Bridge Preliminary Design Report

Ramp B over Norfolk Southern Tracks
SCI-823-1598

SCI-823-10.13
PID No. 79977

Prepared for

Ohio Department of Transportation

November 2007

CH2MHILL

Bridge Preliminary Design Report

Ramp B over Norfolk Southern Tracks
SCI-823-1598

SCI-823-10.13
PID No. 79977

Prepared for

Ohio Department of Transportation

November 2007

CH2MHILL

TABLE OF CONTENTS

1. Introduction

Following review and resolution of comments on the Structure Type Study resubmitted in June 2007, a three-span composite curved steel plate girder bridge with reinforced concrete deck and jointed stub abutments behind a MSE wall (rear abutment) and a 2:1 spill through slope (forward abutment) was the structure type selected by the Department on July 9, 2007 for construction of the proposed Ramp B over Norfolk Southern Tracks bridge.

The proposed bridge is a three span curved structure with the substructures located radially to the curve. The spans are 138'-0", 187'-0" and 138'-0". The reinforced concrete deck is 33'-0" wide and is supported by four curved steel plate girders. The two tee type piers are supported on HP piles driven to refusal on rock. The two stub abutments are supported on HP piles driven to refusal on rock. The rear abutment is located behind a MSE wall and the forward abutment is located behind a 2:1 spill through slope.

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 revised to reduce the vertical clearance.

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

- *Bridge Substructure:* The location of the piers and abutments in plan has not changed.

  The bottom of footing elevations for both abutments has changed due to the revised vertical profile grade, a one inch reduction in the depth of the girder's web (to 80" from 81"), and due to a more accurate estimate of the bearing thickness.

  The drainage design for the project has been revised since the submittal of the type study. The proposed culvert under US 23 at Sta. 611+00 has been relocated to Sta. 609+50±.

  A ditch has also been added north of Pier 2. Preliminary ditch design shows the ditch centerline comes within ±30 feet of the pier centerline, and the low point of the ditch bottom adjacent to Pier 2 is at Elev. ±549.7. However, the ditch location and depth does not affect the depth of the footing at Pier 2 due to its distance from the pier. Therefore, it is proposed to maintain the bottom of footing elevation at 542.00, as proposed in the type study. Detailed design locating the ditch centerline and establishing the ditch grade has not been completed. As the drainage design proceeds, the bottom of footing elevation will be adjusted so as to provide a minimum of one foot of cover over the top of footing.

- *Bridge Superstructure:* Preliminary girder designs for an interior girder with different web depths have been completed. The results of the study show that a web depth between 80 inches and 82 inches results in a girder with the least weight. A web depth of 80 inches is proposed for this bridge. This is a revision from the 81 inch web depth shown in the type study.

A deck placement sequence will be prepared during final design development. The following factors support the preparation of a deck placement sequence:

1. *Deck Concrete Volume:* Approximately 450 cy of deck concrete (not including parapets) will be placed, which may require more than one construction day.
2. *Staged Construction:* A concrete deck pour over an existing and active railroad may require more than one construction day.
3. *Horizontal Curvature:* Constructability issues are associated with horizontally curved bridges. Differential deflections between girders will be addressed.

- *Constructability:* ODOT's review of the Structure Type Study resulted in a comment regarding construction of the proposed structure. Specifically, CH2M HILL was asked to provide a girder erection sequence plan. Erection sequence plans are included with this submission in Appendix E. A few things should be noted concerning these plans:

  - It was assumed that the two future Norfolk Southern tracks will have been constructed by the time steel erection occurs.
  - Crane placement on the drawings was such to maintain a minimum 13 feet horizontal clear zone distance at all times throughout steel erection, as per Norfolk Southern standards.
  - The erection sequence scheme shown is only one of many that the contractor may choose.
  - Only one crane is shown on the drawings, but two will probably be required. The second crane would fit on the opposite side of the bridge from that shown on the drawings.
  - During a May 2, 2007 meeting with representatives of Norfolk Southern, the railroad concurred that the contractor could put temporary falsework between the existing tracks as long as 10 feet of clearance from the centerline of track is maintained at all times. Therefore, there is room between the existing tracks for temporary shoring if the contractor wants to use it, as long as the 10′ clearance is maintained. This may allow the use of smaller cranes.
  - Under Section VI. of Norfolk Southern's Overhead Grade Separation Design Criteria, cranes must be adequate for 150% of the actual weight of the pick. CH2M HILL reviewed heavy lift data tables and selected a crane model capable of lifting 150% of the actual pick weight.

- *Aesthetics:* Aesthetic treatments for this structure and site could include concrete staining or coatings, formliners for the substructure, railing on MSE wall, 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 the Norfolk Southern tracks:

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
Required Vertical Clearance:	23'-2 1/8" over eastern two Norfolk Southern tracks, measured six feet from centerline rail	
	23'-3" over western two Norfolk Southern tracks, measured six feet from centerline rail	
Required Horizontal Clearance:	30'-0" from face of MSE wall to edge of pavement	
	25'-0" from face of pier stem to centerline of adjacent Norfolk Southern track	

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 highway traffic during construction of the Ramp B bridge over Norfolk Southern tracks will be limited. With the exception of limited US-23 closures for MSE wall construction, as well as traffic safety precautions throughout bridge construction, no additional maintenance of traffic solutions will need to be investigated.

Coordination with railway traffic below the proposed bridge will be required during construction. All features have been located such that permanent and temporary work will be located outside the permanent or temporary clear zones as applicable. Appropriate railroad flagging and insurance will be required throughout construction.

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.

Three borings at the Ramp B bridge over Norfolk Southern tracks were taken during the first phase and four 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 stub type rear and forward abutments will be supported by HP 12x53 H-piles driven to refusal on bedrock. The rear abutment is situated behind a MSE wall and the forward abutment has a 2:1 spill through slope. The final pile arrangement should consider avoiding potential conflicts with typical MSE reinforcing strap patterns at the rear abutment. Each pier is supported by HP 14x73 H-piles driven to refusal on bedrock. Pier piles will be battered to resist horizontal loads.

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.	Stub	559.40	516.7	HP12x53	70	43.7	45	50
Pier 1	Tee Type	538.00	517.8	HP14x73	95	21.2	25	30
Pier 2	Tee Type	542.00	522.3	HP14x73	95	20.7	25	30
Fwd Abut.	Stub	574.60	525.9	HP12x53	70	49.7	50	55

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

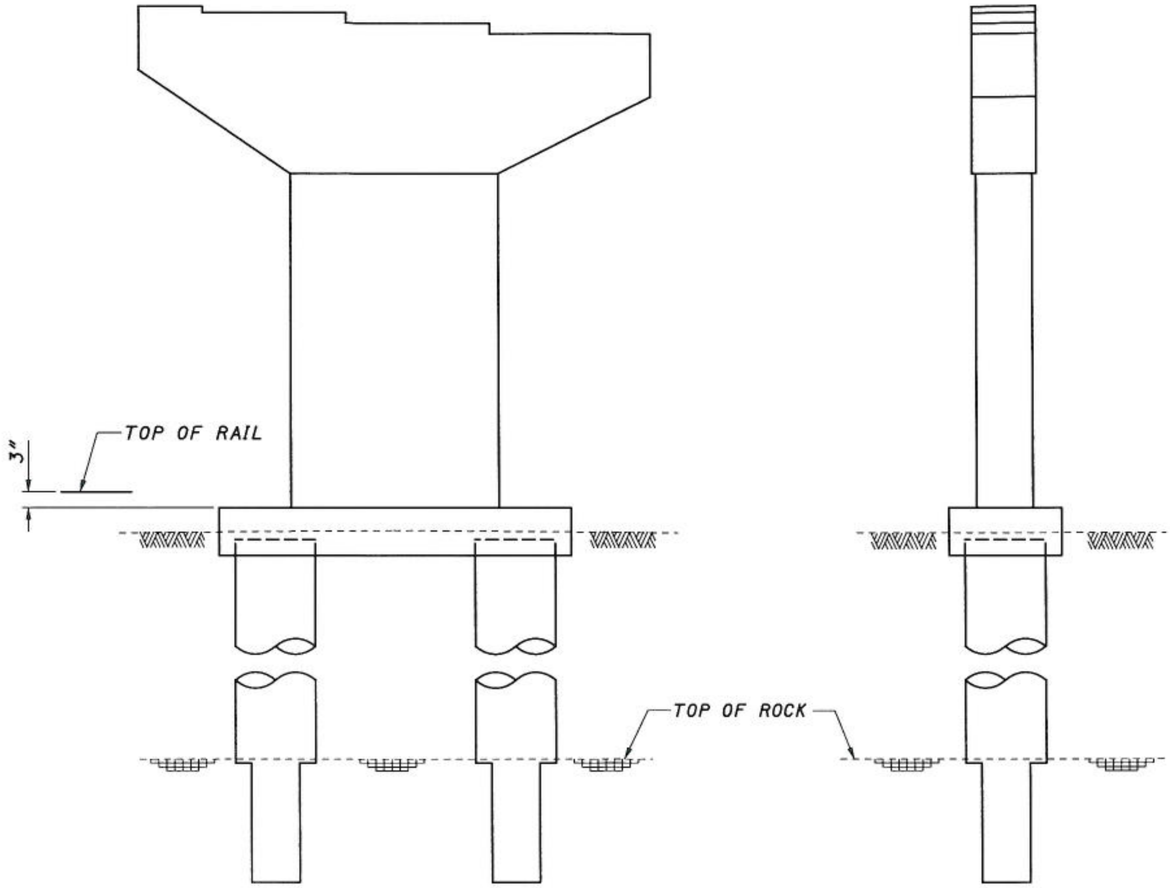
The piers are currently recommended to be supported by steel HP piles driven to refusal on rock. However, the piers could also be supported on drilled shafts that are socketed into rock. At the end of this section is a sketch showing two alternatives for a pier supported on drilled shafts.

Alternative 1 uses large diameter shafts that require a cap beam. Alternative 2 uses smaller diameter shafts that would not require a cap beam. No design or cost estimates have been completed for either alternative. The shafts for Alternative 1 can be sized to meet design requirements. However, since no design work has been completed, it is not known if Alternative 2 is feasible.

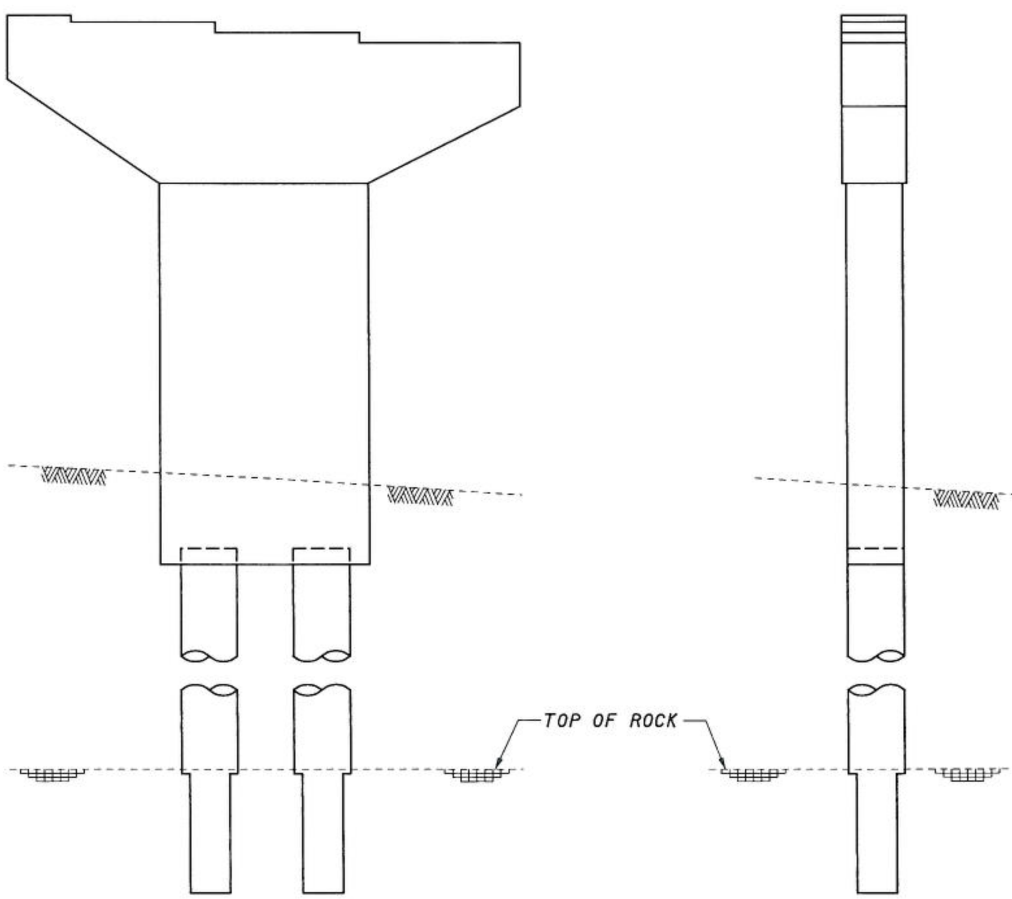
The drilled shaft foundation has several advantages when compared to piles and include:

- Footing excavation is either eliminated or reduced. This could eliminate the need for temporary sheeting or shoring along the Norfolk Southern tracks.
- Less interference with Norfolk Southern operations and potential elimination of their review and approval of temporary sheeting and shoring design calculations and plans.
- May be less costly than a pile foundation.

It is recommended that piers supported by drilled shafts be further evaluated during the next phase of the project.



DRILLED SHAFT PIER ALTERNATIVE 1



DRILLED SHAFT PIER ALTERNATIVE 2

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 the Norfolk Southern tracks is included in Appendix A. 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-823-1598**
Structure File Number:	**7306776**

APPENDIX A

SCI-823-10.13

Ramp B Over Norfolk Southern Tracks

PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[RampB_RR_Structure Cost.xls]Summary

By: DGS   Revised by: SCJ   Date: 5/18/2007   Rev.: 9/25/2007

Checked: SKT   Date: 6/4/2007

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	Superstructure Life Cycle Maintenance Cost	Total Relative Ownership Cost
3	138.00 - 187.00 - 138.00	463.00	4 ~ Steel Plate Girders	80" Steel Plate Girder	**$1,784,000**	**$299,000**	**$333,000**	**$483,000**	**$2,899,000**	**$1,883,000**	**$4,782,000**

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:

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

3. 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, falsework bents, bearings, (minor) temporary shoring, crushed aggregate slope protection, pile driving equipment mobilization, shear connectors, settlement platforms, expansion joints, joint sealers, and joint fillers costs.

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

SCI-823-10.13

Ramp B Over Norfolk Southern Tracks

PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[RampB_RR_Structure Cost.xls]Summary

By: DGS Revised by: SCJ Date: 5/18/2007 Rev.: 9/25/2007

Checked: SKT Date: 6/4/2007

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	Initial Superstructure Cost
3	138.00 - 187.00 - 138.00	463.00	471.36	15,550	598	$293,600	$138,000	$45,300	4 ~ Steel Plate Girders	80" Steel Plate Girder	895,000	$1,306,700	**$1,784,000**

* Deck Length Measured along Centerline of Bridge rather than Baseline

Deck Cross-Sectional Area:

Parapets:

	No.	Individual Area (sq. ft.)	Parapet Area (sq. ft.)
Parapets	2	4.26	8.52

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 steel girder bridges

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

Structural Steel
Unit Costs ($/lb.):

	Cost Ratio	Year 2005	Annual Escalation	Year 2006
Plate Girders - Grade 50 (level 5)	n/a	$1.30	12.0%	$1.46

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

SCI-823-10.13
Ramp B Over Norfolk Southern Tracks
PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[RampB_RR_Structure Cost.xls]Summary

By: DGS Revised by: SCJ Date: 5/18/2007 Rev.: 9/25/2007
Checked: SKT Date: 6/4/2007

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	Temporary Sheeting and Shoring Cost	Initial Substructure Cost
3	138.00 - 187.00 - 138.00	4 ~ Steel Plate Girders	80" Steel Plate Girder	$96,900	$19,400	$63,500	$11,700	$70,600	$37,200	**$299,000**

Pier QC/QA Concrete, Class QSC1 Cost:

Pier 1

	Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Cap	32.6	$555.68	3.0%	$572.00	$18,600
Stem	36.6	$555.68	3.0%	$572.00	$20,900
Footing	24.0	$300.31	3.0%	$309.00	$7,400
Total Pier 1 Concrete Cost					$46,900

Pier 2

	Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Cap	32.6	$555.68	3.0%	$572.00	$18,600
Stem	42.0	$555.68	3.0%	$572.00	$24,000
Footing	24.0	$300.31	3.0%	$309.00	$7,400
Total Pier 2 Concrete Cost					$50,000

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

Pier Piles:

Number Pier 1	Number Pier 2	Top Elevation Pier 1	Top Elevation Pier 2	Bottom Elevation Pier 1	Bottom Elevation Pier 2	Order Length Per Pier 1 Pile	Order Length Per Pier 2 Pile	Total Pile Order Length	Total Cost	Pile Size
18	18	539.0	543.0	517.8	522.3	30	30	1,080	$39,200	HP14 x 73

Abutment Piles:

Number Rear	Number Forward	Top Elevation Rear	Top Elevation Forward	Bottom Elevation Rear	Bottom Elevation Fwd.	Order Length Per Rear Pile	Order Length Per Forward Pile	Total Pile Order Length	Total Cost	Pile Size
10	10	560.4	575.6	516.7	525.9	50	55	1,050	$31,400	HP12 x 53

Abutment QC/QA Concrete, Class QSC1 Cost:

Component		Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Abutment						
	Rear	63.3	$384.26	3.0%	$396.00	$25,100
	Fwd	64.8	$384.26	3.0%	$396.00	$25,700
Wingwalls						
	Rear	0.0	$384.26	3.0%	$396.00	$0
	Fwd	32.0	$384.26	3.0%	$396.00	$12,700
Total Abutment Cost						$63,500

Temporary Sheeting and Shoring:

	Exposed Wall Height	Depth of Embedment	Total Wall Height	Length	Total Exposed Wall Area	Total Wall Area
At Pier 1:	8.0	8.0	16.0	25	200	400
At Pier 2:	12.0	12.0	24.0	35	420	840
Totals					620	1,240

Costs:

Cantilever Sheet Pile Wall	$30.00	per SF of total wall area
Soldier Pile Wall	$40.00	per SF of total wall area
Tied Back Wall	$50.00	per SF of total wall area

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

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

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[RampB_RR_Structure Cost.xls]Summary

By: DGS Revised by: SCJ Date: 5/18/2007 Rev.: 9/25/2007
Checked: SKT Date: 6/4/2007

LIFE CYCLE MAINTENANCE COST

Span Arrangement		Framing	Structural Steel Painting (4)			Superstructure Sealing (4)		
No. Spans	Lengths	Alternative	Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost	Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost
3	138.00 - 187.00 - 138.00	4 ~ Steel Plate Girders	$524,100	2	$1,048,200	$0	0	$0

Span Arrangement		Framing	Bridge Deck Overlay (4)					Bridge Redecking (4)						Superstructure	Total	Total
No. Spans	Lengths	Alternative	Deck Demo & Chipping	Deck Overlay	Deck Joint Gland (2)	Number of Maintenance Cycles	Total Life Cycle Cost	Deck Concrete Cost (3)	Deck Reinforcing Cost (3)	Deck Joint Cost (2)	Deck Removal Cost	Number of Maintenance Cycles	Total Life Cycle Cost	Life Cycle Maintenance Cost (1)	Initial Construction Cost	Relative Ownership Cost
3	138.00 - 187.00 - 138.00	4 ~ Steel Plate Girders	$50,000	$58,000	$5,200	2	$226,400	$293,600	$138,000	$20,800	$155,500	1	$607,900	$1,883,000	$2,899,000	$4,782,000

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.)
Supstr.	80	4	471.4	20.00	34,566	20%	41,500

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	For I-Girder Superstructure Components

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	2

Bridge Deck Removal Cost:

Deck Area (3) (sq. ft.)	Year 2006	Deck Removal Cost
15,550	$10.00	$155,500

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.:

	Year 2005	Annual Escalation	Year 2006
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.)
15,550	1,728	43	36

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. Strip seal deck joints are 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 (including weathering steel) 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.

SCI-823-10.13

Ramp B Over Norfolk Southern Tracks

PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[RampB_RR_Structure Cost.xls]Summary

By: DGS   Revised by: SCJ   Date: 5/18/2007   Rev.: 9/25/2007
Checked: SKT   Date: 6/4/2007

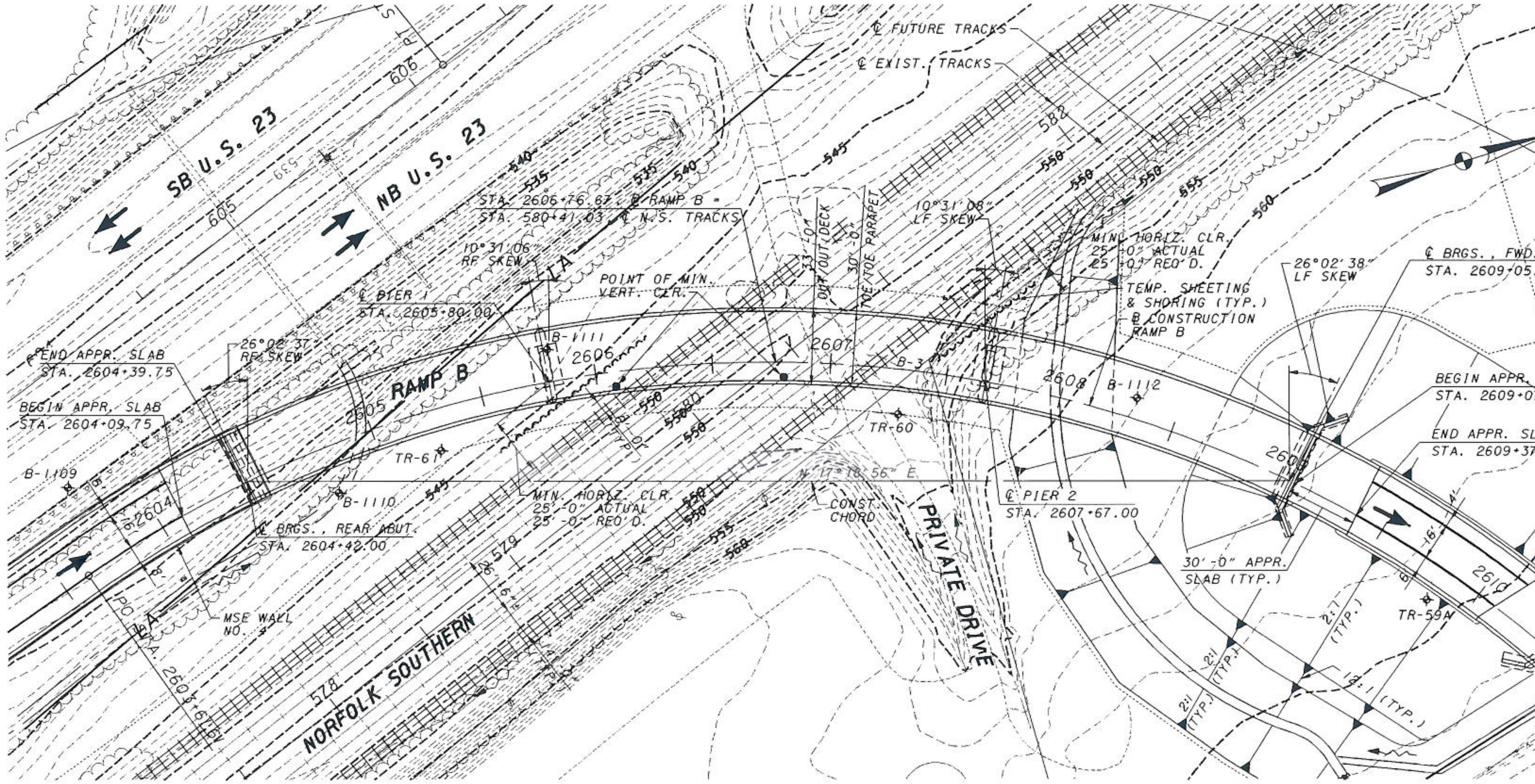
COST SUMMARY

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
3	138.00 - 187.00 - 138.00	4 ~ Steel Plate Girders	80" Steel Plate Girder	$1,784,000	$299,000	$2,899,000	$1,883,000	**$4,782,000**

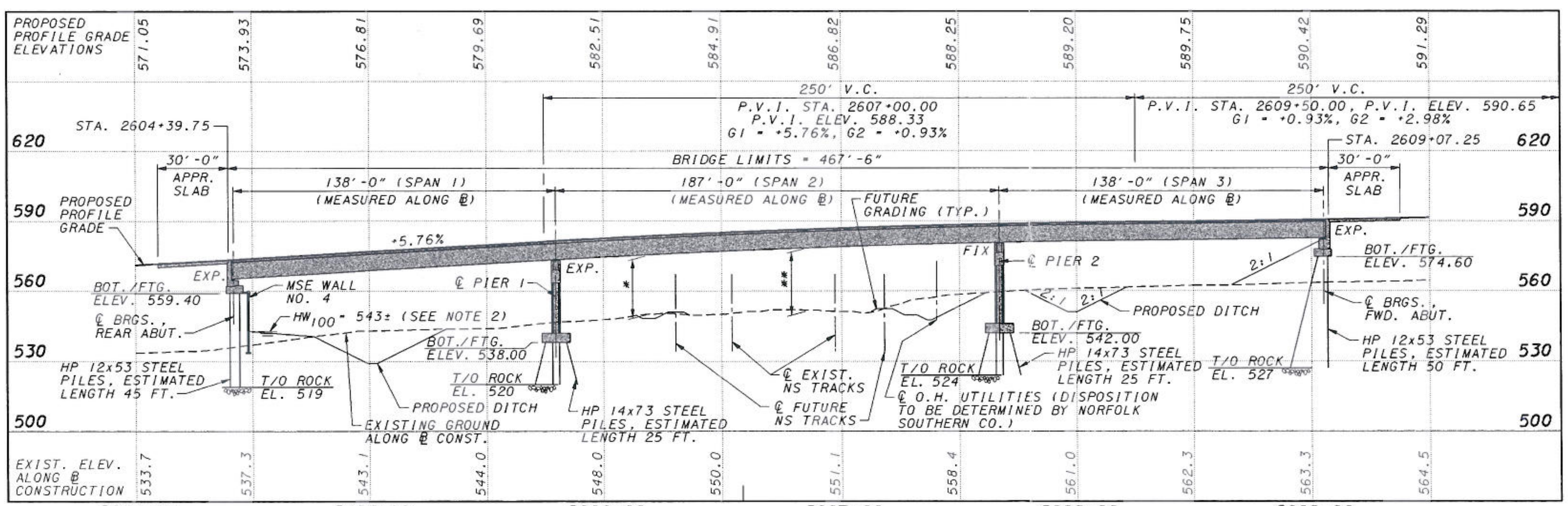
Notes:
1. Includes contingencies and incidental costs.

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

APPENDIX B



PLAN



PROFILE ALONG ℄ CONSTRUCTION, RAMP B

* MIN. VERT. CLR. 23'-7" ACTUAL 23'-3" REQ'D.

** MIN. VERT. CLR. 25'-9" ACTUAL 23'-2⅛" REQ'D.

BENCHMARKS

CURVE B-3

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'
e_{max} = 0.071

TRAFFIC DATA

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

LEGEND

⌖ DENOTES SOIL BORING LOCATION

NOTES:

1. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
2. HIGHWATER (HW) ELEV. 543± IS THE 100 YEAR SCIOTO RIVER BACKWATER ELEVATION AS DETERMINED BY FEMA.

PROPOSED STRUCTURE

TYPE: THREE SPAN COMPOSITE CURVED STEEL PLATE GIRDERS (WEATHERED ASTM A709, GR 50W) WITH REINFORCED CONCRETE DECK ON JOINTED STUB ABUTMENT ON MSE WALL (REAR) AND JOINTED STUB ABUTMENT BEHIND 2:1 EMBANKMENT (FWD.) WITH T-TYPE PIERS

LENGTH OF SPAN: 138'-0", 187'-0", 138'-0", C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-0" TOE/TOE PARAPETS

SIDEWALK: NONE

DESIGN LOADING: HS25 (CASE II) AND THE ALTERNATE MILITARY LOADING, FWS = 60 LB/FT2

SKEW: 26°02'37" RF (REAR ABUTMENT), 10°31'06" RF (PIER 1), 10°31'08" LF (PIER 2), 26°02'38" LF (FORWARD ABUTMENT), 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 FT.)

SUPERELEVATION: 0.071 FT/FT

LATITUDE: N 38°53'28"

LONGITUDE: W 82°59'54"

DESIGN AGENCY	DESIGNED	DRAWN	REVIEWED	DATE
CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	DGS	JBA	VKN	11/07
	CHECKED	REVISED	STRUCTURE FILE NUMBER	
	SCJ		7306776	

SCIOTO COUNTY
STA. 2604+39.75
TO STA. 2609+07.25

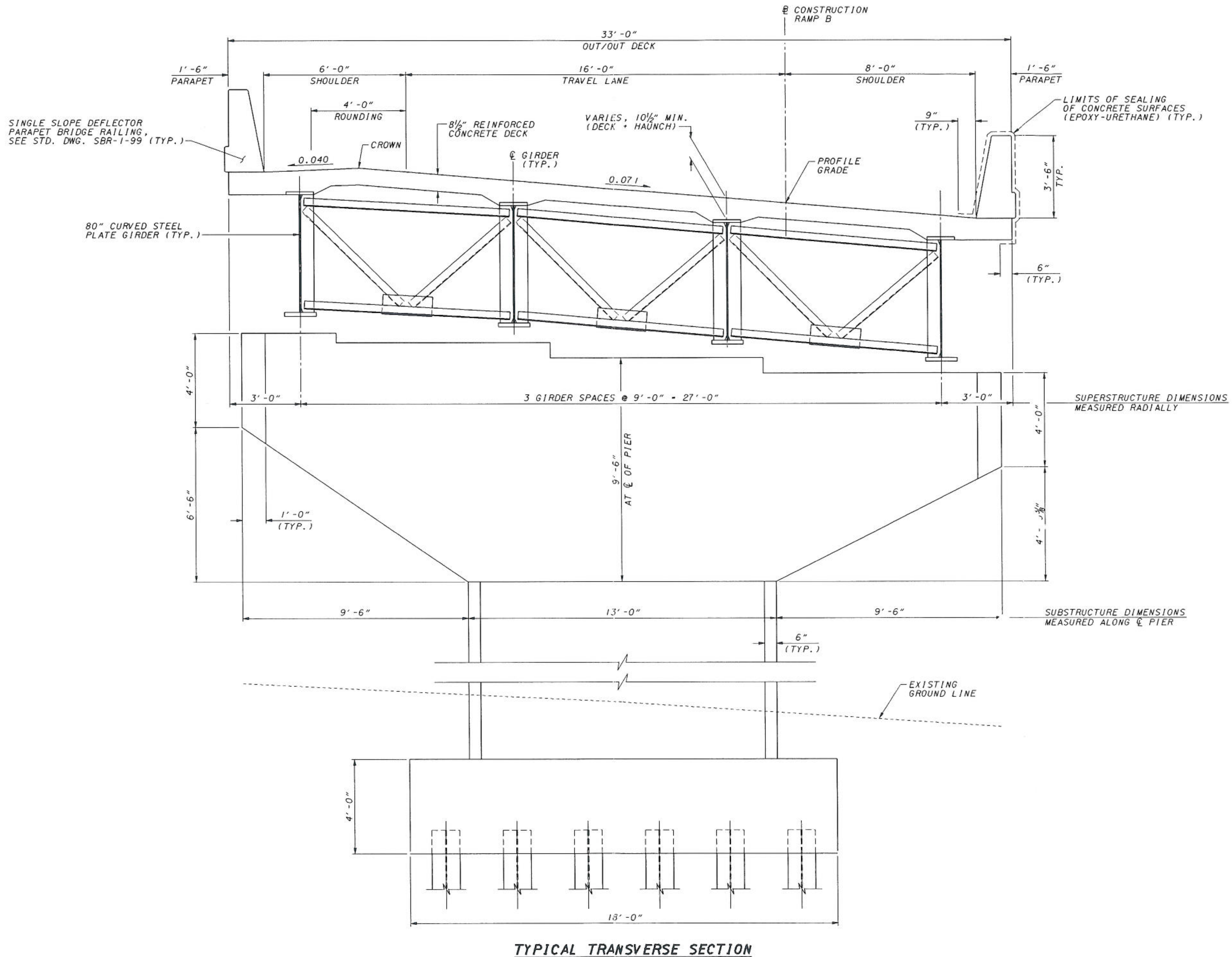
SITE PLAN

BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

1 / 3

833



TYPICAL TRANSVERSE SECTION

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SCJ		7306776	

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

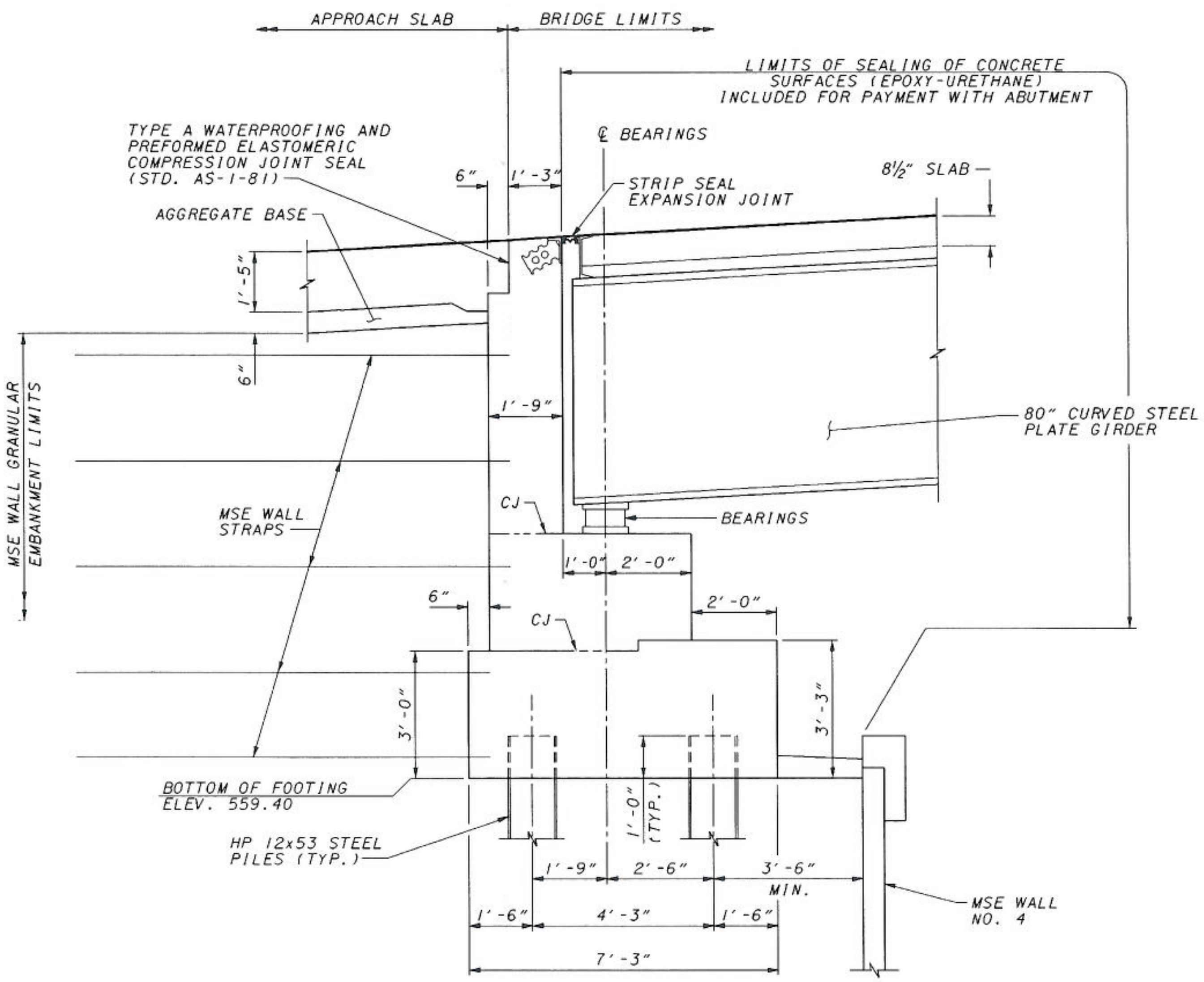
TYPICAL TRANSVERSE SECTION
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

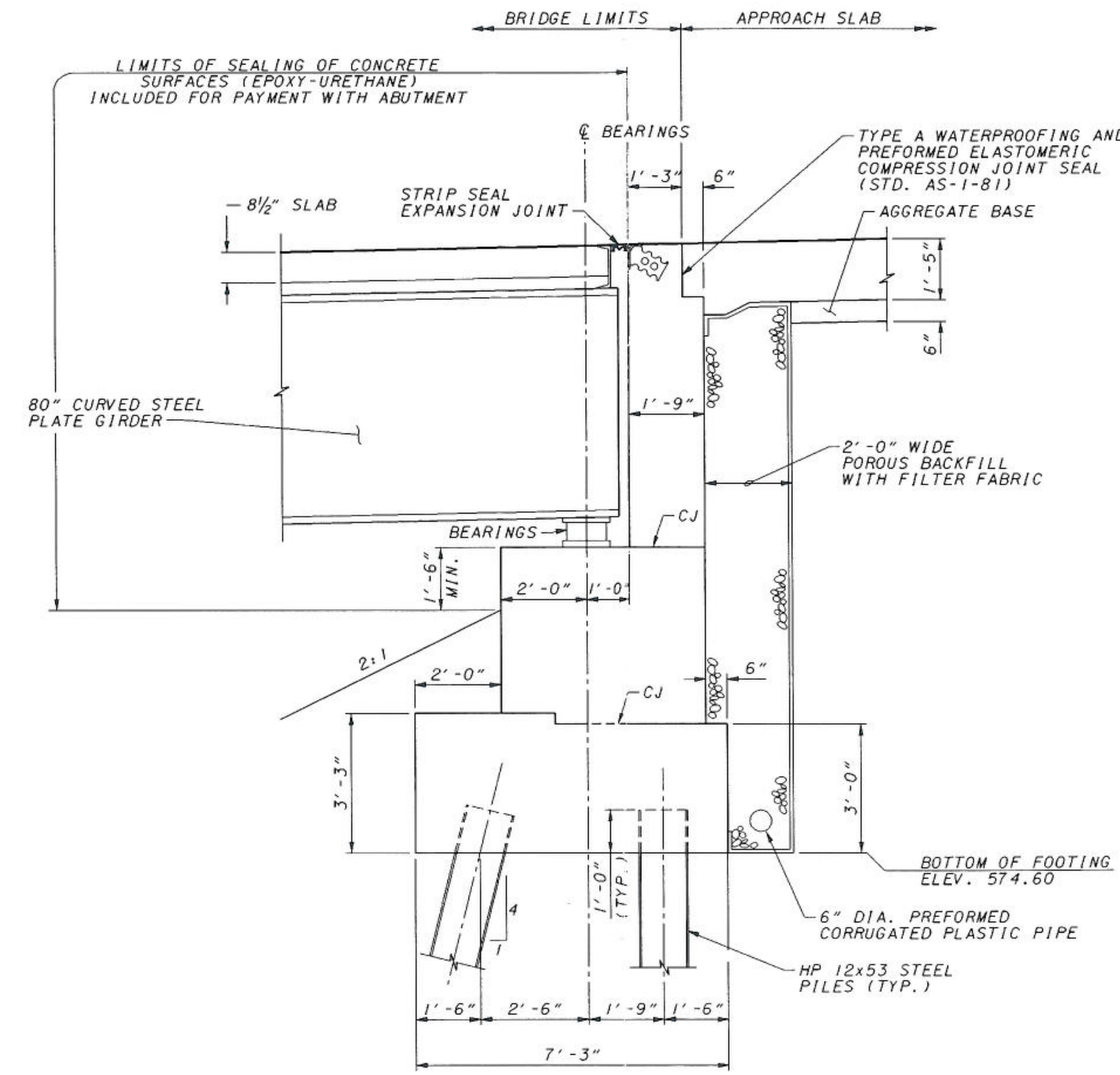
2 / 3

834

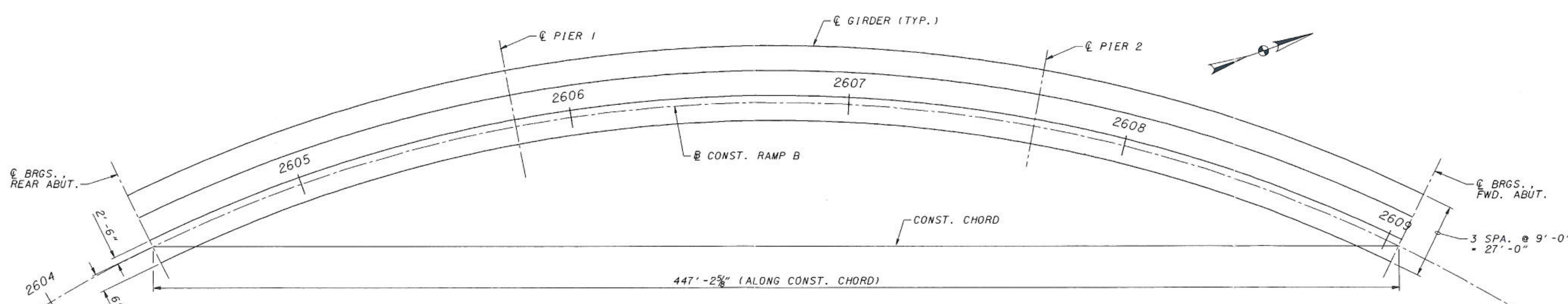
melben
823_1598pts001.dgn
11/27/2007
8:38 AM
\\aries\PlanSets-400 sci823_half.pen



REAR ABUTMENT SECTION



FORWARD ABUTMENT SECTION



FRAMING PLAN

DESIGN AGENCY: CH2MHILL, 5775 Perimeter Drive, Suite 190, Dublin, Ohio 43017	REVIEWED: VKN; DATE: 11/07; STRUCTURE FILE NUMBER: 7306776	DRAWN: JBA; REVISED:	DESIGNED: DGS; CHECKED: SCJ	ABUTMENT SECTION AND FRAMING PLAN — BRIDGE NO. SCI-823-1598, RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

3 / 3

835

melben 823_1598par001.dgn 11/27/2007 8:38 AM \\varies\PlanSets-400 sci823_half.pen

APPENDIX C

Review Comments to DLZ's Geotechnical Report MSE Walls 4 and 5 - 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

Geotechnical engineers at CH2M HILL have completed a brief review of the MSE wall recommendations contained in the report, prepared by DLZ, for this bridge and have the following comments.

1. Bearing Capacity and Staged Construction. The *four* phase staged construction concept proposed to accommodate the very low bearing capacity Factor of Safety has several risks:

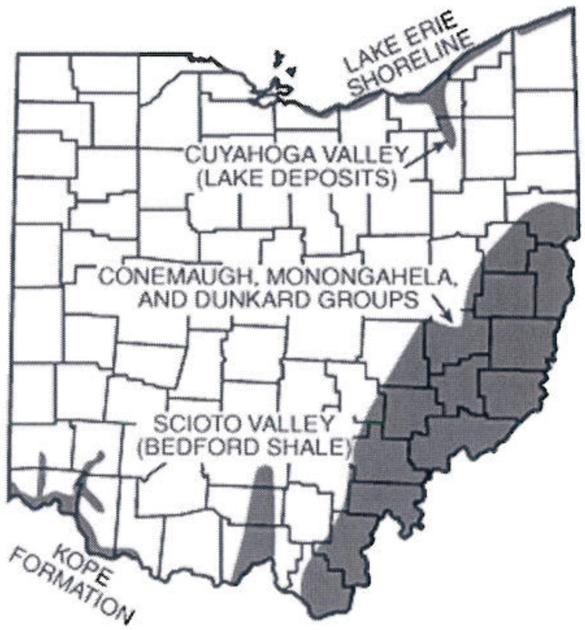
    a. It is time consuming, complex, and has considerable uncertainty for the contractor. The constructor will need to install instrumentation and avoid damaging the instrumentation while placing the stages. If he damages them during placement, he will have to reinstall them during which time there will 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.

2. Slope Stability - Ramps B & C. The borings indicate weathered shale at the soil to rock interface. It is very common in Ohio for there to be a very soft weathered shale layer a few inches thick at the soil to rock interface. This is a notoriously common condition in Ohio that results in one hundred or more landslides annually. 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."*

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.

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.

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

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

Conclusion and Recommendations

1. The consequence of a slip failure of these walls makes avoidance of this risk an overriding priority. It is recommended that alternative construction 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 steel HP 12x53 piles 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.

2. 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.

3. 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 Walls 4 & 5. 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.

4. 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 three sided walls on Ramps B & C.
            - 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.

Report for:

Subsurface Exploration for Bridge and MSE Retaining Walls
US 23 Ramp B Over Norfolk Southern Railroad, (Bridge No. SCI-823-1598)
Project SCI-823-10.13 Portsmouth Bypass (PID 79977)
Scioto County, Ohio

Prepared for:

CH2M Hill
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

DLZ OHIO, INC.
6121 Huntley Road
Columbus, Ohio 43229-1003
Phone: (614) 888-0040
Fax: (614) 888-6415

DLZ Job No. 0121-3070.03

October 19, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

US 23 RAMP B OVER NORFOLK SOUTHERN RAILROAD (BRIDGE NO. SCI-823-1598)

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

SCIOTO COUNTY, OHIO

For:

CH2M Hill
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

By:

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

DLZ Job. No. 0121-3070.03

October 19, 2007

TABLE OF CONTENTS

REPORT
OF
SUBSURFACE EXPLORATION
FOR
BRIDGE AND MSE RETAINING WALLS
US 23 RAMP B OVER NORFOLK SOUTHERN RAILROAD
(BRIDGE NO. SCI-823-1598)
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 Ramp B Interchange bridge over the Norfolk Southern railroad of the Portsmouth bypass project. The findings of other structure evaluations for the Portsmouth bypass project and the US 23 SR 823 Interchange Report will be submitted in separate documents.

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

The structure as planned, is a three-span structure, which utilizes MSE retaining walls to hold back the roadway embankment and contain the rear abutment. It is understood that a spill through slope is currently proposed at the forward abutment location. It is also understood that driven piles will be used to support the abutments and piers of the proposed structures. For more information refer to the Structure Plan and Profile Drawing, presented in Appendix I.

It is understood that MSE walls will be placed along US 23 Ramp B (hereafter referred to as Ramp B) from station 2598+23 to 2604+48 to contain the embankment fill material. As shown on the provided drawings, an MSE wall is planned only on the left side of Ramp B from station 2598+23 to 2600+75. The MSE walls will continue on both sides of Ramp B from station 2600+75 to 2604+48. Also, as part of this retaining wall system, an MSE wall is currently planned at station 2604+48 to contain the rear abutment of the proposed Ramp B structure. Note that only the portions of the Ramp B MSE walls that are adjacent to the bridge, essentially between station 2604+00 and 2604+48, are considered for this report. In this report, the retaining wall system proposed for Ramp B will be generally referred to as Wall No 4.

Based upon the provided drawings and the available cross sections, it is assumed that the maximum height of the proposed Wall No. 4 is approximately 40.5 feet, near the rear abutment location. This height is 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 include the embedment depth. For more information refer to the US 23 Ramp B Plan Drawing, presented in Appendix I.

The analyses and recommendations presented in this report have been made on the basis of the foregoing information. If the proposed locations or structural concept 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 nine borings for the Ramp B bridge and retaining walls. Three structure borings (TR-59 through TR-61) were drilled for previously proposed structure configurations. Six roadway borings (B-1108 through B-1112, and B-1109A) were drilled in the vicinity of the bridge for the proposed roadway, Ramp B retaining walls, and bridge. 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 US 23 Ramp B Plan Drawing presented in Appendix II.

It should be noted that the test results from borings B-45 and B-46, which were drilled for proposed structures over Fairground Road were considered in these evaluations. In addition, borings B-1105A, and B-1113, drilled for other features of Ramp B were also considered in the evaluations for this report. The boring logs and results of testing for these borings are presented in Appendix II and Appendix III, respectively.

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.

Shale and sandstone of the Cuyahoga Formation as well as Sunbury shale were evident in the borings drilled on the eastern end of the interchange. 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 were somewhat varied across the site. In general, the subsoil stratigraphy consisted of shallow surface materials consisting of topsoil or pavement layers underlain by native cohesive and granular soil deposits overlying shale and sandstone bedrock.

4.2.1 Soil Conditions

Borings drilled in the pavement surface for Ramp B generally encountered 4 inches of asphalt concrete pavement at the surface. Below the asphalt concrete pavement, borings generally encountered 4 to 8 inches of aggregate base. Borings drilled off the paved shoulder for Ramp B generally encountered 6 inches of topsoil at the existing ground surface. Below the surface material, cohesive soils ranging from silt (A-4b) to silty clay (A-6b) were encountered to depths ranging from 11.3 to 23.0 feet below the ground surface. Below the cohesive soils, layers of cohesionless soils ranging from gravel with sand (A-1-b) to silt (A-4b) were encountered to depths ranging from 16.0 to 28.9 feet below the ground surface, to the top of bedrock.

Similarly, borings for the proposed bridge generally encountered 1 to 6 inches of topsoil at the ground surface. Below the topsoil, borings generally encountered cohesive soils ranging from silt (A-4b) to silty clay (A-6b) to depths ranging from 10.5 to 13.5 feet below the ground surface. Below the cohesive soils, layers of cohesionless soils ranging from gravel with sand (A-1-b) to silt (A-4b) were generally encountered to depths ranging from 18.5 to 33.0 feet below the ground surface, to the top of bedrock.

4.2.2 Bedrock Conditions

Bedrock was confirmed by coring in all borings considered in this report. Borings B-1108 through B-1112, TR-60, and TR-61 encountered medium hard, black shale (Sunbury shale) at the top of rock. In these borings, bedrock was generally encountered at depths ranging from 16.0 to 33.0 feet below the ground surface. Boring TR-59 encountered soft to medium hard gray shale interbedded with sandstone of the Cuyahoga Formation at the top of rock. In this boring, bedrock was encountered at a depth of 18.5 feet below the ground surface.

The recovery in each core run varied between 87 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 32 and 100 percent with an average of 76 percent, indicating "fair" to "good" quality rock.

4.2.3 Groundwater Conditions

Seepage was observed in all borings considered for this report. In these borings, seepage was first observed at depths ranging from 10.0 to 26.0 feet below the ground surface. Measurable water levels were observed in borings B-1108 through B-1110, TR-60, and TR-61 prior to rock coring at depths ranging from 12.0 to 26.0 feet below the ground surface. Measurable final water levels were present in all borings where rock was cored. In these borings, final water levels were observed at 5.0 to 19.0 feet below the ground surface. 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-1109A to monitor the groundwater level in the area of Ramp B. This piezometer was screened between depths of 11 and 16 feet in the granular layers overlying bedrock. Readings indicate that the phreatic level in the water-bearing granular layers of boring B-1109A ranges from 0.2 to 1.8 feet below the existing ground surface. The average phreatic level is approximately 1.2 feet below the ground surface, corresponding to an elevation of 531.3. For more information, please refer to Appendix II for the piezometer installation report.

5.0 CONCLUSIONS AND RECOMMENDATIONS

It is understood that a three-span structure is proposed to carry one lane of traffic for Ramp B over the Norfolk Southern Railroad. The recommendations contained in this document pertain to the proposed bridge, Ramp B MSE retaining walls adjacent to the bridge, and the forward abutment spill through slope, essentially between stations 2604+00 and 2609+50. Note that there are portions of the Ramp B MSE retaining walls that extend beyond these limits that are not considered in this report.

It is understood that driven HP 14x73 piles are preferred to support the proposed structure. Additionally, it is understood that MSE retaining walls are preferred to retain the fill for Ramp B and to contain the rear abutment.

5.1 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

Recommendations for the MSE walls are presented in the following sections. For the purposes of performing stability analyses and settlement calculations for the proposed MSE walls, it was assumed that deep foundations would be used to support the bridges.

Because the walls are continuous, and due to the varied soil strength characteristics along wall locations, several analyses were performed to determine the most critical profile and wall configuration combination for performing stability and settlement analyses.

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 the Ramp B location in accordance with ODOT and AASHTO guidelines. The MSE walls were also analyzed for sliding, overturning, and settlement.

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.

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.

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, hand penetrometer values, typical correlations, and engineering judgment. Table 1 presents the strength parameters assumed in analyses for the analyses. A summary of the strength and consolidation testing is included in Appendix III. The results of testing are also included in Appendix III.

Table 1 - Soil Parameters Used in Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	ϕ	c'	ϕ'
Reinforced Fill - MSE	Select Granular Backfill	120	0	34	0	34
Retained Soil - MSE	Compacted Embankment Fill	120	0	30	0	30
Embankment Fill	Compacted Embankment Fill	120	2000	0	300	28
Foundation Soil (Wall No. 4) (B-1109 & B-1108)	Very Stiff Clay*	120	2000	0	0	29
	Med. Stiff Clay*	120	900	0	0	28
	Sand and Gravel	120	0	29	0	29

*Bearing capacity analyses required an assumed value for the angle of shearing resistance ($\mathbf{F}_{cu}$) for staged construction evaluations of MSE Wall No. 4 only.

Consolidated undrained triaxial testing (CIU) was performed on representative samples to determine required parameters for staged construction evaluations of MSE Wall No. 4. Tests run on silty clay (A-6b) samples obtained from borings B-1105A and B-1108 reported the angle of shearing resistance (from total stress curve, $\mathbf{F}_{cu}$) ranging from 20.4 to 22.2 degrees. Considering these test results, we selected 21.3 degrees for the angle of shearing resistance for the staged construction analyses of the weaker clay layer. The results of these tests are 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 an effective 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.

It should be noted that testing was also performed to determine the strength of the granular layers that exhibited low SPT N-values. Granular material from boring TR-61 was remolded in a very loose condition and a direct shear test was performed. The sample was saturated in the mold with free water. Furthermore, the sample was stirred prior to commencing the test to ensure a loose condition. The results of the tests indicated friction angles between 42.1 and 45.7 degrees. However, to be conservative, a value of 29 degrees was selected for the purposes of these analyses. The results of this test are presented in Appendix III.

5.1.3 MSE Wall Evaluations and Recommendations US 23 Ramp B – Wall No. 4

Although this report pertains only to the retaining walls adjacent to the Ramp B bridge over the Norfolk Southern Railroad, results of the analyses of adjacent sections of Wall No. 4 are also included. For additional information concerning the analyses and recommendations of the Ramp B retaining walls, please refer to the Final Report of Subsurface Exploration and MSE Wall and Embankment Evaluations for Proposed US 23/SR 823 Interchange.

For MSE Wall No. 4, which will contain the fill for Ramp B, the subsurface profile generally encountered by borings B-1109 and B-1108 were assumed to be the most critical with respect to stability.

It should be noted that the maximum wall height (measured to the top of the coping) was approximately 40.5 feet. 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. It is assumed that the top of leveling pad for this wall will be placed at approximate elevation 532.2.

Borings B-1109 and B-1108 generally encountered very stiff sandy silt (A-4a) and silt and clay (A-6a) from the bottom of the leveling pad excavation (el. 532.2) to approximate elevation 527.6. Below this layer, borings generally encountered medium stiff silty clay (A-6b) to approximately elevation 522.6. Below this layer, borings generally encountered cohesionless silt (A-4b) to approximate elevation 517.1, 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 Wall No. 4 was found to be 1.0, which is below the minimum required value of 2.5.

In order to construct the MSE wall while maintaining the minimum factor of safety against undrained bearing capacity, the use of staged construction was investigated. Additional analyses were performed, assuming that an increase in the undrained shear strength of the foundation soils will occur due to the consolidation from the loading of each stage. These analyses indicate that MSE Wall No. 4 could be built in four 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 these additional analyses, the first stage of 13.0 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 1.08 psi prior to placing the next stage. After excess pore pressures have sufficiently dissipated, the second stage of 11.0 feet may be constructed. 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 0.92 psi prior to placing the next stage. After excess pore pressures have sufficiently dissipated, the third 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 0.67 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 a consolidation period will be required after the first, second, and third stages to allow the excess pore water pressures to dissipate in the foundation soils. Time-rate of consolidation calculations have indicated that a consolidation period of approximately 45 days will be required after each stage to achieve ninety percent (U=90%) consolidation. It is anticipated that a significant portion of the pressures may 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.

The use of prefabricated vertical drains (wick drains) may be considered to accelerate the consolidation of foundation soils under the proposed ramp. The estimated time to ninety percent consolidation using various wick drain spacing options is presented in Table 2. The consolidation times (U=90%) cited in Table 2 are required after each stage to allow pore pressures to dissipate prior to the application of subsequent stages.

Table 2 - Wick Drain Spacing and Consolidation Periods

US 23 Ramp B, MSE Wall No. 4		
Spacing (ft)	Time to U=90% (days)	Approximate Depth of Wick Drains (ft)
5	20	25
7	30	25
9	35	25

It is recommended that wick drains be installed within and to 15 feet beyond (where possible) the limits of the proposed ramp embankment using a triangular spacing pattern. Wick drains should be installed to a depth which is sufficient to penetrate the upper, fine-grained layer. This corresponds to an approximate depth of 25 feet below the existing ground surface. Three feet of sand (ODOT Item 703.02) should be placed over the treated area prior to construction of the embankment. This layer will provide a free draining layer beneath any embankment fill, allowing pore water to be expelled.

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

The length of the reinforcing straps is limited by the width of the ramp for sections with higher walls. At the maximum wall height, a reinforcing strap length of approximately 0.85(H+D) is the maximum length that can be accommodated by the ramp dimensions. As the height of the ramp lessens, while the width remains constant (34 feet), it is recommended that a minimum reinforcing length of 1.0(H+D) be used as allowed by the ramp dimensions.

The maximum settlement at the centerline of the rear abutment retaining wall (station 2604+48) was estimated to be approximately 6 inches. Differential settlement at the wall face, between station 2604+48 and 2603+98 was calculated to be approximately 0.3 percent. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1/100). Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankment loading for the abutment wall. Settlement calculations are presented in Appendix IV.

Table 3 presents the MSE retaining wall parameters and results of analyses for MSE Wall No. 4.

Table 3 - MSE Retaining Wall Parameters and Analyses Results
MSE Wall No. 4, US 23 Ramp B MSE Wall

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 *(Staged Construction)*+ q_{all} *Stg.1=2,287 psf* q_{all} *Stg.2=3,508 psf* q_{all} *Stg.3=4,541 psf* q_{all} *Stg.4=5,293 psf*
Allowable Bearing Capacity – Drained Condition q_{all} = 7,206 psf
Global Stability Factor of Safety – Undrained Condition = 1.5 Factor of Safety – Drained Condition = 2.2 Factor of Safety – Drained Seismic Condition = 2.1
Estimated Settlement of MSE Volume δ_A = 4 inches (Corner, abutment wall, sta. 2604+48) δ_B = 6 inches (At abutment wall centerline, sta. 2604+48) δ_C = 6 inches (At wall face, sta. 2603+98) δ_D = 8 inches (At ramp centerline, sta. 2603+98) Differential Settlement = 0.30% (maximum allowable is 1.0% ODOT BDM 204.6.2.1)
Maximum Full Height of MSE Wall = 40.5 feet (including embedment depth) Minimum Embedment Depth = 3.0 feet* Minimum Length of Reinforcement for External Stability, 1.0(H+D)++

* Assumed top of leveling pad elevation is 532.7. Embedment depth may vary depending on actual top of leveling pad. Minimum embedment depth of 3.0 feet.
+ See Section 5.1.3 for staged construction details.
++ Use 1.0(H+D) where allowed by ramp width. For higher wall sections, reductions of reinforcement length due to limiting ramp width is permissible given that the reinforcement overlap is 0.3 (H+D) or greater.

5.2 Bridge Foundation Recommendations

It is understood that driven HP 14x73 piles are preferred to support the proposed structure. It is also understood that uplift is not anticipated at any of the foundation locations for the proposed bridge. Due to the multi-span bridge and the poor soil conditions encountered, it is assumed that spread footing foundations will not be considered. Also, recommendations for drilled shaft foundations are not presented. Recommendations can be provided for additional foundation alternatives upon request.

It is recommended that HP 14x73 piles, driven to refusal on the top of rock be used to support the proposed bridge. Table 4 summarizes the site conditions and foundation recommendations for the proposed bridge. It should be noted that the bedrock surface varies across the project area. The approximate pile tip elevations presented in Table 4 indicate the approximate elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 4 - Summary of Driven Pile Tip Elevations, HP 14x73*
US 23 Ramp B over Norfolk Southern Railroad

Substructure	Boring Number	Existing Ground Surface Elevation (Ft)	Estimated Pile Tip Elevation (Ft)
Rear Abutment	B-1110	542.3	516.7
Pier 1	B-1111	543.8	517.8
Pier 2	TR-60	552.3	522.3
Forward Abutment	B-1112	560.9	525.9

** 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 25.6 to 35.0 feet below the ground surface. Based upon the degree of weathering and the strength characteristics of the shale bedrock, it is anticipated that the piles will penetrate approximately 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. It is anticipated that medium hard, black (Sunbury) shale bedrock will be encountered at the top of rock at the foundation locations for the proposed bridge. Therefore, based upon guidance from ODOT's Bridge Design Manual (BDM), it is not necessary to use reinforced pile points to protect the piles. 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. At the rear abutment, 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 90 and 95 percent of the primary consolidation has occurred at the rear and forward abutments, respectively. No waiting periods are required at the pier locations. At the abutments, fill should be placed to the proposed roadway grade level and allowed to consolidate prior to driving piles. Without using wick drains, the estimated consolidation periods (prior to driving piles) are approximately 45 and 143 days at the rear and forward abutments, respectively. Time-rate of consolidation calculations are presented in Appendix IV.

Due to schedule requirements, it may be desirable to use wick drains to accelerate the consolidation of the foundation and shorten the waiting period prior to driving piles. At the rear abutment location, the consolidation periods presented in Table 2 may be used for various drain spacing options. Similarly, at the forward abutment location, consolidation periods have been developed for various drain spacing options and are presented in Table 5.

Table 5 - Wick Drain Spacing and Consolidation Periods

US 23 Ramp B, Forward Abutment Location		
Spacing (ft)	Time to U=95% (days)	Approximate Depth of Wick Drains (ft)
5	25	20
7	35	20
9	50	20

5.3 Embankment Evaluations and Recommendations US 23 Ramp B

Global stability analyses were performed for the earthen embankment at the forward abutment. For the purposes of analyses, it was assumed that deep foundations would be used to support the structures. The assumed maximum height of embankment constructed for Ramp B near the forward abutment is approximately 30.8 feet. As per ODOT Office of Geotechnical Engineering, the following material properties were assumed for the stability analyses; 1) a cohesion value of 2000 pound per square foot (psf) and a friction angle of zero was used for the undrained analysis and 2) a cohesion value of 300 psf and a friction angle of 28 was used for the drained and seismic analyses. Based on the soils encountered in the borings and the stability analyses that were performed, this site appears to be suitable for earthen embankments, provided they are constructed with side slopes of no steeper than 2H:1V. A drawing illustrating the results of the stability analyses is included in Appendix IV.

The maximum settlement at the centerline of the forward abutment (sta. 2609+07) was estimated to be approximately 8 inches. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankment loading at the abutment location. Settlement calculations are presented in Appendix IV.

5.4 General Earthwork Recommendations

The proposed alignment traverses a gently to moderately sloping area and the proposed grade is anticipated to be a maximum of 40.5 feet higher than the existing grade. Consequently, the placement of fill will be required to construct the approach embankments at the abutments. Some excavation is anticipated for the construction of the pier foundations and the MSE wall leveling pads.

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

Five samples from four different borings drilled for other features of the interchange were tested to determine the organic content. Samples from borings B-1102, B-1103, B-1129, and B-1150 were tested. The results indicate organic contents ranging from 3.74 to 6.12 percent, which is considered to be slightly to moderately organic. Organic soils were not observed in any of the borings drilled adjacent to the bridge. 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.

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.

5.5 Groundwater Considerations

Seepage was first observed in the borings at depths between 10.0 and 26.0 feet below the ground surface. Measurable water levels were observed in some of the borings prior to rock coring at depths between 12.0 and 26.0 feet below the ground surface. However, excavations could encounter significant seepage based upon piezometer observations. The piezometer installed in boring B-1109A indicated that the phreatic level in the water-bearing granular layers is at approximate elevation 531.3, or 1.2 feet below the existing ground surface.

Based upon the borings in the area of the proposed US 23 Ramp B MSE walls, a layer of fine-grained soil is overlying the water-bearing granular layer, which is directly above the top of rock. The contractor should be prepared to dewater the excavations. In addition to groundwater, the contractor should be prepared to deal with water flow and precipitation that may enter any excavations. For additional information, refer to the boring logs presented in Appendix II.

6.0 CLOSING REMARKS

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

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams

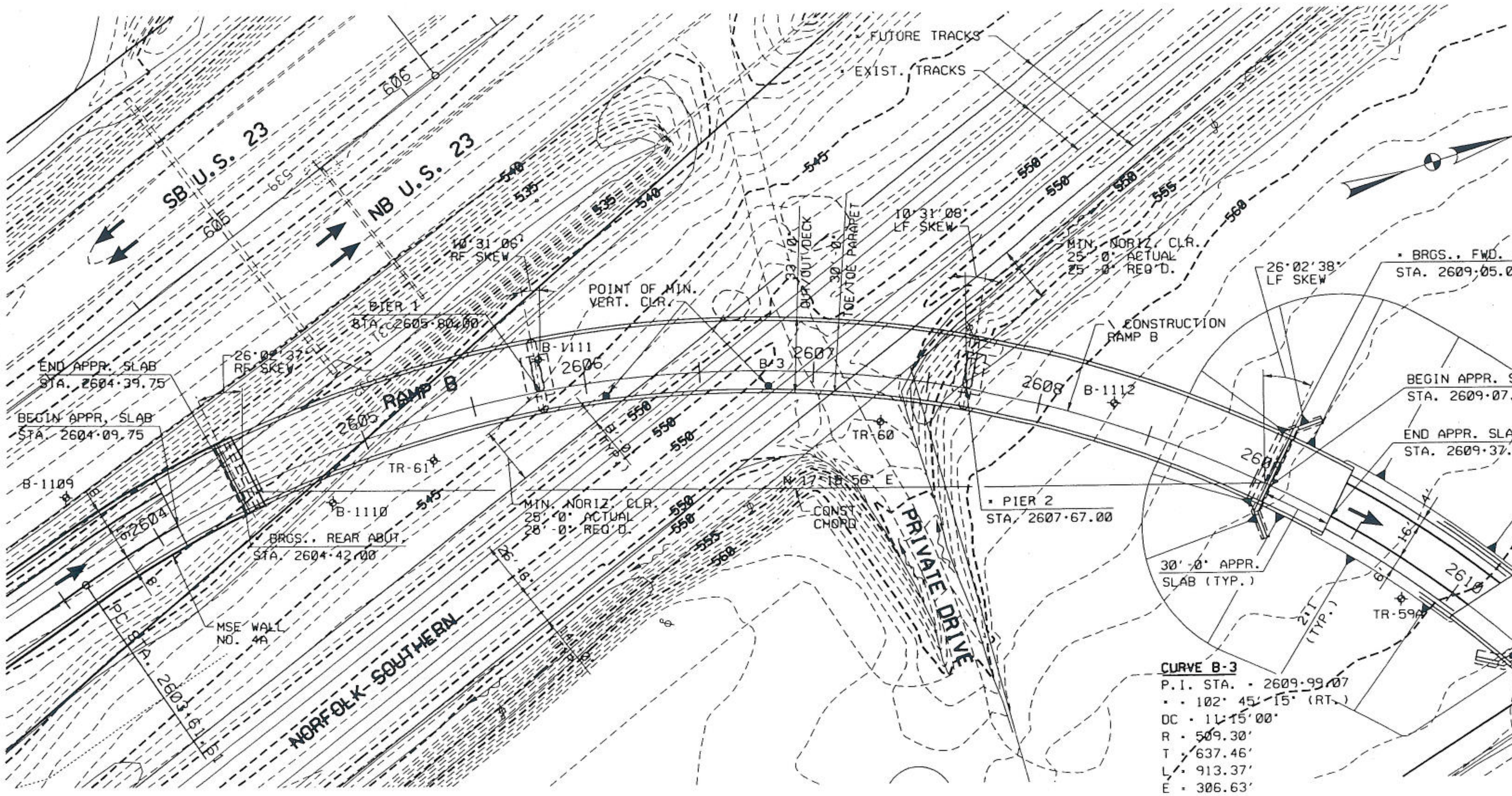
Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

sjr

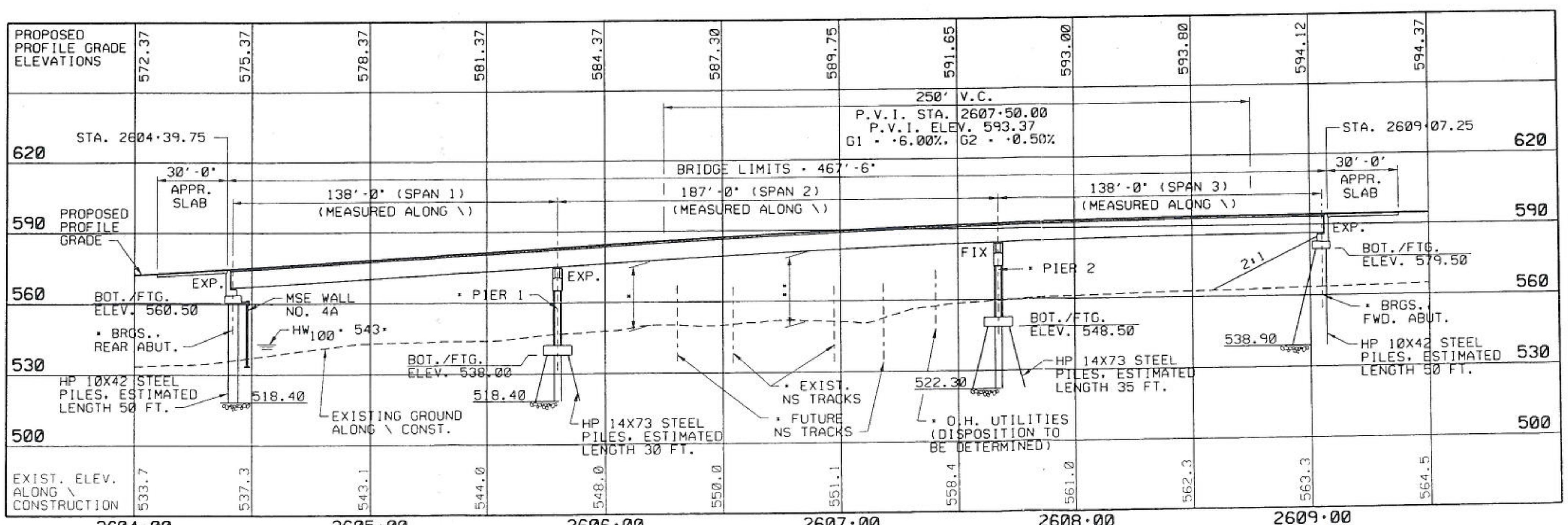
M:\proj\0121\3070.03\Interchanges\US 23\Correspondence with CH2\Final Reports\Ramp B over N-S RR\US 23 Ramp B over N-S RR-combined-Structure Report 10-19-07 sjr.doc

APPENDIX I

Structure Plan and Profile Drawings – 11"x17"
US 23 Ramp B Plan Drawing – 11"x17"



PLAN



PROFILE ALONG \ SURVEY AND CONSTRUCTION, RAMP B

• MIN. VERT. CLR. 25'-5" ACTUAL 23'-3" REQ'D.

•• MIN. VERT. CLR. 28'-5" ACTUAL 23'-2" REQ'D.

BENCHMARKS

TRAFFIC DATA

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

LEGEND

⌖ DENOTES SOIL BORING LOCATION

NOTE

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

PROPOSED STRUCTURE

TYPE: THREE SPAN COMPOSITE CURVED STEEL PLATE GIRDERS (WEATHERED ASTM A709, GR 50W) WITH REINFORCED CONCRETE DECK ON JOINTED STUB ABUTMENT ON MSE WALL (REAR) AND JOINTED STUB ABUTMENT BEHIND 2:1 EMBANKMENT (FWD.) WITH T-TYPE PIERS

LENGTH OF SPAN: 131'-0", 187'-0", 131'-0", C-C BEARINGS, MEASURED ALONG \ CONSTRUCTION

ROADWAY: 30'-0" TOE/TOE PARAPETS

SIDEWALK: NONE

DESIGN LOADING: HS25 (CASE II) AND THE ALTERNATE MILITARY LOADING, FWS · 60 LB/FT²

SKEW: 26°02'37" RF (REAR ABUTMENT), 10°31'06" RF (PIER 1), 10°31'08" LF (PIER 2), 26°02'38" LF (FORWARD ABUTMENT), 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 FT.)

