Bridge Preliminary Design Report

SR-823 over Norfolk Southern Tracks and US-23
SCI-823-1601

SCI-823-10.13
PID No. 79977

Prepared for

Ohio Department of Transportation

November 2007

CH2MHILL

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1. Introduction

Following review and resolution of comments on the Structure Type Study resubmitted in May 2007, a three-span composite steel plate girder bridge with reinforced concrete deck and jointed stub abutments behind MSE walls was the structure type selected by the Department on June 19, 2007 for construction of the proposed SR 823 over Norfolk Southern Tracks and US-23 bridge.

The proposed bridge is a three span structure with a 22°-36'-19" RF skew. The spans are 123'-0", 144'-10" and 97'-5". The reinforced concrete deck is 66'-0" wide and is supported by eight steel plate girders. The two piers are the cap and column type and are supported on HP piles driven to refusal on rock. Each pier has five columns. The two semi-integral abutments are supported on HP piles driven to refusal on rock. Both abutments are located behind MSE walls.

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 SR 823 has not changed and is the same as shown on the type study drawings.

- *Horizontal Geometry:* The horizontal geometry of SR 823 has not changed since the type study.

- *Bridge Substructure:* The location of the piers and abutments has changed. The location of Pier 2 was moved to be coincident with the centerline of construction for US 23. The bottom of footing elevation has been revised to 534.20 from 533.60 to provide one foot of cover over the top of footing.

  A preliminary design of Pier 1 was completed. In the type study, 36 inch diameter columns were assumed and horizontal clearance to the railroad track was calculated accordingly. However, the preliminary pier design indicates that a 36 inch diameter column may not be adequate due to the column's slenderness and the associated moment magnification. Therefore, Pier 1 has been shifted to the west (further from the tracks) to permit the use of larger diameter columns. This has the added benefit of reducing the center span from 146'-0" to 144'-10", which may result in a slightly lighter plate girder.

  The MSE wall at the forward abutment as shown in the type study was straight and on a 22°-36'-19" right forward skew. Although this skew is parallel to US 23, the horizontal clearance to the face of wall is controlled by the distance to the edge of pavement of the Ramp D taper, which is not parallel to US 23. As a result, in the type study, a minimum horizontal clearance of 30'-0" was provided at the far north end of the wall and that resulted in a horizontal clearance of 34'-8" at the north end of the abutment. The layout of the MSE wall has been revised to provide the minimum 30'-0" horizontal clearance at the north end of the abutment and also at the far north end of the MSE wall. This results in a "kink" in the MSE wall at the north end of the abutment. The angle of the bend is approximately 178°-44'-51". This change results in the end span of the bridge being revised from 103'-0" to 97'-5".

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±. This results in a ditch running parallel to Pier 1. The bottom of the ditch is at Elev. ±535.70. The ditch may be lined with up to 30 inches of rock channel protection, therefore, the top of footing is placed three feet below the flowline of the ditch or one foot below the bottom of the rock channel protection. This results in a bottom of footing elevation of 529.00, as compared to elevation 533.00 shown on the Site Plan of the type study. This results in a theoretical pile length of 10 feet from the bottom of footing to rock. 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 three feet 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 52 inches and 54 inches results in a girder with the least weight. A web depth of 52 inches is proposed for this bridge. This is a revision from the 60 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 700 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 not only an existing State Route, but also an existing and active railroad may require more than one construction day.

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

2. Design Criteria

The following design criteria apply to this structure, SR 823 over US 23 and the Norfolk Southern tracks:

Functional Classification:	Urban Principal Arterial	
Traffic Data:	ADT (2010)	8,900
	ADT (2030)	13,000
	ADTT (2030)	1,820
	Design Speed	70 mph
	Legal Speed	65 mph

Required Vertical Clearance:	17'-0" over US 23
	23'-5 1/8" over eastern two Norfolk Southern tracks, measured six feet from centerline rail
	23'-6 1/2" 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 column to centerline of adjacent Norfolk Southern track

3. Maintenance of Traffic

The proposed SR-823 alignment will carry traffic exiting southbound US-23 onto southbound SR-823 and exiting northbound SR-823 onto southbound US-23. Because the SR-823 alignment is new construction, maintenance of highway traffic during construction of the SR-823 bridge over Norfolk Southern Tracks and US-23 will be limited. With the exception of limited US-23 closures for superstructure beam setting, existing culvert replacement, and US-23 acceleration lane 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.

Six borings at the SR-823 bridge over Norfolk Southern tracks and US-23 were taken during the first phase and three were taken during the second phase. Based on these borings, foundation recommendations have been made by DLZ. Geotechnical engineers at CH2M HILL performed a brief review of the MSE wall/bridge foundation recommendations contained in the final subsurface exploration report prepared by DLZ, and provided written comments in a technical memorandum. A copy of DLZ's foundation report and CH2M HILL's review comments are included with this submission in Appendix C.

The semi-integral rear and forward abutments, behind a MSE wall, will be supported by HP 12x53 H-piles driven to bedrock. The final pile arrangement should consider avoiding potential conflicts with typical MSE reinforcing strap patterns. Each pier is supported by HP 12x53 H-piles driven to bedrock. Pier piles will be battered to resist horizontal loads.

It is anticipated that most of the piles will be driven to refusal on shale or sandstone. While weathered shale bedrock is present at the top of rock near Pier 1, Pier 2, and the forward abutment, the shale layer is thin and it is possible that some piles could be driven through the shale to refusal on the sandstone. Therefore, it is recommended that reinforced pile points be used to protect all the proposed piles while driving at Pier 1, Pier 2, and the forward abutment.

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

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

Substructure Unit	Type	Bottom of Footing Elev.	Estimated Pile Tip Elev.	Pile Type	Max. Design Load (tons)	Distance: Top of Pile[1] to Estimated Pile Tip	Estimated Pile Length	Pile Order Length
Rear Abut.	Semi - Integral	573.00	523.0	HP12x53	70	51.0	55	60
Pier 1	Cap & Column	529.00	516.0	HP12x53	70	14.0	15	20
Pier 2	Cap & Column	534.20	511.8	HP12x53	70	23.4	25	30
Fwd Abut.	Semi - Integral	562.00	506.7	HP12x53	70	56.3	60	65

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

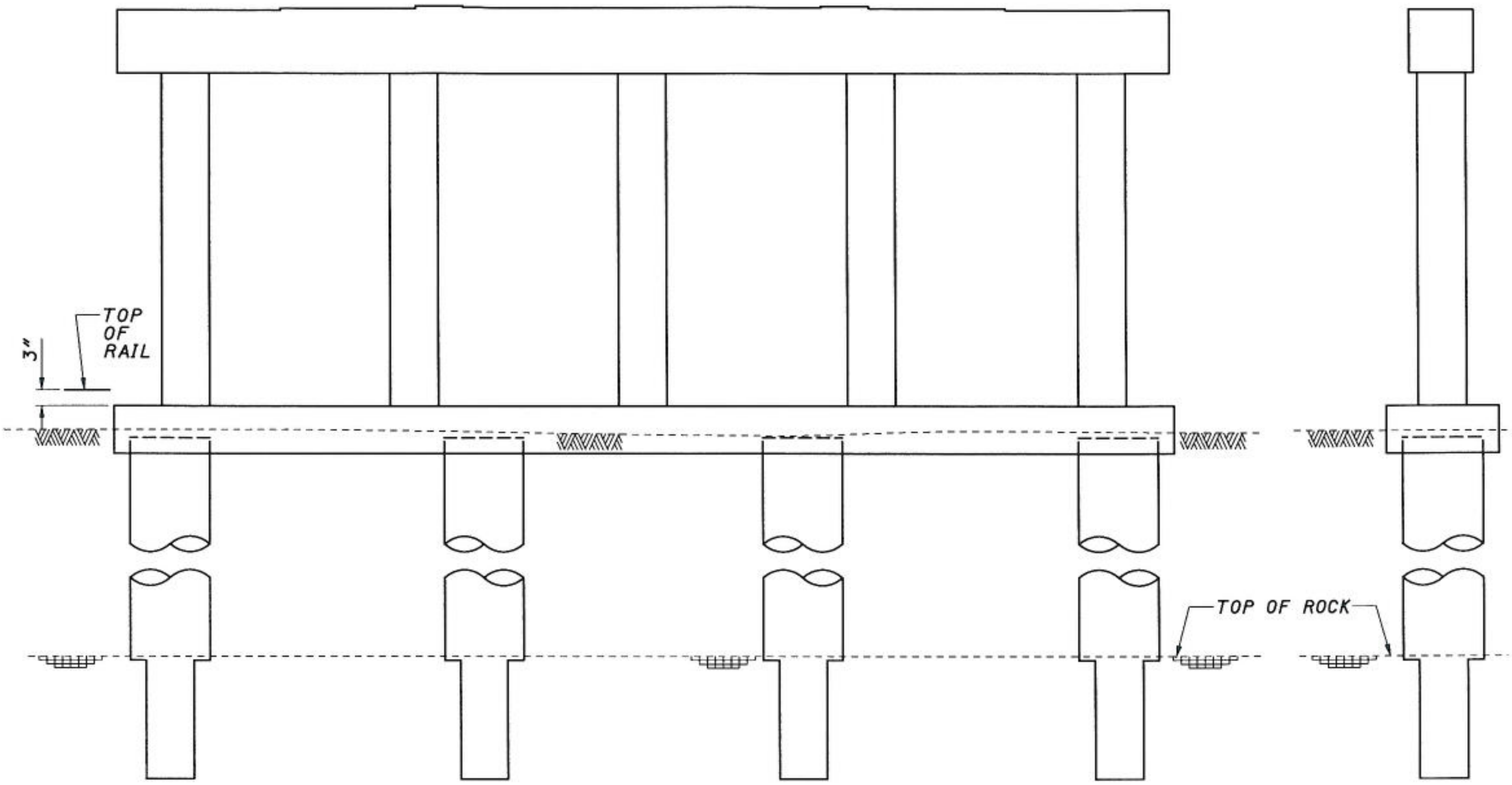
The piers are currently recommended to be supported by steel HP piles driven to refusal on rock. However, the piers could also be supported on drilled shafts that are socketed into rock. At the end of this section is a sketch showing two alternatives for a pier supported on drilled shafts.

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.

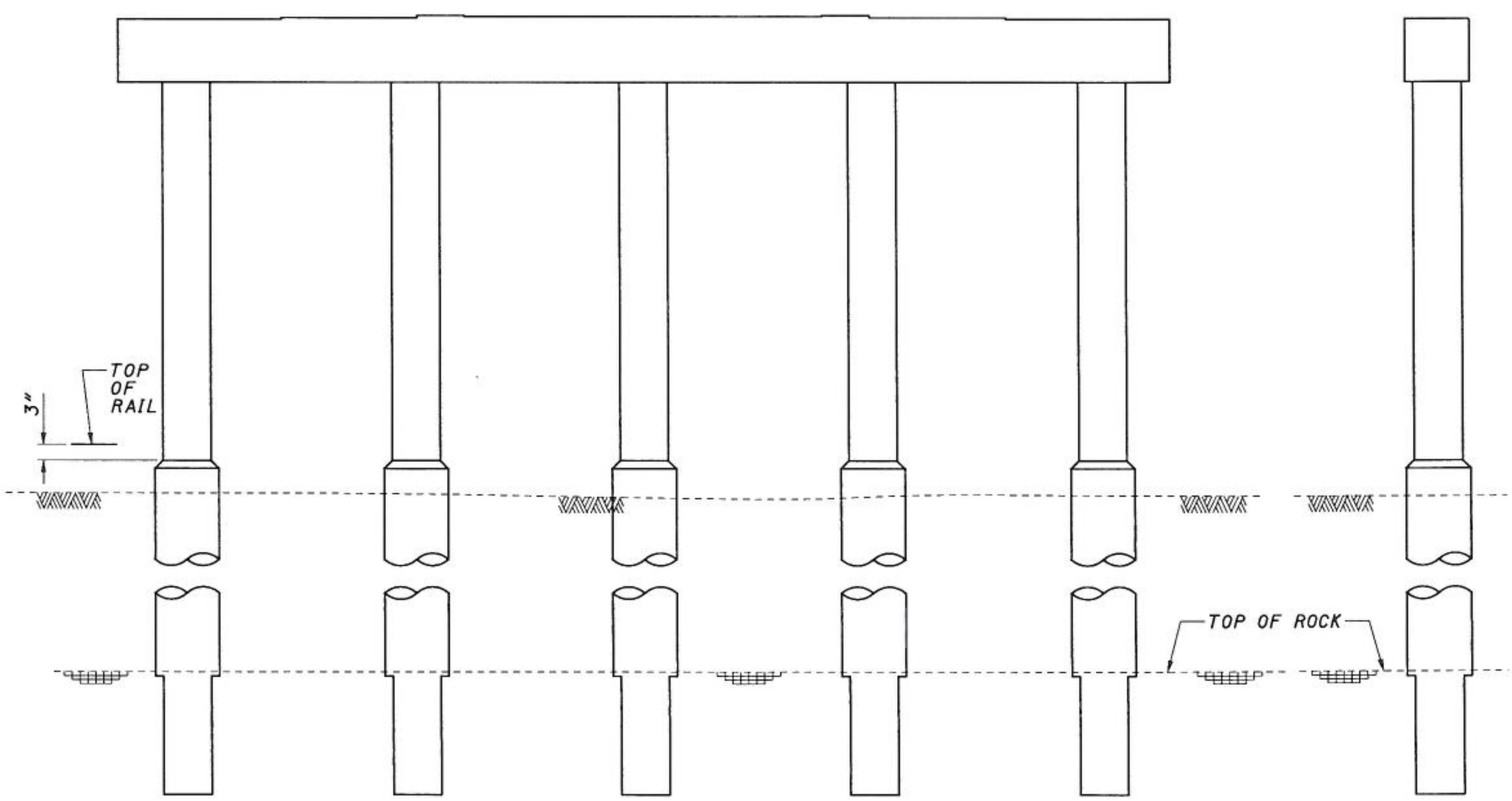
The drilled shaft foundation has several advantages when compared to piles and include:

- 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 SR 823 bridge over US 23 and 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-823-1601**
Structure File Number:	**7306792**

APPENDIX A

SCI-823-10.13

SCI-823 Over Norfolk Southern Tracks & US-23

PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1601C 823 over Railroad_US23\Report\[Cost Estimate.xls]Summary

By: DGS Revised: SCJ Date: 4/9/2007 Revised: 7/19/2007
Checked: JTC Date: 4/20/2007

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	123.00 - 144.83 - 97.42	365.25	8 ~ Steel Plate Girders	Steel Plate Girder (52" Web)	**$1,800,000**	**$716,000**	**$403,000**	**$584,000**	**$3,503,000**	**$2,379,000**	**$5,882,000**

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.
3. Use 2006 Concrete Barrier, Single Slope Median, Type B1 cost = $64.00 /ft.
   Use 2006 Concrete Barrier, Single Slope, Type D cost = $81.00 /ft.
4. Structure incidental cost allowance includes provision for structure excavation, porous backfill & drainage pipe, sealing of concrete surfaces, structural steel painting, 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

SCI-823 Over Norfolk Southern Tracks & US-23

PRELIMINARY BRIDGE DESIGN COST ESTIMATE

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1601C 823 over Railroad_US23\Report\[Cost Estimate.xls]Summary

By: DGS Revised: SCJ Date: 4/9/2007 Revised: 7/19/2007

Checked: JTC Date: 4/20/2007

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	123.00 - 144.83 - 97.42	365.25	367.25	24,200	942	$461,400	$217,400	$90,600	8 ~ Steel Plate Girders	Steel Plate Girder (52" Web)	800,000	$1,030,400	**$1,800,000**

Deck Cross-Sectional Area:

Parapets:

	No.	Individual Area (sq. ft.)	Parapet Area (sq. ft.)
Parapets	2	4.26	8.52
Median	1	9.29	9.29

Slab:

T (ft.)	Ave. W (ft.)	Slab Area	Haunch & Overhang Area	Total Concrete Area (sq. ft.)
0.708	66.00	46.7	4.7	69.2

Note: 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 =			$490.00

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 4)	n/a	$1.15	12.0%	$1.29

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

Length = 30 ft. Width = 66.00 ft
Area = 220 sq. yd.

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

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SUBSTRUCTURE

Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Proposed Stringer Section	Pier Concrete Cost	Pier Reinforcing Cost	Abutment Concrete Cost	Abutment Reinforcing Cost	Pile Foundation Cost	Temporary Sheeting and Shoring	Initial Substructure Cost
3	123.00 - 144.83 - 97.42	8 ~ Steel Plate Girders	Steel Plate Girder (52" Web)	$165,700	$34,700	$125,200	$23,000	$140,600	$227,000	**$716,000**

Pier QC/QA Concrete, Class QSC1 Cost:

Pier 1

	Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Cap	52.0	$555.68	3.0%	$572.00	$29,700
Columns	79.9	$555.68	3.0%	$572.00	$45,700
Footings	58.0	$300.31	3.0%	$309.00	$17,900
Total Pier 1 Cost					$93,300

Pier 2

	Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Cap	42.3	$555.68	3.0%	$572.00	$24,200
Columns	52.9	$555.68	3.0%	$572.00	$30,300
Footings	58.0	$300.31	3.0%	$309.00	$17,900
Total Pier 2 Cost					$72,400

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
44	44	531.0	535.2	516.0	511.8	20	30	2,200	$65,800	HP12 x 53

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
20	20	574.0	563.0	523.0	506.7	60	65	2,500	$74,800	HP12 x 53

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

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

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

Abutment QC/QA Concrete, Class QSC1 Cost:

Component		Volume (cu. yd.)	Year 2005	Annual Escalation	Year 2006	Total Cost
Abutment						
	Rear	122.2	$384.26	3.0%	$396.00	$48,400
	Fwd	122.2	$384.26	3.0%	$396.00	$48,400
Wingwalls						
	Rear	35.8	$384.26	3.0%	$396.00	$14,200
	Fwd	35.8	$384.26	3.0%	$396.00	$14,200
Total Abutment Cost						$125,200

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

Temporary Sheeting and Shoring:

	Exposed Wall Height	Depth of Embedment	Total Wall Height	Length	Total Exposed Wall Area	Total Wall Area
At Pier 1:	10.0	10.0	20.0	227	2,270	4,540

Costs:

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

LIFE CYCLE MAINTENANCE COST

Span Arrangement No. Spans	Lengths	Framing Alternative	Structural Steel Painting (4) Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost	Superstructure Sealing (4) Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost
3	123.00 - 144.83 - 97.42	8 ~ Steel Plate Girders	$560,800	2	$1,121,600	$0	4	$0

Span Arrangement No. Spans	Lengths	Framing Alternative	Bridge Deck Overlay (4) Deck Demo & Chipping	Deck Overlay	Deck Joint Gland (2)	Number of Maintenance Cycles	Total Life Cycle Cost	Bridge Redecking (4) Deck Concrete Cost (3)	Deck Reinforcing Cost (3)	Deck Joint Cost (2)	Deck Removal Cost	Number of Maintenance Cycles	Total Life Cycle Cost	Superstructure Life Cycle Maintenance Cost (1)	Total Initial Construction Cost	Total Relative Ownership Cost
3	123.00 - 144.83 - 97.42	8 ~ Steel Plate Girders	$77,800	$90,300	$0	2	$336,200	$461,400	$217,400	$0	$242,000	1	$920,800	$2,379,000	$3,503,000	$5,882,000

