

Top of leveling pad el. 562.0'

Bot. of excavation el. 560.5

Depths measured from bottom of leveling pad excavation, below MSE retaining wall

Stage 1 Embankment First Stage Embankment Height H_1= **19.0** Average Percent Consolidation U= **90%**

Depth	Soil Type	Initial Undrained Shear Strength, c_{ui} (psf)	$\Delta\sigma'$ (psf)	ϕ_{cu} (deg)	Δc_u (psf)	c_u (psf), *After Consolidation*	Percent Increase
0-12.5	A-6b/A-2-6	1500	2052	15.0	550	2050	37%

Stage 2 Embankment Second Stage Embankment Height H_2= **8.0** Average Percent Consolidation U= **90%**

0-12.5	A-6b/A-2-6	2050	864	15.0	232	2282	11%

Stage 3 Embankment Third Stage Embankment Height H_3= **2.0** Average Percent Consolidation U= **0%**

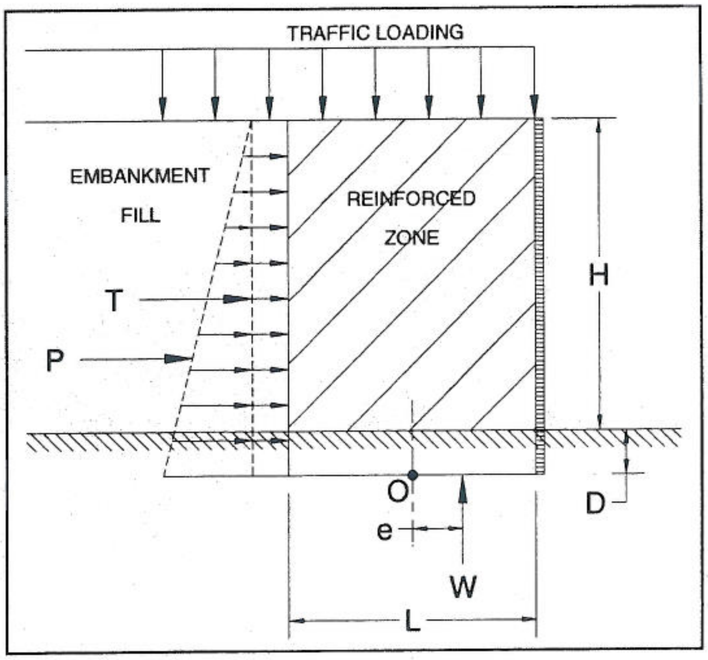
0-12.5	A-6b/A-2-6	2282	0	15.0	0	2282	0%

SUBJECT	Client CH2M Hill	JOB NUMBER	0121-3070.03
	Project SCI-823 Portsmouth Bypass	SHEET NO.	7 OF 21
	Item MSE Wall Bearing Capacity	COMP. BY	SAR DATE 8-13-07
	Wall No. 2, Fairground Road	CHECKED BY	DAA DATE 8-30-07

Stage 1, H+D=22', based on B-45 & B-1113

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	1500	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

***L based on H+D=32'**

ω_t	=	240	psf	Traffic loading
L=B	=	**35.2**	ft	Length of MSE reinforcement
L factor	=	1.1		Length factor-range (0.7 - 1.1)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	22	ft	
H	=	19	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	7.3333	ft	Moment arm
Γ Wt	=	11	ft	Moment arm
B'	=	33.44	ft	
γ'	=	57.6	pcf	
W_t		8,448	lb/ft of wall	Weight from traffic
W_{mse}	=	92,928	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_V = 3,032 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 7,883 psf

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

q_{ALL} = 3,153 psf

Factor of Safety = **2.60** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 21,467 psf

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

q_{ALL} = 8,587 psf

Factor of Safety = **7.08** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 0.88 ft

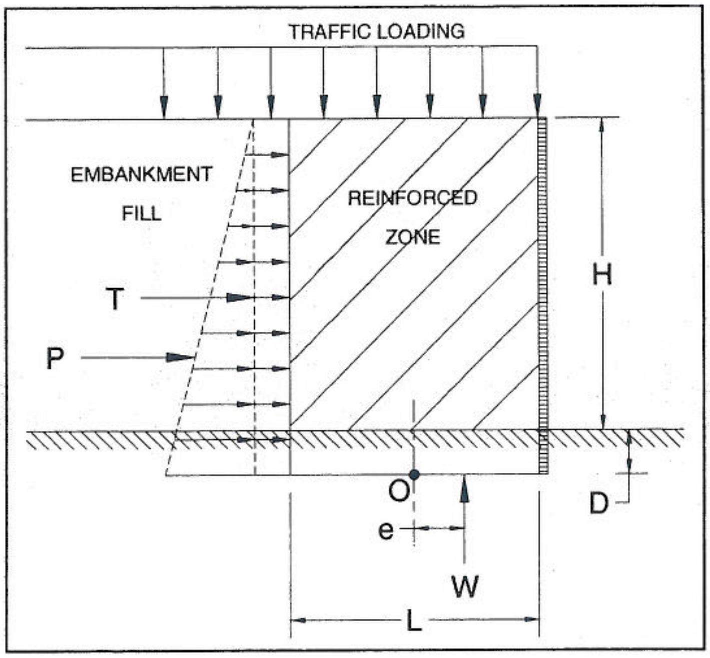
Kern

e < L/6 = 5.87 ft

Stage 2, H+D=27'+3'=30', based on B-45 & B-1113

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2050	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

***L based on H+D=32'**

ω_t	=	240	psf	Traffic loading
L=B	=	**35.2**	ft	Length of MSE reinforcement
L factor	=	1.1		Length factor-range (0.7 - 1.1)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	30	ft	
H	=	27	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	10	ft	Moment arm
Γ Wt	=	15	ft	Moment arm
B'	=	32.04	ft	
γ'	=	57.6	pcf	
W_t		8,448	lb/ft of wall	Weight from traffic
W_{mse}	=	126,720	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,219 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 10,710 psf

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

q_{ALL} = 4,284 psf

Factor of Safety = **2.54** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 20,687 psf

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

q_{ALL} = 8,275 psf

Factor of Safety = **4.90** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.58 ft

Kern

e < L/6 = 5.87 ft

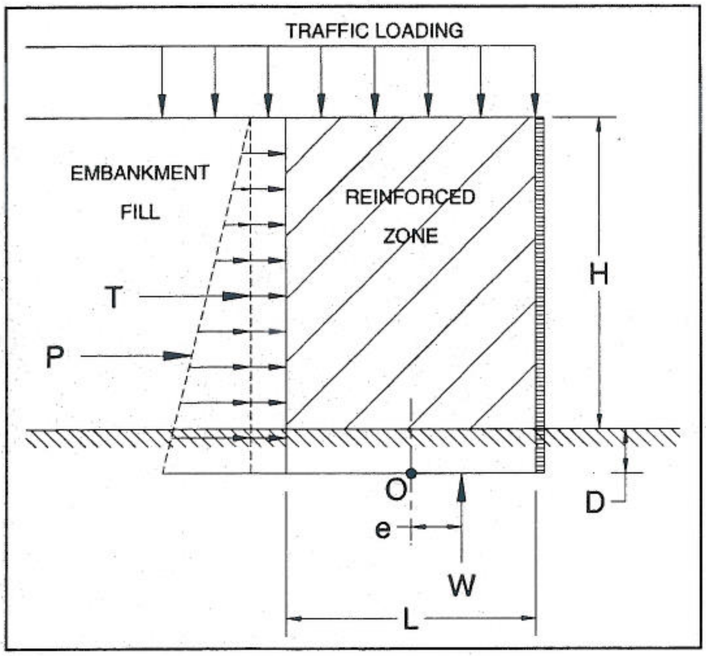
DLZ

SUBJECT			
Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	9 OF 21
Item	MSE Wall Bearing Capacity	COMP. BY	SAR DATE 8-13-07
Wall No. 2, Fairground Road		CHECKED BY	DAA DATE 8-31-07

Stage 3, H+D=29'+3'=32', based on B-45 & B-1113

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	2282	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

***L based on H+D=32'**

ω_t	=	240	psf	Traffic loading
L=B	=	**35.2**	ft	Length of MSE reinforcement
L factor	=	1.1		Length factor-range (0.7 - 1.1)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	32	ft	
H	=	29	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	10.667	ft	Moment arm
Γ Wt	=	16	ft	Moment arm
B'	=	31.62	ft	
γ'	=	57.6	pcf	
W_t		8,448	lb/ft of wall	Weight from traffic
W_{mse}	=	135,168	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,542 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 11,902 psf

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

q_{ALL} = 4,761 psf

Factor of Safety = **2.62** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 20,453 psf

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

q_{ALL} = 8,181 psf

Factor of Safety = **4.50** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	27.86
N_q	1.00	N_q	16.44
N_γ	0.00	N_γ	19.34

Eccentricity of Resultant Force

e = 1.79 ft

Kern

e < L/6 = 5.87 ft

DLZ — ENGINEERS • ARCHITECTS • SCIENTISTS, PLANNERS • SURVEYORS	CLIENT: CH2M Hill	PROJECT NO.: 0121-3070.03	
	PROJECT: SCI-823 Portsmouth Bypass	SHEET NO.: 10	OF: 21
	SUBJECT: Fairgrounds Rd Structures	COMP. BY: SAK	DATE: 8-15-07
	Staged Construction Details	CHECKED BY: DAA	DATE: 8-31-07