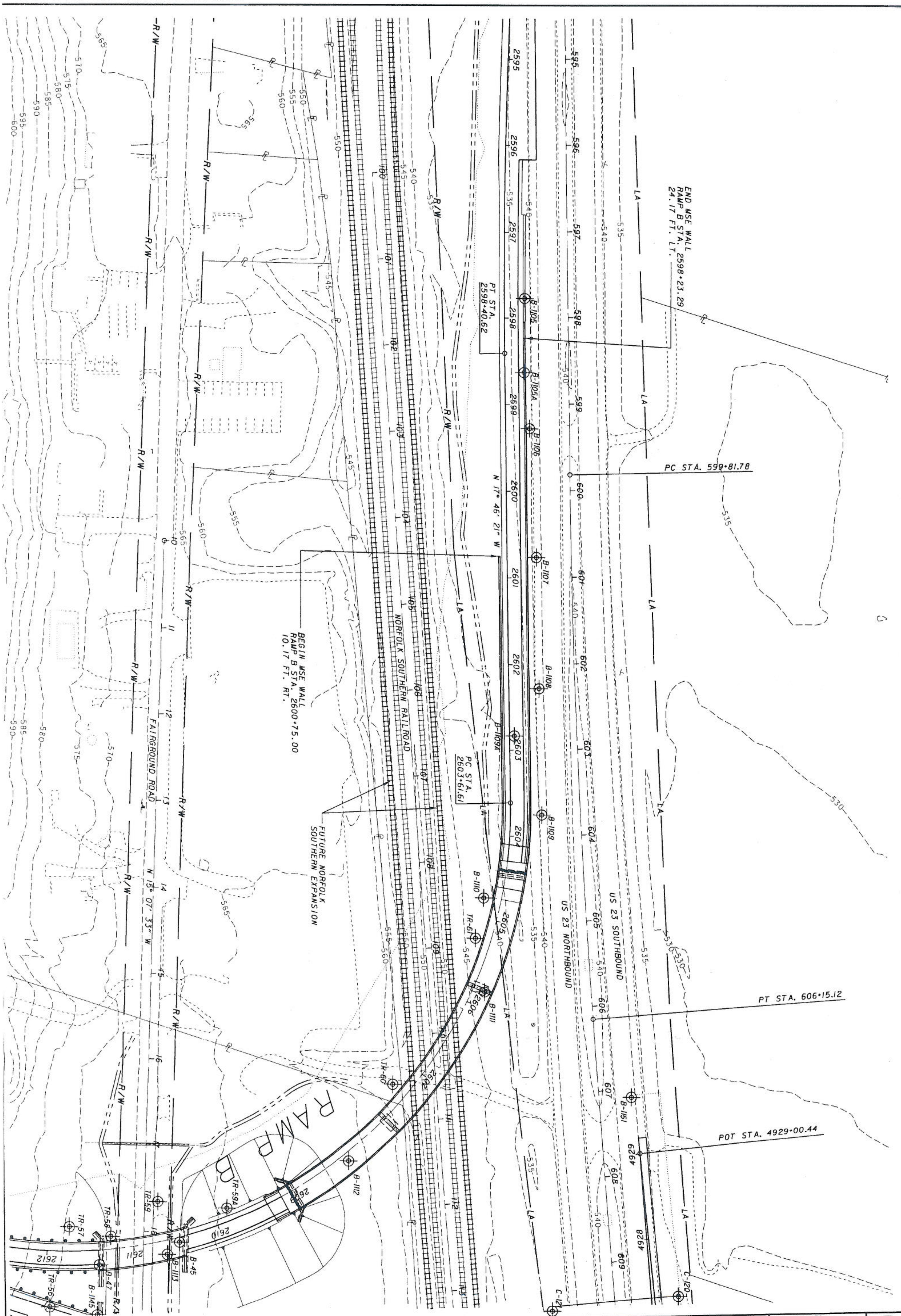
SUPERELEVATION: 0.071 FT/FT

LATITUDE: N 38°53'28"

LONGITUDE: W 82°59'54"

DESIGN AGENCY	CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017
REVIEWED WRT	DATE 06/07
STRUCTURE FILE NUMBER	7306717
DRAWN JBA	REVISED
DESIGNED DGS	CHECKED SCJ
SCIOTO COUNTY	STA. 2604·39.75 TO STA. 2609·07.25
SITE PLAN	BRIDGE NO. SCI-823-1598 RAMP B OVER NORFOLK SOUTHERN - ALT. 1
SCI-823-10.13	PID 79977
1 / 3	

0/18/2007
:47:54 PM
:\PROJ\0121\3070.03\PROJECT PLANS\STEVEN\RAMPB_PLANV8.DGN



SCI-823	PID NO. 79977	US 23 RAMP B PLAN DRAWING

HORIZONTAL SCALE IN FEET

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Thirteen (13) Borings
Piezometer Installation Report

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

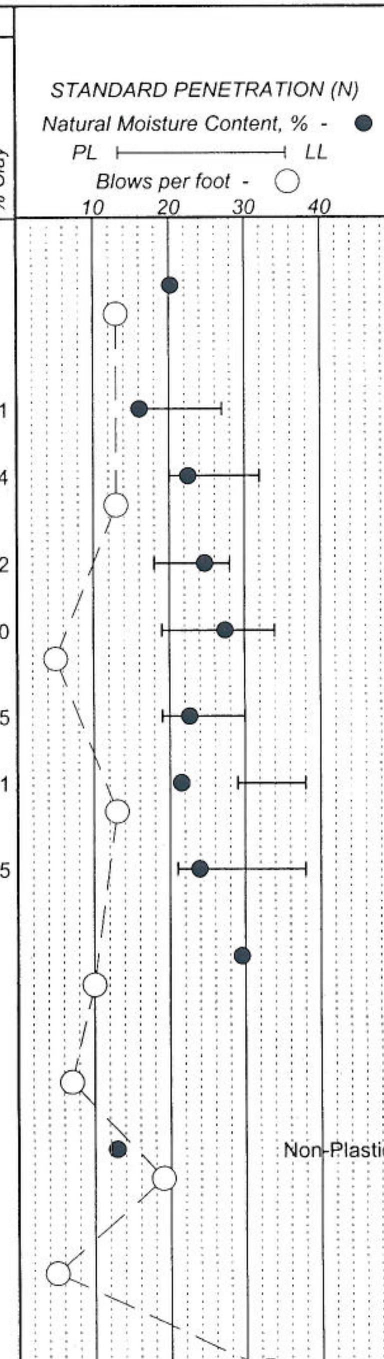
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-1105A | Location: Sta. 2598+63.3, 22.4 ft. LT of US 23 Ramp B BL | Date Drilled: 07/12/07

WATER OBSERVATIONS: Water seepage at: 22.3'-29.7'
Water level at completion: 21.0'

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	538.0												
0.3	537.7						Topsoil - 4"						
		3 5 8	13	1		4.5+	Very stiff to hard brown SILT AND CLAY (A-6a), some fine sand, trace coarse sand, trace gravel; damp to moist.						
5					ST1	2.5		1	3	--	22	43	31
							@ 5.5', stiff to very stiff, brownish gray, trace fine sand, moist.						
		4 6 7	18	2		2.0		1	1	--	3	61	34
7.8	530.2				ST2	1.0	Medium stiff to stiff mottled brown and gray SILT (A-4b), some clay, trace fine to coarse sand; trace gravel; moist to wet.	2	3	--	6	57	32
10.0	528.0	WOH 2 3	18	3		0.25	Very soft to soft mottled brown and gray SILT AND CLAY (A-6a), trace fine to coarse sand; moist.	0	4	--	5	51	40
					ST3	2.0	@ 12.9', stiff to very stiff, gray.	0	2	--	5	58	35
14.0	524.0	4 5 8	18	4		3.0	Very stiff mottled brown and gray SANDY SILT (A-4a), trace fine sand; damp.	0	0	--	2	47	51
15													
16.0	522.0				ST4	2.25	Very stiff mottled brown and gray SILTY CLAY (A-6b), trace fine to coarse sand; damp to moist.	0	1	--	2	42	55
		3 4 6	12	5		2.5	@ 18.5', little to some gravel.						
20							@ 20.5', medium stiff, little fine to coarse sand, moist.						
22.0	516.0	2 3 4	18	6A 6B		0.75	Loose to medium dense brown GRAVEL WITH SAND (A-1-b), trace to little silty clay; wet.						
		3 5 14	14	7				39	40	--	10	11	
25													
		2 2 3	18	8									
		4 16 17	12	9A			@ 29.5', contains sandstone fragments.						
30													



FILE: 0121-3070-03 | 10/19/2007 12:07 PM

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

Client: TranSystems, Inc.

Project: SCI-823-0.00

Job No. 0121-3070.03

LOG OF: Boring B-1105A

Location: Sta. 2598+63.3, 22.4 ft. LT of US 23 Ramp B BL

Date Drilled: 07/12/07

WATER OBSERVATIONS: Water seepage at: 22.3'-29.7'
Water level at completion: 21.0'

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30.0	508.0			9B			Severely weathered gray SANDSTONE.							
31.3	506.7	50/3	3	10			Bottom of Boring - 31.3'							50+

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

Depth scale (ft): 35, 40, 45, 50, 55, 60

FILE: 0121-3070-03 (10/19/2007 12:07 PM)

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

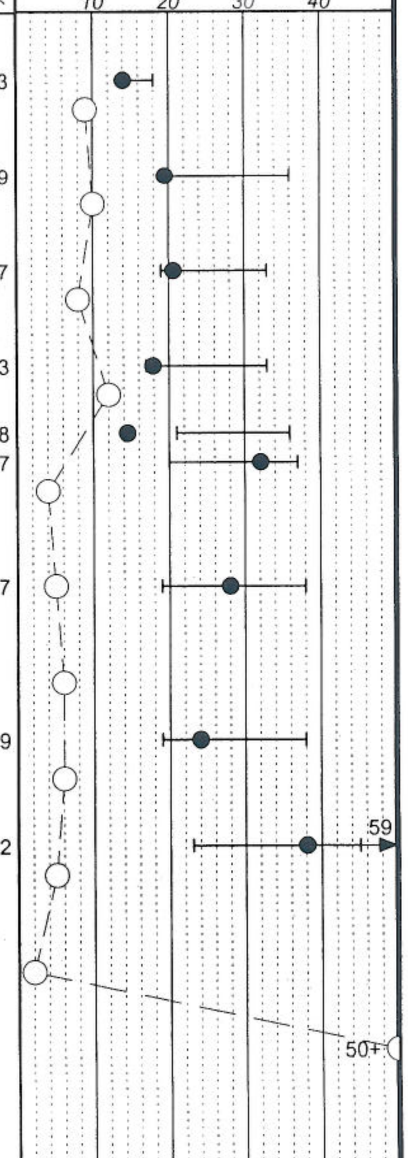
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-1108 | *Location:* Sta. 2602+29.5, 35.3 ft. LT of US 23 Ramp B BL | *Date Drilled:* 07/21/05

WATER OBSERVATIONS: Water seepage at: 25.0'
Water level at completion: 25.0' (prior to coring)
8.5' (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	540.7						Asphalt - 4" Aggregate Base - 8"						
1.0	539.7	3 5 4	12	1		1.5	FILL: Stiff dark brown SANDY SILT (A-4a), little clay, little gravel; damp to moist.	10	18	--	28	31	13
3.0	537.7	3 4 6	10	2		3.0	FILL: Very stiff dark brown SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; damp to moist.	6	4	--	8	43	39
5.5	535.2	2 3 5	16	3		--	FILL: Stiff to very stiff gray SILT AND CLAY (A-6a), little fine to coarse sand, little gravel; slightly organic; moist.	10	4	--	8	51	27
8.0	532.7	3 5 7	13	4		4.5+	Hard brown and gray SILTY CLAY (A-6b), trace to little fine to coarse sand, trace gravel; moist.	7	6	--	10	44	33
10.0	530.7				P1	3.75	Very stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand; damp.	0	1	--	3	58	38
		2 2 2	14	5		1.0		3	0	--	1	59	37
12.0	528.7						Stiff brown SILTY CLAY (A-6b), trace to little fine to coarse sand, trace gravel; moist.						
		2 2 3	16	6		1.5							
15					P2			2	4	--	9	48	37
		1 3 3	15	7		1.75							
		2 2 4	15	8	P3	1.0		1	4	--	5	51	39
20													
21.5	519.2	1 2 3	18	9		1.5	Stiff brown CLAY (A-7-6), trace gravel, trace fine sand; moist.	2	0	--	1	25	72
23.0	517.7	WOH 1 1	15	10			Very loose to loose gray COARSE AND FINE SAND (A-3a), little gravel, trace clay; wet.						
25													
26.5	514.2	6 50/4	9	11			Severely weathered black SILTSTONE, carbonaceous.						
28.9	511.8						Hard gray SANDSTONE interbedded with SILTSTONE.						
30													

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



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

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

LOG OF: Boring B-1108 | Location: Sta. 2602+29.5, 35.3 ft. LT of US 23 Ramp B BL | Date Drilled: 07/21/05

WATER OBSERVATIONS: Water seepage at: 25.0'
Water level at completion: 25.0' (prior to coring)
8.5' (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) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ 10 20 30 40
30	510.7	Core 60"	Rec 53"	RQD 78%	R1		Hard gray SANDSTONE interbedded with SILTSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, medium bedded, slightly fractured. @ 28.9'-29.1', 31.4'-31.7', 32.2'-33.2', high angle fractures.							
33.5	507.2						Bottom of Boring - 33.5'							
35														
40														
45														
50														
55														
60														

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

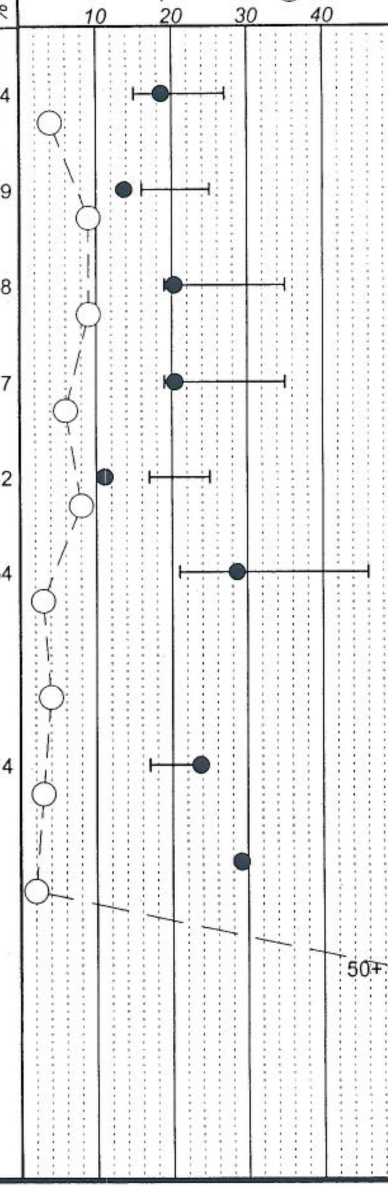
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring B-1109 | *Location:* Sta. 2603+75.4, 36.5 ft. LT of US 23 Ramp B BL | *Date Drilled:* 07/22/05

WATER OBSERVATIONS: Water seepage at: 19.0'-22.0'
Water level at completion: 19.0' (prior to coring)
8.0' (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	540.6						Asphalt - 4" Aggregate Base - 8"						
1.0	539.6	3 2 2	12	1		1.5	FILL: Stiff dark brown SILT AND CLAY (A-6a), some fine to coarse sand, trace gravel; moist.	7	10	--	11	48	24
3.0	537.6	2 4 5	14	2		2.0	FILL: Stiff to very stiff dark brown SANDY SILT (A-4a), some gravel, little clay; contains wood fragments; damp.	22	15	--	12	32	19
5.5	535.1	3 4 5	15	3		3.5	POSSIBLE FILL: Very stiff to hard grayish brown SILTY CLAY (A-6b), trace fine to coarse sand, trace gravel; slightly organic; moist.	1	3	--	7	51	38
		2 3 3	13	4		4.25		1	4	--	11	47	37
10.5	530.1	1 4 4	10	5			Medium stiff brown SANDY SILT (A-4a), some gravel, little clay; moist.	33	17	--	14	24	12
13.0	527.6	1 1 2	15	6		1.5	Stiff gray CLAY (A-7-6), some silt, trace fine to coarse sand, trace gravel; moist to wet.	1	3	--	7	35	54
		1 1 3	12	7		2.0							
18.0	522.6	1 1 2	18	8		--	Soft brown SANDY SILT (A-4a), little clay, trace gravel; wet.	3	12	--	32	39	14
		WOH 1 1	16	9		--		0	1	--	52	47	
23.5	517.1	10 50/5	9	10			Severely weathered black SHALE, carbonaceous.						
25.0	515.6	Core 60"	Rec 60"	RQD 77%	R1		Soft to medium hard black SHALE; very fine grained, moderately weathered to decomposed, carbonaceous, thinly laminated, highly fractured. @ 28.0'-28.1', 28.3'-28.6', high angle fracture.						
28.3	512.3						Medium hard to hard gray SANDSTONE; fine grained, highly weathered, micaceous, argillaceous, massive, slightly						
30.0	510.6												

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



FILE: 0121-3070-03 (10/19/2007 12:07 PM)

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

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

LOG OF: Boring B-1109 | Location: Sta. 2603+75.4, 36.5 ft. LT of US 23 Ramp B BL | Date Drilled: 07/22/05

WATER OBSERVATIONS: Water seepage at: 19.0'-22.0'
Water level at completion: 19.0' (prior to coring)
8.0' (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) Natural Moisture Content, % - ● PL ├─┤ LL Blows per foot - ○ 10 20 30 40
30	510.6						fractured. Bottom of Boring - 30.0'							
35														
40														
45														
50														
55														
60														

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

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

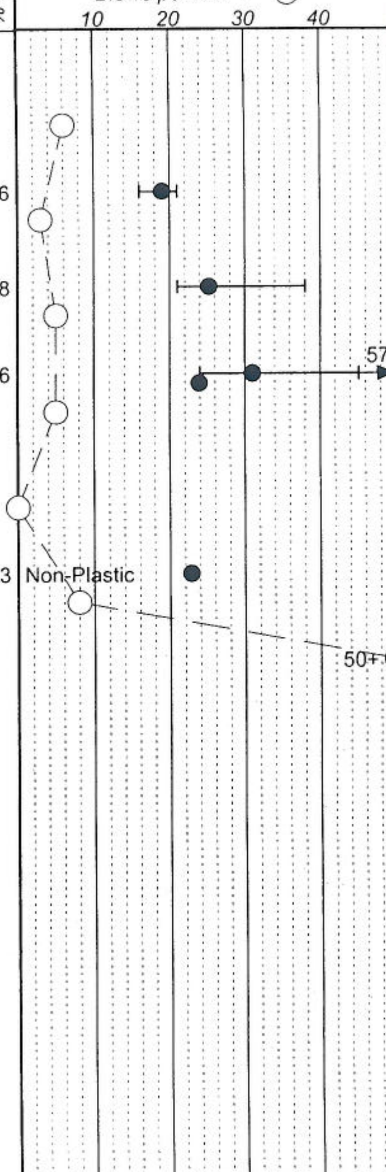
LOG OF: Boring B-1109A

Location: Sta. 2602+83.8, 5.6 ft. LT of US 23 Ramp B BL | Date Drilled: 07/12/07

WATER OBSERVATIONS: Water seepage at: 13.5'-15.0'
Water level at completion: 1.8' (includes surface 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	532.5												
0.5	532.0						Topsoil - 6"						
		WOH 2 4	12	1		1.75	POSSIBLE FILL: Stiff brownish gray SANDY SILT (A-4a), little clay, little gravel; contains roots, coal fragments, and plant debris; moist. @ 3.0', wet.						
		WOH 2 1	8	2		-		13	22	--	21	28	16
5.5	527.0	1 2 3	18	3	ST1	1.5	Stiff mottled brown and gray SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; moist.	3	6	--	10	33	48
8.0	524.5	1 2 3	18	4	ST2	1.25 1.0	Medium stiff to stiff mottled brown and gray CLAY (A-7-6), little to some silt, trace fine to coarse sand; moist.	0	1	--	3	20	76
					ST3								
11.3	521.2	W O H	3	5		0.5	Very loose to loose brown SANDY SILT (A-4a), little clay, trace gravel; wet.						
		WOH 2 6	10	6			Note: No recovery in ST1 and ST3.	9	8	--	38	32	13
16.0	516.5	50/3	3	7			Severely weathered black carbonaceous SHALE.						
16.3	516.2						Bottom of Boring - 16.3'						

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



FILE: 0121-3070-03 [10/19/2007 12:08 PM]

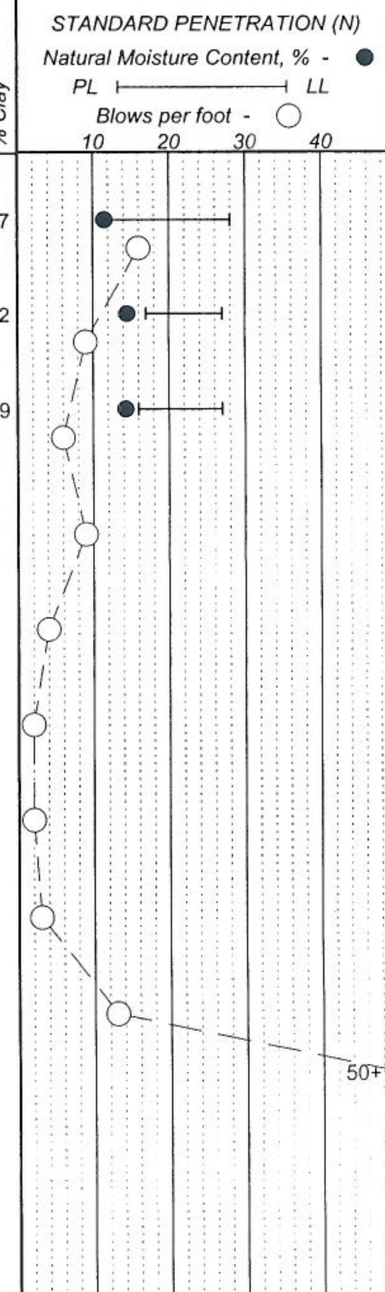
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-1110 | Location: Sta. 2604+76.5, 20.7 ft. RT of US 23 Ramp B BL | Date Drilled: 07/14/05

WATER OBSERVATIONS: Water seepage at: 12.0'-25.0'
Water level at completion: 12.0' (prior to coring)
5.0' (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	542.3												
0.5	541.8						Topsoil - 6"						
		6 8 8	18	1		4.5+	Hard brown SILTY CLAY (A-6b), little fine to coarse sand, trace gravel; damp.	1	4	--	11	57	27
3.0	539.3												
		6 5 4	18	2		3.5	Very stiff brown SILT (A-4b), some clay, little fine to coarse sand; damp.	0	7	--	11	60	22
5.5	536.8												
		2 3 3	15	3		4.0	Very stiff brown SILT AND CLAY (A-6a), "and" fine to coarse sand, trace gravel; moist.	3	15	--	25	28	29
		1 4 5	16	4		2.5							
		2 2 2	18	5		2.0							
13.0	529.3												
		1 1 1	6	6			Very loose brown COARSE AND FINE SAND (A-3a), little clay, little gravel; wet.						
		1 1 1	10	7									
18.0	524.3												
		2 1 2	16	8			Loose to medium dense brown GRAVEL WITH SAND (A-1-b), little clay, little silt; wet.						
		9 6 7	9	9									
23.0	519.3												
		50/5	7	10			Severely weathered black SHALE, carbonaceous.						
25.0	517.3												
25.6	516.7						Soft black SHALE; decomposed, carbonaceous, thinly laminated, moderately fractured.						
		Core 60"	Rec 52"	RQD 32%	R1		Medium hard black SHALE; unweathered, carbonaceous, thinly laminated, slightly fractured. @ 27.8'-28.0', 29.3'-29.5', high angle fractures.						
30.0	512.3						Bottom of Boring - 30.0'						



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

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

LOG OF: Boring B-1110 | Location: Sta. 2604+76.5, 20.7 ft. RT of US 23 Ramp B BL | Date Drilled: 07/14/05

WATER OBSERVATIONS:
Water seepage at: 12.0'-25.0'
Water level at completion: 12.0' (prior to coring)
5.0' (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	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	512.3													
35														
40														
45														
50														
55														
60														

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

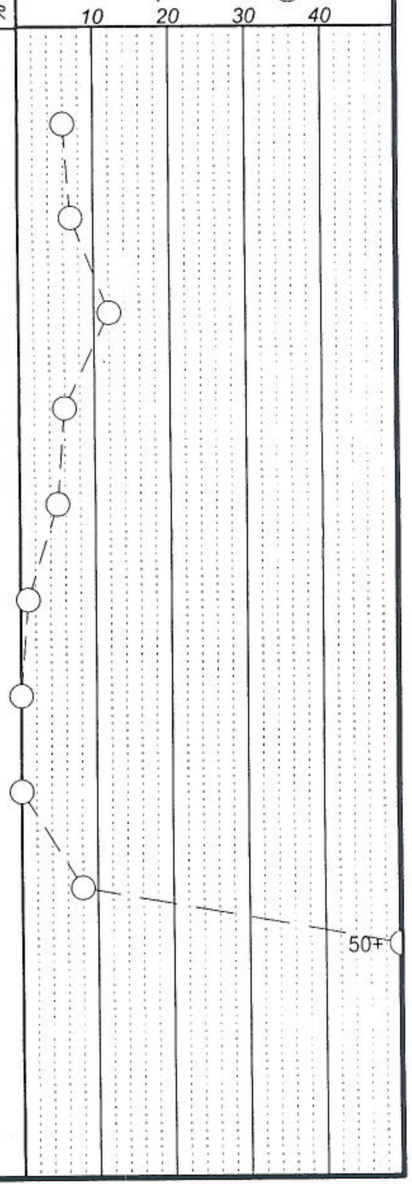
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1111 | Location: Sta. 2605+81.7, 14.0 ft. LT of US 23 Ramp B BL | Date Drilled: 11/2/05

WATER OBSERVATIONS: Water seepage at: 10.0'-22.5'
Water level at completion: 8.0' (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	543.8												
0.3	543.5						Topsoil - 3"						
		3 3 3	13	1		2.5	FILL: Very stiff to hard brown SANDY SILT (A-4a), little clay, trace gravel; damp.						
		4 3 4	10	2		4.5+							
5.5	538.3						Very stiff to hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; moist.						
		4 5 7	15	3		4.25							
		2 3 3	12	4		2.0	@ 8.5', some gravel, some fine to coarse sand.						
10.5	533.3						Loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.						
		2 2 3	12	5									
13.0	530.8						Very loose brown COARSE AND FINE SAND (A-3a), little clay, little gravel; wet.						
		WOH WOH 1	16	6									
		W O H	18	7									
		W O H	11	8									
20.5	523.3						Loose brown GRAVEL WITH SAND AND SILT (A-2-4), little clay; wet.						
		WOH 2 6	12	9									
23.5	520.3	50/4	2	10			Severely weathered black SHALE, carbonaceous.						
25.0	518.8						@ 25.0'-25.4', broken zone.						
		Core 120"	Rec 120"	RQD 57%	R1		Medium hard to hard black SHALE; slightly to moderately weathered, argillaceous, carbonaceous, thinly laminated, moderately fractured, contains turbidity. @ 26.4'-26.5', 27.2', low angle fractures.						
30													

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



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

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

LOG OF: Boring B-1111 | Location: Sta. 2605+81.7, 14.0 ft. LT of US 23 Ramp B BL | Date Drilled: 11/2/05

WATER OBSERVATIONS: Water seepage at: 10.0'-22.5'
Water level at completion: 8.0' (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
30.0	513.8						Hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, medium bedded to thickly bedded, moderately to highly fractured. @ 31.5'-31.7', 32.1', low angle fractures. @ 32.7', 32.9', 33.6'-34.2', fractures with decomposed zones.							
35.0	508.8						Bottom of Boring - 35.0'							
40														
45														
50														
55														
60														

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

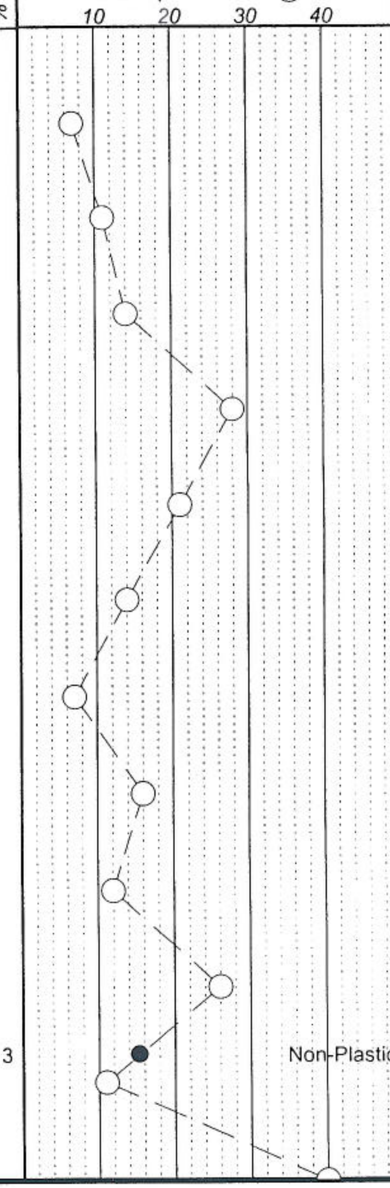
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1112 | Location: Sta. 2608+31.5, 9.0 ft. LT of US 23 Ramp B BL | Date Drilled: 10-12-05

WATER OBSERVATIONS: Water seepage at: 26.0'-30.0'
Water level at completion: None (prior to coring)
6.6' (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	560.9												
0.3	560.6						Topsoil - 3"						
		3 3 4	9	1		4.0	FILL: Very stiff to hard brown SILT AND CLAY (A-6a), little gravel, trace fine to coarse sand; moist.						
3.0	557.9	5 5 6	18	2			FILL: Medium dense brown and dark gray SANDY SILT (A-4a), trace clay, trace gravel; damp.						
5.5	555.4	5 7 7	15	3			POSSIBLE FILL: Medium dense brown COARSE AND FINE SAND (A-3a), trace to little gravel; dry.						
10		13 14 14	10	4									
		4 10 11	11	5									
15		8 8 6	9	6									
		3 3 4	17	7			@ 16.0', little silt, little clay; damp to moist.						
20		5 8 8	6	8									
		5 6 6	13	9									
23.0	537.9	9 11 15	1	10			POSSIBLE FILL: Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little silt, trace clay; wet.						
25		3 5 6	14	11				26	39	--	19	13	3
30		6 25 15	11	12									

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



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

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

LOG OF: Boring B-1112 | *Location:* Sta. 2608+31.5, 9.0 ft. LT of US 23 Ramp B BL | *Date Drilled:* 10-12-05

WATER OBSERVATIONS: Water seepage at: 26.0'-30.0'
Water level at completion: None (prior to coring)
6.6' (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	530.9						POSSIBLE FILL: Medium dense to dense brown GRAVEL WITH SAND (A-1-b), little silt, trace clay; wet.							
33.0	527.9	Core 120"	Rec 120"	RQD 83%	R1		Medium hard black SHALE; moderately to highly weathered, carbonaceous, thinly bedded, moderately fractured. @ 33.9'-34.0', broken zone.							
43.0	517.9						Bottom of Boring - 43.0'							

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

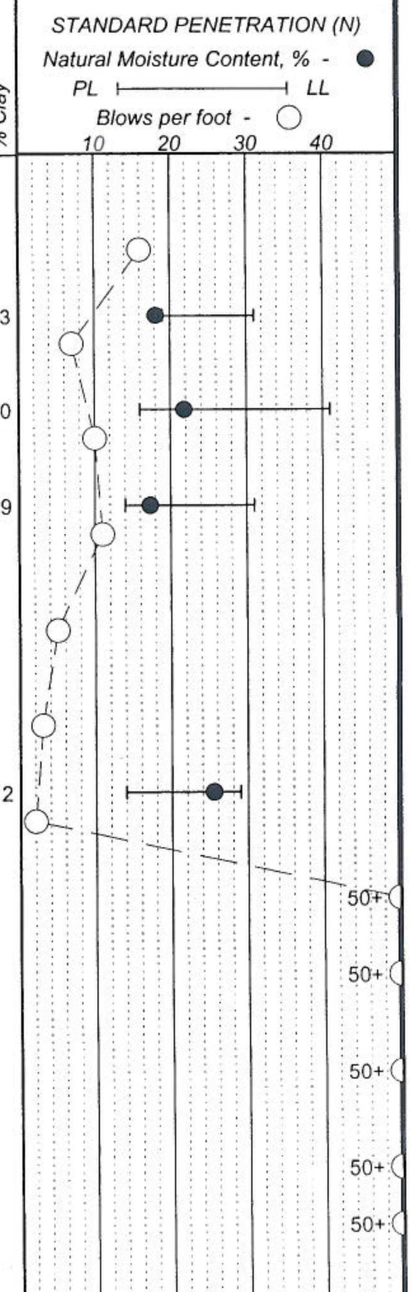
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring B-1113 | Location: Sta. 2610+64.3, 16.0 ft. LT of US 23 Ramp B BL | 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	3 4 6	16	3		4.0	Very stiff to hard brown and gray CLAY (A-7-6), "and" silt, little fine to coarse sand; moist.	0	1	--	12	47	40
8.0	558.8	5 4 7	18	4		1.5	Stiff brown and gray SILTY CLAY (A-6b), "and" fine to coarse sand, little gravel; moist.	15	15	--	28	23	19
		2 3 2	17	5		1.5							
		2 1 2	16	6		1.0							
15.5	551.3	2 1 1	18	7			Very loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.	18	37	--	25	8	12
18.0	548.8	34 50/5	11	8			Severely weathered gray SHALE, micaceous.						
		50/5	5	9									
		50/4	4	10									
		50/4	4	11									
		50/4	4	12									
30													



FILE: 0121-3070-03 (10/19/2007 12:08 PM)

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

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

LOG OF: Boring B-1113 | Location: Sta. 2610+64.3, 16.0 ft. LT of US 23 Ramp B BL | 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 Penetro-meter (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N)
30	536.8	Core 48"	Rec 33"	RQD 0%	R1		Severely weathered gray SHALE, micaceous.							
		50/5	5	13										50+
		50/4	4	14										50+
		50/2	2	15										50+
44.0	522.8	Core 60"	Rec 56"	RQD 58%	R2		Medium hard black SHALE; slightly to moderately weathered, carbonaceous, thinly laminated, slightly fractured. @ 45.1', 47.2', 48.9', decomposed fractures.							
49.0	517.8						Bottom of Boring - 49.0'							

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

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

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

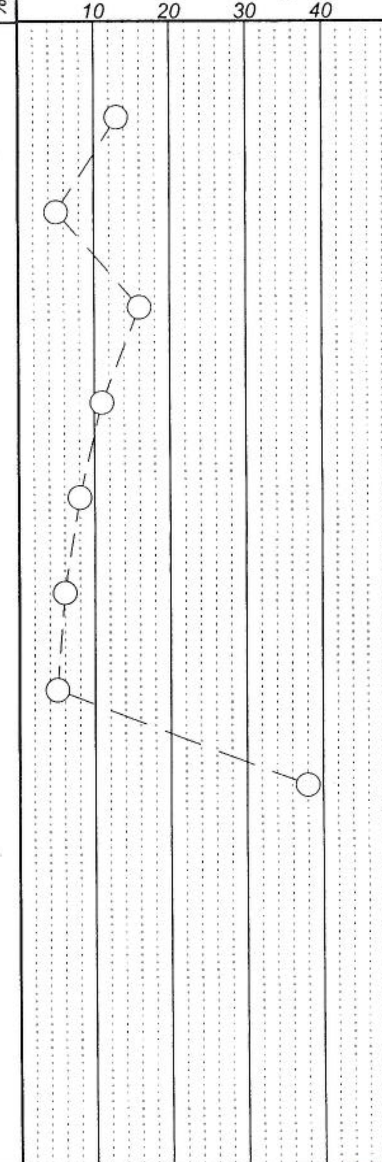
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-59 | Location: Sta. 2610+61.7, 45.7 ft. RT of US 23 Ramp B BL | 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 (10/19/2007 12:09 PM)

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

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

LOG OF: Boring TR-59 | Location: Sta. 2610+61.7, 45.7 ft. RT of US 23 Ramp B BL | 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)
30	537.3						Medium hard gray SANDSTONE.							
31.0	536.3						Bottom of Boring - 31.0'							

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

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

FILE: 0121-3070-03 [10/19/2007 12:09 PM]

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

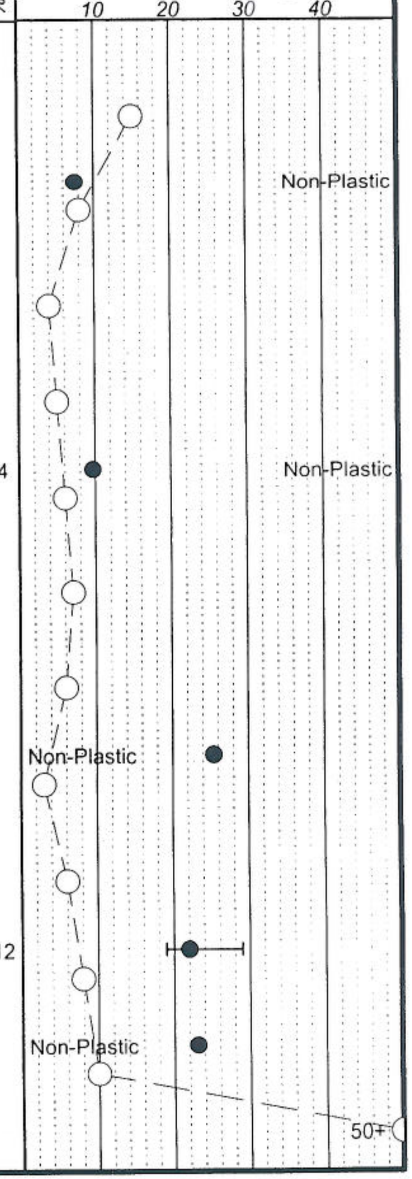
Client: TranSystems, Inc. | *Project:* SCI-823-0.00 | *Job No.* 0121-3070.03

LOG OF: Boring TR-60 | *Location:* Sta. 2607+31.2, 19.6 ft. RT of US 23 Ramp B BL | *Date Drilled:* 3/14/05

WATER OBSERVATIONS:
Water seepage at: 18.0'-28.0'
Water level at completion: 26.0' (prior to coring)
19.0' (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	552.3												
0.1	552.2						Topsoil - 1"						
		4 8 7	12	1			FILL: Medium dense brown SANDY SILT (A-4a), little gravel, little clay; damp.						
3.0	549.3	4 4 4	12	2			Loose brown GRAVEL WITH SAND (A-1-b), little silt, trace clay; damp.	33	43	--	11	13	
5		3 2 2	9	3									
		3 2 3	13	4									
10		3 3 3	14	5				50	20	--	9	17	4
		3 3 4	1	6			@ 13.5', moist.						
15		2 3 3	14	7									
18.0	534.3	1 1 2	17	8			Very loose to loose brown COARSE AND FINE SAND (A-3a), little clay, trace gravel; wet.	10	53	--	20	17	
20		4 3 3	16	9									
23.0	529.3	7 4 4	18	10			Loose brown GRAVEL WITH SAND AND SILT (A-2-4), little clay; wet.	31	27	--	12	18	12
25 / 25.5	526.8	3 6 4	18	11			Loose reddish brown COARSE AND FINE SAND (A-3a), some clay, trace gravel; wet.	7	14	--	58	21	
28.0	524.3	50/4	4	12			Severely weathered black SHALE.						
30													

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



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

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

LOG OF: Boring TR-60

Location: Sta. 2607+31.2, 19.6 ft. RT of US 23 Ramp B BL | Date Drilled: 3/14/05

WATER OBSERVATIONS: Water seepage at: 18.0'-28.0'
Water level at completion: 26.0' (prior to coring)
19.0' (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)
30.0	522.3	Core 120"	Rec 119"	RQD 79%	R-1		Soft to medium hard black SHALE; carbonaceous, moderately weathered to decomposed, laminated, slightly fractured. @ 30.0'-32.3', clay seam. @ 32.3', hard. @ 33.2', 38.0'-38.2', clay seams. @ 39.4'-39.8', high angle fracture. @ 39.9'-40.0', hard gray SANDSTONE.							
40.0	512.3						Bottom of Boring - 40.0'							

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

FILE: 0121-3070-03 [10/19/2007 12:09 PM]

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

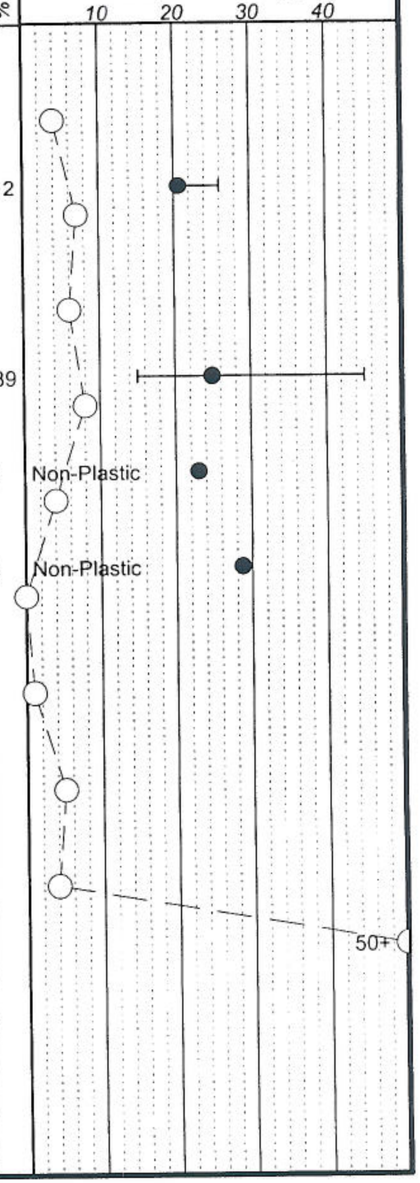
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-61 | Location: Sta. 2605+26.3, 18.1 ft. RT of US 23 Ramp B BL | Date Drilled: 3/16/05

WATER OBSERVATIONS: Water seepage at: 13.5'-23.0'
Water level at completion: 14.0' (prior to coring)
9.0' (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	543.4												
0.3	543.1						Topsoil - 4"						
		2 2 2	2	1			FILL: Loose black SANDY SILT (A-4a), little clay, little gravel; organic; dry to damp.						
		3 4 3	1	2				14	20	--	26	28	12
5.5	537.9	2 3 3	16	3		2.5	Very stiff light brown CLAY (A-7-6), some fine to coarse sand, trace gravel; damp.						
		1 3 5	12	4		2.25	@ 8.5', brown.	8	12	--	12	29	39
10.5	532.9	1 2 2	13	5			Very loose brown GRAVEL WITH SAND (A-1-b), little silty clay; moist to wet.	9	46	--	32	13	
13.0	530.4	W O H	16	6			Very loose brown COARSE AND FINE SAND (A-3a), little silty clay, trace gravel; wet.	1	22	--	62	15	
		WOH WOH 1	18	7									
17.0	526.4	1 3 2	18	8			Very loose to loose brown GRAVEL WITH SAND (A-1-b), little silty clay; moist to wet.						
		1 1 3	18	9									
23.0	520.4	50/3	3	10			Severely weathered black SHALE.						
25.0	518.4	Core 120"	Rec 114"	RQD 92%	R-1		Hard black SHALE; carbonaceous, moderately weathered, thinly bedded, moderately fractured. @ 25.0'-25.2', 27.5'-27.6', 28.1'-28.2', 29.3'-30.0', high angle fractures						
30													

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-61 | *Location:* Sta. 2605+26.3, 18.1 ft. RT of US 23 Ramp B BL | *Date Drilled:* 3/16/05

WATER OBSERVATIONS:
Water seepage at: 13.5'-23.0'
Water level at completion: 14.0' (prior to coring)
9.0' (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	513.4													
30.5	512.9						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, thinly to medium bedded, slightly fractured. @ 31.2'-31.6', high angle fracture. @ 33.7'-33.9', clay seam.							
35.0	508.4						Bottom of Boring - 35.0'							
40														
45														
50														
55														
60														

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

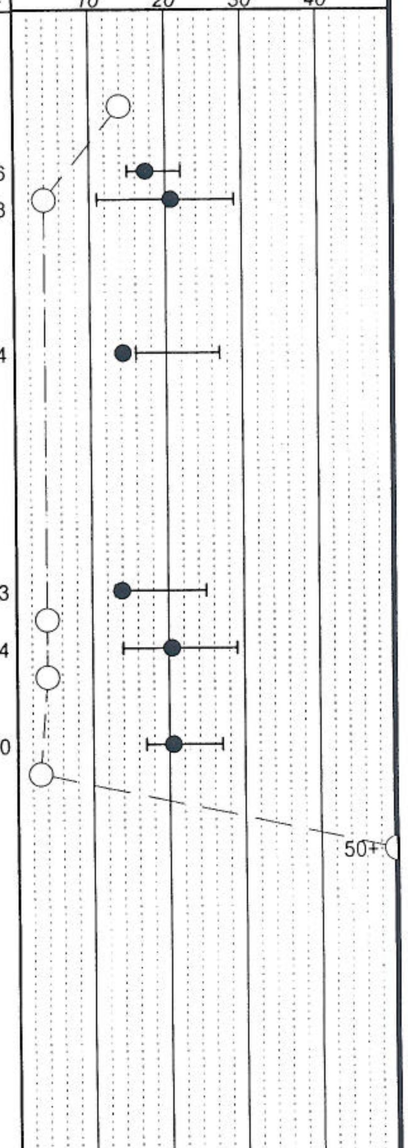
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-45 | Location: Sta. 2610+47.6, 5.8 ft. LT of US 23 Ramp B BL | 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
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.						
							@ 3.0', brown, trace gravel.						
		3 2 2	15	2		1.0		0	5	--	18	51	26
5.0	561.0							0	3	--	20	49	28
					P-1	1.25	Stiff brown SILTY CLAY (A-6b), little to some fine sand, trace coarse sand; moist.						
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
10													
					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						Loose brown GRAVEL WITH SAND AND SILT (A-2-4), some silty clay; moist to wet.						
		WOH 1 2	8	5				47	20	--	7	17	10
20													
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 - ○
10 20 30 40



FILE: 0121-3070-03 [10/19/2007 12:09 PM]

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

LOG OF: Boring B-45 | *Location:* Sta. 2610+47.6, 5.8 ft. LT of US 23 Ramp B BL | *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)
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														

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

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

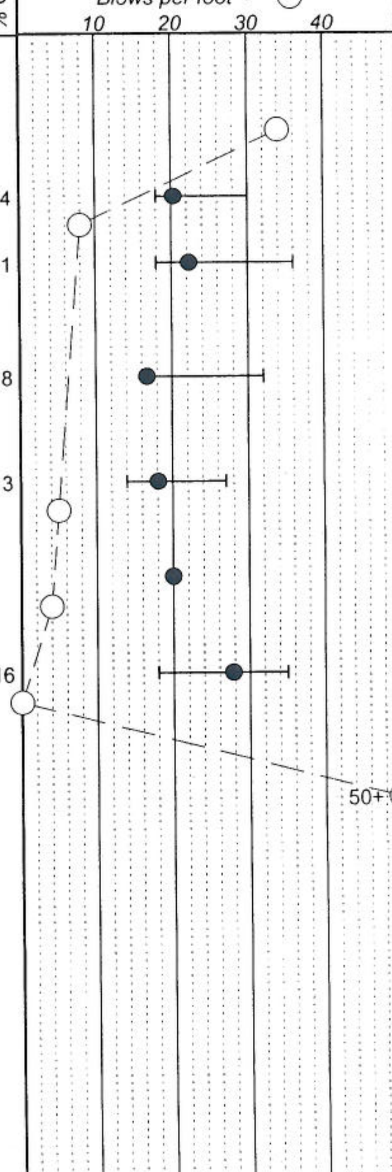
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-46 | Location: Sta. 3892+81.3, 20.3 ft. LT of US 23 Ramp C BL | 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		Core	Rec	RQD	R-2		@ 25.2', qu=4,011 psi.						
30													

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 B-46 | Location: Sta. 3892+81.3, 20.3 ft. LT of US 23 Ramp C BL | Date Drilled: 6/15/07

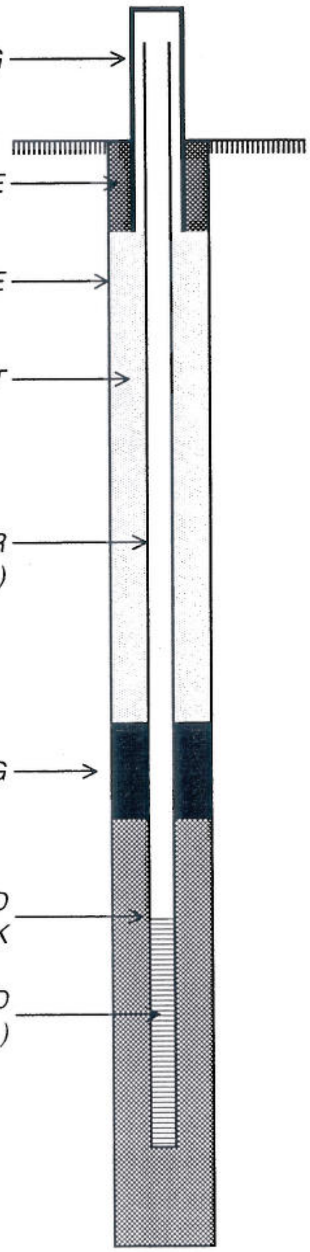
WATER OBSERVATIONS: Water seepage at: 15.5'
Water level at completion: 4.3' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand 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	535.6	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

MONITORING WELL INSTALLATION REPORT

PROJECT:	SCI-823 Portsmouth Bypass	BORING NO.:	B-1109A
PROJECT NO:	0121-3070.03	DATE INSTALLED:	7/30/2007
GROUND SURFACE ELEVATION:	532.5'	LOGGED BY:	S. Larimer
TOP OF RISER PIPE ELEVATION:	535.2	GROUND WATER LEVEL AT COMPLETION:	El. 530.7
		Average Reading	***El. 531.3***



DEPTH BELOW GROUND SURFACE	
7.5'	TOP OF PLUG
9.0'	TOP OF SAND
11.0'	TOP OF SCREEN
16.0'	BOTTOM OF SCREEN
NA	BOTTOM OF SAND PACK

REMARKS AND NOTES

Steel Protective Cover Top 3.1' AGL

Top of PVC Riser 2.7' AGL

1" Diameter

No Sand Pack Under Screen at 16.0'

DLZ

DLZ Ohio, Inc.

ENGINEERS * ARCHITECTS * SCIENTISTS
PLANNERS * SURVEYORS

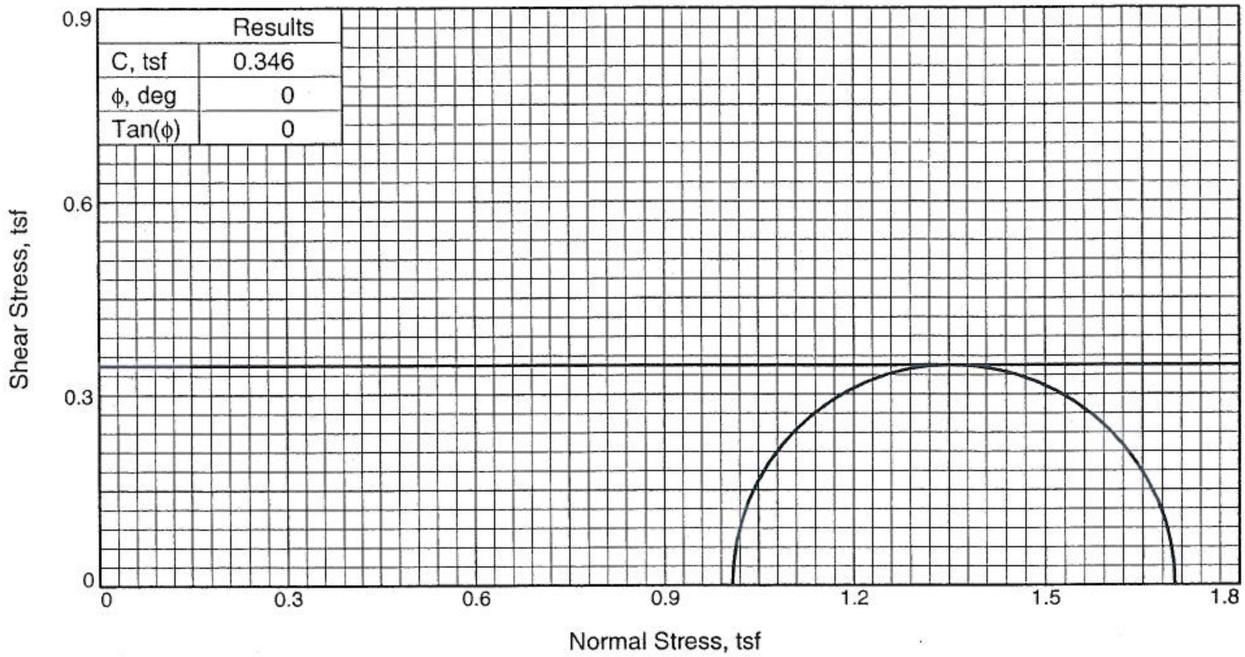
APPENDIX III

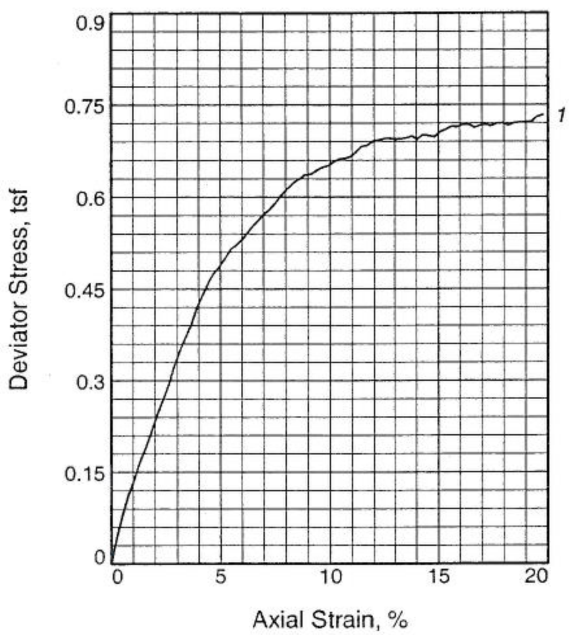
Summary of Strength and Consolidation Test Results
Strength and Consolidation Test Results

PROJECT SCI-823-0.00
US 23 Interchange
SUMMARY TEST RESULTS US 23 Ramp B

Boring	Sample	Depth (ft.)	Test Performed	Results											
				ODOT Classification	γ_D (pcf)	WC (%)	e_0	Cc	Cr	p_c (psf)	c (psf)	c' (psf)	φ (deg)	φ' (deg)	q_u (tsf)
B-1105A	ST2	8.0	UU	A-4b	97.0	27.5					692				
B-1105A	ST2	8.0	CIU	A-4b	98.7	24.7					0	0	22.4	34.9	
B-1105A	ST3	12.0	UU / Consol.	A-6a	100.0	24.1	0.694	0.240	0.060	5100	1396				
B-1105A	ST4	16.0	CIU	A-6b	91.6	27.6					182	0	20.4	36.8	
B-1105A	In-situ	10.0	FVS Test	A-4b/A-6a							1093*				
B-1105A	In-situ	11.6	FVS Test	A-6a							779*				
B-1105A	In-situ	13.7	FVS Test	A-6a							3933*				
B-1108	P1	10.0	UC / Consol.	A-6a / A-6b	103.4	22.4	0.639	0.170	0.030	3700					2.618
B-1108	P2	14.0	UU	A-6b	95.2	30.2					896				
B-1108	P3	18.0	CIU / Consol.	A-6b	95.8	28.4	0.734	0.210	0.050	2160	0	0	22.2	37.4	
B-1109A	ST2	8.0	UU / Consol.	A-7-6	97.9	26.2	0.883	0.150	0.040	3000	978				
B-1109A	In-situ	8.0	FVS Test	A-7-6							1687*				
B-45	P-1	5.0	UU	A-6b	103.5	22.1					1488				
B-45	P-1	5.0	Consol.	A-6b	105.2	20.0	0.632	0.090	0.010	600					
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	Consol.	A-6b	100.0	23.1	0.692	0.240	0.040	2700					
B-46	P-2	8.0	CIU	A-6b	108.2	18.9					256	0	23.8	35.8	
TR-61	6	13.5	Direct Shear	A-3a / A-1-b	101.8	28.7								42.1	

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





	Sample No.	1
Initial	Water Content,	27.5
	Dry Density, pcf	97.0
	Saturation,	98.2
	Void Ratio	0.7700
	Diameter, in.	2.84
	Height, in.	5.22
At Test	Water Content,	27.4
	Dry Density, pcf	97.0
	Saturation,	97.9
	Void Ratio	0.7700
	Diameter, in.	2.84
	Height, in.	5.22
	Strain rate, in./min.	0.06
	Back Pressure, tsf	0.00
	Cell Pressure, tsf	1.01
	Fail. Stress, tsf	0.69
	Ult. Stress, tsf	0.69
	σ_1 Failure, tsf	1.70
	σ_3 Failure, tsf	1.01

Type of Test:
Unconsolidated Undrained

Sample Type: Press Tube

Description:

Assumed Specific Gravity= 2.75

Remarks:

Figure _______

Client: TranSystems, Inc.

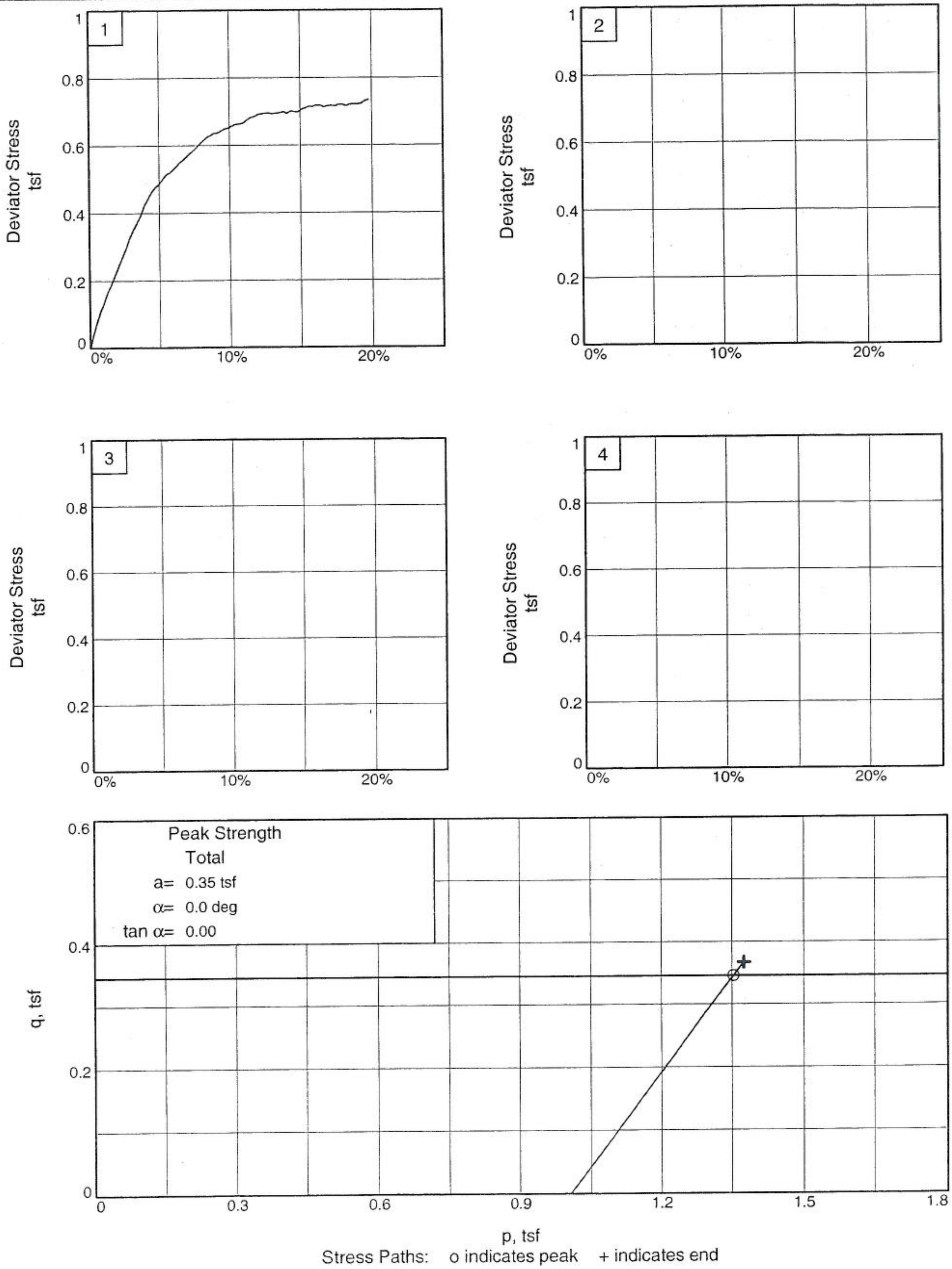
Project: SCI-823-0.00

Source of Sample: B-1105A **Depth:** 8.0

Sample Number: ST2

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

DLZ



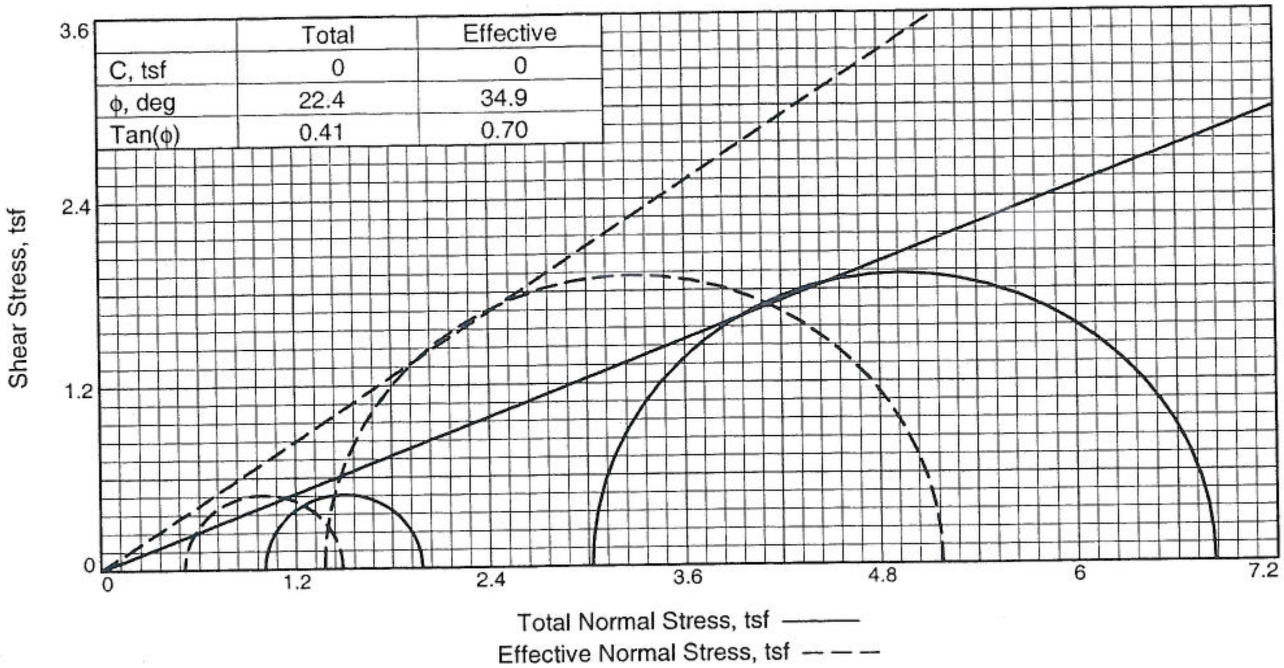
Client: TranSystems, Inc.

Project: SCI-823-0.00

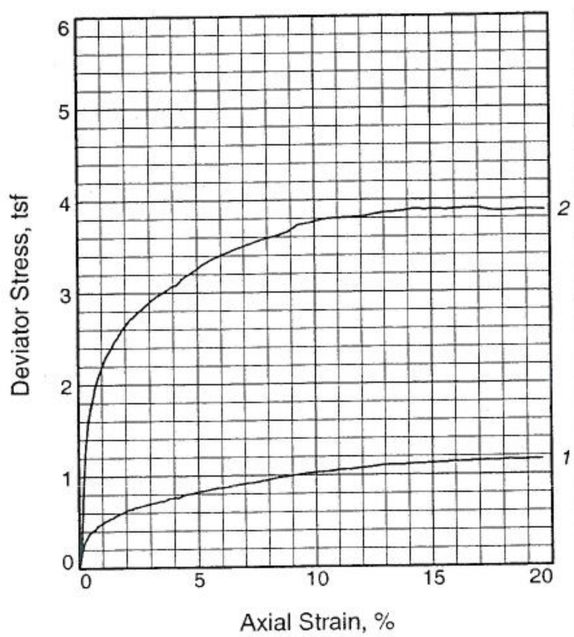
Source of Sample: B-1105A **Depth:** 8.0 **Sample Number:** ST2

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

DLZ, INC.



	Total	Effective
C, tsf	0	0
ϕ, deg	22.4	34.9
Tan(ϕ)	0.41	0.70



Sample No.		1	2
Initial	Water Content,	24.7	26.0
	Dry Density, pcf	98.7	97.8
	Saturation,	92.0	94.6
	Void Ratio	0.7397	0.7559
	Diameter, in.	2.84	2.84
	Height, in.	5.32	5.29
At Test	Water Content,	24.2	22.2
	Dry Density, pcf	101.0	105.0
	Saturation,	95.2	96.1
	Void Ratio	0.6992	0.6354
	Diameter, in.	2.82	2.76
	Height, in.	5.27	5.21
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		0.98	3.80
Total Pore Pr., tsf		3.81	4.96
Ult. Stress, tsf		0.98	3.80
Total Pore Pr., tsf		3.81	4.96
$\bar{\sigma}_1$ Failure, tsf		1.49	5.18
$\bar{\sigma}_3$ Failure, tsf		0.51	1.38

Type of Test:
CU with Pore Pressures

Sample Type: Press Tube

Description: Lean clay

LL= 28 **PL=** 18 **PI=** 10

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

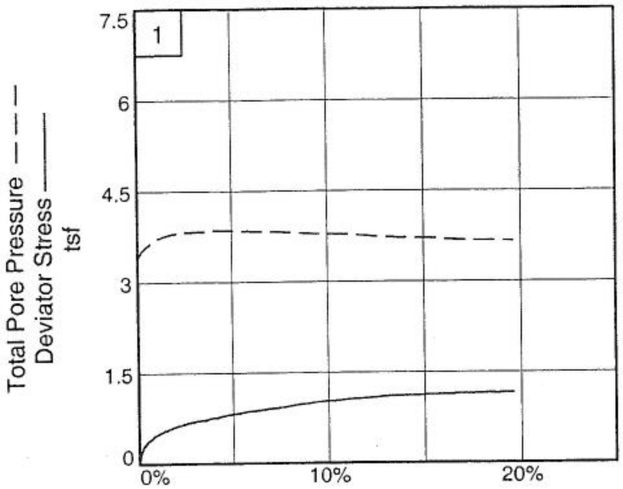
Project: SCI-823-0.00

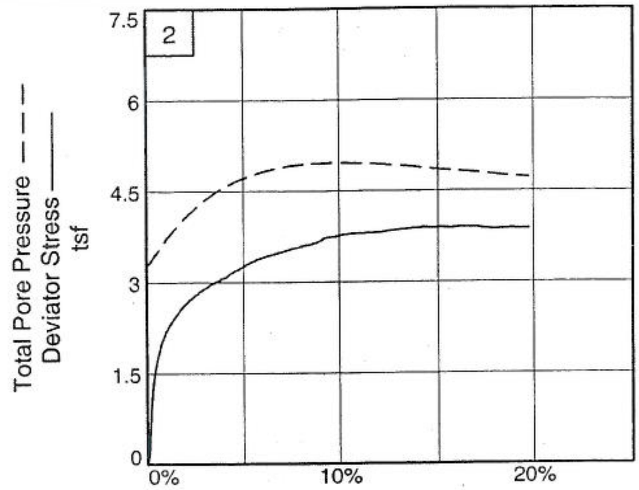
Source of Sample: B-1105A **Depth:** 8.0

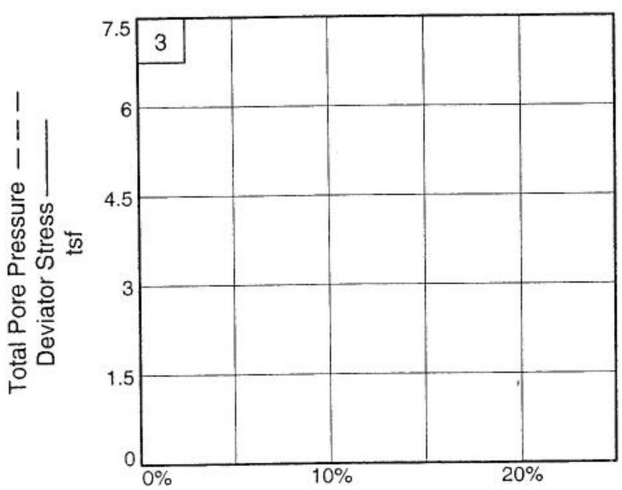
Sample Number: ST2

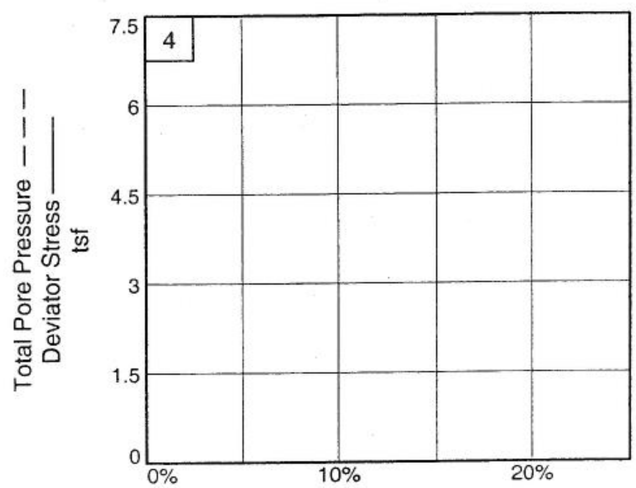
Proj. No.: 0121-3070.03 **Date:** 8/24/07

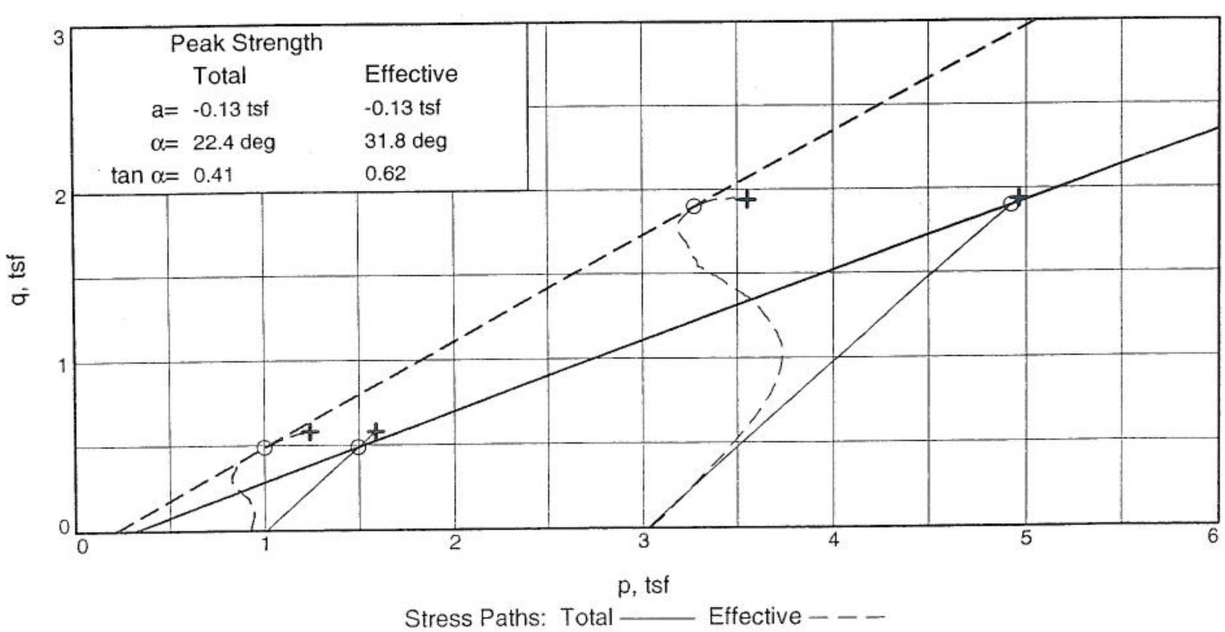
DLZ











Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1105A **Depth:** 8.0 **Sample Number:** ST2
Project No.: 0121-3070.03 **Figure** ______

DLZ, INC.