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.)
52	8	365.3	16.00	37,012	20%	44,400

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

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
66.00	0

Bridge Deck Removal Cost:

Deck Area (3) (sq. ft.)	Year 2006	Deck Removal Cost
24,200	$10.00	$242,000

Bridge Deck Overlay (Item 848):

Bridge Deck MSC Overlay Cost per sq. yd.:

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

Bridge Deck MSC Overlay Cost per cu. yd.:

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

Deck Area (3) (sq. ft.)	Deck Area (sq. yd.)	Hand Chipping (sq. yd.)	Variable Thickness Repair (cu. yd.)
24,200	2,689	67	56

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

Bridge Deck Joint Gland Replacement Cost per foot:

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

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

NOTES:

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

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By: DGS Revised: SCJ — Date: 4/9/2007 Revised: 7/19/2007

Checked: JTC — Date: 4/20/2007

COST SUMMARY

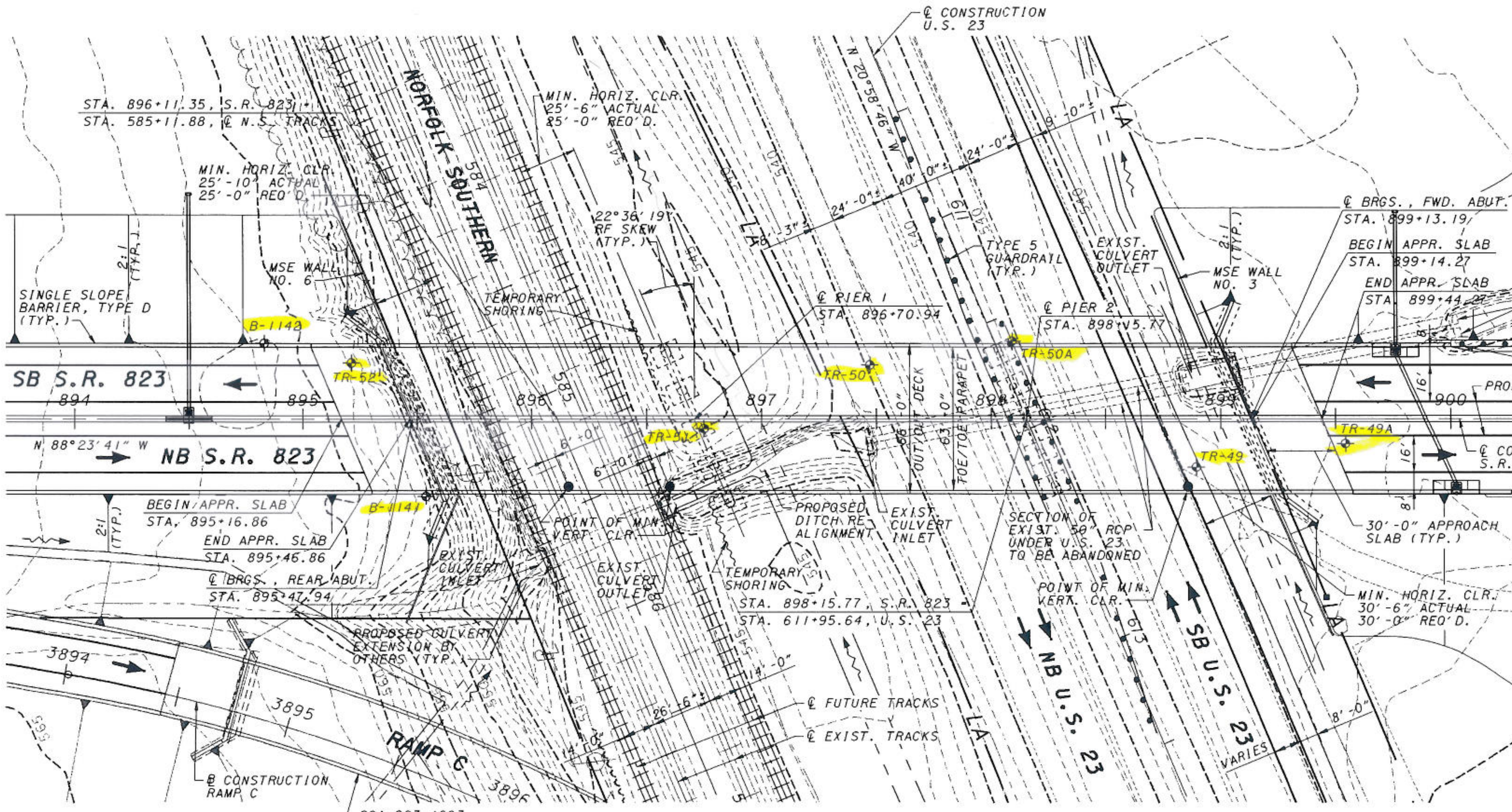
Span Arrangement No. Spans	Span Arrangement Lengths	Framing Alternative	Proposed Stringer Section	Total Initial Superstructure Cost	Total Initial Substructure Cost	Total Initial Construction Cost (1)	Superstructure Life Cycle Maintenance Cost	Total Relative Ownership Cost
3	123.00 - 144.83 - 97.42	8 ~ Steel Plate Girders	Steel Plate Girder (52" Web)	$1,800,000	$716,000	$3,503,000	$2,379,000	**$5,882,000**

Notes:
1. Includes contingencies and incidental costs.

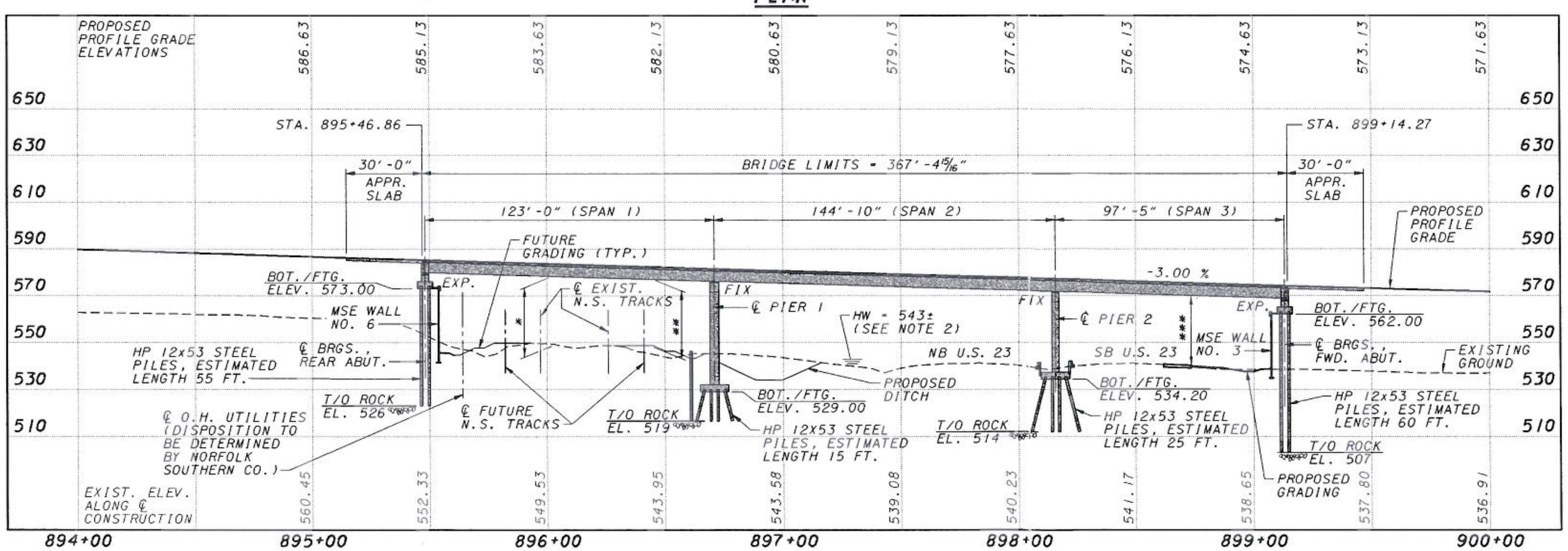
2. The estimate and all unit prices used are based upon 2006 costs.

APPENDIX B

BENCHMARKS



PLAN



* MIN. VERT. CLR. 26'-9" ACTUAL 23'-5 1/8" REQ'D.

** MIN. VERT. CLR. 26'-9" ACTUAL 23'-6 1/2" REQ'D.

PROFILE ALONG PROFILE GRADE LINE. S.R. 823

*** MIN. VERT. CLR. 28'-4" ACTUAL 17'-0" REQ'D.

TRAFFIC DATA

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

LEGEND

INDICATES 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.
3. MEDIAN GUARDRAIL AT PIER 2 TO BE IN ACCORDANCE WITH STANDARD DRAWING GR-6.2, DESIGN A.

PROPOSED STRUCTURE

TYPE: THREE-SPAN COMPOSITE STEEL PLATE GIRDERS (WEATHERING ASTM A709, GR 50W) WITH REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS BEHIND MSE WALLS WITH CAP AND COLUMN PIERS

LENGTH OF SPAN: 123'-0", 144'-10", 97'-5" C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-1 1/8" TOE/TOE BARRIERS (RB) 30'-1 1/8" TOE/TOE BARRIERS (LB)

SIDEWALK: NONE

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

SKEW: 22°36'19" RIGHT FORWARD

WEARING SURFACE: MONOLITHIC CONCRETE

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

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

LATITUDE: N 38°53'32"

LONGITUDE: W 82°59'58"

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	
SKT		7306792	

SCIOTO COUNTY
STA. 895+46.86
TO STA. 899+14.27

SITE PLAN

BRIDGE NO. SCI-823-1601
S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23

SCI-823-10.13
PID 79977

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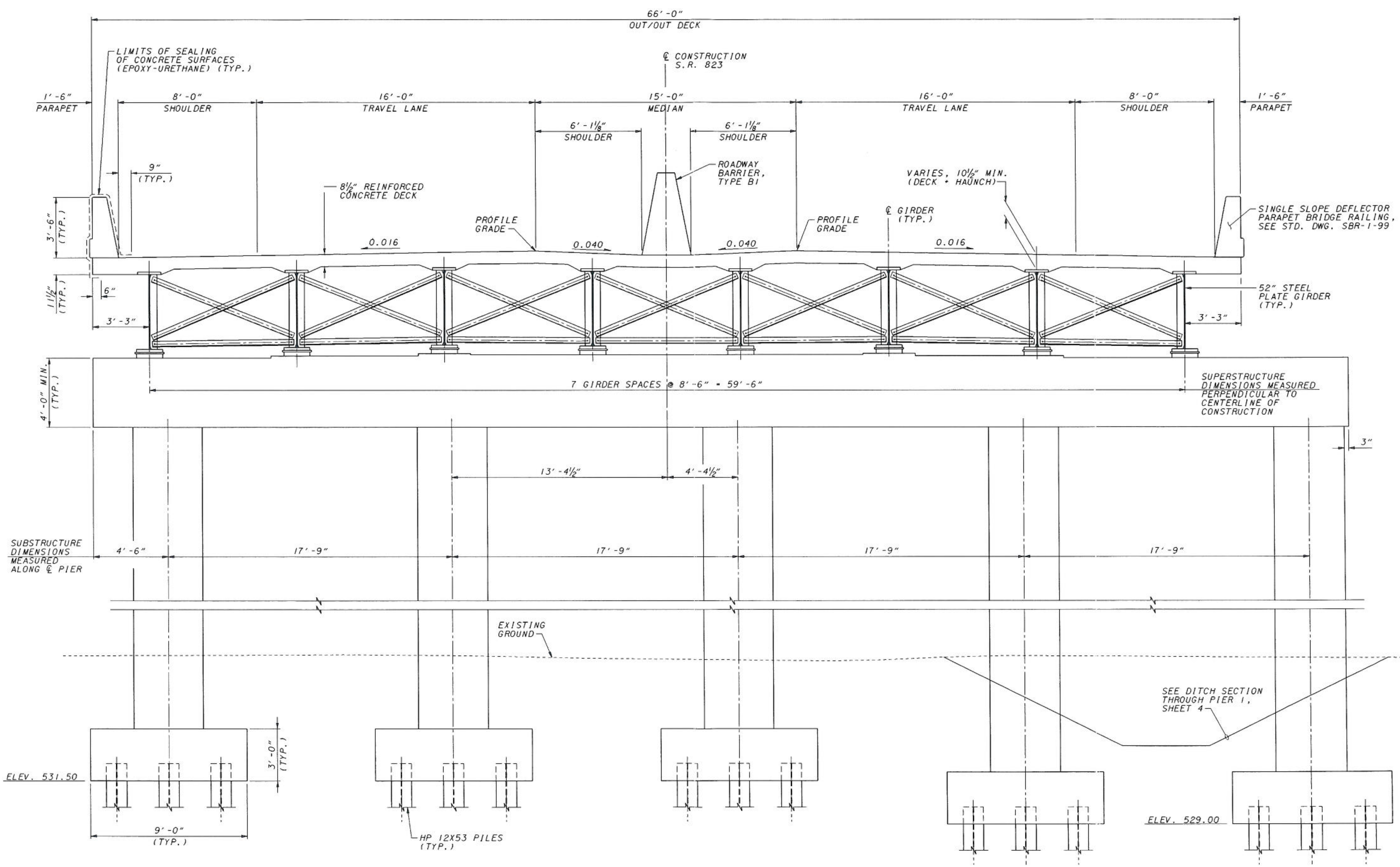

TYPICAL TRANSVERSE SECTION AND PIER 1

DESIGN AGENCY: CH2MHILL, 5775 Perimeter Drive, Suite 190, Dublin, Ohio 43017	DESIGNED DGS / CHECKED SKT	DRAWN JBA / REVISED	REVIEWED VKN / DATE 11/07 / STRUCTURE FILE NUMBER 7306792	PIER 1 ELEVATION & TYPICAL TRANSVERSE SECTION — BRIDGE NO. SCI-823-1601 — S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23

SCI-823-10.13
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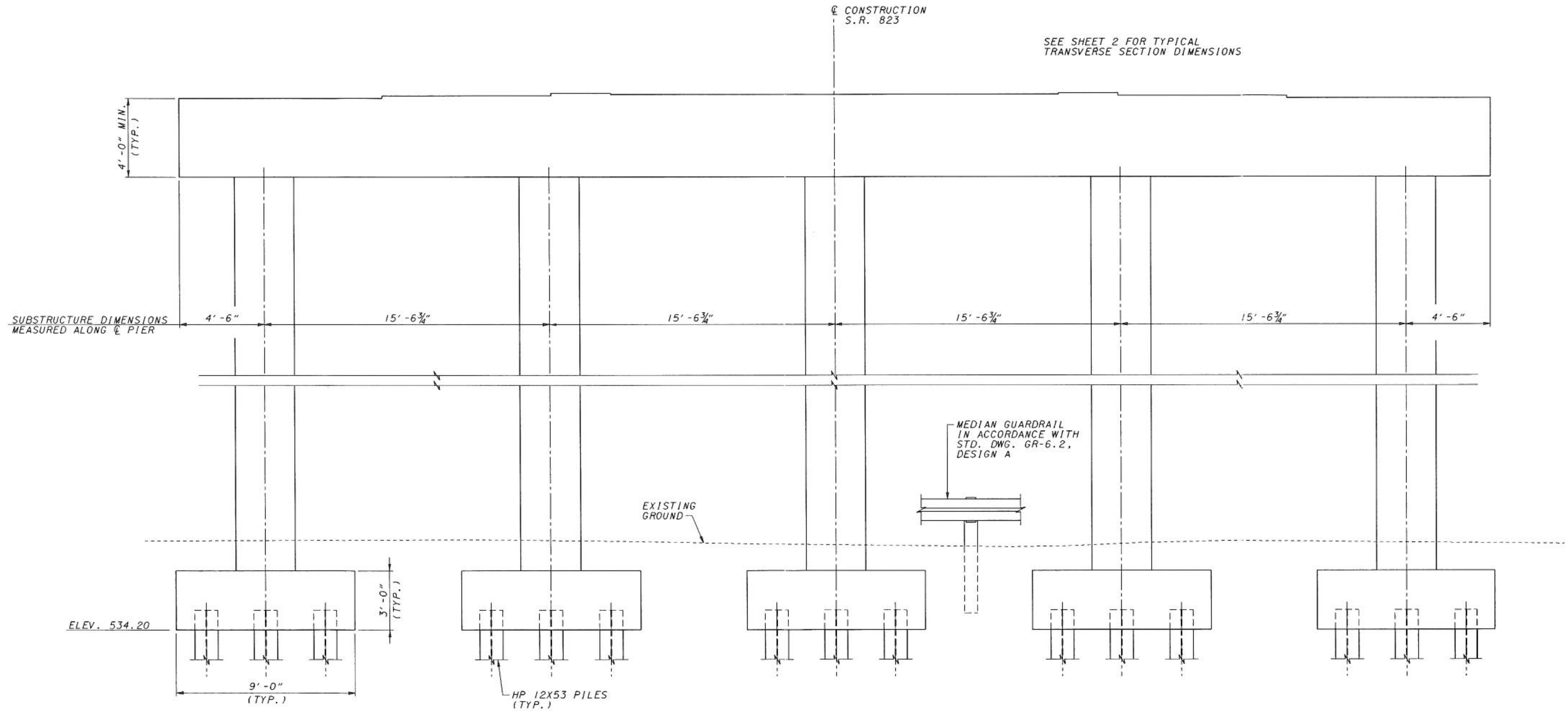

PIER 2 ELEVATION

DESIGNED DGS	DRAWN JBA	REVIEWED VKN	DATE 11/07
CHECKED SKT	REVISED	STRUCTURE FILE NUMBER 7306792	

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

PIER 2 ELEVATION
BRIDGE NO. SCI-823-1601
S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23