* Based on bearing capacity calculations, staged construction is required for Wall No. 2

- Height of 1st stage; $H_1 = 19.0'$

  Maximum excess pore pressure; $u_e = 19.0'(120\ pcf) = 2280\ psf = 15.8\ psi$

  * Prior to placing 2nd stage, excess pore pressures should be allowed to dissipate to $U = 90\%$.

  $$u_{e_{90}} = (1-0.90)(15.8\ psi) = \underline{1.6\ psi}$$

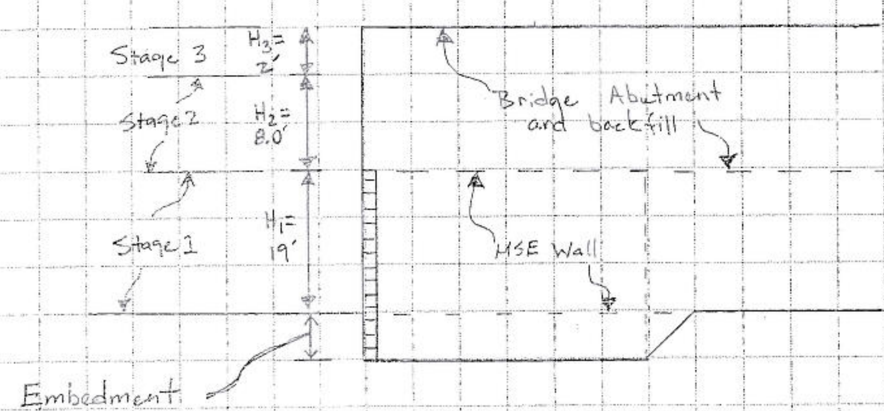
- Height of 2nd stage; $H_2 = 8.0'$

  $$u_e = 8.0'(120\ pcf) = 960\ psf = 6.7\ psi$$

  * Prior to placing Final (3rd Stage), excess pore pressures should be allowed to dissipate to $U = 90\%$

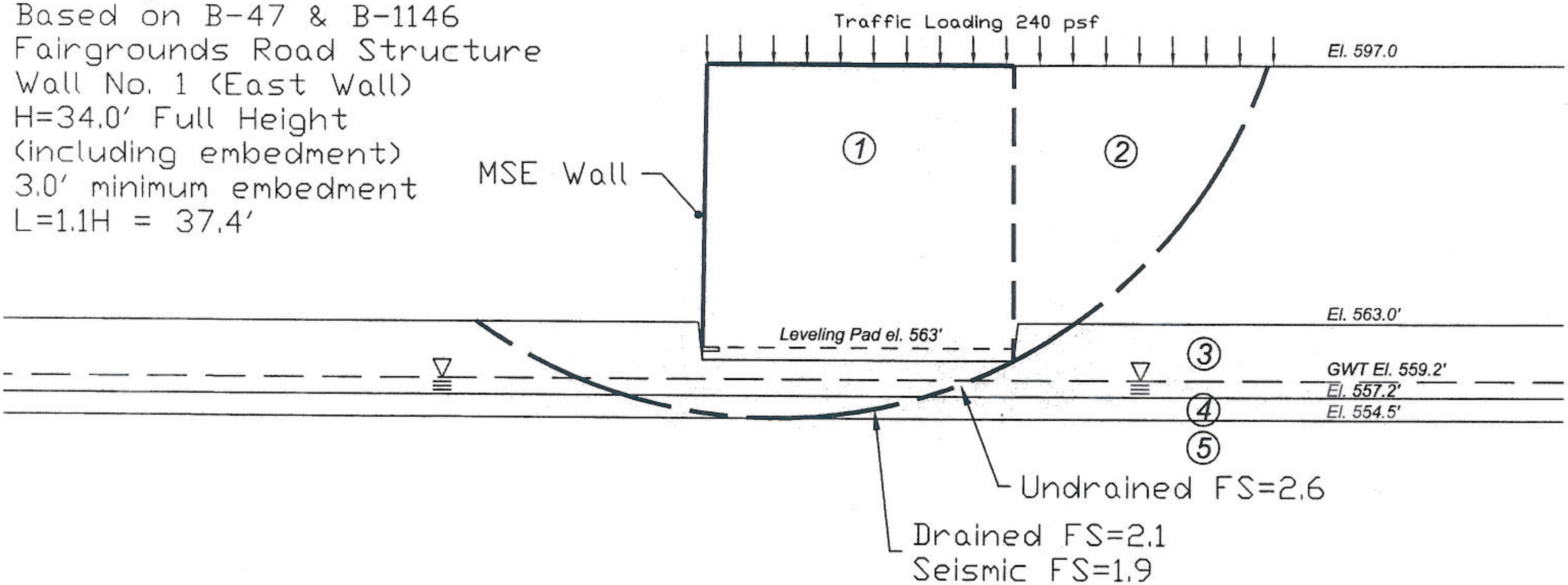
  $$u_{e_{90}} = (1-0.90)(6.7\ psi) = \underline{0.7\ psi}$$

Wall No. 2 Only



Material	Consistency	Soil Type	Undrained c (psf)	Undrained Φ (deg)	Drained c' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-4a/A-6a	2350	0	0	29	120
Material 4	Loose	Silt	0	29	0	29	120
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-47 & B-1146
Fairgrounds Road Structure
Wall No. 1 (East Wall)
H=34.0' Full Height
(including embedment)
3.0' minimum embedment
L=1.1H = 37.4'



Sheet 11 of 21

US-23 Interchange
Fairground Road Wall No. 1
Based on Borings B-47 & B-1146

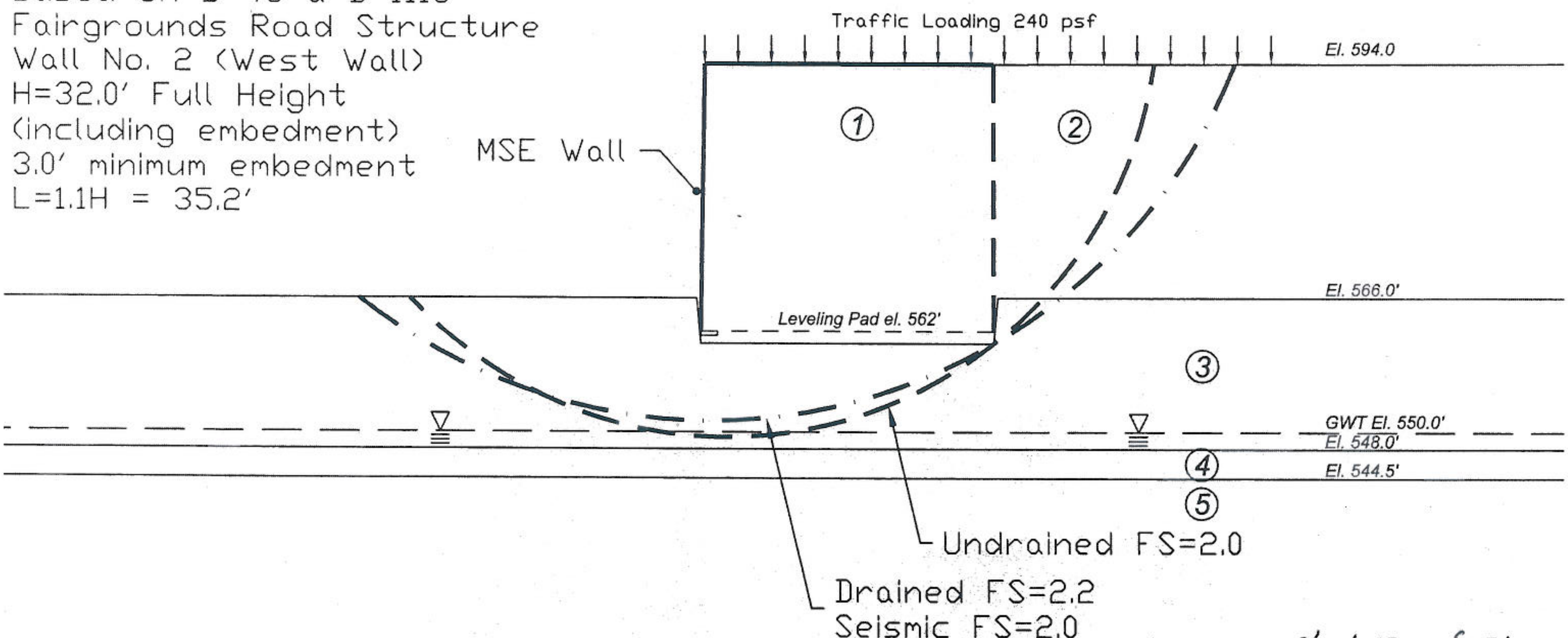
MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 8/14/07

Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-6b/A-2-6	1500	0	0	29	120
Material 4	Loose	A-2-4	0	30	0	30	120
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-45 & B-1113
Fairgrounds Road Structure
Wall No. 2 (West Wall)
H=32.0' Full Height
(including embedment)
3.0' minimum embedment
L=1.1H = 35.2'