Vane Shear Test Report

Project	SCI-823-0.00	Date and Time	7/27/2007	Begin 10:50am End 11:30am
Project No.	0121-3070-03	Boring Number	B-1105a	Depth 10.0' First
Client	ODOT			
Drill Rig & Crew	D Wamsley			
Tested By	B Mott			
Weather / Temp.	overcast 75			
Soil Type				

DRILLING

Hollowstem augers to depth	D_a	9
Vane Depth below bottom of augers	D_s	1
Augers above ground surface	H_a	6
Depth to vane tip	D_t	10

SHEAR VANE

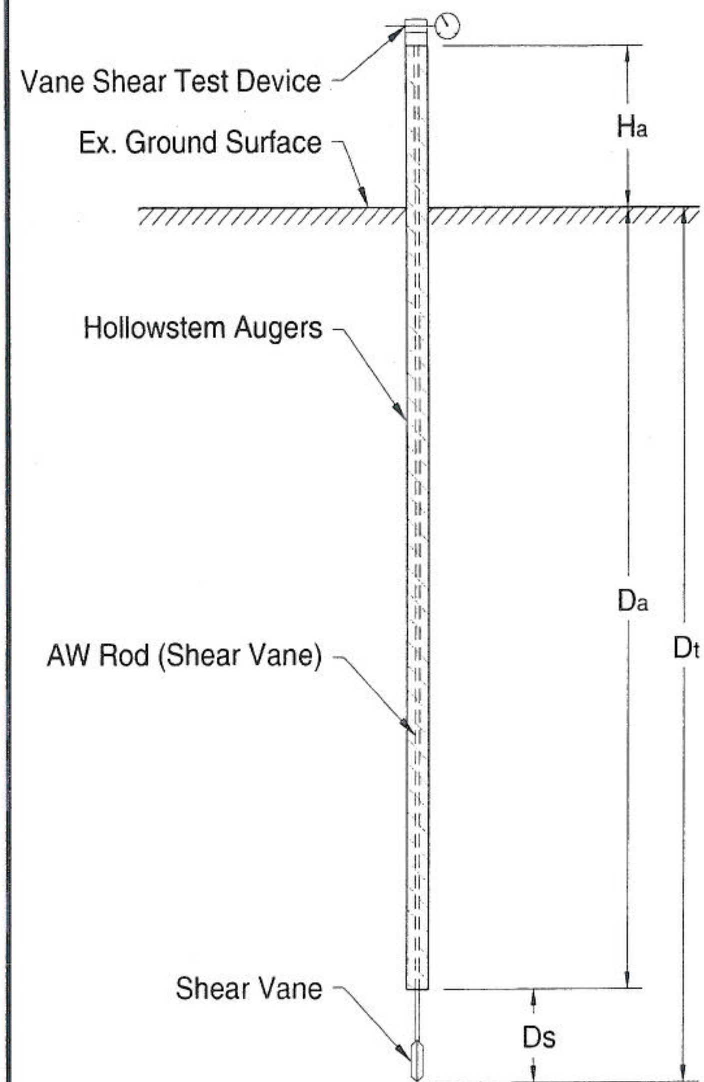
Vane Used	**2.0"**	2.5"	3.625"
Vane constant, k (lb-in to psf)	**5.17**	2.59	0.905

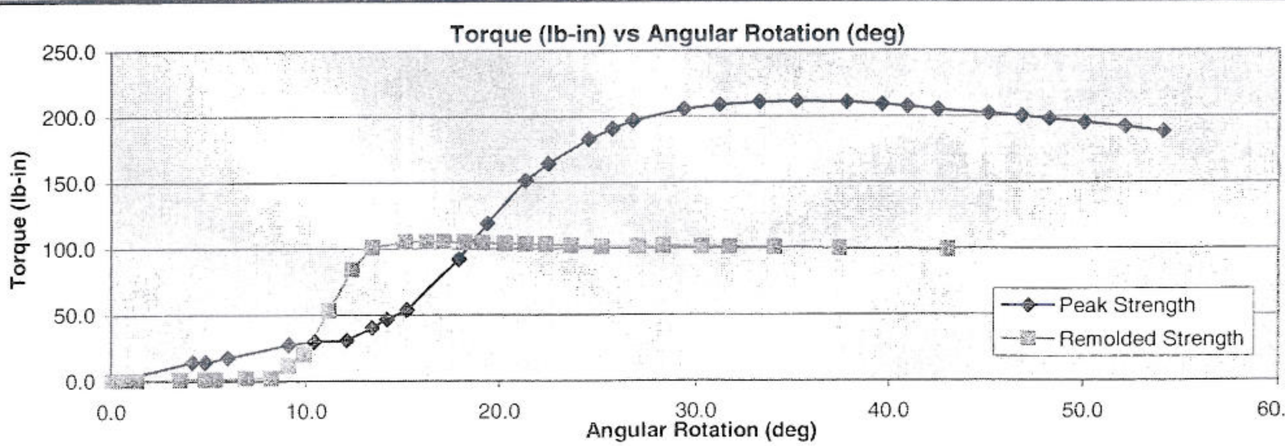
Masurement by: Automatic/torque cell

Max Torque: 211 lb-in

Max UD Shear Strength 1093 psf

Test Setup





DLZ

DLZ Ohio, Inc.
ENGINEERS · ARCHITECTS · SCIENTISTS
PLANNERS · SURVEYORS

Vane Shear Test Report

Peak Strength Test

Read Time	Δ time	Rotation *(degrees)*	Torque *(in-lbs.)*
11:04:12	0:00:00	0.0	0.0
11:04:44	0:00:32	4.2	14.5
11:04:49	0:00:37	4.8	14.5
11:04:58	0:00:46	6.0	17.6
11:05:22	0:01:10	9.1	27.3
11:05:32	0:01:20	10.4	29.7
11:05:45	0:01:33	12.1	30.7
11:05:55	0:01:43	13.4	40.4
11:06:01	0:01:49	14.2	46.8
11:06:09	0:01:57	15.2	54.0
11:06:30	0:02:18	17.9	92.4
11:06:41	0:02:29	19.4	119.2
11:06:56	0:02:44	21.3	151.4
11:07:05	0:02:53	22.5	164.2
11:07:21	0:03:09	24.6	182.6
11:07:30	0:03:18	25.7	190.7
11:07:38	0:03:26	26.8	196.8
11:07:58	0:03:46	29.4	205.4
11:08:12	0:04:00	31.2	208.6
11:08:28	0:04:16	33.3	210.8
11:08:43	0:04:31	35.2	211.5
11:09:03	0:04:51	37.8	211.0
11:09:17	0:05:05	39.7	209.2
11:09:27	0:05:15	41.0	207.6
11:09:39	0:05:27	42.5	205.0
11:09:59	0:05:47	45.1	202.4
11:10:12	0:06:00	46.8	200.2
11:10:23	0:06:11	48.2	198.0
11:10:37	0:06:25	50.0	195.3
11:10:53	0:06:41	52.1	192.0
11:11:08	0:06:56	54.1	188.6

Remolded Strength Test

Read Time	Δ time	Rotation *(degrees)*	Torque *(in-lbs.)*
11:20:02	0:00:00	0.0	0.0
11:20:07	0:00:05	0.6	0.1
11:20:12	0:00:10	1.3	0.5
11:20:29	0:00:27	3.5	1.1
11:20:39	0:00:37	4.8	1.5
11:20:43	0:00:41	5.3	1.4
11:20:55	0:00:53	6.9	2.5
11:21:05	0:01:03	8.2	2.2
11:21:12	0:01:10	9.1	11.7
11:21:18	0:01:16	9.9	20.4
11:21:28	0:01:26	11.2	52.9
11:21:37	0:01:35	12.3	84.4
11:21:45	0:01:43	13.4	101.1
11:21:58	0:01:56	15.1	105.4
11:22:07	0:02:05	16.2	105.4
11:22:14	0:02:12	17.2	105.4
11:22:22	0:02:20	18.2	105.0
11:22:29	0:02:27	19.1	104.4
11:22:38	0:02:36	20.3	103.9
11:22:46	0:02:44	21.3	103.4
11:22:54	0:02:52	22.4	103.2
11:23:04	0:03:02	23.7	102.2
11:23:16	0:03:14	25.2	101.4
11:23:30	0:03:28	27.0	101.6
11:23:40	0:03:38	28.3	102.0
11:23:55	0:03:53	30.3	101.4
11:24:06	0:04:04	31.7	101.2
11:24:24	0:04:22	34.1	101.1
11:24:50	0:04:48	37.4	100.1
11:25:33	0:05:31	43.0	99.2

Peak Torque	*211.5*	(lb-in)
Vane Constant	*5.17*	
Peak Shear Strength	*1093*	*psf*

Remolded Torque	*105.4*	(lb-in)
Vane Constant	*5.17*	
Remolded Shear Strength	*545*	*psf*
Sensitivity	*2.0*	

Vane Shear Test Report

Project	SCI-823-0.00	Date and Time	7/27/2007	Begin 1:50pm End 2:25pm
Project No.	0121-3070-03	Boring Number	B-1105A	Depth 11.6'
Client	ODOT			
Drill Rig & Crew	D Wamsley			
Tested By	B Mott			
Weather / Temp.	sunny 90			
Soil Type				

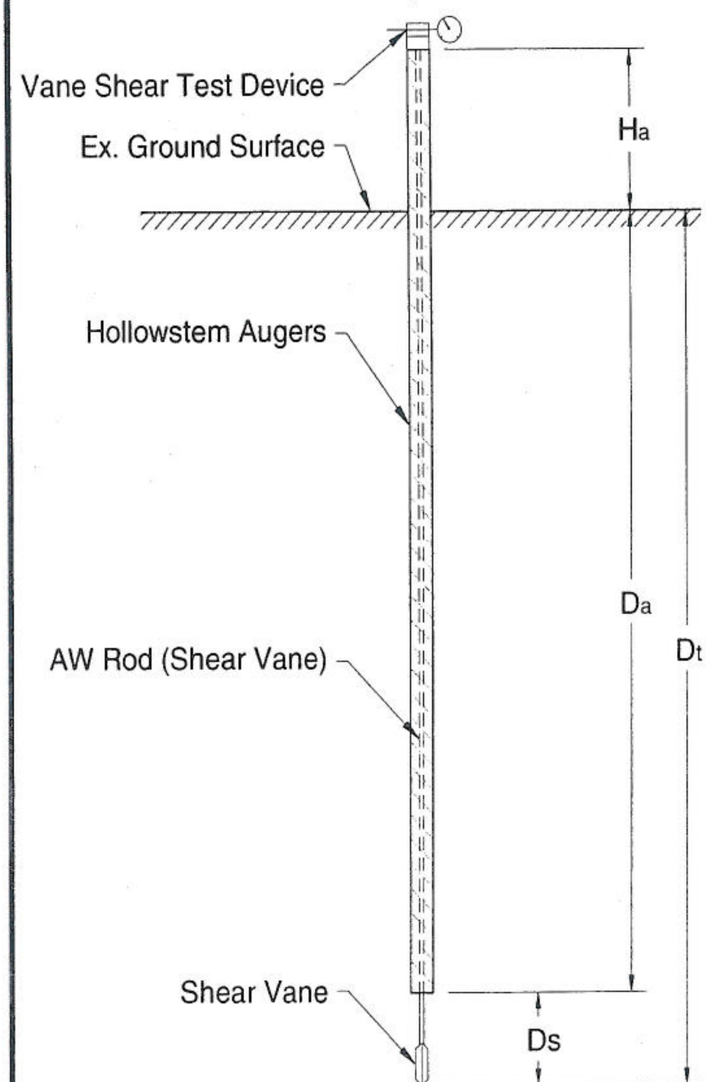
DRILLING

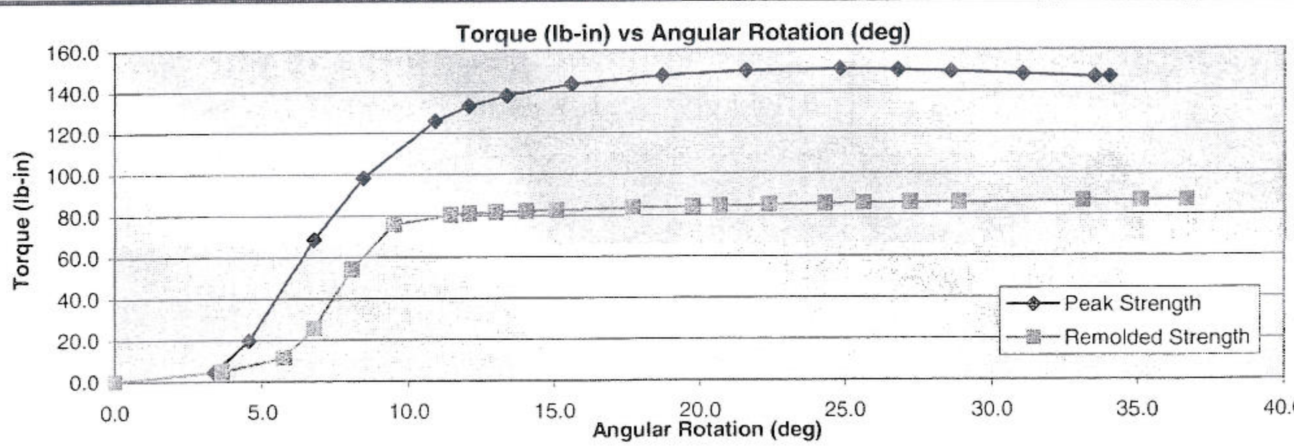
Hollowstem augers to depth	D_a	10.8
Vane Depth below bottom of augers	D_s	0.8
Augers above ground surface	H_a	7
Depth to vane tip	D_t	11.6

SHEAR VANE

Vane Used	**2.0"**	2.5"	3.625"
Vane constant, k (lb-in to psf)	**5.17**	2.59	0.905
Masurement by	Automatic/torque cell		
Max Torque	151	lb-in	
Max UD Shear Strength	**779**	**psf**	

Test Setup





DLZ

DLZ Ohio, Inc.
ENGINEERS · ARCHITECTS · SCIENTISTS
PLANNERS · SURVEYORS

Vane Shear Test Report

Peak Strength Test

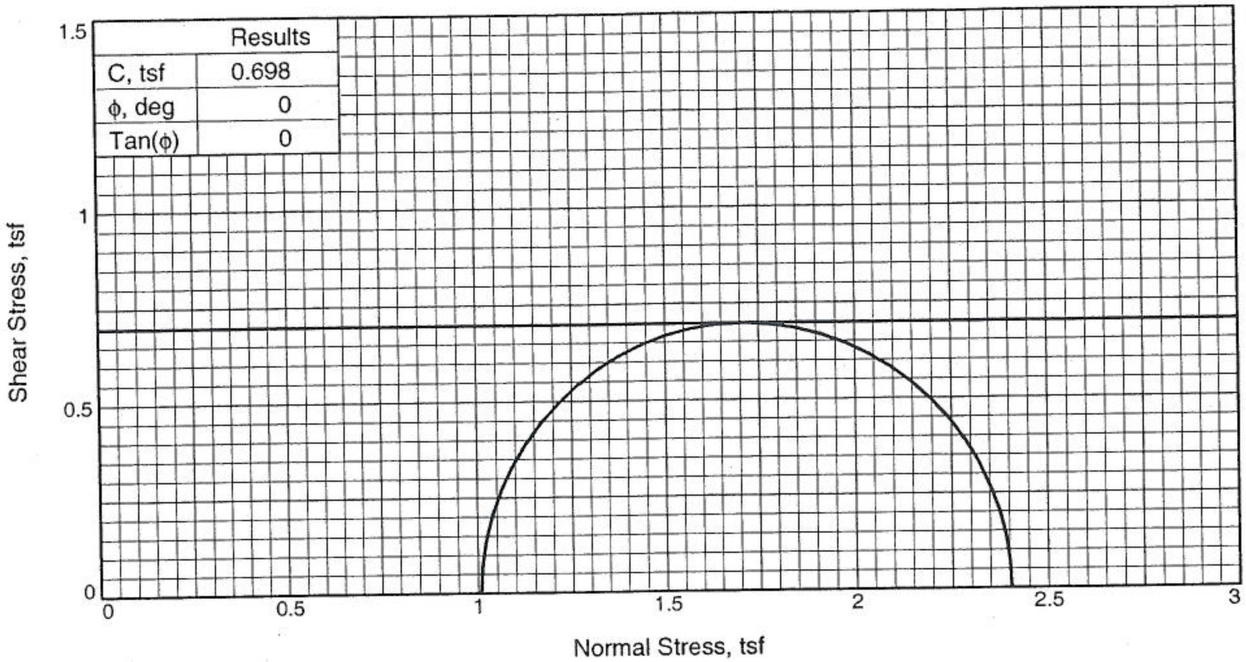
Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
14:03:57	0:00:00	0.0	0.0
14:04:23	0:00:26	3.4	4.6
14:04:32	0:00:35	4.5	19.7
14:04:49	0:00:52	6.8	68.6
14:05:02	0:01:05	8.5	98.3
14:05:21	0:01:24	10.9	125.9
14:05:30	0:01:33	12.1	133.1
14:05:40	0:01:43	13.4	138.1
14:05:57	0:02:00	15.6	143.7
14:06:21	0:02:24	18.7	147.7
14:06:43	0:02:46	21.6	149.9
14:07:08	0:03:11	24.8	150.7
14:07:23	0:03:26	26.8	150.1
14:07:37	0:03:40	28.6	149.4
14:07:56	0:03:59	31.1	148.0
14:08:15	0:04:18	33.5	146.6
14:08:19	0:04:22	34.1	146.6

Peak Torque	150.7	(lb-in)
Vane Constant	5.17	
Peak Shear Strength	*779*	*psf*

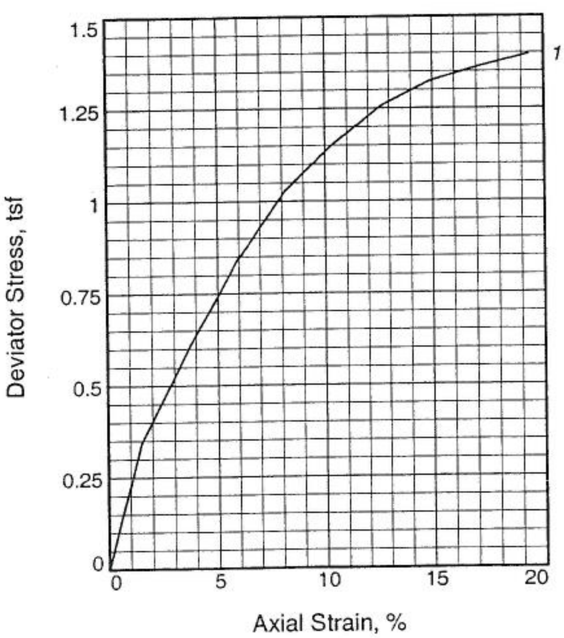
Remolded Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
14:14:10	0:00:00	0.0	0.0
14:14:38	0:00:28	3.6	5.0
14:14:54	0:00:44	5.7	11.5
14:15:02	0:00:52	6.8	25.7
14:15:12	0:01:02	8.1	54.1
14:15:23	0:01:13	9.5	75.8
14:15:38	0:01:28	11.4	80.2
14:15:43	0:01:33	12.1	80.9
14:15:50	0:01:40	13.0	81.4
14:15:58	0:01:48	14.0	82.1
14:16:06	0:01:56	15.1	82.5
14:16:26	0:02:16	17.7	83.6
14:16:42	0:02:32	19.8	83.9
14:16:49	0:02:39	20.7	84.4
14:17:02	0:02:52	22.4	84.7
14:17:17	0:03:07	24.3	85.2
14:17:27	0:03:17	25.6	85.5
14:17:39	0:03:29	27.2	85.8
14:17:52	0:03:42	28.9	85.8
14:18:25	0:04:15	33.2	86.5
14:18:40	0:04:30	35.1	86.7
14:18:52	0:04:42	36.7	86.8

Remolded Torque	86.8	(lb-in)
Vane Constant	5.17	
Remolded Shear Strength	*449*	*psf*
Sensitivity	*1.7*	



Results	
C, tsf	0.698
ϕ, deg	0
Tan(ϕ)	0



Sample No.		1
Initial	Water Content,	24.1
	Dry Density, pcf	100.0
	Saturation,	92.4
	Void Ratio	0.7176
	Diameter, in.	2.82
	Height, in.	5.54
At Test	Water Content,	24.1
	Dry Density, pcf	100.0
	Saturation,	92.4
	Void Ratio	0.7176
	Diameter, in.	2.82
	Height, in.	5.54
Strain rate, in./min.		0.06
Back Pressure, tsf		0.00
Cell Pressure, tsf		1.01
Fail. Stress, tsf		1.40
Ult. Stress, tsf		
σ_1 Failure, tsf		2.40
σ_3 Failure, tsf		1.01

Type of Test:
Unconsolidated Undrained
Sample Type: Press Tube
Description:

LL= 30 **PL=** 19 **PI=** 11
Assumed Specific Gravity= 2.75
Remarks:

Figure ______

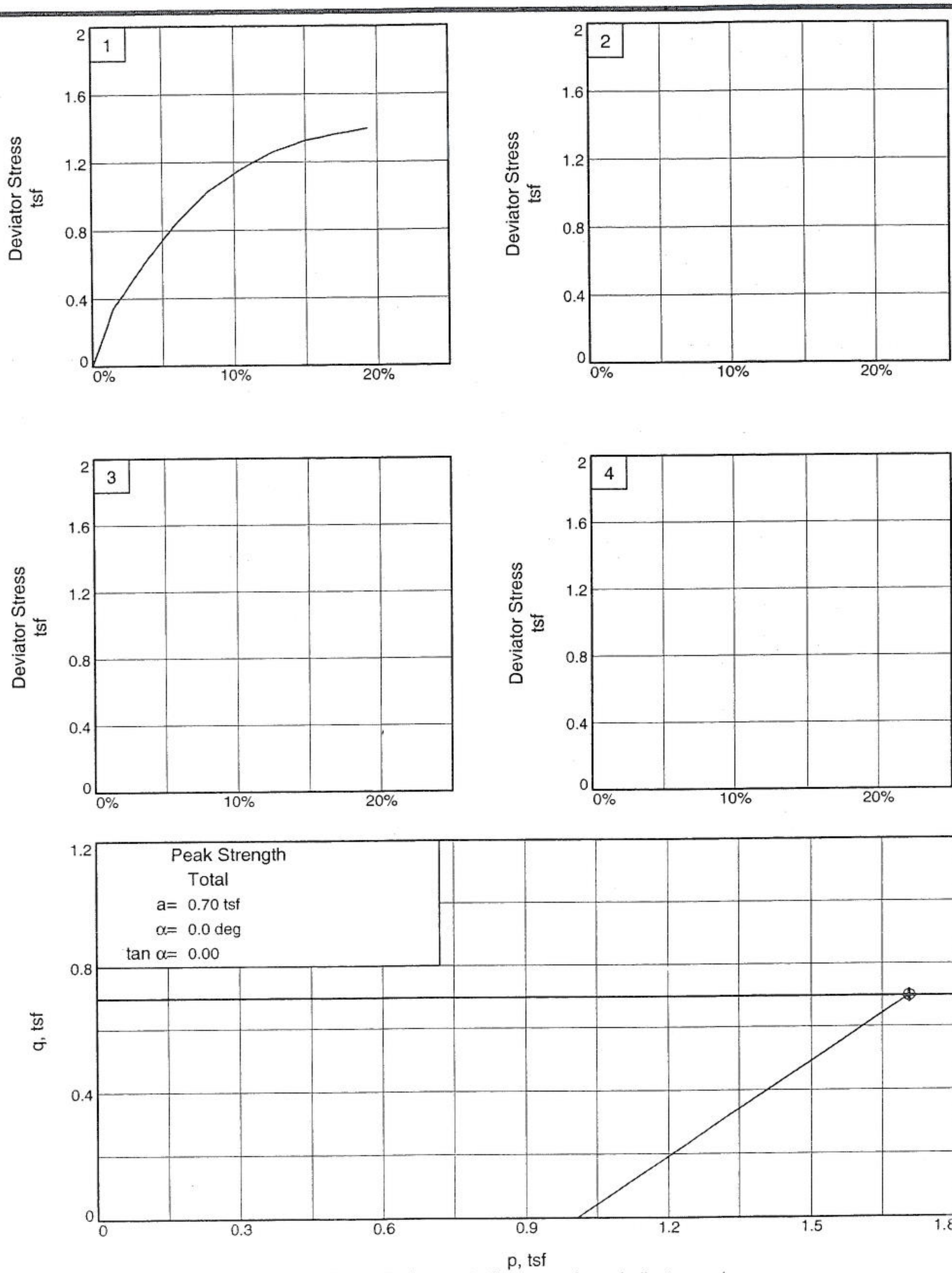
Client: TranSystems, Inc.

Project: SCI-823-0.00

Source of Sample: B-1105A **Depth:** 12.0
Sample Number: ST3

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

DLZ



Stress Paths: o indicates peak + indicates end

Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1105A
Project No.: 0121-3070.03

Depth: 12.0
Figure ________

Sample Number: ST3

DLZ, INC.

Vane Shear Test Report

Project	SCI-823-0.00	Date and Time	7/27/2007	Begin 2:30pm End 3:00pm
Project No.	0121-3070-03	Boring Number	B-1105A	Depth 13.7'
Client	ODOT			
Drill Rig & Crew	D Wamsley			
Tested By	B Mott			
Weather / Temp.	sunny 90			
Soil Type				

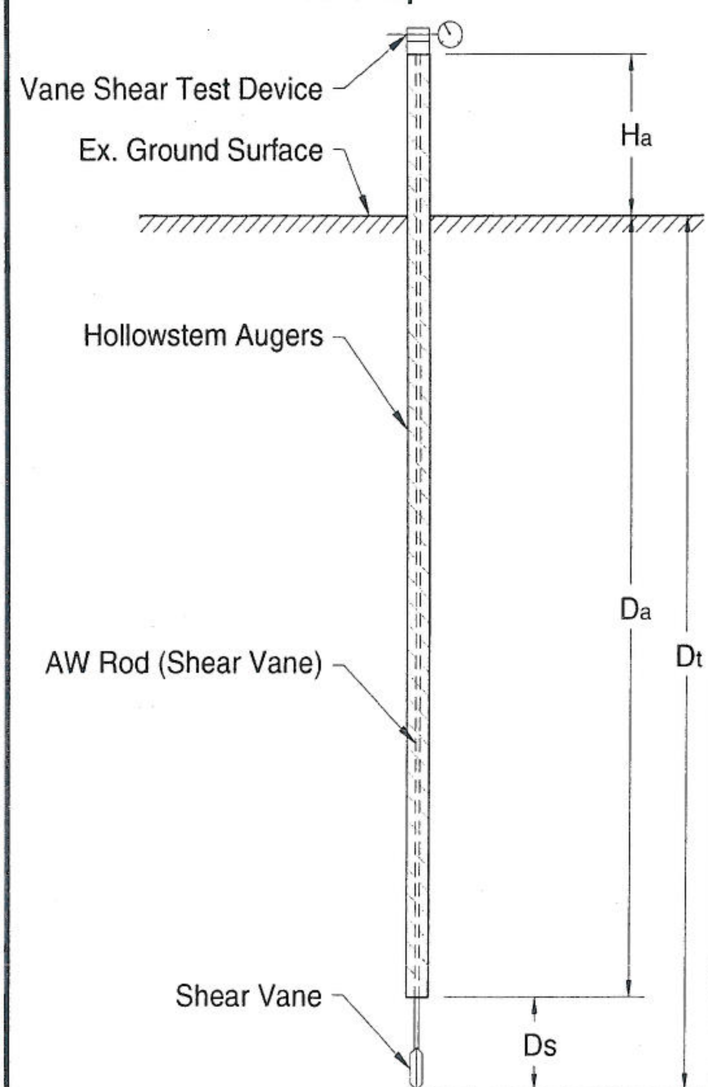
DRILLING

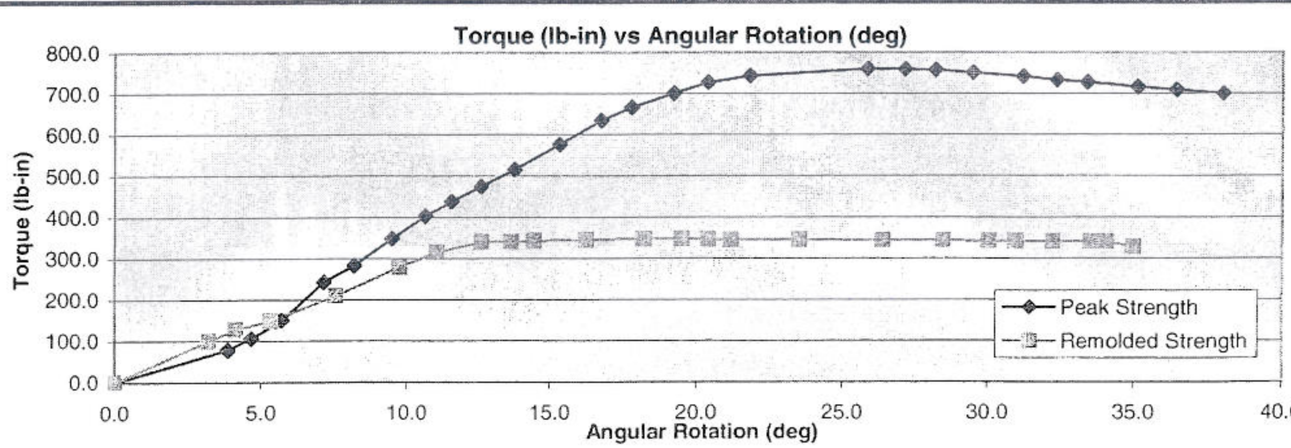
Hollowstem augers to depth	D_a	12.9
Vane Depth below bottom of augers	D_s	0.8
Augers above ground surface	H_a	7
Depth to vane tip	D_t	13.7

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		761	lb-in
Max UD Shear Strength		**3933**	**psf**

Test Setup





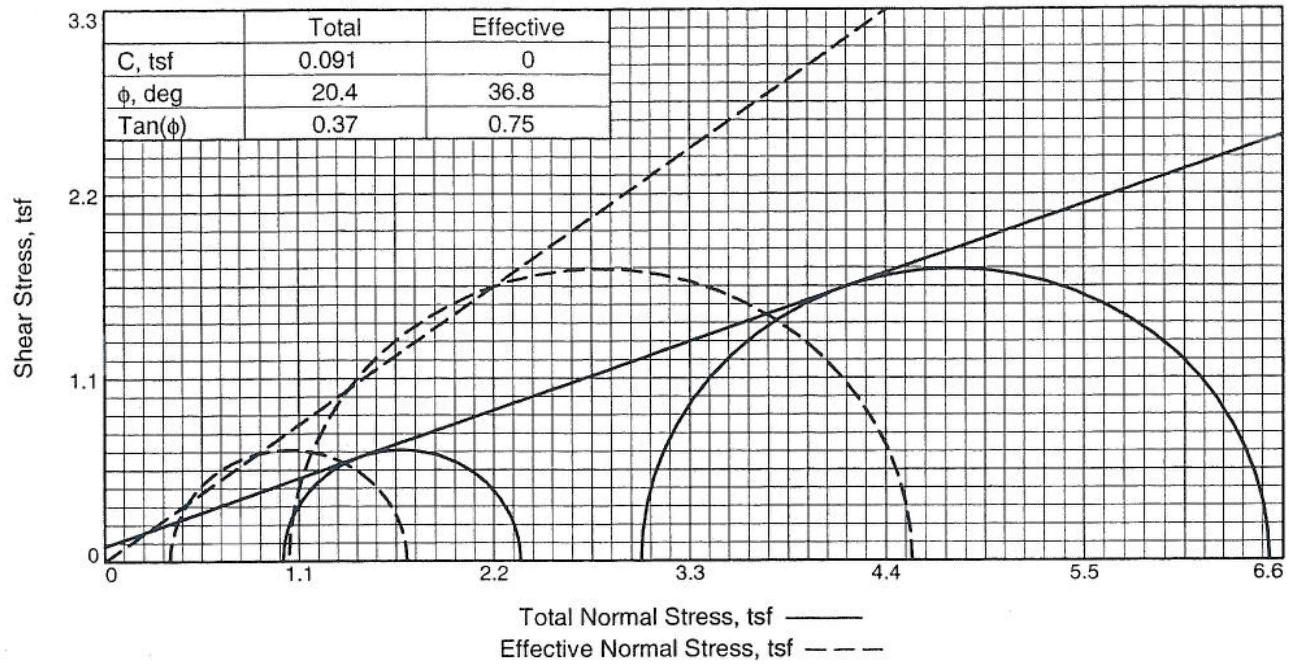
Vane Shear Test Report

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
Peak Strength Test			
14:34:18	0:00:00	*0.0*	0.0
14:34:48	0:00:30	*3.9*	78.2
14:34:54	0:00:36	*4.7*	106.8
14:35:02	0:00:44	*5.7*	151.1
14:35:13	0:00:55	*7.2*	243.3
14:35:21	0:01:03	*8.2*	282.7
14:35:31	0:01:13	*9.5*	348.3
14:35:40	0:01:22	*10.7*	402.7
14:35:47	0:01:29	*11.6*	437.9
14:35:55	0:01:37	*12.6*	474.1
14:36:04	0:01:46	*13.8*	515.1
14:36:16	0:01:58	*15.3*	576.4
14:36:27	0:02:09	*16.8*	634.8
14:36:35	0:02:17	*17.8*	666.4
14:36:46	0:02:28	*19.2*	700.1
14:36:55	0:02:37	*20.4*	727.2
14:37:06	0:02:48	*21.8*	743.2
14:37:37	0:03:19	*25.9*	760.7
14:37:47	0:03:29	*27.2*	759.3
14:37:55	0:03:37	*28.2*	757.0
14:38:05	0:03:47	*29.5*	750.8
14:38:18	0:04:00	*31.2*	741.6
14:38:27	0:04:09	*32.4*	733.3
14:38:35	0:04:17	*33.4*	727.3
14:38:48	0:04:30	*35.1*	716.1
14:38:58	0:04:40	*36.4*	708.7
14:39:10	0:04:52	*38.0*	699.8

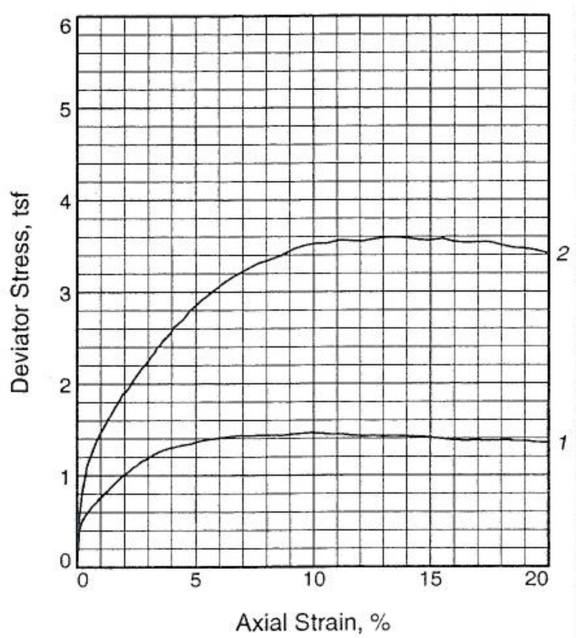
Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
Remolded Strength Test			
14:46:36	0:00:00	0.0	0.0
14:47:01	0:00:25	3.3	100.6
14:47:08	0:00:32	4.2	128.0
14:47:17	0:00:41	5.3	149.2
14:47:34	0:00:58	7.5	209.5
14:47:51	0:01:15	9.8	279.2
14:48:01	0:01:25	11.1	315.0
14:48:13	0:01:37	12.6	339.6
14:48:21	0:01:45	13.7	340.0
14:48:27	0:01:51	14.4	342.0
14:48:41	0:02:05	16.3	345.0
14:48:56	0:02:20	18.2	346.9
14:49:06	0:02:30	19.5	347.5
14:49:13	0:02:37	20.4	346.2
14:49:19	0:02:43	21.2	345.2
14:49:37	0:03:01	23.5	344.9
14:49:59	0:03:23	26.4	344.0
14:50:15	0:03:39	28.5	343.6
14:50:27	0:03:51	30.0	342.4
14:50:34	0:03:58	30.9	342.3
14:50:44	0:04:08	32.2	341.5
14:50:54	0:04:18	33.5	340.9
14:50:58	0:04:22	34.1	340.5
14:51:05	0:04:29	35.0	327.4

Peak Torque	*760.7*	(lb-in)
Vane Constant	*5.17*	
Peak Shear Strength	*3933*	*psf*

Remolded Torque	*347.5*	(lb-in)
Vane Constant	*5.17*	
Remolded Shear Strength	*1796*	*psf*
Sensitivity	*2.2*	



	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
$\overline{\sigma}_1$ Failure, tsf		1.71	4.55
$\overline{\sigma}_3$ 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.

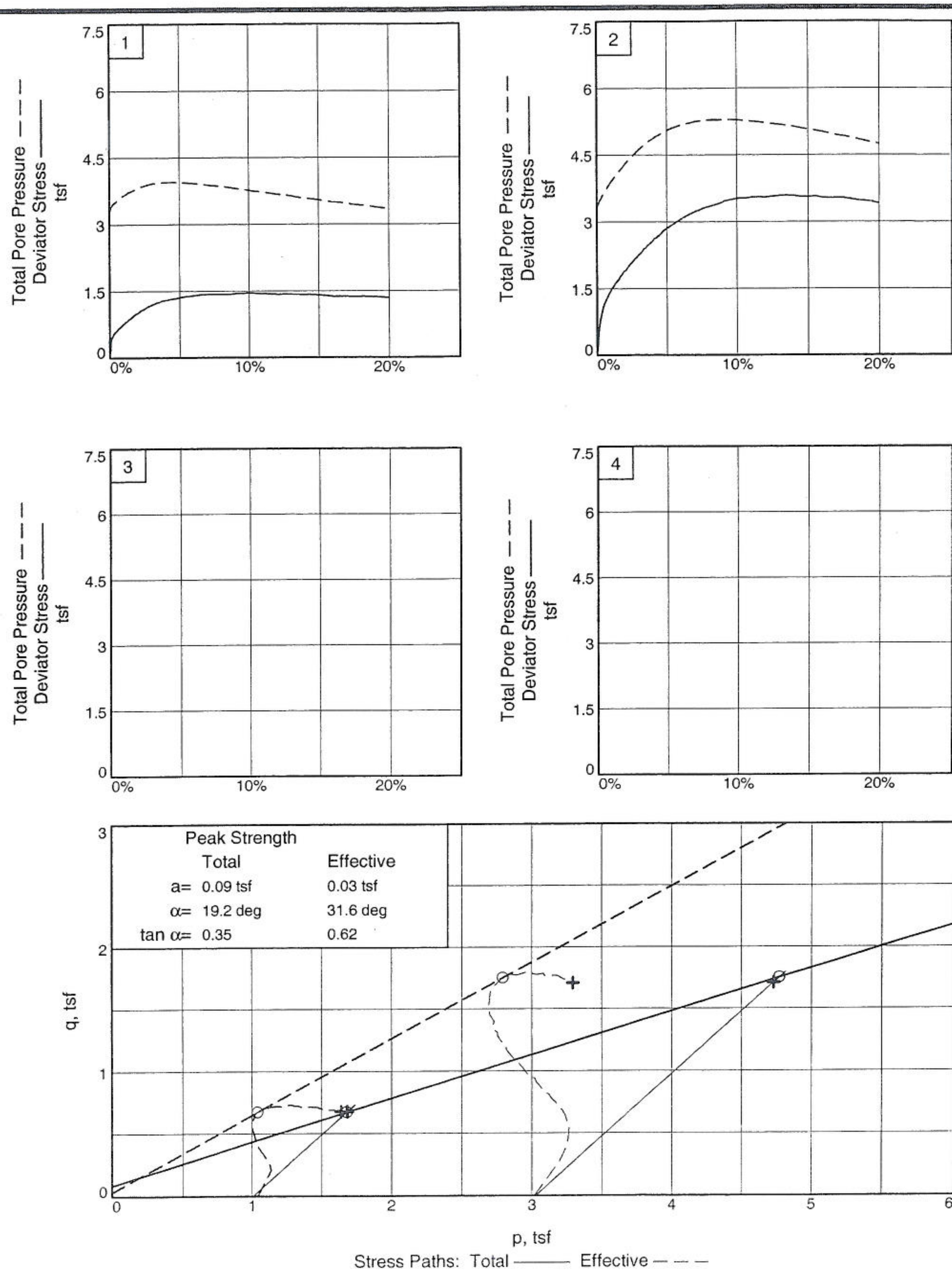
Project: SCI-823-0.00

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

Sample Number: ST4

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

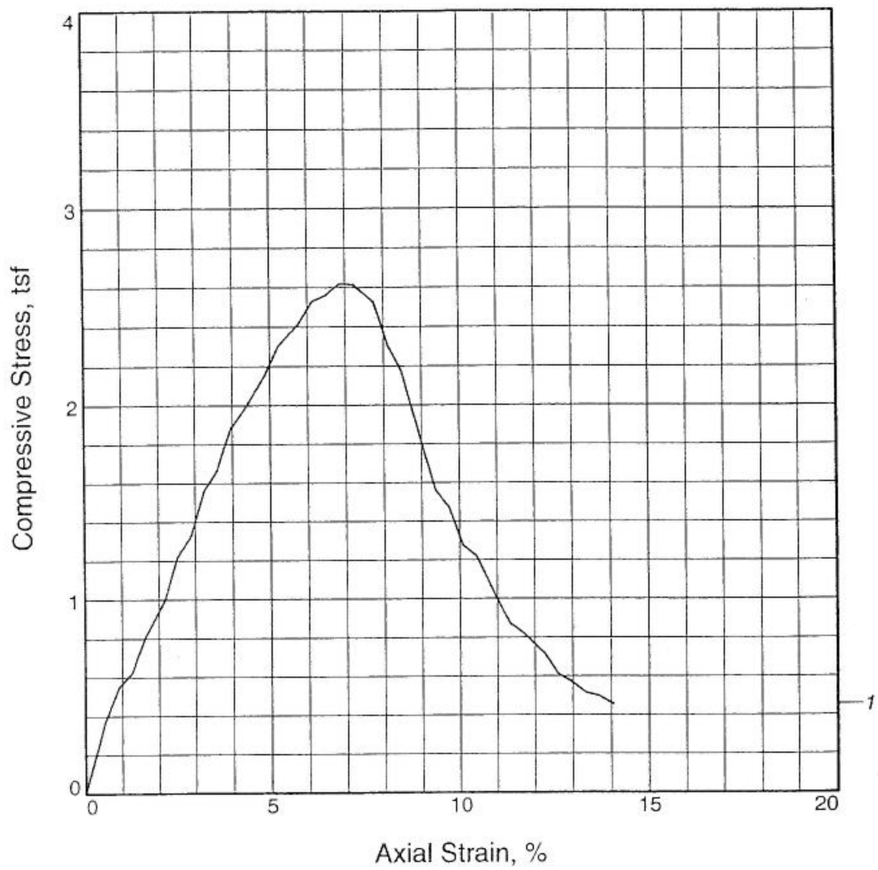
DLZ



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.

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	2.618			
Undrained shear strength, tsf	1.309			
Failure strain,	6.8			
Strain rate, in./min.	0.06			
Water content, %	22.4			
Wet density, pcf	126.5			
Dry density, pcf	103.4			
Saturation, %	93.1			
Void ratio	0.6602			
Specimen diameter, in.	2.83			
Specimen height, in.	5.55			
Height/diameter ratio	1.96			

Description: Moisture Content = 22.4%

LL = 36	**PL =** 21	**PI =** 15	**Assumed GS=** 2.75	**Type:** 3" Press Tubes

Project No.: 0121-3070.03

Date: 08/16/06

Remarks:

Client: TranSystems, Inc.

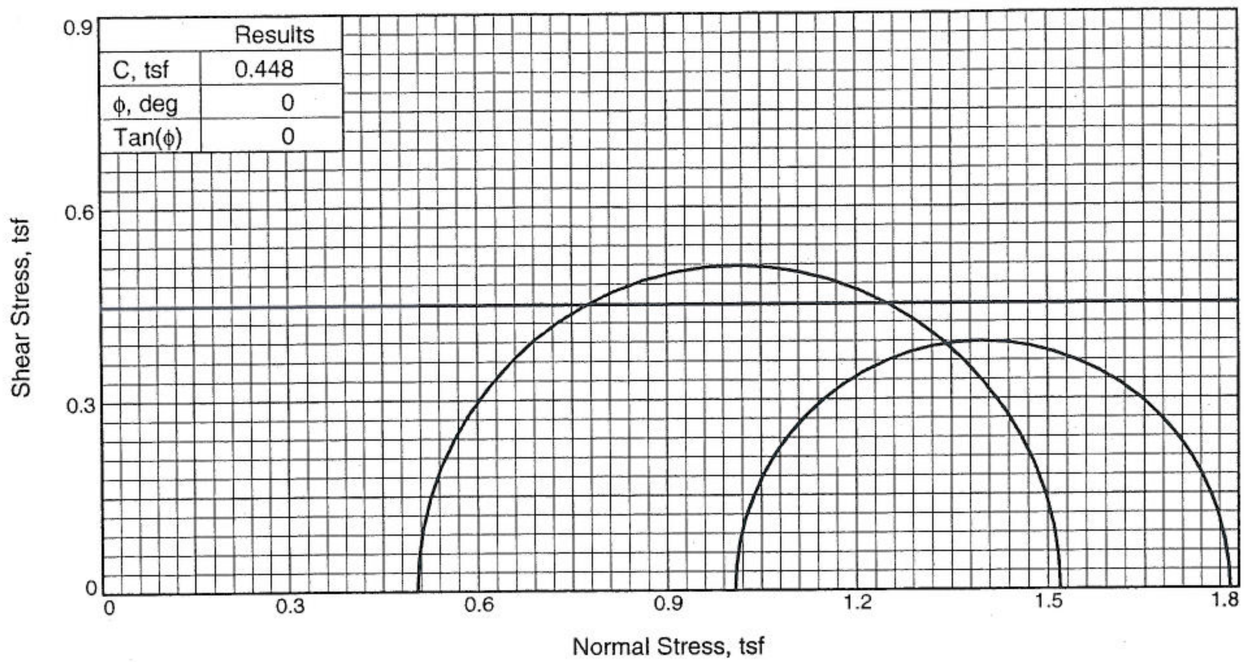
Project: SCI-823-0.00

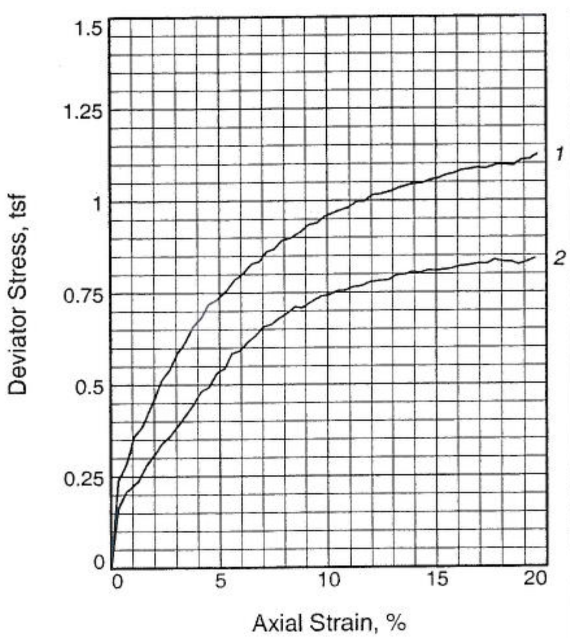
Source of Sample: B-1108 **Depth:** 10.0

Sample Number: P1

DLZ

Figure _______





Sample No.		1	2
Initial	Water Content,	30.2	32.6
	Dry Density, pcf	95.2	89.5
	Saturation,	103.3	97.8
	Void Ratio	0.8041	0.9172
	Diameter, in.	2.83	2.84
	Height, in.	5.56	5.54
At Test	Water Content,	27.0	31.8
	Dry Density, pcf	95.2	89.5
	Saturation,	92.2	95.2
	Void Ratio	0.8041	0.9172
	Diameter, in.	2.83	2.84
	Height, in.	5.56	5.54
Strain rate, in./min.		0.06	0.06
Back Pressure, tsf		0.00	0.00
Cell Pressure, tsf		0.50	1.01
Fail. Stress, tsf		1.02	0.78
Ult. Stress, tsf		1.02	0.78
σ_1 Failure, tsf		1.52	1.79
σ_3 Failure, tsf		0.50	1.01

Type of Test:
Unconsolidated Undrained

Sample Type: 3" Press Tube

Description: Lean clay with sand

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

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

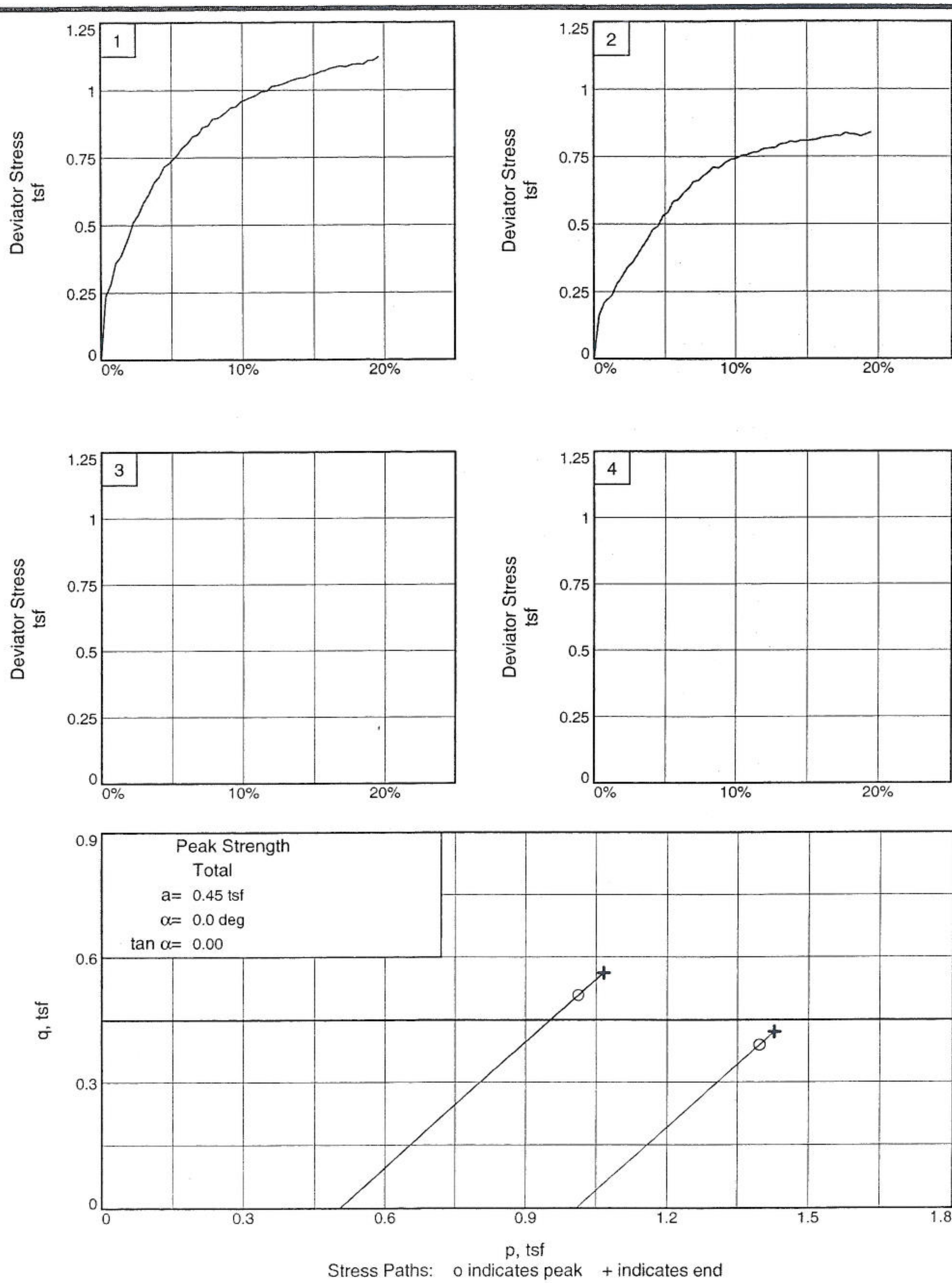
Project: SCI-823-0.00

Source of Sample: B-1108 **Depth:** 14.0

Sample Number: P2

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

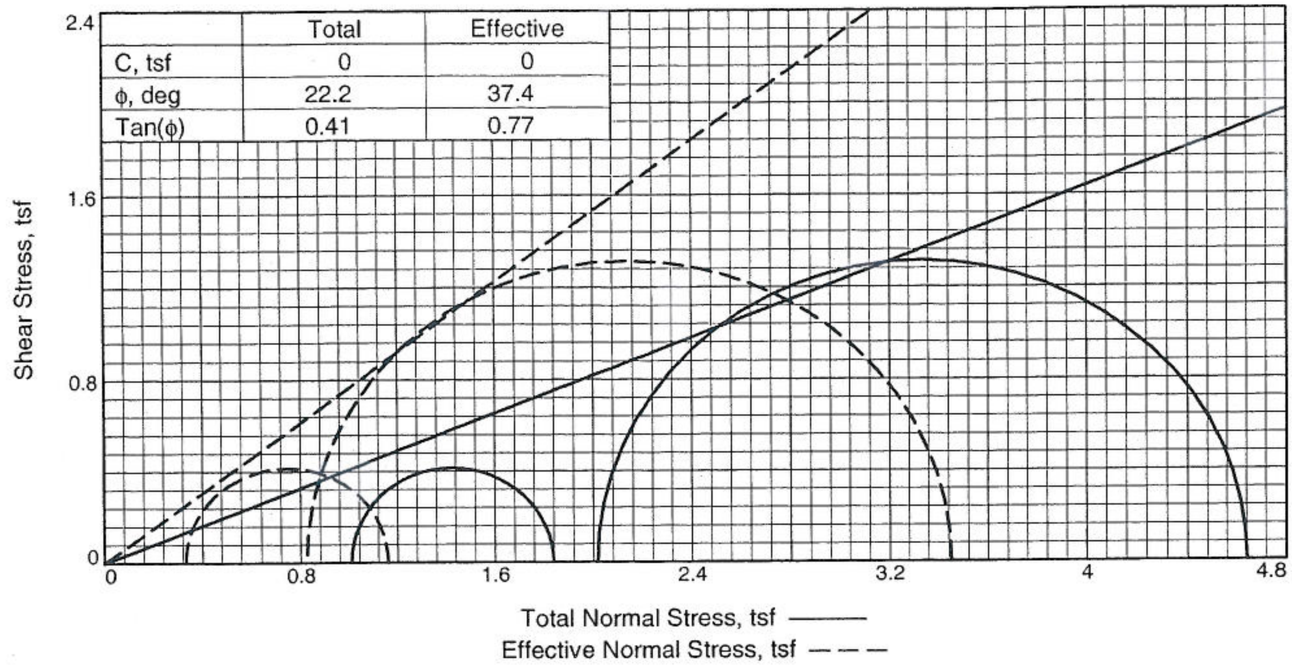
DLZ

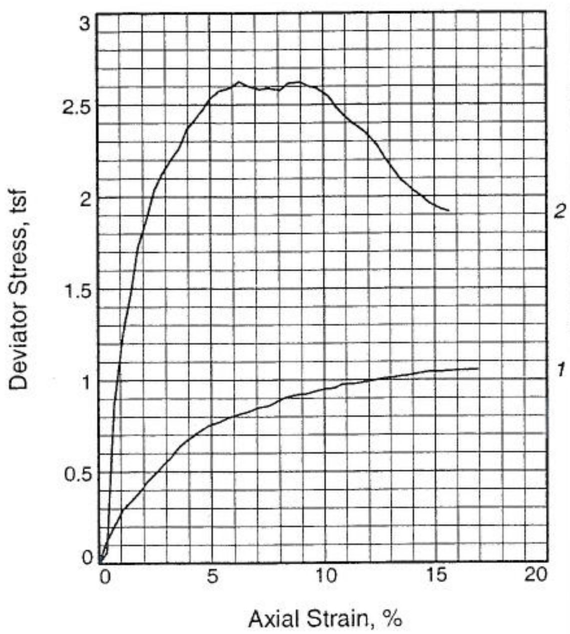


Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1108 **Depth:** 14.0 **Sample Number:** P2
Project No.: 0121-3070.03 **Figure** _______

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.

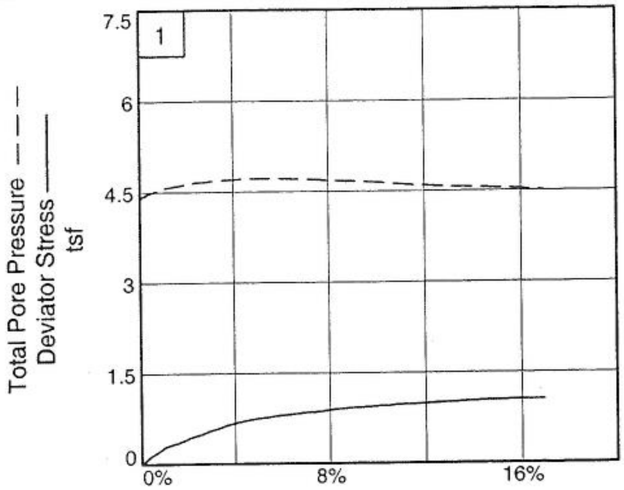
Project: SCI-823-0.00

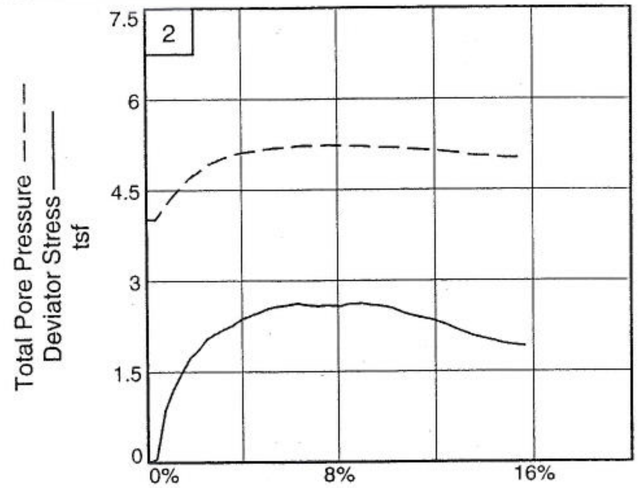
Source of Sample: B-1108 **Depth:** 18.0

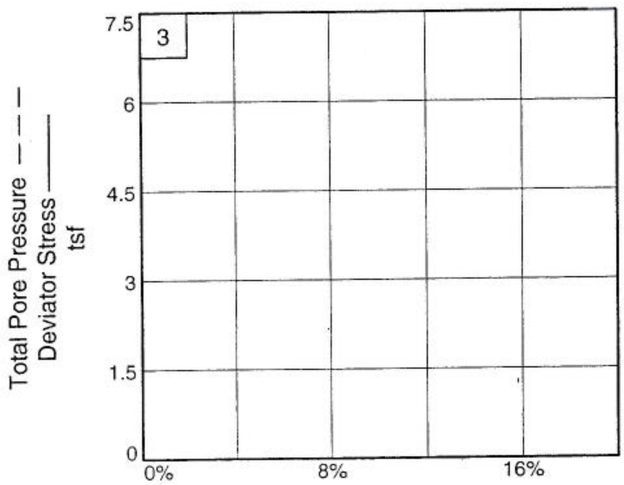
Sample Number: P3

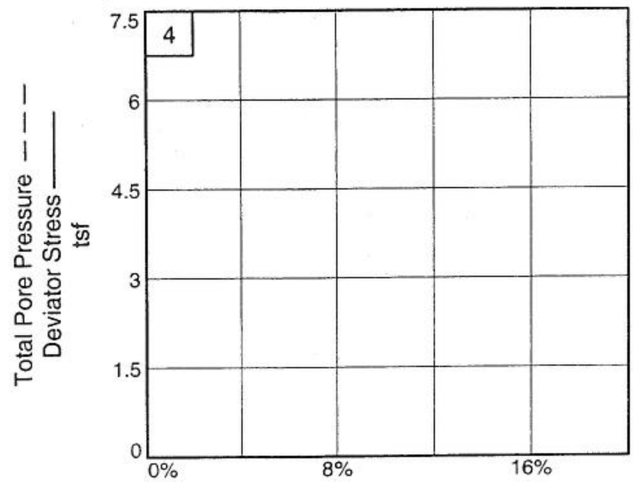
Proj. No.: 0121-3070.03 **Date:** 08/16/06

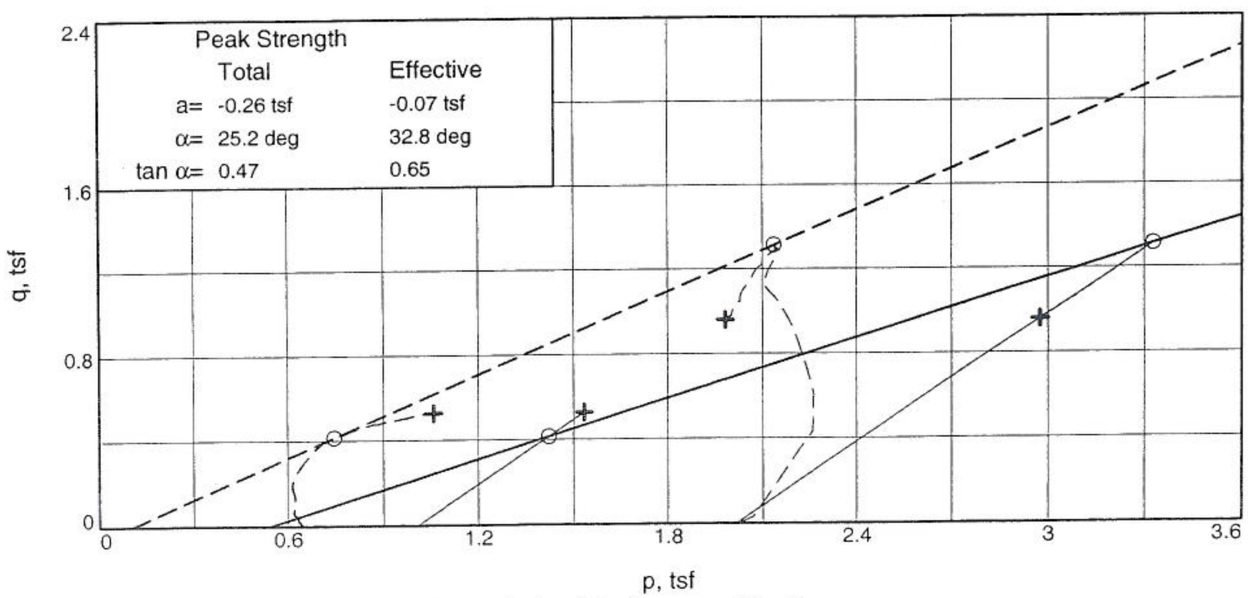
DLZ











Stress Paths: Total ——— Effective – – –

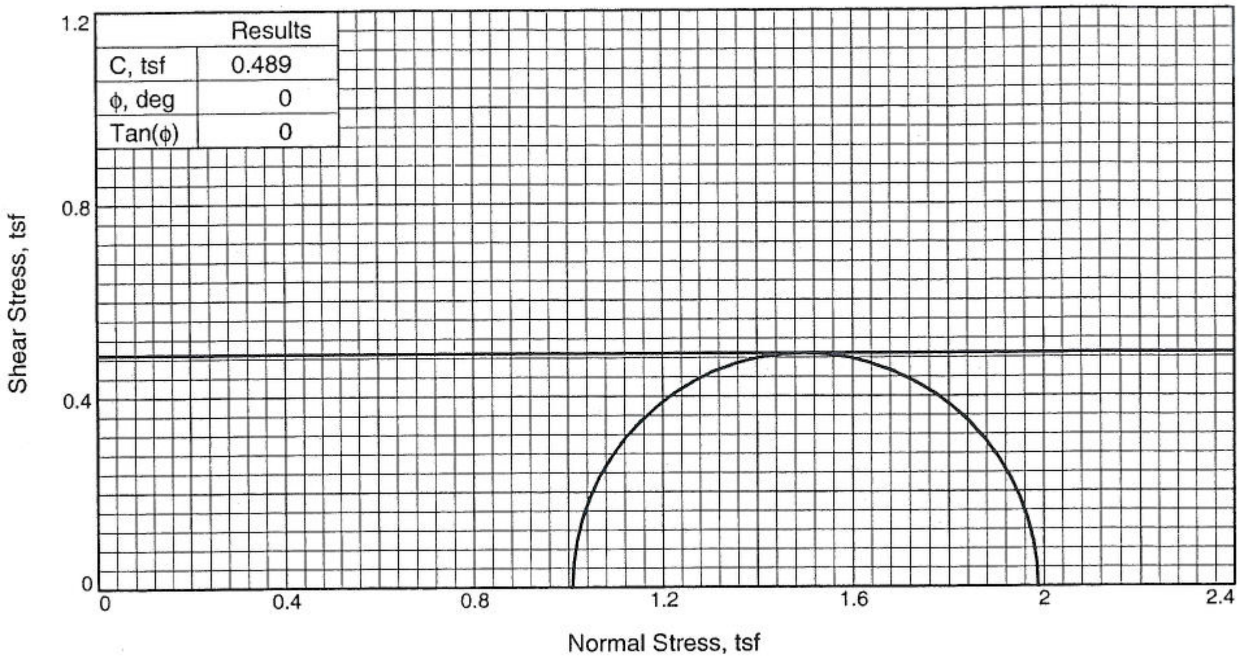
Client: TranSystems, Inc.

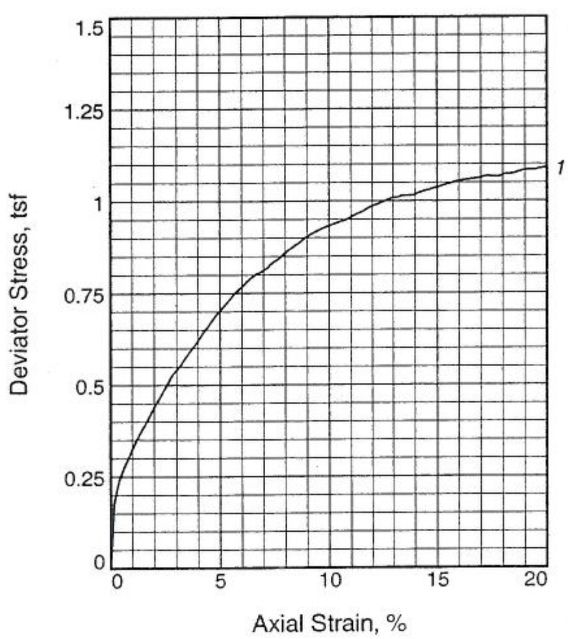
Project: SCI-823-0.00

Source of Sample: B-1108 **Depth:** 18.0 **Sample Number:** P3

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

DLZ, INC.





	Sample No.	1
Initial	Water Content,	26.2
	Dry Density, pcf	97.9
	Saturation,	96.8
	Void Ratio	0.7405
	Diameter, in.	2.84
	Height, in.	5.56
At Test	Water Content,	26.8
	Dry Density, pcf	97.9
	Saturation,	98.7
	Void Ratio	0.7405
	Diameter, in.	2.84
	Height, in.	5.56
	Strain rate, in./min.	0.06
	Back Pressure, tsf	0.00
	Cell Pressure, tsf	1.01
	Fail. Stress, tsf	0.98
	Ult. Stress, tsf	0.98
	σ_1 Failure, tsf	1.99
	σ_3 Failure, tsf	1.01

Type of Test:
Unconsolidated Undrained

Sample Type: Press Tube

Description:

LL= 57 **PL=** 30 **PI=** 27

Assumed Specific Gravity= 2.73

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

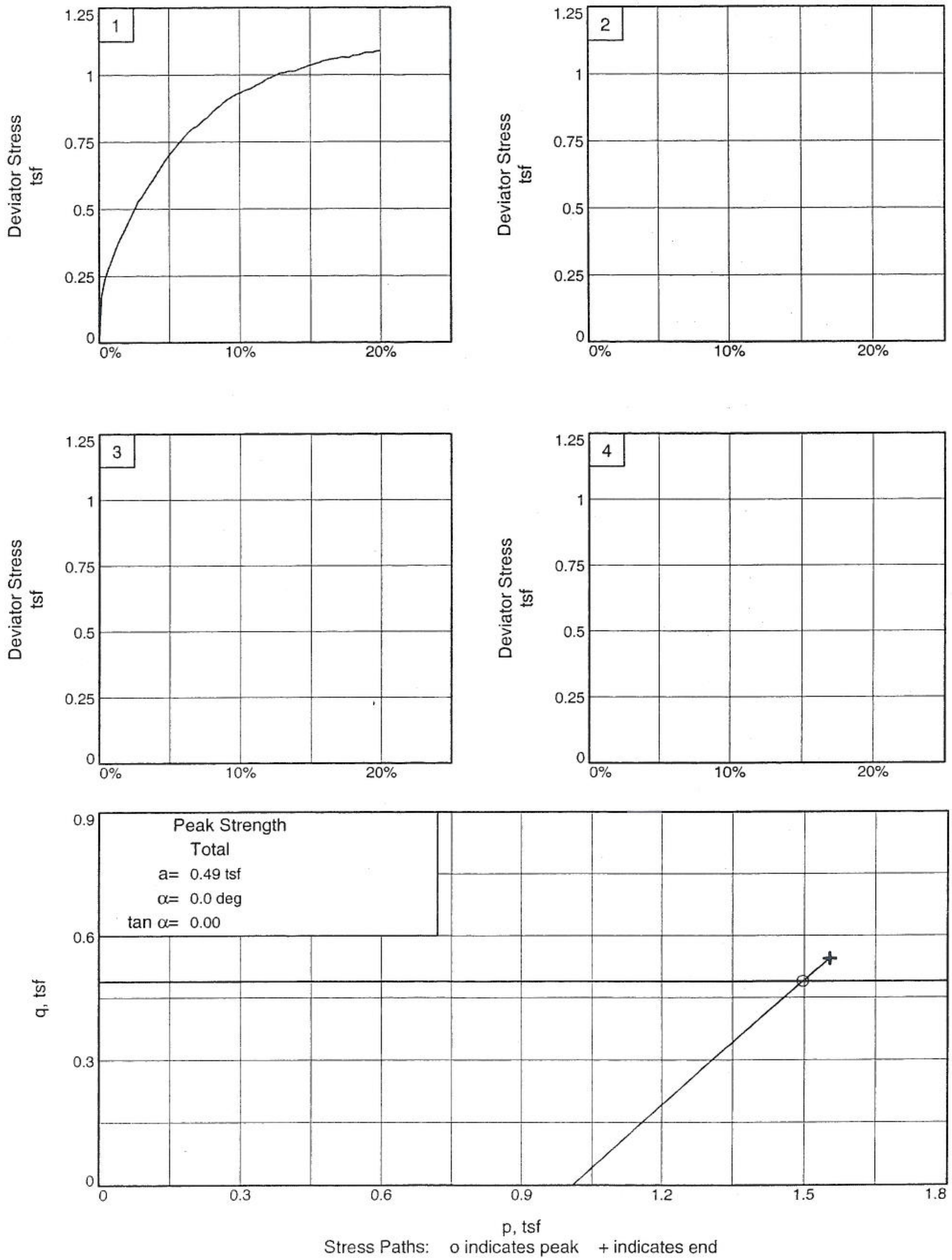
Source of Sample: B-1109A **Depth:** 8.0

Sample Number: ST2

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

DLZ

Figure ________



Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1109A **Depth:** 8.0 **Sample Number:** ST2
Project No.: 0121-3070.03 **Figure** ________

DLZ, INC.