SCI-823-10.13
PID 79977

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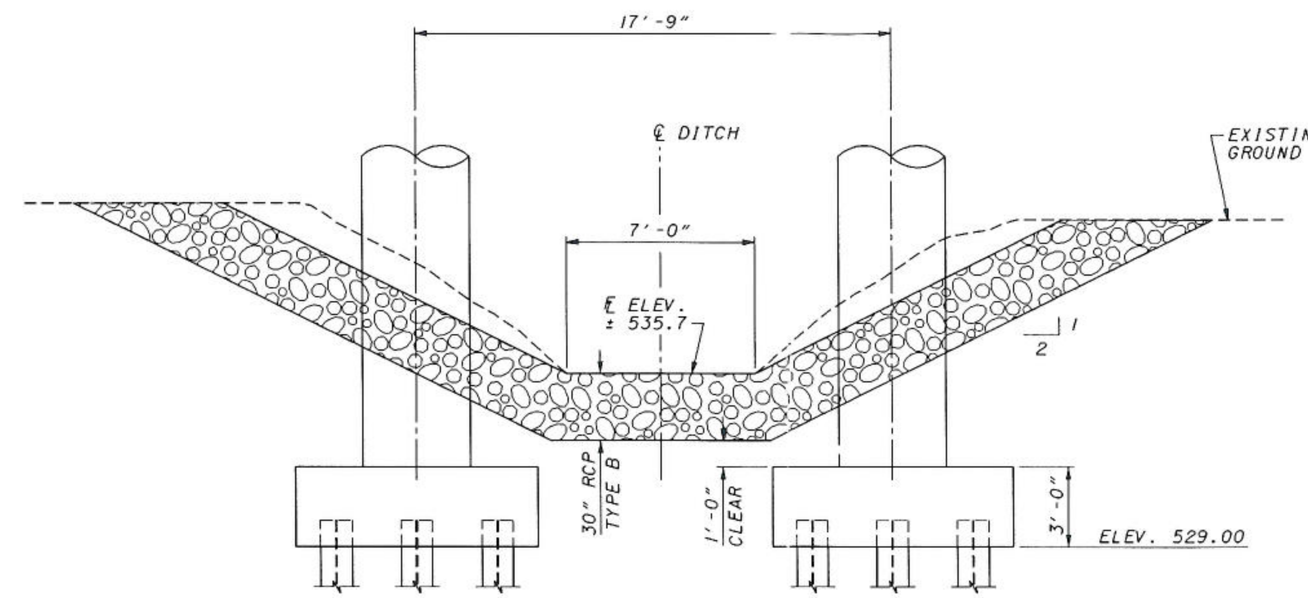
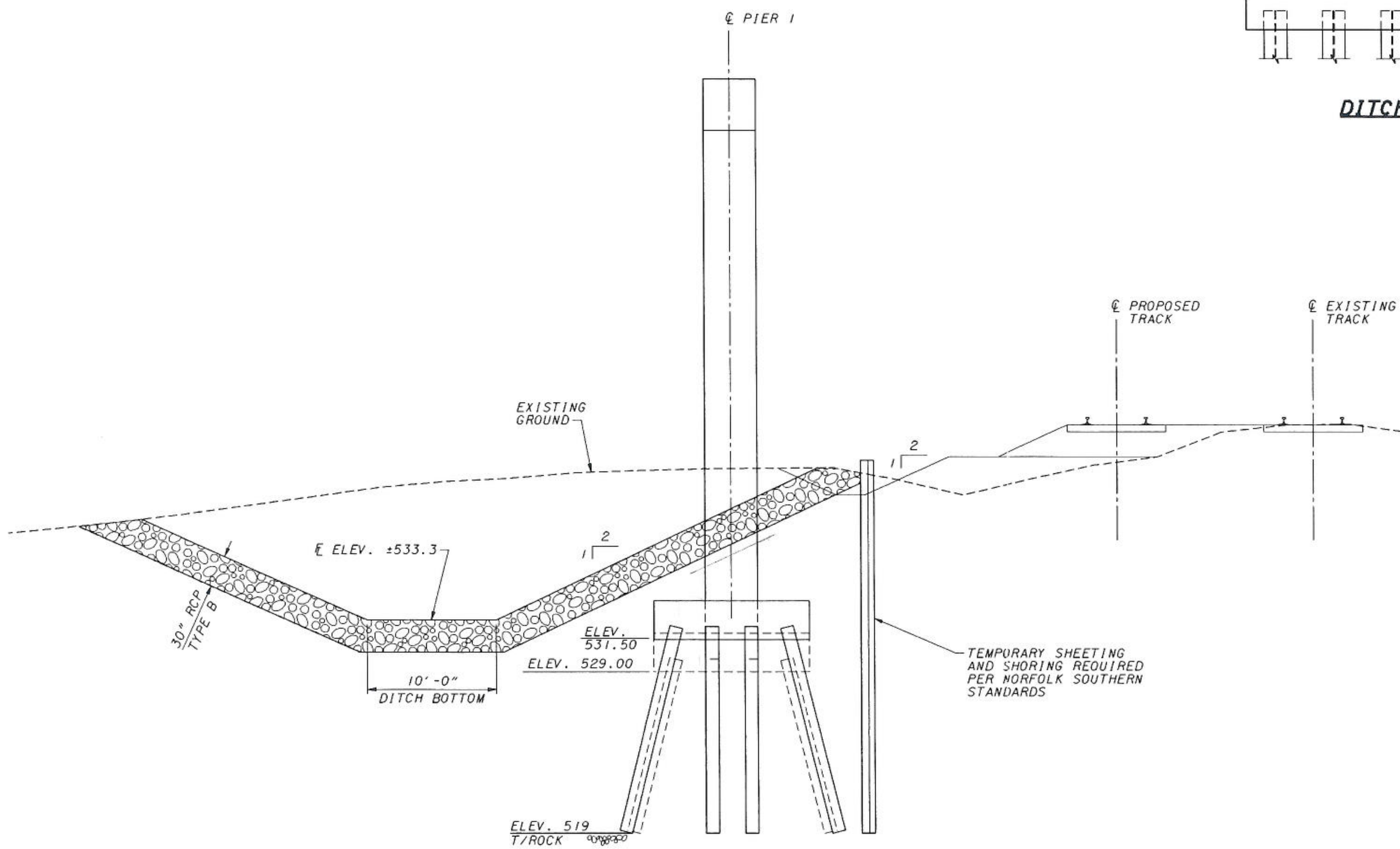


DITCH SECTION THROUGH PIER 1



SECTION AT PIER 1 LOOKING NORTH

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

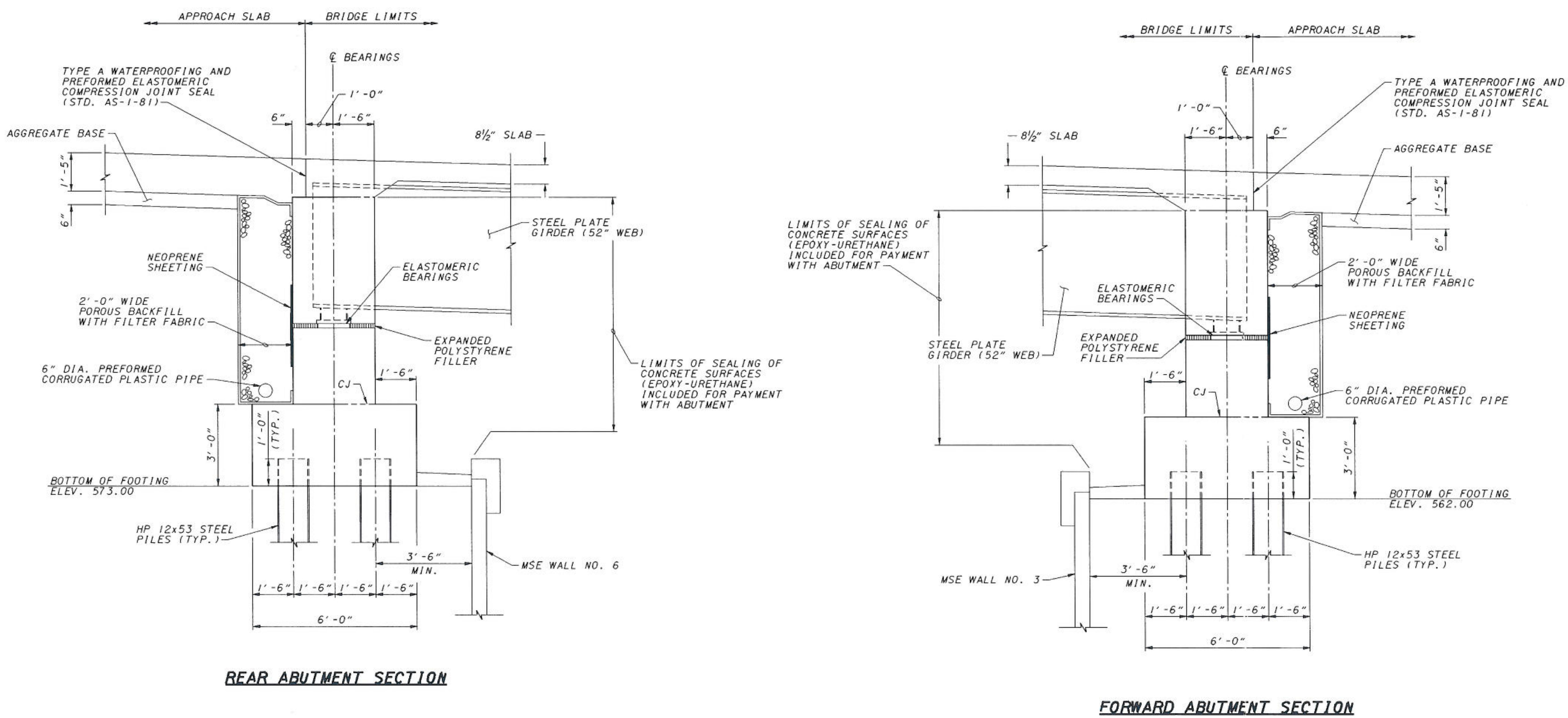
DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	VKN	11/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT		7306792	

SUBSTRUCTURE DETAILS

BRIDGE NO. SCI-823-1601

S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23

SCI-823-10.13
PID 79977



DESIGNED DGS	DRAWN JBA	REVIEWED VKN	DATE 11/07	DESIGN AGENCY CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017
CHECKED SKT	REVISED	STRUCTURE FILE NUMBER 7306792		

ABUTMENT SECTION

BRIDGE NO. SCI-823-1601

S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23

SCI-823-10.13
PID 79977

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APPENDIX C

Review Comments to DLZ's Geotechnical Report MSE Walls 3 and 6 – 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 4, 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.

Wall 6 (43.3 feet high)

Page 7 of the report, and the calculations for the wall, show the bottom of the leveling pad to be at elevation 541.2 (3-ft of embedment). Bearing Capacity, Sliding, and Global Stability Analysis show the bottom of the leveling pad being in loose, free draining sand (A-2-6). However:

- Borings TR-52 shows the leveling pad to be in stiff siltly clay (A-6a).
- Boring B-1141 shows the leveling pad to be in loose, dense gray sandy silt (A-4a).
- Page seven of the subject report also states that the leveling pad is placed upon these soils.

As a result,

1. I believe that bearing capacity, sliding and global stability analysis should be evaluated using the two soils encountered in the borings and denoted on page 7 of the report.
2. As per FHWA Manual "Mechanically Stabilized Earth Walls and Slopes", global stability analysis should include circular, deep seated, wedge, and block failures. The analysis provided was only for a deep seated circular failure in the A-2-6 material (granular soil). A block-wedge failure would be the appropriate failure mechanism for this soil.
3. Please clarify if buoyant unit weights of soil were used in the global, bearing, and sliding calculations.
4. Please clarify that the high water table was considered/included in the bearing capacity calculations.
5. Page 4 of 23 of the calculations notes the presence of the A-6a and A-4a material, but states to neglect them for settlement computations. Please clarify.

6. The global stability analysis used a c = 2000 psf for material #3 (A-6b/A-4a). Please clarify and amplify which specific borings and lab tests were used to establish this value.
7. Construction of this wall will require an approximate 15-ft cut into material three. The report doesn't address an acceptable short/long term slope angle to lay back this slope for reinforcement installation.
8. FHWA Manual "Mechanically Stabilized Earth Walls and Slopes" recommends on page 38 that for end slopes at bridges, wall embedment should be H/10 or 4-ft in our case.

Wall 3

1. Global Stability, Bearing Capacity, and Sliding calculation used Layer 4 values of c = 900 psf and an effective phi of 28-degrees. Triaxial tests on Sample ST3 from boring B-54 (at Wall 3) showed c = 336 psf. Please clarify the use of c = 900 psf. In addition, using the c = 900 resulted in a factor of safety of 1.3, which is the minimum allowed. Therefore, please clarify the sensitivity of the factor of safety to 10%-20% variations in c for layer 4.
2. Global Stability evaluations should also consider block and wedge failures along the interface between layer 3 (c = 2000) and layer 4 (c = 900), as well as at the interface with the gravel layer #5.
3. The undrained sliding resistance factor of safety was show as 0.89 on page 10 of 23 of the calculations. In hand-written lettering, it says drained sliding FS = 1.68. Please clarify and expand on this calculation.
4. Page 9 of the report states that staged construction of the wall will not work because of bearing capacity weakness (Factor of Safety as low as 0.8 using c = 900 psf). Therefore, they recommend preloading the wall area with a 1:1 steepened reinforced soil slope (RSS). As per FHWA Manual "Mechanically Stabilized Earth Walls and Slopes", Chapter 7, Section 7.2 Reinforced Slope Design Guidelines, further analysis should be performed:
   - Sliding
   - Stability. Per page 228, use both circular-arc and sliding-wedge methods, and consider failure through the toe, through the face (at several elevations), and deep seated below the toe (circular and wedge).
   - Local Bearing Failure (lateral squeeze). See page 239 for computation and note warning of FS less than 2.
5. Based on the low bearing capacity factor of safety (0.8), I do not believe that the reinforced slope can be constructed in a single stage without local bearing failure occurring.
6. In my opinion, the most technically and economically viable way to construct this wall is staged construction using a wire face MSE wall system with a permanent facing applied after completion of settlement. The settlement has been estimated to be in upwards of 9-inches. I believe that the differential settlement between the wall face and back of the reinforced zone (where the vertical overburden stress is greater) will overstress the strap connections and tilt/damage the panels. Please refer to FHWA manual pages 39, 61-62, and 82.

   I have attached a pdf file containing the pertinent FHWA manual pages.
7. I believe the transverse differential settlement issue (between the wall face and the back of the straps) should be evaluated for each of the walls on this project as per the FHWA

manual. Vertical slip joints only accommodate differential movement along the face of the wall caused by changes in the wall height and/or settlement of foundation soils.

8. Settlement issues contained in my supplemental technical memorandum for Ramps B and C, dated November 5th, 2007, should be resolved with the items within this technical memorandum before proceeding further with the current MSE wall schemes.

Report for:

Subsurface Exploration for Bridge and MSE Retaining Walls SR 823 Over Norfolk Southern Railroad and US 23, (Bridge No. Sci-823-1601) Project SCI-823-10.13 Portsmouth Bypass (PID 79977) Scioto County, Ohio

Prepared for:

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DLZ Job No. 0121-3070.03

November 4, 2007

Prepared by:

DLZ

REPORT

OF

SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

SR 823 OVER NORFOLK SOUTHERN RAILROAD AND US 23
(BRIDGE NO. SCI-823-1601)

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

SCIOTO COUNTY, OHIO

For:

CH2M Hill
5775 Perimeter Drive, Suite 190
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By:

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

DLZ Job. No. 0121-3070.03

November 4, 2007

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(BRIDGE NO. SCI-823-1601)
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 SR 823 bridge over the Norfolk Southern railroad and US 23 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 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 abutments. It is understood that driven piles will be used to support the abutments and piers, of the proposed structure. The Structure Plan and Profile Drawing is attached in Appendix I.

It is understood that MSE walls will be constructed at approximately stations 895+54 and 899+07 to contain the embankment fill material at the rear and forward abutment locations, respectively. The proposed retaining wall systems at the rear and forward abutments will hereafter be referred to as Wall No. 6 and Wall No. 3, respectively.

Based upon the provided drawings, it is estimated that the maximum height of Wall No. 6 is approximately 43.3 feet and the maximum height of Wall No. 3 is approximately 39.4 feet. These heights are based upon the difference between the proposed grade of SR 823 at the wall locations and the leveling pad elevations shown on the provided drawings. It should be noted that these wall heights include the embedment depth of 3.0 feet.

The analyses and recommendations presented in this report are 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 nine borings for the SR 823 bridge and retaining walls. Six structure borings (TR-49 through TR-52, TR-50A, and TR-49A) were drilled for previously proposed structure configurations. Two roadway borings (B-1141 and B-1142) were drilled in the vicinity of the bridge for the proposed roadway. Boring B-54 was drilled near the forward abutment location for the currently proposed structure configuration. The boring logs are presented in Appendix II. Information concerning the drilling procedures is also present in Appendix II.

The boring locations were planned and staked in the field by representatives of DLZ and Lockwood, Lanier, Mathias & Noland, Inc. (2LMN). The surveyed locations and ground surface elevations of the borings were determined by representatives of 2LMN. The surveyed locations of the borings are shown on the Structure Plan and Profile Drawing presented in Appendix I.

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 this structure 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, that are generally composed of silty clay, coarse sand, gravel, and cobbles. 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. These deposits ranged 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 are evident in the borings drilled on the eastern end of the interchange. Borings drilled west of the Fairground Road site encounter progressively thinner layers of the shale bedrock. West of the Norfolk and Southern Railroad and immediately east of US 23, the shale is no longer encountered at the top of rock. West of the Norfolk and Southern railroad, Berea Sandstone is 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.

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 generally encountered 1 to 4 inches of topsoil at the existing ground surface. Below the surface material, cohesive soils consisting of sandy silt (A-4a) to silty clay (A-6b) were encountered to depths ranging from 13.5 to 20.5 feet below the ground surface. Below the cohesive soils, layers of cohesionless soils consisting of gravel with sand and silt (A-2-4) to coarse and fine sand (A-3a) were encountered to the top of bedrock at depths ranging from 21.0 to 34.5 feet below the ground surface.

4.2.2 Bedrock Conditions

Bedrock was confirmed by coring in all borings. Borings B-1141, B-1142, TR-51 and TR-52 were drilled for the rear abutment and Pier 1 locations. In these borings, bedrock was encountered at depths ranging from 25.5 to 34.5 feet below the ground surface, or from elevation 519.0 to 526.0. Soft to medium hard black shale (Sunbury shale) was encountered at the top of rock. Also in these borings, hard to very hard sandstone was encountered below the shale layers, at depths ranging from 28.5 to 45.1 feet below the ground surface, or from elevation 513.6 to 517.6. Borings TR-49 through TR-50, TR-49A, TR-50A, and B-54 were drilled near Pier 2 and the forward abutment of the proposed structure. These borings encountered hard to very hard sandstone at the top of rock at depths ranging from 21.0 to 31.0 feet below the ground surface, or from elevation 507.7 to 519.5.

The recovery in each core run varied between 45 and 100 percent. The rock quality designation (RQD) of the bedrock ranged between 0 and 100 percent with an average of 59 percent, indicating fair rock quality.

4.2.3 Groundwater Conditions

Seepage was observed in all borings. Seepage was first observed at depths ranging from 11.0 to 28.0 feet below the ground surface. Measurable water levels were observed in borings B-1141, B-1142, B-54, and TR-51 at depths ranging from 20.2 to 31.0 feet prior to rock coring operations. Final water levels were present in all borings, except boring B-54 and were observed at 6.0 to 23.3 feet below the ground surface. The final water levels included water that was used for rock coring 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. Therefore, the readings indicated on the boring logs may not be representative of the long-term groundwater level. Long-term monitoring would be necessary to obtain a more accurate estimate of the groundwater table elevation.