Sheet 12 of 21

US-23 Interchange
Fairground Road Wall No. 2
Based on Borings B-45 & B-1113

MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC. SJR | DATE 8/14/07

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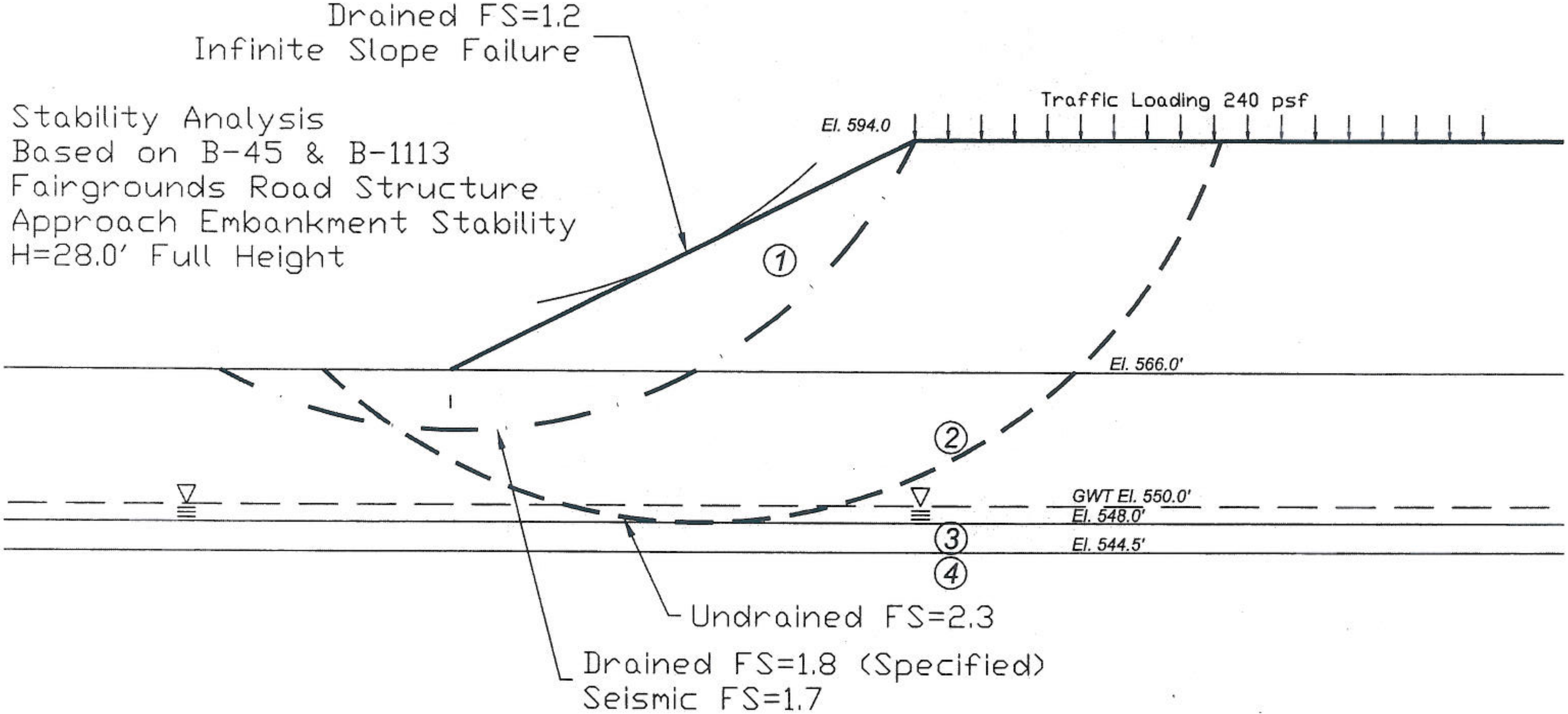
Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c′ (psf)	Drained φ′ (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	V. Stiff	A-6b/A-2-6	1500	0	0	29	120
Material 3	Loose	A-2-4	0	30	0	30	120
Material 4		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-45 & B-1113
Fairgrounds Road Structure
Approach Embankment Stability
H=28.0′ Full Height



Sheet 13 of 21

US-23 Interchange
Fairground Road Wall Approach Embankment
Based on Borings B-45 & B-1113

EMBANKMENT STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 8/14/07

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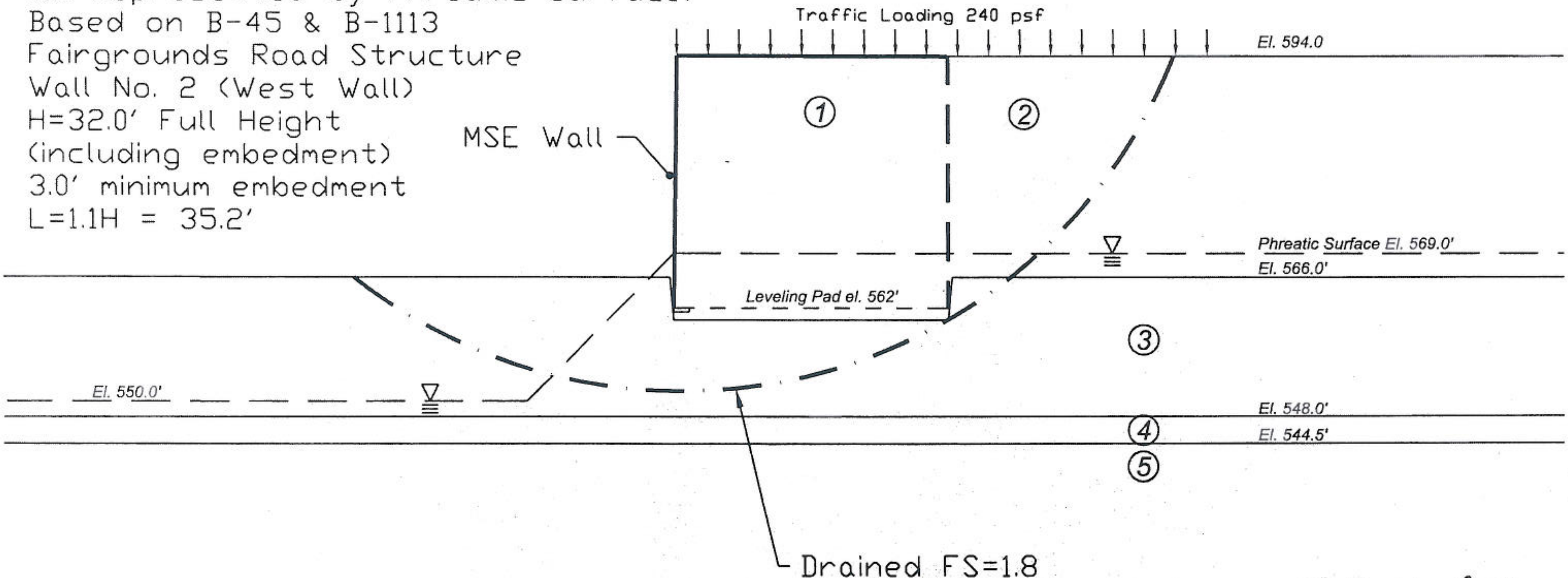
Material	Consistency	Soil Type	Undrained c (psf)	Undrained Φ (deg)	Drained c' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-6b/A-2-6	1500	0	0	29	120
Material 4	Loose	A-2-4	0	30	0	30	120
Material 5		Bedrock	10000	45	10000	45	145

EFFECTIVE STRESS ANALYSIS
Determine stability at theoretical
Maximum pore pressure following staged
construction guidelines.
(u_e Represented by Phreatic Surface)
Based on B-45 & B-1113
Fairgrounds Road Structure
Wall No. 2 (West Wall)
H=32.0' Full Height
(including embedment)
3.0' minimum embedment
L=1.1H = 35.2'



Sheet 14 of 21

US-23 Interchange
Fairground Road Wall No. 2
Based on Borings B-45 & B-1113

MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 8/15/07

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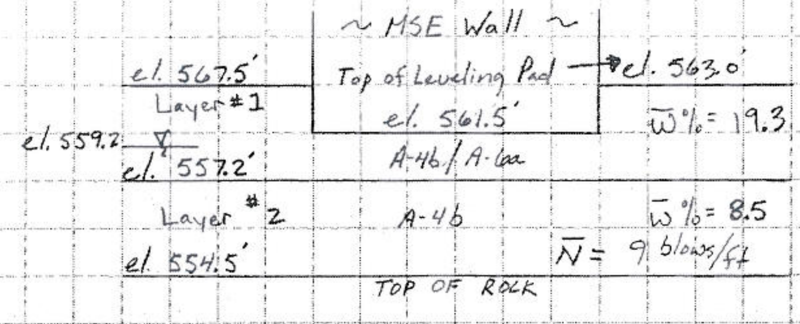
CLIENT: CH2 M Hill | PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass | SHEET NO. 15 OF 21
SUBJECT: Fairgrounds Rd Structures | COMP. BY SJK DATE 8-14-0
Consolidation / Settlement under MSE wall | CHECKED BY DAA DATE 8-31-0

$H = 597.0 - 567.5' = 29.5'$

• Wall No. 1 — East Wall - Fairgrounds Rd.