Vane Shear Test Report

Project	SCI-823-0.00	Date and Time	7/27/2007	Begin 3:30pm End 3:50pm
Project No.	0121-3070-03	Boring Number	B-1109a	Depth 8.0
Client	ODOT			
Drill Rig & Crew	D Wamsley			
Tested By	B Mott			
Weather / Temp.	sunny 85			
Soil Type				

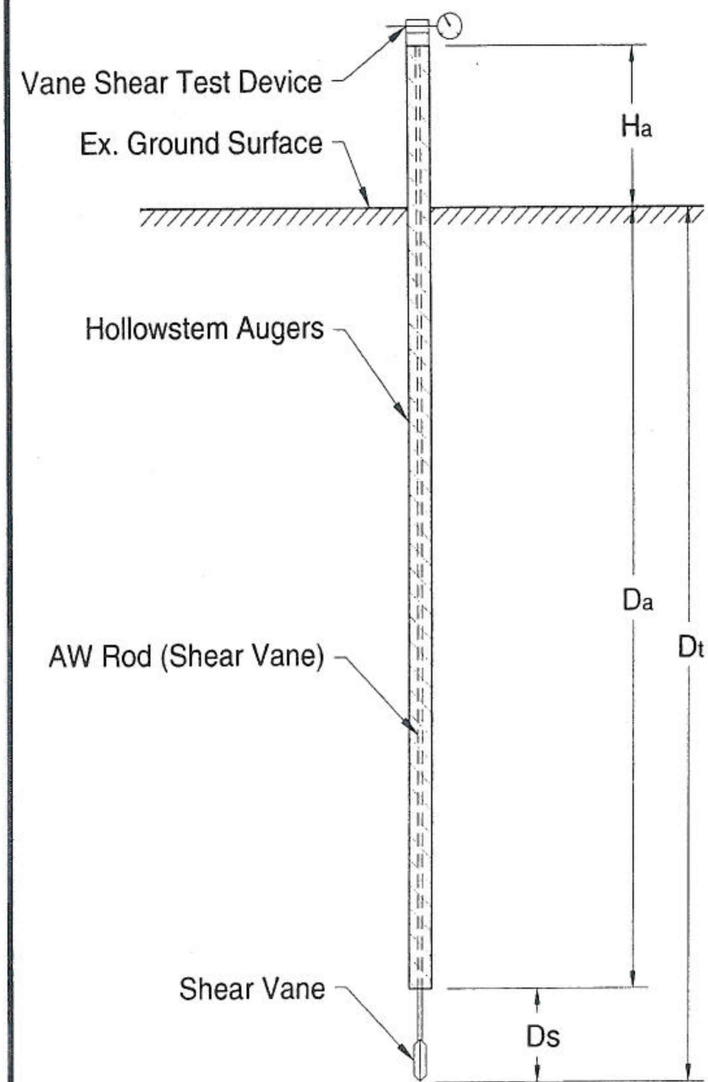
DRILLING

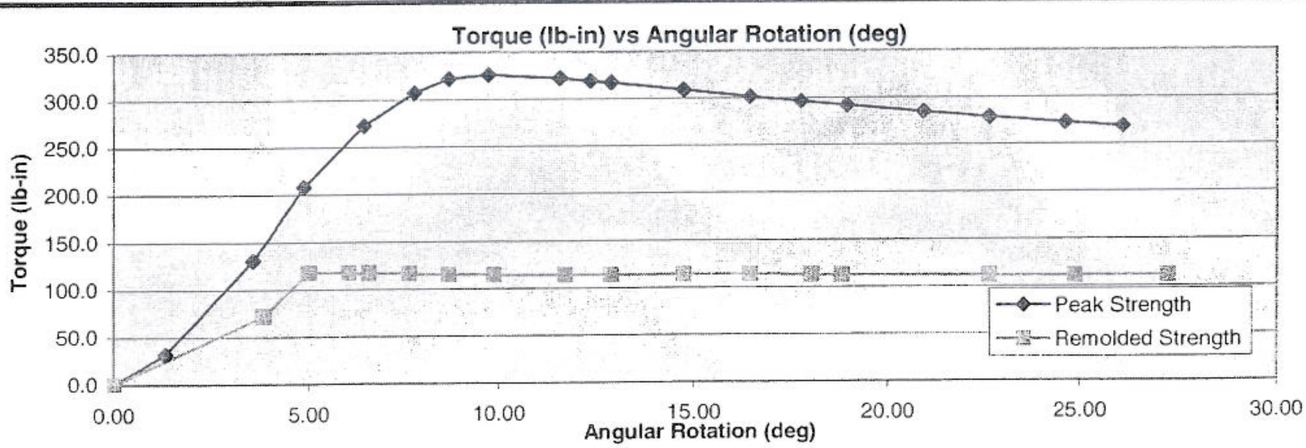
Hollowstem augers to depth	D_a	7.5
Vane Depth below bottom of augers	D_s	1
Augers above ground surface	H_a	7
Depth to vane tip	D_t	8

SHEAR VANE

Vane Used	**2.0"**	2.5"	3.625"
Vane constant, k (lb-in to psf)	**5.17**	2.59	0.905
Masurement by	Automatic/torque cell		
Max Torque	326	lb-in	
Max UD Shear Strength	**1687**	**psf**	

Test Setup





Vane Shear Test Report

Peak Strength Test

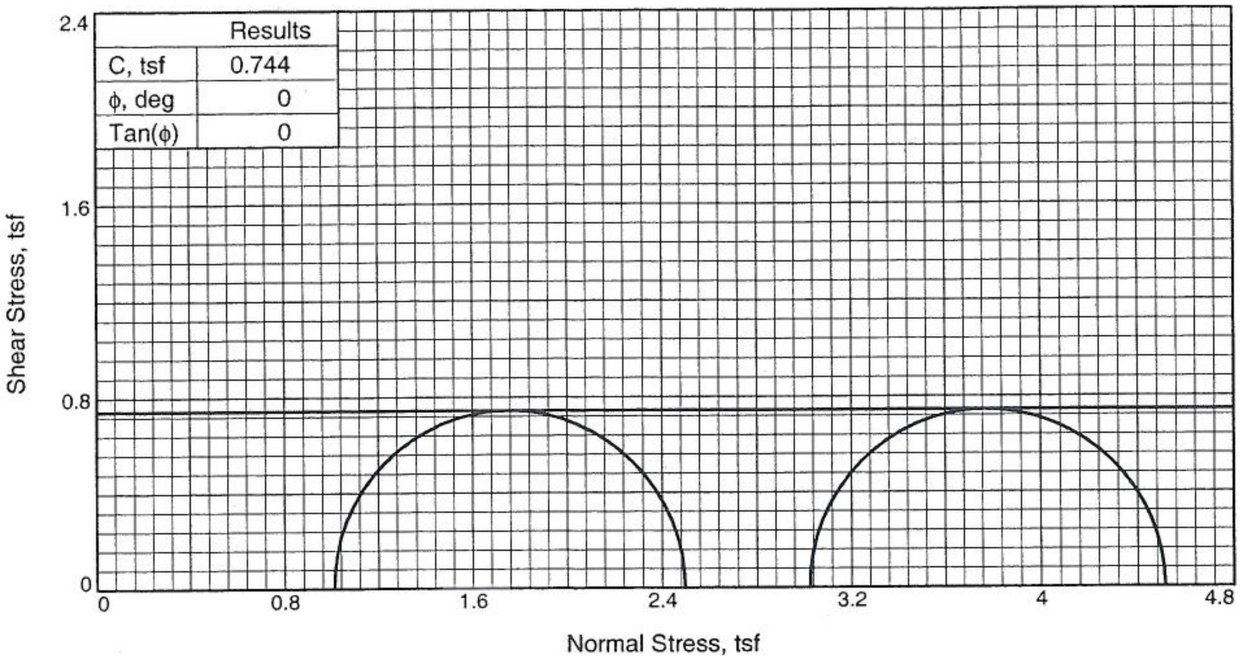
Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
15:32:12	0:00:00	0.00	0.0
15:32:22	0:00:10	1.32	32.0
15:32:39	0:00:27	3.55	131.0
15:32:49	0:00:37	4.87	207.9
15:33:01	0:00:49	6.45	272.8
15:33:11	0:00:59	7.77	307.7
15:33:18	0:01:06	8.69	322.0
15:33:26	0:01:14	9.74	326.3
15:33:40	0:01:28	11.59	322.1
15:33:46	0:01:34	12.38	319.2
15:33:50	0:01:38	12.90	317.3
15:34:04	0:01:52	14.75	309.2
15:34:17	0:02:05	16.46	301.6
15:34:27	0:02:15	17.77	296.3
15:34:36	0:02:24	18.96	291.8
15:34:51	0:02:39	20.93	284.9
15:35:04	0:02:52	22.65	279.1
15:35:19	0:03:07	24.62	273.0
15:35:30	0:03:18	26.07	268.7

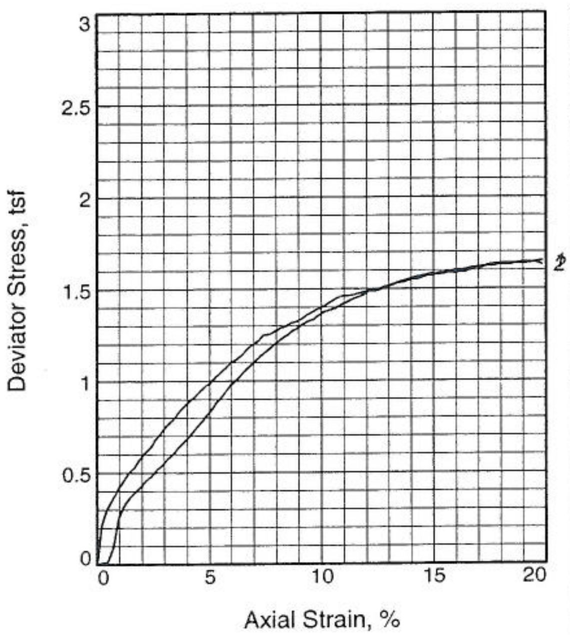
Remolded Strength Test

Read Time	Δ time	Rotation (degrees)	Torque (in-lbs.)
15:42:11	0:00:00	0.00	0.0
15:42:40	0:00:29	3.82	72.1
15:42:49	0:00:38	5.00	117.3
15:42:57	0:00:46	6.06	117.3
15:43:01	0:00:50	6.58	116.8
15:43:09	0:00:58	7.64	115.8
15:43:17	0:01:06	8.69	114.8
15:43:26	0:01:15	9.87	114.4
15:43:40	0:01:29	11.72	113.8
15:43:49	0:01:38	12.90	113.5
15:44:03	0:01:52	14.75	113.7
15:44:16	0:02:05	16.46	113.4
15:44:28	0:02:17	18.04	112.7
15:44:34	0:02:23	18.83	112.4
15:45:03	0:02:52	22.65	111.3
15:45:20	0:03:09	24.89	111.1
15:45:38	0:03:27	27.25	111.0

Peak Torque *326.3333* (lb-in)
Vane Constant *5.17*
Peak Shear Strength *1687* *psf*

Remolded Torque *117.35* (lb-in)
Vane Constant *5.17*
Remolded Shear Strength *606.69* *psf*





	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:

Client: TranSystems, Inc.

Project: SCI-823-0.00

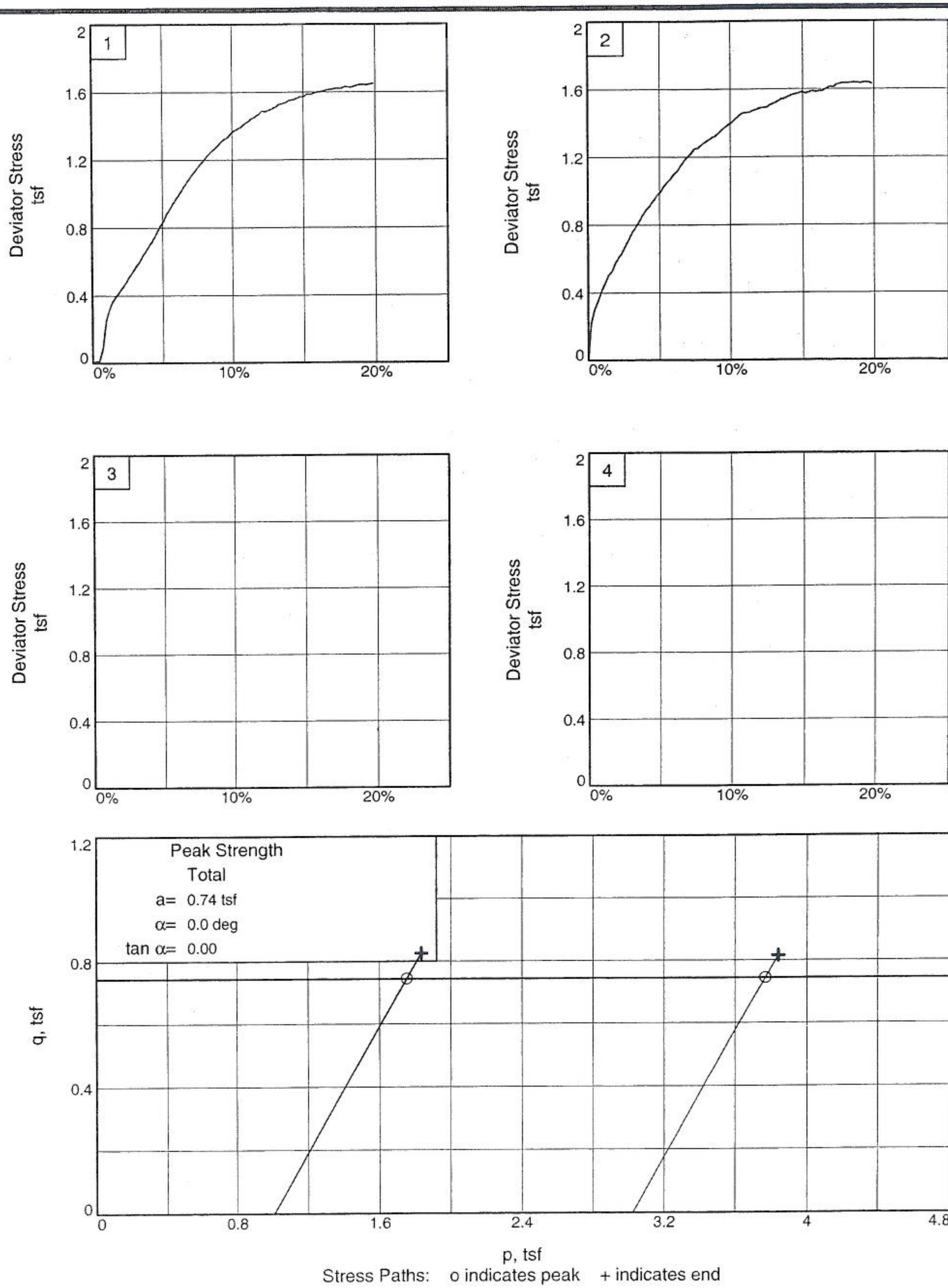
Source of Sample: B-45 **Depth:** 5.0

Sample Number: P-1

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

DLZ

Figure ________



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.

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)

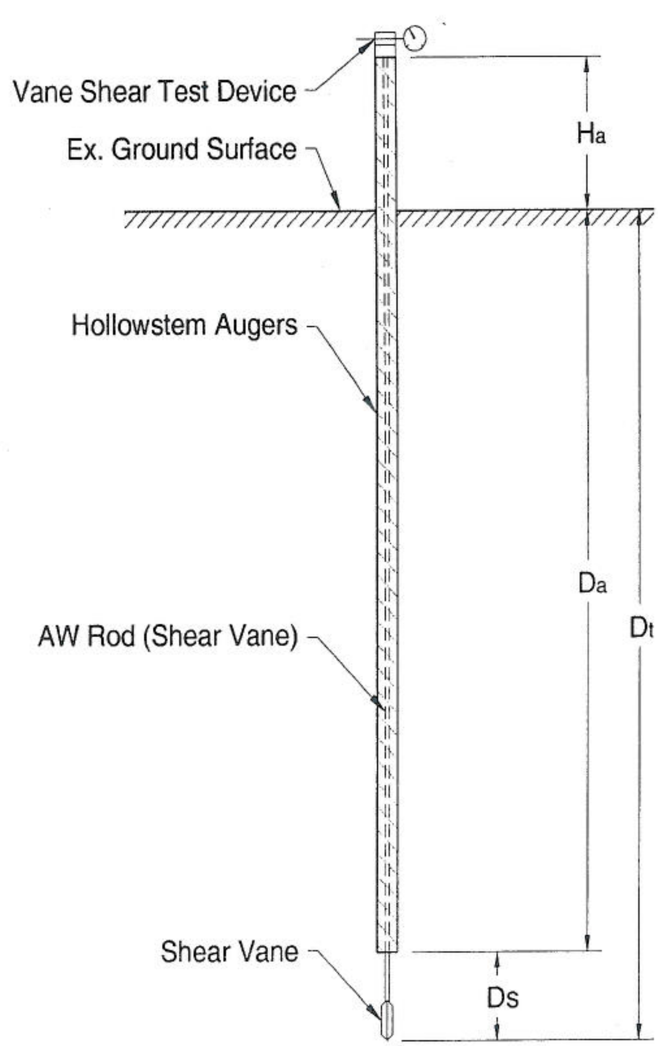
DRILLING

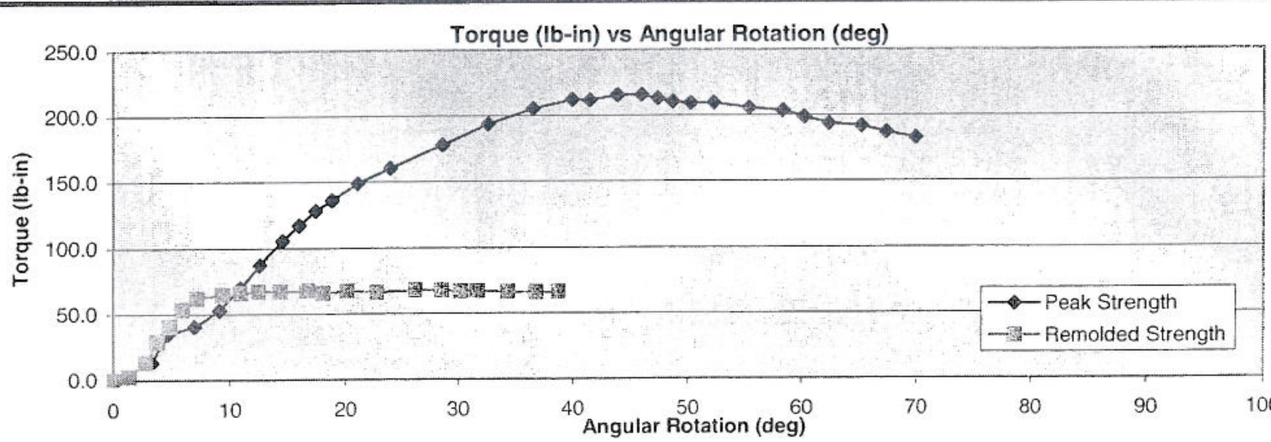
Hollowstem augers to depth	D_a	5'
Vane Depth below bottom of augers	D_s	1'
Augers above ground surface	H_a	2'
Depth to vane tip	D_t	6'

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

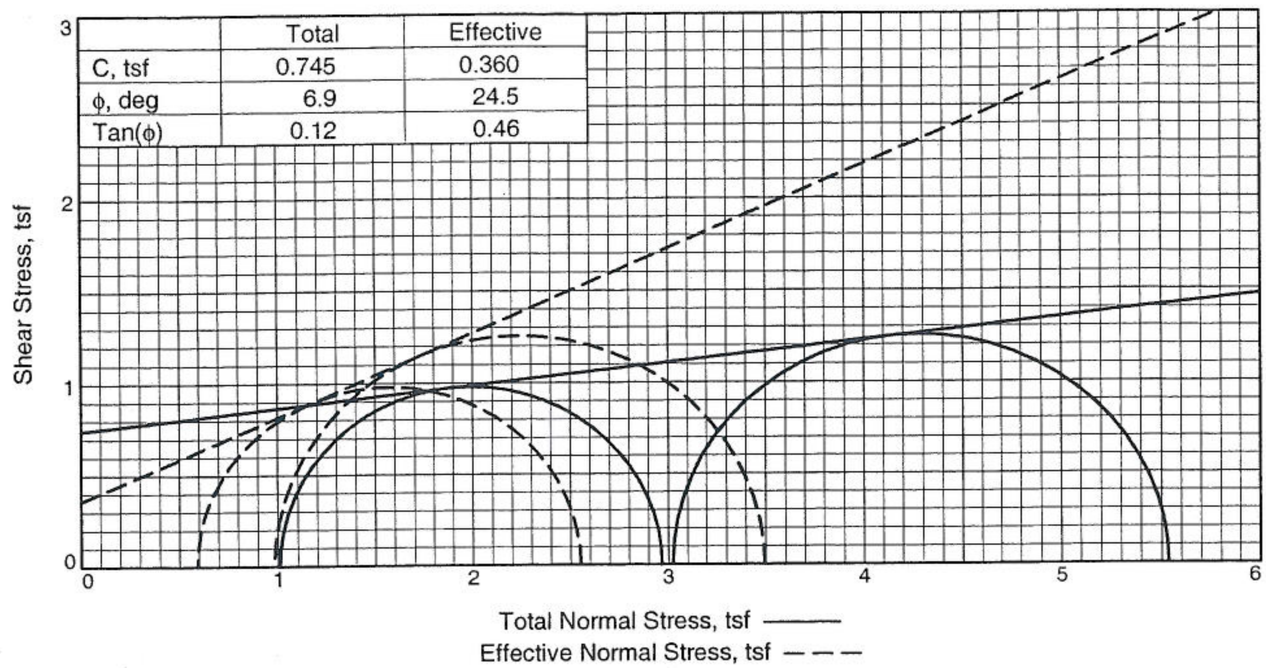
Peak Torque *215.7953* (lb-in)
Vane Constant *5.17*
Peak Shear Strength ***1116*** *psf*

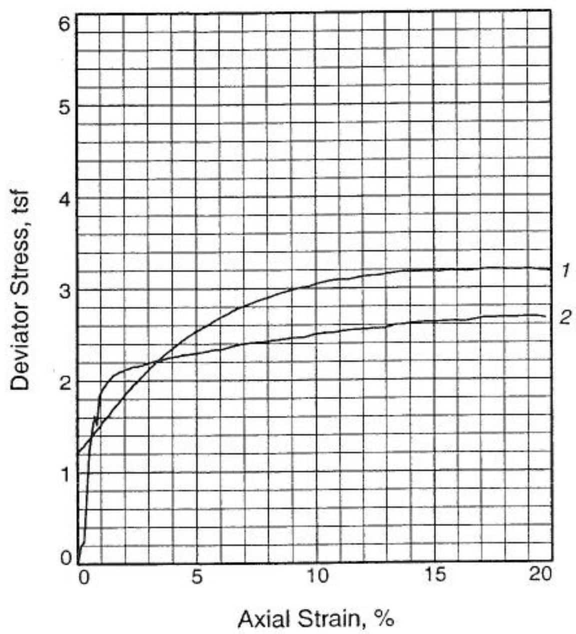
Remolded Strength Test

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

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

	Total	Effective
C, tsf	0.745	0.360
ϕ, deg	6.9	24.5
Tan(ϕ)	0.12	0.46





	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.

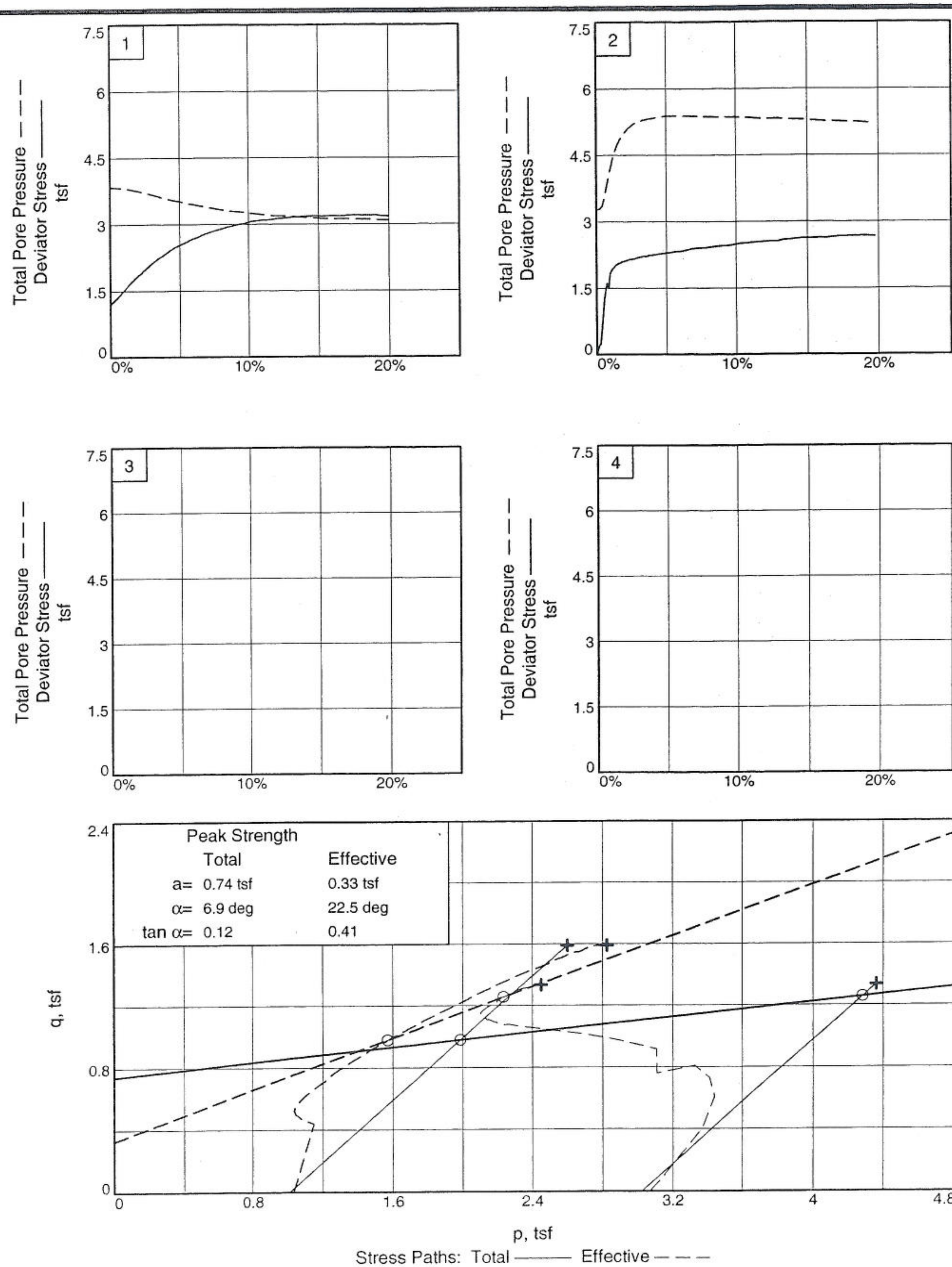
Project: SCI-823-0.00

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

Sample Number: P-2

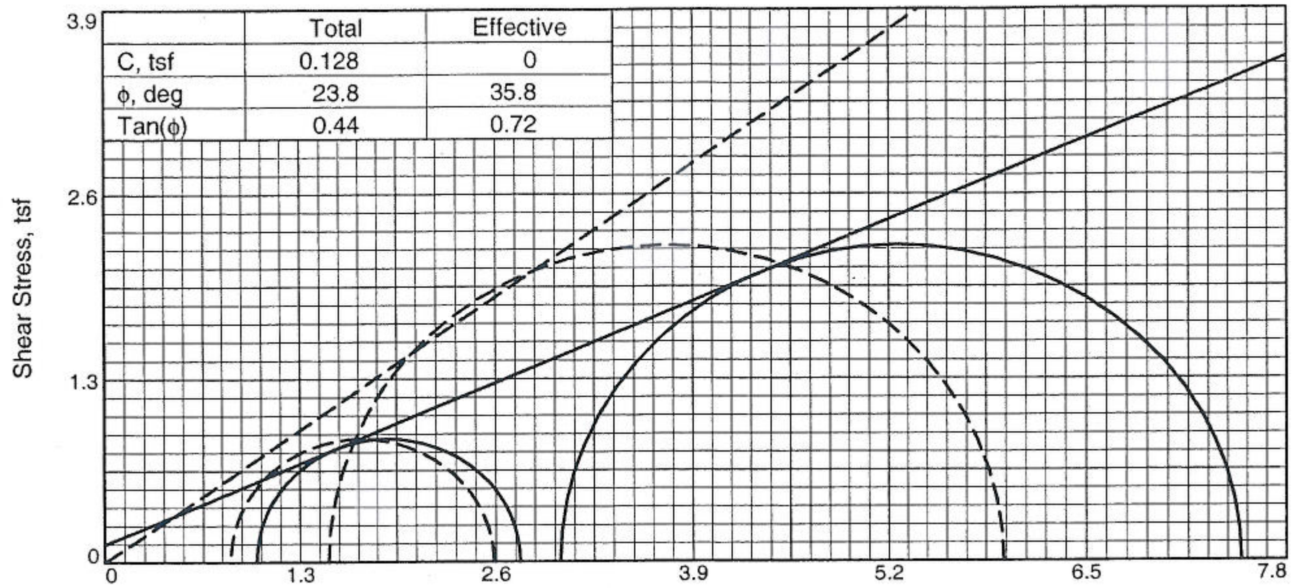
Proj. No.: 0121-3070.03 **Date:** 7/20/07

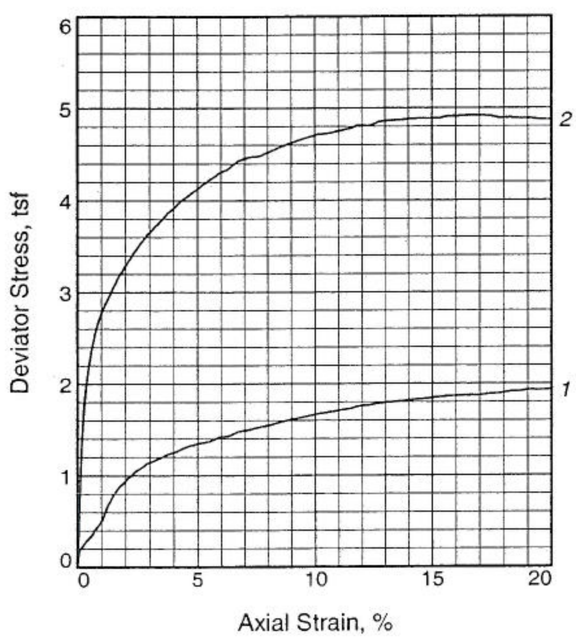
DLZ



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,	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.

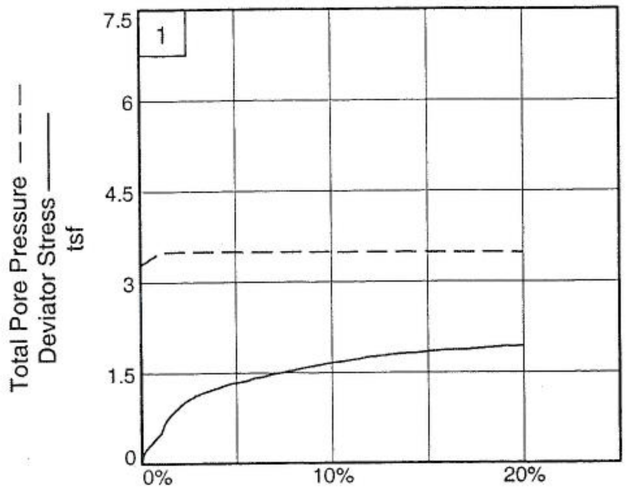
Project: SCI-823-0.00

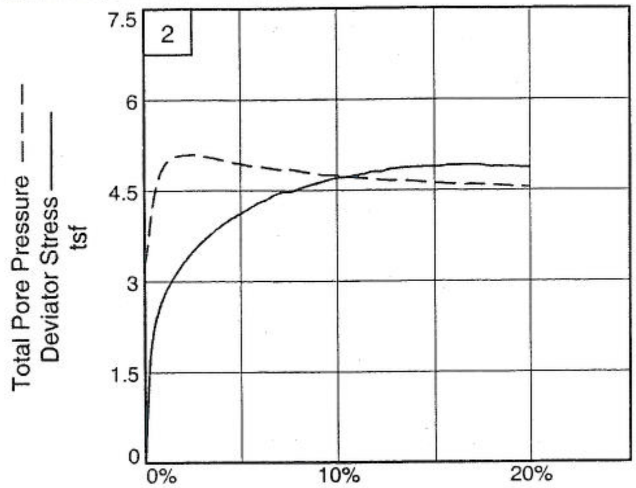
Source of Sample: B-46 **Depth:** 8.0

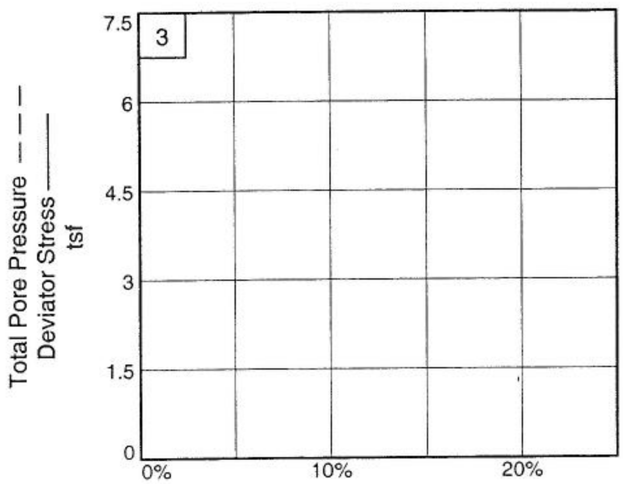
Sample Number: P-2

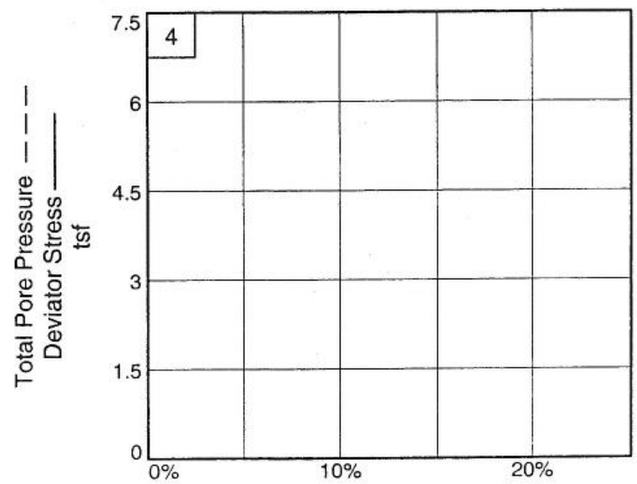
Proj. No.: 0121-3070.03 **Date:** 7/20/07

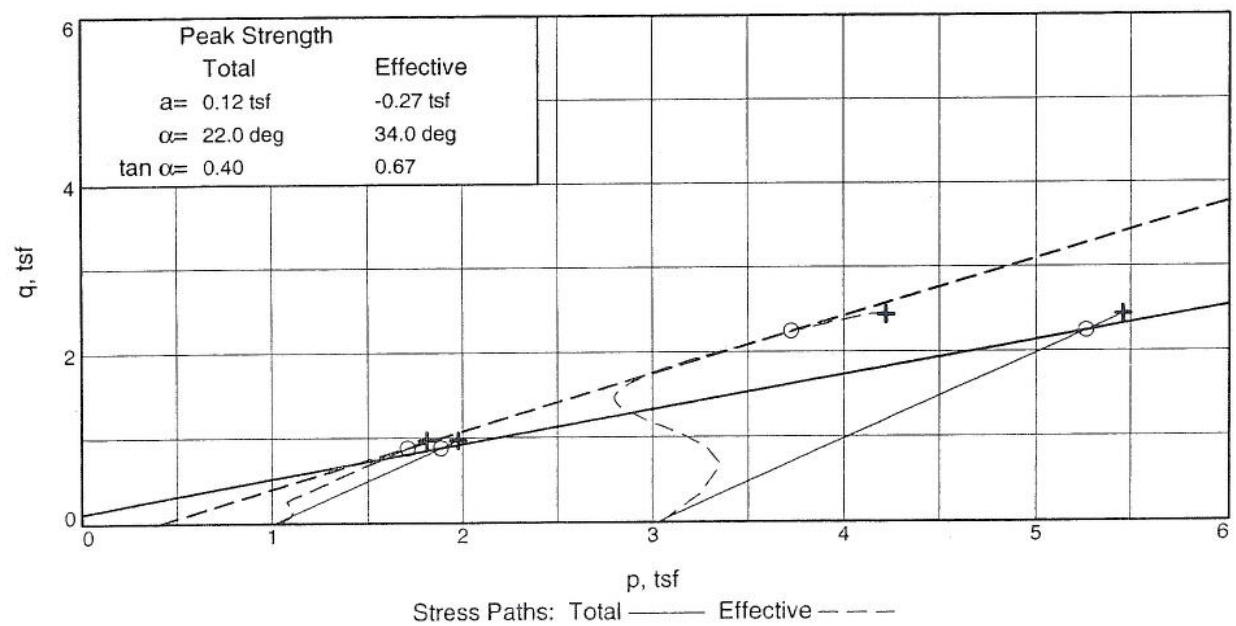
DLZ





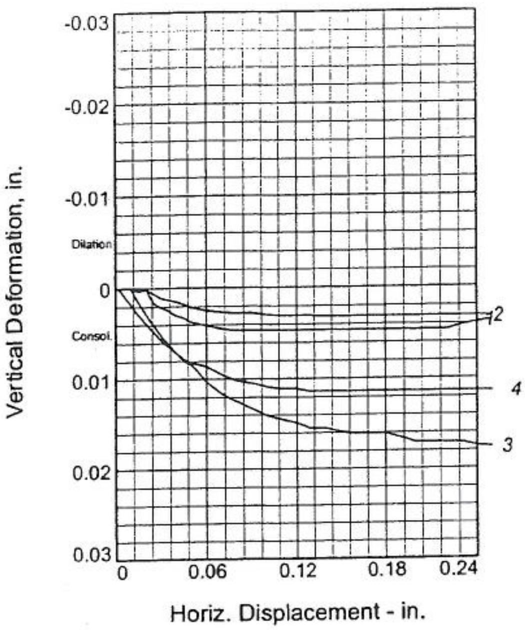


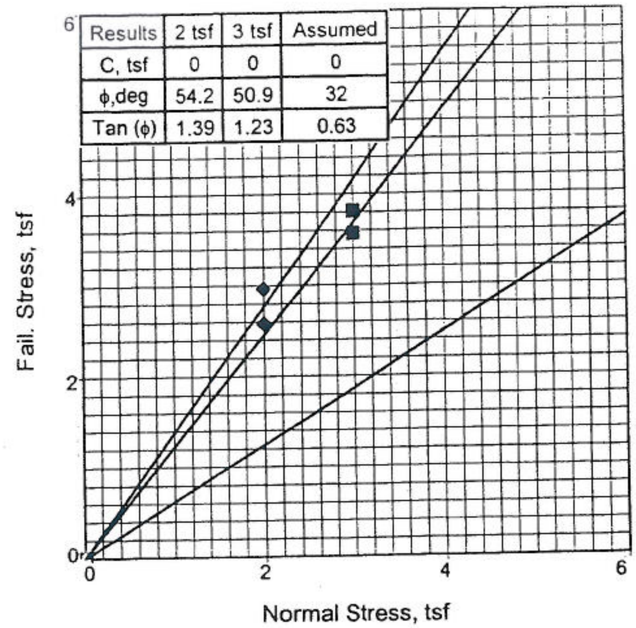




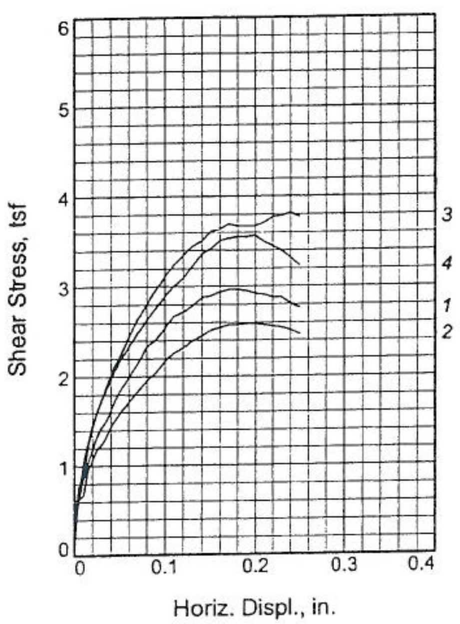
Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-46 **Depth:** 8.0 **Sample Number:** P-2
Project No.: 0121-3070.03 **Figure** ________

DLZ, INC.





Results	2 tsf	3 tsf	Assumed
C, tsf	0	0	0
φ,deg	54.2	50.9	32
Tan (φ)	1.39	1.23	0.63



	Sample No.	1	2	3	4
Initial	Water Content, %	28.7	28.7	28.7	28.7
	Dry Density, pcf	106.4	101.0	98.5	101.4
	Saturation, %	132.3	115.7	108.8	116.8
	Void Ratio	0.5849	0.6691	0.7111	0.6628
	Diameter, in.	2.50	2.50	2.50	2.50
	Height, in.	1.02	1.21	1.27	1.21
At Test	Water Content, %	21.2	21.2	19.4	19.4
	Dry Density, pcf	111.1	104.8	102.5	104.9
	Saturation, %	110.7	94.1	81.2	86.3
	Void Ratio	0.5172	0.6089	0.6451	0.6069
	Diameter, in.	2.50	2.50	2.50	2.50
	Height, in.	0.98	1.16	1.22	1.17
	Normal Stress, tsf	2.000	2.000	3.000	3.000
	Fail. Stress, tsf	2.963	2.582	3.814	3.564
	Displacement, in.	0.17	0.19	0.24	0.20
	Ult. Stress, tsf				
	Displacement, in.				
	Strain rate, in./min.	0.01	0.01	0.01	0.01

Sample Type: Standard Penetration Test
Description: Silty sand

LL= NP **PL=** NP **PI=** NP
Assumed Specific Gravity= 2.7
Remarks: Due to small REC, S-6 & S-7 were combined for testing. Samples were completely saturated and contained "free water". Sample was stirred prior to testing, to incorporate excess water.
Figure ________

Client: TranSystems, Inc.

Project: SCI-823-0.00

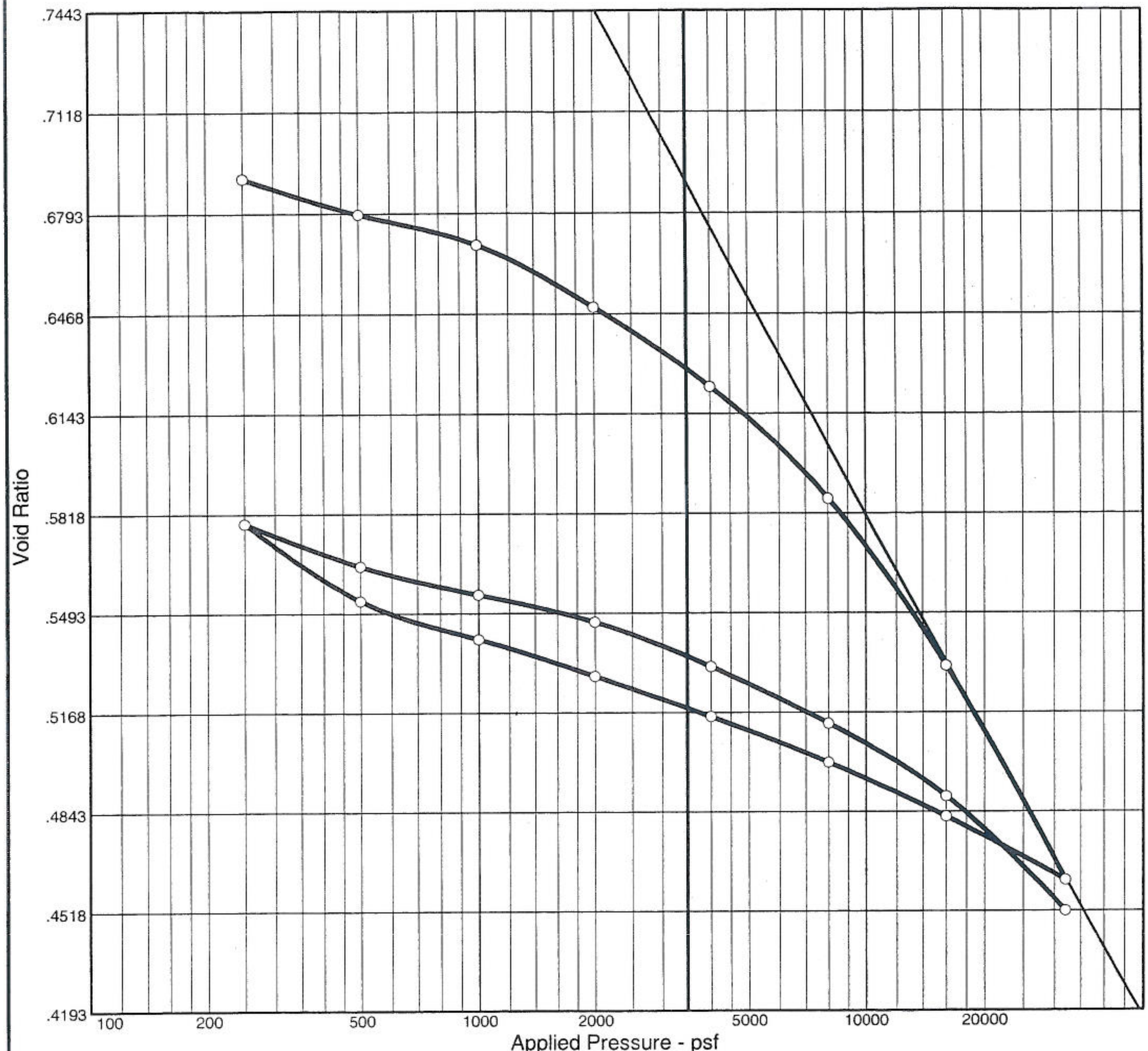
Source of Sample: TR-61 **Depth:** 13.5
Sample Number: 6

Proj. No.: 0121-3070.03 **Date:** 11/7/05

DLZ

Tested By: JN

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
84.9 %	21.9 %	99.2	30	11	2.69			0.694

MATERIAL DESCRIPTION

Silt and Clay (A-6a)

Project No. 0121-

Client: TranSystems, Inc.

Project: SCI-823-0.00

Source: B-1105A **Sample No.:** ST3 **Elev./Depth:** 12.0

DLZ

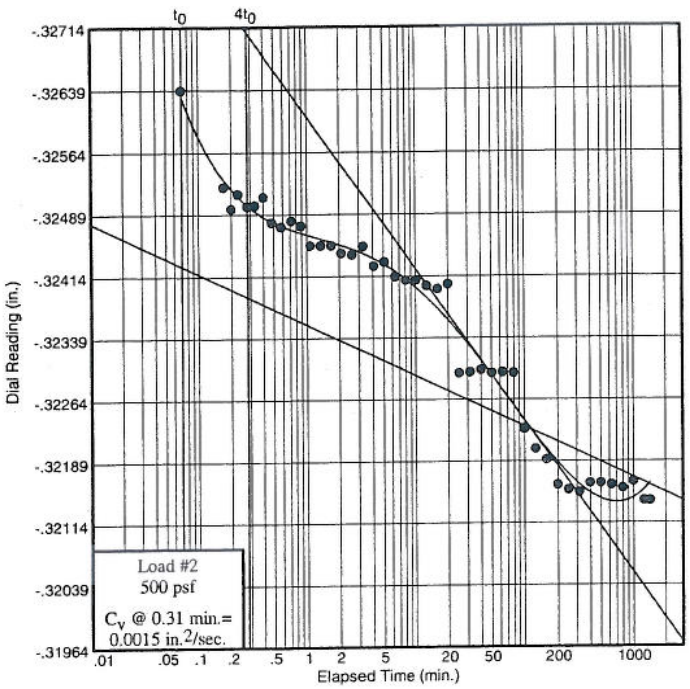
Remarks:

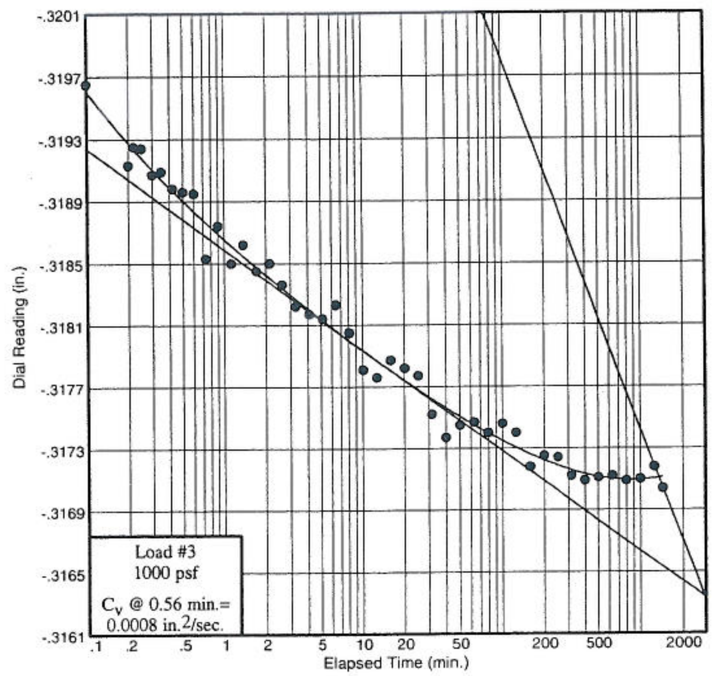
Figure

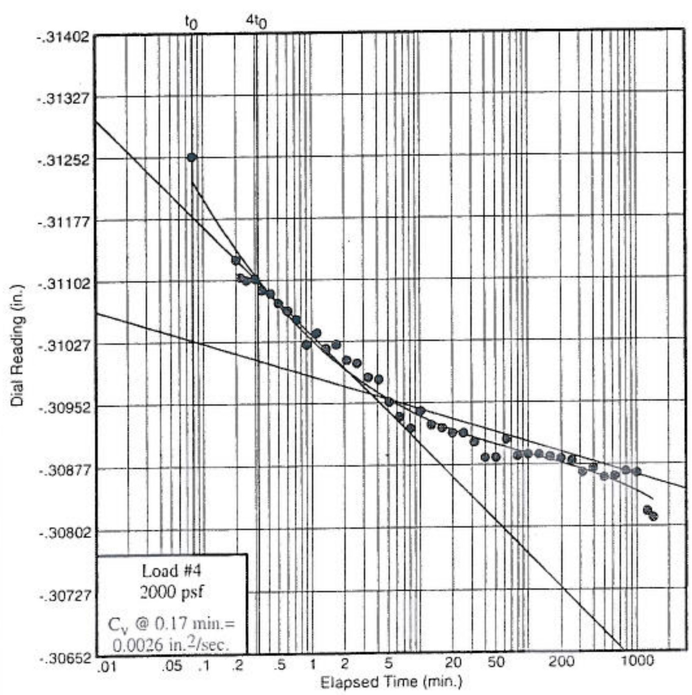
Dial Reading vs. Time

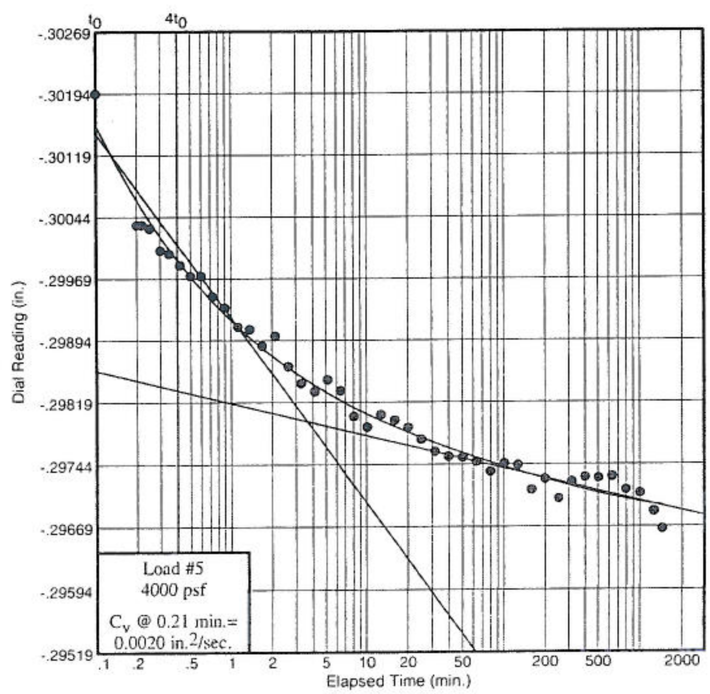
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1105A   Sample No.: ST3   Elev./Depth: 12.0









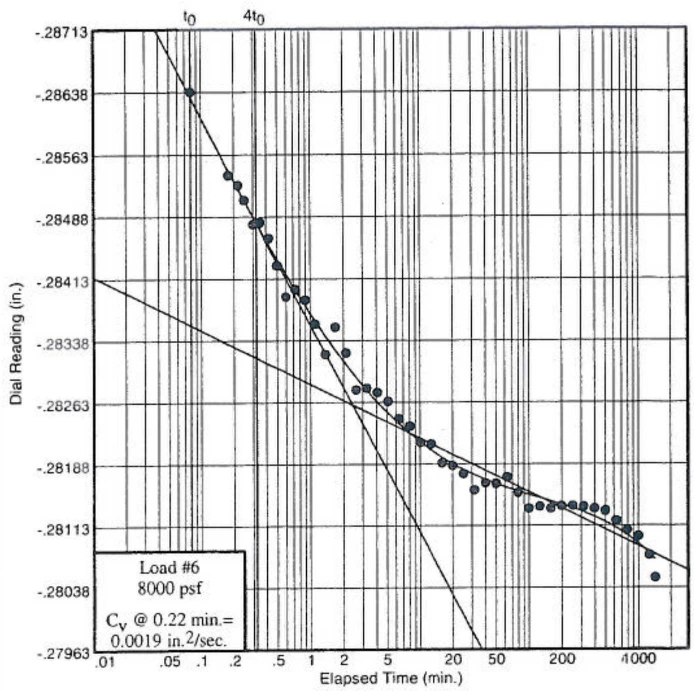
DLZ

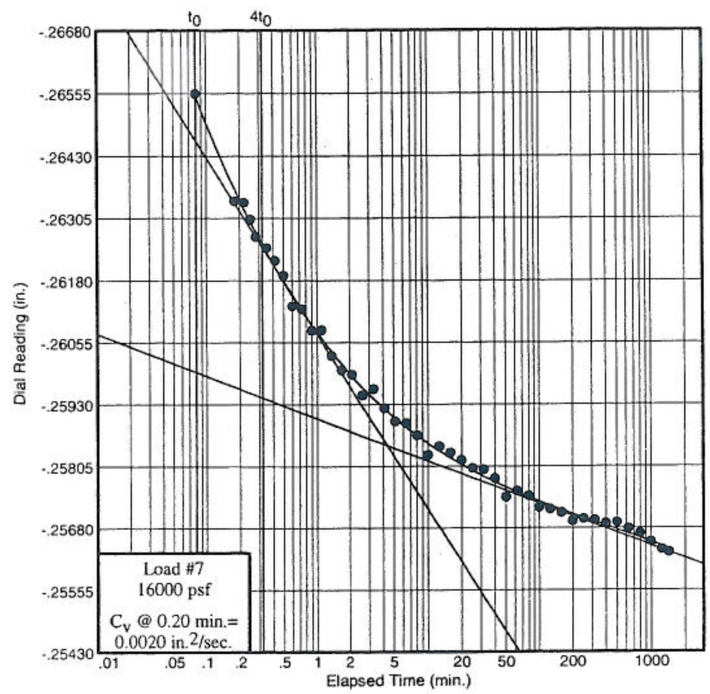
Figure

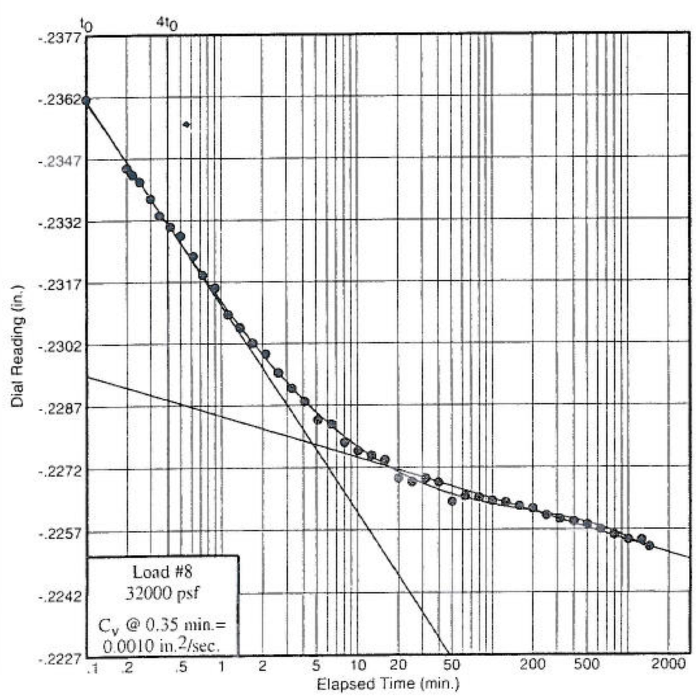
Dial Reading vs. Time

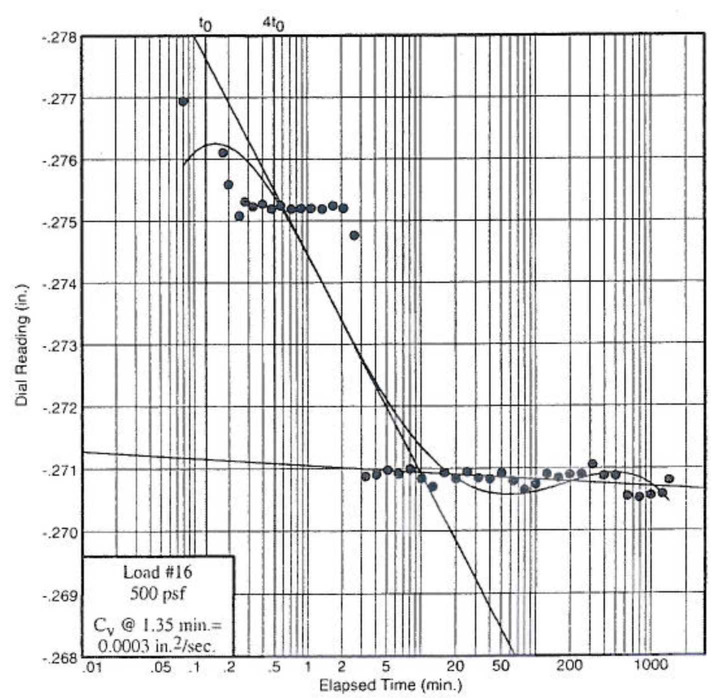
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1105A | Sample No.: ST3 | Elev./Depth: 12.0









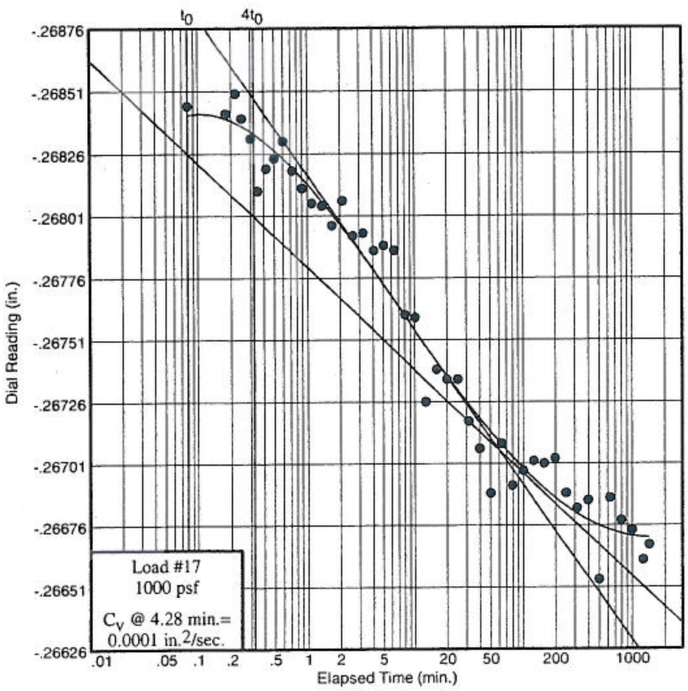
DLZ

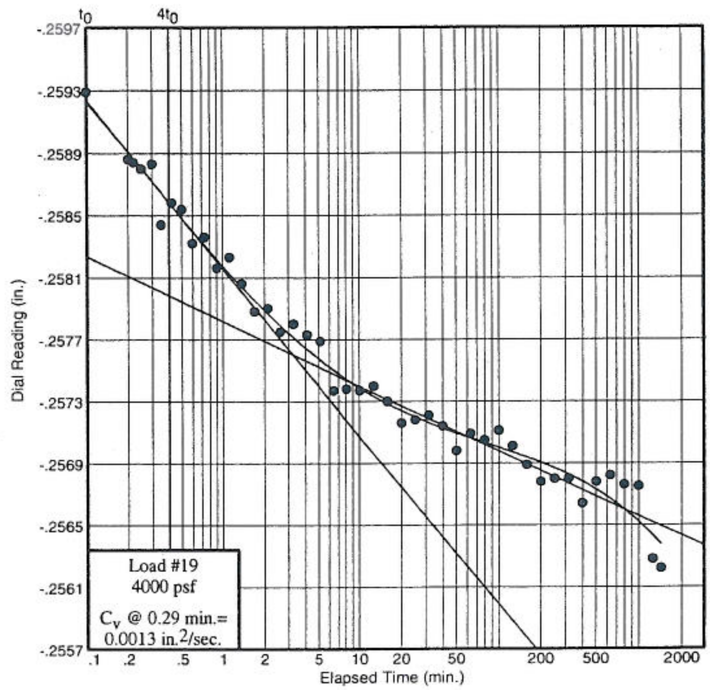
Figure

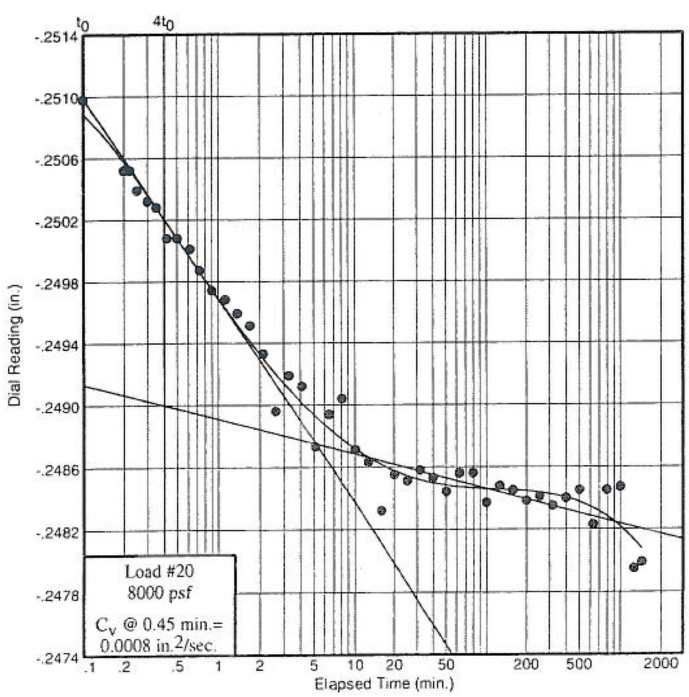
Dial Reading vs. Time

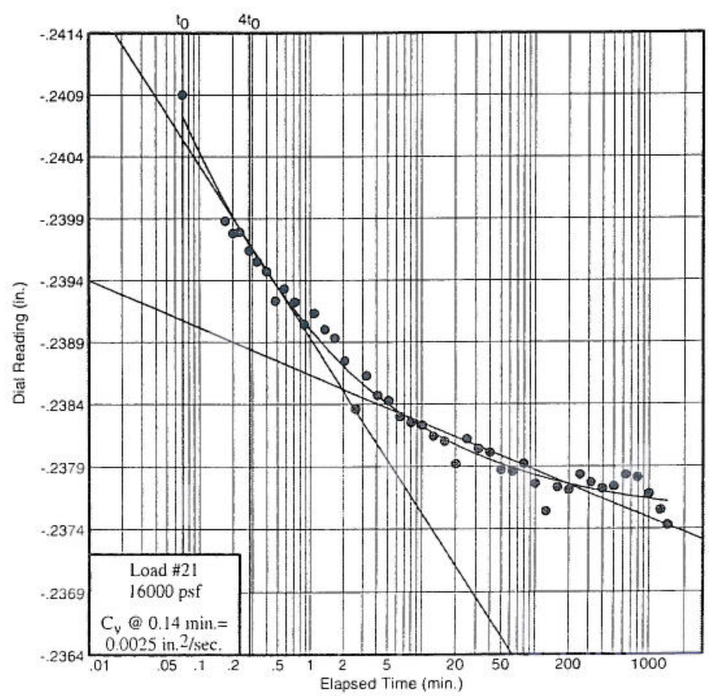
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1105A   Sample No.: ST3   Elev./Depth: 12.0









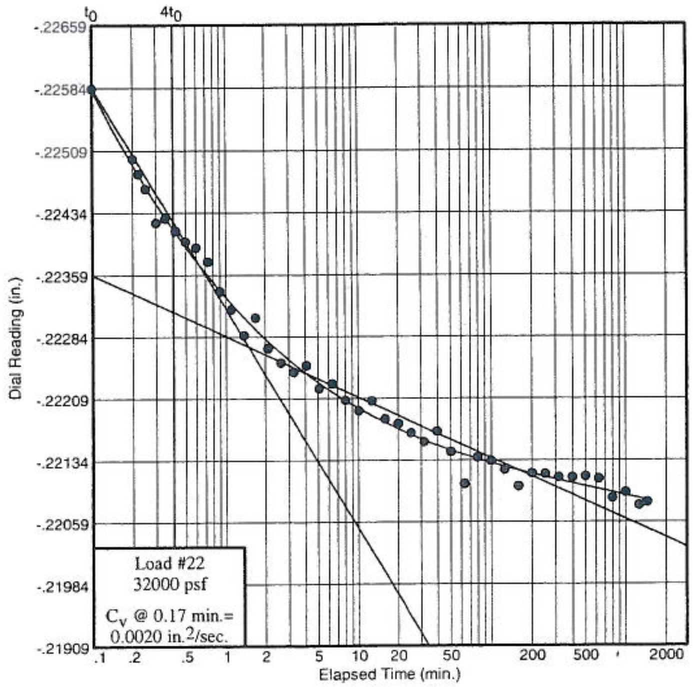
DLZ

Figure

Dial Reading vs. Time

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

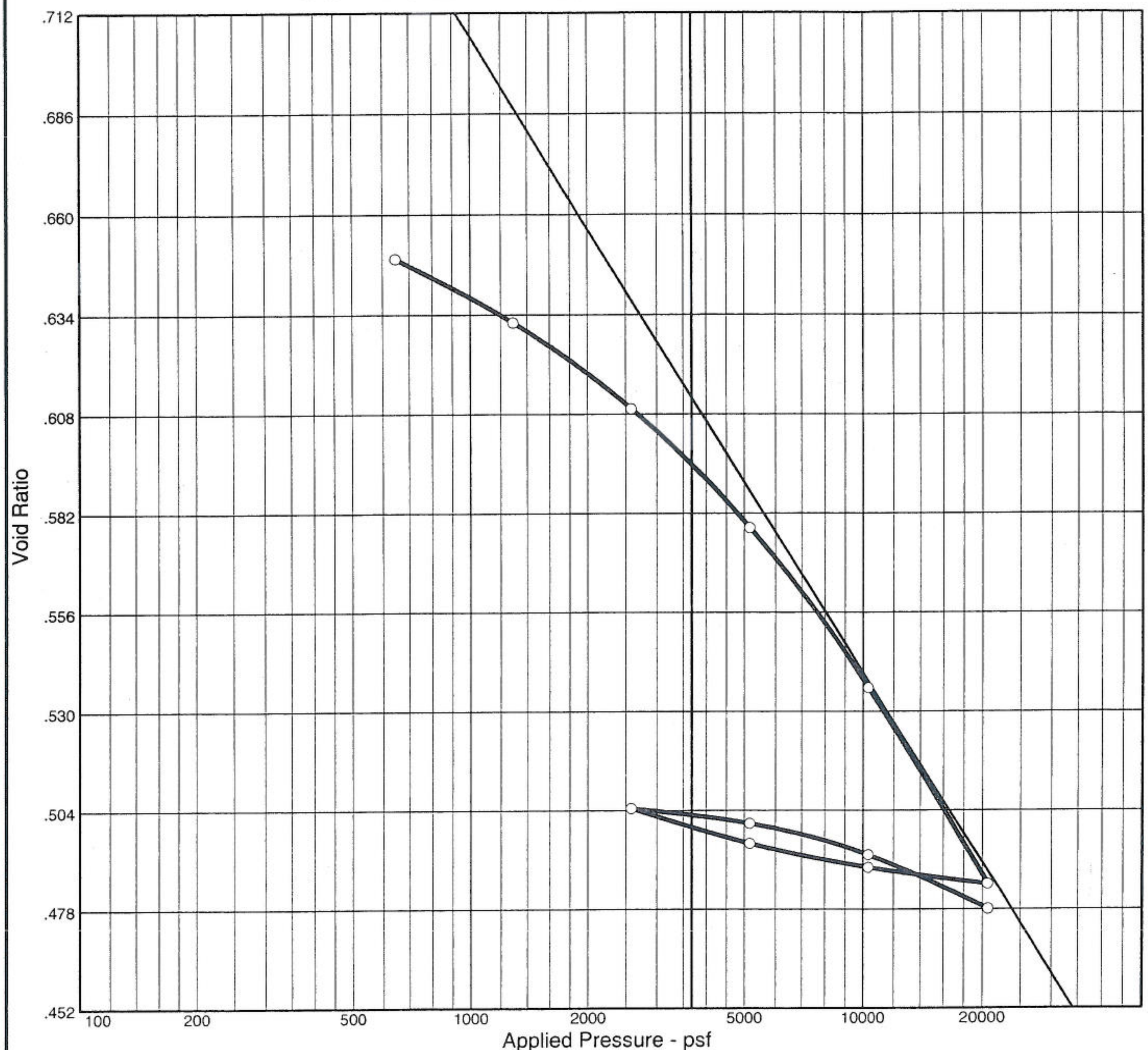
Source: B-1105A Sample No.: ST3 Elev./Depth: 12.0



DLZ

Figure

CONSOLIDATION TEST REPORT



Natural Saturation	Natural Moisture	Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
95.5 %	23.0 %	101.0	36	15	2.65	CL	A-6(15)	0.639

MATERIAL DESCRIPTION

Silt and Clay (A-6b)

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

Project: SCI-823-0.00

Source: B-1108 **Sample No.:** P1 **Elev./Depth:** 10.0

DLZ

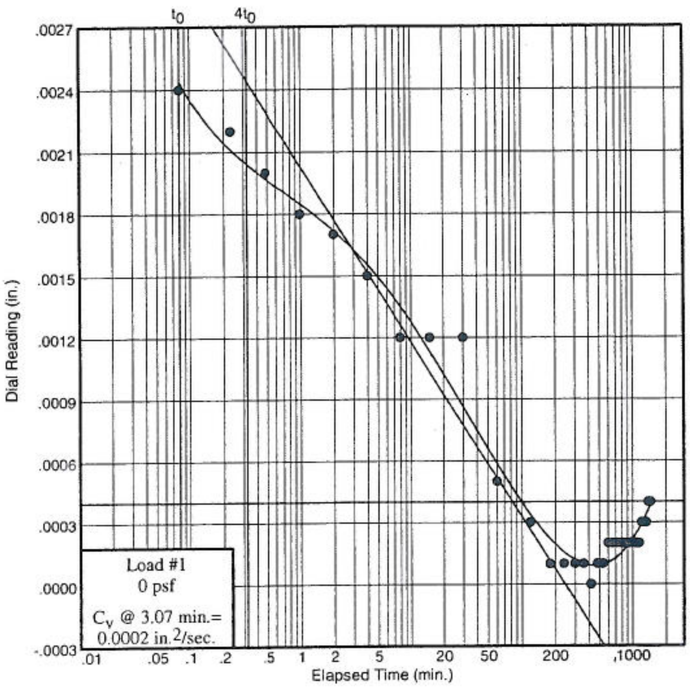
Remarks:

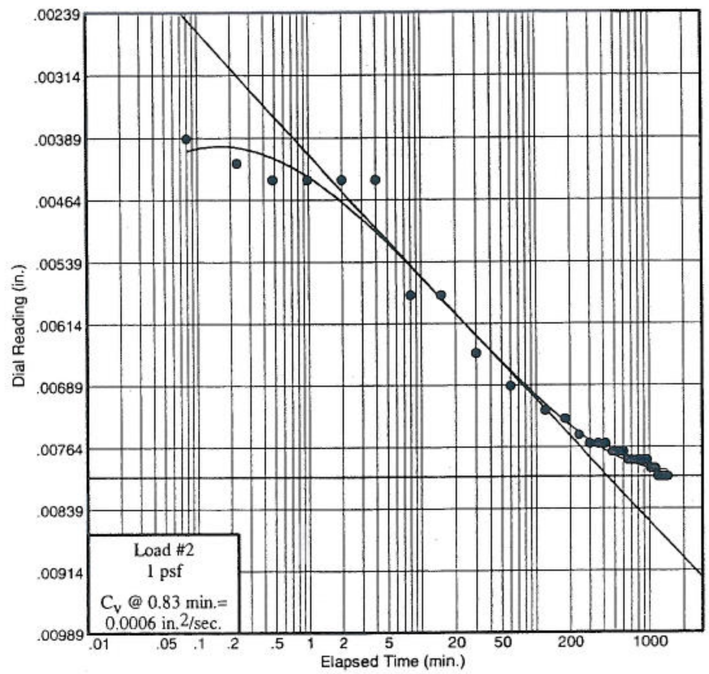
Figure

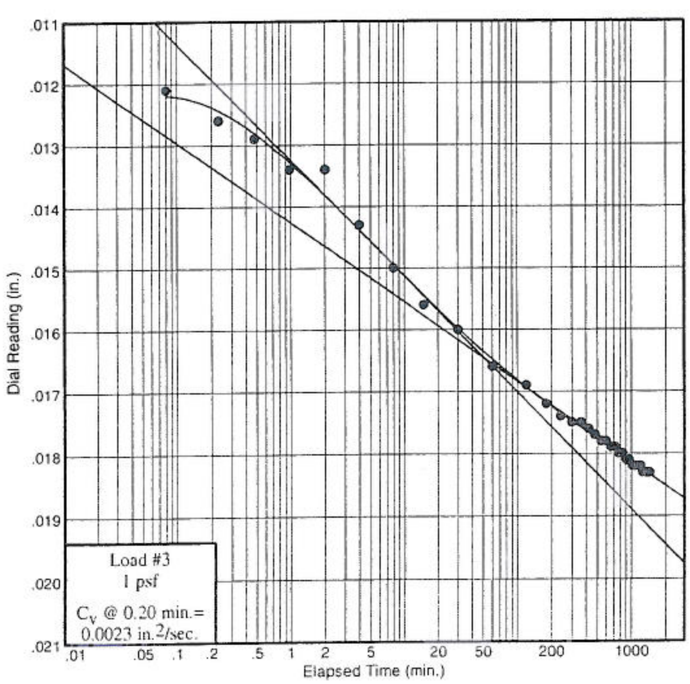
Dial Reading vs. Time

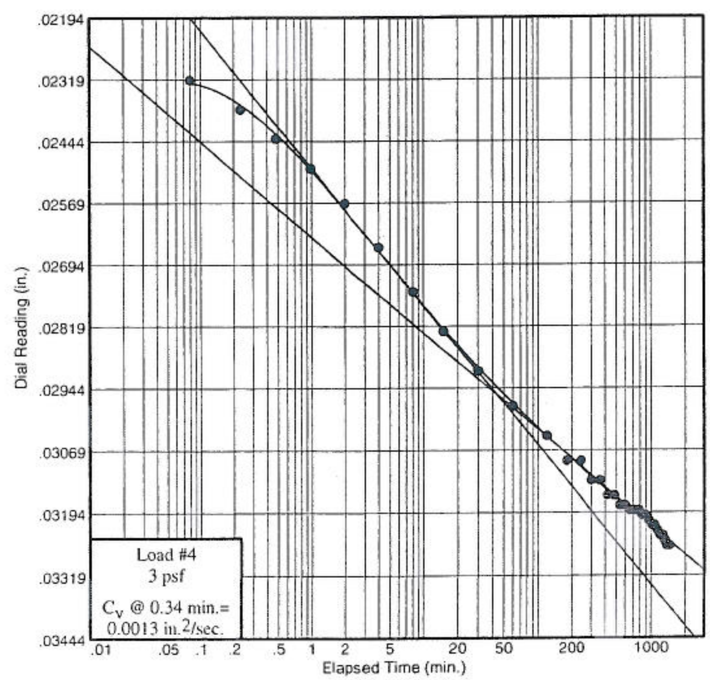
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1108 | Sample No.: P1 | Elev./Depth: 10.0