A piezometer was installed in boring B-54, in the area of the proposed forward abutment, to monitor the groundwater level. This piezometer was screened in the granular layers overlying bedrock. The groundwater levels in this piezometer ranged from 20.2 to 23.5 feet below the existing ground surface (el 518.6 to 515.3). The average phreatic level was approximately 21.4 feet below the ground surface, corresponding to an elevation of 517.4.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The recommendations contained in this section pertain to the proposed bridge, abutment retaining walls, and approach embankments adjacent to the bridge essentially between stations 895+17 and 899+44. The recommendations for portions of the embankments that extend beyond these limits are not included in this report, but are presented in the Final Report of Subsurface Exploration and MSE Wall and Embankment Evaluations for Proposed US 23/SR 823 Interchange.

It is understood that driven HP 14x73 piles are preferred to support the proposed structure. Based on this assumption, stability analyses and settlement calculations were performed for the proposed MSE walls.

5.1 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

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 a bearing capacity analysis were performed for the MSE walls at the abutment locations 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 are based on the results of the laboratory strength testing, in-situ moisture content, hand penetrometer values, typical correlations, and engineering judgment. Table 1 presents the strength parameters assumed in the analyses. A summary of the strength and consolidation testing is included in Appendix III. The results of laboratory testing are also included in Appendix III.

Because of the low undrained shear strengths of the foundation soils, consolidated undrained triaxial testing (CIU) was performed on a selected sample to determine required parameters for staged construction evaluations of MSE Wall No. 3. Tests run on a cohesive silt (A-4b) sample obtained from boring B-54 reported an angle of shearing resistance (from total stress curve, F_{cu}) of 17.2 degrees. The results of shear strength testing are included in Appendix III.

Table 1- Soil Parameters Used in Stability Analyses

Zone	Soil Type	Unit Weight	Strength Parameters	
			Undrained	Drained

		(pcf)	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. 6) (B-1141 & TR-52)	Very Stiff Clay	120	2000	0	0	29
	Med. Dense Gravel, Sand, and Silt	120	0	28	0	28
Foundation Soil (Wall No. 3) (B-54 & TR-49)	Very Stiff Clay	120	2000	0	0	29
	Med. Stiff Clay	120	900	0	0	28
	Gravel and Sand	120	0	32	0	32

+ Use shear strength values for internal design of geogrid/geotextile slope reinforcement.

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. 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.

Laboratory strength testing was performed to determine the strength of the granular layers that exhibited low SPT N-values. A direct shear test was performed on loosely remolded samples of granular material from boring TR-61, located approximately 600 feet south of the proposed bridge. Before the direct shear test commenced, the sample was saturated in the mold with free water. The sample was subsequently stirred prior to beginning the test to ensure a loose condition. The results of the tests indicated friction angles between 42.1 and 45.7 degrees. However, in the stability analyses, a friction angle ranging from 28 to 32 degrees was used for the granular layers. These values were selected in part based upon the gradations and past experience with similar materials. Results of the direct shear test are presented in Appendix III.

5.1.3 MSE Wall Evaluations and Recommendations Rear Abutment – Wall No. 6

In the analysis of MSE Wall No. 6, the subsurface profile encountered by borings B-1141 and TR-52 are considered to be the most critical with respect to stability.

As per ODOT's Supplemental Specification 840 (SS 840), Section 840.04 A, the maximum wall height, measured to the proposed grade, is estimated to be

43.3 feet. As per the provided drawings, the top of the leveling pad for this wall is assumed to be at elevation 541.7.

Borings B-1141 and TR-52 generally encountered very stiff sandy silt (A-4a) and silt and clay (A-6a) between elevation 541.2 (the bottom of the leveling pad) and approximate elevation 539.9. Below these cohesive soil layers, borings generally encountered loose gravel with sand, silt and clay (A-2-6) to the top of bedrock at approximately elevation 523.0.

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 indicate that the factors of safety for global stability, bearing capacity and stability (overturning and sliding) are all above the minimum recommended values.

The stability analyses indicate that a minimum reinforcement length of 0.85(H+D) or 37.1 feet should be used for this wall. The reinforcement length may be increased to achieve the required internal stability.

The maximum settlement occurring at the centerline of Wall No. 6 is estimated to be approximately 1 inch. Differential settlement at the wall face, between the centerline and the toe of the proposed MSE wall was calculated to be approximately 0.1 percent. Generally, 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) or one percent of the wall length considered. As a result, the calculated differential settlement is acceptable. Settlement is 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 2 presents the MSE retaining wall parameters and results of the analyses for MSE Wall No. 6.

Table 2 - MSE Retaining Wall Parameters and Analysis Results
Rear Abutment Location, MSE Wall No. 6

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi' = 30°$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 28°(0.67) = 0.36
Allowable Bearing Capacity – Undrained Condition q_{all} = *NA (granular foundation soils below MSE wall leveling pad)*
Allowable Bearing Capacity – Drained Condition q_{all} = 6,933 psf
Global Stability Factor of Safety – Undrained Condition = 2.0 Factor of Safety – Drained Condition = 1.8 Factor of Safety – Drained Seismic Condition = 1.7
Estimated Settlement of MSE Volume δ_A = Negligible (toe of embankment / wall) δ_B = 1 inch (centerline of embankment / wall) Differential Settlement = 0.1% (maximum allowable is 1.0% ODOT BDM 204.6.2.1)
Maximum Full Height of MSE Wall = 43.3 feet (Including Embedment Depth) Minimum Embedment Depth = 3.0 feet* Minimum Length of Reinforcement for External Stability, 0.85(H+D)

* Assumed top of leveling pad elevation is 541.7. Embedment depth may vary depending on actual top of leveling pad., however, a minimum embedment depth of 3.0 feet is required.

5.1.4 MSE Wall Evaluations and Recommendations Forward Abutment – Wall No. 3

In the analysis of MSE Wall No. 3, the subsurface profile encountered by borings B-54 and TR-49 are considered to be the most critical with respect to stability.

As per ODOT's Supplemental Specification 840 (SS 840), Section 840.04 A, the maximum wall height, measured to the proposed grade, is estimated to be 39.4 feet. As per the provided drawings, the top of the leveling pad for this wall is assumed to be at elevation 535.0.

Borings B-54 and TR-49 generally encountered soils consisting of silt (A-4b) to silt and clay (A-6a) between elevation 534.5 (the bottom of the leveling pad) and approximate elevation 520.7. The consistency of these cohesive soils was very stiff between elevation 534.5 and elevation 533.7, but stiff to medium stiff below elevations 533.7. Below the cohesive soil layers, borings generally encountered loose gravel with sand (A-1-b) and coarse and fine sand (A-3a) to the top of bedrock at approximate elevation 507.7.

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 indicate that the factors of safety for global stability, drained bearing capacity and overturning are all above the minimum recommended values. However, the factor of safety against undrained bearing capacity failure is found to be 0.81, which is well below the minimum required value of 2.5. The factor of safety against sliding under the undrained condition is found to be 0.89, which is well below the minimum required value of 1.5.

In order to construct the MSE wall while maintaining the minimum factor of safety against undrained bearing capacity and sliding, the possible use of staged construction was investigated. Additional analyses were performed, assuming that an increase in the undrained shear strength of the foundation soils would occur due to the consolidation after each stage of wall construction. Calculations using the maximum wall height of 39.4 feet, indicate that the required factor of safety of 2.5 against undrained bearing capacity failure will not be achieved with the estimated increase in shear strength in the foundation soils. As a result, the wall cannot be constructed using staged construction.

The use of preloading to surcharge the foundation soils prior to construction of the MSE wall was also investigated. The toe of this preloading embankment should be located approximately at station 898+82, or 25 feet east (down-station) of the face of Wall No. 3. Analyses indicate that an embankment using 1H:1V side slopes, which is approximately 36 feet tall would sufficiently surcharge the existing foundation soils. Flatter slopes could be used, but would require the toe of the slope to extend into the pavement surface of existing US 23, which may require lane closures. Therefore, options using flatter slopes were not explored.

A reinforced soil slope, using a geogrid or geotextile fabric, is required for the stability of the recommended 1H:1V side slopes. Global stability analyses were performed to determine the minimum required reinforcement length. The analyses indicate that a minimum reinforcement length of 20 feet should be required for external stability. Other analyses are required to determine the internal stability of the reinforced slopes and are not considered to be within the scope of work for this report. The geotextile supplier should determine the length and the spacing of the reinforcement required for internal stability. The minimum required reinforcement length may be increased if necessary.

As with the MSE walls, the analyses for the reinforced soil slopes assume a unit weight of 120 pounds per cubic foot (pcf) and an effective friction angle of 34 degrees for the backfill material in the reinforced zone. In accordance with ODOT guidelines, 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.

The preloading embankment should be constructed up to elevation 574.1 (approximately 36 feet high) and allowed to consolidate to at least ninety percent of primary consolidation (U=90%). After the foundation soils have sufficiently consolidated under the surcharge load, the preloading embankment should be removed, and the proposed MSE wall constructed. Stability analyses indicate that a reinforcement length of 0.85(H+D) or 33.5 feet should be used for Wall No. 3. This value is a minimum and may be increased for internal stability.

Calculations indicate that approximately 9 inches of settlement will occur at the proposed wall face during the preloading. When the surcharge embankment is removed, it is anticipated that the foundation soils will rebound slightly before they consolidate again under the weight of the MSE wall and fill. Calculations using the recompression index, for the fine-grained foundation soils indicate that the maximum settlement beneath the MSE wall will be approximately 2 inches with differential settlement of approximately 0.1 percent. Generally, 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) or one percent of the wall length considered. As a result, the calculated differential settlement is acceptable. Settlement is 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.

Based on the results of time-rate of consolidation calculations, the time to 90 percent consolidation (U=90%) is estimated to be 81 days. This duration can be shortened through the use of prefabricated vertical drains (wick drains). The consolidation times using various wick drain spacing are presented in Table 3.

Table 3 - Wick Drain Spacing and Consolidation Periods (Preloading)

SR 823 over Norfolk Southern Railroad and US 23, MSE Wall No. 3		
Spacing (ft)	**+Time to U=90% (days)**	**Approximate Depth of Wick Drains (ft)**
5	20	20
7	30	20
9	40	20

+ U=90%, required minimum consolidation of foundation soils under preloading embankment.

It is recommended that wick drains in a triangular pattern be installed a minimum of 15 feet beyond the limits of the proposed embankment, wherever possible. 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 20 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 of sand will provide a free draining layer beneath any embankment fill, allowing pore water to be expelled.

Pore water pressures and soil settlements should be monitored in the fine-grained layers of the foundation soils, during construction of the embankments and throughout the preloading process. 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.

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

Table 4 - MSE Retaining Wall Parameters and Analysis Results Forward Abutment Location, MSE Wall No. 3

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi' = 30°$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 28°(0.67) = 0.36
Allowable Bearing Capacity – Undrained Condition *q_{all} = NA (undrained condition not mobilized due to preloading)*
Allowable Bearing Capacity – Drained Condition q_{all} = 6,559 psf
Global Stability Factor of Safety – Undrained Condition = 1.3 *(Using Initial Shear Strengths)* Factor of Safety – Drained Condition = 1.9 Factor of Safety – Drained Seismic Condition = 1.8
Estimated Settlement of MSE Volume δ_C = Negligible (toe of embankment / wall) *(After Preloading)* δ_D = 2 inches (centerline of embankment / wall) *(After Preloading)* Differential Settlement = 0.1% (maximum allowable is 1.0% ODOT BDM 204.6.2.1)
Maximum Full Height of MSE Wall = 39.4 feet (Including Embedment Depth) Minimum Embedment Depth = 3.0 feet* Minimum Length of Reinforcement for External Stability, 0.85(H+D)

* Assumed top of leveling pad elevation is 535.0. Embedment depth may vary depending on actual top of leveling pad., however, a minimum embedment depth of 3.0 feet is required.

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 proposed use of a multi-span bridge and the soft soil conditions encountered by the borings, it is assumed that spread footing foundations will not be considered. Also, recommendations for drilled shaft foundations are not presented in this report. Recommendations for drilled shaft and additional foundation alternatives can be provided 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 estimated pile tip elevations for the proposed bridge. The bedrock surface varies across the project area. The approximate pile tip elevations presented in Table 5 indicate the approximate bedrock elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

**Table 5-Summary of Driven Pile Tip Elevations, HP 14x73*
SR 823 over Norfolk Southern Railroad and US 23**

Substructure	Boring Number	Existing Ground Surface Elevation (Ft)	Estimated Pile Tip Elevation (Ft)
Rear Abutment	TR-52	558.0	523.0
Pier 1	TR-51	544.5	516.0
Pier 2	TR-50A	539.3	511.8
Forward Abutment	B-54	538.7	506.7

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

It is anticipated that piles will encounter refusal upon shale or sandstone bedrock at a depth of approximately 27.5 to 35.0 feet below the ground surface. Based upon the degree of weathering and the strength characteristics of the shale bedrock evident from the borings, it is anticipated that the piles driven at the rear abutment and Pier 1 locations will penetrate approximately three feet below the top of rock elevation in boring TR-52 and TR-51.

If driven to refusal, the allowable structural 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 rear abutment location. Based upon guidance from the ODOT's Bridge Design Manual (BDM), reinforced pile points should not be used when the piles are driven to bear on Shale as anticipated at the rear abutment location. Medium hard shale bedrock was also encountered at the top of rock in the area of Pier 1. However, the shale layer was approximately 2.5 feet thick and was underlain by hard sandstone. Consequently, reinforced pile points may be considered when the piles are driven at the Pier 1 location. Borings drilled for the foundations of Pier 2 and the forward abutment encountered hard to very hard sandstone at the top of rock. It is recommended that reinforced pile points be used to protect the piles at these locations.

At the abutment locations, 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 proposed MSE walls. Due to the tendency of certain shales to "relax", it is recommended that the contractor restrike the piles at the rear abutment and Pier 1 locations, seven days after the pile installation to ensure that the allowable bearing capacity of the pile is achieved.

At the rear abutment, due to the granular nature of the foundation soils, it is anticipated that most of the settlement will occur as the MSE wall is being constructed. Consequently, downdrag should not be a concern. Piles may be driven without a waiting period after the rear abutment MSE wall has been constructed. Conversely, to mitigate the effect of downdrag forces on the pile foundation at forward abutment, fill should be placed to the proposed roadway grade level and allowed to consolidate prior to driving piles. The piles should not be driven until at least 70 percent of the primary consolidation has occurred (U=70%) at the forward abutment location. Without using wick drains, the estimated consolidation period (prior to driving piles) would be approximately 38 days. Time-rate of consolidation calculations are presented in Appendix IV. It may be desirable to use wick drains to accelerate the consolidation of the foundation soil and shorten the waiting period prior to driving piles at the forward abutment. The estimated consolidation periods (U=70%) for various spacing options are presented in Table 6. No waiting periods are required at the pier locations.

Table 6 - Wick Drain Spacing and Consolidation Periods (Driven Piles)

SR 823 over Norfolk Southern Railroad and US 23, Forward Abutment		
Spacing (ft)	+Time to U=70% (days)	Approximate Depth of Wick Drains (ft)
5	10	20
7	15	20
9	20	20

+ U=70%, required minimum consolidation prior to driving piles for the forward abutment.

5.3 Embankment Evaluations and Recommendations
Approach Embankments

Global stability analyses were performed for the earthen approach embankments near the rear and forward abutments. The maximum height of the approach embankment near the rear abutment is estimated to be 34.5 feet, while the maximum height of the approach embankment near the forward abutment is estimated to be 35.9 feet. As per ODOT Office of Geotechnical Engineering, the following material properties were used for the stability analyses of the embankments; 1) a cohesion value of 2000 pounds per square foot (psf) and a friction angle of zero degrees were used for the undrained analysis and 2) a cohesion value of 300 psf and a friction angle of 28 degrees were used for the drained and seismic analyses. Based on the results of the analyses, factors of safety greater than the minimum recommended values for undrained, drained, and seismic conditions can be achieved for the earthen embankments with side slopes of no steeper than 2H:1V. Drawings illustrating the results of the stability analyses are included in Appendix IV.

5.4 General Earthwork Recommendations

The proposed alignment traverses a gently to moderately sloping area. The proposed grade is anticipated to be a maximum of 35.9 feet higher than the existing grade.

Consequently, the placement of fill will be required to construct the approach embankments at the abutment locations. However, excavations in excess of 15 to 18 feet deep are anticipated for the construction of the pier foundations and the MSE walls.

Approximately 1 to 5 inches of topsoil were encountered at the ground surface. All topsoil and vegetation within the footprint of the new embankment and roadway should be removed prior to any 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 borings (B-1102, B-1103, B-1129, and B-1150) drilled for other features of the interchange were tested to determine the organic content. The results indicate organic contents ranging from 3.74 to 6.12 percent, which are considered to be slightly to moderately organic. Although no organic soils were encountered in borings drilled for the proposed structure, 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 or organic soils at the proposed bridge and embankment areas, and to 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. In order to determine the suitability of the supporting soils, excavation bottoms should be examined by the geotechnical engineer prior to placement of reinforcing steel and concrete

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 borings at depths between 11.0 and 28.0 feet below the ground surface. Measurable water levels were observed in some of the borings prior to rock coring at depths between 20.2 and 31.0 feet below the ground surface. As a result, shallow excavations, less than five feet deep, are expected to encounter little if any seepage.

Due to the existing slope geometry relative to the proposed rear abutment (Wall No. 6) MSE wall, significant excavations will be required to accommodate the installation of the soil reinforcing straps. Based upon the provided drawings, the bottom of the MSE wall leveling pad is anticipated to be at elevation 541.2. Borings B-1141, B-1142, and TR-52, drilled in the area of the rear abutment encountered water-bearing granular soils at approximate elevation 539.9. Seepage was observed in these borings at approximate elevation 535.0. Although groundwater or seepage was not observed above the bottom of excavation, the contractor should be prepared to deal with any seepage zones that may be present which were not observed in the borings.

At the Pier 1 location, boring TR-51 first encountered seepage at elevation 531.5. The bottom of the proposed pile cap, as shown on the provided drawings, is elevation 530.0. As a result, the Contractor should anticipate encountering seepage and/or high groundwater levels while excavating for the Pier 1 structure.

A piezometer was installed in boring B-54, located in the area of the proposed forward abutment. The boring encountered a layer of fine-grained soil underlain by a water-bearing granular layer, which in turn overly the bedrock. The average phreatic level in the piezometer was at approximate elevation 517.4 or 21.4 feet below the existing ground surface. The anticipated bottom of leveling pad excavation is approximately elevation 534.5. Consequently, little if any seepage is anticipated for the excavations of the wall foundation near the forward abutment.

Note that the actual groundwater levels could vary from those observed at the time the borings were drilled. The contractor should be prepared to dewater any excavations where seepage or groundwater is encountered. In addition to groundwater, the contractor should be prepared to deal with water flow and precipitation that may enter any excavations.