* moisture content from B-47
* N-values from B-47
* Profile based on B-47 and B-1140



} Compacted MSE Fill Assume Incompressible

} $C_c = 0.19$ $e_o = 0.52$ *See Below #1

} * See below #2

#1 * Sample Calculation: $\bar{\omega} = 19.3\%$, $\overline{LL} = 24.5\%$, $\overline{PL} = 18.0\%$, $\overline{PI} = 6.5$

*Assume soil is normally consolidated.
* Assume soil is saturated.

$$e_o = \frac{G_s\,\omega}{100} = \frac{2.70\,(19.3)}{100} = 0.52$$

$$C_c = \frac{\omega}{100} = \frac{19.3}{100} = 0.19$$ Ref [FHWA NHI-00-045] "Soils and Foundation Workshop Ref Manual"

#2 * Sample Calculation: Ref [FHWA NHI-00-045]

Average N-value = 9 blows/ft

$\sigma_{ov} = 1290$ psf $\quad N'/N \simeq 1.1 \longrightarrow N' = N(1.1) = 9(1.1) = 9.9$ say 10

$C' \doteq 30$

The computer program EMBANK requires input for C_c and e_o. To evaluate the settlement of granular layers, we must compute equivalent consolidation parameters from C'.

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

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

When $C' = 30 \longrightarrow$ Use $C_c = \frac{2}{30} = 0.07$ when $e_o = 1.0$

* From EMBANK

Total Settlement = 3.3" at $x = 59'$ and = 0.3" at $x = 0'$

$$\text{Differential Settlement, } DS = \frac{(3.3'' - 0.3'')\,1/12}{59' - 0'} = 0.004$$

SJK

Sheet 16 of 21
SJR 8-15-07
DAA 8-31-07

Fairground Road Wall No 1

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

Project Name : SCI-823
File Name : FRW1
Date : 8/14/10

Client : CH2M Hill
Project Manager : Nix
Computed by : sjr

Settlement for X-Direction

Embank. slope, x direc. = 59.00 (ft)
y direc. = 00.10 (ft)
Embankment top width = 273.00 (ft)
Embankment bottom width = 391.00 (ft)
Ground Surface Elev. = 567.50 (ft)
Water table Elev. = 559.20 (ft)

Height of fill H = 29.50 (ft)
Unit weight of fill = 120.00 (pcf)
p load/unit area = 3540.00 (psf)
Foundation Elev. = 567.50 (ft)
Unit weight of Wat. = 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	6.0	-----	-----	-----	120.00	----	----
2	COMP.	4.3	0.190	0.000	0.000	120.00	2.65	0.52
3	COMP.	2.7	0.070	0.000	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	4.30	559.35	978.00	978.00
3	2.70	555.85	1188.96	1188.96

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 59.00 Stress (psf)	Sett. (in.)	X = 118.00 Stress (psf)	Sett. (in.)	X = 177.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	73.58	0.20	1683.33	2.80	1755.80	2.88	1756.07	2.88
3	107.27	0.04	1653.75	0.43	1759.24	0.45	1760.03	0.45
		------		------		------		------
		0.25		3.23		3.33		3.33
		Say 0.3		Say 3.3"				

Layer	X = 236.00 Stress (psf)	Sett. (in.)	X = 295.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	1756.04	2.88	1755.14	2.88
3	1759.93	0.45	1757.39	0.45
		------		------
		3.33		3.33

proposed grade = 597.0'
existing grade = 567.5'
Assumed bottom of excavation = 561.5'

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: CHZM Hill
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Fairgrounds Rd. Structures
Consolidation / Settlement under MSE wall

PROJECT NO. 0121-3070.03
SHEET NO. 17 OF 21
COMP. BY SJK DATE 8-15-
CHECKED BY DAA DATE 8-31-0

$H = 594.0 - 565.6 = 28.4'$

• Wall No. 2 West Wall - Fairgrounds Rd.

* Profile based on B-46

~ MSE Wall ~

Top of Leveling Pad el. 562.0'

el. 565.6' — el. 560.5' } Compacted MSE Fill, Assume Incompressible

550 ▽ Layer #1 A-6b } $C_c = .24$ $e_o = .692$ from B-46

el. 555.1'

Layer #2 A-2-6 $\bar{N} \approx 4$ $N' = 4$ $C' \approx 42$ } equivalent $C_c = 0.05$ * See sample calculation, pg 15

el. 547.6'

TOP OF ROCK

* Assume A-6b layer is normally consolidated. Based upon in-situ moisture content, this would be a prudent assumption.

* From EMBANK

Total settlement = 4.8" @ 56.8' and 0.4" @ 0'

Differential Settlement, $DS = \dfrac{(4.8" - 0.4")(1'/12")}{56.8'} = 0.006$

SJK

FRW2

Sheet 18 of 21
SJR 8-15-07 DAA 8-31-07

Assumes Normally consolidated Soil

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name	: SCI-823		Client	: CH2M Hill	
File Name	: FRW2		Project Manager	: Nix	
Date	: 8/14/10		Computed by	: sjr	

Settlement for X-Direction

Embank. slope, x direc.	=	56.80 (ft)	Height of fill H	=	28.40 (ft)
y direc.	=	00.10 (ft)	Unit weight of fill	=	120.00 (pcf)
Embankment top width	=	331.00 (ft)	p load/unit area	=	3408.00 (psf)
Embankment bottom width	=	444.60 (ft)	Foundation Elev.	=	565.60 (ft)
Ground Surface Elev.	=	565.60 (ft)			
Water table Elev.	=	550.00 (ft)	Unit weight of Wat.	=	62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	5.1	-----	-----	-----	120.00	----	----
2	COMP.	5.4	0.240	0.040	0.000	120.00	2.65	0.69
3	COMP.	7.5	0.050	0.050	0.050	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	5.40	557.80	936.00	936.00
3	7.50	551.35	1710.00	1710.00

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	28.40 Sett. (in.)	X = Stress (psf)	56.80 Sett. (in.)	X = Stress (psf)	85.20 Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	70.38	0.29	848.38	2.58	1620.45	4.02	1688.52	4.12
3	131.36	0.07	849.87	0.39	1565.69	0.64	1687.74	0.67
		------		------		------		------
		0.36		2.97		4.65		4.79

Say 0.4"

Say 4.8"

Layer	X = Stress (psf)	113.60 Sett. (in.)	X = Stress (psf)	142.00 Sett. (in.)
1	INCOMP.	INCOMP.		
2	1689.74	4.12	1689.95	4.12
3	1694.31	0.67	1695.54	0.67
		------		------
		4.80		4.80

ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ

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CLIENT CH2M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Fairgrounds Rd Structures
Time-rate of consolidation

PROJECT NO. 0121-3070.03
SHEET NO. 19 OF 21
COMP. BY SAK DATE 8-15-0
CHECKED BY DAA DATE 8-31-

Wall No. 1 East Wall - Fairgrounds Rd.

Based on boring B-47

$\overline{LL} \simeq 25$, $H_v = 4.3'/2$ *Assumes double drainage

$H_v = 2.2'$

$^*c_v \simeq 0.65$ ft²/day

*Ref {FHWA HI-97-021, figure 9-5, "Subsurface Investigations"}

$$t_{90} = \frac{T_v (H_v)^2}{c_v} = \frac{(0.848)(2.2')^2}{0.65\ ft^2/day} = 6.3 \text{ days} \quad \text{say } 7 \text{ days}$$

Wall No. 2 West Wall - Fairgrounds Rd

Based on boring B-45 & B-46

$\overline{LL} \simeq 34$ (B-46) $H_v = 5.4/2 = 2.7'$ *Assumes double drainage

$^*c_v \simeq 0.35$ ft²/day

$$t_{90} = \frac{T_v (H_v)^2}{c_v} = \frac{(0.848)(2.7')^2}{0.35\ ft^2/day} = 17.7 \text{ days} \quad \text{say } 18 \text{ days}$$

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CLIENT CH2M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Downdrag Forces on Piles
* Calculation of waiting period

PROJECT NO. 0121-3070.03
SHEET NO. 20 OF 21
COMP. BY SAK DATE 8-15-0
CHECKED BY DAA DATE 8-31-0

MSE Wall No. 1

Total Consolidation Settlement, $\delta_c = 2.8''$

To limit remaining settlements to 0.4" or less;

$\frac{0.4''}{2.8''} = 0.14$ $U_{Req} = 1 - 0.14 = 0.86$ Say $U = 90\%$

*From time-rate calculations, $t_{90} = 7$ days

MSE Wall No. 1

Total Consolidation Settlement, $\delta_c = 4.0''$

$\frac{0.4''}{4.0} = 0.10$ $U_{Req} = 1 - 0.10 = 0.90$ $U = 90\%$

* From time-rate calculations, $t_{90} = 18$ days.