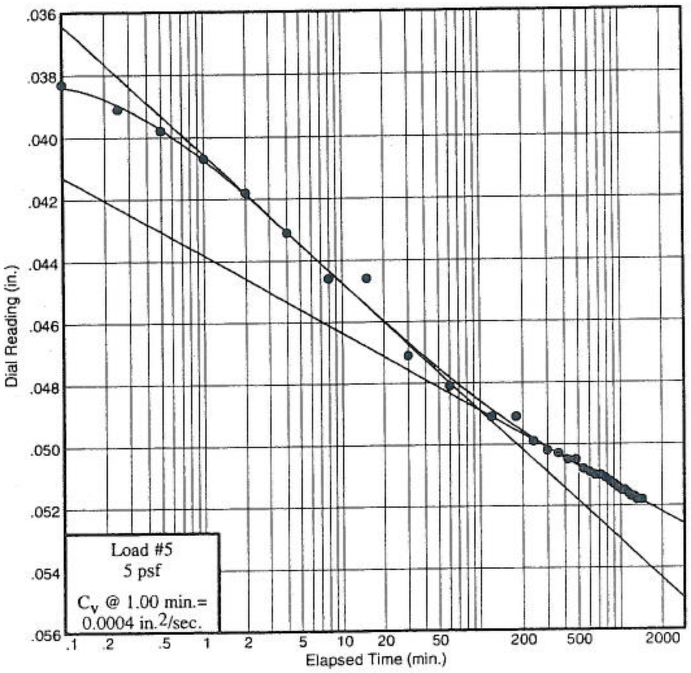
DLZ

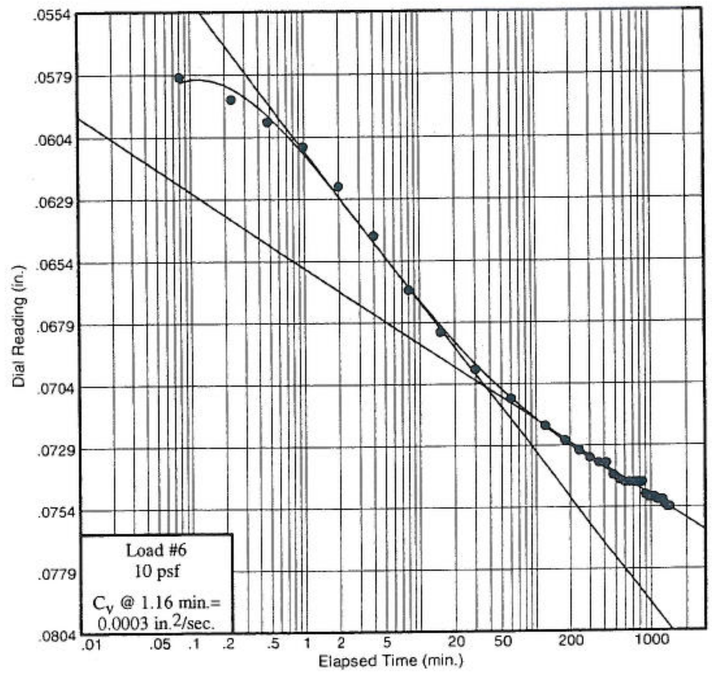
Figure

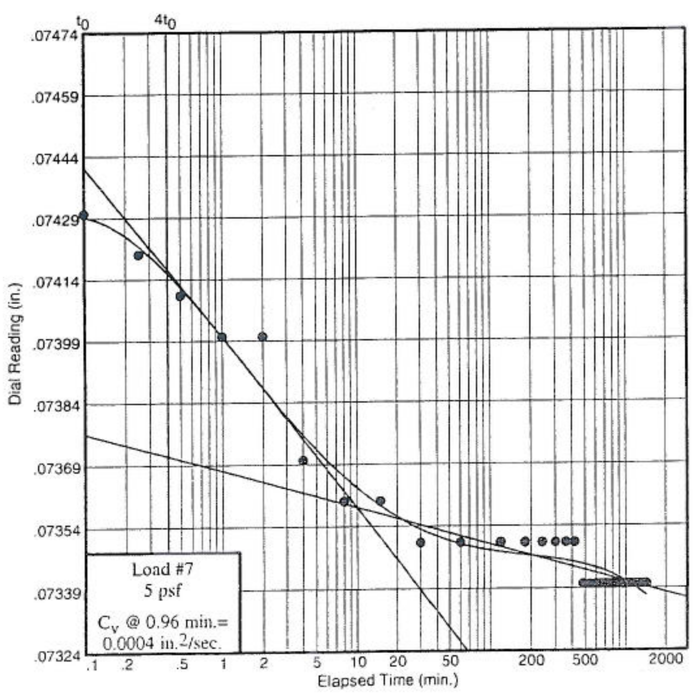
Dial Reading vs. Time

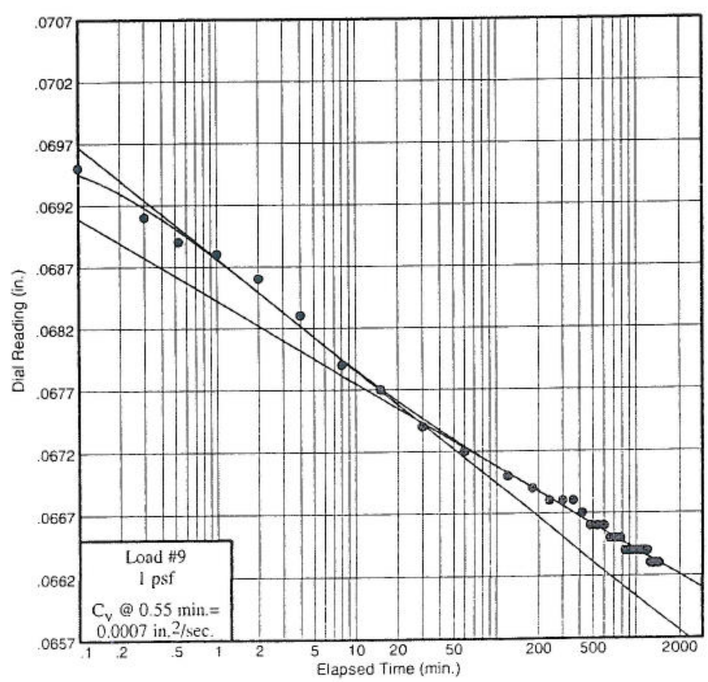
Project No.: 0121-3070.03
Project: SCI-823-0.00

Source: B-1108 Sample No.: P1 Elev./Depth: 10.0









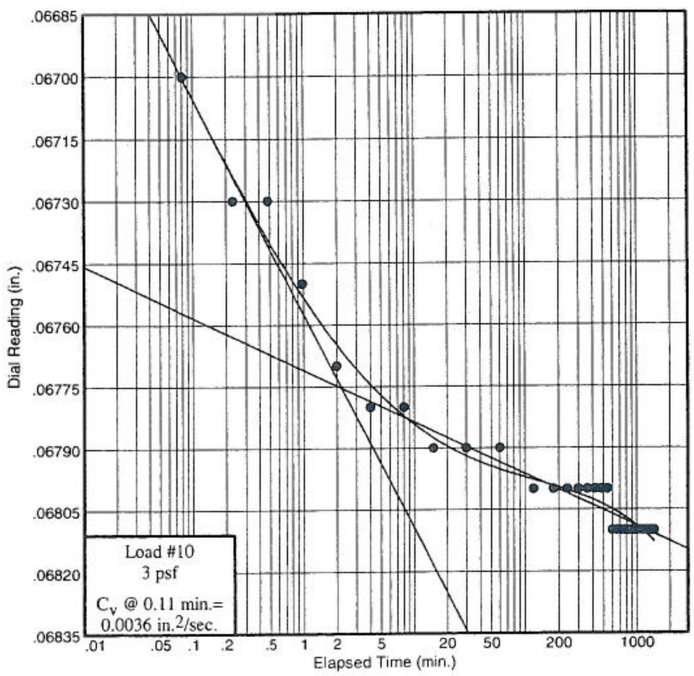
DLZ

Figure

Dial Reading vs. Time

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

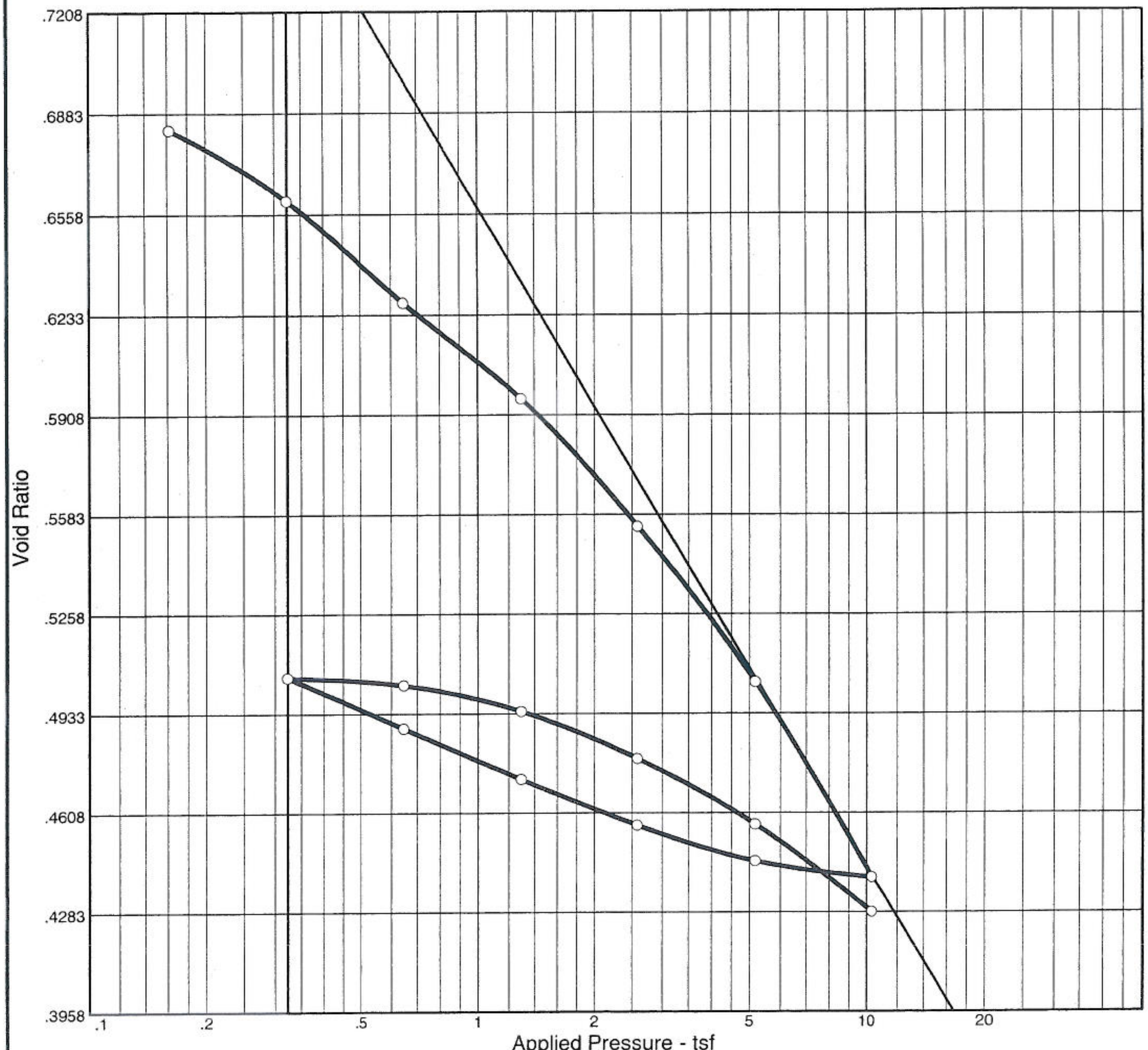
Source: B-1108 | Sample No.: P1 | Elev./Depth: 10.0



DLZ

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
104.7 %	29.1 %	95.0	38	19	2.64	CL	A-6(17)	0.734

MATERIAL DESCRIPTION

Silt Clay (A-6b)

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

Project: SCI-823-0.00

Source: B-1108 **Sample No.:** P3 **Elev./Depth:** 18.0

Remarks:

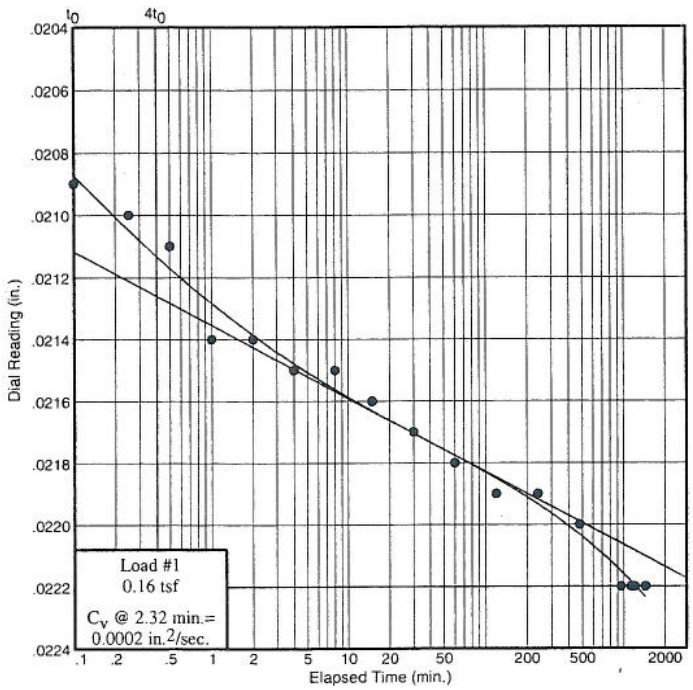
DLZ

Figure

Dial Reading vs. Time

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

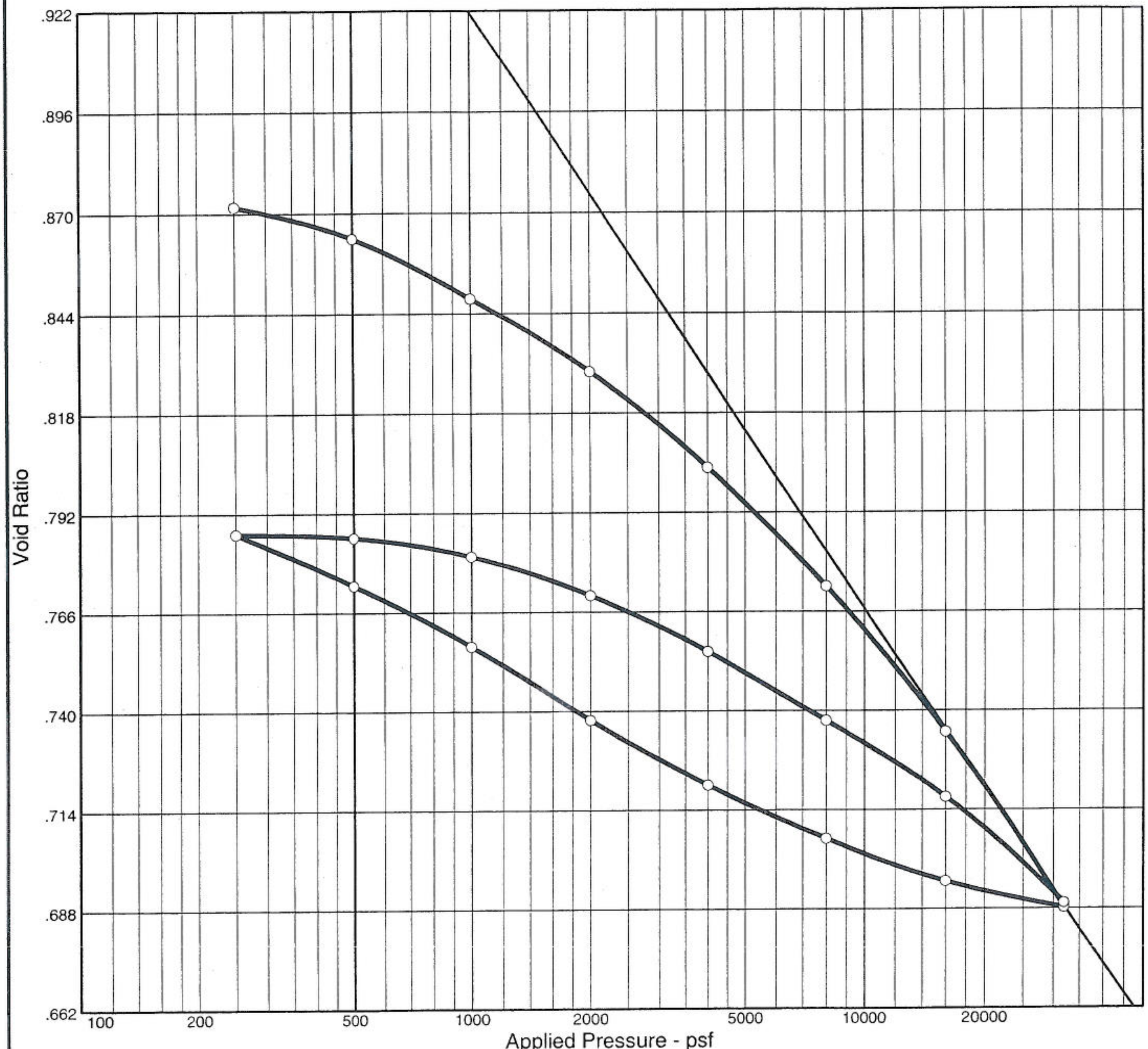
Source: B-1108 | Sample No.: P3 | Elev./Depth: 18.0



DLZ

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
95.5 %	30.9 %	90.5	57	33	2.73		A-7-6	0.883

MATERIAL DESCRIPTION

Project No. 0121-

Client: TranSystems, Inc.

Project: SCI-823-0.00

Source: B-1109A

Elev./Depth: 8.0

DLZ

Remarks:

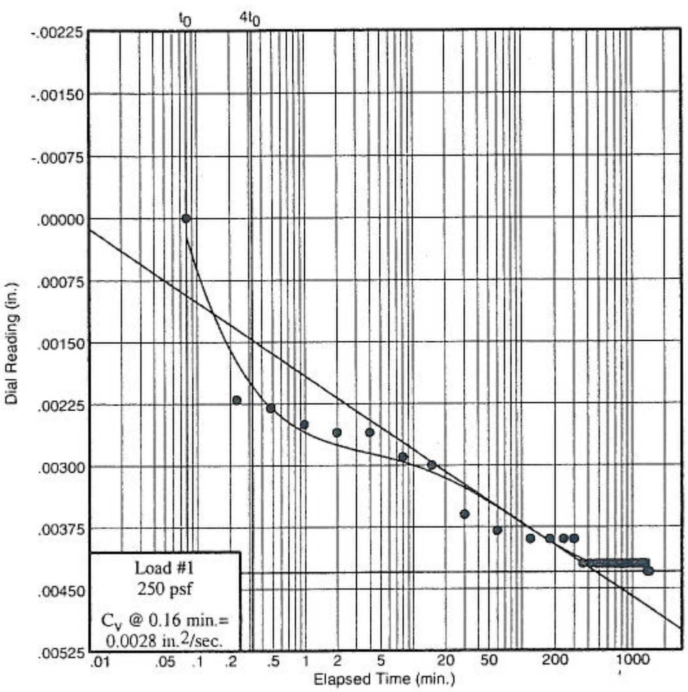
Figure

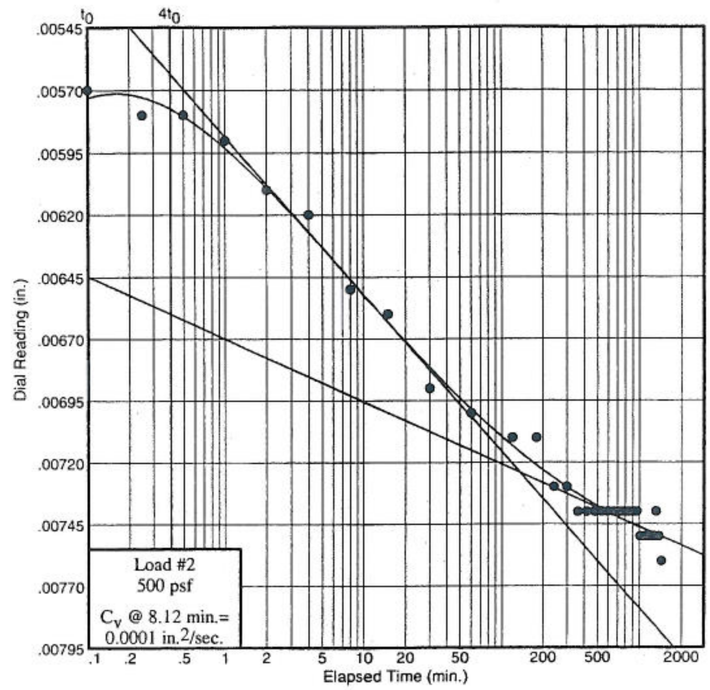
Dial Reading vs. Time

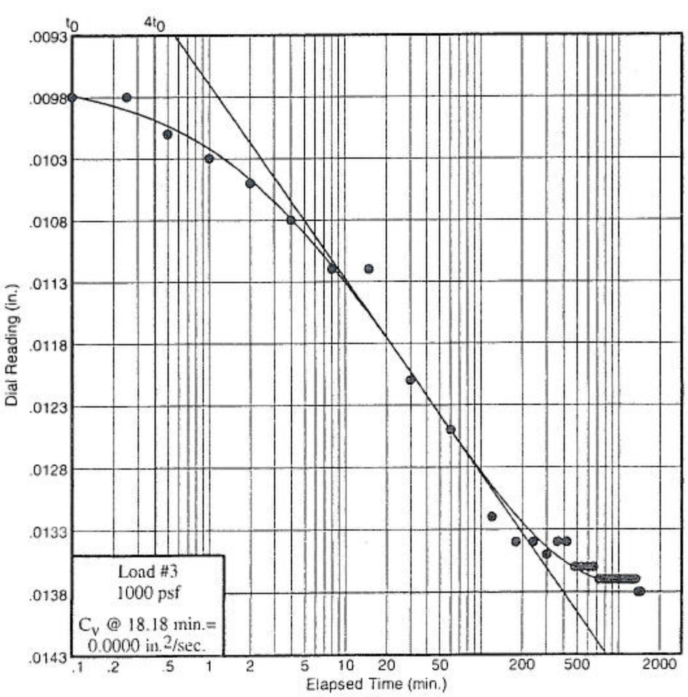
Project No.: 0121-3070.03
Project: SCI-823-0.00

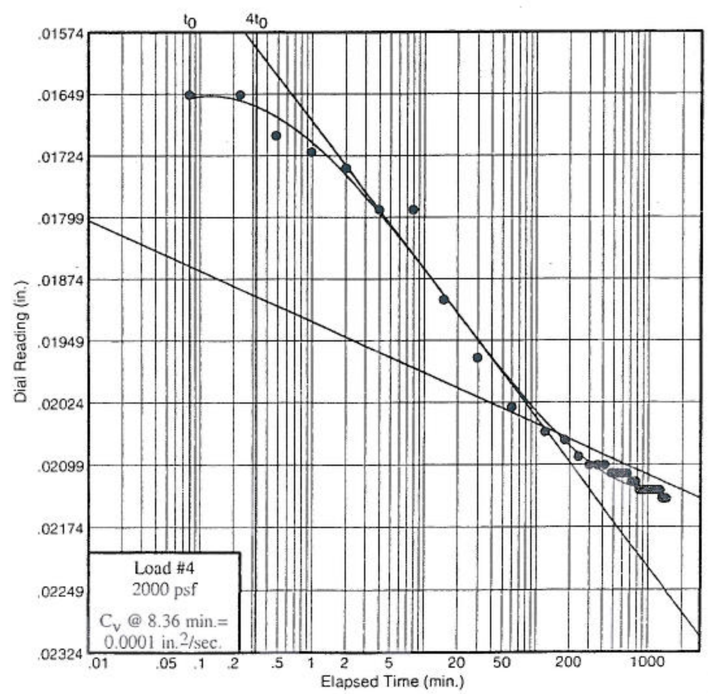
Source: B-1109A

Elev./Depth: 8.0









DLZ

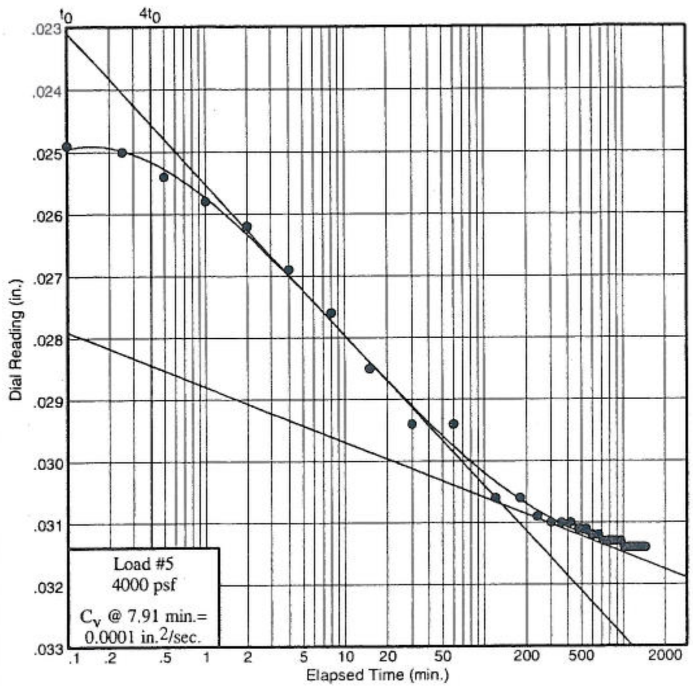
Figure

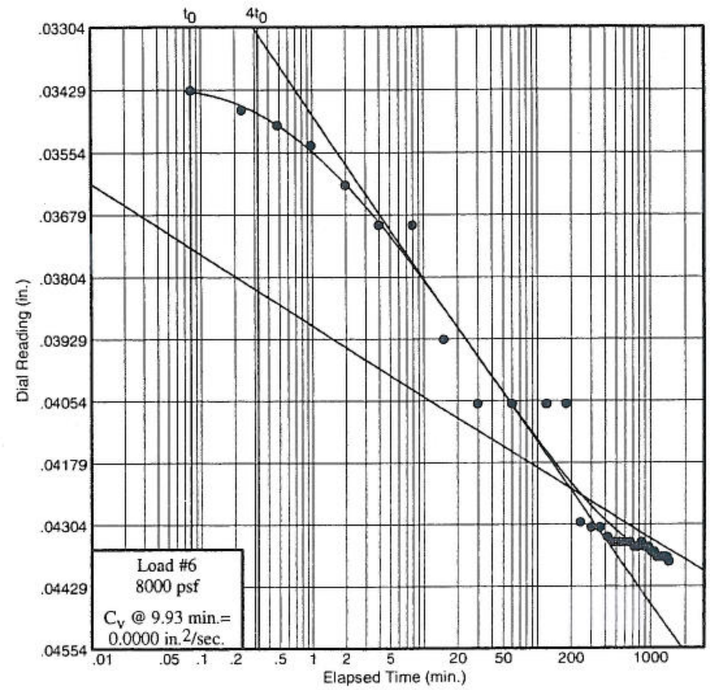
Dial Reading vs. Time

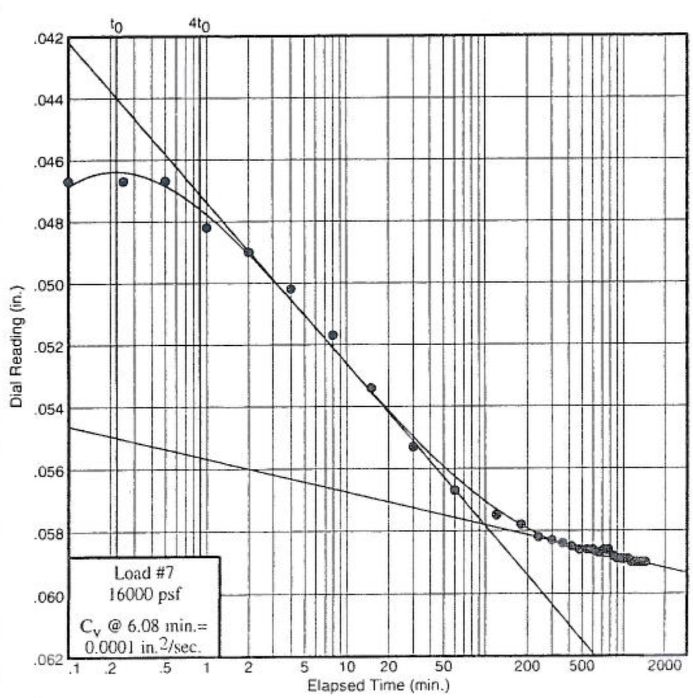
Project No.: 0121-3070.03
Project: SCI-823-0.00

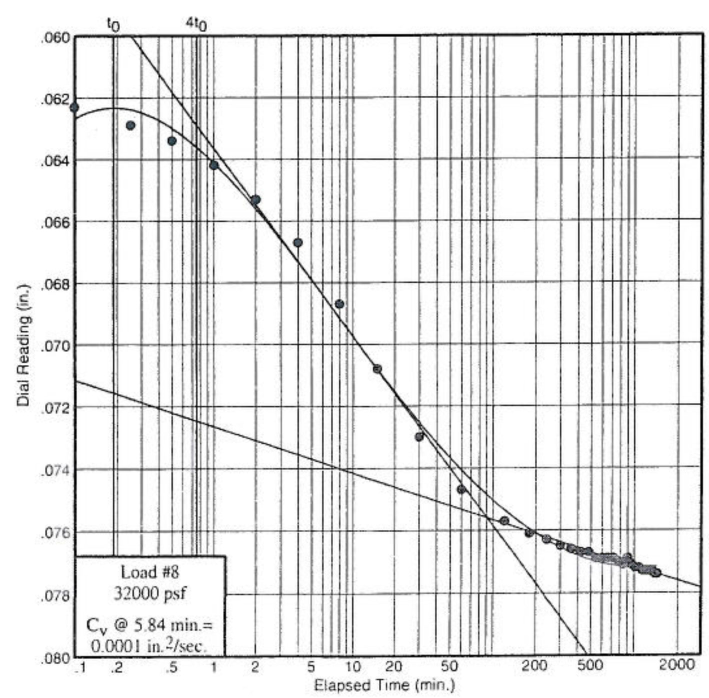
Source: B-1109A

Elev./Depth: 8.0









DLZ

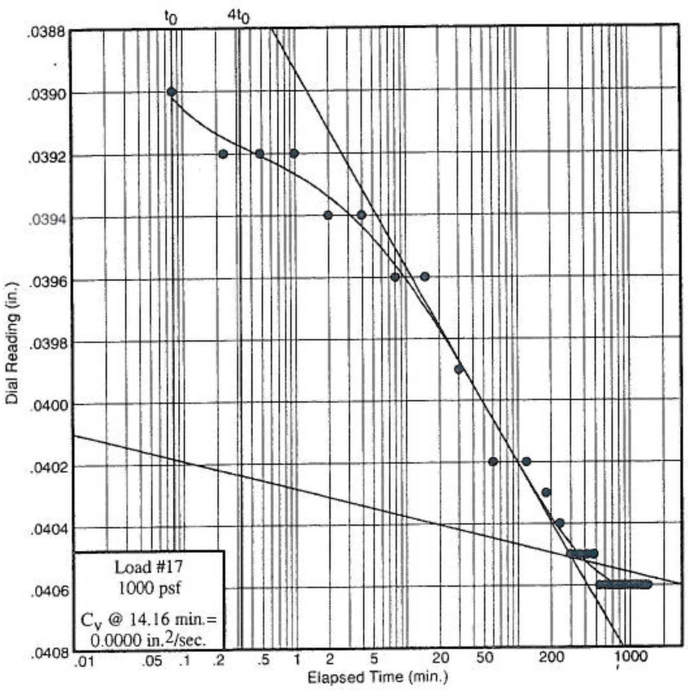
Figure

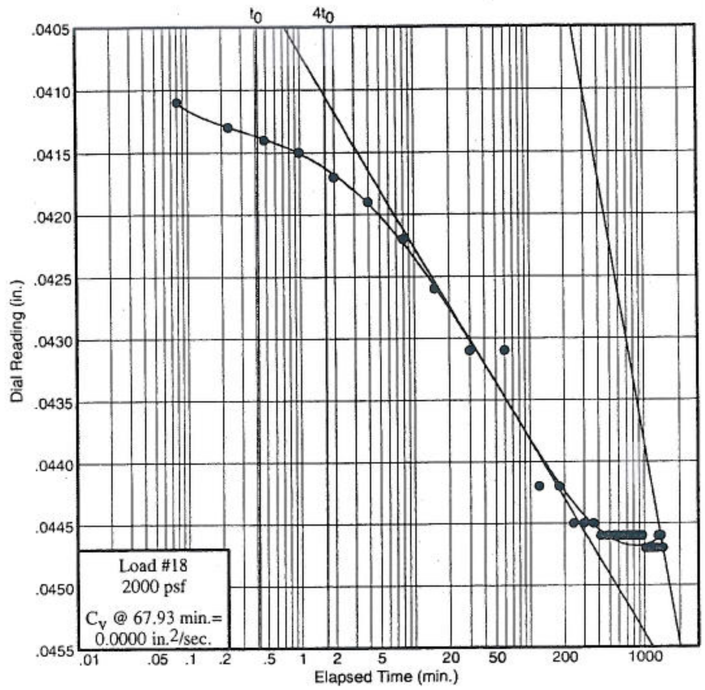
Dial Reading vs. Time

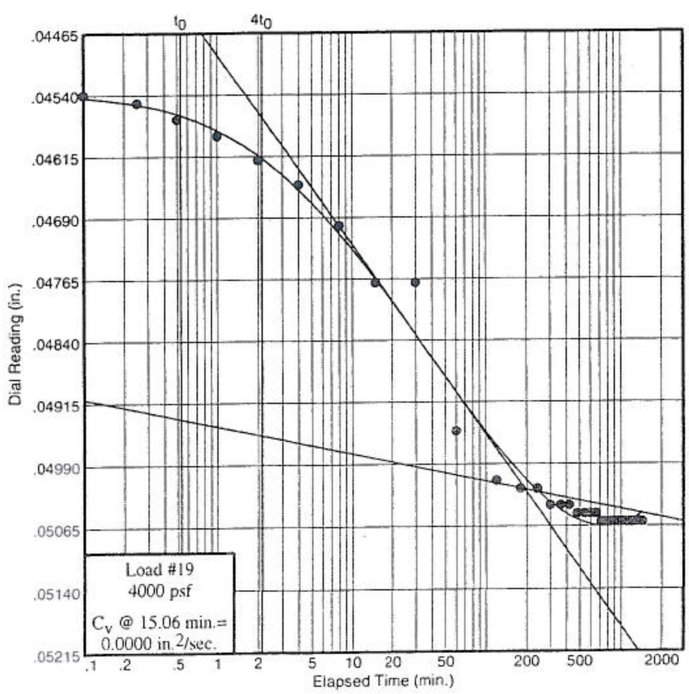
Project No.: 0121-3070.03
Project: SCI-823-0.00

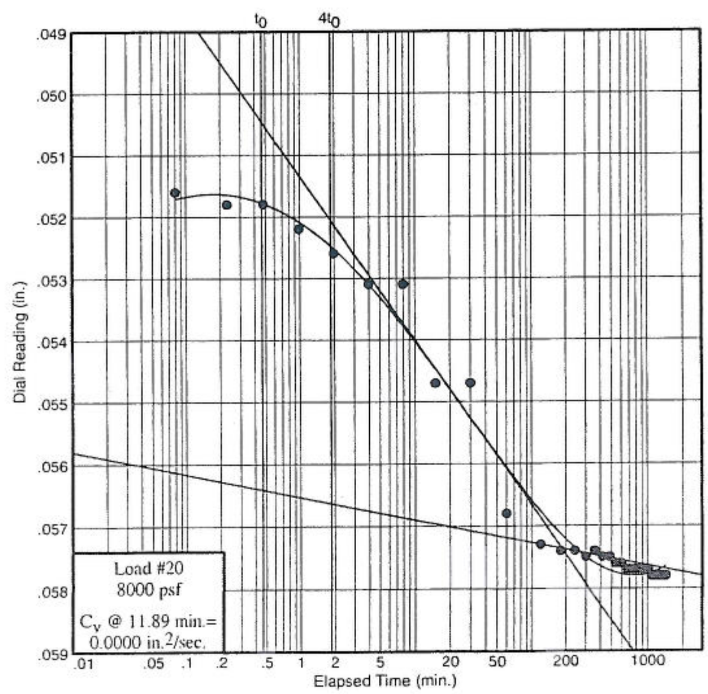
Source: B-1109A

Elev./Depth: 8.0









DLZ

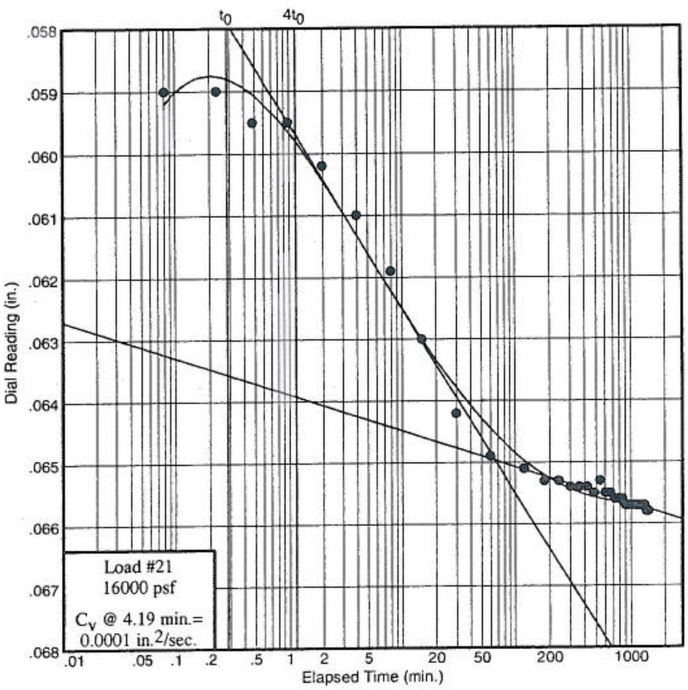
Figure

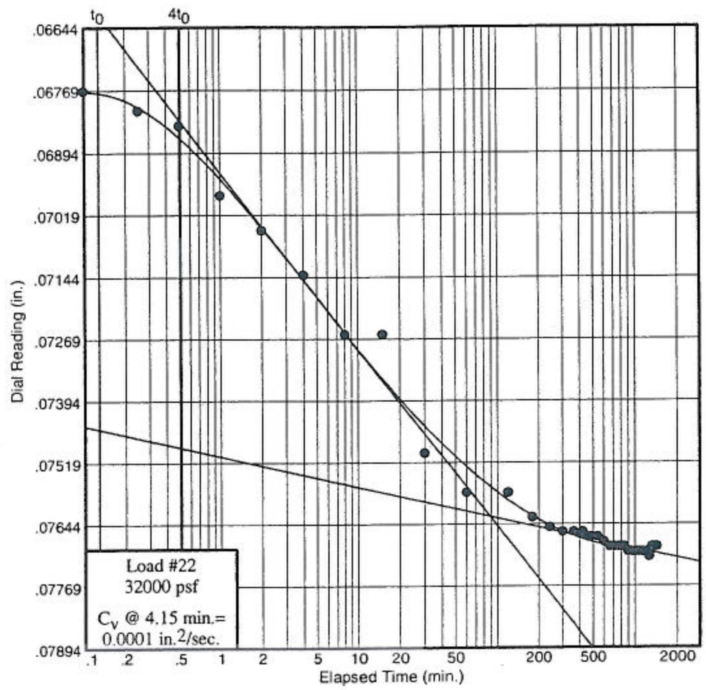
Dial Reading vs. Time

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

Source: B-1109A

Elev./Depth: 8.0

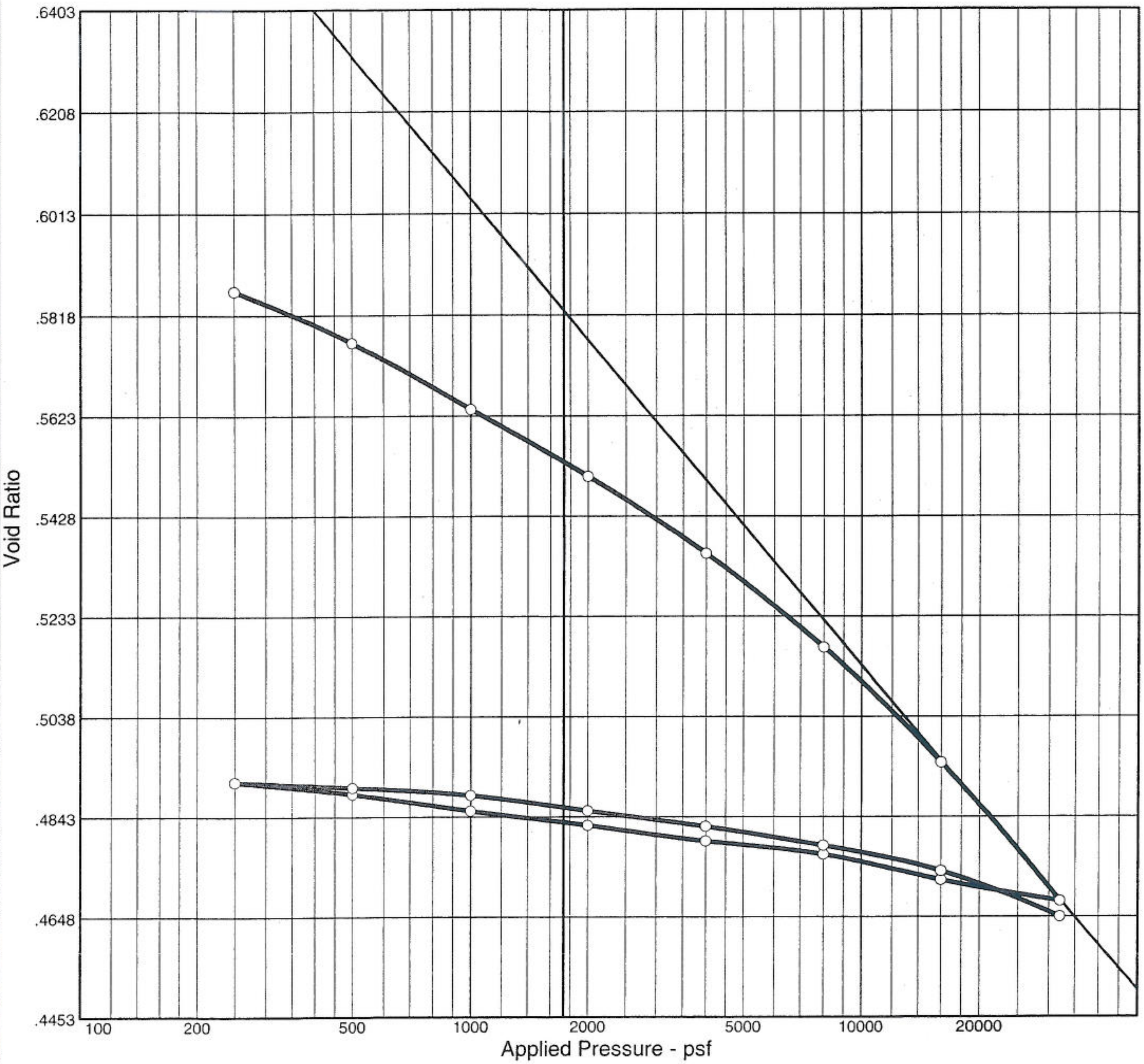




DLZ

Figure

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	USCS	AASHTO	Initial Void Ratio
Saturation	Moisture							
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

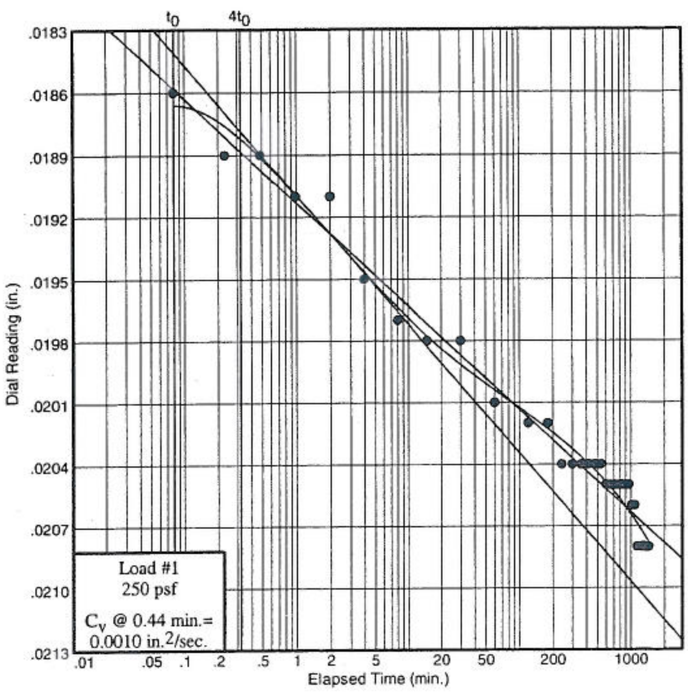
Remarks:

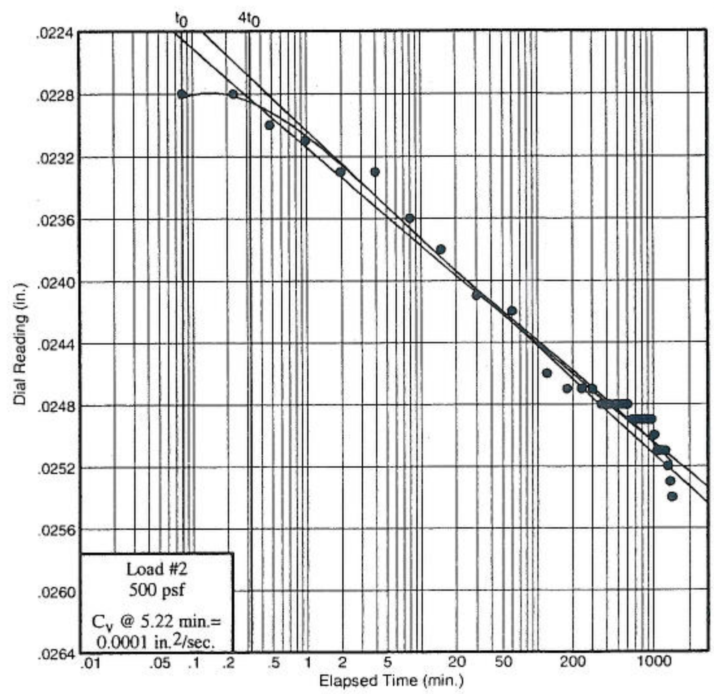
Figure

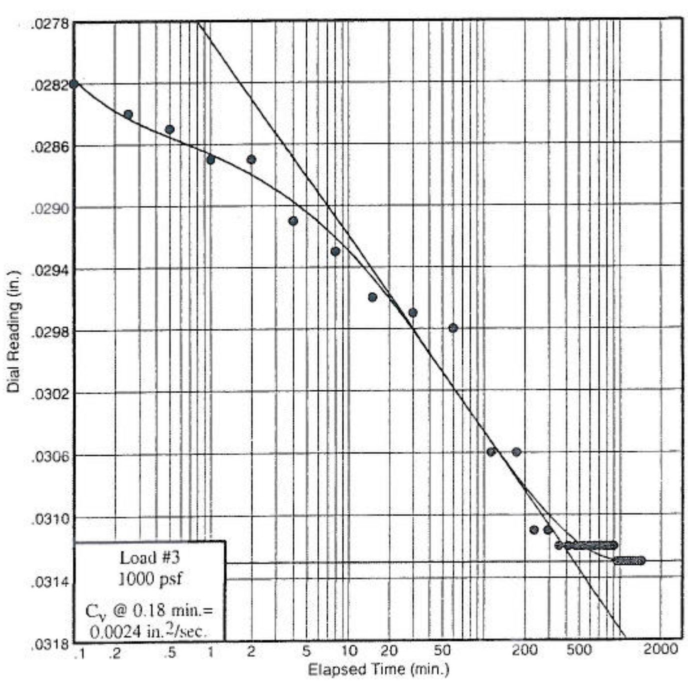
Dial Reading vs. Time

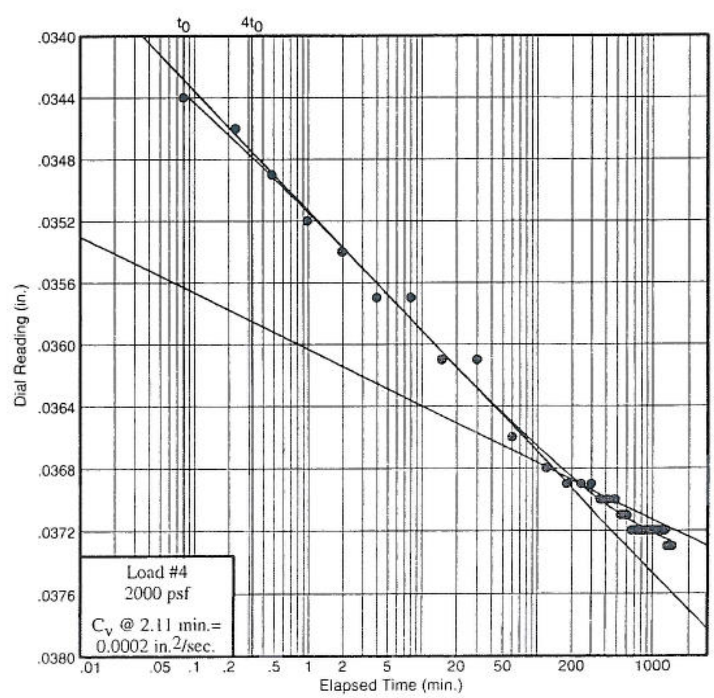
Project No.: 0121-3070.03
Project: SCI-823-0.00

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









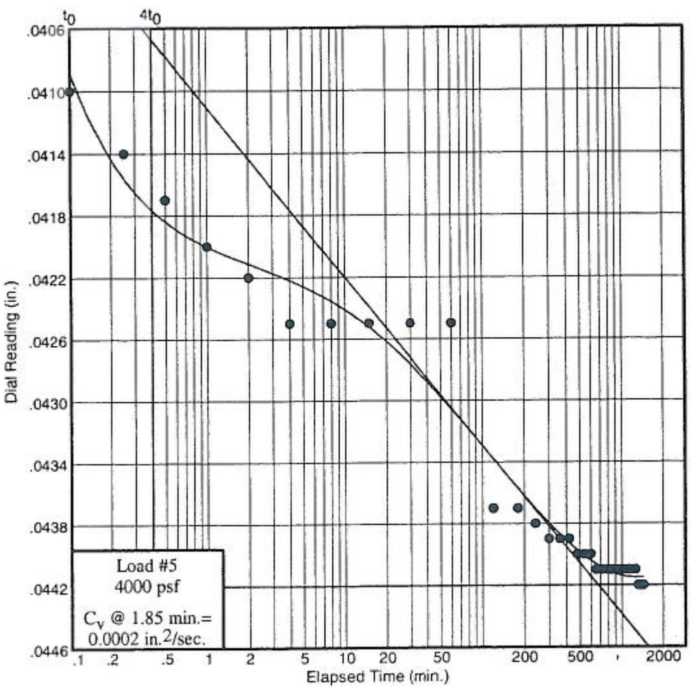
DLZ

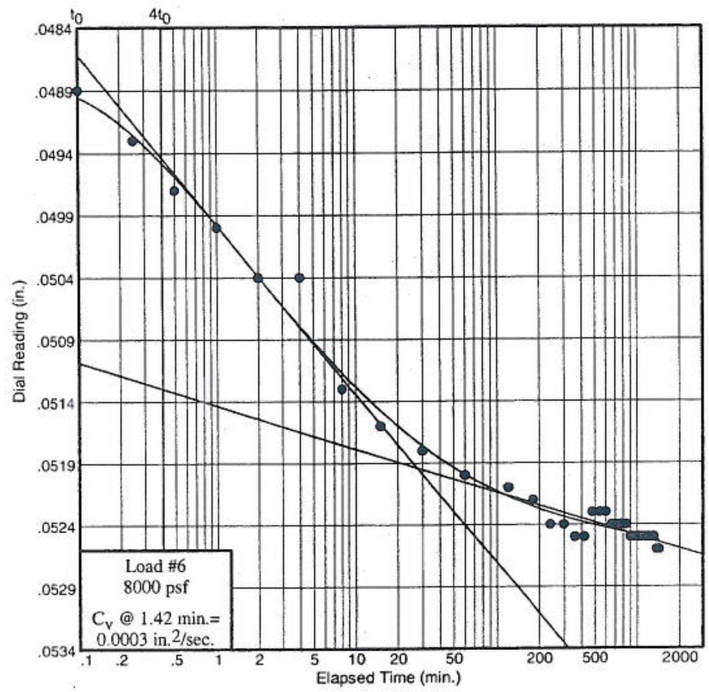
Figure

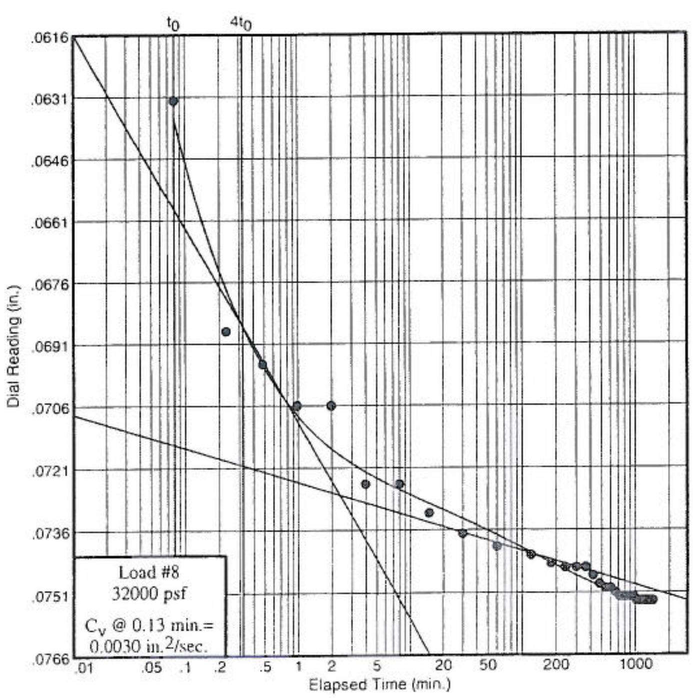
Dial Reading vs. Time

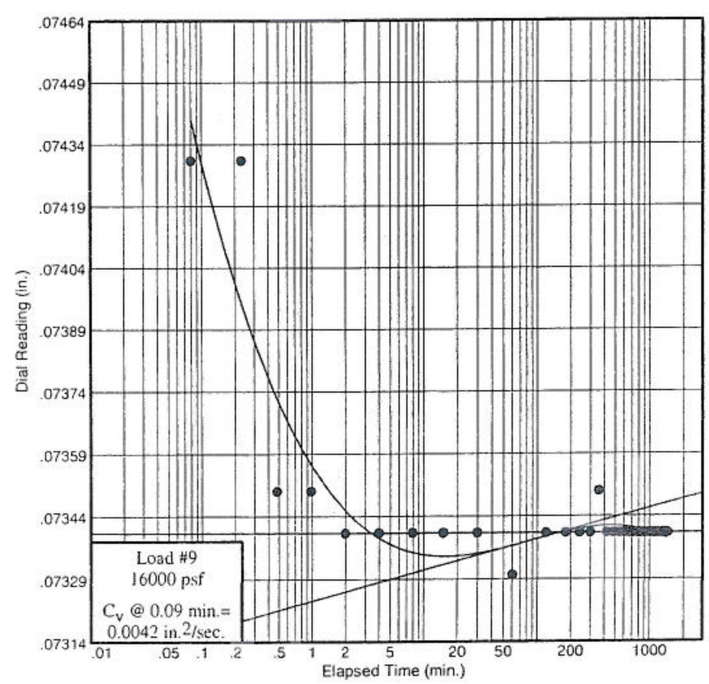
Project No.: 0121-3070.03
Project: SCI-823-0.00

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









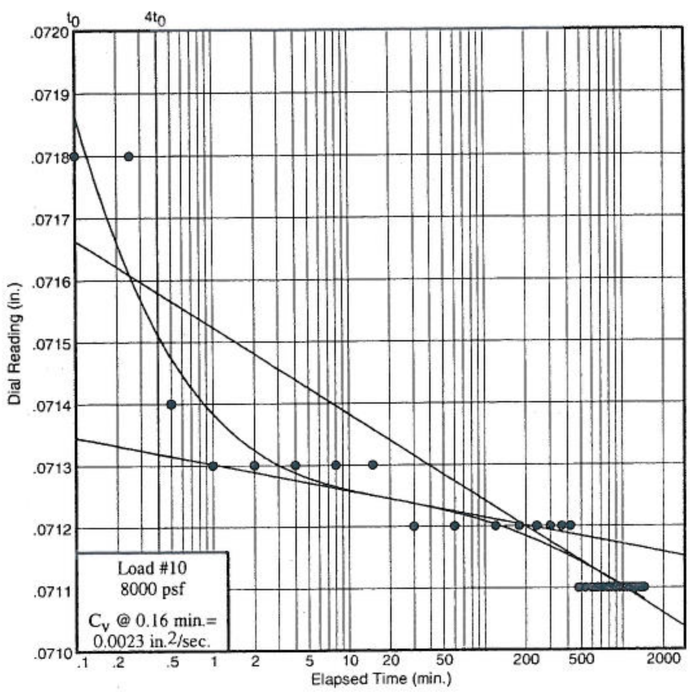
DLZ

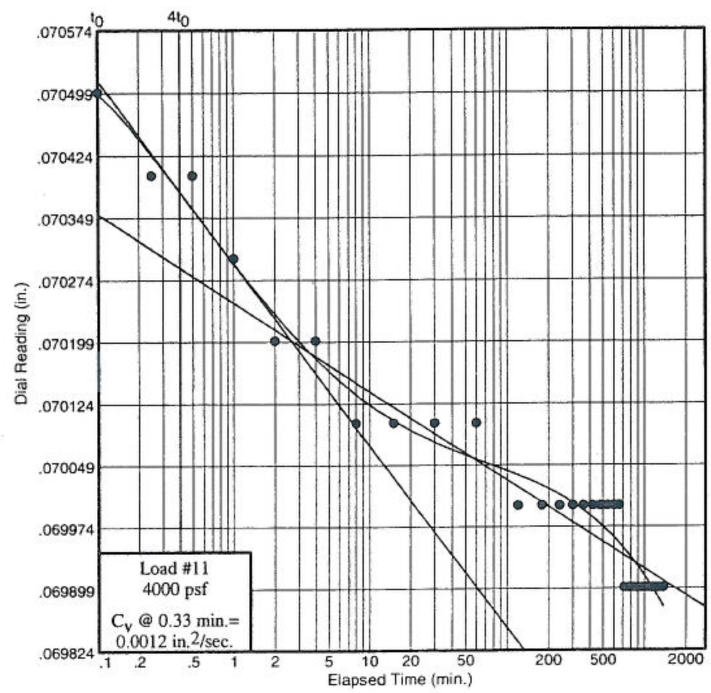
Figure

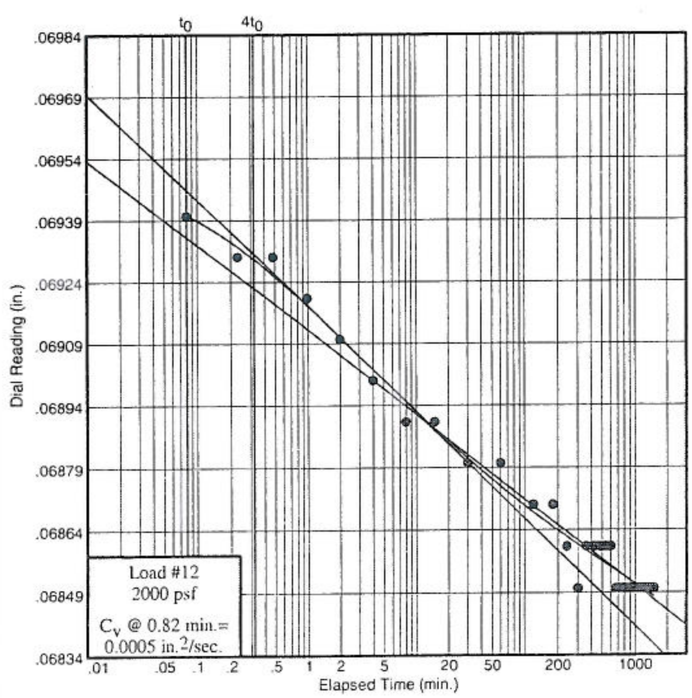
Dial Reading vs. Time

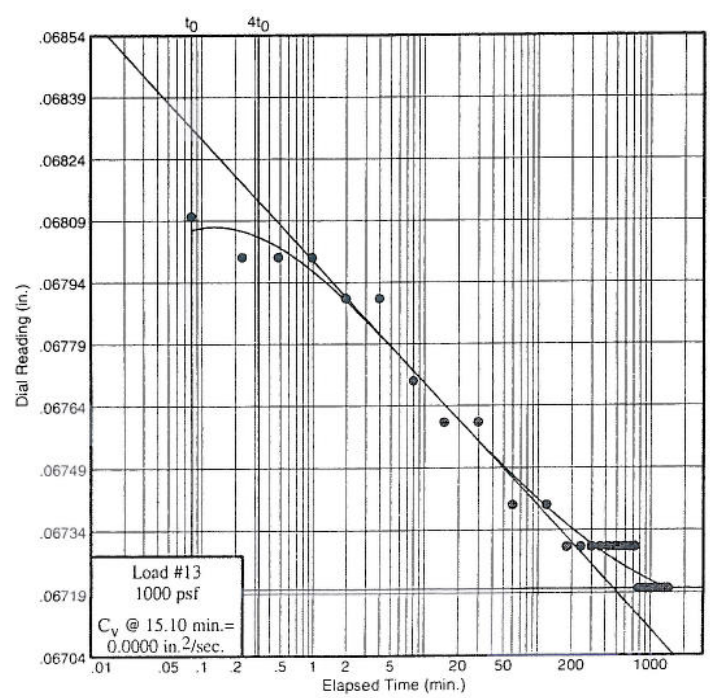
Project No.: 0121-3070.03
Project: SCI-823-0.00

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





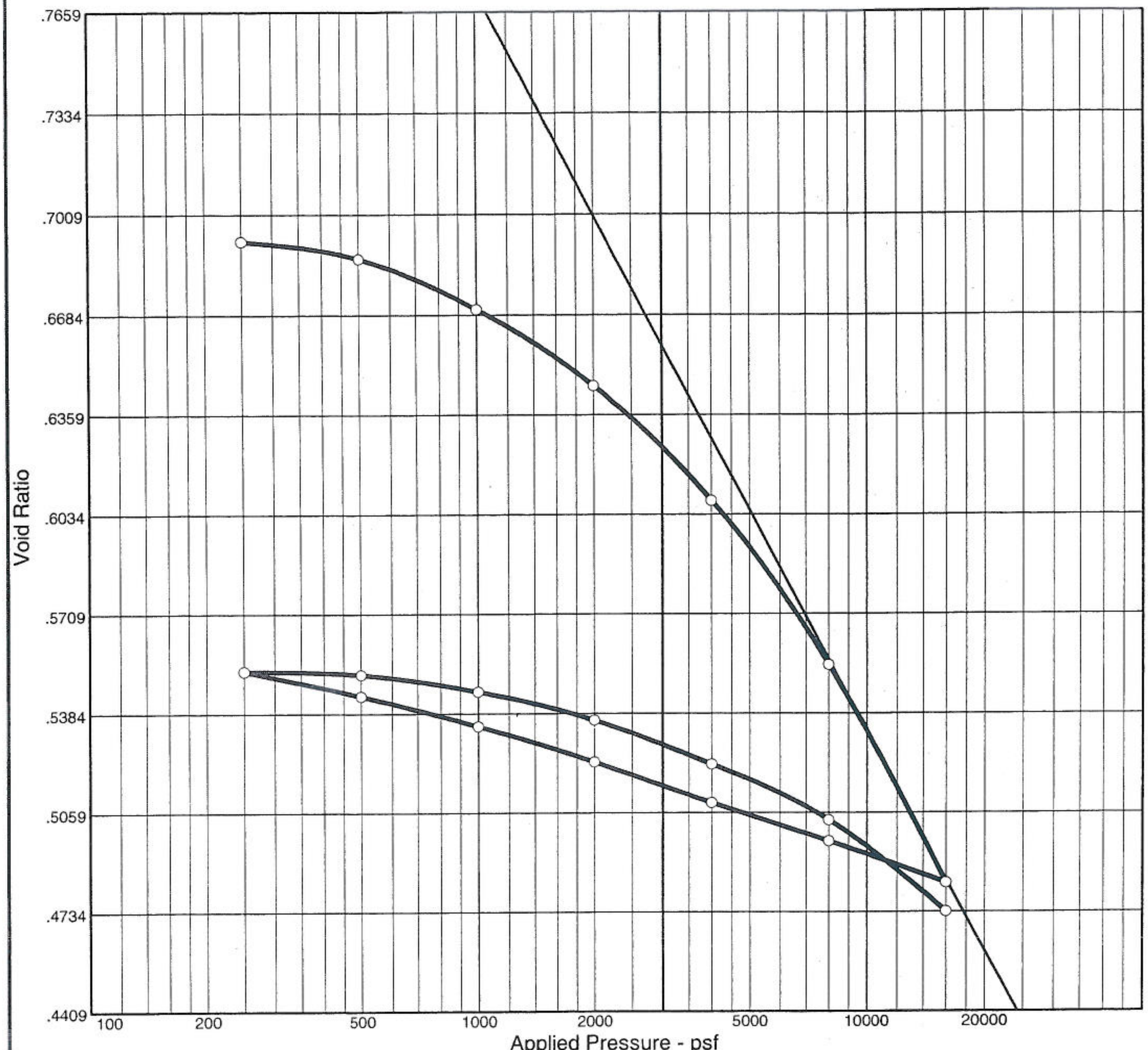




DLZ

Figure

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.
Project: SCI-823-0.00

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

DLZ

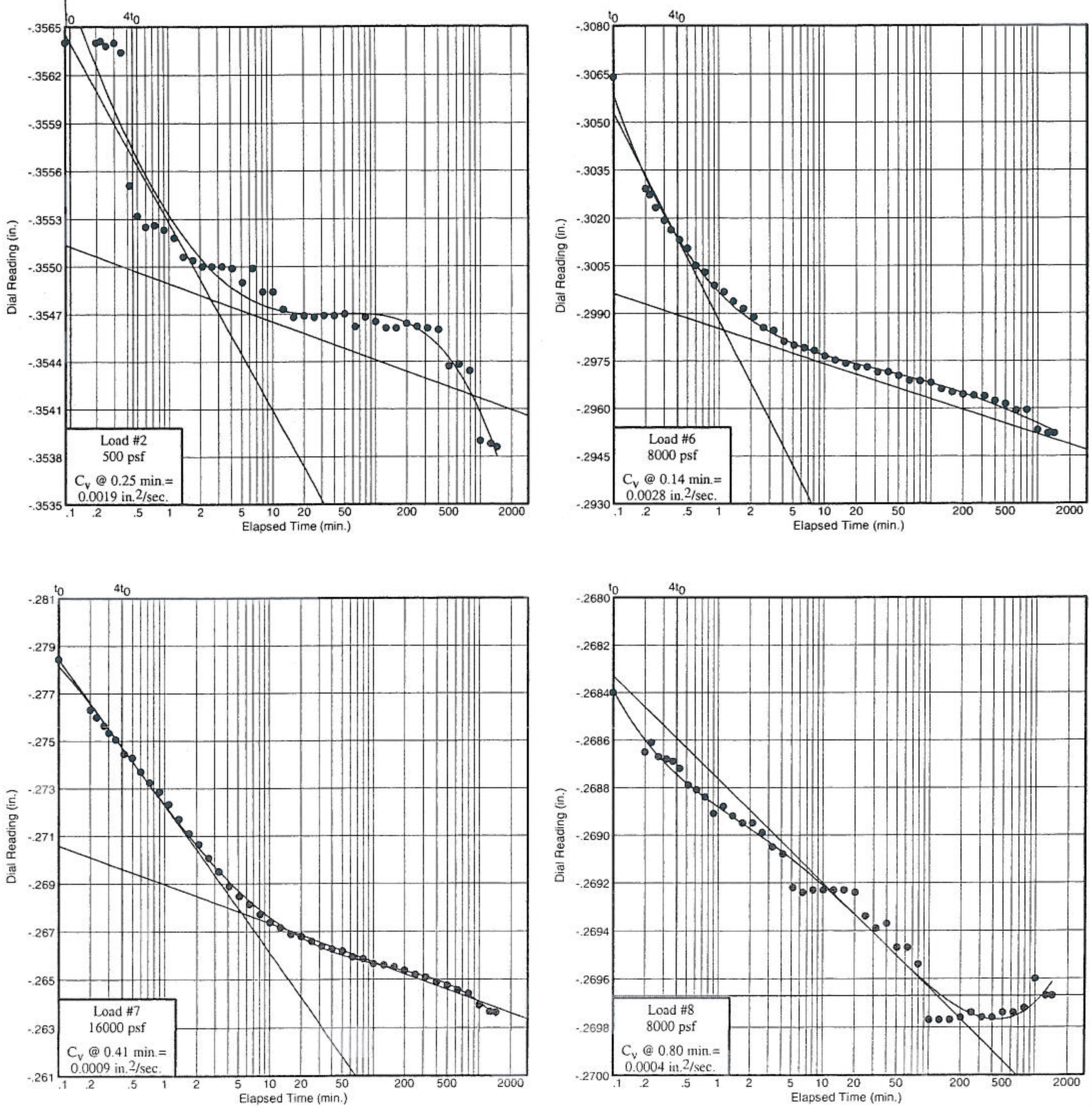
Remarks:

Figure

Dial Reading vs. Time

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

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



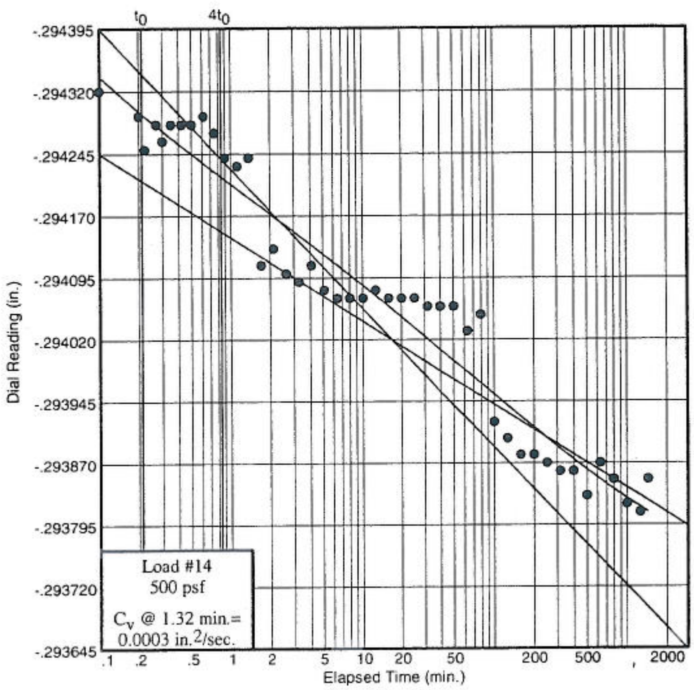
DLZ

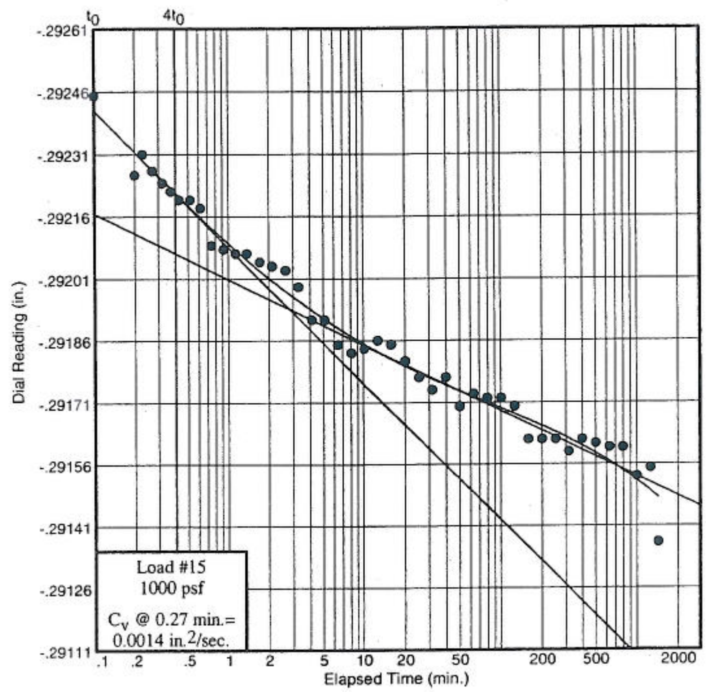
Figure

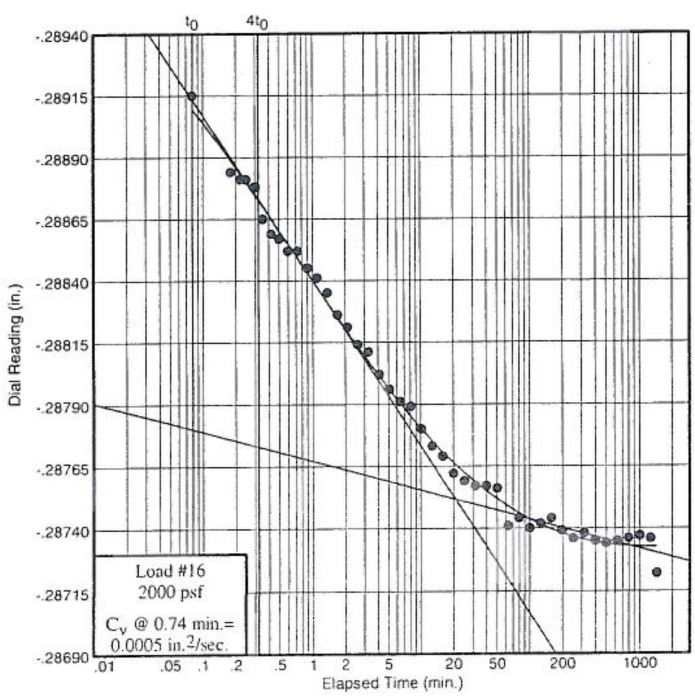
Dial Reading vs. Time

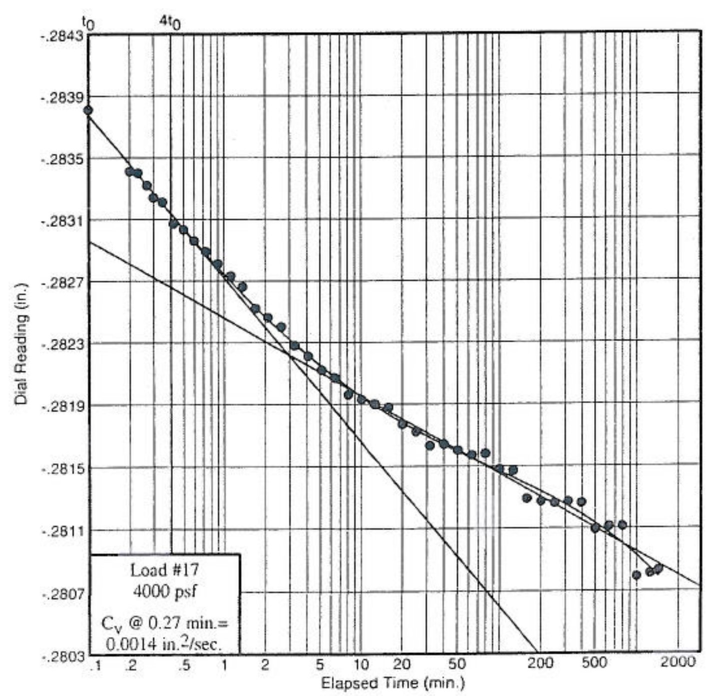
Project No.: 0121-3070.03
Project: SCI-823-0.00

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









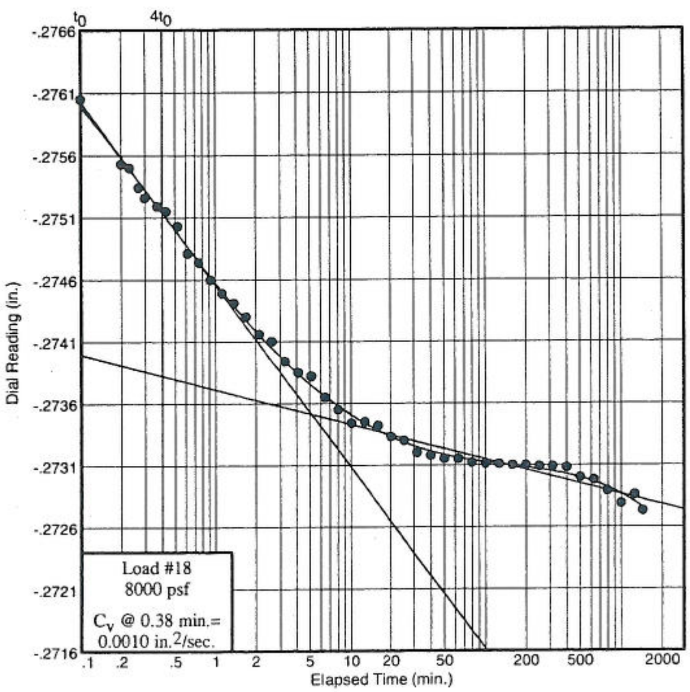
DLZ

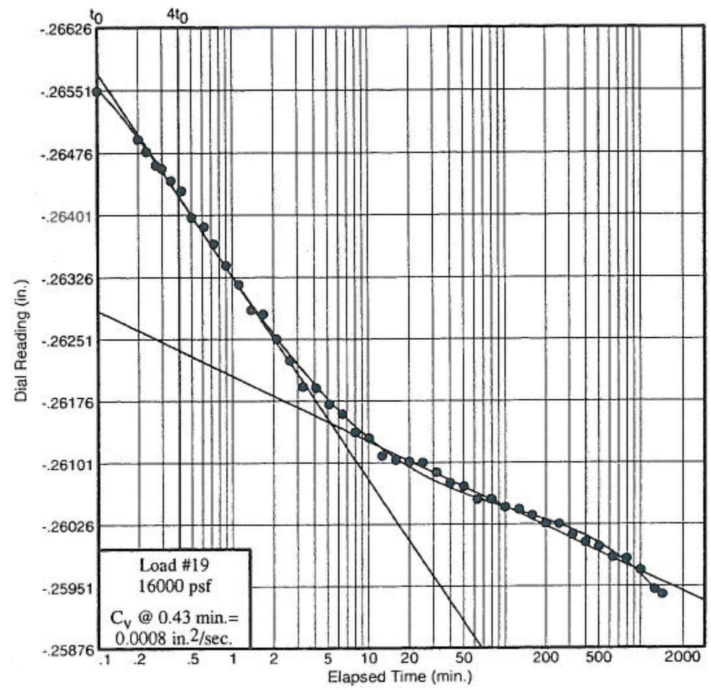
Figure

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

APPENDIX IV

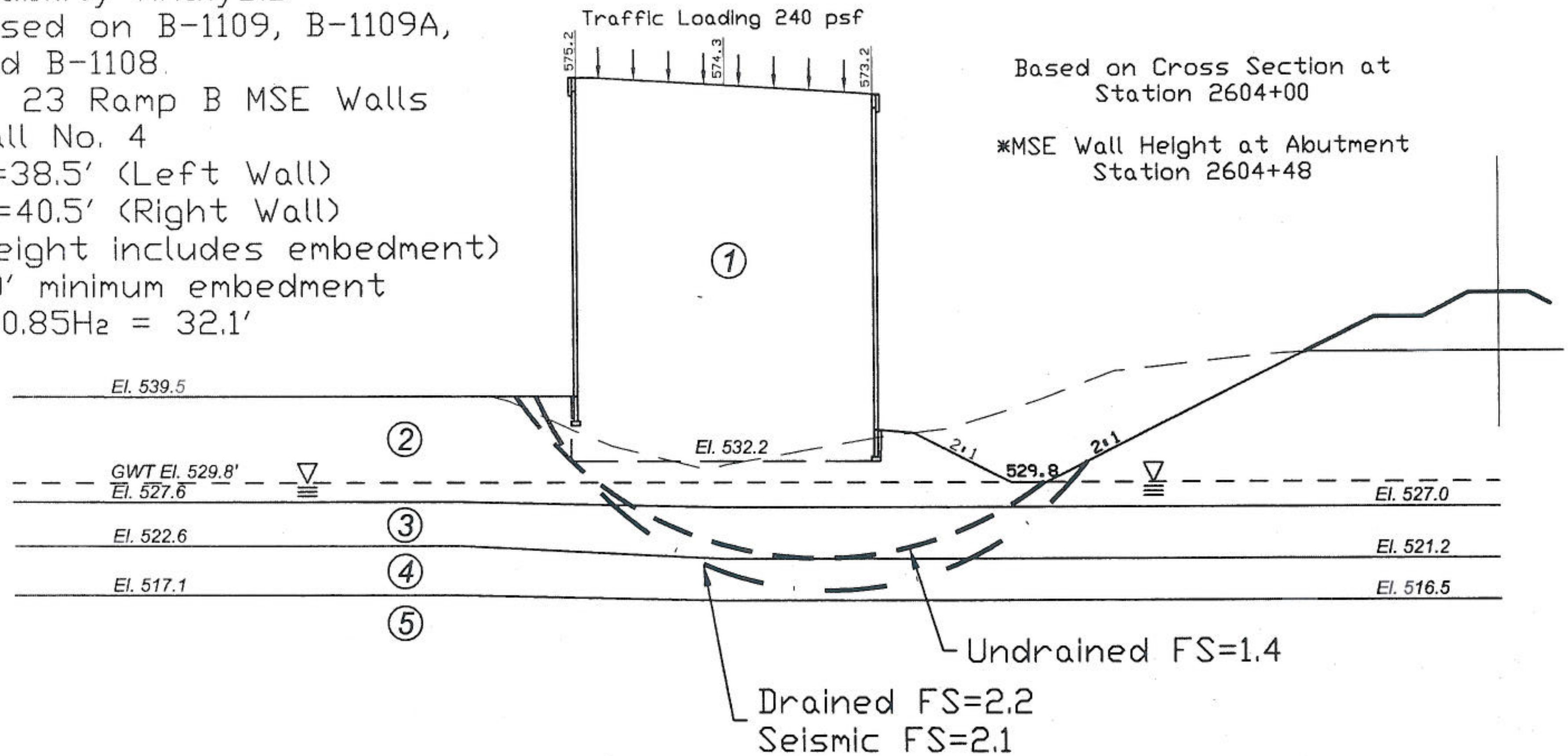
MSE Wall Bearing Capacity and Stability Calculations
MSE Wall Global Stability Analysis Results
MSE Wall Settlement Calculations
Time-Rate of Consolidation Calculations

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	Possible Fill	A-4a/A-6a	2000	0	0	29	120
Material 3	Medium Stiff	A-7-6	900	0	0	28	120
Material 4	Loose	A-4a/A-4b	0	29	0	29	120
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-1109, B-1109A, and B-1108
US 23 Ramp B MSE Walls
Wall No. 4
H_1=38.5′ (Left Wall)
H_2=40.5′ (Right Wall)
(Height includes embedment)
3.0′ minimum embedment
$L=0.85H_2$ = 32.1′

Based on Cross Section at Station 2604+00

*MSE Wall Height at Abutment Station 2604+48



US-23 Interchange
Ramp B Wall No. 4, Station 2604+48
Based on Borings B-1109, B-1109A, &B-1108

MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 10/01/07

Stability Analyses performed using UTEXAS3 Version 1.201

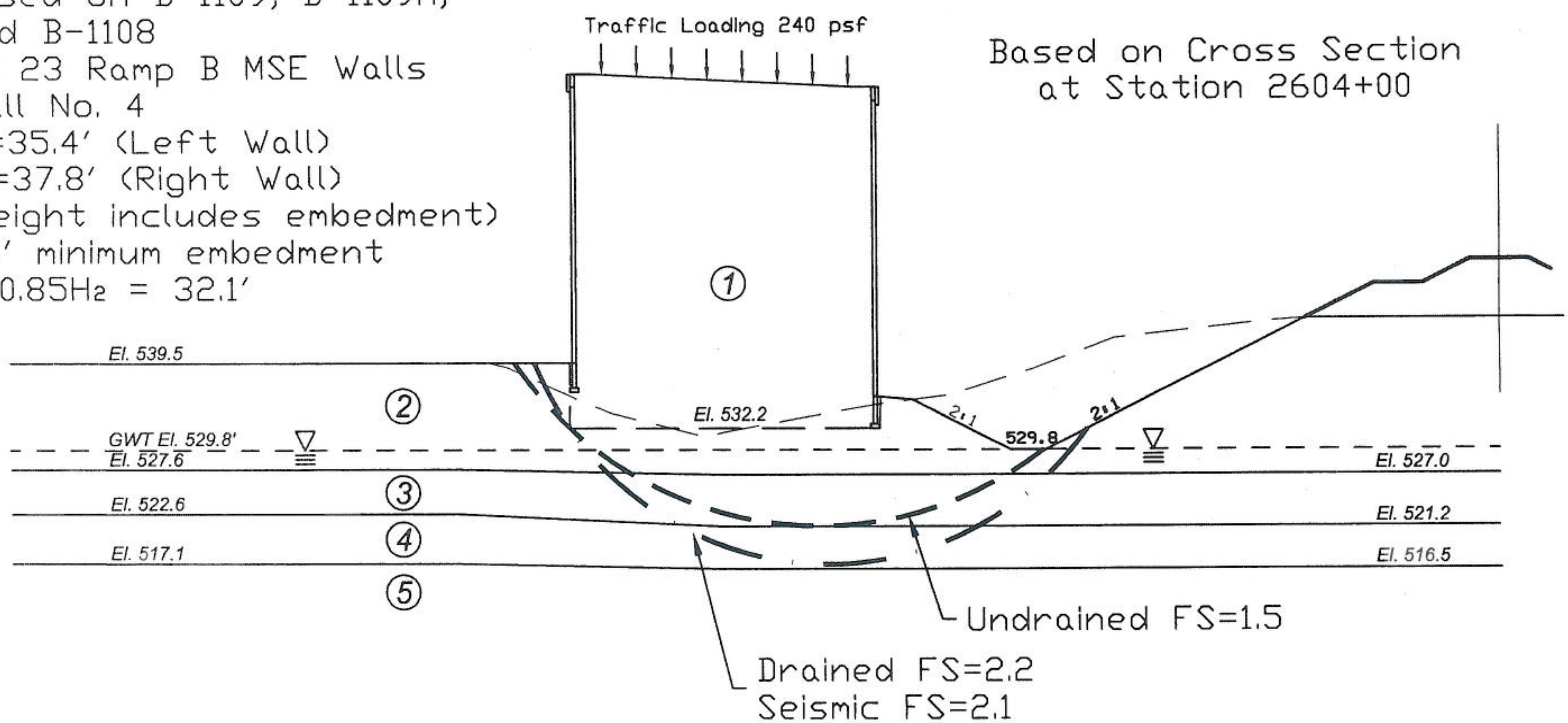
Sheet 1 of 42

M:\proj\0121\3070.03\Interchanges\US 23\embankment stability.dwg, 10/16/2007 11:49:56 AM, Xerox WorkCentre Pro 245 PS

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	Possible Fill	A-4a/A-6a	2000	0	0	29	120
Material 3	Medium Stiff	A-7-6	900	0	0	28	120
Material 4	Loose	A-4a/A-4b	0	29	0	29	120
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-1109, B-1109A, and B-1108
US 23 Ramp B MSE Walls
Wall No. 4
H_1=35.4' (Left Wall)
H_2=37.8' (Right Wall)
(Height includes embedment)
3.0' minimum embedment
L=0.85H_2 = 32.1'

Based on Cross Section at Station 2604+00



Stability Analyses performed using UTEXAS3 Version 1.201

US-23 Interchange Ramp B Wall No. 4, Station 2604+00 Based on Borings B-1109, B-1109A, &B-1108		
MSE GLOBAL STABILITY ANALYSIS SCI-823-0.00		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 9/12/07

M:\proj\0121\3070.03\Interchanges\US 23\embankment stability.dwg, 10/16/2007 11:49:43 AM, Xerox WorkCentre Pro 245 PS

Sheet 2 of 42

Mechanically Stabilized Earth Walls and Reinforced Slopes
Design and Const. Guidelines, FHWA-NHI-00-043

5.4 BACK-TO-BACK WALLS

For walls which are built back-to-back as shown in figure 50, a modified value of backfill thrust influences the external stability calculations. As indicated in figure 50, two cases can be considered.

- For Case I, the overall base width is large enough so that each wall behaves and can be designed independently. In particular, there is no overlapping of the reinforcements. Theoretically, if the distance, D, between the two walls is shorter than:

$$D = H_1 \tan(45° - \varphi/2) \tag{55}$$

then the active wedges at the back of each wall cannot fully spread out and the active thrust is reduced. However, it is assumed that for values of:

$$D > H_1 \tan(45° - \varphi/2) \approx 0.5\, H_1 \tag{56}$$

full active thrust is mobilized.

- For Case II, there is an overlapping of the reinforcements such that the two walls interact. When the overlap, L_R, is greater than 0.3 H_2, where H_2 is the shorter of the parallel walls, no active earth thrust from the backfill needs to be considered for external stability calculations. For intermediate geometries between Case I and Case II, the active earth thrust may be linearly interpolated from the full active case to zero. For Case II geometries with overlaps greater than 0.3 H_2, L/H ratios for each wall as low as 0.6 may be considered.

Considering this case, designers might be tempted to use single reinforcements connected to both wall facings. This alternative completely changes the strain patterns in the structure and results in higher reinforcement tensions such that the design method in this manual is no longer applicable. In addition, difficulties in maintaining wall alignment could be encountered during construction, especially when the walls are not in a tangent section.

Based on a performance review, back-to-back walls with overlapping reinforcements may be designed for static load conditions with a distance between parallel facing as low as L/H = 0.6, where H is the height of each wall, and for conditions where the seismic horizontal accelerations at the foundation level is less than 0.05g. For walls in more seismically active areas (up to 0.19g) a distance of 1.1H_1 is presently recommended. For walls subjected to significant seismic loading (up to 0.40g) successful performance has been observed when the distance between parallel facings was at least 1.2H_1.

Justification of narrower back-to-back distances (< 1.1H_1) between faces in seismically active areas require a more detailed analysis be performed to include effects of potential non-uniform distribution of seismic and inertial forces within the wall, as suggested by numerical studies and not provided for in the present design methodology.

DLZ

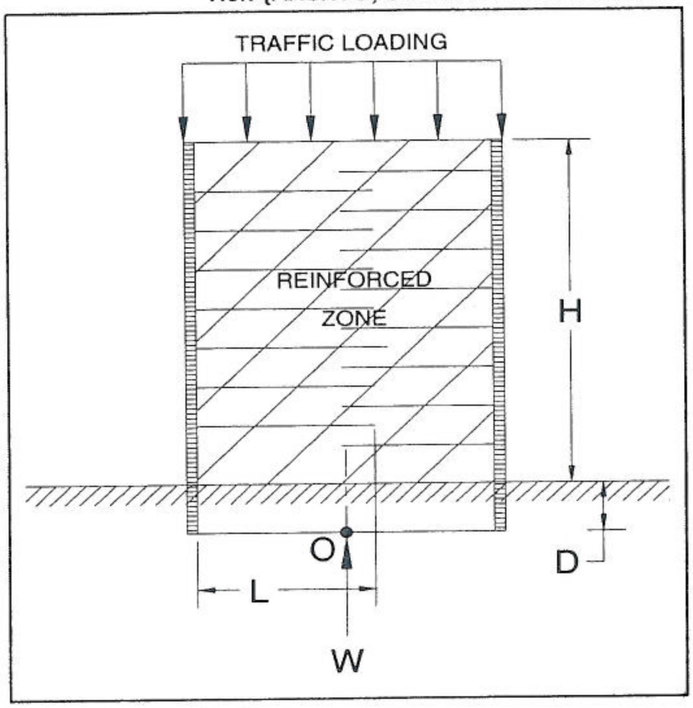
SUBJECT			
Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	4 OF 42
Item	MSE Wall Bearing Capacity	COMP. BY	SJK DATE 10-10-07
Wall No 4, Station 2604+00, BACK-TO-BACK WALLS		CHECKED BY	DAA DATE 10-17-07

Ka=0.0, for soil reinforcement overlap greater than 0.3*H

* Full height L is limited by Ramp width.

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	=	900	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	32.13	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	37.8	ft	
H	=	34.8	ft	Height of wall
Ka	**=**	**0.00**		
Γ Pa	=	12.6	ft	Moment arm
Γ Wt	=	18.9	ft	Moment arm
B'	=	32.13	ft	
γ'	=	57.6	pcf	
W_t		7,711	lb/ft of wall	Weight from traffic
W_{mse}	=	145,742	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,776 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} = 4,799 psf

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

q_{ALL} = 1,920 psf

Factor of Safety = **1.00** **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} = 18,015 psf

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

q_{ALL} = 7,206 psf

Factor of Safety = **3.77** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 0.00 ft

Kern

e < L/6 = 5.36 ft

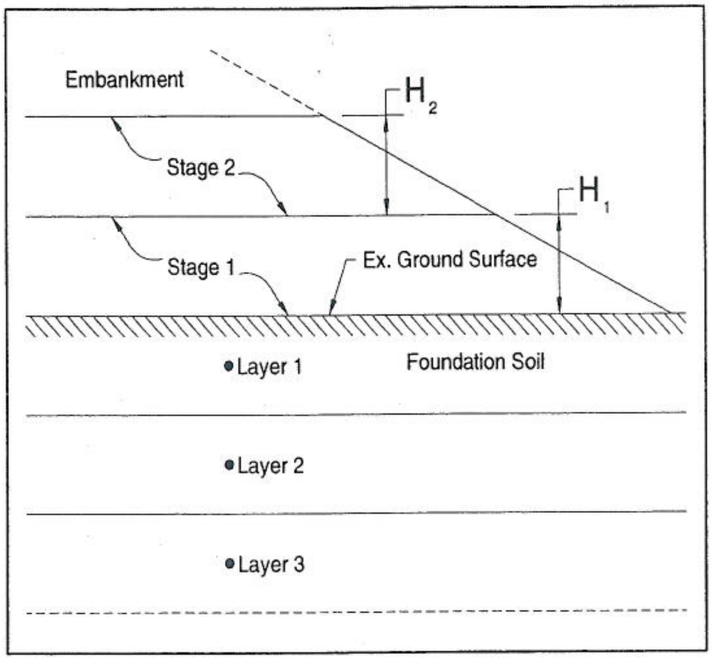
DLZ SUBJECT

Client	CH2M Hill	JOB NUMBER	0121-3070.03		
Project	SCI-823 Portsmouth Bypass	SHEET NO.	5	OF	42
Item	Undrained Strength Analysis - Staged Const.	COMP. BY	SAK	DATE	10-10-07
MSE wall No. 4, US 23 Ramp B, Sta 2604+00		CHECKED BY	DAA	DATE	10-17-07

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

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

Stage 1 Embankment First Stage Embankment Height H_1= **13.0** 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
3.0-7.2	A-6a	2000	1404	20.0	511	2511	26%
7.2-14.3	A-7-6	900	1404	21.3	547	1447	61%

Stage 2 Embankment Second Stage Embankment Height H_2= **11.0** Average Percent Consolidation U= **90%**

3.0-7.2	A-6a	2511	1188	20.0	432	2943	17%
7.2-14.3	A-7-6	1447	1188	21.3	463	1910	32%

Stage 3 Embankment Third Stage Embankment Height H_3= **8.0** Average Percent Consolidation U= **90%**

3.0-7.2	A-6a	2943	864	20.0	314	3257	11%
7.2-14.3	A-7-6	1910	864	21.3	337	2247	18%

The fill may be placed up to the finisehd grade after consolidation under the stage 3 embankment/MSE wall

H_4=5.5' For highest wall section at the abutment MSE wall

DLZ

ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT CH2M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT US 23 Ramp B, MSE Wall No. 4
Staged Construction Details

PROJECT NO. 0121-3070.03
SHEET NO. 6 OF 42
COMP. BY SJK DATE 10-15-0
CHECKED BY DAA DATE 10-17-0

* Based on bearing capacity calculations, staged construction is required for Wall No. 4, Ramp B.

- Height of 1st Stage; $H_1 = 13.0'$
  Max excess pore pressures; $u_e = 13.0'(120\ pcf) = 1560\ psf = 10.8\ psi$
  Prior to placing 2nd stage, u_e should dissipate to $U = 90\%$
  $$u_{e_{90}} = (1-.90)(10.8\ psi) = \underline{1.08\ psi}$$

- Height of 2nd Stage; $H_2 = 11.0'$
  Max excess pore pressures; $u_e = 11.0'(120\ pcf) = 1320\ psf = 9.2\ psi$
  Prior to placing 3rd stage, u_e should dissipate to $U = 90\%$
  $$u_e = (1-.90)(9.2\ psi) = \underline{0.92\ psi}$$

- Height of 3rd Stage; $H_3 = 8.0'$
  Max excess pore pressures; $u_e = 8.0'(120\ pcf) = 960\ psf = 6.7\ psi$
  Prior to placing final stage, u_e should dissipate to $U = 90\%$
  $$u_e = (1-.90)(6.7\ psi) = \underline{0.67\ psi}$$

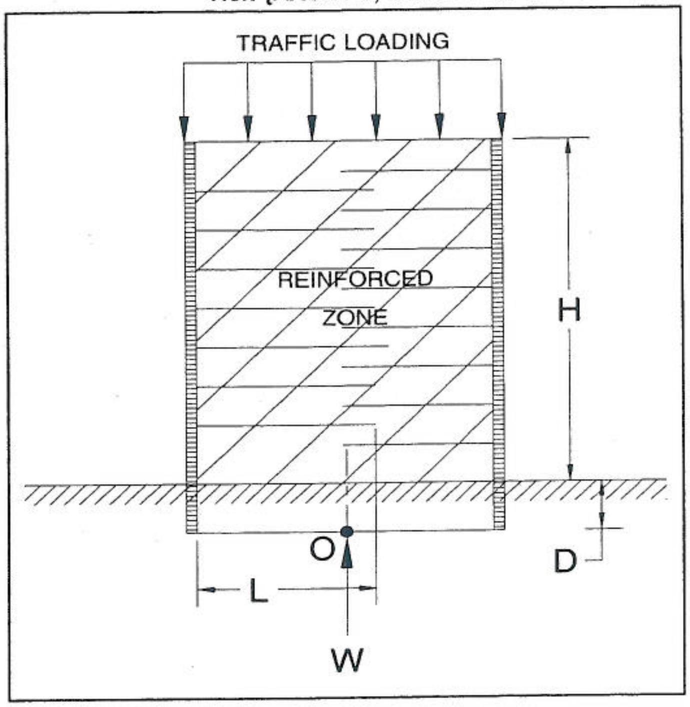
Place remaining fill; $H_4 = 5.5'$ $H_{full\ height} = 40.5'$ at abutment

DLZ	SUBJECT	Client	CH2M Hill	JOB NUMBER	0121-3070.03		
		Project	SCI-823 Portsmouth Bypass	SHEET NO.	7	OF	42
		Item	MSE Wall Bearing Capacity	COMP. BY	SJK	DATE	10-10-07
		Wall No 4, Station 2604+00, BACK-TO-BACK WALLS		CHECKED BY	DAA	DATE	10-17-07

Stage 1 controlled by wall section with Ka=0.33 **STAGE 1**

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	=	900	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	**32.13**	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	16	ft	
H	=	13	ft	Height of wall
Ka	**=**	**0.33**		
Γ Pa	=	5.3333	ft	Moment arm
Γ Wt	=	8	ft	Moment arm
B'	=	31.05	ft	
γ'	=	57.6	pcf	
W_t		7,711	lb/ft of wall	Weight from traffic
W_{mse}	=	61,690	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 2,235 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} = 4,799 psf

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

q_{ALL} = 1,920 psf

Factor of Safety = *2.15 **No Good**

*See multi-layered bearing capacity calculations page 8 of 42

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 17,495 psf

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

q_{ALL} = 6,998 psf

Factor of Safety = **7.83** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force / Kern

e = 0.54 ft   e < L/6 = 5.36 ft

DLZ

Bearing Capacity
Stiff Over Soft Clay

CLIENT	CH2M Hill	JOB NUMBER	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	8 of 42
SUBJECT	Multi-Layered Bearing Capacity	COMP. BY	SJK Date 10-10-07
MSE wall No. 4, US 23 Ramp B, Sta 2604+00		CHECKED BY	DAA Date 10-17-07

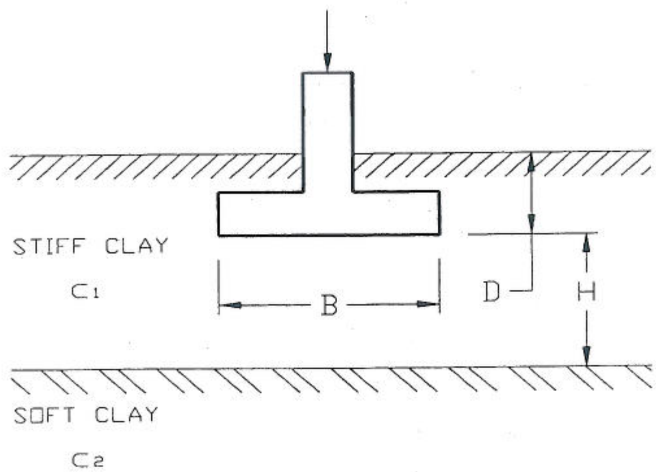
Stage 1

AASHTO, Standard Specifications for Highway Bridges, 17th Ed - 2002

Note:
Used for analysis of ramp B MSE retaining walls.
Stage 1, Using initial undrained strengths

Input

Footing Width (ft)	B	34
Length of Footing (ft)	L	200
Depth below footing to Soil Layer 2	H	4.2
Cohesion of upper soil layer 1	c_1	2000
Cohesion of lower soil layer 2	c_2	900
Bearing Capacity Factors (φ=0)	N_c	5.14
Bearing Capacity Factors (φ=0)	N_q	1.00
Effective overburnden pressure, D*γ	q	360
Factor of Safety	FS	2.5



Shape Factor	s_c	1.03	$s_c = 1 + \left(B/L\right)\left(N_q/N_c\right)$	Eq: 4.4.7.1.1.2-1
Ratio of Cohesion Values	κ	0.45	$\kappa = c_2/c_1$	
Punching Index	β_m	3.46	$\beta_m = BL/[2(B+L)H]$	
Modified Bearing Capacity Factor	N_m	2.68	$N_m = (1/\beta_m + \kappa s_c N_c) \le s_c N_c$	Eq: 4.4.7.1.1.7-2
Ultimate Undrained Bearing Capacity (psf)	q_{ULT}	5,717	$q_{ULT} = c_1 N_m + q$	Eq: 4.4.7.1.1.7-1
Allowable Undrained Bearing Capacity (psf)	q_{ALL}	**2,287**	$q_{ALL} = q_{ULT}/FS$	

Using FS = 2.5 for undrained bearing capacity;

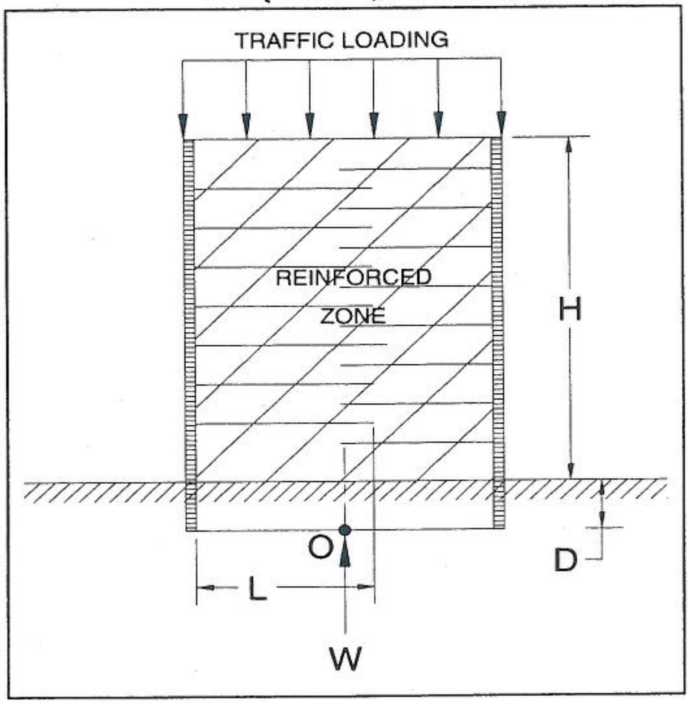
q_{all} = 2,287 psf > 2,235 = σ_v for Stage 1 = 16'

Stage 1 = 16' - 3'(embedment) = 13'
Treat 13' as consolidating stress.

SUBJECT		JOB NUMBER	0121-3070.03
Client	CH2M Hill	SHEET NO.	9 OF 42
Project	SCI-823 Portsmouth Bypass	COMP. BY	SJK DATE 10-10-07
Item	MSE Wall Bearing Capacity	CHECKED BY	DAA DATE 10-19-07
Wall No 4, Station 2604+00, BACK-TO-BACK WALLS			
Ka=0.0, for soil reinforcement overlap greater than 0.3*H		STAGE 2	

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	=	1447	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	**32.13**	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	27	ft	
H	=	24	ft	Height of wall
Ka	=	**0.00**		
Γ Pa	=	9	ft	Moment arm
Γ Wt	=	13.5	ft	Moment arm
B'	=	32.13	ft	
γ'	=	57.6	pcf	
W_t		7,711	lb/ft of wall	Weight from traffic
W_{mse}	=	104,101	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 3,480 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} = 7,610 psf

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

q_{ALL} = 3,044 psf

Factor of Safety = 2.19* **No Good**

* See multi-layered bearing capacity calculations page 10 of 42

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 18,015 psf

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

q_{ALL} = 7,206 psf

Factor of Safety = **5.18** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 0.00 ft

Kern

e < L/6 = 5.36 ft

DLZ

Bearing Capacity
Stiff Over Soft Clay

CLIENT: CH2M Hill
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Multi-Layered Bearing Capacity
MSE wall No. 4, US 23 Ramp B, Sta 2604+00
Stage 2

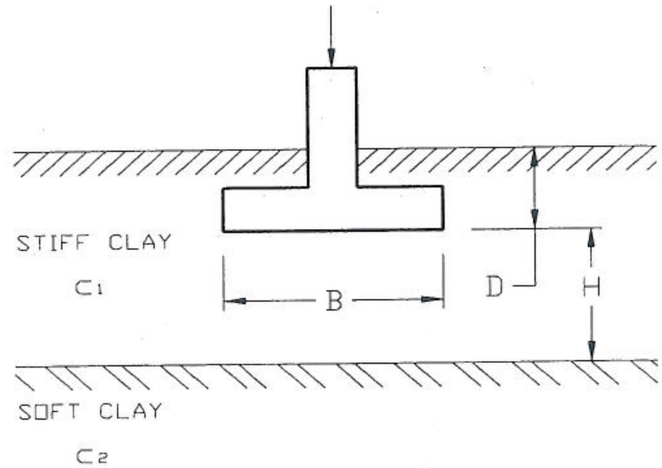
JOB NUMBER: 0121-3070.03
SHEET NO. 10 of 42
COMP. BY: SJK Date 10-10-07
CHECKED BY: DAA Date 10-17-07

AASHTO, Standard Specifications for Highway Bridges, 17th Ed - 2002

Note:
Used for analysis of ramp B MSE retaining walls.
Stage 2, Using undrained strengths after consolidation under stage 1

Input

Footing Width (ft)	B	34
Length of Footing (ft)	L	200
Depth below footing to Soil Layer 2	H	4.2
Cohesion of upper soil layer 1	c_1	2511
Cohesion of lower soil layer 2	c_2	1447
Bearing Capacity Factors (φ=0)	N_c	5.14
Bearing Capacity Factors (φ=0)	N_q	1.00
Effective overburnden pressure, D*γ	q	360
Factor of Safety	FS	2.5



Shape Factor	s_c	1.03	$s_c = 1 + \left(B/L\right)\left(N_q/N_c\right)$	Eq: 4.4.7.1.1.2-1
Ratio of Cohesion Values	κ	0.57626	$\kappa = c_2/c_1$	
Punching Index	β_m	3.46	$\beta_m = BL/[2(B+L)H]$	
Modified Bearing Capacity Factor	N_m	3.35	$N_m = (1/\beta_m + \kappa s_c N_c) \le s_c N_c$	Eq: 4.4.7.1.1.7-2
Ultimate Undrained Bearing Capacity (psf)	q_{ULT}	8,769	$q_{ULT} = c_1 N_m + q$	Eq: 4.4.7.1.1.7-1
Allowable Undrained Bearing Capacity (psf)	q_{ALL}	$\boxed{3,508}$	$q_{ALL} = q_{ULT}/FS$	

Using FS = 2.5 for undrained bearing capacity;

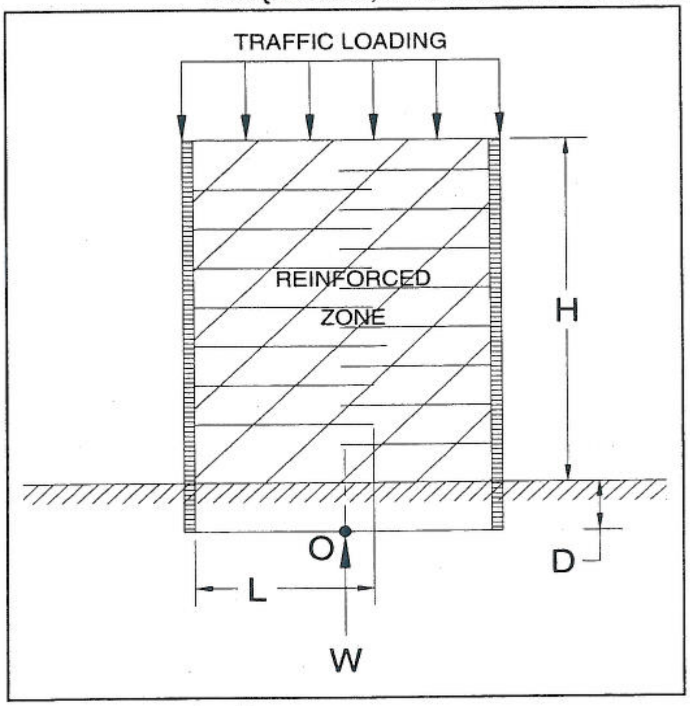
$q_{all} = 3,508 > 3,480 = \sigma_v$ for Stage 2 = 24'

stage 2; $H_2 = 24 - 13 = 11'$

DLZ	SUBJECT				
	Client	CH2M Hill	JOB NUMBER	0121-3070.03	
	Project	SCI-823 Portsmouth Bypass	SHEET NO.	11	OF 42
	Item	MSE Wall Bearing Capacity	COMP. BY	SAK	DATE 10-10-07
	Wall No 4, Station 2604+00, BACK-TO-BACK WALLS		CHECKED BY	DAA	DATE 10-17-07
	Ka=0.0, for soil reinforcement overlap greater than 0.3*H		STAGE 3		

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	=	1910	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	**32.13**	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	35	ft	
H	=	32	ft	Height of wall
Ka	**=**	**0.00**		
Γ Pa	=	11.667	ft	Moment arm
Γ Wt	=	17.5	ft	Moment arm
B'	=	32.13	ft	
γ'	=	57.6	pcf	
W_t		7,711	lb/ft of wall	Weight from traffic
W_{mse}	=	134,946	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 0.00 ft

Kern

e < L/6 = 5.36 ft

Effective Bearing Pressure

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

σ_v = 4,440 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} = 9,990 psf

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

q_{ALL} = 3,996 psf

Factor of Safety = **2.25*** **No Good**

* See multi-layered bearing capacity calculations page 12 of 42

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 18,015 psf

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

q_{ALL} = 7,206 psf

Factor of Safety = **4.06** **OK**

DLZ	CLIENT	CH2M Hill	JOB NUMBER 0121-3070.03
	PROJECT	SCI-823 Portsmouth Bypass	SHEET NO. 12 of 42
	SUBJECT	Multi-Layered Bearing Capacity	COMP. BY SMK Date 10-10-07
	MSE wall No. 4, US 23 Ramp B, Sta 2604+00		CHECKED BY DAA Date 10-17-07

Bearing Capacity
Stiff Over Soft Clay

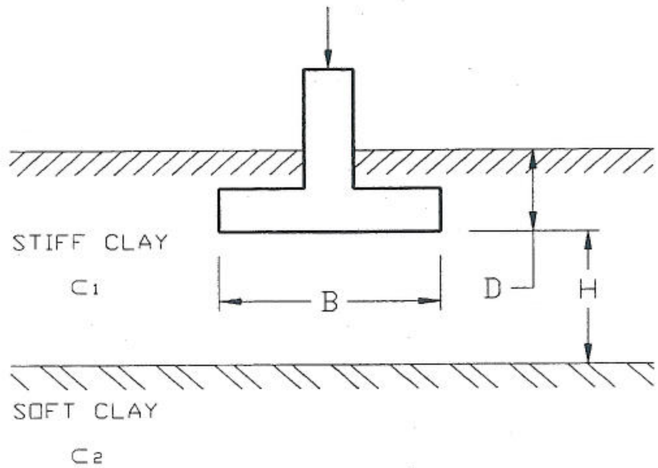
Stage 3

AASHTO, Standard Specifications for Highway Bridges, 17th Ed - 2002

Note:
Used for analysis of ramp B MSE retaining walls.
Stage 3, Using undrained strengths after consolidation under stage 2

Input

Footing Width (ft)	B	34
Length of Footing (ft)	L	200
Depth below footing to Soil Layer 2	H	4.2
Cohesion of upper soil layer 1	c_1	2943
Cohesion of lower soil layer 2	c_2	1910
Bearing Capacity Factors (φ=0)	N_c	5.14
Bearing Capacity Factors (φ=0)	N_q	1.00
Effective overburnden pressure, D*γ	q	360
Factor of Safety	FS	2.5



Shape Factor	s_c	1.03	$s_c = 1 + \left(B/L\right)\left(N_q/N_c\right)$	Eq: 4.4.7.1.1.2-1
Ratio of Cohesion Values	κ	0.649	$\kappa = c_2/c_1$	
Punching Index	β_m	3.46	$\beta_m = BL/[2(B+L)H]$	
Modified Bearing Capacity Factor	N_m	3.74	$N_m = (1/\beta_m + \kappa s_c N_c) \leq s_c N_c$	Eq: 4.4.7.1.1.7-2
Ultimate Undrained Bearing Capacity (psf)	q_{ULT}	11,353	$q_{ULT} = c_1 N_m + q$	Eq: 4.4.7.1.1.7-1
Allowable Undrained Bearing Capacity (psf)	q_{ALL}	$\boxed{4,541}$	$q_{ALL} = q_{ULT}/FS$	

Using FS = 2.5 for undrained bearing capacity;
q_{all} = 4,541 psf > 4,440 psf = σ_v for Stage 2 = 32'

Stage 3; $H_3 = 32' - 24' = 8'$

DLZ

Bearing Capacity
Stiff Over Soft Clay

CLIENT	CH2M Hill	JOB NUMBER	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	13 of 42
SUBJECT	Multi-Layered Bearing Capacity	COMP. BY	SJK Date 10-10-07
MSE wall No. 4, US 23 Ramp B, Sta 2604+00		CHECKED BY	DAA Date 10-17-07

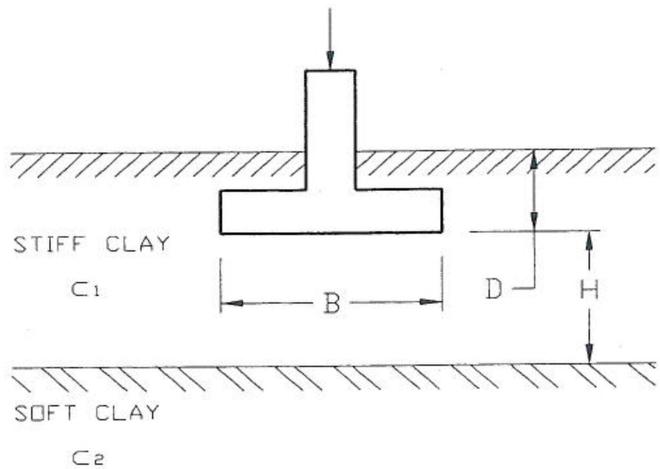
Stage 4

AASHTO, Standard Specifications for Highway Bridges, 17th Ed - 2002

Note:
Used for analysis of ramp B MSE retaining walls.
Stage 4, Using undrained strengths after consolidation under stage 3

Input

Footing Width (ft)	B	34
Length of Footing (ft)	L	200
Depth below footing to Soil Layer 2	H	4.2
Cohesion of upper soil layer 1	c_1	3257
Cohesion of lower soil layer 2	c_2	2247
Bearing Capacity Factors (φ=0)	N_c	5.14
Bearing Capacity Factors (φ=0)	N_q	1.00
Effective overburnden pressure, D*γ	q	360
Factor of Safety	FS	2.5



Shape Factor	s_c	1.03	$s_c = 1 + \left(\frac{B}{L}\right)\left(\frac{N_q}{N_c}\right)$	Eq: 4.4.7.1.1.2-1
Ratio of Cohesion Values	κ	0.6899	$\kappa = \frac{c_2}{c_1}$	
Punching Index	β_m	3.46	$\beta_m = BL/[2(B+L)H]$	
Modified Bearing Capacity Factor	N_m	3.95	$N_m = (1/\beta_m + \kappa s_c N_c) \leq s_c N_c$	Eq: 4.4.7.1.1.7-2
Ultimate Undrained Bearing Capacity (psf)	q_{ULT}	13,233	$q_{ULT} = c_1 N_m + q$	Eq: 4.4.7.1.1.7-1
Allowable Undrained Bearing Capacity (psf)	q_{ALL}	$\boxed{5,293}$	$q_{ALL} = \frac{q_{ULT}}{FS}$	

Prior to placing remaining fill, $q_{all} = 5{,}293$ psf

DLZ

SUBJECT

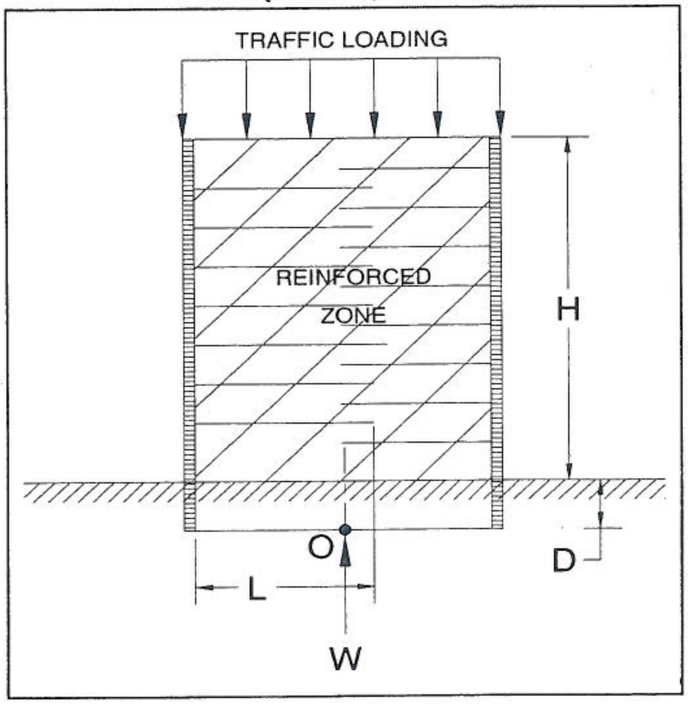
Client	CH2M Hill
Project	SCI-823 Portsmouth Bypass
Item	MSE Wall Bearing Capacity
Wall No 4, Station 2604+00, BACK-TO-BACK WALLS	

JOB NUMBER	0121-3070.03		
SHEET NO.	14	OF	42
COMP. BY	SAK	DATE	10-10-07
CHECKED BY	DAA	DATE	10-17-0

Ka=0.0, for soil reinforcement overlap greater than 0.3*H   ABUTMENT WALL - Full Height

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Reinforced Zone
ϕ'_{EMB}	=	34	deg.	Friction ang.	Reinforced Zone
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	900	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	**32.13**	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	40.5	ft	
H	=	37.5	ft	Height of wall
Ka	=	**0.00**		
Γ Pa	=	13.5	ft	Moment arm
Γ Wt	=	20.25	ft	Moment arm
B'	=	32.13	ft	
γ'	=	57.6	pcf	
W_t		7,711	lb/ft of wall	Weight from traffic
W_{mse}	=	156,152	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 5,100 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 4,799 psf

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

q_{ALL} = 1,920 psf

Factor of Safety = **0.94** **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} = 18,015 psf

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

q_{ALL} = 7,206 psf

Factor of Safety = **3.53** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 0.00 ft

Kern

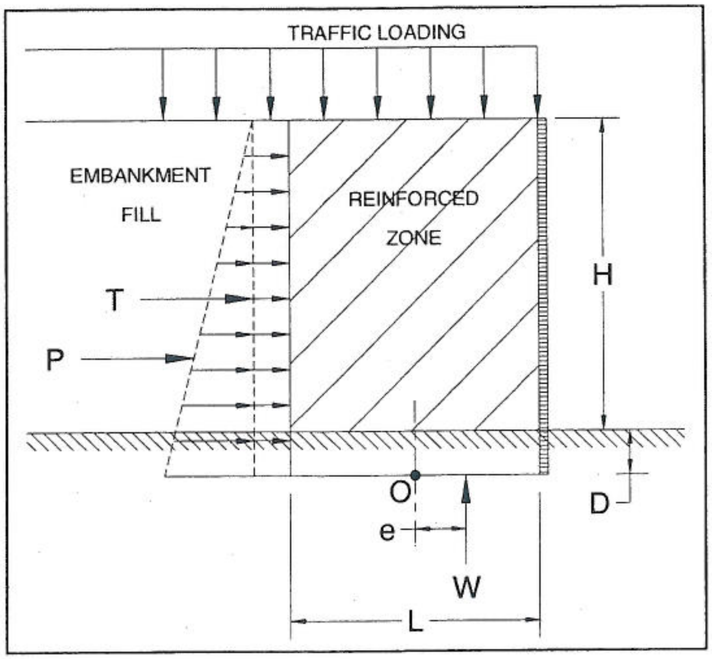
e < L/6 = 5.36 ft

DLZ	SUBJECT	Client	CH2M Hill	JOB NUMBER	0121-3070.03
		Project	SCI-823 Portsmouth Bypass	SHEET NO.	15 OF 42
		Item	MSE Wall Bearing Capacity	COMP. BY	SAK DATE 10-10-07
		Wall No 4, Station 2602+00, DOUBLE WALL		CHECKED BY	DAA DATE 10-17-07

Use strength values from weaker (lower) soil layer *Maximum wall height with full Ka, H=20'

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	=	900	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	20	ft	Length of MSE reinforcement
L factor	=	1		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	20	ft	
H	=	17	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	6.6667	ft	Moment arm
Γ Wt	=	10	ft	Moment arm
B'	=	17.40	ft	
γ'	=	57.6	pcf	
W_t		4,800	lb/ft of wall	Weight from traffic
W_{mse}	=	48,000	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 3,034 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 4,799 psf

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

q_{ALL} = 1,920 psf

Factor of Safety = **1.58** **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} = 10,922 psf

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

q_{ALL} = 4,369 psf

Factor of Safety = **3.60** **OK**

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 1.30 ft

Kern: e < L/6 = 3.33 ft

DLZ

SUBJECT

Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	16 OF 42
Item	MSE Wall Stability	COMP. BY	SAK DATE 10-10-07
	Wall No 4, Station 2602+00, Double Wall	CHECKED BY	DAA DATE 10-17-07

*Maximum wall height with full Ka, H=20.0'

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=17.0'

2 Analysis of double wall, but not reduction in Ka.

3 Ground water; Dw=0.0'

4 Traffic loading is neglected in resisting forces

5 For stability use strength values of upper soil layer

Wall Properties

$H+D$ = 20 feet

γ_{mse} = 120 pcf

L = 20 feet

L factor = 1.00

ϕ = 30 deg

Foundational Soil Properties

c = 2000 psf Cohesion

ϕ' = 29 deg Friction angle

ω_T = 240 psf Traffic loading

Length factor-range (0.7 - 1.0)

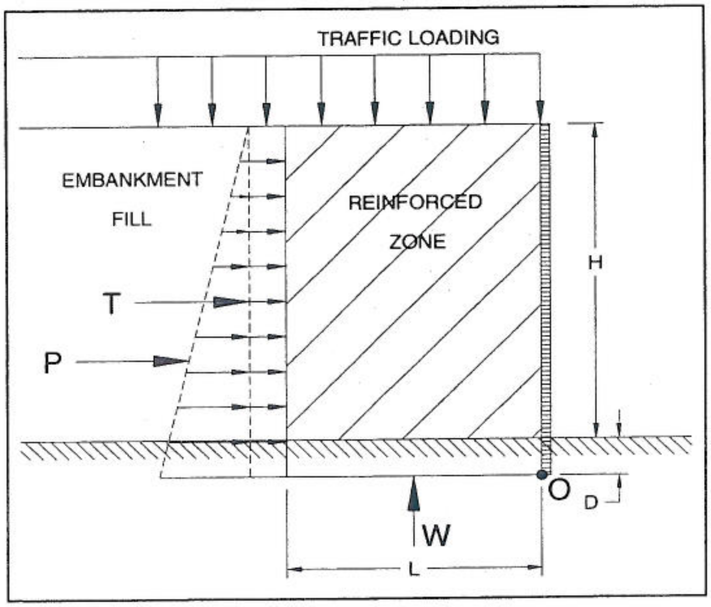
Friction Angle of Embankment Fill

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 = 9,504 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 = 17,760 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 40,000 lbs per foot of wall

Use Drained Value

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

Calculated FS = **1.87**

Required FS = 1.50

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 480,000 lb-ft

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

$\Sigma M_{overturning}$ = 68,640 lb-ft

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

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

Calculated FS = **6.99**

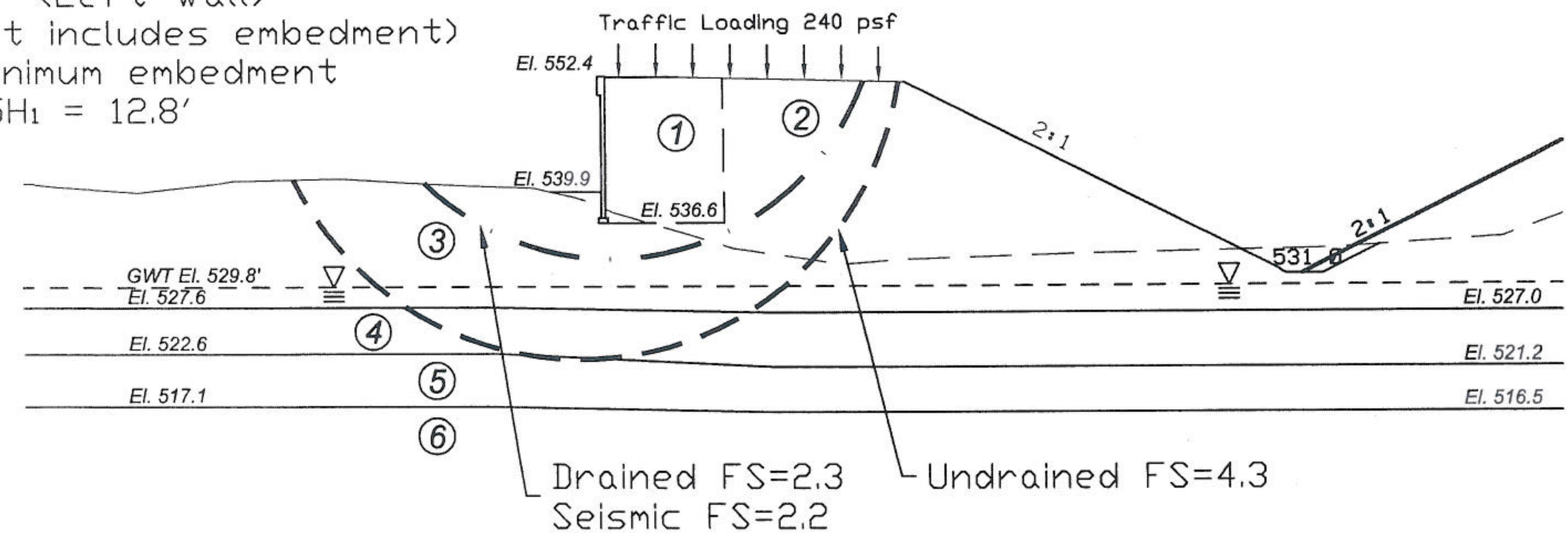
Required FS = 2.00

Resistance Against Overturning is **OK**

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	Embank Fill	0	30	0	30	120
Material 3	Possible Fill	A-4a/A-6a	2000	0	0	29	120
Material 4	Medium Stiff	A-7-6	900	0	0	28	120
Material 5	Loose	A-4a/A-4b	0	29	0	29	120
Material 6		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-1109, B-1109A,
and B-1108
US 23 Ramp B MSE Wall
Wall No. 4
H_1=15.1' (Left Wall)
(Height includes embedment)
3.0' minimum embedment
L=0.85H_1 = 12.8'

Based on Cross Section
at Station 2600+75



Stability Analyses performed using UTEXAS3 Version 1.201

US-23 Interchange
Ramp B Wall No. 4, Station 2600+75
Based on Borings B-1109, B-1109A, &B-1108

MSE GLOBAL STABILITY ANALYSIS

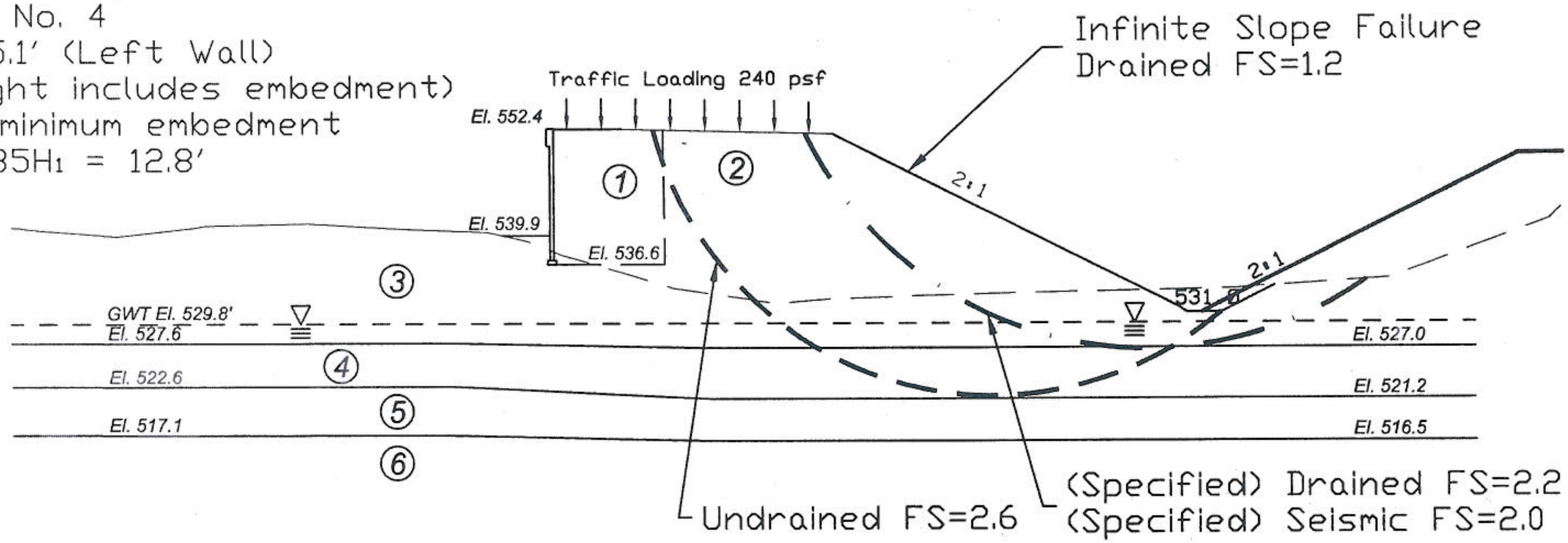
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 9/14/07

			Undrained		Drained		
Material	Consistency	Soil Type	c (psf)	φ (deg)	c' (psf)	φ' (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Embank Fill	0	30	0	30	120
Material 3	Possible Fill	A-4a/A-6a	2000	0	0	29	120
Material 4	Medium Stiff	A-7-6	900	0	0	28	120
Material 5	Loose	A-4a/A-4b	0	29	0	29	120
Material 6		Bedrock	10000	45	10000	45	145

Stability Analysis
Based on B-1109, B-1109A,
and B-1108
US 23 Ramp B MSE Walls
Wall No. 4
H_1=15.1' (Left Wall)
(Height includes embedment)
3.0' minimum embedment
L=0.85H_1 = 12.8'

Based on Cross Section
at Station 2600+75



Stability Analyses performed using UTEXAS3 Version 1.201

US-23 Interchange Ramp B Wall No. 4, Station 2600+75 Based on Borings B-1109, B-1109A, &B-1108		
MSE GLOBAL STABILITY ANALYSIS SCI-823-0.00		
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 9/14/07

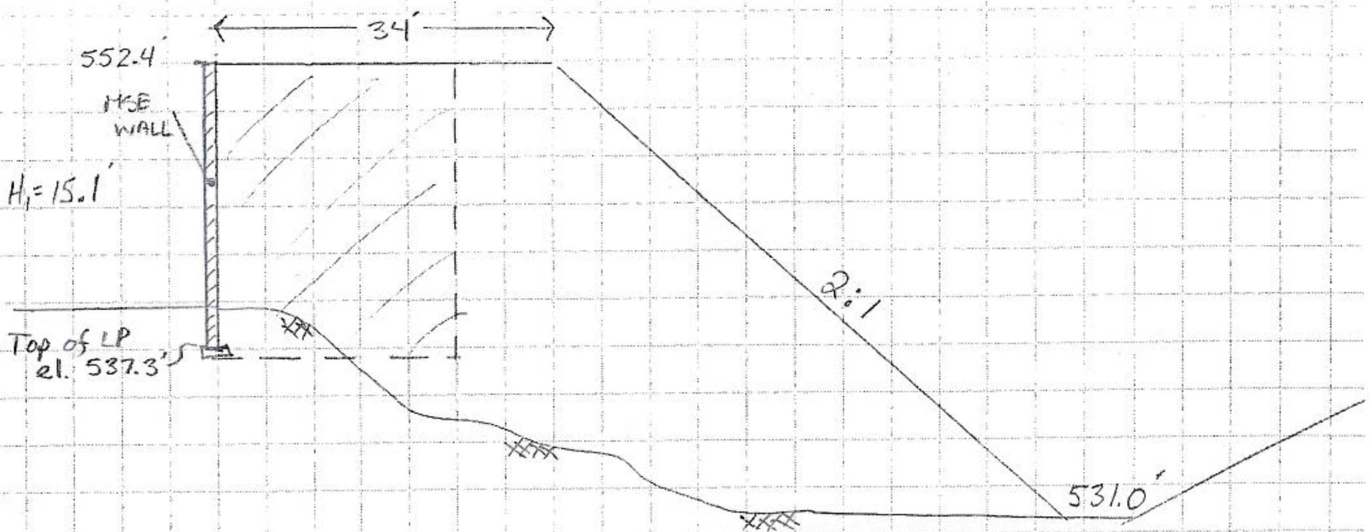
Sheet 18 of 42

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT CH2 M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT MSE Wall Stability

PROJECT NO. 0121-3070.03
SHEET NO. 19 OF 42
COMP. BY SAK DATE 10-10-0
CHECKED BY DAA DATE 10-17-0

Highest Single Wall Section
At Station 2600+75 el. = 552.4'

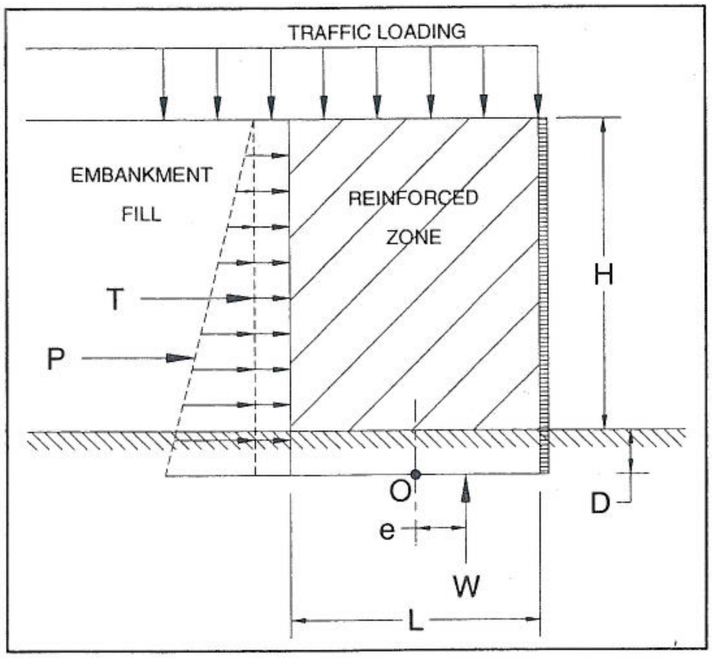


* Nearest boring B-1107, However, use profile and strengths based upon B-1109, B-1109A and, B-1108. (More Critical)

Use strength values from weaker (lower) soil layer

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	=	900	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	28	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
L=B	=	15.1	ft	Length of MSE reinforcement
L factor	=	1		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	15.1	ft	
H	=	12.1	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	5.0333	ft	Moment arm
Γ Wt	=	7.55	ft	Moment arm
B'	=	13.06	ft	
γ'	=	57.6	pcf	
W_t		3,624	lb/ft of wall	Weight from traffic
W_{mse}	=	27,361	lb/ft of wall	Weight from MSE wall

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 1.02 ft

Kern: e < L/6 = 2.52 ft

Effective Bearing Pressure

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

σ_v = 2,373 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 4,799 psf

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

q_{ALL} = 1,920 psf

Factor of Safety = 2.02* **No Good**

*See multi-layer bearing capacity calculations See page 21 of 42

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 8,832 psf

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

q_{ALL} = 3,533 psf

Factor of Safety = 3.72 **OK**

DLZ

Bearing Capacity
Stiff Over Soft Clay

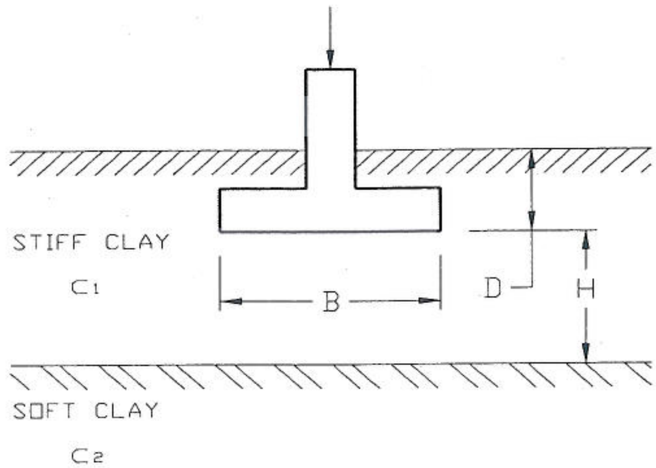
CLIENT	CH2M Hill	JOB NUMBER	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	21 of 42
SUBJECT	Multi-Layered Bearing Capacity	COMP. BY SJW	Date 10-10-07
MSE wall No. 4, US 23 Ramp B, Sta 2600+75		CHECKED BY DAA	Date 10-17-07

AASHTO, Standard Specifications for Highway Bridges, 17th Ed - 2002

Note:
Used for analysis of ramp B MSE retaining walls.
Using initial undrained strengths

Input

Footing Width (ft)	B	15.1
Length of Footing (ft)	L	200
Depth below footing to Soil Layer 2	H	9.6
Cohesion of upper soil layer 1	c_1	2000
Cohesion of lower soil layer 2	c_2	900
Bearing Capacity Factors (φ=0)	N_c	5.14
Bearing Capacity Factors (φ=0)	N_q	1.00
Effective overburnden pressure, D*γ	q	360
Factor of Safety	FS	2.5



Shape Factor	s_c	1.01	$s_c = 1 + \left(B/L\right)\left(N_q/N_c\right)$	Eq: 4.4.7.1.1.2-1
Ratio of Cohesion Values	κ	0.45	$\kappa = c_2/c_1$	
Punching Index	β_m	0.73	$\beta_m = BL/[2(B+L)H]$	
Modified Bearing Capacity Factor	N_m	3.71	$N_m = (1/\beta_m + \kappa s_c N_c) \le s_c N_c$	Eq: 4.4.7.1.1.7-2
Ultimate Undrained Bearing Capacity (psf)	q_{ULT}	7,789	$q_{ULT} = c_1 N_m + q$	Eq: 4.4.7.1.1.7-1
Allowable Undrained Bearing Capacity (psf)	q_{ALL}	$\boxed{3,116}$	$q_{ALL} = q_{ULT}/FS$	

Using a FS of 2.5 ;

q_{all} = 3,116 psf > 2,373 = σ_v for 15.1' MSE Wall

Undrained Bearing Capacity → $\boxed{OK}$

DLZ

SUBJECT			
Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	22 OF 42
Item	MSE Wall Stability	COMP. BY	SAM DATE 10-10-07
Wall No 4, Station 2600+75, SINGLE WALL		CHECKED BY	DAA DATE 10-17-07

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=12.1'
2 Analysis of single wall with 2:1 slopes
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 For stability use strength values of upper soil layer

Wall Properties

H+D	=	15.1	feet
γ_{mse}	=	120	pcf
L	=	15.1	feet
L factor	=	1.00	
ϕ	=	30	deg

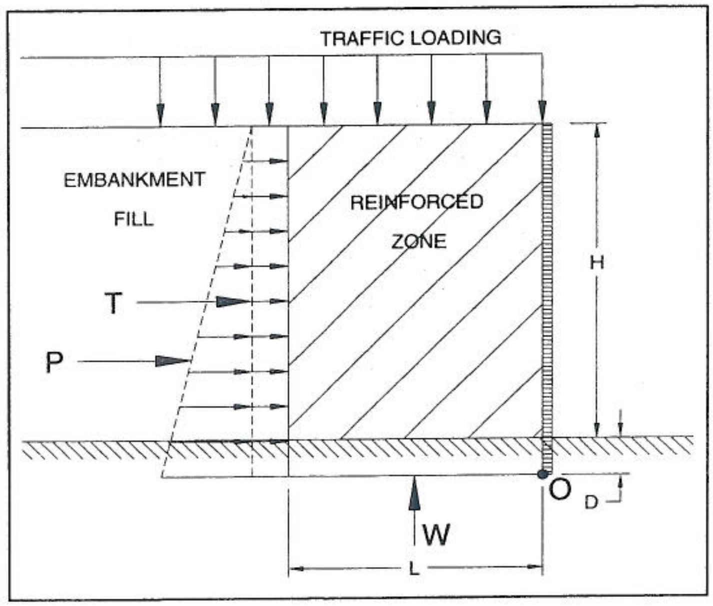
Foundational Soil Properties

c	=	2000	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 = 5,711 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 = 10,124 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 30,200 lbs per foot of wall

Use Drained Value

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

Calculated	Required
FS = **1.77**	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

$\Sigma M_{resisting}$ = 206,577 lb-ft

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

$\Sigma M_{overturning}$ = 31,753 lb-ft

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

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

Calculated	Required
FS = **6.51**	FS = 2.00

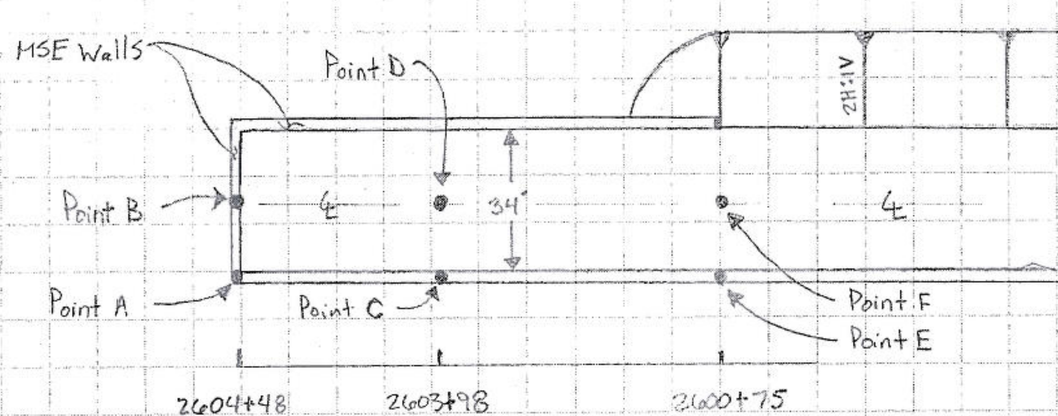
Resistance Against Overturning is **OK**

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT CHZM Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Consolidation Results
Settlement - Ramp B MSE Walls

PROJECT NO. 0121-3070.03
SHEET NO. 23 OF 42
COMP. BY SAK DATE 10-11-0
CHECKED BY DAA DATE 10-17-0

Plan View - Ramp B



• At abutment location: Sta. 2604+48

Point A, δ = 3.8" (at wall face / corner)
Point B, δ = 5.6" (at abutment wall centerline)

• 50' from abutment location: Sta. 2603+98

Point C, δ = 5.6" (at wall face)
Point D, δ = 8.4" (at ramp centerline)

• At station 2600+75

Point E, δ = 1.5" (at wall face)
Point F, δ = 2.6" (at ramp centerline)

Differential Settlement measured at wall face; between pt A and pt C

$$DS = \frac{(5.6" - 3.8")(1/12")}{50'} = 0.003 = 0.3\% < 1.0\% \checkmark \boxed{OK}$$

Differential Settlement measured at wall face; between pt C and pt E.