6.0 CLOSING REMARKS

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

Respectfully submitted,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Eric Tse, P.E.
Senior Geotechnical Engineer

sjr

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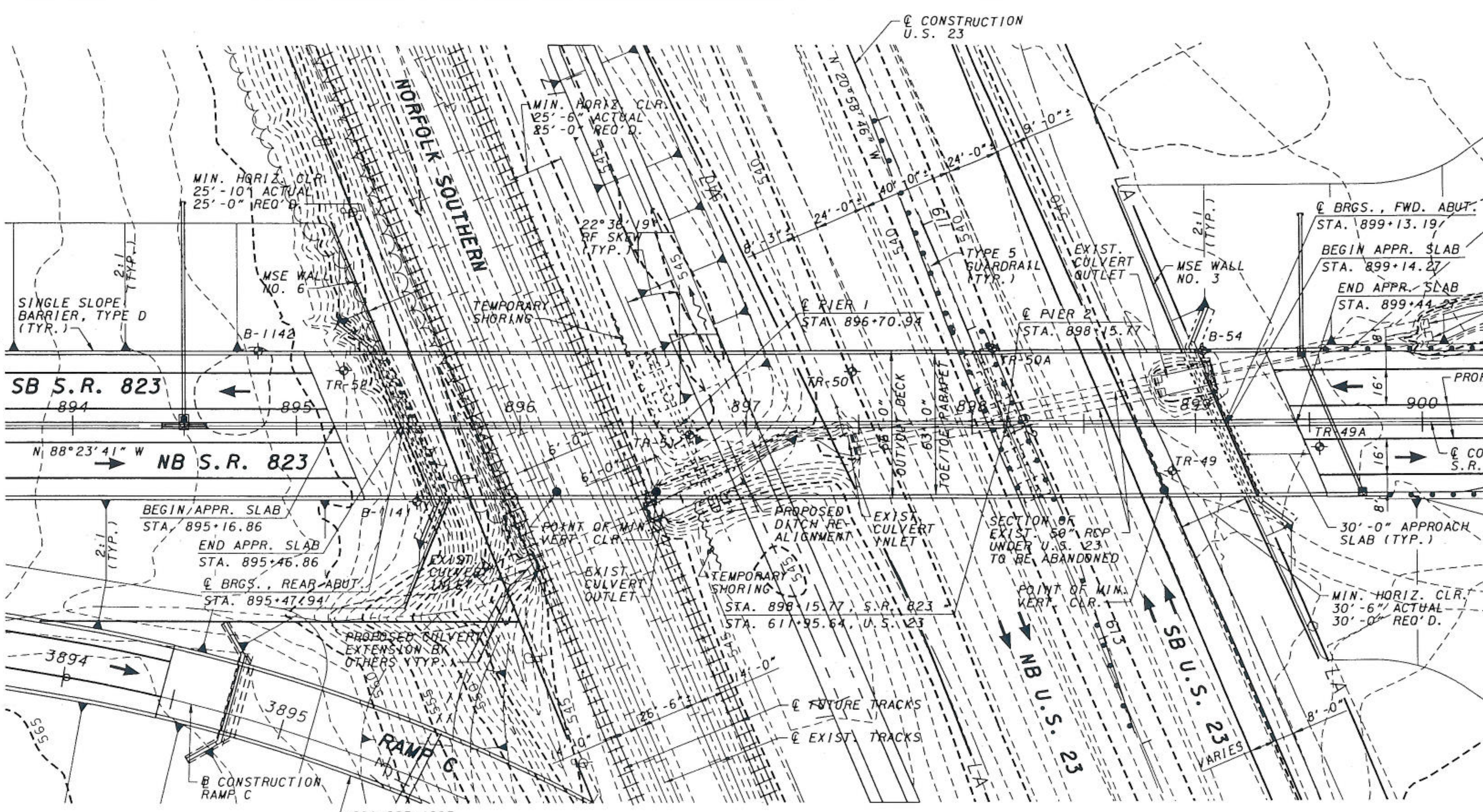
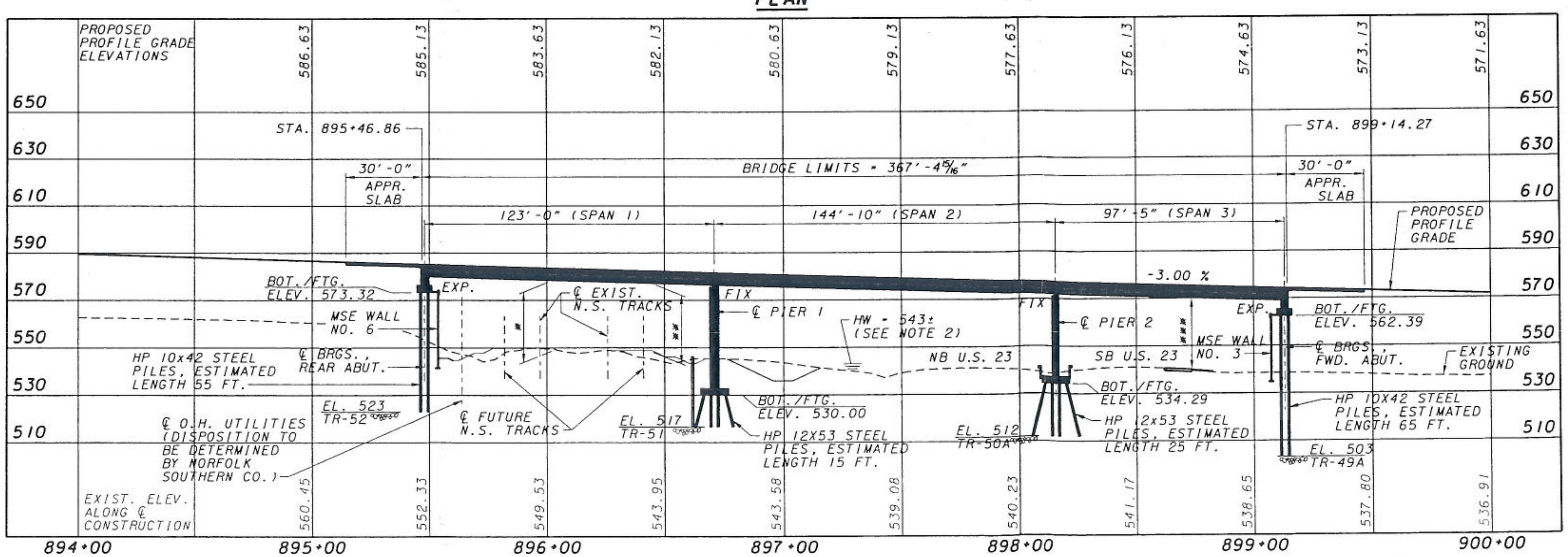
APPENDIX I

Structure Plan and Profile Drawings - 11"x17"



PLAN



* MIN. VERT. CLR. 27'-1" ACTUAL 23'-5½" REQ'D.

** MIN. VERT. CLR. 27'-1" ACTUAL 23'-5½" REQ'D.

*** MIN. VERT. CLR. 28'-8" ACTUAL 17'-0" REQ'D.

PROFILE ALONG PROFILE GRADE LINE, S.R. 823

BENCHMARKS

TRAFFIC DATA

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

LEGEND

INDICATES 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.
3. MEDIAN GUARDRAIL AT PIER 2 TO BE IN ACCORDANCE WITH STANDARD DRAWING GR-6.2, DESIGN A.

PROPOSED STRUCTURE

TYPE: THREE-SPAN COMPOSITE STEEL PLATE GIRDERS (WEATHERING ASTM A709, GR 50W) WITH REINFORCED CONCRETE DECK ON SEMI-INTEGRAL ABUTMENTS BEHIND MSE WALLS WITH CAP AND COLUMN PIERS

LENGTH OF SPAN: 123'-0", 144'-10", 97'-5" C-C BEARINGS, MEASURED ALONG ℄ CONSTRUCTION

ROADWAY: 30'-1½" TOE/TOE BARRIERS (RB)
30'-1⅛" TOE/TOE BARRIERS (LB)

SIDEWALK: NONE

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

SKEW: 22°36'19" RIGHT FORWARD

WEARING SURFACE: MONOLITHIC CONCRETE

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

ALIGNMENT: TANGENT

CROWN: 0.016 FT/FT

LATITUDE: N 38°53'32"

LONGITUDE: W 82°59'58"

DESIGN AGENCY	DESIGNED	DRAWN	REVIEWED	DATE			
CH2MHILL 5775 Perimeter Drive, Suite 190 Dublin, Ohio 43017	DGS	JBA	SCJ	09/07	SCIOTO COUNTY STA. 895+46.86 TO STA. 899+14.27	SITE PLAN BRIDGE NO. SCI-823-1601 S.R. 823 OVER NORFOLK SOUTHERN & U.S. 23	SCI-823-10.13 PID 79977
	CHECKED	REVISED	STRUCTURE FILE NUMBER				
	SKT		7306792				

1 / 5

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Nine (9) Borings

GENERAL INFORMATION
DRILLING PROCEDURES AND LOGS OF BORINGS

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

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

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

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

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

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

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

LEGEND – BORING LOG TERMINOLOGY

Explanation of each column, progressing from left to right

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

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

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

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

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

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

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

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

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

9. Soil Description

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

   Granular Soils – Compactness

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

   Cohesive Soils – Consistency

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

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

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

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

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

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

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

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

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

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

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

10. Rock Hardness and Rock Quality Designation

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

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

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

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

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

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

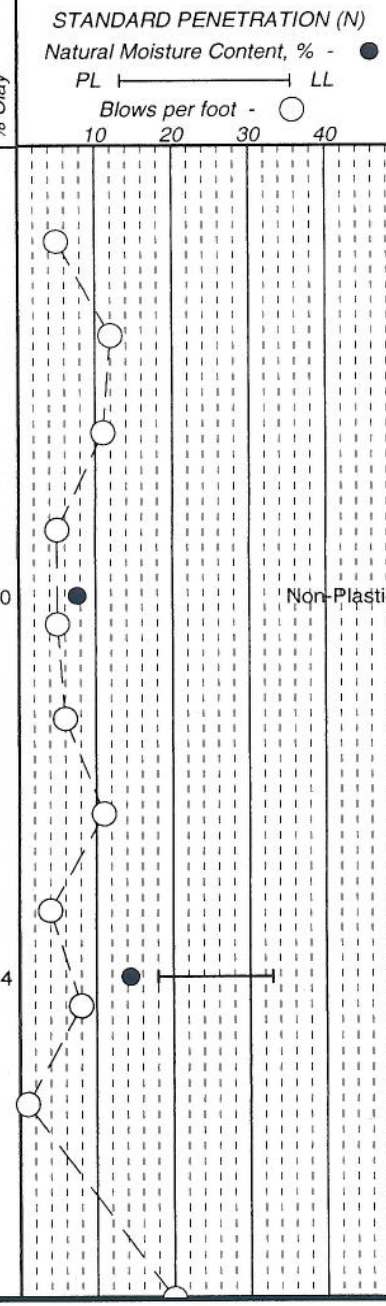
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-52 | Location: Sta. 895+21.1, 24.8 ft. LT of SR 823 CL | Date Drilled: 03/15/05

WATER OBSERVATIONS: Water seepage at: 23.0'-30.0'
Water level at completion: 27.0' (prior to coring)
6.0' (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
	558.0												
0.1	557.9						Topsoil - 1"						
		2 2 3	4	1		4.5+	Hard gray SILTY CLAY (A-6b), trace to little fine to coarse sand; contains shale fragments; damp.						
5		3 5 7	10	2		4.5+							
		10 7 4	9	3		4.5+							
10		1 2 3	8	4		4.5+							
10.5	547.5	2 2 3	7	5			Loose gray GRAVEL WITH SAND AND SILT (A-2-4), trace to little clay; damp.	66	4	--	2	18	10
13.0	545.0	2 3 3	16	6		3.75	Very stiff gray SILT AND CLAY (A-6a), trace fine to coarse sand, trace fine to coarse gravel; moist.						
15		3 5 6	16	7		4.0	@ 16.0', brown.						
		2 2 2	18	8		1.0	@ 18.5'-20.0', stiff, moist to wet.						
20 / 20.5	537.5	2 4 4	12	9			Loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); damp.	35	20	--	17	14	14
23.0	535.0	WOH WOH 1	18	10		<0.25	Very soft brown CLAY (A-7-6), trace fine sand; wet.						
25 / 25.5	532.5	W O H	18	11			Very loose brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.						
30		13 15 5	17	12			@ 28.5-30.0', medium dense.						



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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-52 | Location: Sta. 895+21.1, 24.8 ft. LT of SR 823 CL | Date Drilled: 03/15/05

WATER OBSERVATIONS: Water seepage at: 23.0'-30.0'
Water level at completion: 27.0' (prior to coring)
6.0' (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	528.0						Medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); wet.							
32.0	526.0	22 50/5	10	13			Severely weathered black SHALE.							50+
35.0	523.0	Core 120"	Rec 120"	RQD 35%	R1		Medium hard black SHALE; moderately weathered, carbonaceous, laminated, broken to moderately fractured.							
40.4	517.6						Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thickly bedded, slightly fractured.							
45.0	513.0						Bottom of Boring - 45.0'							

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

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

FILE: 0121-3070-03 | 10/31/2007 2:17 PM |

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

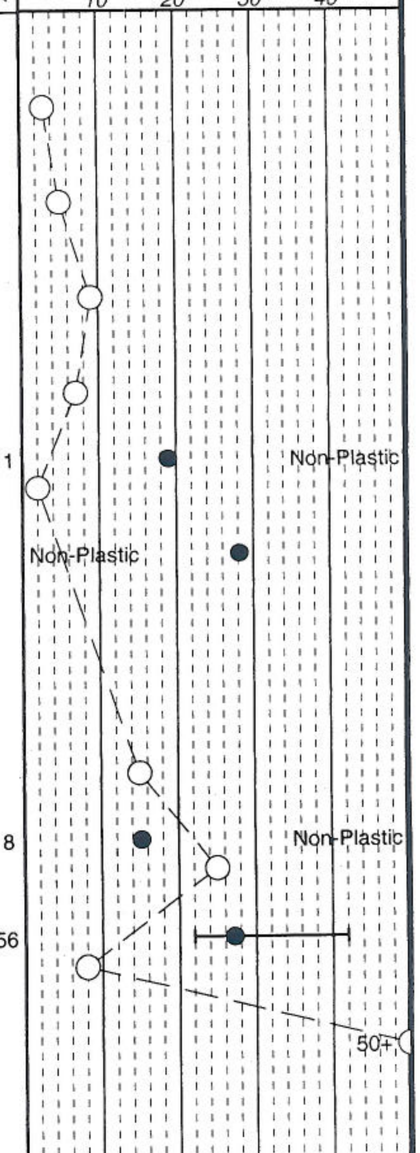
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-51 | Location: Sta. 896+74.9, 4.3 ft. RT of SR 823 CL | Date Drilled: 03/17/05

WATER OBSERVATIONS: Water seepage at: 13.0'-18.0'
Water level at completion: 21.0' (prior to coring)
13.0' (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
	544.5												
0.1	544.4						Topsoil - 2"						
		1 2 1	7	1		2.0	Stiff dark brown SILT AND CLAY (A-6a), little fine to coarse sand, trace fine to coarse gravel; damp to moist.						
		1 2 3	13	2		1.0							
5.5	539.0	2 3 6	8	3		3.5	Very stiff brown SILTY CLAY (A-6b), trace fine to coarse sand, trace fine to coarse gravel; damp.						
8.0	536.5	3 3 4	10	4			Very loose to loose brown GRAVEL WITH SAND (A-1-b), little clay, trace silt; damp.						
		1 1 1	7	5			@ 11.0', moist.	43	28	--	11	7	11
13.0	531.5	WOH	18	6			Very loose brown COARSE AND FINE SAND (A-3a), little gravel, trace clay; wet.	12	28	--	45	15	
		WOH	18	7									
18.0	526.5	16 7 8	18	8			Medium dense reddish brown GRAVEL WITH SAND AND SILT (A-2-4), trace clay; contains sandstone fragments; damp to moist.						
		7 14 11	14	9				19	19	--	30	24	8
23.0	521.5	1 3 5	11	10		1.5	Stiff gray CLAY (A-7-6), trace fine sand; moist.	0	0	--	1	43	56
25.5	519.0	20 50/3	8	11			Severely weathered black SHALE carbonaceous.						
27.5	517.0						Medium hard black SHALE; moderately weathered, pyritic, laminated, broken. @ 28.1'-28.2', gray.						
30													

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



FILE: 0121-3070-03 [10/31/2007 2:17 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-51 | Location: Sta. 896+74.9, 4.3 ft. RT of SR 823 CL | Date Drilled: 03/17/05

WATER OBSERVATIONS: Water seepage at: 13.0'-18.0'
Water level at completion: 21.0' (prior to coring)
13.0' (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	514.5	Core 120"	Rec 116"	RQD 71%	R1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thickly bedded, moderately fractured. @ 28.7'-28.8', pyritic. @ 31.8', very thin clay seam. @ 33.1'-33.3', clay and gravel infilled fracture. @ 34.5', very thin clay infilled fracture. @ 35.5'-36.2', broken zone with clay infilling. @ 36.6'-36.8', highly weathered.							
37.5	507.0						Bottom of Boring - 37.5'							

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

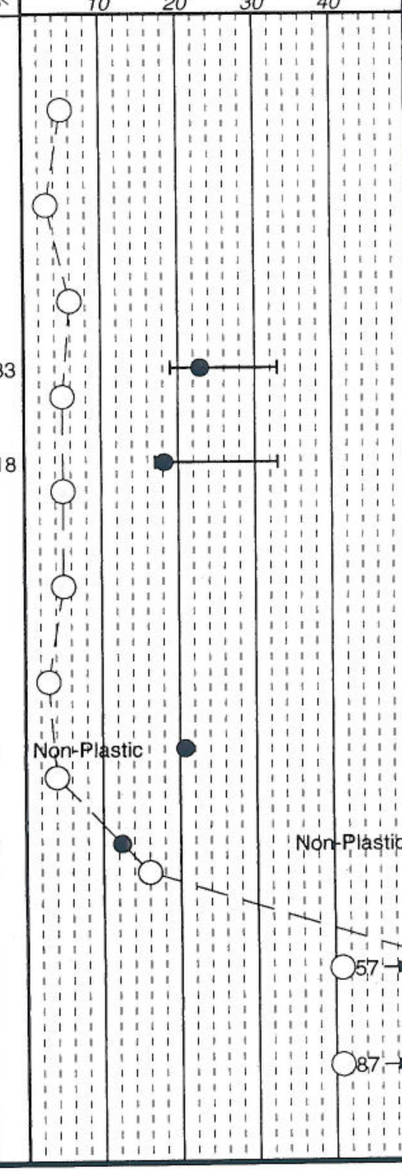
LOG OF: Boring TR-50A | Location: Sta. 898+09.3, 34.1 ft. LT of SR 823 CL | Date Drilled: 3/22/05

WATER OBSERVATIONS: Water seepage at: 18.0'-25.0'
Water level at completion: 18.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	539.3												
0.1	539.2						Topsoil - 1"						
		3 2 3	10	1			FILL: Loose dark brown SANDY SILT (A-4a), trace gravel; contains roots; damp.						
3.0	536.3	2 1 2	8	2		1.0	Stiff brown SILT AND CLAY (A-6a), trace to little fine to coarse sand, trace gravel; moist.						
		2 3 3	13	3		2.0							
		1 2 3	16	4		1.5		1	4	--	6	56	33
10.5	528.8	1 2 3	18	5		1.25	Stiff brown SILTY CLAY (A-6b), some gravel, some fine to coarse sand; moist.	32	20	--	12	18	18
		WOH 2 3	18	6		1.5							
		1 1 2	18	7		1.25							
18.0	521.3	WOH 1 3	16	8			Very loose brown GRAVEL WITH SAND (A-1-b), little to some clay; wet.	16	40	--	23	21	
		2 5 11	16	9			@ 21.0', medium dense.	42	24	--	20	14	
24.5	514.8	2 7 50	18	10			Severely weathered brownish gray SANDSTONE.						
27.5	511.8	25 37 50/4	10	11			Hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, medium bedded, moderately fractured.						