* From testing on rock cores from borings B-45, B-46, B-47
Use lower bound; $q_u = 1{,}971$ psi

End bearing: FHWA-IF-99-025 Eq. 11.6 $q_{max}(MPa) = 4.83\,[q_u(MPa)]^{0.51}$
For RQD between 70-100 percent

$q_u = 1{,}971 \text{ psi} = 13.6 \text{ MPa}$

$q_{max} = 4.83\,[13.6\text{ MPa}]^{0.51} = 18.3 \text{ MPa} = 2{,}652 \text{ psi} = 382 \text{ ksf}$

$q_{allow} = \frac{q_{max}}{F.S.} = \frac{382 \text{ ksf}}{3.0} = 127 \text{ ksf}$

* However, for this type and quality of Shale with Sandstone, we typically use:
$q_a = 40 \text{ ksf}$ (20 tsf)

Side Friction: FHWA-IF-99-025 Eq. 11.24

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

$f_c' \simeq 4500$ psi $\quad q_u \simeq 1{,}971$ psi $\quad$ * q_u governs

$f_{max} = 0.65\,(14.7\text{ psi})\left[\frac{1{,}971\text{ psi}}{14.7\text{ psi}}\right]^{0.5} = 110.6 \text{ psi}$

$f_{allow} = \frac{f_{max}}{F.S.} = \frac{110\text{ psi}}{3.0} = 37 \text{ psi} = 5280 \text{ psi}$

Use $f_{allow} = 3{,}750$ psf *Reduction for argillaceous rock

APPENDIX D

SCI-823-10.13
RAMP B OVER FAIRGROUND ROAD
VERTICAL CLEARANCES

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1593C Ramp B over Fairground\[RampB_Fairground_Vert_Clr.xls]Vertical Clearance

By: DGS Date: 8/1/2007
Checked: SKT Date: 9/26/2007 LEGEND:

User Input - Not Critical
User Input - Critical to Output

AASHTO Type 4 (54") Concrete I-Beams

PROFILE DATA - Fairground Road

Use existing pavement elevations as Fairground Road will not be reconstructed in this project

POINT	FAIRGROUND ROAD LOCATION	FAIRGROUND ROAD STATION	FAIRGROUND ROAD - EXISTING ELEV. @ POINT
1	E/Pavement SB	n/a	567.67
2	Centerline	n/a	567.90
3	E/Pavement NB	n/a	567.75

PROFILE DATA - RAMP B

Vertical Curve: PVC Sta. 2608+25.00; PVC Elev. 589.49; g1 0.93%; g2 2.98%; LVC 250; PVI Sta. 2609+50.00; PVI Elev. 590.65; PVT Sta. 2610+75.00; PVT Elev. 594.38

Vertical Curve: PVT Sta. 2610+75.00; PVT Elev. 594.38; g 2.98%; PVI Sta. 2613+75.00; PVI Elev. 603.32

Superelevation Data:

Station	Left Shoulder	Pavement	Right Shoulder
2604+08.84	-4.0%	7.1%	-7.1%
2612+25.25	-4.0%	7.1%	-7.1%

	RAMP B LOCATION			RAMP B	LT. SHOULDER		RT. SHOULDER	RAMP B - FINISHED
POINT	DESCRIPTION	STA.	OFF.*	PG ELEV.	X-SLOPE	PVMT X-SLOPE	X-SLOPE	GRADE @ POINT
1	RT. FASCIA BEAM	2610+76.19	7.84	594.42	-4.0%	7.1%	-7.1%	593.86
2	RT. FASCIA BEAM	2610+87.00	7.97	594.74	-4.0%	7.1%	-7.1%	594.17
3	RT. FASCIA BEAM	2610+98.54	7.87	595.08	-4.0%	7.1%	-7.1%	594.52

* For Offsets allow positive (+) to denote an offset to the right of the baseline and negative (-) to denote an offset to the left of the baseline

STRUCTURE DEPTH Haunch + Max. Top Flange = 4.0 in

POINT	BEAM DESCRIPTION	Slab	Haunch	Top Flange	Web	Bot. Flange	Splice	Total
1	AASHTO TYPE 4	8.50	4.00		54		-	66.50 in
2	AASHTO TYPE 4	8.50	4.00		54		-	66.50 in
3	AASHTO TYPE 4	8.50	4.00		54		-	66.50 in

VERTICAL CLEARANCE - RAMP B OVER FAIRGROUND RD.

POINT	LOCATION	RAMP B - FINISHED GRADE @ POINT	STRUCTURE DEPTH (in.)	BOT. BEAM ELEVATION	FAIRGROUND RD. - FINISHED GRADE @ POINT	VERTICAL CLEARANCE (ft.)	
1	RT. FASCIA BEAM	593.86	66.50	588.32	567.67	**20.65**	OK
2	RT. FASCIA BEAM	594.17	66.50	588.63	567.90	**20.73**	OK
3	RT. FASCIA BEAM	594.52	66.50	588.98	567.75	**21.23**	OK

APPENDIX E

Ohio Prestressers Association
51 Mallard Point Hebron Ohio 43025-9688 Phone: 740-928-2727 Email: mekllc@columbus.rr.com

July 11, 2007

Doug Stachler, P.E.
CH2M HILL - Columbus, OH Office
5775 Perimeter Drive, Suite 190
Dublin, OH 43017

Re: ODOT - Portsmouth Bypass Project - Prestressed Beam Design

Dear Doug:

Thank you for the opportunity to provide input for your prestressed concrete bridge design. Pursuant to your e-mail, and on behalf of my member PCI producers, Prestress Services Industries, LLC, and United Precast, Inc., I offer the following:

Bridge 1 - Ramp B Bridge over Fairground Rd.
Project Name: SCI-823-10.13 (Portsmouth Bypass)
PID: 79977
Bridge No. SCI-823-1593
SFN: 7306717
Span Length = 98'-10"
No. of Beams = 5
Beam Type: AASHTO Type 4 (54")
Concrete 28 day strength fc' = 7000 psi
Concrete strength @ release fci' = 5500 psi
No. of Strands = 49

Bridge 2 - SR-823 Bridge over Fairground Rd.
Project Name: SCI-823-10.13 (Portsmouth Bypass)
PID: 79977
Bridge No. SCI-823-1594
SFN: 7306725
Span Length = 101'-4"
No. of Beams = 9
Beam Type: Modified AASHTO Type 4 (60")
Concrete 28 day strength fc' = 7000 psi
Concrete strength @ release fci' = 5500 psi
No. of Strands = 49

Bridge 3 - Ramp C Bridge over Fairground Rd.
Project Name: SCI-823-10.13 (Portsmouth Bypass)
PID: 79977
Bridge No. SCI-823-1595
SFN: 7306733
Span Length = 106'-10"
No. of Beams = 5
Beam Type: Modified AASHTO Type 4 (60")
Concrete 28 day strength fc' = 7000 psi
Concrete strength @ release fci' = 5500 psi
No. of Strands = 50

1. Producing Type 4 I-Beams is no problem for either member producer.
2. Release strengths and 28 day strengths you propose will not add any additional cost to the beams.
3. The beams will be able to be delivered safely to the jobsite.

Both Ohio Prestressers Association members are looking forward to competing on this project when it comes to sale. If you need any additional information, please call.

Sincerely,
Ohio Prestressers Association

Mary Ellen Kimberlin
Executive Director

APPENDIX F

inter-office communication

to: James A. Brushart, District 9 Deputy Director **date:** Apr. 19, 2007

from: Timothy J. Keller, Administrator, Office of Structural Engineering **by:** Ananda Dharma, P.E.

subject: SCI-823-6.81; PID 19415; Bridge No. SCI-823-1593; Ramp B over Fairground Road; Revised Structure Type Study Review

Attn.: Thomas M. Barnitz, District 9 Production Administrator

We have briefly reviewed Revised Structure Type Study submission from CH2MHill for the proposed bridge along Ramp B over Fairground Road. Our comments are shown below.

General Comments

1. We agree that the proposed structure should consist of a single span composite prestressed concrete I-beams with reinforced concrete deck and semi-integral abutments supported on MSE walls. Also, see the next comment regarding the use of MSE walls.

2. The determination of the most suitable soil improvement alternative for the proposed MSE walls is contingent upon approval of the Wall Type Study which will be submitted in a separate IOC by Peter Narsavage. Please incorporate Mr. Narsavage's comments prior to proceeding with Preliminary Design.

3. Even though the reinforced concrete deck slab will be curved, the prestressed concrete I-beams will be placed parallel. Please make sure the 2" Preformed Expansion Joint Filler separating the concrete superstructure from the wingwalls will be placed parallel to the prestressed concrete I-beams. Refer to Standard Bridge Drawing No. SICD-1-96.

4. Please reduce the deck overhang as much as possible. We would like to encourage the Design Consultant to attend a seminar on State of Practice for Highly Skewed Bridges which will be held on April 24, 2007 at ODOT Central Office Auditorium. Additional information for the seminar and how to register can be found from the Office of Structural Engineering's website at the following website address: **http://www.dot.state.oh.us/se/skew/skew.htm**
The Design Consultant will find the seminar to be very informative because not only will it discuss the design/construction of skewed bridges, but also problems associated with the construction of deck overhang.

5. We could not verify the 30'-0" proposed horizontal clearance after referring to the ODOT's L&D Manual, Volume 1, Fig. 600-1. Please make sure that approval of horizontal clearance is obtained

from **ODOT - Office of Roadway Engineering Services** prior to proceeding.