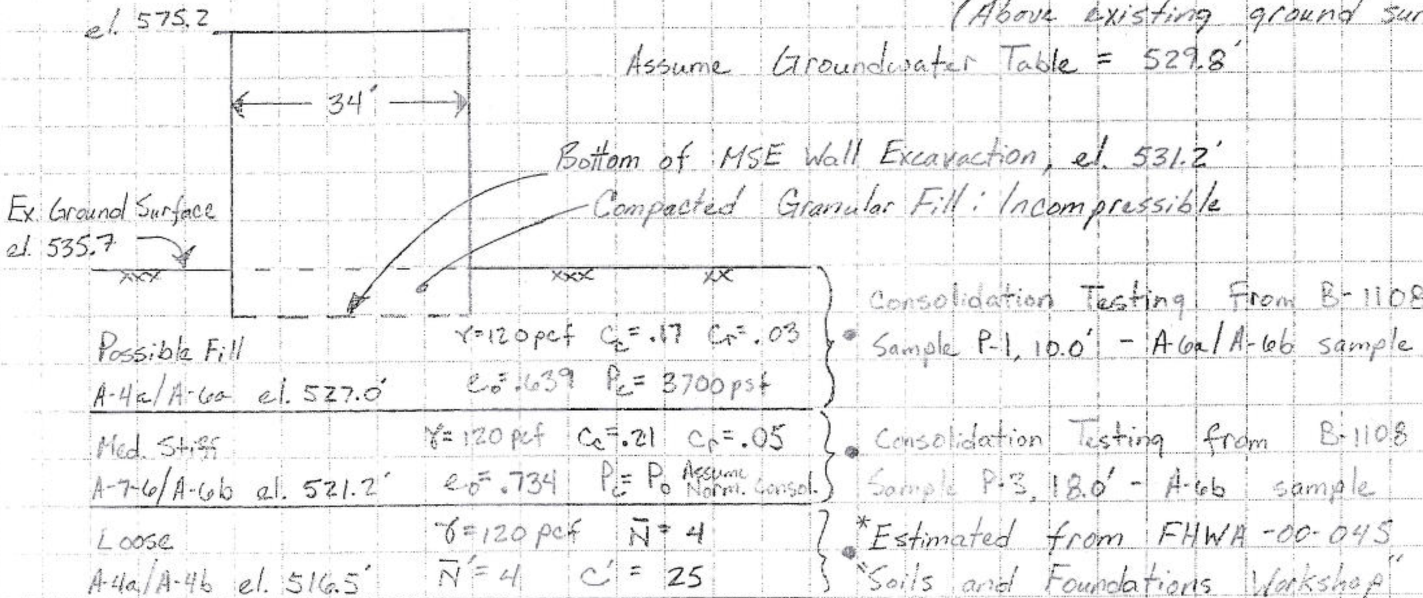
$$DS = \frac{(5.6 - 1.5")(1/12)}{323'} = 0.001 = 0.1\% < 1.0\% \checkmark \boxed{OK}$$

Evaluate Settlement at Station 2604+48; End Ramp B MSE Wall
Profile based upon borings B-1109, B-1109A, and B-1108

Cross Section View at Sta. 2604+48 - Abutment Location

$H = 575.2 - 535.7' = 39.5'$
(Above existing ground surface)

Assume Groundwater Table = 529.8'



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

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

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

When $C' = 25$, $C_c = \frac{2.0}{25} = 0.08$

* At Station 2603+98, Use H = 572.3 - 535.7 = 36.6'

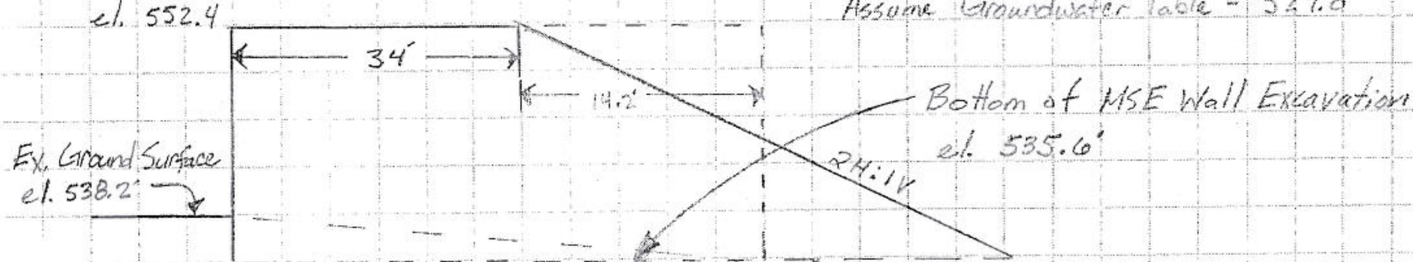
Evaluate Settlement at Station 2600+75 ; Ramp B MSE Wall
Profile based upon borings B-1109, B-1109A, and B-1108.

Cross Section View at Sta. 2600+75

* Transition from Single to back-to-back MSE Walls.

H = 552.4 - 538.2' = 14.2'

Assume Groundwater Table = 529.8'



Possible Fill A-4a / A-6a el. 527.6'	γ=120 pcf C_c=.17 C_r=.03 e_o=.639 P_c=3700 psf	Consolidation Testing from B-1108 Sample P-1, 10.0' - A-6a/A-6b sample
Med. Stiff A-7-6/A-6b el. 522.6'	γ=120 pcf C_c=.21 C_r=.05 e_o=.734 $P_c \cong P_o$ Assume Norm. Consol.	Consolidation Testing from B-1108 Sample P-3, 18.0' - A-6b sample
Loose A-4a / A-4b el. 516.5'	γ=120 pcf $\bar{N}$ = 4 $\bar{N}'$= 4 C'= 25	*Estimated from FHWA-00-045 "Soils and Foundations Workshop"

*See Sample Calculation pg. 24 of 42
* C_α = 0.08

Ramp B 2604+48 Abutment MSE Wall back-to-back

SJK 10-10-07
DAA 10-17-07

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	: RB1	Project Manager	: Nix
Date	: 10/11/07	Computed by	: sjr

Settlement for X-Direction

Embank. slope, x direc. = 0.10 (ft)	Height of fill H = 39.50 (ft)
y direc. = 0.10 (ft)	Unit weight of fill = 120.00 (pcf)
Embankment top width = 34.00 (ft)	p load/unit area = 4740.00 (psf)
Embankment bottom width = 34.20 (ft)	Foundation Elev. = 535.70 (ft)
Ground Surface Elev. = 535.70 (ft)	
Water table Elev. = 529.80 (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.	4.5	-----	-----	-----	120.00	----	----
2	COMP.	4.2	0.170	0.030	0.000	120.00	2.65	0.63
3	COMP.	5.8	0.210	0.050	0.000	120.00	2.65	0.73
4	COMP.	4.7	0.080	0.080	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	4.20	529.10	748.32	3700.00
3	5.80	524.10	1036.32	1036.32
4	4.70	518.85	1338.72	1338.72

Layer	X = 0.10 Stress (psf)	Sett. (in.)	X = 3.50 Stress (psf)	Sett. (in.)	X = 6.90 Stress (psf)	Sett. (in.)	X = 10.30 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	1204.52	0.39	1872.13	0.50	2183.41	0.55	2294.48	0.57
3	1180.61	2.79	1595.19	3.42	1896.95	3.82	2072.42	4.03
4	1147.13	0.61	1430.59	0.71	1665.89	0.79	1831.19	0.84
	Pt. A	3.78		4.64		5.16		5.44

Layer	X = 13.70 Stress (psf)	Sett. (in.)	X = 17.10 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	2333.53	0.57	2343.40	0.57
3	2157.71	4.13	2182.78	4.16
4	1925.76	0.87	1956.18	0.88
		5.57	Pt. B	5.61

Ramp B 2603+98 MSE Wall back-to-back

SJK 10-10-07
DAA 10-17-07

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	: RB1	Project Manager	: Nix
Date	: 10/11/07	Computed by	: sjr

Settlement for X-Direction

Embank. slope, x direc.	= 0.10 (ft)	Height of fill H	= 36.60 (ft)
y direc.	= 0.10 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 34.00 (ft)	p load/unit area	= 4392.00 (psf)
Embankment bottom width	= 34.20 (ft)	Foundation Elev.	= 535.70 (ft)
Ground Surface Elev.	= 535.70 (ft)		
Water table Elev.	= 529.80 (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.	4.5	-----	-----	-----	120.00	----	----
2	COMP.	4.2	0.170	0.030	0.000	120.00	2.65	0.63
3	COMP.	5.8	0.210	0.050	0.000	120.00	2.65	0.73
4	COMP.	4.7	0.080	0.080	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	4.20	529.10	748.32	3700.00
3	5.80	524.10	1036.32	1036.32
4	4.70	518.85	1338.72	1338.72

Y = 50.0

Layer	X = 0.10 Stress (psf)	Sett. (in.)	X = 3.50 Stress (psf)	Sett. (in.)	X = 6.90 Stress (psf)	Sett. (in.)	X = 10.30 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	2210.04	0.55	3433.80	0.92	4005.41	1.22	4209.76	1.31
3	2172.41	4.15	2935.61	4.93	3491.34	5.41	3814.65	5.66
4	2108.14	0.93	2630.60	1.06	3064.33	1.17	3369.09	1.23
		------		------		------		------
A. C		5.63		6.92		7.79		8.21

Layer	X = 13.70 Stress (psf)	Sett. (in.)	X = 17.10 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	4281.68	1.34	4299.86	1.35
3	3971.87	5.78	4018.10	5.81
4	3543.46	1.27	3599.55	1.28
		------		------
		8.39	A. D	8.45

Ramp B 2600+75 MSE Wall Single Wall

SJR 10-10-07
DAA 10-17-07

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

Project Name : SCI-823
File Name : RB1
Date : 10/11/07

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

Settlement for X-Direction

Embank. slope, x direc. =	0.10 (ft)	Height of fill H =	14.20 (ft)
y direc. =	0.10 (ft)	Unit weight of fill =	120.00 (pcf)
Embankment top width =	34.00 (ft)	p load/unit area =	1704.00 (psf)
Embankment bottom width =	34.20 (ft)	Foundation Elev. =	535.70 (ft)
Ground Surface Elev. =	538.20 (ft)		
Water table Elev. =	529.80 (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.	2.6	-----	-----	-----	120.00	----	----
2	COMP.	8.0	0.170	0.030	0.000	120.00	2.65	0.63
3	COMP.	5.0	0.210	0.050	0.000	120.00	2.65	0.73
4	COMP.	6.1	0.080	0.080	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	8.00	531.60	792.00	3700.00
3	5.00	525.10	1278.72	1278.48
4	6.10	519.55	1598.40	1598.26

Layer	X = 0.10 Stress (psf)	Sett. (in.)	X = 3.50 Stress (psf)	Sett. (in.)	X = 6.90 Stress (psf)	Sett. (in.)	X = 10.30 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	439.08	0.34	761.01	0.52	837.67	0.55	854.61	0.56
3	426.25	0.91	588.32	1.20	700.66	1.38	761.97	1.48
4	414.25	0.29	520.98	0.36	608.70	0.41	669.38	0.44
	Pt. E	1.54		2.07		2.35		2.49

Layer	X = 13.70 Stress (psf)	Sett. (in.)	X = 17.10 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	859.40	0.56	860.51	0.56
3	790.33	1.52	798.46	1.54
4	703.60	0.46	714.52	0.47
		2.55		2.57 Pt. F

Rear Abutment Location

* Station 2604+48 Profile based upon boring B-1109

el. 531.2 Bottom of Excavation

A-4a / A-6a $\overline{LL} \simeq 29$

$c_v \simeq 0.5$ ft²/day

el. 527.6

A-7-6 $\overline{LL} \simeq 46$

$c_v \simeq 0.25$ ft²/day

el. 522.6

A-4a / A-4b Assume Free-Draining

el. 517.1

TOP OF ROCK

Assume double drainage

$H_v = (531.2 - 522.6)/2 = 4.3'$

Use "weighted" c_v value;

$$c_v = \frac{(3.6')(0.5 \text{ ft}^2/\text{day}) + (5.0')(0.25 \text{ ft}^2/\text{day})}{8.6'}$$

$c_v = 0.35$ ft²/day

Time to $U = 90\%$;

$$t_{90} = \frac{T_v \cdot H_v^2}{c_v} = \frac{0.848\,(4.3')^2}{0.35 \text{ ft}^2/\text{day}} = 45 \text{ days}$$

* Downdrag at Rear Abutment

To prevent downdrag forces from reducing the allowable capacity of the piles, the remaining settlement should be limited to 0.4 inches or less prior to driving piles.

Settlement at the ℄ of abutment wall, $\delta = 5.6''$

Of the 5.6", 4.7" is consolidation settlement.

$$\left(1 - \frac{0.4''}{4.7''}\right) = 0.91 \quad \text{OR} \quad U = 91\%$$

Prior to driving piles, $U = 91\%$ * Say $U = 90\%$

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2598+23 to 2604+48**
Reference: FHWA-RD-86-168

SAK 10-10-07
DAA 10-17-07

Wick Drain Spacing **5.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	5.25	**0.35**	**4.3**	**4.7**
5	0.0635	0.0946	0.29	0.35	53.4	2.5				
10	0.1270	0.1893	0.49	0.51	75.1	3.5				
15	0.1905	0.2839	0.63	0.62	86.1	4.0				
20	0.2540	0.3786	0.74	0.69	91.8	4.3				
25	0.3175	0.4732	0.81	0.74	95.0	4.5				
30	0.3810	0.5679	0.86	0.78	96.9	4.6				
35	0.4444	0.6625	0.89	0.83	98.1	4.6				
40	0.5079	0.7572	0.91	0.87	98.8	4.6				
45	0.5714	0.8518	0.93	0.90	99.3	4.7				

Assumes a Triangular Grid Layout

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2598+23 to 2604+48**
Reference: FHWA-RD-86-168

Wick Drain Spacing **7.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	7.35	0.35	4.3	4.7
5	0.0324	0.0946	0.17	0.35	45.5	2.1				
10	0.0648	0.1893	0.29	0.51	65.7	3.1				
15	0.0972	0.2839	0.40	0.62	77.3	3.6				
20	0.1296	0.3786	0.49	0.69	84.3	4.0				
25	0.1620	0.4732	0.57	0.74	88.9	4.2				
30	0.1944	0.5679	0.64	0.78	92.2	4.3				
35	0.2268	0.6625	0.70	0.83	94.7	4.5				
40	0.2592	0.7572	0.74	0.87	96.6	4.5				
45	0.2915	0.8518	0.78	0.90	97.8	4.6				
50	0.3239	0.9465	0.82	0.91	98.3	4.6				
55	0.3563	1.0411	0.84	0.88	98.1	4.6				
60	0.3887	1.1357	0.86	0.78	97.0	4.6				

Assumes a Triangular Grid Layout

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2598+23 to 2604+48**
Reference: FHWA-RD-86-168

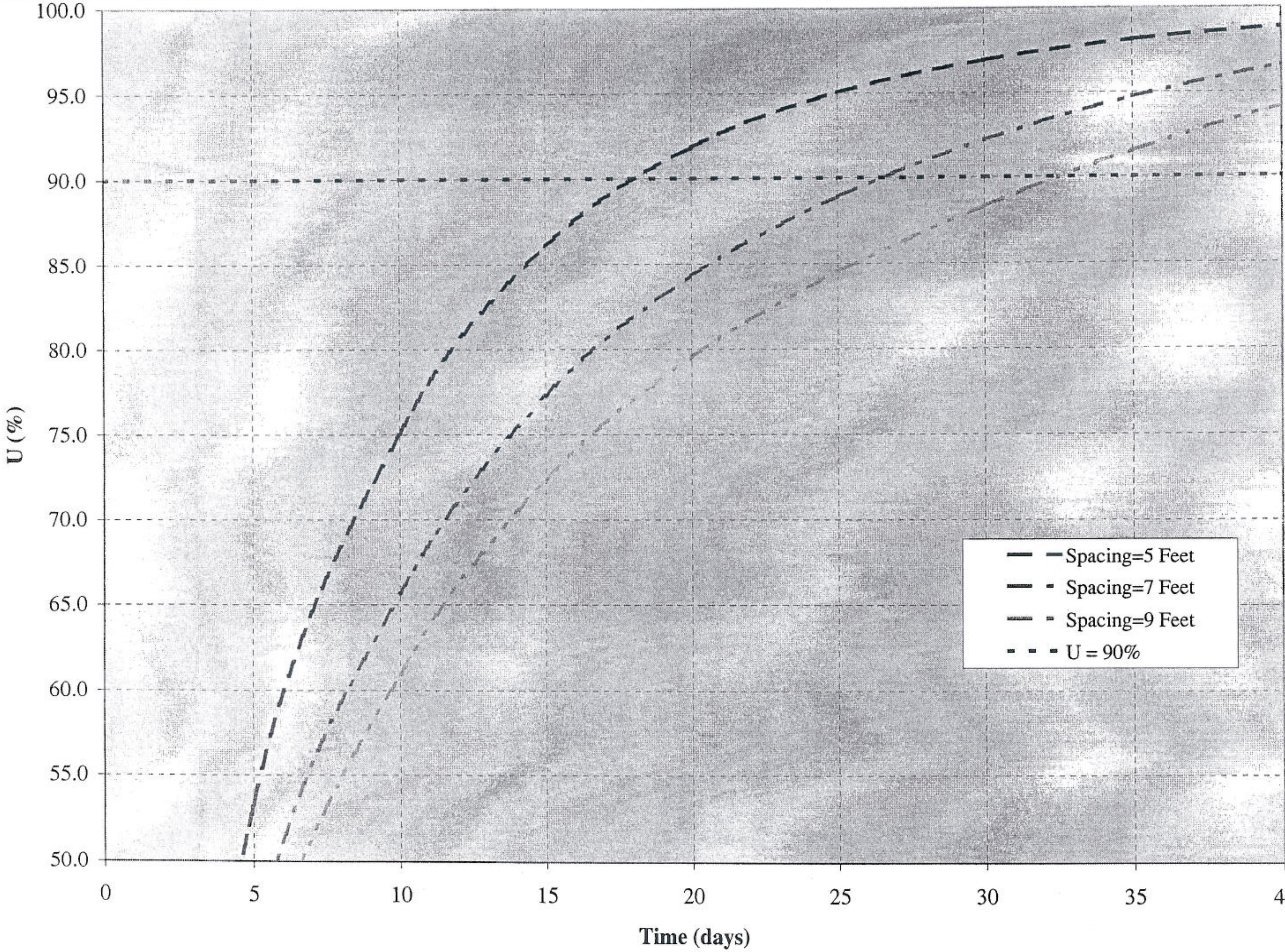
Wick Drain Spacing **9.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	9.45	**0.35**	**4.3**	**4.7**
5	0.0196	0.0946	0.11	0.35	41.9	2.0				
10	0.0392	0.1893	0.20	0.51	60.9	2.9				
15	0.0588	0.2839	0.27	0.62	72.4	3.4				
20	0.0784	0.3786	0.34	0.69	79.6	3.7				
25	0.0980	0.4732	0.40	0.74	84.5	4.0				
30	0.1176	0.5679	0.46	0.78	88.3	4.2				
35	0.1372	0.6625	0.51	0.83	91.5	4.3				
40	0.1568	0.7572	0.56	0.87	94.1	4.4				
45	0.1764	0.8518	0.60	0.90	96.0	4.5				
50	0.1960	0.9465	0.64	0.91	96.8	4.5				
55	0.2156	1.0411	0.68	0.88	96.1	4.5				
60	0.2352	1.1357	0.71	0.78	93.6	4.4				
65	0.2548	1.2304	0.74	0.58	89.1	4.2				

Assumes a Triangular Grid Layout

DLZ

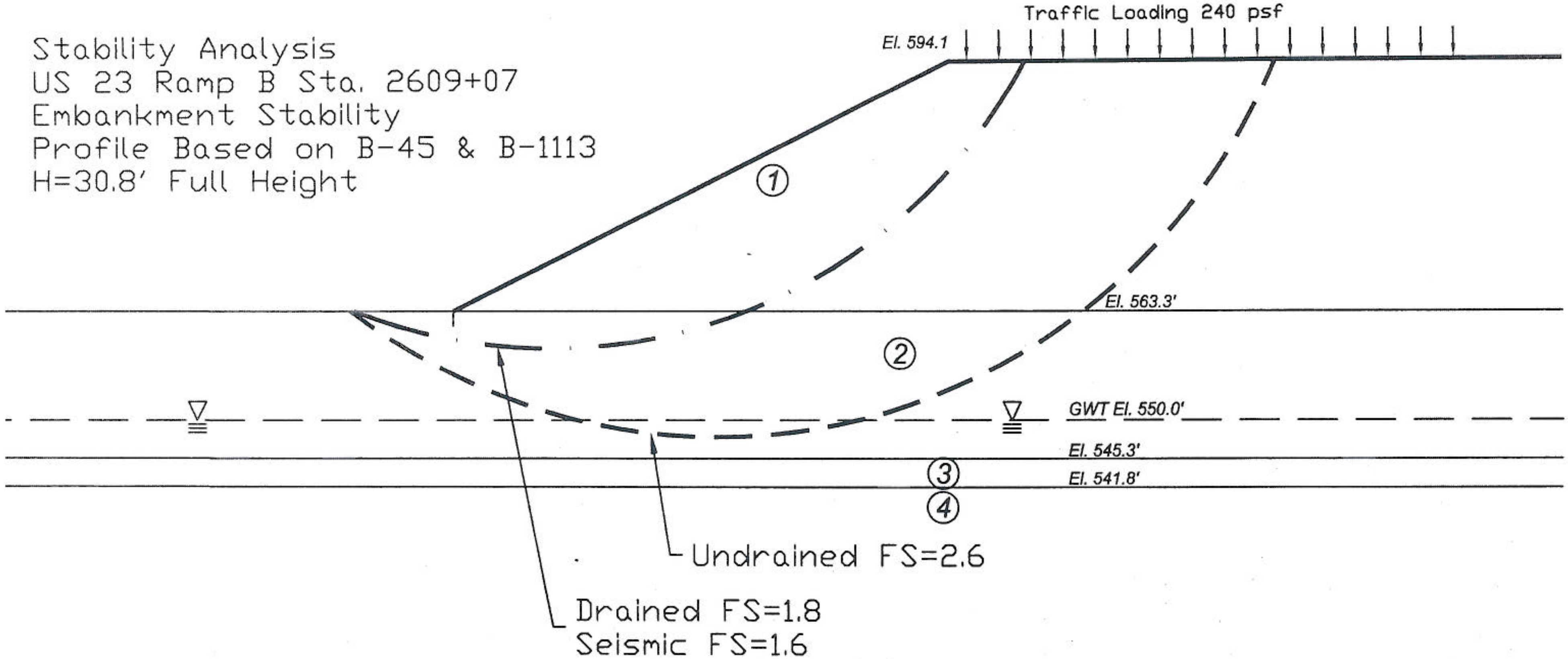
Percent Consolidation vs Time Using Prefabricated Vertical "Wick" Drains
US-23 Interchange, Ramp B (Wall No 4)



Sheet 33 of 42

Material	Consistency	Soil Type	Undrained c (psf)	Undrained ϕ (deg)	Drained c' (psf)	Drained ϕ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	2000	0	300	28	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
US 23 Ramp B Sta. 2609+07
Embankment Stability
Profile Based on B-45 & B-1113
H=30.8' Full Height



Stability Analyses performed using UTEXAS3 Version 1.201

US-23 Interchange
Ramp B Embankment, Station 2609+07
Profile Based on B-45 and B-1113

MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 9/14/07

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT: CH2M Hill
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Consolidation Parameters
Settlement - Ramp B, Station 2609+07

PROJECT NO. 0121-3070.03

COMP. BY SAK DATE 10-11-0
CHECKED BY DAA DATE 10-17-0

Evaluate Settlement at station 2609+07, using 2H:1V Slopes
Profile based upon boring B-45 and B-1113

el. 594.1

*Assume groundwater table = 550.0'

Embankment Fill
$\gamma = 120$ pcf

el. 568.3'

A-6b/A-2-6 — $\gamma = 120$ pcf $C_c = .09$ $C_r = .01$
$e_o = .632$ $P_c \cong P_o$
el. 545.3'
— Consolidation Testing from B-45 Sample P-1, 5.0' - A-6b sample

$\gamma = 120$ pcf $\bar{N} \cong 4$
$\bar{N} = 4$ $C' = 25$
el. 541.8'
— Estimated from FHWA-00-045 "Soils and Foundations Workshop"

*See Sample Calculation pg. 24 of 42
$C_c = 0.08$

Ramp B 2609+07 Spill through slopes

SJR 10-10-07
DAA 10-17-07

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 : RBS.emb Project Manager : Nix
Date : 10/11/07 Computed by : sjr

Settlement for X-Direction

Embank. slope, x direc. = 61.60 (ft) Height of fill H = 30.80 (ft)
y direc. = 61.60 (ft) Unit weight of fill = 120.00 (pcf)
Embankment top width = 40.00 (ft) p load/unit area = 3696.00 (psf)
Embankment bottom width = 163.20 (ft) Foundation Elev. = 563.30 (ft)
Ground Surface Elev. = 563.30 (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	COMP.	18.0	0.090	0.010	0.000	120.00	2.65	0.63
2	COMP.	3.5	0.080	0.080	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	18.00	554.30	1080.00	1080.00
2	3.50	543.55	1967.52	1967.52

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	16.32 Sett. (in.)	X = Stress (psf)	32.64 Sett. (in.)	X = Stress (psf)	48.96 Sett. (in.)
1	161.55	0.72	987.55	3.36	1947.19	5.34	2880.17	6.73
2	343.85	0.12	1024.86	0.31	1877.60	0.49	2659.62	0.62
		0.84		3.67		5.83		7.35

Layer	X = Stress (psf)	65.28 Sett. (in.)
1	3471.90	7.45
2	3132.20	0.69
		8.15

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

Ramp B 2609+57 Spill through slopes SJK 10-10-07 DAA 10-17-07

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name : SCI-823
File Name : RBS.emb
Date : 10/11/07

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

Settlement for X-Direction

Embank. slope, x direc. =	61.60 (ft)	Height of fill H	=	30.80 (ft)
y direc. =	61.60 (ft)	Unit weight of fill	=	120.00 (pcf)
Embankment top width =	40.00 (ft)	p load/unit area	=	3696.00 (psf)
Embankment bottom width =	163.20 (ft)	Foundation Elev.	=	563.30 (ft)
Ground Surface Elev. =	563.30 (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	COMP.	18.0	0.090	0.010	0.000	120.00	2.65	0.63
2	COMP.	3.5	0.080	0.080	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	18.00	554.30	1080.00	1080.00
2	3.50	543.55	1967.52	1967.52

Layer	X = Stress (psf)	0.00 Sett. (in.)	X = Stress (psf)	16.32 Sett. (in.)	X = Stress (psf)	32.64 Sett. (in.)	X = Stress (psf)	48.96 Sett. (in.)
1	162.95	0.73	990.79	3.37	1956.07	5.35	2915.84	6.78
2	356.61	0.12	1051.81	0.31	1938.74	0.50	2803.87	0.65
		------ 0.85		------ 3.68		------ 5.85		------ 7.42

Layer	X = Stress (psf)	65.28 Sett. (in.)
1	3609.61	7.61
2	3396.62	0.73
		------ 8.34

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

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

CLIENT CH2M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT Time - rate of Consolidation
Downdrag on piles - Ramp B

PROJECT NO. 0121-3070.03
SHEET NO. 38 OF 42
COMP. BY SML DATE 10-10-0
CHECKED BY DAA DATE 10-17-0

Forward Abutment Location
Station 2609+07 Profile based upon borings B-45 & B-1113

el. 563.3' Ex. Ground Surface
A-6b/A-2-6 $\overline{LL} \approx 26$
el. 545.3' $c_v \approx 0.65$ ft²/day
A-2-4 Assume Free Draining
el. 541.8'
TOP OF ROCK

Assume Double Drainage
$H_v = (563.3 - 545.3)/2 = 9$ ft.

Use $c_v = 0.65$ ft²/day

* Time to $U = 90\%$

$$t_{90} = \frac{T_v\, H_v^2}{c_v} = \frac{0.848\,(9')^2}{0.65\ \text{ft}^2/\text{day}} = 106 \text{ days}$$

* Downdrag at Forward Abutment

To prevent downdrag forces from reducing the allowable capacity of the piles, the remaining settlement should be limited to 0.4 inches or less prior to driving piles.

Settlement at ℄ of forward abutment, $\delta = 8.2''$
Of the 8.2", 7.5" is consolidation settlement

$$\left(1 - \frac{0.4}{7.5}\right) = 0.95 \quad \text{OR} \quad U = 95\%$$

Prior to driving piles, $U = 95\%$

* Time to $U = 95\%$ for $U = 95\%$, $T_v = 1.15$

$$t_{95} = \frac{1.15\,(9')^2}{0.65\ \text{ft}^2/\text{day}} = 143 \text{ days}$$

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2609+07**
Reference: FHWA-RD-86-168

SAK 10-10-07
DAA 10-17-07

Wick Drain Spacing **5.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	5.25	**0.65**	**9**	**7.5**
5	0.1179	0.0401	0.46	0.21	57.3	4.3				
10	0.2358	0.0802	0.71	0.31	80.1	6.0				
15	0.3537	0.1204	0.84	0.40	90.4	6.8				
20	0.4717	0.1605	0.90	0.47	94.8	7.1				
25	0.5896	0.2006	0.93	0.53	96.9	7.3				
30	0.7075	0.2407	0.96	0.58	98.3	7.4				
35	0.8254	0.2809	0.98	0.62	99.3	7.4				
40	0.9433	0.3210	0.99	0.65	99.7	7.5				
45	1.0612	0.3611	0.96	0.68	98.8	7.4				

Assumes a Triangular Grid Layout

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2609+07**
Reference: FHWA-RD-86-168

Wick Drain Spacing **7.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	7.35	**0.65**	**9**	7.5
5	0.0602	0.0401	0.28	0.21	42.5	3.2				
10	0.1203	0.0802	0.47	0.31	63.4	4.8				
15	0.1805	0.1204	0.61	0.40	76.7	5.8				
20	0.2406	0.1605	0.72	0.47	85.1	6.4				
25	0.3008	0.2006	0.79	0.53	90.3	6.8				
30	0.3610	0.2407	0.84	0.58	93.5	7.0				
35	0.4211	0.2809	0.88	0.62	95.5	7.2				
40	0.4813	0.3210	0.91	0.65	96.7	7.3				
45	0.5414	0.3611	0.92	0.68	97.5	7.3				
50	0.6016	0.4012	0.94	0.70	98.1	7.4				
55	0.6618	0.4414	0.95	0.72	98.6	7.4				
60	0.7219	0.4815	0.96	0.74	99.0	7.4				

Assumes a Triangular Grid Layout

DLZ

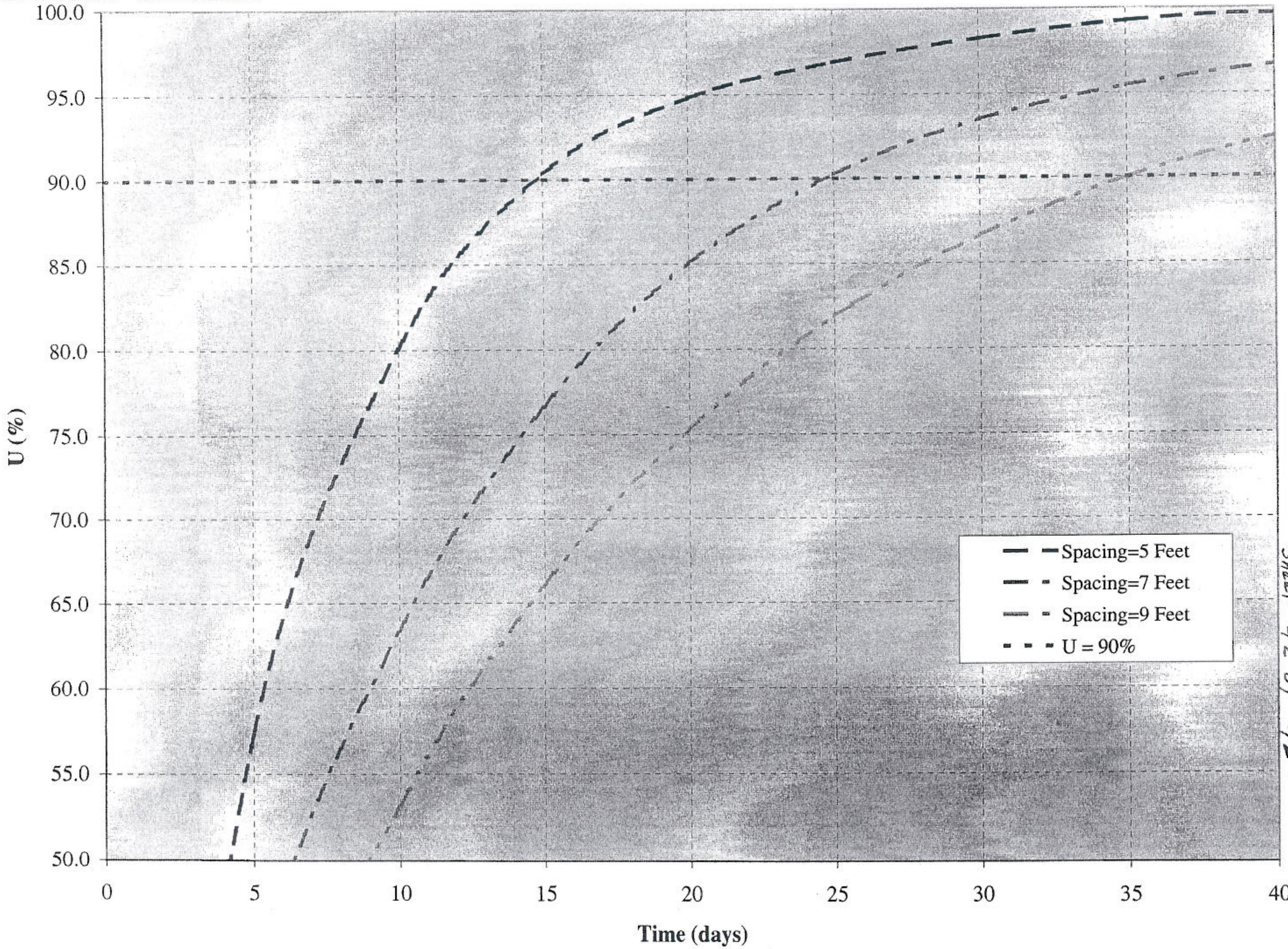
Time Rate of Consolication of Foundation Soils with Wick Drians
US 23 Ramp B **Station** **2609+07**
Reference: FHWA-RD-86-168

Wick Drain Spacing **9.0** feet Use $\eta = 10$

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	9.45	0.65	9	7.5
5	0.0364	0.0401	0.18	0.21	35.3	2.6				
10	0.0728	0.0802	0.32	0.31	53.2	4.0				
15	0.1092	0.1204	0.44	0.40	66.1	5.0				
20	0.1456	0.1605	0.53	0.47	75.3	5.7				
25	0.1820	0.2006	0.62	0.53	81.9	6.1				
30	0.2184	0.2407	0.68	0.58	86.6	6.5				
35	0.2548	0.2809	0.74	0.62	90.0	6.7				
40	0.2911	0.3210	0.78	0.65	92.4	6.9				
45	0.3275	0.3611	0.82	0.68	94.2	7.1				
50	0.3639	0.4012	0.85	0.70	95.5	7.2				
55	0.4003	0.4414	0.87	0.72	96.4	7.2				
60	0.4367	0.4815	0.89	0.74	97.1	7.3				
65	0.4731	0.5216	0.90	0.76	97.7	7.3				

Assumes a Triangular Grid Layout

Percent Consolidation vs Time Using Prefabricated Vertical "Wick" Drains
US-23 Interchange, Ramp B. Roadway Embankment-Sta. 2609+07



Sheet 42 of 42

APPENDIX D

SCI-823-10.13
RAMP B OVER NORFOLK SOUTHERN TRACKS
VERTICAL CLEARANCES

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1598C Ramp B over Railroad\Report\[Ramp B_RR_Vert_Clr.xls]Vertical Clearance
By: SCJ Date: 08/16/07
Checked: SKT Date: 11/20/2007

LEGEND:
User Input - Not Critical
User Input - Critical to Output

80" Curved Steel Plate Girder

PROFILE DATA - NORFOLK SOUTHERN TRACKS

Use existing top of high rail elevations, as profile adjustments to the railroad are not anticipated in this project.

POINT	RAILROAD LOCATION		RAILROAD STATION	RAILROAD - EXISTING ELEV. @ POINT
1	Top of Rail	West	n/a	550.97
2	Top of Rail	West	n/a	551.00
3	Top of Rail	East	n/a	551.98
4	Top of Rail	East	n/a	552.01

PROFILE DATA - RAMP B

Linear: PVT Sta. 2600+75.00; PVT Elev. 552.33; g 5.76%; PVC Sta. 2605+75.00; PVC Elev. 581.13

Vertical Curve: PVC Sta. 2605+75.00; PVC Elev. 581.13; g1 5.76%; g2 0.93%; LVC 250; PVI Sta. 2607+00.00; PVI Elev. 588.33; PVT Sta. 2608+25.00; PVT Elev. 589.49

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

Superelevation Data:

Station	Left Shoulder	Pavement	Right Shoulder
2603+79.13	-4.0%	7.1%	-7.1%
2611+95.54	-4.0%	7.1%	-7.1%

POINT	RAMP B LOCATION			RAMP B	LT.	PVMT X-	RT.	RAMP B -
	LOCATION	STA.	OFF.*	PG ELEV.	SHOULDER	SLOPE	SHOULDER	FINISHED
1	RT. EXT. GIRDER	2606+08.17	6.50	582.93	-4.0%	7.1%	-7.1%	582.47
2	RT. EXT. GIRDER	2606+34.58	6.50	584.22	-4.0%	7.1%	-7.1%	583.76
3	RT. EXT. GIRDER	2606+80.15	6.50	586.12	-4.0%	7.1%	-7.1%	585.66
4	RT. EXT. GIRDER	2607+02.09	6.50	586.89	-4.0%	7.1%	-7.1%	586.43

* 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

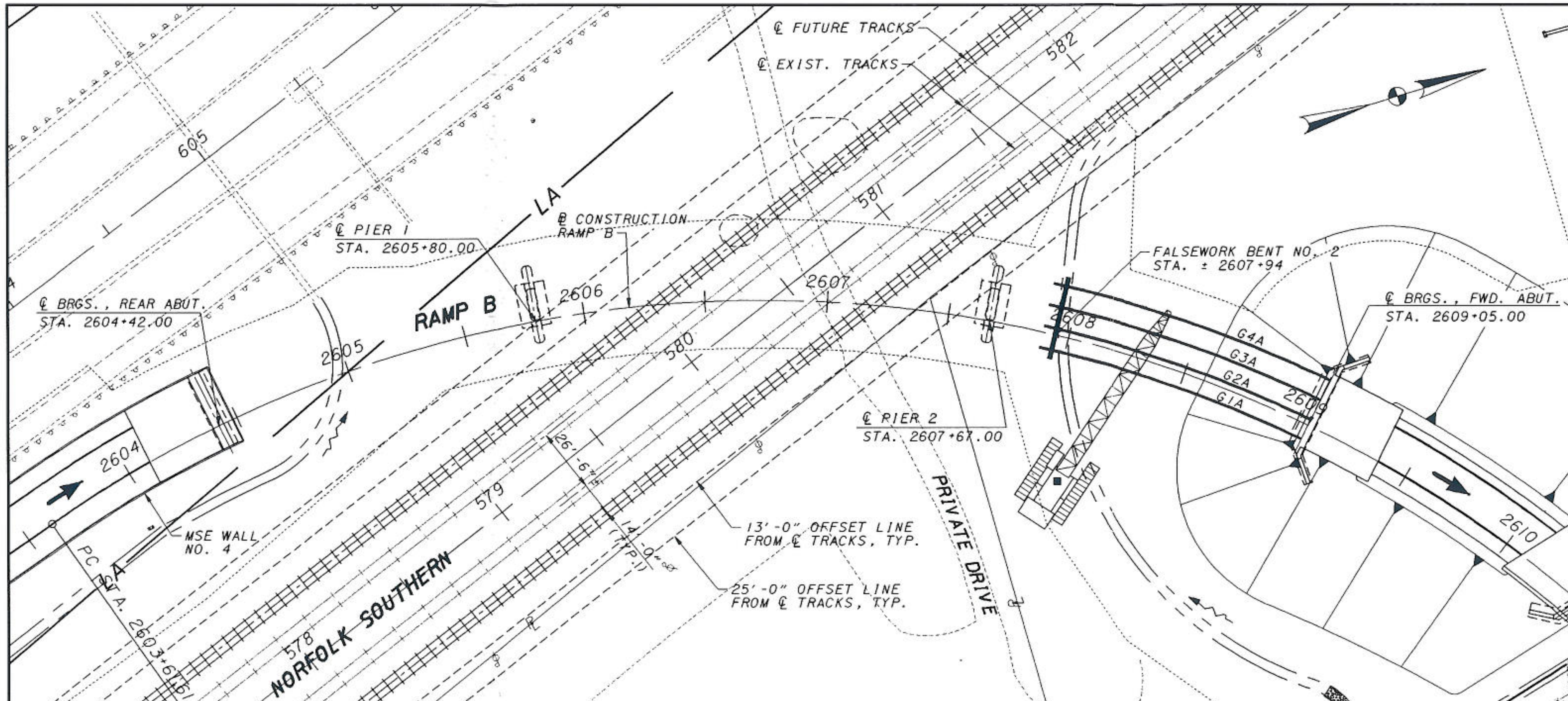
POINT	GIRDER DESCRIPTION	Slab	Haunch	Top Flange	Web	Bot. Flange	Splice	Total (inches)
1	PLATE GIRDER	8.50	2.50	1.5	80	2.00	-	94.50
2	PLATE GIRDER	8.50	2.50	1.5	80	2.00	1.50	96.00
3	PLATE GIRDER	8.50	2.50	1.5	80	2.00	-	94.50
4	PLATE GIRDER	8.50	2.50	1.5	80	2.00	-	94.50

VERTICAL CLEARANCE - RAMP B OVER NORFOLK SOUTHERN TRACKS

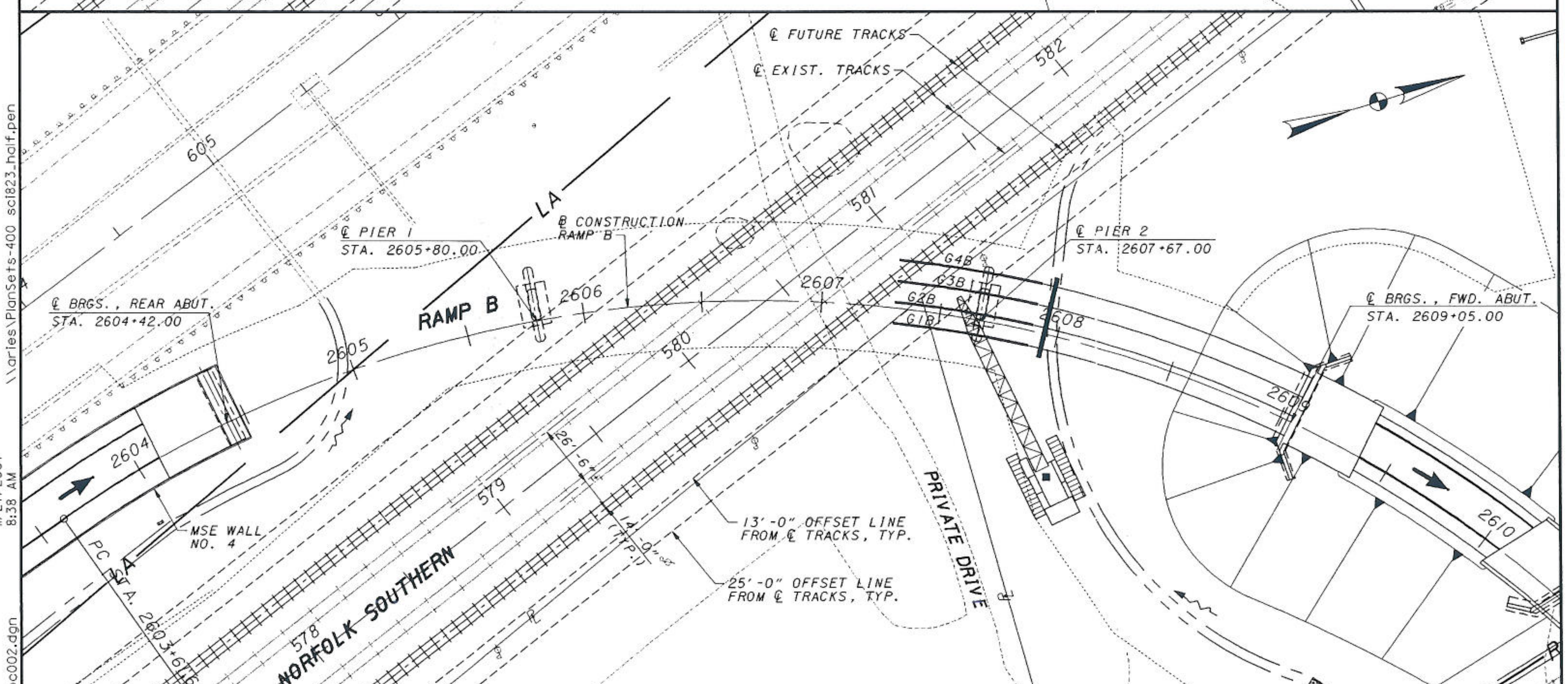
POINT	LOCATION	RAMP B - FINISHED GRADE @ POINT	STRUC. DEPTH (in.)	BOT. GIRDER ELEV.	RAILROAD - FINISHED GRADE @ POINT	VERTICAL CLEARANCE (ft.)	CHECK MINIMUM REQUIRED VERTICAL CLEARANCE *	
1	RT. EXT. GIRDER	582.47	94.500	574.60	550.97	**23.63**	OK	MIN. VERT. CLR =
2	RT. EXT. GIRDER	583.76	96.000	575.76	551.00	**24.76**	OK	23.25'
3	RT. EXT. GIRDER	585.66	94.500	577.78	551.98	**25.80**	OK	MIN. VERT. CLR =
4	RT. EXT. GIRDER	586.43	94.500	578.55	552.01	**26.54**	OK	23.18'

* REQUIRED MINIMUM VERTICAL CLEARANCE OVER RR WAS INCREASED ABOVE 23'-0" TO ALLOW FOR REMOVAL OF APPARENT SETTELEMENT OF EXISTING TRACK.

APPENDIX E



PLAN - ERECTION STAGE A



PLAN - ERECTION STAGE B

GENERAL NOTES:

1. THE FOLLOWING ASSUMPTIONS WERE MADE IN PREPARATION OF THE PRELIMINARY ERECTION PLAN:
   A. THE TWO FUTURE NORFOLK SOUTHERN TRACKS WILL HAVE BEEN CONSTRUCTED BY THE TIME STEEL ERECTION OCCURS;
   B. THE EXISTING RAILROAD COMMUNICATION LINES WILL LIKELY BE RELOCATED IF THE TWO ADDITIONAL TRACKS ARE CONSTRUCTED AND WERE NOT CONSIDERED IN DEVELOPING THE PRELIMINARY ERECTION PROCEDURE.
2. PER NORFOLK SOUTHERN PUBLICATION "OVERHEAD GRADE SEPERATION DESIGN CRITERIA"
   A. MINIMUM TEMPORARY HORIZONTAL CLEARANCE IS 13'-0".
   B. PERMANENT HORIZONTAL CLEARANCE IS 25'-0".
   C. CRANES MUST BE ADEQUATE FOR 150% THE WEIGHT OF THE PICK.
3. A MANITOWOC MODEL 999 CRANE (275 TON) WAS ASSUMED FOR THE DEVELOPMENT OF THIS PLAN. ERECTION OF THE GIRDERS OVER THE NORFOLK SOUTHERN TRACKS (ERECTION STAGE E) CONTROLS THE CAPACITY OF THE CRANE SELECTED. STAGES A, B, C, AND D COULD UTILIZE A SMALLER CRANE AT THE ERECTORS DISCRETION.
4. CROSS FRAMES/LATERAL BRACING ARE NOT SHOWN ON THE DRAWINGS, BUT IT IS ASSUMED THAT THEY WILL BE INSTALLED AS ERECTION PROCEEDS TO BRACE THE GIRDERS.
5. SITE PREPARATION NECESSARY FOR THE CRANE AND TO PROVIDE ACCESS FOR MATERIALS ARE NOT SHOWN ON THE DRAWINGS.
6. THE ERECTION SEQUENCE SHOWN ON THE DRAWINGS IS NOT INTENDED FOR CONSTRUCTION. ACTUAL ERECTION METHODS AND PROCEDURES TO BE DETERMINED BY THE CONTRACTOR.
7. THE LOCATION OF FALSEWORK BENTS IS SUBJECT TO CHANGE BASED ON FINAL DESIGN LOCATION OF FIELD SPLICE POINTS.

ERECTION NOTES - STAGE A:

1. DUE TO THE LOAD CAPACITY OF THE CRANE, IT MAY BE POSITIONED AT ALMOST ANY POSITION ON THE EAST SIDE OF THE BRIDGE TO ERECT STAGE A GIRDERS.
2. THE LENGTH OF GIRDER G4A IS ±120 FEET WITH AN ESTIMATED MAXIMUM LIFTING WEIGHT OF 50,000 POUNDS. THE OTHER GIRDERS ARE SHORTER AND LIGHTER.
3. THE ERECTION PROCEDURE ASSUMES THAT EACH GIRDER WILL BE ERECTED SEPARATELY. HOWEVER, DUE TO THE CAPACITY OF THE CRANE, THE ERECTOR COULD INSTALL CROSS-FRAMES AND BOLT TWO GIRDERS TOGETHER ON THE GROUND AND LIFT THEM INTO PLACE AS A UNIT.
4. BEARINGS TO BE WELDED TO GIRDERS AS THE GIRDERS ARE SET.
5. THE SUGGESTED ERECTION SEQUENCE IS:
   A. DESIGN AND CONSTRUCT FALSEWORK BENT NO. 2;
   B. SET BEARINGS AT FORWARD ABUTMENT;
   C. ERECT GIRDERS AND CROSS-FRAMES.

ERECTION NOTES - STAGE B:

1. DUE TO THE LOAD CAPACITY OF THE CRANE, IT MAY BE POSITIONED AT ALMOST ANY POSITION ON THE EAST SIDE OF THE BRIDGE TO ERECT STAGE B GIRDERS.
2. THE LENGTH OF GIRDER G4B IS ±61 FEET WITH AN ESTIMATED MAXIMUM LIFTING WEIGHT OF 35,000 POUNDS. THE OTHER GIRDERS ARE SHORTER AND LIGHTER.
3. THE ERECTION PROCEDURE ASSUMES THAT EACH GIRDER WILL BE ERECTED SEPARATELY. HOWEVER, DUE TO THE CAPACITY OF THE CRANE, THE ERECTOR COULD INSTALL CROSS-FRAMES AND BOLT TWO GIRDERS TOGETHER ON THE GROUND AND LIFT THEM INTO PLACE AS A UNIT.
4. BEARINGS TO BE WELDED TO GIRDERS AS THE GIRDERS ARE SET.
5. MINIMUM 50% OF FIELD SPLICE HOLES TO BE FILLED WITH BOLTS. NUMBER OF HOLES TO BE FILLED TO BE DETERMINED BY CONTRACTOR.
6. THE SUGGESTED ERECTION SEQUENCE IS:
   A. ERECT BEARINGS ON PIER NO. 2;
   B. ERECT GIRDERS AND CROSS-FRAMES.

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SCJ		7306776	

ERECTION SEQUENCE PLAN

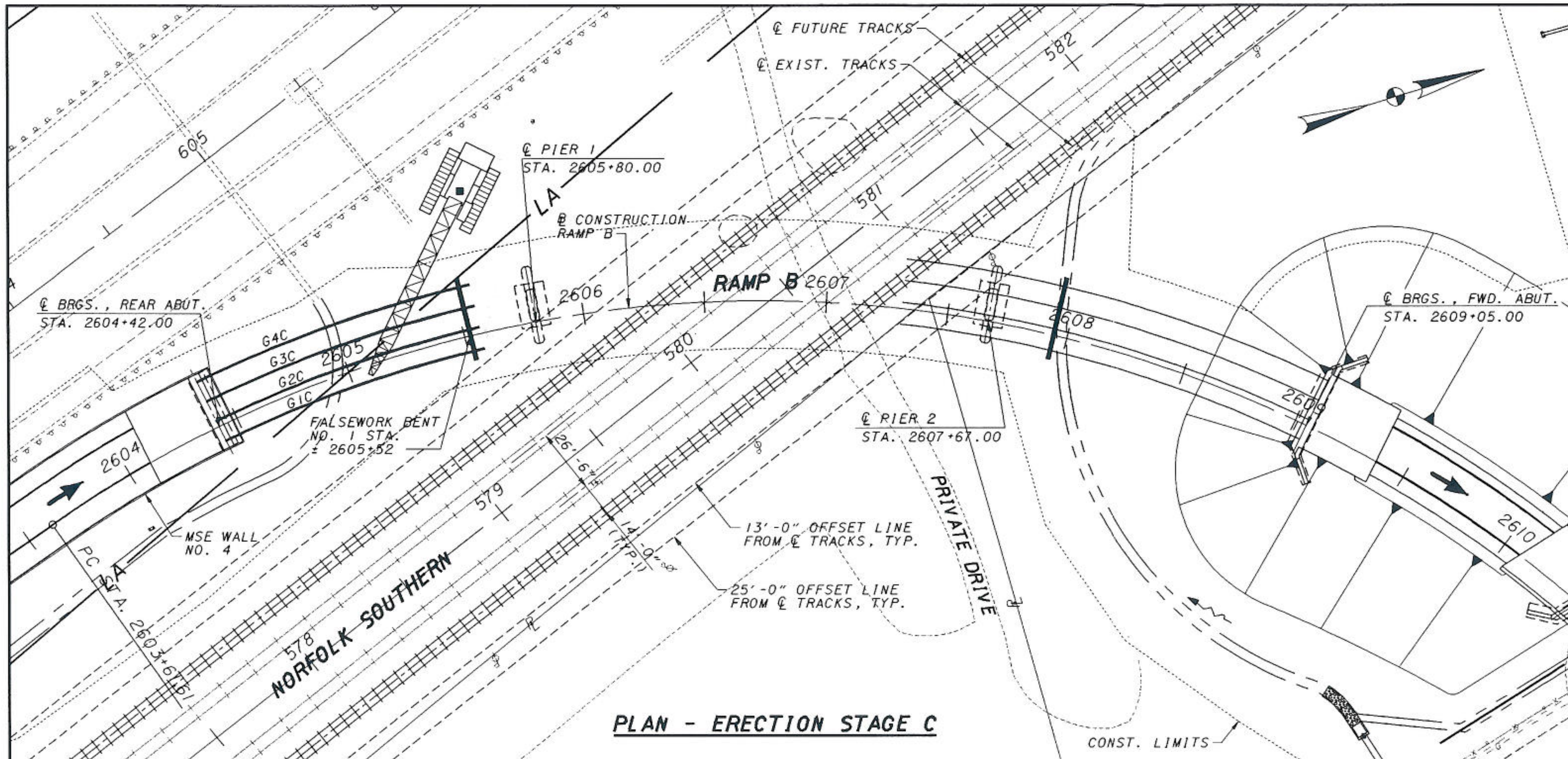
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

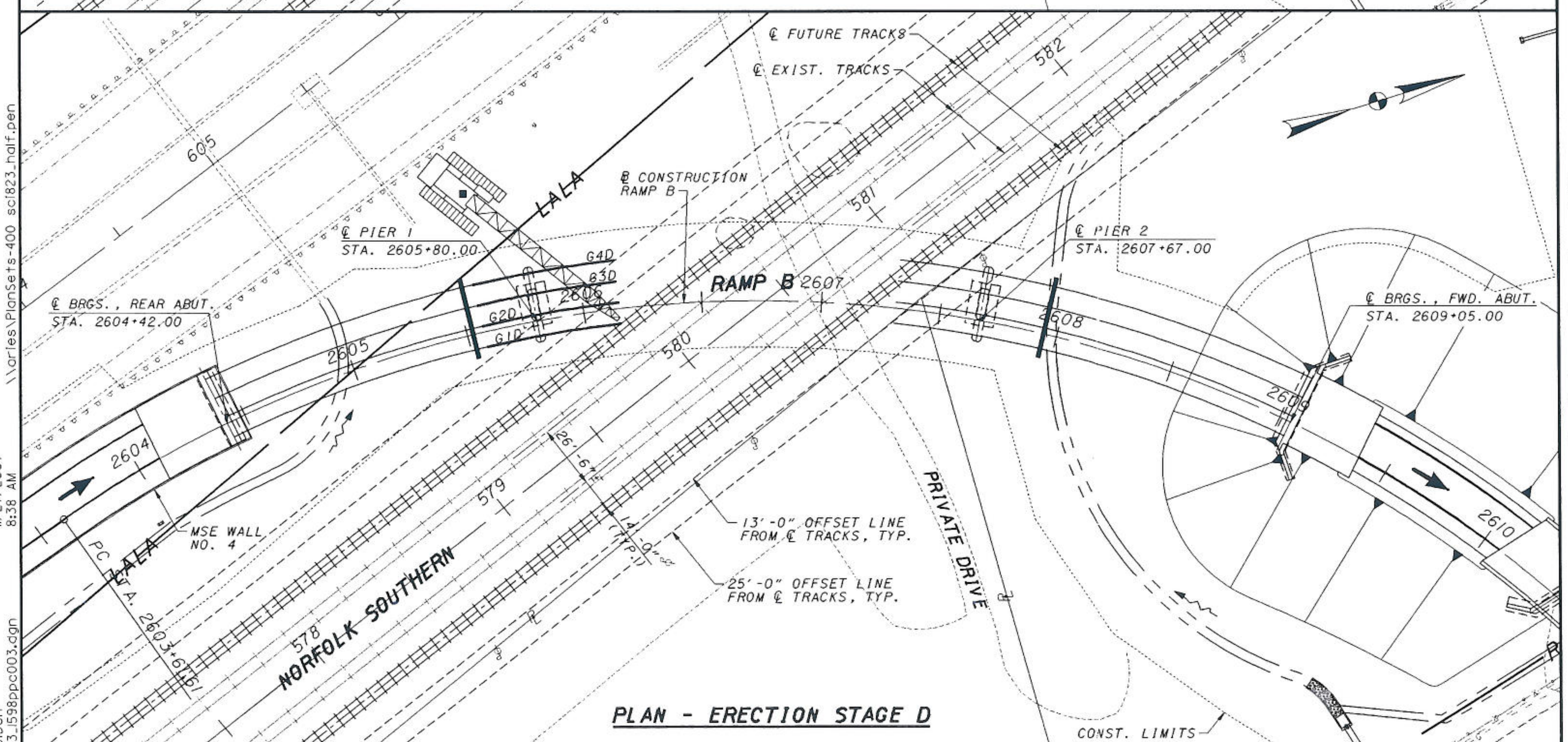
1 / 5

836

\\arles\PlanSets-400 sci823_half.pen
11/27/2007
8:38 AM
melben
823_1598ppc002.dgn



PLAN - ERECTION STAGE C



PLAN - ERECTION STAGE D

ERECTION NOTES - STAGE C:

1. DUE TO THE LOAD CAPACITY OF THE CRANE, IT MAY BE POSITIONED AT ALMOST ANY POSITION ON THE WEST SIDE OF THE BRIDGE TO ERECT STAGE C GIRDERS.
2. THE LENGTH OF GIRDER G4C IS ±120 FEET WITH AN ESTIMATED MAXIMUM LIFTING WEIGHT OF 50,000 POUNDS. THE OTHER GIRDERS ARE SHORTER AND LIGHTER.
3. THE ERECTION PROCEDURE ASSUMES THAT EACH GIRDER WILL BE ERECTED SEPARATELY. HOWEVER, DUE TO THE CAPACITY OF THE CRANE, THE ERECTOR COULD INSTALL CROSS-FRAMES AND BOLT TWO GIRDERS TOGETHER ON THE GROUND AND LIFT THEM INTO PLACE AS A UNIT.
4. BEARINGS TO BE WELDED TO GIRDERS AS THE GIRDERS ARE SET.
5. THE SUGGESTED ERECTION SEQUENCE IS:
   A. DESIGN AND CONSTRUCT FALSEWORK BENT NO. 1;
   B. SET BEARINGS AT REAR ABUTMENT;
   C. ERECT GIRDERS AND CROSS-FRAMES.

ERECTION NOTES - STAGE D:

1. DUE TO THE LOAD CAPACITY OF THE CRANE, IT MAY BE POSITIONED AT ALMOST ANY POSITION ON THE WEST SIDE OF THE BRIDGE TO ERECT STAGE D GIRDERS.
2. THE LENGTH OF GIRDER G4D IS ±61 FEET WITH AN ESTIMATED MAXIMUM LIFTING WEIGHT OF 35,000 POUNDS. THE OTHER GIRDERS ARE SHORTER AND LIGHTER.
3. THE ERECTION PROCEDURE ASSUMES THAT EACH GIRDER WILL BE ERECTED SEPARATELY. HOWEVER, DUE TO THE CAPACITY OF THE CRANE, THE ERECTOR COULD INSTALL CROSS-FRAMES AND BOLT TWO GIRDERS TOGETHER ON THE GROUND AND LIFT LEFT THEM INTO PLACE AS A UNIT.
4. BEARINGS TO BE WELDED TO GIRDERS AS THE GIRDERS ARE SET.
5. MINIMUM 50% OF FIELD SPLICE HOLES TO BE FILLED WITH BOLTS. NUMBER OF HOLES TO BE FILLED TO BE DETERMINED BY CONTRACTOR.
6. THE SUGGESTED ERECTION SEQUENCE IS:
   A. ERECT BEARINGS ON PIER NO. 1;
   B. ERECT GIRDERS AND CROSS-FRAMES.

DESIGN AGENCY	DESIGNED	DRAWN	REVIEWED	DATE
CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	DGS	JBA	VKN	11/07
	CHECKED	REVISED	STRUCTURE FILE NUMBER	
	SCJ		7306776	

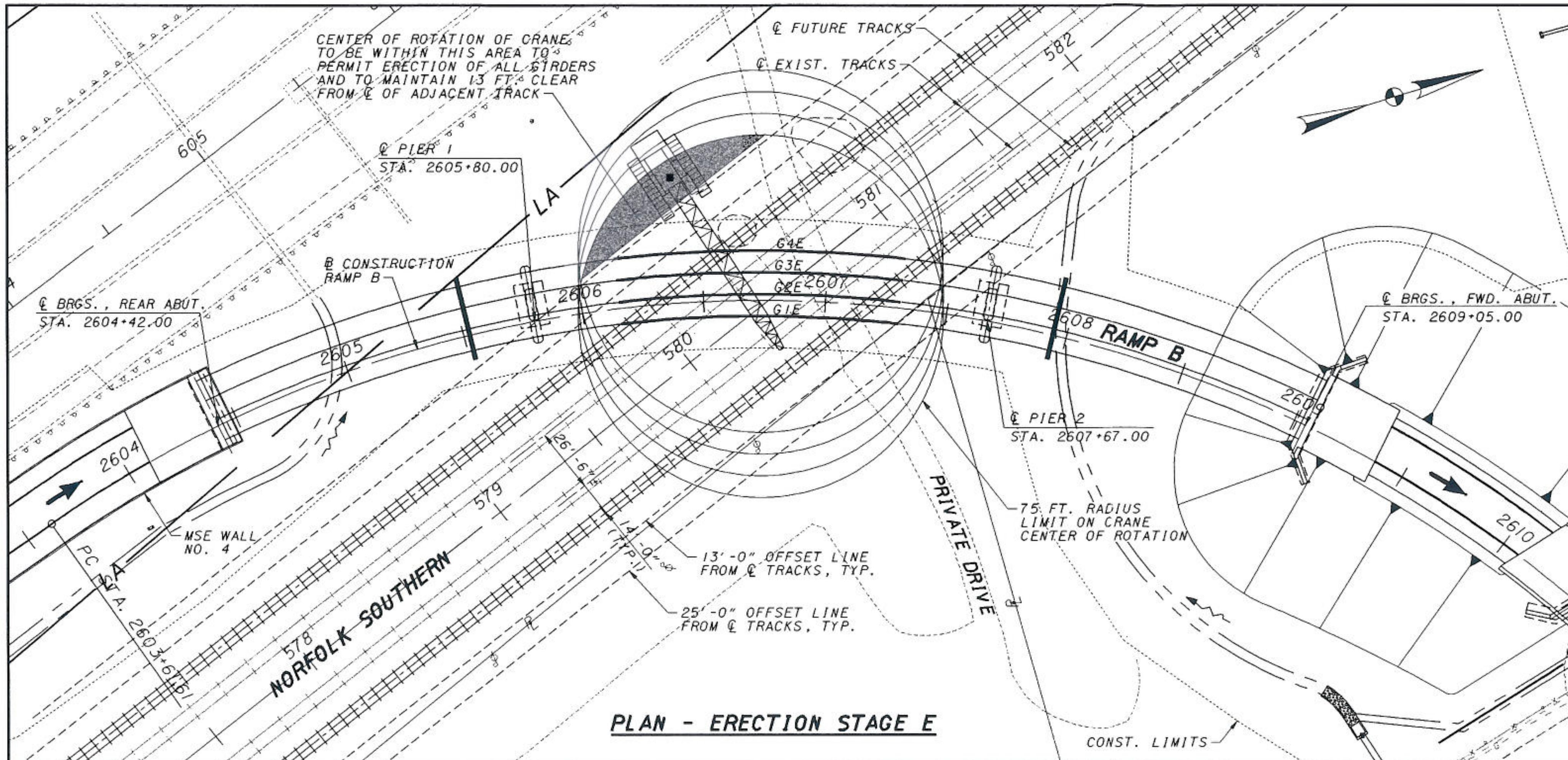
ERECTION SEQUENCE PLAN
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

2/5

837

melben 823_1598ppc003.dgn 11/27/2007 8:38 AM \\orles\PlanSets-400 sci823_half.pen



PLAN - ERECTION STAGE E

ERECTION NOTES - STAGE E:

1. A REVIEW OF THE HEAVY LIFT DATA TABLES FOR A MANITOWOC MODEL 999 CRANE INDICATES THAT IT CAN LIFT 75,000 POUNDS (1.5 TIMES ACTUAL WEIGHT) AT A 75' OPERATING RADIUS. SUFFICIENT, BUT LIMITED, SPACE IS AVAILABLE FOR THE LIFT WHILE STILL MAINTAINING A MINIMUM 13' CLEAR ZONE FROM THE CENTERLINE OF THE ADJACENT PROPOSED NORFOLK SOUTHERN TRACK.
2. THE LENGTH OF GIRDER G4E IS ±120 FEET WITH AN ESTIMATED MAXIMUM LIFTING WEIGHT OF 50,000 POUNDS. THE OTHER GIRDERS ARE SHORTER AND LIGHTER.
3. THE ERECTION PROCEDURE ASSUMES THAT EACH GIRDER WILL BE ERECTED SEPARATELY.
4. MINIMUM 50% OF FIELD SPLICE HOLES TO BE FILLED WITH BOLTS. NUMBER OF HOLES TO BE FILLED TO BE DETERMINED BY CONTRACTOR.
5. THE SUGGESTED ERECTION SEQUENCE IS:
   A. ERECT GIRDERS AND CROSS-FRAMES AND COMPLETE ALL FIELD SPLICES;
   B. REMOVE FALSEWORK BENTS.

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SCJ		7306776	

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

ERECTION SEQUENCE PLAN
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

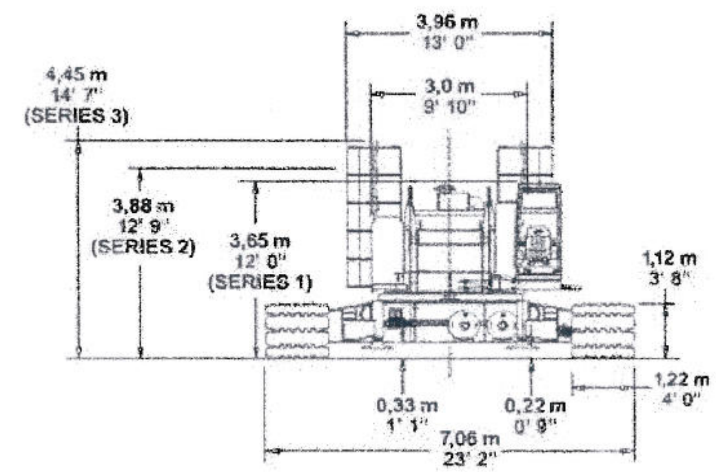
SCI-823-10.13
PID 79977

3/5

838

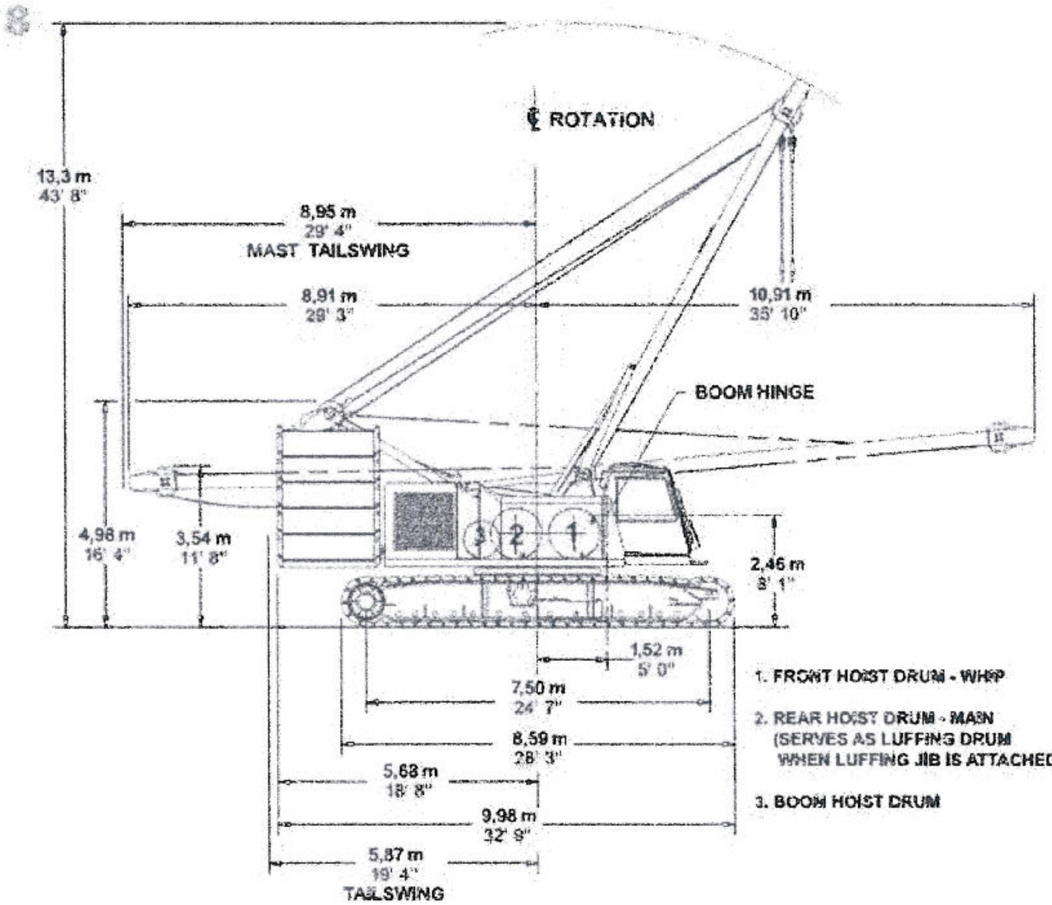
melben
823_1598ppc004.dgn
11/27/2007
8:38 AM
\\aries\PlanSets-400 sci823_half.pen

outline dimensions



model 999

outline dimensions



1. FRONT HOIST DRUM - WHIP
2. REAR HOIST DRUM - MAIN (SERVES AS LUFFING DRUM WHEN LUFFING JIB IS ATTACHED)
3. BOOM HOIST DRUM

model 999

CRANE DATA - MANITOWOC MODEL 999
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SCJ		7306776	

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

SCI-823-10.13
PID 79977

4 / 5

839

heavy-lift boom load chart

Liftcrane Boom Capacities - Series 3
Boom No. 82 or No. 22EL

99 609 kg (219,600 lb) Upperworks Counterweight 36 288 kg (80,000 lb) Carbody Counterweight

360° Rating kg (lb) x 1 000

Boom m (ft) / Radius	21,3 (70)	27,4 (90)	33,5 (110)	39,6 (130)	45,7 (150)	51,8 (170)	57,9 (190)	64,0 (210)	70,1 (230)	76,2 (250)	82,3 (270)	85,4 (290)
4,6 (15)	250,0* (551.0)*											
5,6 (17)	219,9 (462.5)											
6,0 (20)	183,0 (397.5)	180,7 (398.4)										
8,0 (26)	139,0 (309.3)	126,5 (308.3)	124,1 (307.5)	133,3 (295.0)	120,1 (265.5)							
10,0 (32)	111,5 (252.2)	111,1 (251.2)	110,8 (250.5)	110,5 (249.9)	110,6 (248.9)	100,8 (224.1)	88,6 (196.8)					
11,0 (36)	100,2 (221.8)	100,3 (222.0)	100,4 (222.1)	100,3 (221.9)	99,9 (221.0)	97,6 (215.7)	86,3 (190.5)	76,7 (169.2)	63,1 (139.3)			
12,0 (40)	88,2 (190.3)	88,3 (190.4)	88,3 (190.4)	88,3 (190.4)	88,2 (190.0)	88,3 (189.7)	81,4 (177.0)	74,9 (164.5)	62,8 (138.4)	51,8 (114.1)	- (95.1)	
14,0 (46)	70,9 (156.1)	70,9 (156.2)	70,9 (156.1)	70,9 (156.0)	70,6 (155.5)	70,5 (155.1)	70,3 (154.6)	67,9 (149.6)	62,1 (137.1)	51,2 (113.0)	42,6 (94.1)	35,3 (77.9)
15,0 (50)	64,4 (139.0)	64,4 (139.0)	64,4 (138.9)	64,3 (138.8)	64,1 (138.3)	63,9 (137.9)	63,6 (137.3)	63,6 (136.7)	60,7 (132.5)	50,9 (112.2)	42,3 (93.3)	35,0 (77.2)
18,0 (60)	50,1 (108.2)	50,1 (108.1)	50,0 (108.0)	49,9 (107.8)	49,6 (107.2)	49,5 (106.8)	49,1 (106.1)	48,9 (105.5)	48,5 (104.7)	48,2 (104.7)	41,5 (91.4)	34,3 (75.5)
24,0 (80)		33,6 (72.6)	33,6 (72.5)	33,5 (72.3)	33,2 (71.7)	33,0 (71.2)	32,6 (70.4)	32,3 (69.8)	31,9 (68.9)	31,9 (68.9)	31,5 (68.0)	31,3 (67.2)
30,0 (100)			24,4 (52.7)	24,3 (52.5)	24,1 (51.9)	23,8 (51.4)	23,5 (50.6)	23,1 (49.9)	22,7 (49.0)	22,7 (48.9)	22,3 (48.0)	21,9 (47.2)
34,0 (110)			- (41.6)	20,2 (45.6)	19,9 (45.0)	19,7 (44.4)	19,3 (43.8)	19,0 (42.9)	18,6 (42.0)	18,6 (42.0)	18,1 (41.0)	17,7 (40.2)
36,0 (120)				18,5 (39.8)	18,2 (39.2)	18,0 (38.7)	17,6 (37.9)	17,3 (37.2)	16,9 (36.3)	16,8 (36.2)	16,4 (35.3)	16,0 (34.5)
40,0 (130)				- (30.9)	15,3 (34.5)	15,1 (34.0)	14,8 (33.2)	14,4 (32.4)	14,0 (31.6)	14,0 (31.5)	13,5 (30.5)	13,2 (29.7)
42,0 (140)					13,9 (29.3)	13,9 (29.0)	13,5 (29.1)	13,3 (28.4)	12,8 (27.5)	12,8 (27.4)	12,3 (26.5)	12,0 (25.6)
46,0 (150)					- (22.4)	11,7 (26.4)	11,5 (25.7)	11,1 (24.0)	10,7 (24.0)	10,7 (24.0)	10,2 (23.0)	9,9 (22.1)
50,0 (165)						8,7 (21.4)	9,7 (22.6)	9,4 (21.0)	9,0 (21.0)	8,9 (20.0)	8,5 (20.0)	8,1 (19.1)
52,0 (170)							8,5 (19.1)	8,6 (19.3)	8,2 (18.4)	8,2 (18.3)	7,8 (17.4)	7,4 (16.5)
58,0 (190)								5,8 (13.0)	6,1 (13.6)	6,2 (13.9)	5,8 (12.9)	5,4 (12.1)
64,0 (210)									3,4 (7.6)	4,1 (9.1)	3,9 (8.8)	3,8 (8.5)
70,0 (230)										1,8 (4.1)	1,9 (4.2)	1,8 (4.0)

*NOTE: Special equipment required.