Depth scale: 5, 10, 15, 20, 25, 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-50A | Location: Sta. 898+09.3, 34.1 ft. LT of SR 823 CL | Date Drilled: 3/22/05

WATER OBSERVATIONS: Water seepage at: 18.0'-25.0'
Water level at completion: 18.0' (includes drilling water)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay	STANDARD PENETRATION (N) Natural Moisture Content, % - ● PL ⊢⊣ LL Blows per foot - ○ 10 20 30 40
30	509.3	Core 120"	Rec 117"	RQD 68%	R1		Hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, medium bedded, moderately fractured. @ 28.1',-28.7', 29.0'-29.1', filled fractures. @ 33.3', 34.3'-34.4', 36.2', 37.2', clay-filled fractures.							
35														
37.5	501.8						Bottom of Boring - 37.5'							
40														
45														
50														
55														
60														

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

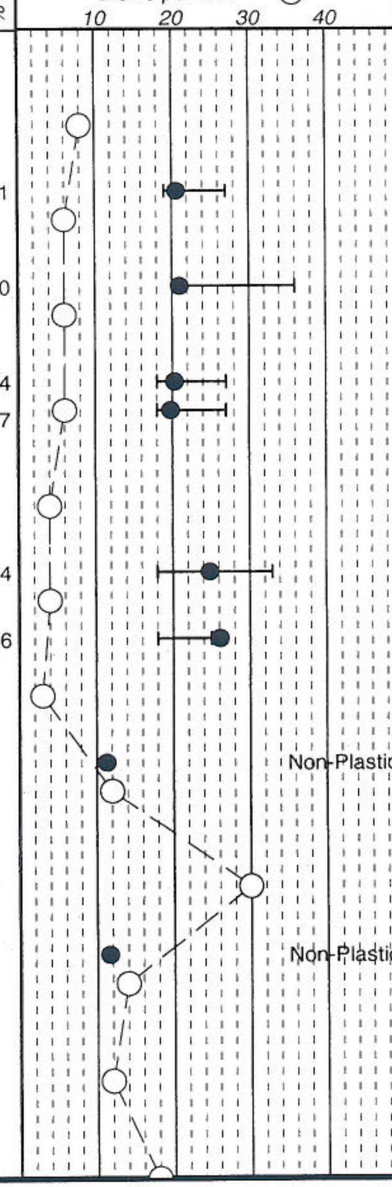
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-54 | Location: Sta. 899+03.6, 36.8 ft. LT of SR 823 CL | Date Drilled: 07/31/07

WATER OBSERVATIONS: Water seepage at: 11.0', 14.5', 16.5', 18.5'
Water level at completion: 20.2' (prior to coring)

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	538.7												
0.4	538.3						Topsoil - 5"						
		3 3 5	15	1		4.0	Very stiff to hard brown SANDY SILT (A-4a), little clay; damp.						
3.5	535.2	2 2 4	17	2		3.0	Very stiff brown SILT (A-4b), some clay; moist.	0	1	--	3	65	31
5.0	533.7	2 3 3	14	3	ST1	1.0	Medium stiff to stiff brown SILT AND CLAY (A-6a), trace to little fine to coarse sand; damp to moist.	0	1	--	9	60	30
8.5	530.2	2 3 3	16	4		1.5	Stiff brown SANDY SILT (A-4a), some clay, little to some fine to coarse sand; trace gravel; moist.	7	10	--	11	48	24
10								9	10	--	9	45	27
		2 2 2	18	5	ST2	1.25							
13.0	525.7	1 2 2	18	6A		1.25	Stiff brown SILT AND CLAY (A-6a), trace fine to coarse sand; moist.	0	1	--	4	61	34
15.0	523.7	WOH 1 2	18	7	ST3	1.0	Medium stiff to stiff brown SILT (A-4b), little fine to coarse sand, trace gravel; wet.	4	5	--	7	58	26
18.0	520.7	6 6 6	14	8			Medium dense brown GRAVEL WITH SAND (A-1-b), trace to little silty clay; moist to wet.	42	36	--	9	13	
20		10 18 12	13	9									
		3 7 7	16	10				41	41	--	10	8	
25													
25.5	513.2	3 4 8	12	11			Medium dense brown COARSE AND FINE SAND (A-3a); wet. @ 27.5', water added to prevent sand heave.						
30		8 9 9	12	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-54 | Location: Sta. 899+03.6, 36.8 ft. LT of SR 823 CL | Date Drilled: 07/31/07

WATER OBSERVATIONS: Water seepage at: 11.0', 14.5', 16.5', 18.5'
Water level at completion: 20.2' (prior to coring)

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	508.7													
31.0	507.7	50/2	2	13			Severely weathered brown SANDSTONE.							50+
32.0	506.7	Core 60"	Rec 57"	RQD 71%	R-1		Medium hard gray SANDSTONE; very fine to fine grained, moderately weathered, argillaceous, micaceous, thinly bedded, moderately fractured, with typical low angle clay-filled fractures. @ 36.8'-37.0', lost recovery, washed out clay.							
		Core 60"	Rec 60"	RQD 70%	R-2		@ 32.0'-33.7', iron stained zone. @ 34.8'-34.9',37.9'-38.1', clay-filled broken zones.							
42.0	496.7						Bottom of Boring - 42.0'							

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 [10/31/2007 2:17 PM]

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

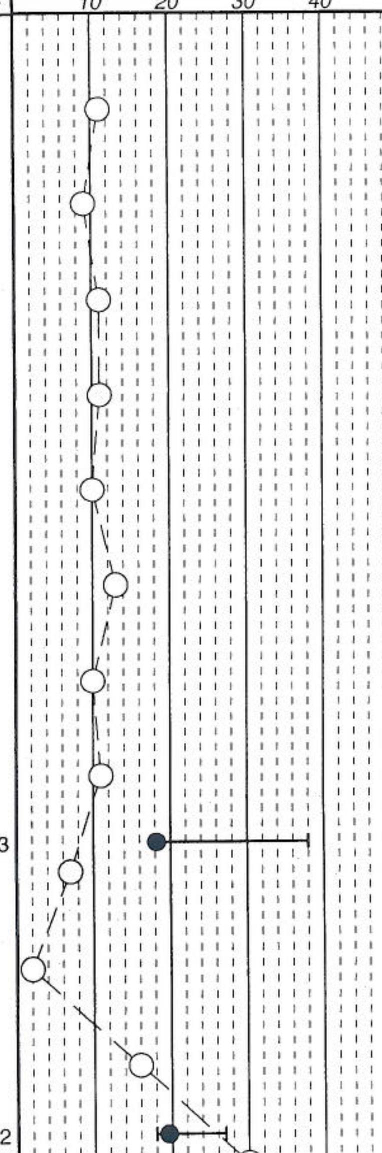
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1141 | Location: Sta. 895+53.5, 34.0 ft. RT of SR 823 CL | Date Drilled: 10/12/05

WATER OBSERVATIONS: Water seepage at: 21.0'-30.0'
Water level at completion: 31.0' (prior to coring)
7.7' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0.2	556.2 / 556.0						Topsoil - 2"						
		5 5 6	18	1			FILL: Loose to medium dense gray SANDY SILT (A-4a), trace clay, trace gravel; contains shale fragments; damp to moist.						
5		4 4 5	18	2									
		2 6 5	6	3									
10		4 8 3	8	4									
		15 7 3	5	5			@ 11.0', contains wood fragments.						
15		4 5 8	3	6									
15.5	540.7	3 5 5	0	7			Loose to medium dense brown and gray GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); moist to wet. @ 16.0'-20.0', contains wood fragments.						
20		5 5 6	3	8			@ 21.0', brown.						
		3 4 3	14	9				57	15	--	7	8	13
25		2 1 1	18	10			@ 23.5'-25.0', very loose.						
		1 1 15	5	11									
28.0	528.2	10 15 15	8	12		1.5	Stiff brown SANDY SILT (A-4a), some clay, little gravel; wet.	11	8	--	27	32	22
30													

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



FILE: 0121-3070-03 [10/31/2007 2:17 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-1141 | Location: Sta. 895+53.5, 34.0 ft. RT of SR 823 CL | Date Drilled: 10/12/05

WATER OBSERVATIONS: Water seepage at: 21.0'-30.0'
Water level at completion: 31.0' (prior to coring)
7.7' (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	526.2						Stiff brown SANDY SILT (A-4a), some clay, little gravel; wet.							
33.0	523.2	Core 120"	Rec 120"	RQD 78%	R1		Medium hard to hard black SHALE; moderately to highly weathered, carbonaceous, thinly laminated to laminated, highly fractured. @ 33.3', 33.4', 33.5', 33.9', 34.7'-34.8', 35.1', 35.3', 35.4', 36.0', 37.2', 37.4'- 37.6', 38.0', 38.3', 38.8', 39.2', 39.3', 39.9', 40.1', 40.8', 41.1', low angle fractures.							
42.6	513.6						Hard gray SANDSTONE; fine grained, slightly weathered, micaceous, medium bedded, unfractured.							
43.0	513.2						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

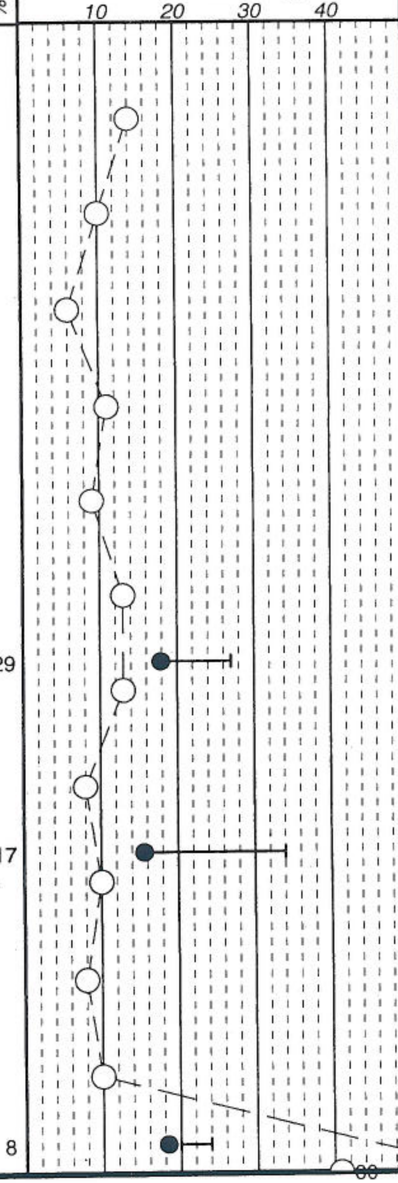
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1142 | Location: Sta. 894+83.0, 33.3 ft. LT of SR 823 CL | Date Drilled: 9/21/05 to 9/22/05

WATER OBSERVATIONS: Water seepage at: 28.0'
Water level at completion: 23.3' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	560.4												
0.3	560.1						Topsoil - 4"						
		5 7 7	5	1		4.0	FILL: Very stiff to hard gray SILTY CLAY (A-6b), trace gravel; damp.						
5		3 5 5	6	2		4.25							
		2 1 5	5	3		2.5	@ 6.0', moist.						
8.0	552.4	11 5 6	6	4		1.0	FILL: Stiff brown SILT AND CLAY (A-6a), little gravel, trace fine to coarse sand; moist.						
10		5 5 4	4	5		1.0	@ 11.0', brown and gray.						
13.0	547.4	5 5 8	18	6		2.0	FILL: Stiff to very stiff black SANDY SILT (A-4a), trace gravel; contains brick fragments and organic material; moist.						
15 / 15.5	544.9	5 7 6	18	7		3.5	Very stiff brown SILT (A-4b), some clay, trace fine to coarse sand, trace gravel; damp to moist.	0	3	--	6	62	29
20		5 4 4	18	8		3.0							
20.5	539.9	4 5 5	8	9			Loose to medium dense brown GRAVEL WITH SAND, SILT, AND CLAY (A-2-6); contains red sandstone fragments; moist.	30	26	--	16	11	17
23.0	537.4	5 4 4	2	10		0.5	Soft brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; wet.						
25 / 25.5	534.9	5 5 5	1	11			Loose to medium dense brown COARSE AND FINE SAND (A-3a), "and" clay, little silt, little gravel; wet.						
29.0	531.4	18 50 10	12	12A 12B			Very dense brown GRAVEL WITH SAND AND SILT (A-2-4),	31	28	--	12	21	8
30													

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



FILE: 0121-3070-03 [10/31/2007 2:17 PM]

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

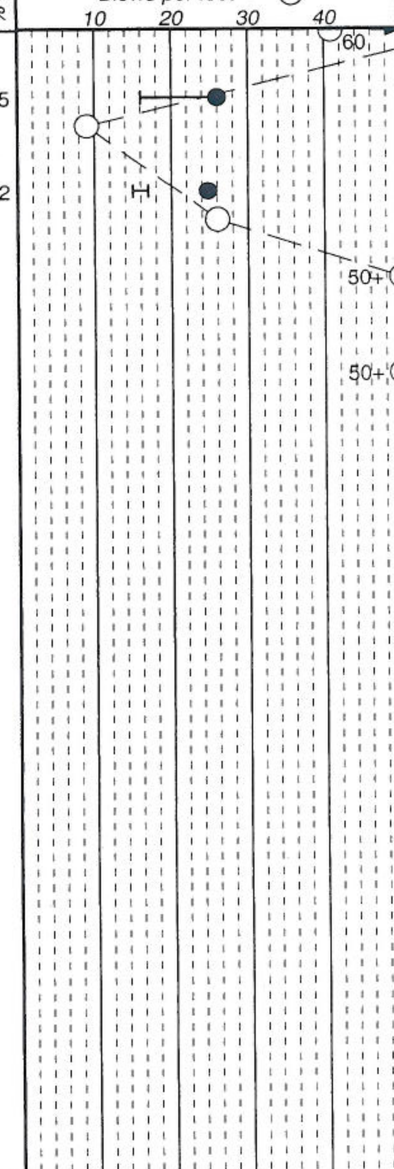
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring B-1142 | Location: Sta. 894+83.0, 33.3 ft. LT of SR 823 CL | Date Drilled: 9/21/05 to 9/22/05

WATER OBSERVATIONS: Water seepage at: 28.0'
Water level at completion: 23.3' (inside hollowstem augers)

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf) / * Point-Load Strength (psi)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
30.0	530.4						trace clay; moist.						
		5 4 5	18	13			Loose to medium dense gray SANDY SILT (A-4a), little clay, trace gravel; wet.	0	1	--	37	47	15
		3 2 24	18	14A 14B				1	1	--	42	44	12
34.5	525.9						Severely weathered black SHALE, carbonaceous.						
		50/4	4	15									
		50/2	2	16									
43.5	516.9						Medium hard black SHALE; moderately to highly weathered, carbonaceous, medium bedded, moderately fractured. @ 43.6'-44.0', high angle fracture.						
45.1	515.3	Core 60"	Rec 60"	RQD 75%	R1		Very hard gray SANDSTONE; very fine to fine grained, moderately to highly weathered, argillaceous, medium bedded, slightly fractured to unfractured. @ 48.0'-48.5', multiple fractures. @ 48.0', moderate siltstone and sandstone laminae.						
48.5	511.9						Bottom of Boring - 48.5'						

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



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

FILE: 0121-3070-03 [10/31/2007 2:17 PM]

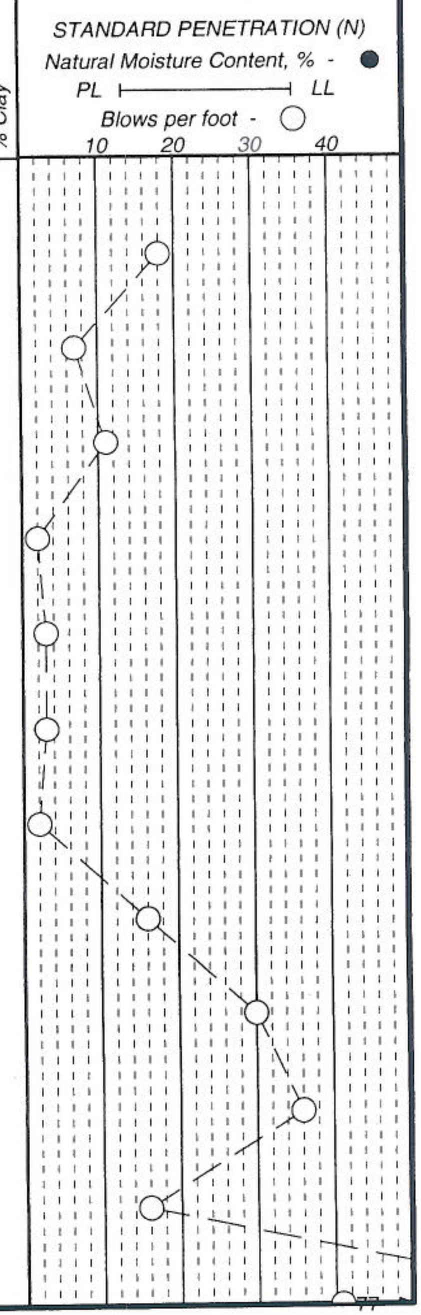
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-49 | Location: Sta. 898+89.3, 21.2 ft RT of SR 823 CL | Date Drilled: 7/7/04

WATER OBSERVATIONS: Water seepage at: 19.0'
Water level at completion: 22.0' (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	539.8												
0.3	539.5						Aggregate - 3"						
		8 9 9	15	1		4.5+	Stiff to very stiff brown SILT AND CLAY (A-6a), little fine to coarse sand, little to some gravel; damp. @ 0.3'-2.5', hard. @ 3.5'-7.5', contains rock fragments.						
5		3 3 4	16	2		1.75							
		3 6 5	15	3		3.0							
10		1 1 1	18	4		1.75							
		1 1 2	15	5		2.25							
13.5	526.3	1 1 2	16	6			Very loose brown SANDY SILT (A-4a), some clay; moist.						
15													
16.0	523.8	1 1 1	18	7		0.5	Soft to medium stiff brown SILT AND CLAY (A-6a), little fine to coarse sand; moist.						
18.5	521.3	6 6 10	13	8			Medium dense to dense brown GRAVEL WITH SAND (A-1-b), some fine to coarse sand, little to some silty clay; wet.						
20		4 9 21	14	9									
25		13 18 18	17	10									
		5 9 7	13	11									
28.5	511.3	4 27 50/3	11	12			Severely weathered gray SANDSTONE fragments.						
30													