6. The superelevation rate at the proposed Ramp B is 7% which corresponds to 35 mph Design Speed. What is the Design Speed of the northbound U.S.R. 23 at the proposed location? The proposed Ramp B will carry traffic exiting northbound U.S.R. 23 onto eastbound S.R. 823. Ramp B would preferably be designed with the same design speed as northbound U.S.R. 23. However, our office cannot comment on the alignment and profile grade because the geometric review of the proposed alignment will need to be reviewed by the **ODOT - Office of Roadway Engineering Services**. Please make sure that an approval from the **ODOT - Office of Roadway Engineering Services** prior to proceeding with the next design stage.

7. Include the Structure File Number in the Title block. Structure File Number for this bridge is **7306717**. For future projects, Structure File Number can be obtained by contacting Ms. Kathy J. Keller, Office of Structural Engineering, Bridge Inventory section (Phone: 614-752-9973)

Our office recommends that the District approves the Revised Structure Type Study submission subject to resolution of these comments. Your concurrence with the above comments submitted <u>in writing</u> constitutes compliance.

Nothing in these comments is to be construed as authorizing extra work for which additional compensation may be claimed. If you have reason to believe that these comments require work outside the limits of your Scope of Services, please contact this office before proceeding.

Should you have any questions concerning our review comments for the above referenced project, please contact our office.

TJK:JS:ad

c: John K. Wetzel, ODOT District 9
Lawrence A. Wills, ODOT District 9
Timothy J. Keller, Office of Structural Engineering
Jawdat Siddiqi, Office of Structural Engineering
Richard A. Bruce, Office of Roadway Engineering Services
file

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 04/26/07

Bridge SCI-823-1593: Ramp B over Fairground Road

PROJECT: SCI-823-10.13: Portsmouth Bypass PROJ. NO: 319861.08.03

REVIEWER: ODOT OSE - Ananda Dharma, P.E. PHASE: Type Study

Reference Page/Sheet No.	Review Comment	Designer Response
	ODOT Comments	
General	1. We agree that the proposed structure should consist of single span composite prestressed concrete I-beams with reinforced concrete deck and semi-integral abutments supported on MSE walls. Also, see the next comment regarding the use of MSE walls.	Will comply.
General	2. The determination of the most suitable soil improvement alternative for the proposed MSE walls is contingent upon approval of the Wall Type Study, which will be submitted in a separate IOC by Peter Narsavage. Please incorporate Mr. Narsavage's comments prior to proceeding with Preliminary Design.	Will comply. Per the Wall Type Study IOC from Peter Narsavage, dated April 23, 2007, ODOT OSE believes that MSE walls at the Fairground Road location can be built in two stages without any surcharging or ground improvement.
Site Plan (1/3)	3. Even though the reinforced concrete deck slab will be curved, the prestressed concrete I-beams will be placed parallel. Please make sure the 2" Preformed Expansion Joint Filler separating the concrete superstructure from the wingwalls will be placed parallel to the prestressed concrete I-beams. Refer to Standard Bridge Drawing No. SICD-1-96.	Will comply.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 04/26/07

Bridge SCI-823-1593: Ramp B over Fairground Road

PROJECT: SCI-823-10.13: Portsmouth Bypass PROJ. NO: 319861.08.03

REVIEWER: ODOT OSE – Ananda Dharma, P.E. PHASE: Type Study

Transverse Section (2/3)	4. Please reduce the deck overhang as much as possible. We would like to encourage the Design Consultant to attend a seminar on State of Practice for Highly Skewed Bridges, which will be held on April 24, 2007 at ODOT Central Office Auditorium. Additional information for the seminar and how to register can be found from the Office of Structural Engineering's website at the following website address: http://www.dot.state.oh.us/se/skew.htm. The Design Consultant will find the seminar to be very informative, because not only will it discuss the design/construction of skewed bridges, but also problems associated with the construction of deck overhang.	Will comply. 54" prestressed concrete I-beams were used for all three structures along Fairground Road. However, with span lengths ranging from 98'-10" at Ramp B to 106'-10" at Ramp C, this may require a highly reinforced design. During Preliminary Design Report development, we will investigate the use of deeper prestressed concrete I-beams at the SR-823 over Fairground Road and the Ramp C over Fairground Road bridges, because overhang dimensions will not be an issue and deck geometry will permit a wider top flange. For the Ramp B over Fairground Road bridge, the curved reinforced concrete deck geometry will not allow for a wider top flange, while controlling the overhang dimensions to acceptable values. During Preliminary Design Report development, we will investigate if 4-54" prestressed concrete I-beams will work, and if not, we will add an additional beam line.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 04/26/07

Bridge SCI-823-1593: Ramp B over Fairground Road

PROJECT: SCI-823-10.13: Portsmouth Bypass PROJ. NO: 319861.08.03

REVIEWER: ODOT OSE – Ananda Dharma, P.E. PHASE: Type Study

Site Plan (1/3)	5. We could not verify the 30'-0" proposed horizontal clearance after referring to ODOT's L&D Manual, Volume 1, Fig. 600-1. Please make sure that approval of horizontal clearance is obtained from **ODOT - Office of Roadway Engineering Services** prior to proceeding.	See attached documentation pertaining to design/posted speed and design year ADT along Fairground Road to justify the required 30'-0" clear zone distance to the MSE walls. Since the PAVR submittal, CH2M HILL has had discussions with the Scioto County Engineer's Office. The county stated that Fairground Road will be improved to 2-12' lanes and that the speed limit is 55 mph. With a design speed of 60 mph and an ADT greater than 3000 vpd, Fig 600-1E recommends a clear zone distance of 30'-0" when the ditch foreslope varies between 6:1 to 4:1. Due to additional culverts being added along Fairground Road, new ditches are being designed with foreslopes varying from 6:1 to 4:1; the steeper than 6:1 foreslope also provides the avoidance of utilities, while also using existing drainage structures. Providing lateral bridge clearance equal to the clear zone provides a safer roadway and allows for future improvements.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT   DATE: 04/26/07

Bridge SCI-823-1593: Ramp B over Fairground Road

PROJECT: **SCI-823-10.13: Portsmouth Bypass**   PROJ. NO: 319861.08.03

REVIEWER: ODOT OSE - Ananda Dharma, P.E.   PHASE: Type Study

Transverse Section (2/3)	6. The superelevation rate at the proposed Ramp B is 7%, which corresponds to 35 mph Design Speed. What is the Design Speed of the northbound U.S.R. 23 at the proposed location? The proposed Ramp B will carry traffic exiting northbound U.S.R. 23 onto eastbound S.R. 823. Ramp B would preferably be designed with the same design speed as northbound U.S.R. 23. However, our office cannot comment on the alignment and profile grade, because the geometric review of the proposed alignment will need to be reviewed by the **ODOT - Office of Roadway Engineering Services.** Please make sure to obtain an approval from the **ODOT - Office of Roadway Engineering Services** prior to proceeding with the next design stage.	The design speed of the northbound U.S.R. 23 at the proposed location is currently 40 mph. Attached is an email from ODOT District 9 approving our geometric design of the interchange. Also attached is the Technical Memorandum explaining our geometric design of the interchange, which ODOT District 9 was provided.
General	7. Include the Structure File Number in the Title Block. The Structure File Number for this bridge is 7306717. For future projects, the Structure File Number can be obtained by contacting Ms. Kathy J. Keller, Office of Structural Engineering, Bridge Inventory Section (Ph. 614-752-9973).	Will comply.

Thompson, Shawn/COL

From: mdweeks@transystems.com
Sent: Thursday, September 01, 2005 4:19 PM
To: Thompson, Shawn/COL
Cc: Miller, Robert/COL
Subject: FW: SCI-823 Fairground Road

Shawn,
See the information from Dave Norris concerning Fairground Rd (CR55). You will need to take this info into account when determining the required horizontal clearances to your overhead bridge substructures.

Michael D. Weeks, PE, PS
TranSystems Corporation
5747 Perimeter Drive, Suite 240
Dublin, OH 43017
Ph: (614) 336-8480 Fax: (614) 336-8540

From: David Norris [mailto:David.Norris@dot.state.oh.us]
Sent: Thursday, September 01, 2005 4:08 PM
To: CO-Michael Weeks
Subject: SCI-823 Fairground Road

Mike,

I spoke with Clyde Willis, Scioto County Engineer today.
He said he has no plans to widen CR 55 in the future, but he thought it would be a good idea to allow for 24' pavement. There's not much traffic, except for the fair, and for the swap days, and other events.
He said the speed limit is 55 mph.
I found the Functional Classification to be Minor Collector, per CO Planning
http://www.dot.state.oh.us/planning/Functional%20Class/2004FuncClass/District09/Scioto.pdf

Clyde also said the speed limit on CR-28 (Lucasville-Minford) is 55 mph where we cross.