Fixed Jib No. 134 on Heavy-Lift Boom No. 82 or No. 22EL

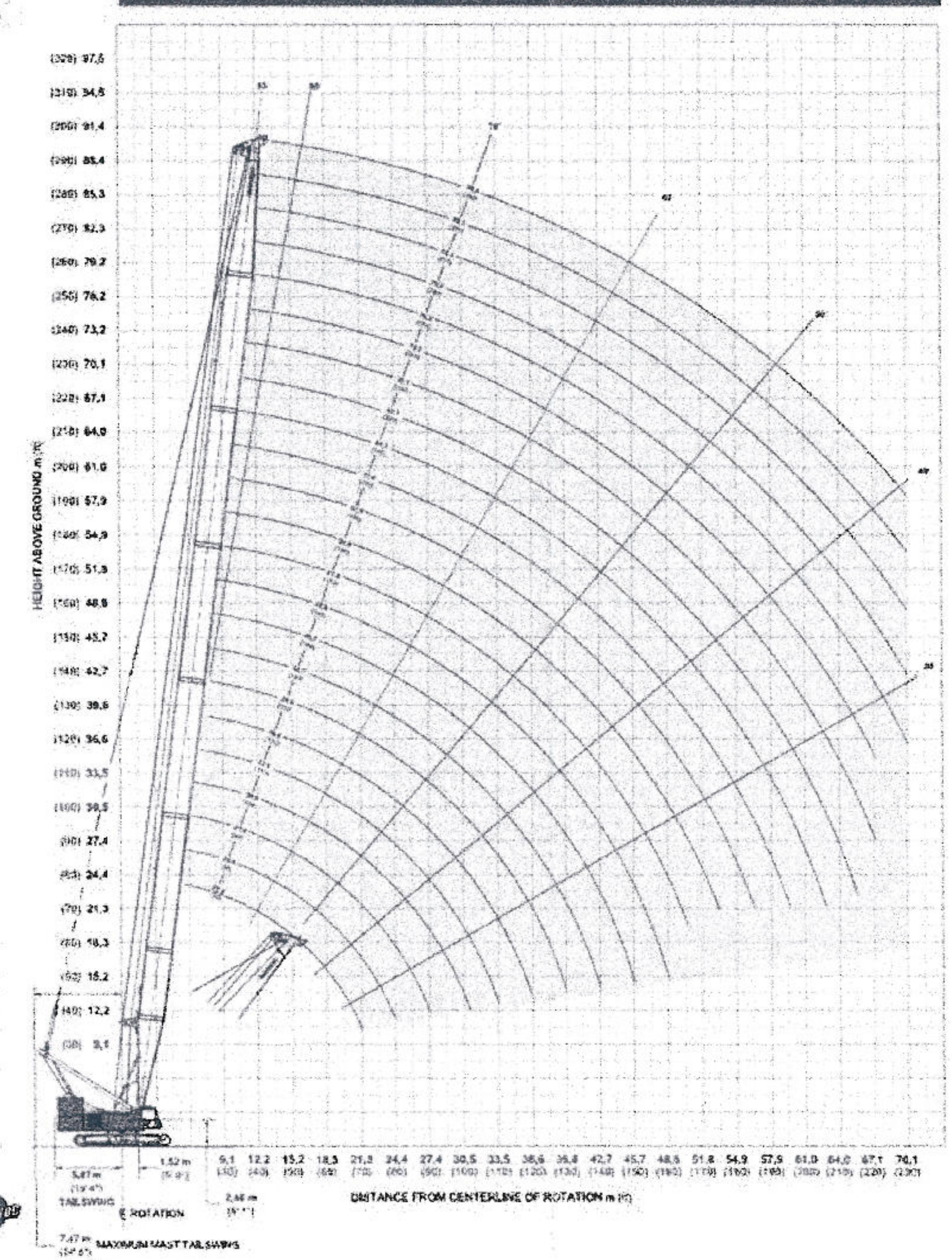
Jib Length m (ft)	Deduct from Capacity when fixed jib is attached kg (lb)
9,1 (30)	1 220 (2,700)
12,2 (40)	1 590 (3,500)
15,2 (50)	1 950 (4,300)
18,3 (60)	2 310 (5,100)
21,3 (70)	2 770 (6,100)
24,4 (80)	3 220 (7,100)

Meets ANSI B30.5 Requirements - Capacities do not exceed 75% of static tipping load.
NOTICE: This capacity chart is for reference only and must not be used for lifting purposes.

Manitowoc model 999

heavy-lift boom range diagram

No. 82 or No. 22EL Main Boom



Manitowoc model 999

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SCJ		7306776	

CRANE DATA - MANITOWOC MODEL 999
BRIDGE NO. SCI-823-1598
RAMP B OVER NORFOLK SOUTHERN

SCI-823-10.13
PID 79977

5 / 5

840

melben
823_1598ppc006.dgn
11/27/2007
8:39 AM
\\arles\PlanSets-400 sci823_half.pen

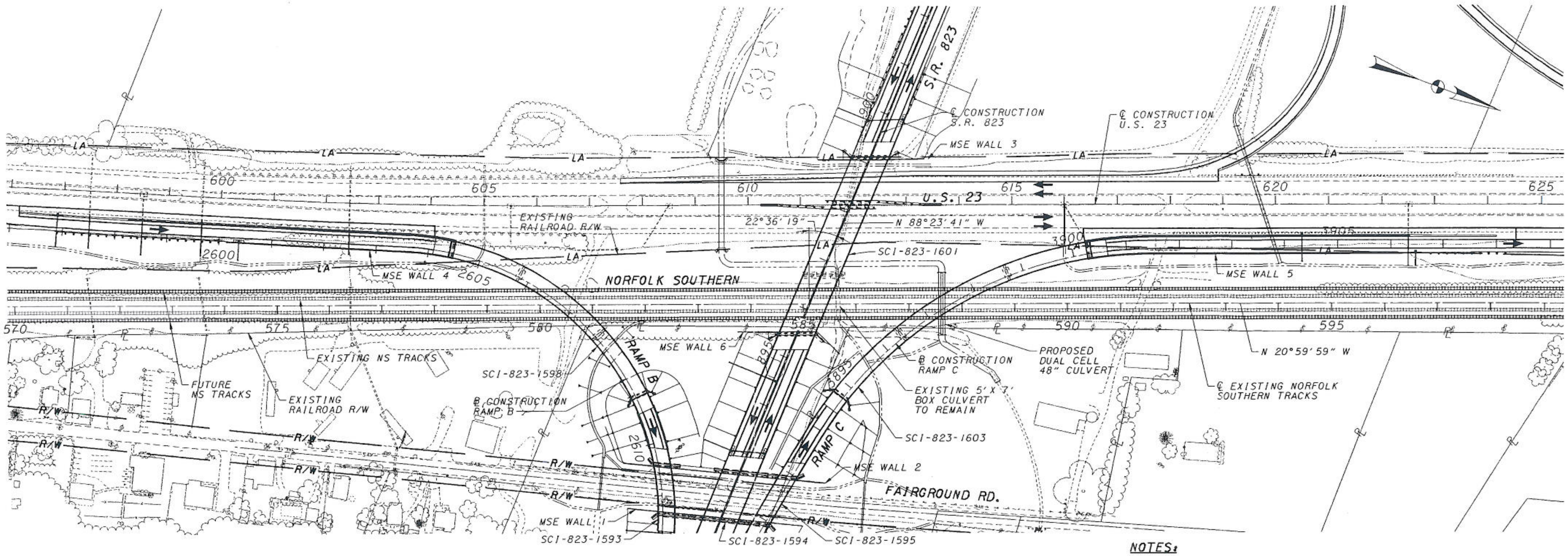
APPENDIX F

SUPPLEMENTAL SITE PLAN

BRIDGE NOS. SCI-823-1598, SCI-823-1601, SCI-823-1603
S.R. 823 INTERCHANGE OVER NORFOLK SOUTHERN TRACKS

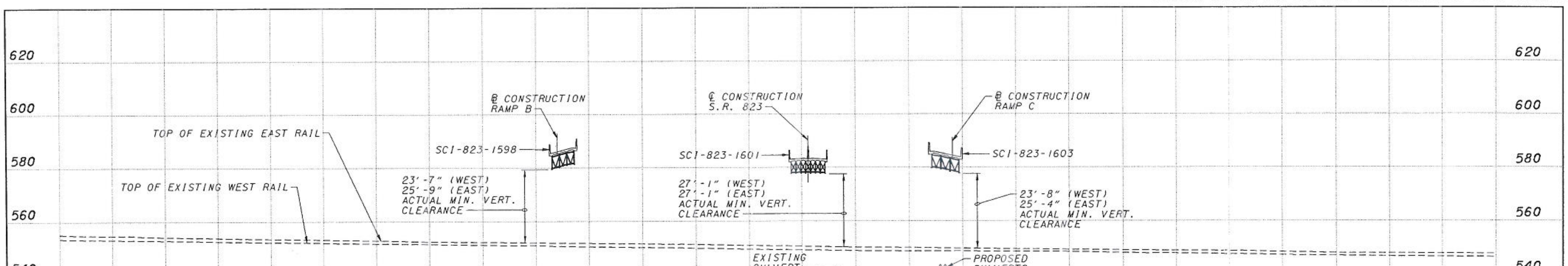
DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			



PLAN

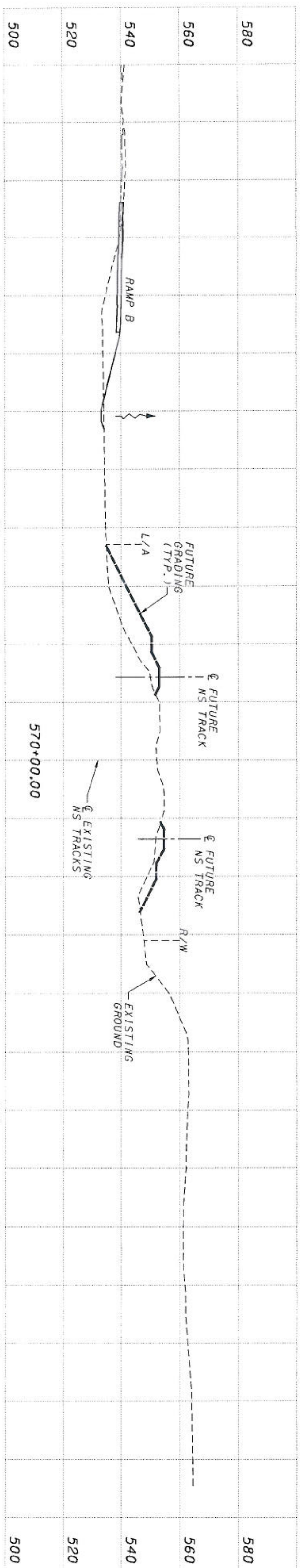
NOTES:
1. RAILROAD TRAFFIC COUNT: AVERAGE 25-35 FREIGHT TRAINS PER DAY (SPEED - 50 - 60 MPH)
2. RAILROAD STATION 585+00 - RR MP 618.51
3. MINIMUM STRUCTURE CLEARANCES SHOWN BELOW FOR THE WEST TRACK ARE LOCATED ALONG THE FUTURE WEST TRACK LOCATION.
4. PROPOSED DUAL CELL 48" CULVERT WILL BE JACKED UNDER THE EXISTING TRACKS. THE PROPOSED INVERT ELEVATION TO BE ±538.2'.

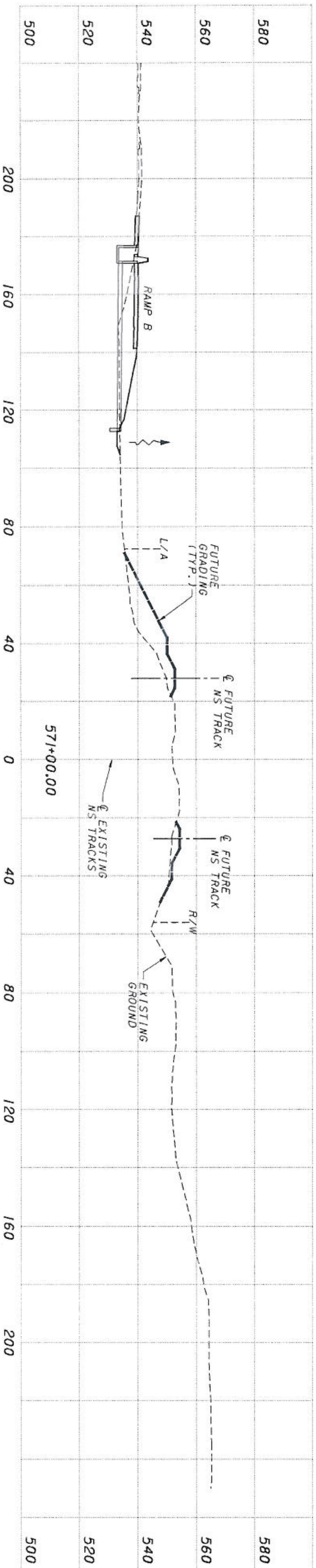


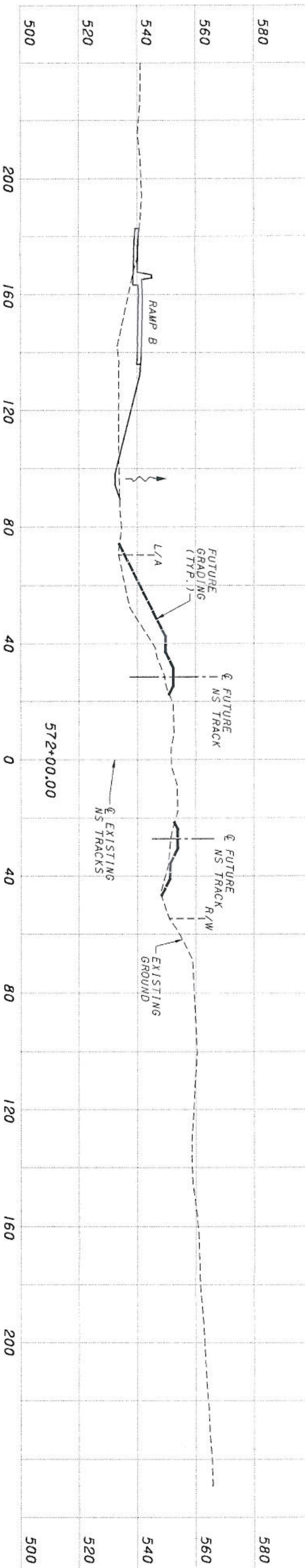
EXIST. TOP OF RAIL ELEV. WEST TRACK	552.94	552.66	552.38	552.06	551.71	551.49	551.18	550.92	550.47	550.18	549.85	549.46	549.14	548.88	548.72	548.72	548.49	548.40	548.07	547.85	547.53	547.22	546.97	546.77	546.59	546.49
EXIST. TOP OF RAIL ELEV. EAST TRACK	554.26	553.87	553.53	553.25	552.80	552.50	552.28	552.07	551.88	551.52	551.17	550.82	550.54	550.27	549.98	549.69	549.42	549.26	549.14	549.01	548.69	548.35	547.98	547.89	547.73	547.61

575+00 580+00 585+00 590+00 595+00

RAIL PROFILES

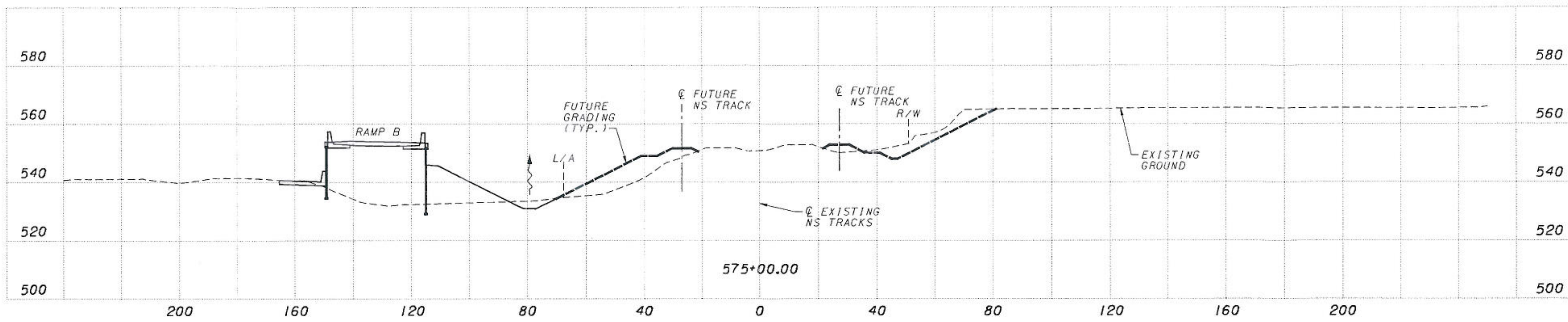


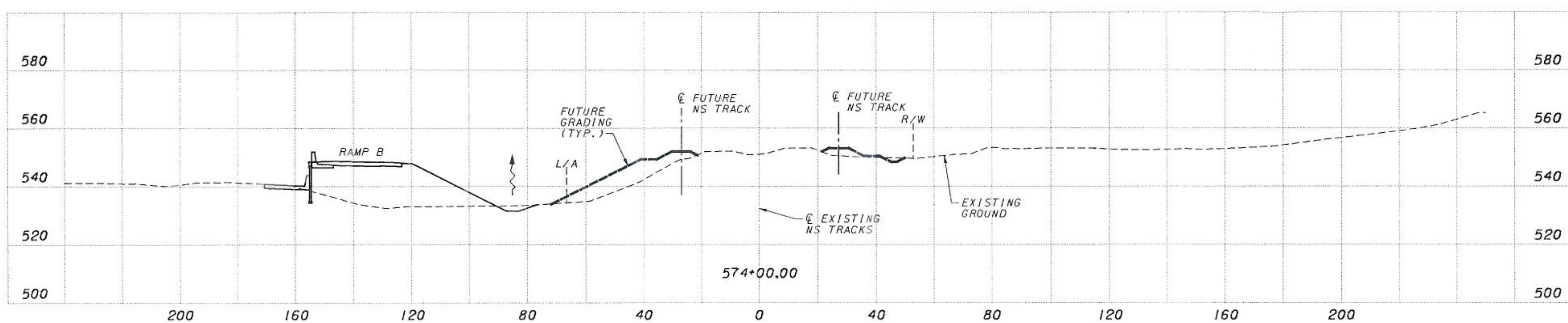


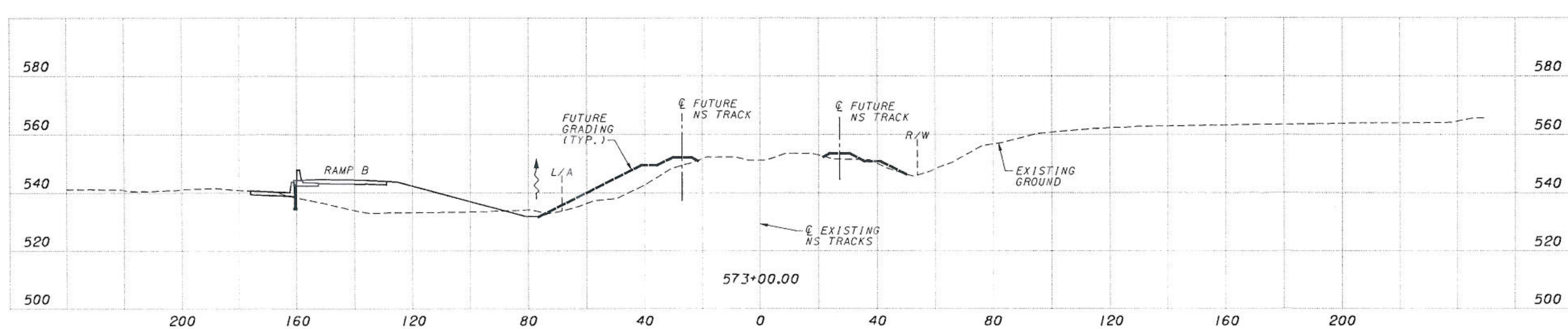


855	2/11	SCI-823-10.13 PID 79977	RAILROAD CROSS SECTIONS 570+00.00 TO 572+00.00	DESIGNED JBA CHECKED SKT	DRAWN JBA REVISED	REVIEWED VKN DATE 11/07 STRUCTURE FILE NUMBER

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





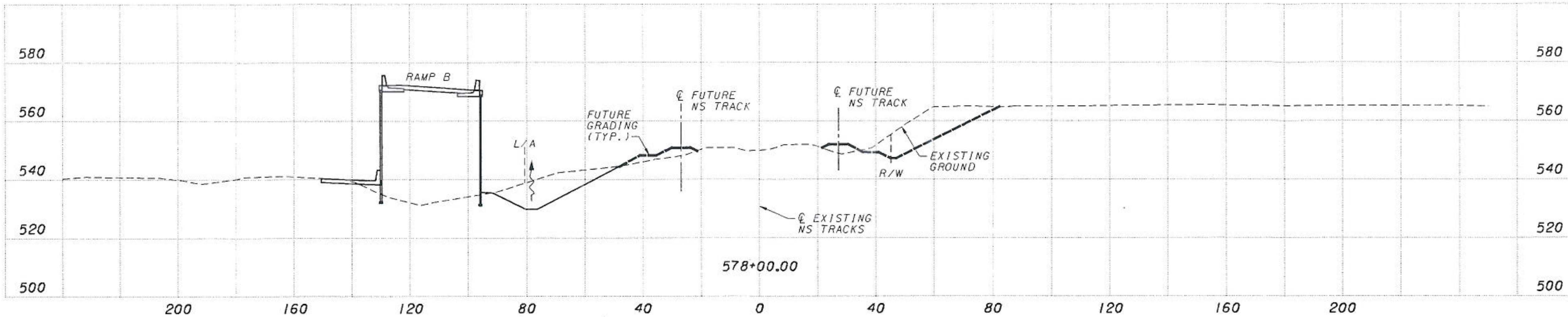


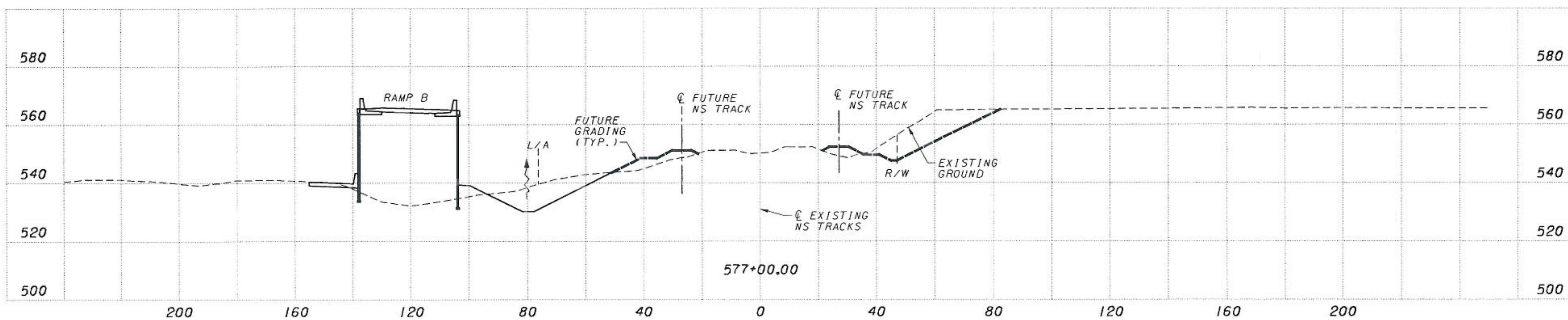
DESIGN AGENCY: CH2MHILL, 5775 Perimeter Drive, Suite 190, Dublin, Ohio 43017	REVIEWED VKN / DATE 11/07 / STRUCTURE FILE NUMBER	DRAWN JBA / REVISED	DESIGNED JBA / CHECKED SKT	RAILROAD CROSS SECTIONS 573+00.00 TO 575+00.00	SCI-823-10.13 PID 79977

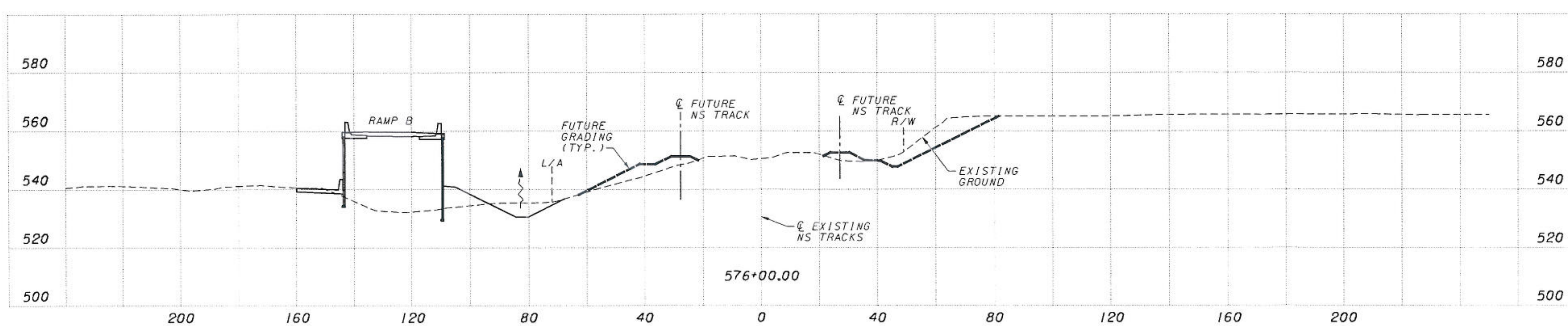
3/11

856

melben
823_160lcxs002.dgn
11/23/2007
10:01 AM
\\aries\PlanSets-400 sci823_half.pen







melben
823_I60Icxs003.dgn
11/23/2007
10:01 AM
\\aries\PlanSets-400 scI823_half.pen

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

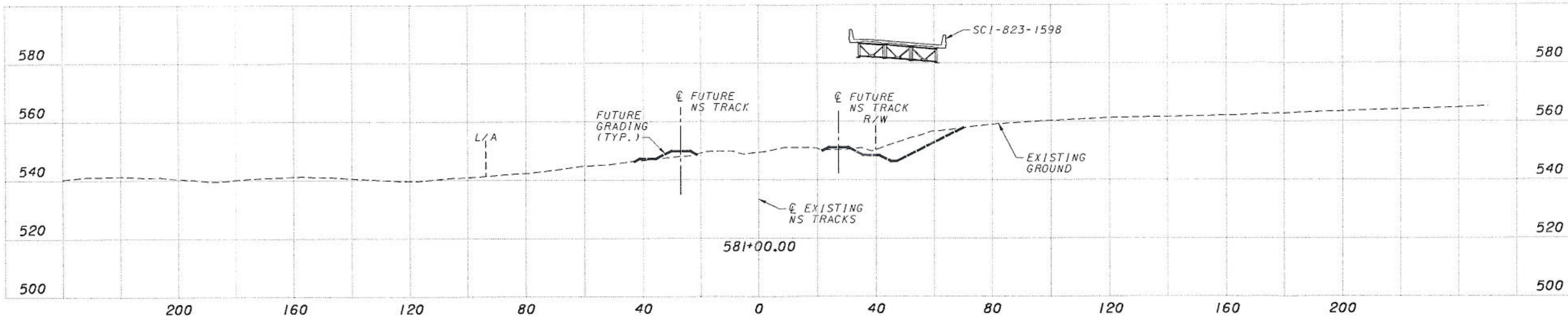
DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

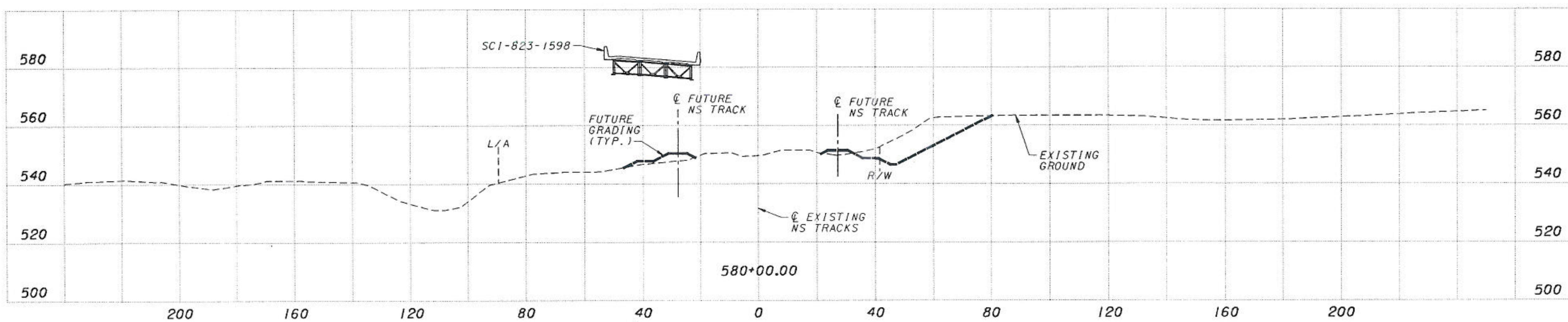
RAILROAD CROSS SECTIONS
576+00.00 TO 578+00.00

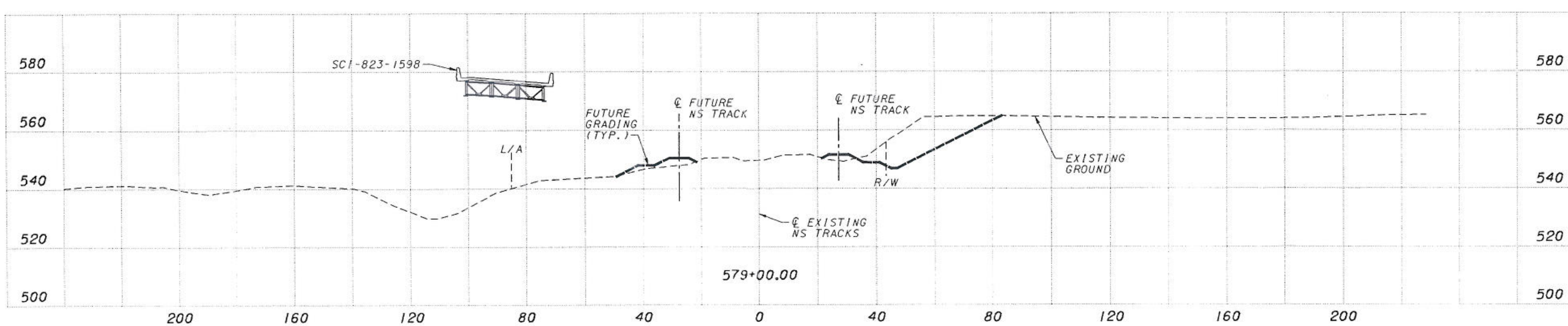
SCI-823-10.13
PID 79977

4 / 11

867







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

REVIEWED VKN DATE 11/07 STRUCTURE FILE NUMBER
DRAWN JBA REVISED
DESIGNED JBA CHECKED SKT

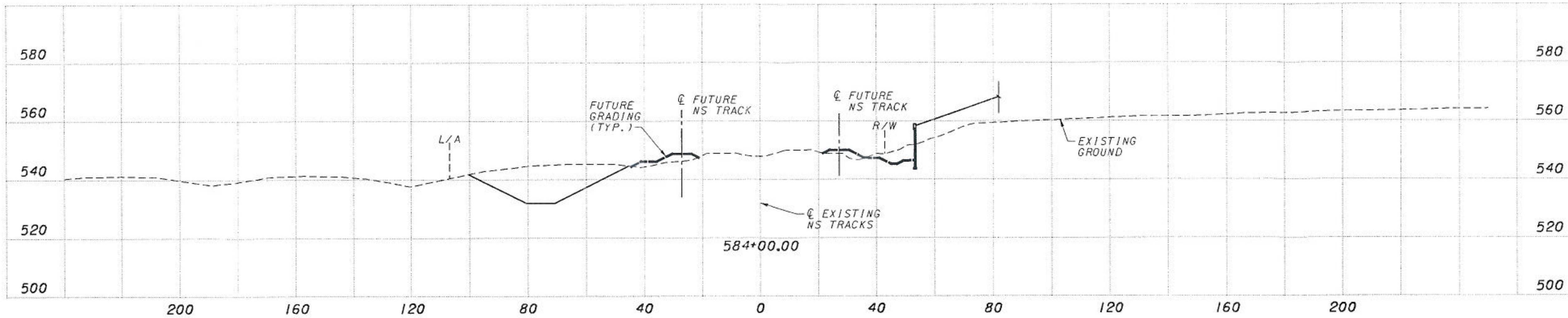
RAILROAD CROSS SECTIONS
579+00.00 TO 581+00.00

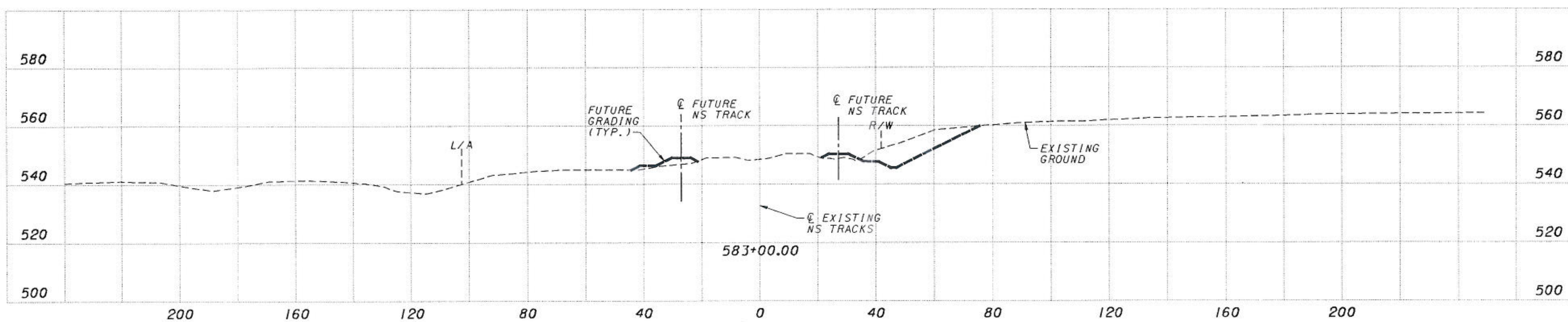
SCI-823-10.13
PID 79977

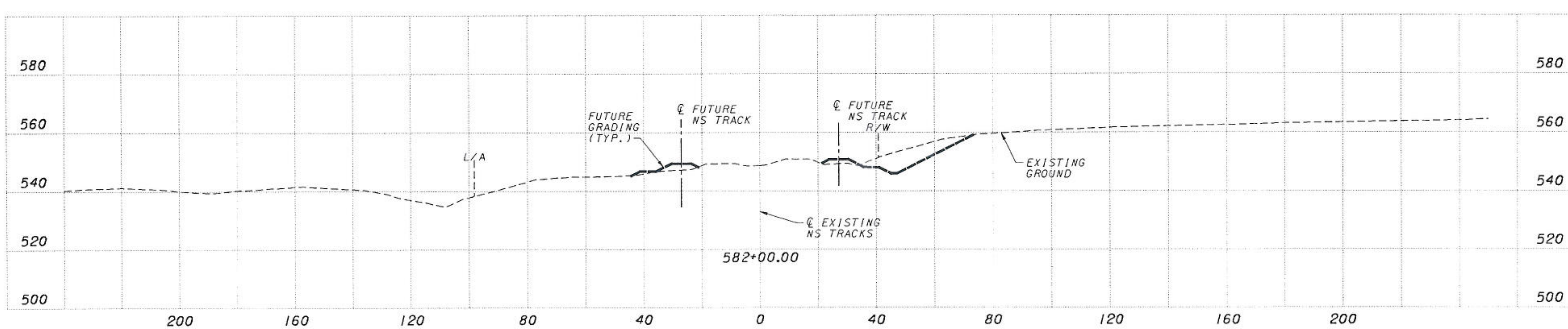
5/11

858

melben
823_160lcx s004.dgn
11/23/2007
10:01 AM
\\aries\PlanSets-400 sci823_half.pen







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

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

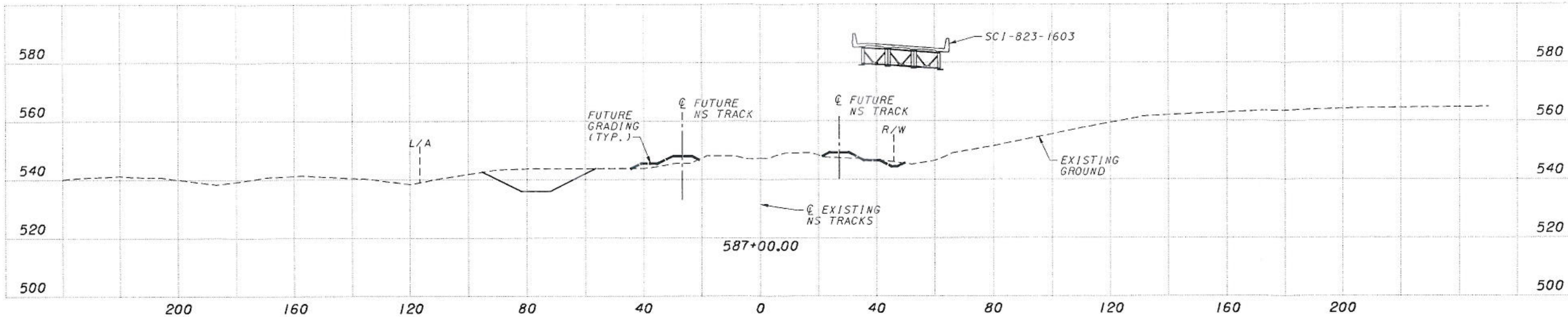
RAILROAD CROSS SECTIONS
582+00.00 TO 584+00.00

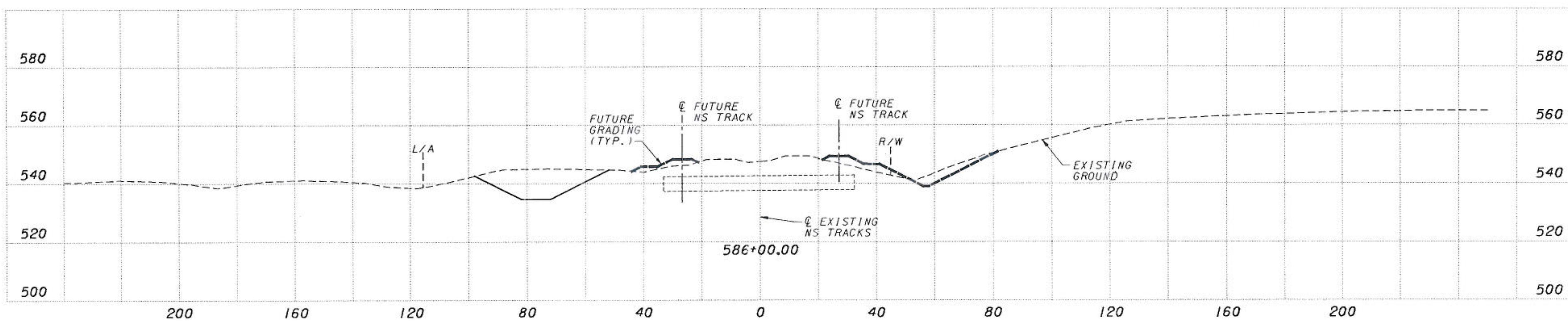
SCI-823-10.13
PID 79977

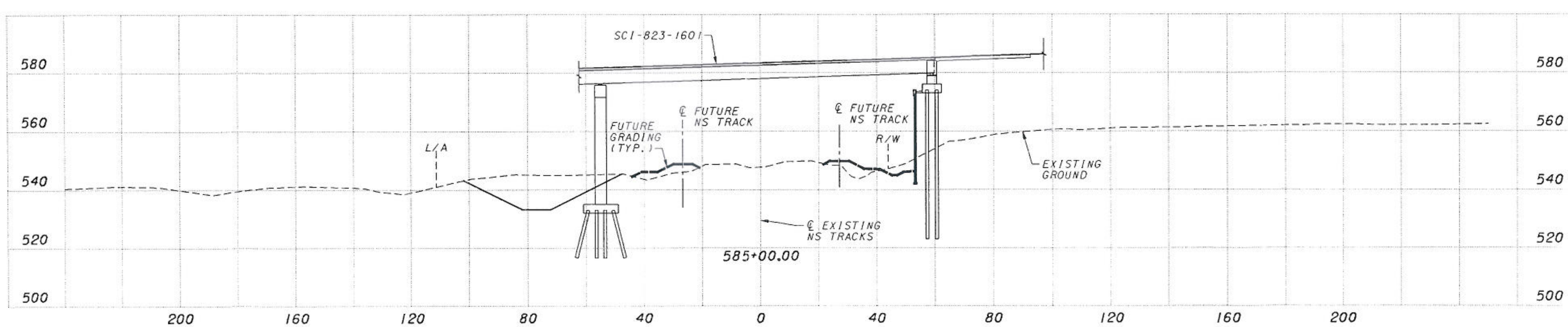
6 / 11

859

melben
823_I60lcxs005.dgn
11/23/2007
10:01AM
\\aries\PlanSets-400 sci823_half.pen







melben
823_1601cxs006.dgn
11/23/2007
10:01 AM
\\aries\PlanSets-400 sci823_half.pen

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

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

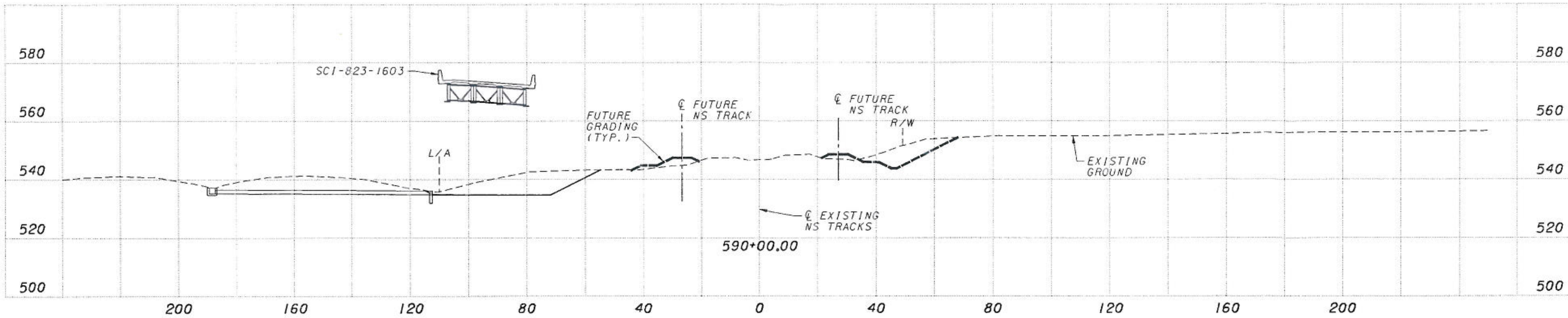
RAILROAD CROSS SECTIONS

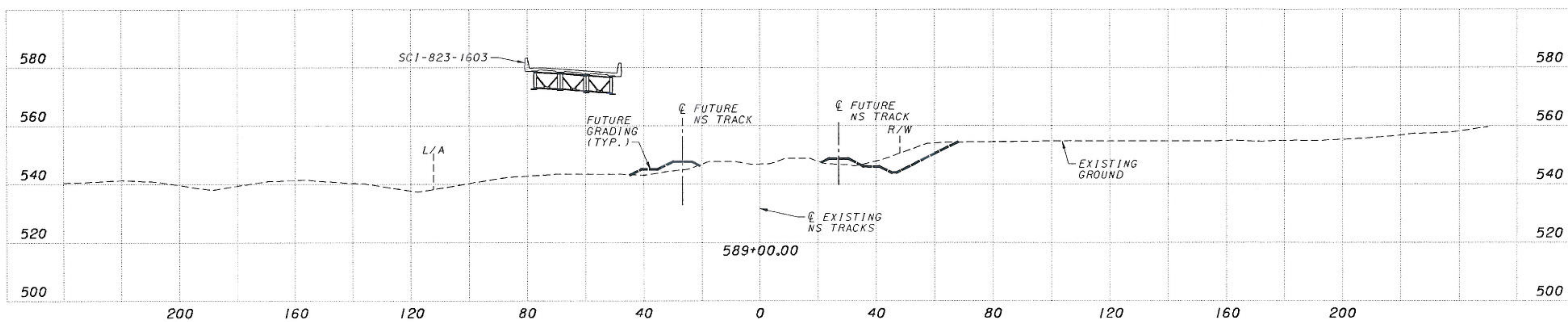
585+00.00 TO 587+00.00

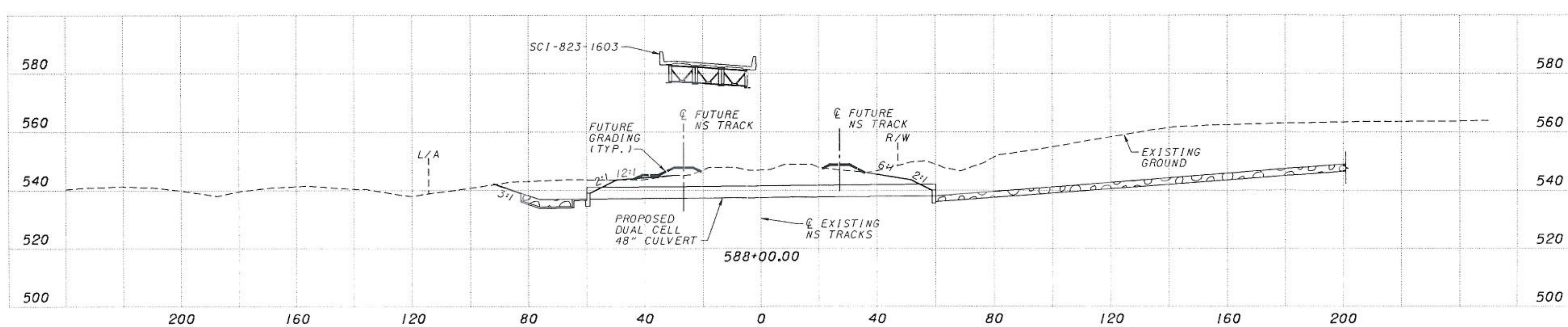
SCI-823-10.13
PID 79977

7 / 11

860







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

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

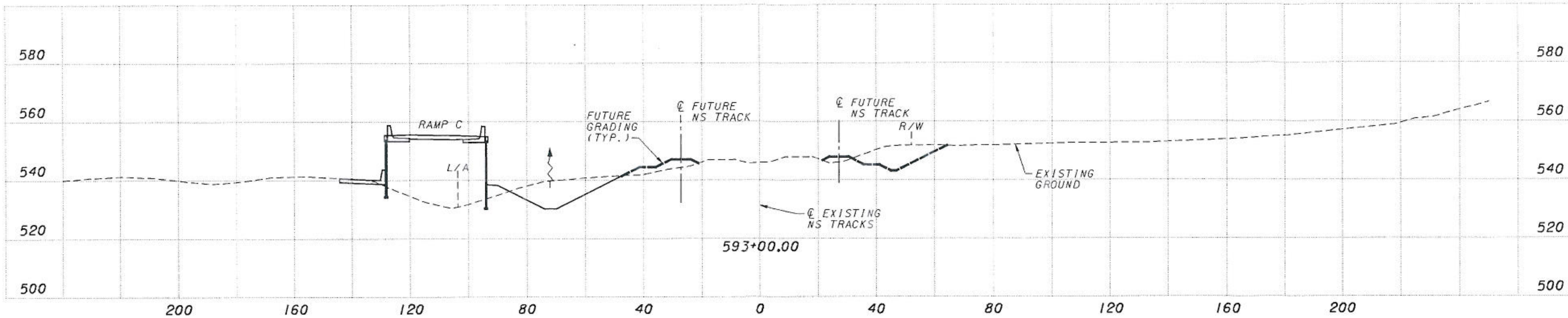
RAILROAD CROSS SECTIONS
588+00.00 TO 590+00.00

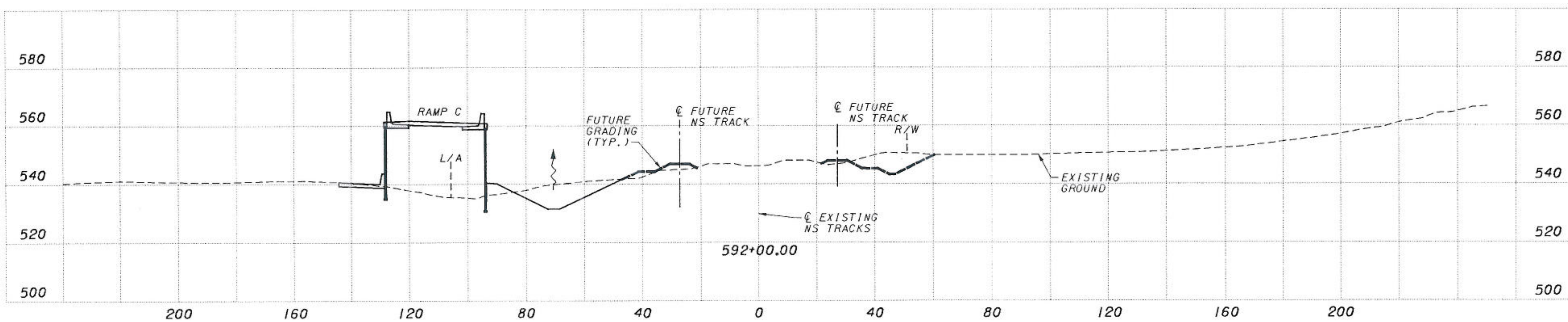
SCI-823-10.13
PID 79977

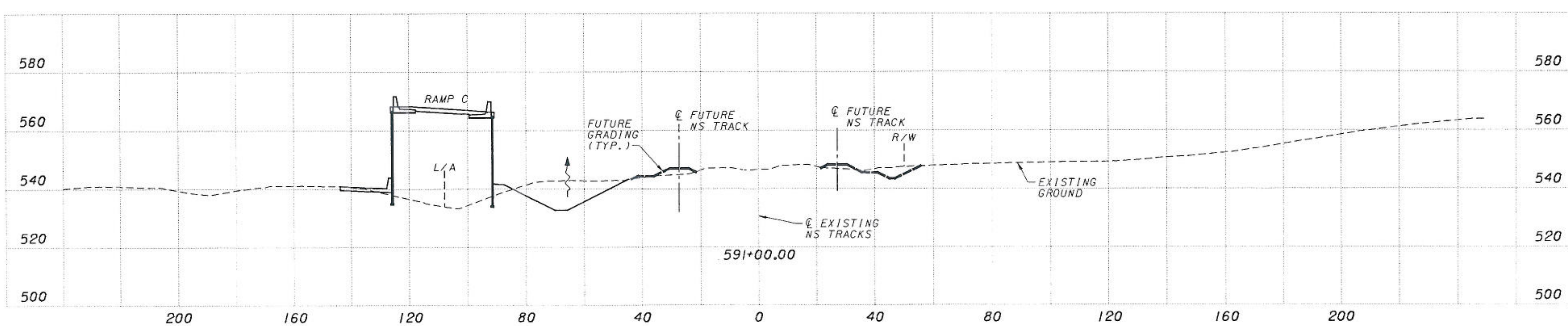
8 / 11

198

melben
823_1601cxs007.dgn
11/23/2007
10:02 AM
\\aries\PlanSets-400 sci823_half.pen







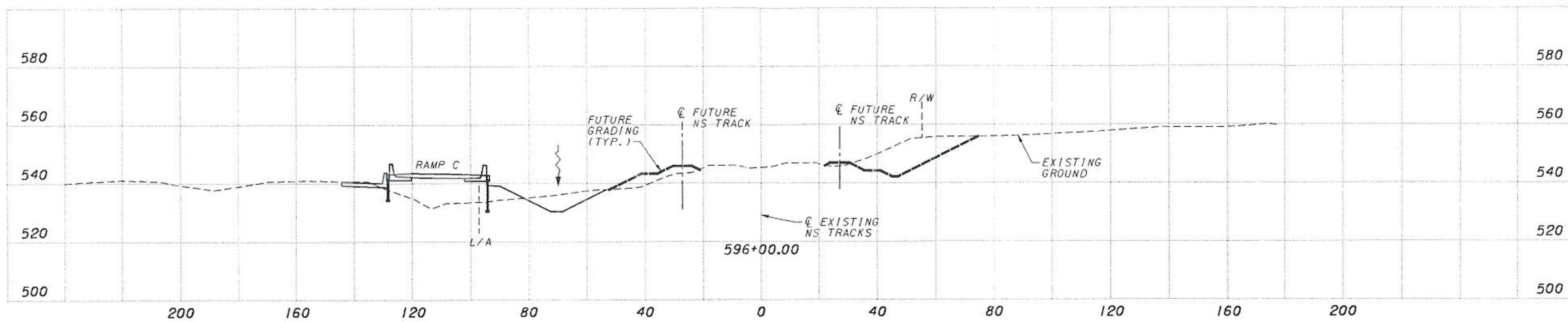
melben
823_160Icxs008.dgn
11/23/2007
10:02 AM
\\aries\PlanSets-400 sci823_half.pen

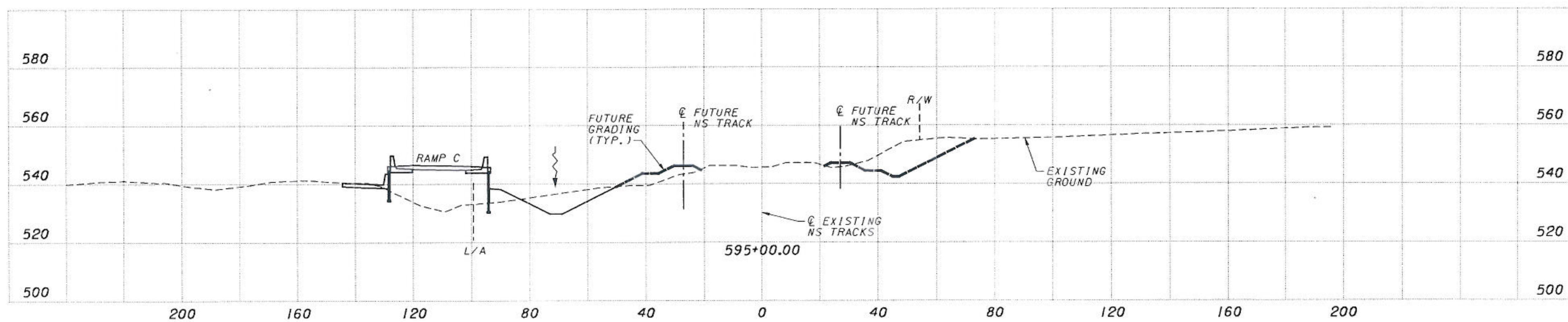
DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

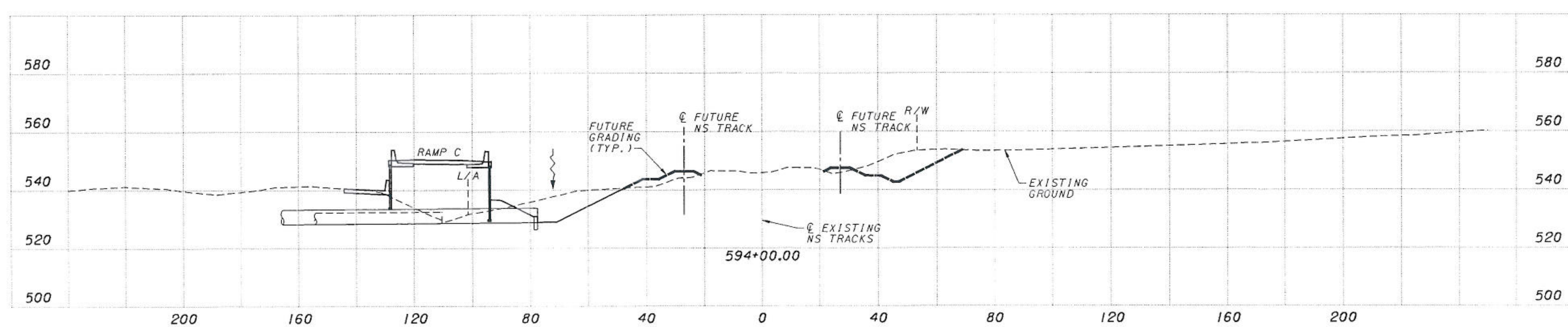
DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

RAILROAD CROSS SECTIONS
591+00.00 TO 593+00.00

SCI-823-10.13
PID 79977







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

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

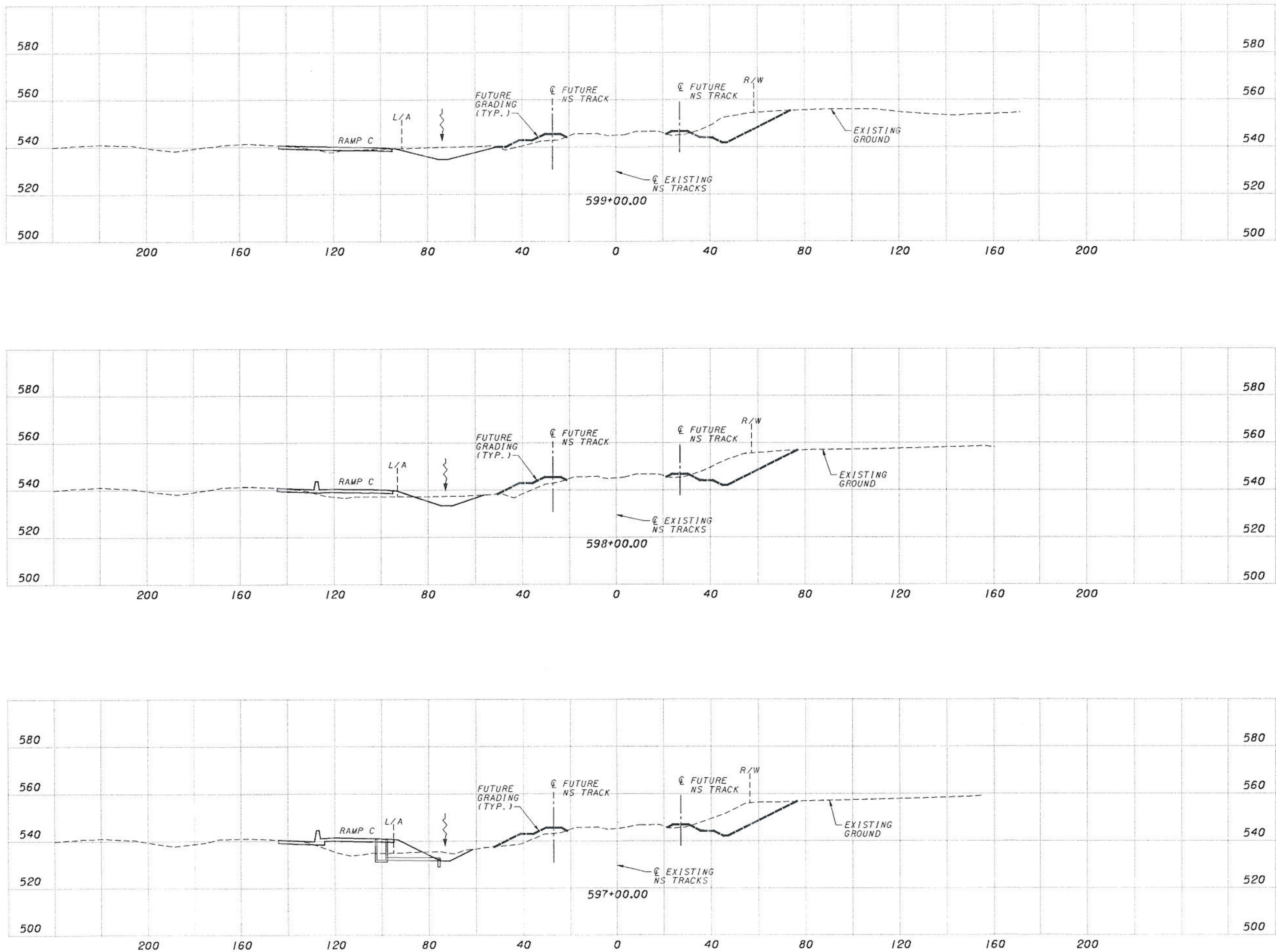
RAILROAD CROSS SECTIONS
594+00.00 TO 596+00.00

SCI-823-10.13
PID 79977

10/11

863

melben
823_160lcxs009.dgn
11/23/2007
10:02 AM
\\varies\PlanSets-400_scl823_half.pen



RAILROAD CROSS SECTIONS
597+00.00 TO 599+00.00

SCI-823-10.13
PID 79977

11 / 11

864

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

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

melben
823_1601cx s010.dgn
11/23/2007
10:02 AM
\\varies\PlanSets-400 sci823_half.pen

APPENDIX G

inter-office communication

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

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

subject: SCI-823-10.13; PID 79977; Bridge No. SCI-823-1598; Ramp B over Norfolk Southern Railroad; Revised Structure Type Study Review

Attn.: John K. Wetzel, District 9 Project Manager

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

General Comments

1. We agree that the proposed structure should consist of a three span composite curved steel plate girders (ASTM A709, Grade 50W) supported on jointed stub abutments and T-type piers. Tim Keller, Jawdat Siddiqi, Jeff Crace and myself had a brief meeting to discuss the best structure type at the proposed site. We all are in agreement that we do not feel the need in requesting the Design Consultant to investigate a one-span (approximately 200' long) alternative.

2. Please investigate if a 2:1 slope could be utilized on the southeast corner of Ramp B and also in front of rear abutment. We agree that MSE wall would be needed on the southwest corner of Ramp B due to close proximity of U.S.R. 23 northbound.

3. Additional comments on subsurface investigation report for the proposed MSE wall and foundation type will be submitted in a separate IOC by Peter Narsavage. Please incorporate Mr. Narsavage's comments prior to proceeding with Stage 1 Design.

4. As stated in page 12 of the Structure Type Study, the Ramp B profile will need to be lowered to reduce the amount of excess vertical clearance.

5. We encourage the Design Consultant to download the presentation on State of Practice for Highly Skewed Bridges which was held on April 24, 2007 at ODOT Central Office Auditorium. The presentation can be downloaded from the Office Structural Engineering website at the following website address: **http://www.dot.state.oh.us/se/skew/skew.htm**
   The Design Consultant will find the presentation 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.

6. The approximate top of bedrock should only be given to the nearest 1 foot. Please verify the estimated pile lengths in the profile view.

7. The Design Consultant shall perform constructability review of the proposed structure prior to Stage 1 submittal. For the Stage 1 submittal, we would like to request additional information from Design Consultant regarding the construction of the proposed structure. The Structure Type Study report indicates that the proposed alternative will require two (2) temporary bents and none of the temporary bents will be located between the two existing railroad tracks. Where will the temporary bents be placed? Please explain the sequence of girder erection. In other words, how the girders will be erected, how many cranes are needed and where the cranes are going to be located during the placement of the girders.

8. The e-mail from David Wyatt (Norfolk Southern) dated March 22, 2007 mentioned the 26'-0" minimum horizontal clearance from the centerline of future track to the face of proposed pier to accommodate maintenance roadway. Please verify with the Norfolk Southern if the proposed 25'-0" horizontal clearance will be acceptable.

9. Structure File Number for this bridge is **7306776**, not 7306717. For future projects, Structure File Number can be obtained by contacting Ms. Kathy J. Keller, Office of Structural Engineering, Bridge Inventory section at 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 in writing 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: Thomas M. Barnitz, ODOT District 9
Lawrence A. Wills, ODOT District 9
Timothy J. Keller, Office of Structural Engineering
Jawdat Siddiqi, Office of Structural Engineering
Jeffery A. Crace, Office of Structural Engineering
file

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 07-11-07

Bridge SCI-823-1598: Ramp B over Norfolk Southern

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.04

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 three span composite curved steel plate girders (ASTM A709, Grade 50W) supported on jointed stub abutments and T-type piers. Tim Keller, Jawdat Siddiqi, Jeff Crace, and I had a brief meeting to discuss the best structure type at the proposed site. We are all in agreement that we do not feel the need in requesting the Design Consultant to investigate a one-span (approximately 200′ long) alternative.	Will Comply.
General	2. Please investigate if a 2:1 slope could be utilized on the southeast corner of Ramp B and also in front of the rear abutment. We agree that an MSE wall would be needed on the southwest corner of Ramp B due to the close proximity of U.S.R. 23 Northbound.	Will comply.
General	3. Additional comments on the subsurface investigation report for the proposed MSE wall and foundation type will be submitted in a separate IOC by Peter Narsavage. Please incorporate Mr. Narsavage's comments prior to proceeding with Stage 1 Design.	Will comply.
General	4. As stated in page 12 of the Structure Type Study, the Ramp B profile will need to be lowered to reduce the amount of excess vertical clearance.	Will comply.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 07-11-07

Bridge SCI-823-1598: Ramp B over Norfolk Southern

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.04

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

Site Plan (1/3)	5. We encourage the Design Consultant to download the presentation on the State of Practice for Highly Skewed Bridges, which was held on April 24, 2007 at the ODOT Central Office Auditorium. The presentation can be downloaded from the Office of Structural Engineering website at the following website address: **http://www.dot.state.oh.us/se/skew/skew.htm**. The Design Consultant will find the presentation 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.
Site Plan (1/3)	6. The approximate top of bedrock should only be given to the nearest 1 foot. Please verify the estimated pile lengths in the profile view.	Will comply.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 07-11-07

Bridge SCI-823-1598: Ramp B over Norfolk Southern

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.04

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

Site Plan (1/3)	7. The Design Consultant shall perform a constructability review of the proposed structure prior to the Stage 1 submittal. For the Stage 1 submittal, we would like to request additional information from the Design Consultant regarding the construction of the proposed structure. The Structure Type Study report indicates that the proposed alternative will require two (2) temporary bents and none of the temporary bents will be located between the two existing railroad tracks. Where will the temporary bents be placed? Please explain the sequence of girder erection. In other words, how the girders will be erected, how many cranes are needed, and where the cranes are going to be located during the placement of the girders.	Will comply.
Site Plan (1/3)	8. The e-mail from David Wyatt (Norfolk Southern) dated March 22, 2007 mentioned the 26′-0″ minimum horizontal clearance from the centerline of future track to the face of proposed pier to accommodate a roadway maintenance. Please verify with Norfolk Southern if the proposed 25′-0″ horizontal clearance will be acceptable.	Will comply.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 07-11-07

Bridge SCI-823-1598: Ramp B over Norfolk Southern

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.04

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

General	9. The Structure File Number for this bridge is **7306776**, not 7306717. For future projects, the Structure File Number can be obtained by contacting Ms. Kathy J. Keller, Office of Structural Engineering, Bridge Inventory section at 614-752-9973.	Will comply.

Thompson, Shawn/COL

From: Richard.Behrendt@dot.state.oh.us

Sent: Tuesday, October 23, 2007 7:26 AM

To: Thompson, Shawn/COL

Subject: Re: Portsmouth Bypss - Horizontal Clearance at RR Issues (2)

Shawn,
Since it appears that you been forthcoming about laying out the 25' min. horizontal clearance issue, moreso after the 4/4/07 face-to-face meeting, I would conclude that NS will not take issue w/the 25' horizontal clearance, particularly since you've demonstrated through the various meetings and email correspondence that CH2M and DLZ have done their due diligence to provide as much horizontal clearance as possible given the curvature of the new bridges going through the area.

I would be concerned if there had not been the level of interaction that you've done on the project, but considering the level of interaction that you've done to date, and the fact that NS sees that you've tried to accomodate their requests, I don't feel that NS will not accept what you present when you forward them Stage 1 drawings showing the 25' min. clearance dimension.

Let me know if you need anything else...

Rich Behrendt
Program Mgr./State Rail Coordinator
Ohio Department of Transportation
1980 West Broad St.
Columbus, Ohio 43223
Phone: 614-387-3097
FAX: 614-466-0158
email: richard.behrendt@dot.state.oh.us

<Shawn.Thompson@ch2m.com>

10/22/2007 04:23 PM

To <Richard.Behrendt@dot.state.oh.us>

cc

Subject Portsmouth Bypss - Horizontal Clearance at RR Issues

Richard,
Good afternoon. I hope things are going well for you. We continue to coordinate with Norfolk Southern regarding our Portsmouth Bypass project. TranSystems and CH2M HILL plan to submit our Stage 1 plans at the end of November. However, we have one last outstanding issue that just won't seem to close itself. As part of ODOT OSE's Structure Type Study review of several of our bridges over the RR, one of the comments was to verify with Norfolk Southern that a 25' horizontal clearance is acceptable, even though the standards show a 26' minimum clearance from the face of our proposed piers to the centerline of future NS tracks to accommodate a maintenance roadway. I have attached a copy of the ODOT OSE comments of our Ramp B bridge over the RR for your convenience (see highlighted comment #8 on SCI-823-1598 Revised Study.wpd).

Repeated attempts to contact Rhonda Moore and David Wyatt at Norfolk Southern have failed regarding this issue. A few months ago, Rhonda informed me that she was looking into some field data about this, but I never heard back from her. On August 7, 2007, I sent both her and David essentially a copy of the attached technical memorandum (Document.pdf) requesting Norfolk Southern to accept the 25' clearance. Unfortunately, I never received a response.

Again, our Stage 1 submittal date is the end of November. If we don't hear from Norfolk Southern before then, we plan to include a copy of the plan sets for Norfolk Southern review. My only fear is that we've completed preliminary design of the bridges, and I'd hate to have to change span layouts after the Stage 1 submittal if the railroad is not accepting our our proposed clearances.

Any assistance you can provide on this matter would be greatly appreciated. Thanks for your time.

Shawn

Shawn Thompson, P.E.
CH2M HILL
Operations Leader
5775 Perimeter Drive
Suite 190
Dublin, Ohio 43017
Tel – 614.734.7144 ext. 17
Mobile – 614.535.7502
shawn.thompson@ch2m.com

Developing People through Challenging Projects

[attachment "SCI-823-1598 Revised Study.wpd" deleted by Richard Behrendt/RealEstate/CEN/ODOT]
[attachment "Document.pdf" deleted by Richard Behrendt/RealEstate/CEN/ODOT]