FILE: 0121-3070-03 [10/31/2007 2:17 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-49 | Location: Sta. 898+89.3, 21.2 ft RT of SR 823 CL | Date Drilled: 7/7/04

WATER OBSERVATIONS: Water seepage at: 19.0'
Water level at completion: 22.0' (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	509.8						Severely weathered gray SANDSTONE fragments.							
32.0	507.8	Core 120"	Rec 110"	RQD 70%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, massive, slightly fractured. @ 35.4'-35.9', probable core loss. @ 37.6'-37.8',38.5',38.9'- 39.2',39.4'-39.5',40.4'- 40.7', decomposed argillaceous bands with fractures.							
42.0	497.8						Bottom of Boring - 42.0'							

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

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

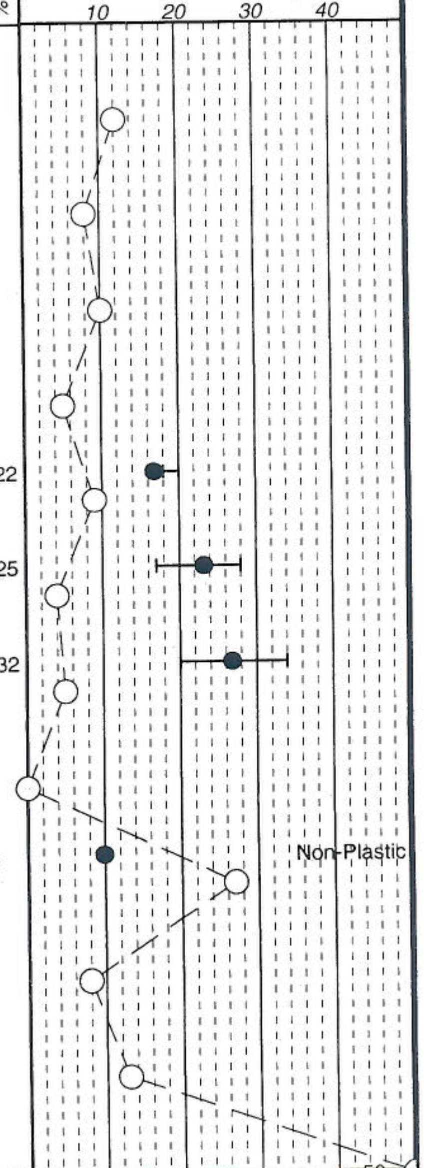
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-49A | Location: Sta. 899+54.2, 10.9 ft. RT of SR 823 CL | Date Drilled: 3/21/05

WATER OBSERVATIONS: Water seepage at: 18.0'-28.0'
Water level at completion: 14.0' (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	538.1						No Topsoil						
		4 5 7	17	1		4.5+	Hard brown SANDY SILT (A-4a), some clay, some gravel; contains sandstone fragments; damp.						
5		3 4 4	16	2		4.0							
		3 5 5	16	3		1.5	@ 6.0', stiff, moist.						
10		2 2 3	17	4		1.0							
		3 4 5	18	5		1.5	@ 11.0', little gravel.	23	16	--	9	30	22
13.0	525.1												
15		WOH 2 2	18	6		<0.25	Very soft brown SILT AND CLAY (A-6a), trace to little fine to coarse sand; wet.	0	5	--	16	54	25
		1 2 3	18	7		<0.25		0	0	--	1	67	32
18.0	520.1												
20		W O H	3	8			Loose to medium dense brown GRAVEL (A-1-a), some to "and" fine to coarse sand, little clay; wet. @ 18.5'-20.0', very loose.						
		5 9 18	12	9				62	16	--	8	14	
25		2 2 6	18	10									
		6 5 8	13	11									
30		10 43 50/4	16	12			@ 28.5'-30.0', very dense.						

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



FILE: 0121-3070-03 [10/31/2007 2:17 PM]

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

LOG OF: Boring TR-49A | Location: Sta. 899+54.2, 10.9 ft. RT of SR 823 CL | Date Drilled: 3/21/05

WATER OBSERVATIONS: Water seepage at: 18.0'-28.0'
Water level at completion: 14.0' (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)
30.0	508.1						Severely weathered gray SANDSTONE							50+
		50/2	1	13										
35.0	503.1						Medium hard to hard gray SANDSTONE; very fine to fine grained, slightly to moderately weathered, argillaceous, micaceous, massive, highly fractured to broken.							
40		Core 120"	Rec 84"	RQD 13%	R1									
45.0	493.1						Bottom of Boring - 45.0'							
50														
55														
60														

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

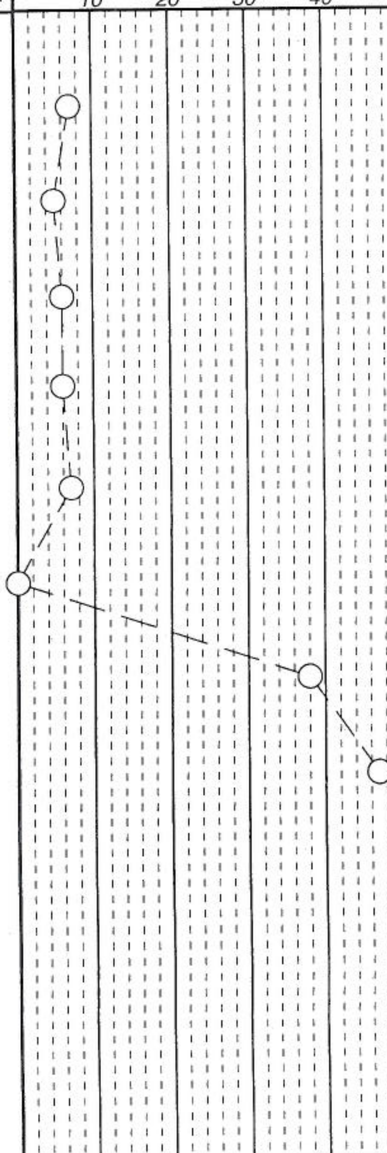
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-50 | Location: Sta. 897+47.1, 23.7 ft. LT of SR 823 CL | Date Drilled: 7/7/04

WATER OBSERVATIONS: Water seepage at: 13.5', 16.0'
Water level at completion: 14.0' (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	540.5												
0.3	540.2						Topsoil - 3"						
		3 3 4	16	1		4.5+	Very stiff to hard brown SILT AND CLAY (A-6a), little fine to coarse sand, trace gravel; damp to moist.						
		1 2 3	14	2		2.0							
		2 3 3	13	3		3.25							
		2 3 3	16	4		2.5							
		2 3 4	13	5		2.75							
		W O H	6	6		1.0	@ 13.5'-17.2', stiff, contains interbedded sand seams.						
		6 7 31	16	7a 7b		1.0 1.25							
17.2	523.3						Severely weathered brown and gray SANDSTONE fragments.						
		10 15 32	4	8									
21.0	519.5	Core 42"	Rec 19"	RQD 0%	R-1		Soft to medium hard gray SANDSTONE; decomposed, argillaceous, broken. @ 24.4'-24.5', dark gray carbonaceous shale.						
25.2	515.3	Core 78"	Rec 70"	RQD 53%	R-2		Hard gray SANDSTONE; very fine to fine grained, micaceous, argillaceous, massive, slightly to highly fractured. @ 25.3'-25.7', 25.8'-25.9', 27.5'-27.8', 30.3'-30.3', high angle fractures. @ 26.1',28.3',29.6', thin clay seams.						
30													

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 TR-50 | Location: Sta. 897+47.1, 23.7 ft. LT of SR 823 CL | Date Drilled: 7/7/04

WATER OBSERVATIONS: Water seepage at: 13.5', 16.0'
Water level at completion: 14.0' (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	510.5	Core 42"	Rec 42"	RQD 100%	R-3		Hard gray SANDSTONE; very fine to fine grained, micaceous, argillaceous, massive, slightly to highly fractured.							
34.5	506.0						Bottom of Boring - 34.5'							

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

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

FILE: 0121-3070-03 [10/31/2007 2:17 PM]

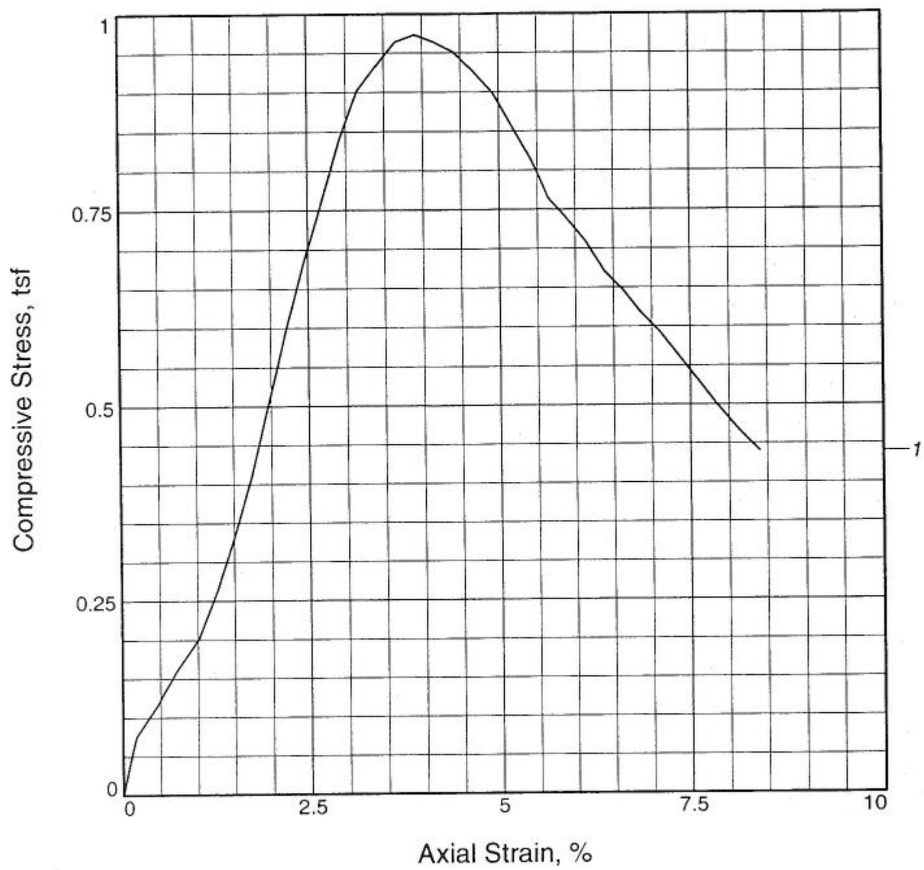
APPENDIX III

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

PROJECT SCI-823-0.00
US 23 Interchange
SUMMARY TEST RESULTS SR 823 Bridge over NS RR and US 23

Boring	Sample	Depth (ft.)	Test Performed	Results											
				ODOT Classification	γ_D (pcf)	WC (%)	e_o	Cc	Cr	p_c (psf)	c (psf)	c' (psf)	φ (deg)	φ' (deg)	q_u (tsf)
B-54	ST1	6.0	UC / Consol.	A-6a	103.9	20.3	0.629	0.170	0.030	2100					0.973
B-54	ST2	10.0	UC	A-6a											0.535
B-54	ST2	10.0	UC	A-6a											1.258
B-54	ST3	15.0	CIU	A-4b	98.3	26.5					336	302	17.2	31.7	

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	0.973			
Undrained shear strength, tsf	0.487			
Failure strain,	3.9			
Strain rate, in./min.	0.06			
Water content, %	21.4			
Wet density, pcf	127.8			
Dry density, pcf	105.3			
Saturation, %	95.5			
Void ratio	0.6065			
Specimen diameter, in.	2.83			
Specimen height, in.	5.52			
Height/diameter ratio	1.95			

Description:

LL = 36	**PL =** 21	**PI =** 15	**Assumed GS=** 2.71	**Type:** 3" press tube

Project No.: 0121-3070.03

Date: 9/17/07

Remarks:

Figure ________

Client: TranSystems, Inc.

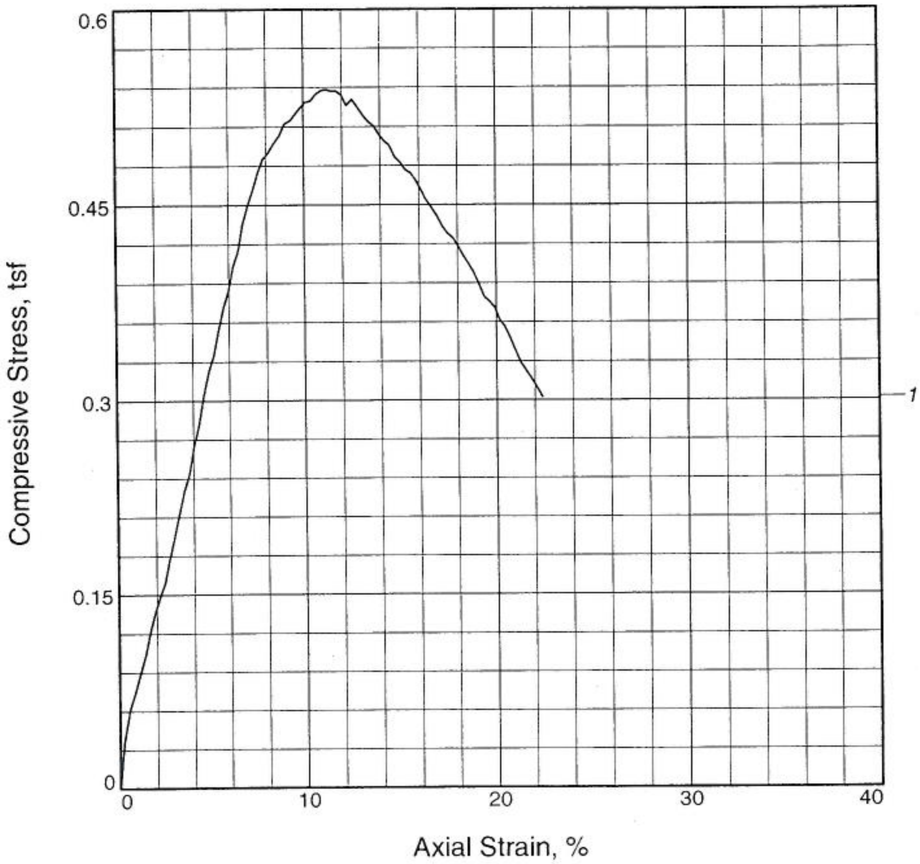
Project: SCI-823-0.00

Source of Sample: B-54 **Depth:** 6.0

Sample Number: ST1

DLZ

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	0.535			
Undrained shear strength, tsf	0.267			
Failure strain,	10.6			
Strain rate, in./min.	0.06			
Water content, %	21.7			
Wet density, pcf	127.6			
Dry density, pcf	104.9			
Saturation, %	93.6			
Void ratio	0.6370			
Specimen diameter, in.	2.84			
Specimen height, in.	4.98			
Height/diameter ratio	1.75			

Description:

LL = 27	**PL =** 18	**PI =** 9	**Assumed GS=** 2.75	**Type:** 3" press tube

Project No.: 0121-3070.03

Date: 9/17/07

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

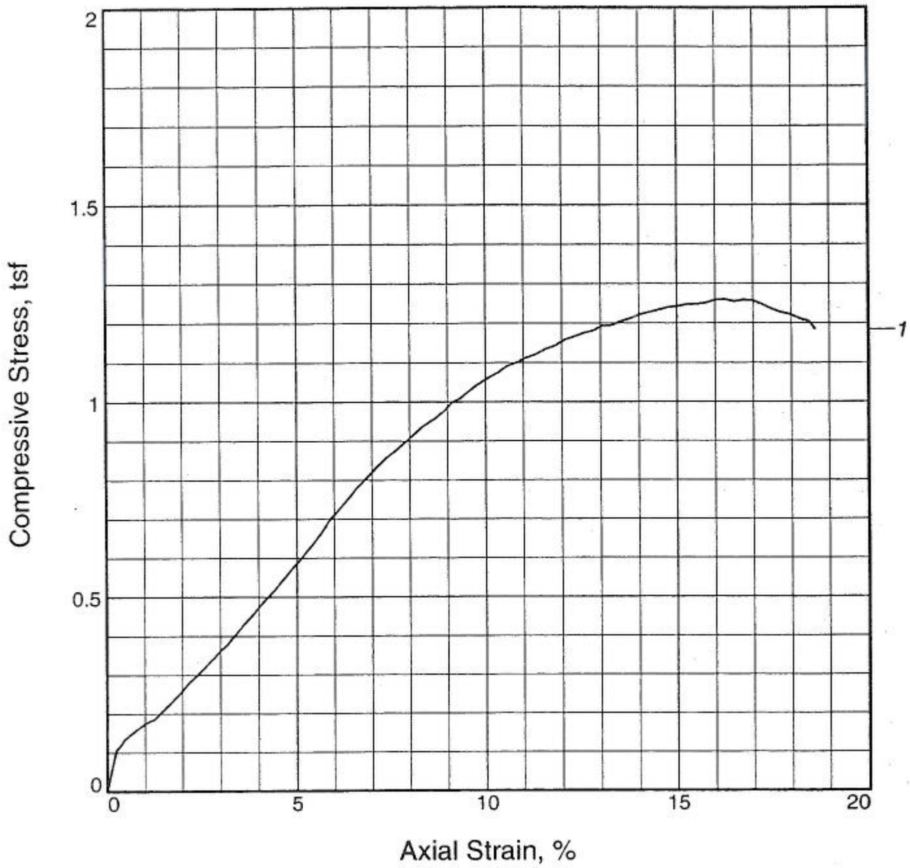
Source of Sample: B-54 **Depth:** 10

Sample Number: ST-2

DLZ

Figure ________

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, tsf	1.258			
Undrained shear strength, tsf	0.629			
Failure strain,	17.0			
Strain rate, in./min.	0.06			
Water content, %	16.4			
Wet density, pcf	134.1			
Dry density, pcf	115.3			
Saturation, %	91.9			
Void ratio	0.4894			
Specimen diameter, in.	2.83			
Specimen height, in.	5.49			
Height/diameter ratio	1.94			

Description:

LL =	PL =	PI =	Assumed GS= 2.75	Type:

Project No.: 0121-3070.03

Date:

Remarks:

Figure _______

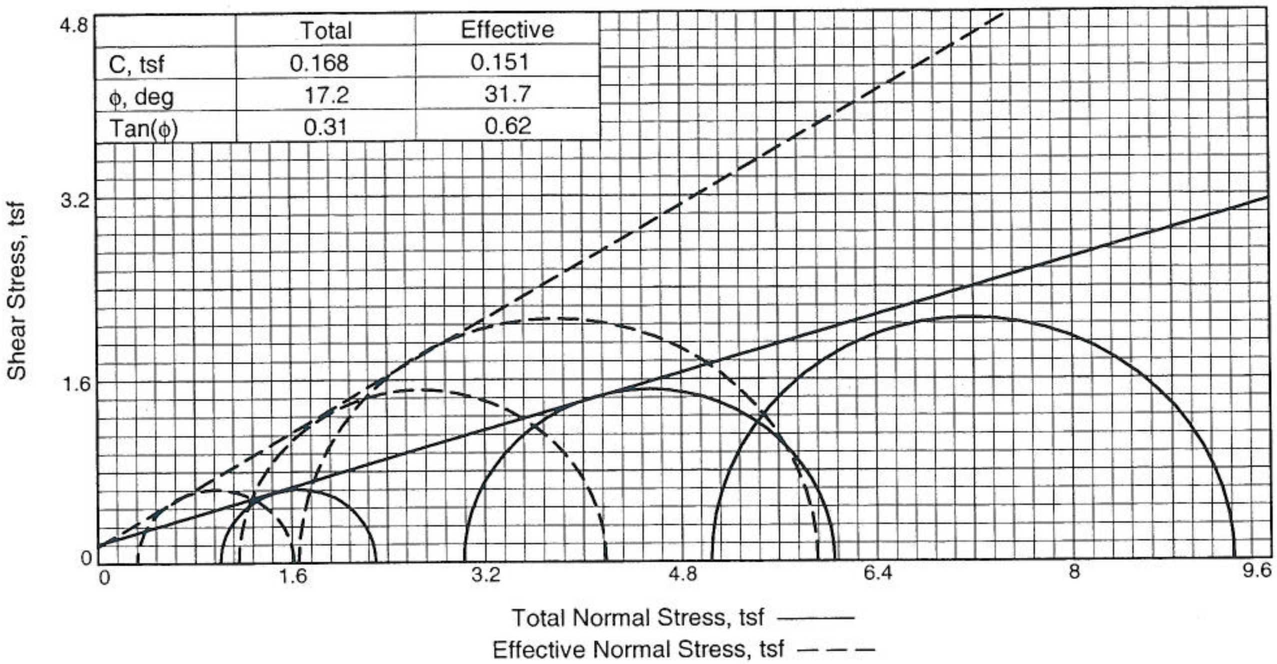
Client: TranSystems, Inc.