--
David A. Norris, PE
ODOT District 9 DDD Engineering Assistant
PO Box 467 Chillicothe, OH 45601
Toll Free: (888) 819-8501
Direct Phone: (740)-774-9061

RECORD OF TELEPHONE CONVERSATION

Date:	June 7, 2006	Job No.	403030064
Time:	11:45 a.m.	Project:	SCI-823-0.00
Contact:	Rita Thoroughman	Subject:	Sideroad Traffic Counts
Phone No.:	740-259-5541	By:	Mike Weeks

Summary of Conversation:

I talked with Rita Thoroughman, Office Manager at the Scioto County Engineer's Office maintenance garage, about the ADTs for the sideroads. She provided me with the following:

Swauger Valley-Minford (CR 31) – north of Shumway = 1,041
Blue Run Rd (CR 29) – north of Flowers-Ison Rd = 937
Morris Lane-Blue Run (CR 54) west of Twp. Rd. 182 = 251
Flatwood-Fallen Timbers (CR 184) south of Blue Run (TR 182) = 768
Fairground Rd. (CR 55) north of Thomas Hollow Rd (TR 158) = 3,056
Highland Bend Rd (TR 248) at Portsmouth Corp. Limit (Slocum Ave. in Portsmouth) = 1,897
Nothing for Pershing Ave. since in Portsmouth (does not think Portsmouth will have any counts)

Ramp Design Speeds (Design/Cost Evaluation) US-23 Interchange Portsmouth Bypass (SCI-823-0.00)

PREPARED FOR:	Mike Weeks/TranSystems Corporation
PREPARED BY:	Ram Nunna/CH2M HILL
	Andy Wolpert/CH2M HILL
DATE:	June 21, 2005
RE:	Ramp Design Speeds

Introduction

This technical memorandum discusses the design speeds for the ramps associated with the proposed US 23/SR 823 interchange (See Exhibit 1 for plan view). As a follow-up of an earlier technical memo (dated May 7, 2005) this memo specifically addresses the design and cost issues arising out of the TranSystems review of the May 7, 2005 technical memo. The design team offers this memo as a means of considering design options and selecting an appropriate, cost effective solution for ramp alignments and structure/roadway widths.

The speeds discussed in this memo pertain to the sharpest curve on the ramp proper. Figure 503-1, L&D Volume 1, shows the range of ramp design speeds that can be used based on the mainline design speeds. ODOT recommends a design speed in the lower range for loop ramps, and a design speed in the middle to upper range for directional ramps. After evaluating the horizontal and vertical constraints for the ramp design, the design team has established per project design criteria a design speed of 35 mph for Ramps A and D, and a design speed of 40 mph for Ramp B and 45 mph for Ramp C. The design speed controls the three dimensional geometry of the ramp (Horizontal curvature, vertical curvature and stopping sight distance).

Overview of Design Trade-offs

The current typical section design for the ramps is consistent with ODOT L&D Volume 1; Fig. 303-1 design requirements for 1-lane directional ramps. However, the standard typical section does not provide full horizontal stopping sight distance (HSSD) requirements for the selected design speeds on ramps A, B and C. due to the restricted line of sight by the roadway features like bridge parapet, median barrier and retaining walls. The proposed vertical alignments for the ramps meet or exceed ODOT's stopping sight distance (SSD) requirement for the established design speed

We note that, according to L&D, Volume 1, Section 105.1, design exceptions for design speed related items are not required for ramps, since ramps do not have continuous design speeds throughout their entire length. Even without the need of the design exception request, we believe it is appropriate to fully address the design issues and trade-offs, and to document the decision regarding the design speeds and the structure widths.

There are cost implications associated with designing the structures and roadway to provide the required SSD for established design speed. In addition, there may be potential operational and safety implications if the structures and roadway is designed with less than minimum SSD.

This memo outlines the available options and is intended for use in decision making and documentation of the design speeds and structure widths.

Other Considerations

Safety, traffic operations, and maintenance are all important considerations when reviewing SSD issues. Safety research suggests that nominal deficiencies in SSD do not usually produce measurable reduction in safety. NCHRP Report 400 confirms that only severe restrictions in SSD lead to measurable reduction in safety. Research by Neuman and Glennon ("Cost Effectiveness of Improvements to Stopping Sight Distance", Transportation Research Record 923) notes that improving geometrics to full SSD requirements are often not cost effective, except where the restriction is severe and the traffic volumes are high. The research also notes that the SSD within 5 to 10 mph of the design speed may be adequate and cost effective, particularly when the cost implications of providing full SSD are significant. Finally the length of the highway over which the restriction in SSD exists is important factor in understanding and gauging the severity of the restriction.

Ramp A (Southbound US 23 to Southbound SR 823)

Ramp A carries a moderate traffic volume of (9400 ADT, 2030 design year from southbound US 23 to southbound SR 823 on a horizontal alignment with 35 mph design speed. The proposed vertical alignment meets ODOT's SSD requirements for 35 mph. Ramp A is separated from loop Ramp D by a concrete median barrier (See Exhibit 1 for plan view).

Horizontal alignment for directional Ramp A adjacent to Ramp D is a circular curve with a length of 764.0 feet and degree of curvature 14º31'. The curve is curving left in the direction of travel. With the standard typical section design, the proposed concrete median barrier separating Ramp A and Ramp D traffic limits the line of sight and the available HSSD required for the 35 mph design speed. The length of HSSD restriction is approximately 700 feet. For analysis purposes following options were considered.

Option 1 Design the ramp for the normal cross section of a 1-lane directional ramp (6' median shoulder, 6' right shoulder) and accept the resultant SSD provided by those dimensions.

Option 2 Design the ramp for the full SSD for 35 mph (12' median shoulder, 6' right shoulder).

Evaluation and Recommendation

Table 1 compares the HSSD, shoulder widths, and corresponding costs associated with all the options. Option 1 would offer the lower construction cost and minimizes the impacts to the wetland area. However, Option 1 does not provide HSSD per ODOT criteria for 35 mph; it meets the criteria for 30 mph. Option 2 meets the HSSD requirement for 35 mph but at the expense of additional cost for shoulder pavement/earthwork and additional impacts to the wetlands. The 12′ median shoulder width would add approximately $35,000 in additional costs for pavement/earthwork and $20,000 for wetland mitigation (onsite or offsite).

It is also important to consider the length of highway over which the sight restriction occurs. The actual length of sight restriction is 700 feet along the horizontal curve adjacent to Ramp D alignment. The approximate travel time along the 700 feet sight distance restriction would be 13.6 seconds (at 35 mph) which is about 5.4 times above the driver reaction time (2.5 seconds) used to calculate the SSD.

Based on the length of HSSD restriction (700 feet), the increased shoulder width would improve the available HSSD with associated operational and safety benefits. The operational and safety benefits would offset the nominal additional cost ($55,000) associated with the increased shoulder width.

Based on construction cost of the pavement/earthwork, cost of additional wetland mitigation, length of HSSD restriction, assessment of safety and operational impacts, the design team recommends Option 2 be considered for Ramp A.

Ramp B (Northbound US 23 to Southbound SR 823)

Ramp B is a directional ramp, carries moderate traffic (3600 ADT, 2030 design year) from northbound US 23 to southbound SR 823 on a horizontal alignment with 40 mph design speed (See Exhibit 1 for plan view). The proposed vertical alignment meets ODOT's SSD requirement for 40 mph design speed.

Horizontal alignment for directional Ramp B is a circular curve with a length of 913.37 feet and a degree of curvature 11°15′. The curve is curving right in the direction of travel. With standard typical section design, the proposed parapet over the retaining wall and the bridge parapets over the Norfolk Southern Railroad and Fairgrounds road, along the right edge of shoulder, limits the line of sight and the available HSSD required for 40 mph design speed. The length of HSSD restriction is approximately 900 feet. The provision of a flatter curvature was not considered as an option, due to severe bridge skew over the Norfolk Southern Railroad and Fairgrounds road bridges. For analysis following options were considered.

Option 1 Design the structure for the normal cross section of a 1-lane directional ramp (6′ left shoulder, 8′ right shoulder) and accept the resultant HSSD.

Option 2 Design the structure for the full SSD for 40 mph (6′ left shoulder, 15′ right shoulder).

Option 3 Design the structure for the same total width as Option 1, but shift 2′ of the bridge width normally provided on the left shoulder to the right shoulder (4′ left shoulder, 10′ right shoulder); therefore, increasing the available SSD without increasing total structure width.

Option 4 Design the structure for a 12′ right shoulder width and accept the resultant HSSD.

Evaluation and Recommendation

Table 2 compares the HSSD, shoulder widths, and corresponding bridge costs associated with all the options. Option 1 and Option 3 would offer the lowest construction cost; however both the options do not provide adequate HSSD required for 40 mph design speed. Option 3 would improve the HSSD slightly over Option 1, but at the expense of a left shoulder dimension less than is normally provide. Option 2 meets the full SSD requirement for 40 mph, but at the cost of a wider bridge deck. The 15′ right shoulder width would add approximately $270,000 and $120,000 to the structure costs over Norfolk Southern Railroad and Fairgrounds road. Option 4 represents an operational compromise, providing some additional stopping sight with additional cost of $150,000 and $60,000 to the structures over Norfolk Southern Railroad and Fairgrounds road.

Options 1 and 3 represent the lowest construction cost alternatives. Of course, the trade off is that both the options do not provide HSSD required for 40 mph design speed and Option 3 has substandard left shoulder. Option 2 provides the required HSSD for 40 mph however at the expense of additional costs and some safety concerns due to the 15 feet wide shoulder (the wide shoulder may encourage unsafe maneuvers). Option 4 improves HSSD nominally over options 1 and 3 for high additional cost and still does not provide the required HSSD for 40 mph design speed. A nominal 5 mph increase in design speed is associated with a huge increase in structure costs. This additional cost would not be expected to produce quantifiable safety or operational benefits.

The reduction in design speed is within 5 mph for all the options and is within the upper and lower range of the ramp design speed as per L&D, Volume 1, Section 500, figure 503-1. According to the L&D, Volume 1, Section 105, design exceptions for speed related items are not required for ramps, since ramps do not have continuous design speeds throughout their entire length. Accordingly the reduction in design speed for Options 1, 3 and 4 on Ramp A does not require a design exception. Of all the options, Option 1 minimizes the bridge width and has the lowest construction cost and the typical section is consistent with the ODOT criteria. Based on the profile design for Ramp B the vertical upgrade of 5.8% (6% maximum) is not conducive to higher (greater than 40 mph) design speeds

It is also important to consider the length of highway over which the sight restriction occurs. The actual length of sight restriction is 900 feet along the horizontal curve. The approximate travel time along the 900 feet sight distance restriction would be

15.3 seconds (at 40 mph) which is about 6 times above the driver reaction time (2.5 seconds) used to calculate the SSD.

Based on the length of HSSD restriction (900 feet), any increase in normal (8 feet) shoulder width would improve the available SSD. However the increase in shoulder width to 15 feet to attain adequate HSSD for 40 mph is associated with additional increase in structural costs. The additional costs would not be expected to produce quantifiable safety or operational benefits.

Based on construction costs, length of HSSD restriction, assessment of safety and operational impacts, design team recommends that option 1 be considered for Ramp B.

Ramp C (Northbound SR 823 to Northbound US 23)

Ramp C is also a directional ramp carries, moderate traffic (9400 ADT, 2030 design year) from northbound SR 823 to northbound US23 on a horizontal alignment with 45 mph design speed (See Exhibit 1 for plan view). The proposed vertical alignment meets ODOT's SSD requirements for 45 mph design speed.

The horizontal alignment for directional Ramp C is a circular curve with a length of 744.85 feet and a degree of curvature of 7º45'. The curve is curving right in the direction of travel. With standard typical section design, the proposed bridge parapets over the Norfolk Southern Railroad and the parapets over the retaining walls, along the right edge of shoulder, limits the line of sight and the available HSSD required for 45 mph design speed. The length of HSSD restriction is approximately 600 feet. The provision of a flatter curvature was not considered as an option, due to severe bridge skew over the Norfolk Southern Railroad and Fairgrounds road bridges. For analysis following options were considered.

Option 1 Design the structure for the normal cross section of a 1-lane directional ramp (6' left shoulder, 8' right shoulder) and accept the resultant HSSD.

Option 2 Design the structure for the full SSD for 45 mph (6' left shoulder, 14' right shoulder).

Option 3 Design the structure for the same total width as Option 1, but shift 2' of the bridge width normally provided on the left shoulder to the right shoulder (4' left shoulder, 10' right shoulder); therefore, increasing the available SSD without increasing total structure width.

Table 3 compares the HSSD, shoulder widths and corresponding bridge costs associated with all the options. Option 1 and Option 3 would offer the lowest construction cost; however both the options do not provide adequate HSSD required for 45 mph design speed. Option 3 would improve the HSSD slightly over option 1, but at the expense of substandard left shoulder. Option 2 meets the full HSSD requirement for 45 mph, but at accost of wider bridge deck. The 14 feet right shoulder width would add approximately add $180,000 to the structure cost over Norfolk Southern Railroad.

Options 1 and 3 represent the lowest construction cost alternatives. Of course, the trade off is that both the options do not provide HSSD for 45 mph design speed and Option 3 has substandard shoulder. Option 2 provides the required HSSD for 45 mph at the expense of high additional bridge costs and some safety concerns due to the 14 feet wide shoulder. For a nominal 5 mph increase in design speed is associated with a high increase in structure costs. This additional cost would not be expected to produce quantifiable safety or operational benefits.

The reduction in design speed for Options 1 and 3, as compared to Option 2, is within 5 mph and in turn it is within the upper and lower ranges of the ramp design speed as per L&D, Volume 1, Section 500, figure 503-1. According to the L&D, Volume 1, Section 105, design exceptions for speed related items are not required for ramps, since ramps do not have continuous design speeds throughout their entire length. Accordingly the reduction in design speed for options 1 and 3 on Ramp A does not require a design exception. Of all the options, option 1 minimizes the bridge width and has the lowest construction cost and the typical section is consistent with the ODOT criteria.

It is also important to consider the length of highway over which the sight restriction occurs. The actual length of sight restriction is 600 feet along the horizontal curve. The approximate travel time along the 600 feet sight distance restriction would be 9.1 seconds (at 45 mph) which is about 3.6 times above the driver reaction time (2.5 seconds) used to calculate the SSD.

Based on the length of HSSD restriction (600 feet), any increase in normal (8 feet) shoulder width would improve the available SSD. However, the increase in shoulder width to 14 feet to attain adequate HSSD for 45 mph is associated with additional increase in structural costs. The additional costs would not be expected to produce quantifiable safety or operational benefits.

Based on construction costs, length of HSSD restriction, assessment of safety and operational impacts and profile design constraints, design team recommends that option 1 be considered for Ramp C.

Ramp D (Northbound SR 823 to Southbound US 23)

Ramp D is a loop ramp, carries moderate traffic volumes (3600, 2030 ADT) from northbound SR 823 to southbound US 23 on a horizontal alignment with 35 mph design speed. The proposed alignment uses the minimum allowable radius for 35 mph and does not cause any geometric or sight distance issues. Right-of-way and wetland impacts are minimized by this design.

Design team recommends that the current design, which utilizes the normal cross section as per L&D Volume 1, Fig. 303-1, be utilized.

Conclusions

Based on the analysis and evaluation following are the recommendations:

RAMP A – 6 feet right shoulder, 16 feet lane and 12 feet median shoulder adjacent to Ramp D segment, resultant design speed is 35 mph

RAMP B- 8' right shoulder, 16 feet lane and 6' left shoulder on Norfolk Southern Railroad and Fairgrounds road bridges, resultant design speed is 35 mph

RAMP C – 8' right shoulder, 16 feet lane and 6' left shoulder on Norfolk Southern Railroad bridge – resultant design speed is 40 mph.

Table 1

Design Options for Ramp A

NO.	Design Option	Shoulder Width (ft)		"d" - Offset to Sight Obstruction (ft)	Stopping Sight Distance/ Design Speed	Total Width of Pavement[1]	Additional Roadway/Earthwork/ Wetland Mitigation Cost
		Left	Right	Ramp Lane			
1	ODOT Typical Section, No Shoulder Widening	6'	6'	13.63'	210' (31 mph)	28'	$0
2	Full SSD for 35 mph	12'	6'	19.27'	250 (35 mph)	34'	$55,000

1 = Total Width of Pavement is equal to the Left and Right Shoulders, and 16' lane

Table 2

Design Options for Ramp B

NO.	Design Option	Shoulder Width (ft)		"d" - Offset to Sight Obstruction (ft)	Stopping Sight Distance/	Total Width	Cost of Structure over	
		Left	Right	Ramp Lane	Design Speed	of Structure	Fairground Rd.	Norfolk Southern
1	ODOT Typical Section, No Shoulder Widening	6'	8'	15.26'	250' (35 mph)	33'	$540,000	$1,170,000
2	Full SSD for 40 mph	6'	15'	22.66'	305' (40 mph)	40'	$660,000	$1,440,000
3	Compromise Design, Shift 2' of Left Shoulder to the Right Shoulder	4'	10'	17.79'	270' (37 mph)	33'	$540,000	$1,170,000
4	Compromise Design, Widen Right Shoulder to 12'	6'	12'	19.12'	280' (38 mph)	35'	$600,000	$1,320,000

1 = Total Width of Structure is equal to the Left and Right Shoulders, 16' lane, and 1.5' of barrier on each side

Table 3

Design Options for Ramp C

NO.	Design Option	Shoulder Width (ft)		"d" - Offset to Sight Obstruction (ft)	Stopping Sight Distance/ Design Speed	Total Width of Structure	Cost of Structure over
		Left	Right	Ramp Lane			Norfolk Southern
1	ODOT Typical Section, No Shoulder Widening	6'	8'	15.68'	305' (40 mph)	33'	$1,860,000
2	Full SSD for 45 mph	6'	14'	21.81'	360' (45 mph)	39'	$2,040,000
3	Compromise Design, Shift 2' of Left Shoulder to the Right Shoulder	4'	10'	17.79'	325' (42 mph)	33'	$1,860,000

1 = Total Width of Structure is equal to the Left and Right Shoulders, 16' lane, and 1.5' of barrier on each side

Wolpert, Andy/COL

From: mdweeks@transystems.com

Sent: Tuesday, June 06, 2006 3:30 PM

To: Miller, Robert/COL

Cc: Wolpert, Andy/COL; Thompson, Shawn/COL; jgbrown@transystems.com; rnunna@transystems.com

Subject: US23 Interchange TM approval

Rob,

D-9 agrees with the recommendations in the TM for the US 23 interchange layout (see below). This was already incorporated in your PAVR design.

Mike,
I have reviewed the report by CH2M, dated June 21, 2005.
District 9 concurs with the recommended options for Ramps B & C.
For Ramp A, the 35 mph design speed is preferable, but if the 12' left shoulder causes significant design, layout or environmental difficulties, the Option 1 (31 mph design speed) should be acceptable.

Sorry for not responding more quickly.

--
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