Project: SCI-823-0.00

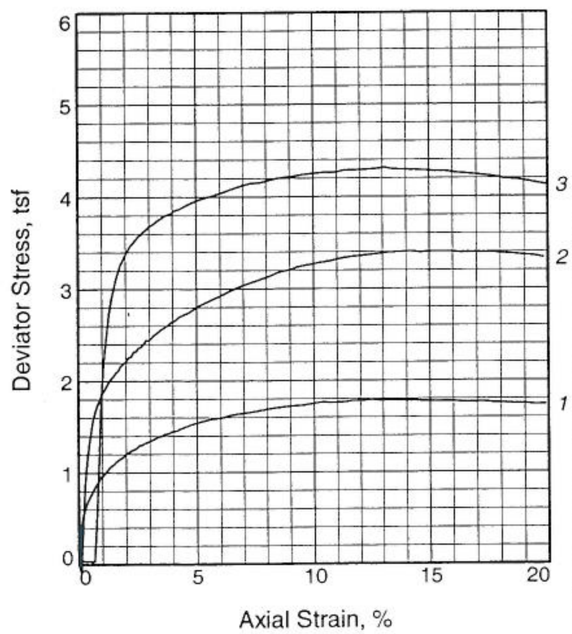
Source of Sample: B-54 **Depth:** 10

Sample Number: ST-2

DLZ



	Total	Effective
C, tsf	0.168	0.151
φ, deg	17.2	31.7
Tan(φ)	0.31	0.62



	Sample No.	1	2	3
Initial	Water Content,	26.5	27.7	28.9
	Dry Density, pcf	98.3	98.1	94.9
	Saturation,	97.8	101.4	98.1
	Void Ratio	0.7457	0.7505	0.8097
	Diameter, in.	2.84	2.83	2.83
	Height, in.	5.46	5.47	5.48
At Test	Water Content,	25.6	22.8	24.5
	Dry Density, pcf	100.8	105.5	102.6
	Saturation,	100.0	100.0	100.0
	Void Ratio	0.7029	0.6274	0.6729
	Diameter, in.	2.80	2.73	2.72
	Height, in.	5.46	5.47	5.48
	Strain rate, in./min.	0.01	0.01	0.01
	Back Pressure, tsf	3.31	3.31	3.31
	Cell Pressure, tsf	4.32	6.34	8.35
	Fail. Stress, tsf	1.29	3.02	4.26
	Total Pore Pr., tsf	4.00	5.18	6.70
	Ult. Stress, tsf	1.32	2.92	4.26
	Total Pore Pr., tsf	3.99	5.21	6.70
	$\bar{\sigma}_1$ Failure, tsf	1.61	4.18	5.91
	$\bar{\sigma}_3$ Failure, tsf	0.32	1.16	1.65

Type of Test:
CU with Pore Pressures

Sample Type: 3" press tube

Description:

LL= 25 **PL=** 18 **PI=** 7

Assumed Specific Gravity= 2.75

Remarks:

Figure ______

Client: TranSystems, Inc.

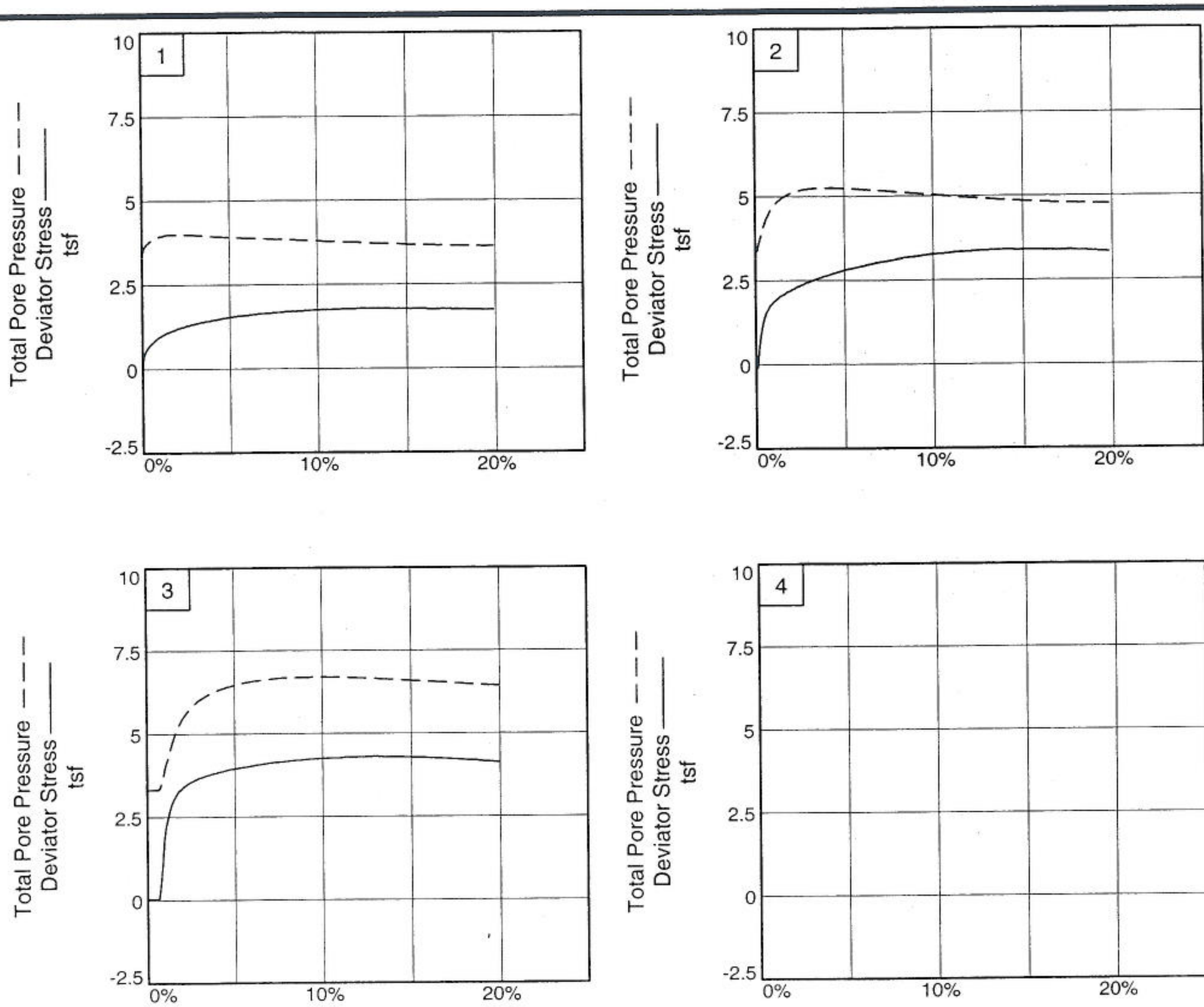
Project: SCI-823-0.00

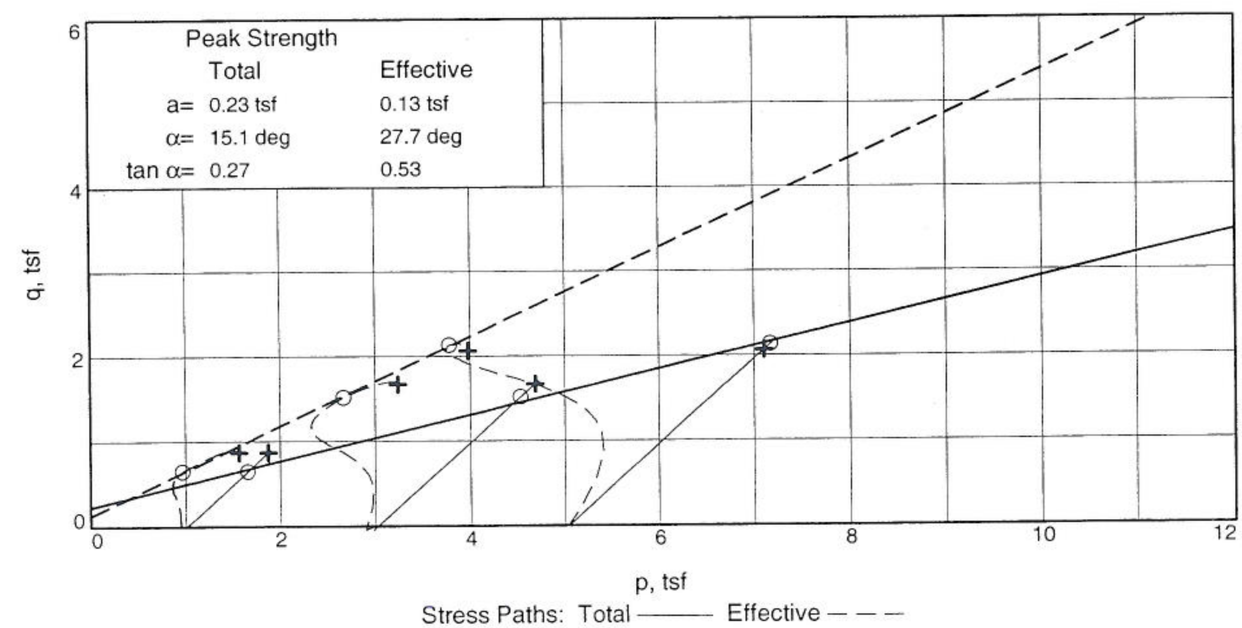
Source of Sample: B-54 **Depth:** 15.0

Sample Number: ST3

Proj. No.: 0121-3070.03 **Date:** 9/18/07

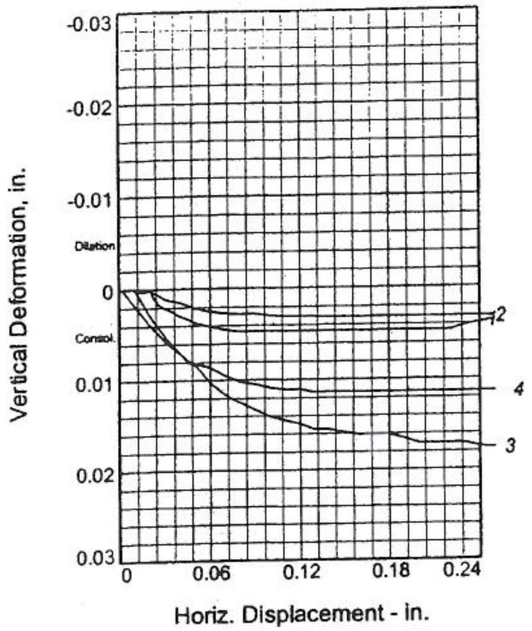
DLZ

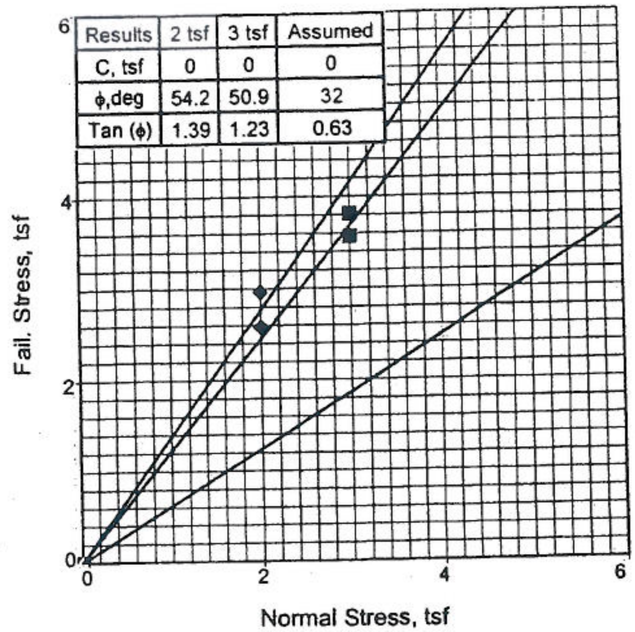




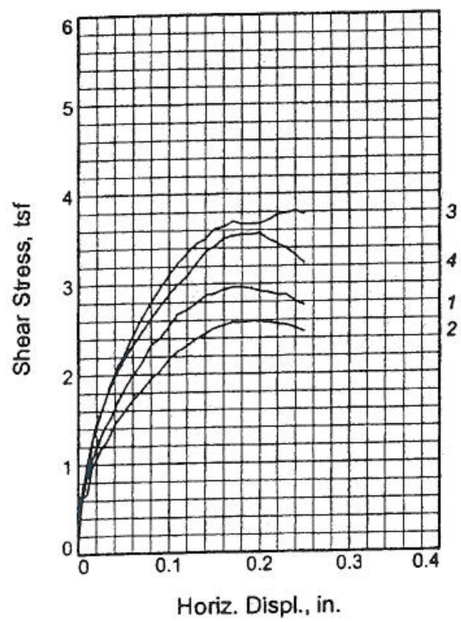
Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-54 **Depth:** 15.0 **Sample Number:** ST3
Project No.: 0121-3070.03 **Figure** ______

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 stired prior to testing, to incorporate excess water.
Figure ________

Client: TranSystems, Inc.

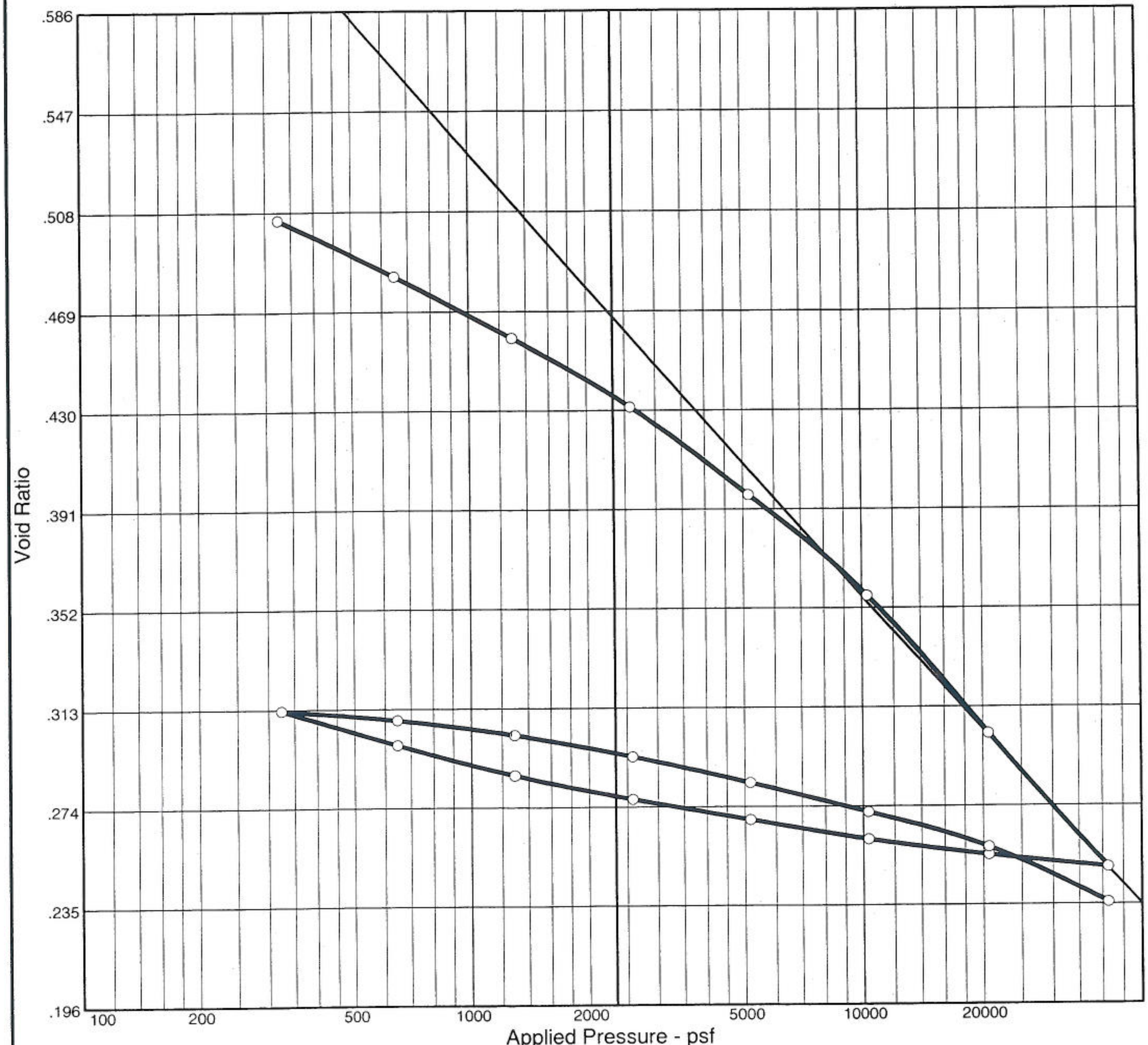
Project: SCI-823-0.00

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							
87.3 %	20.3 %	103.9	36	15	2.71	CL	A-6(14)	0.629

MATERIAL DESCRIPTION

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

Project: SCI-823-0.00

Source: B-54 **Sample No.:** ST1 **Elev./Depth:** 6.0

DLZ

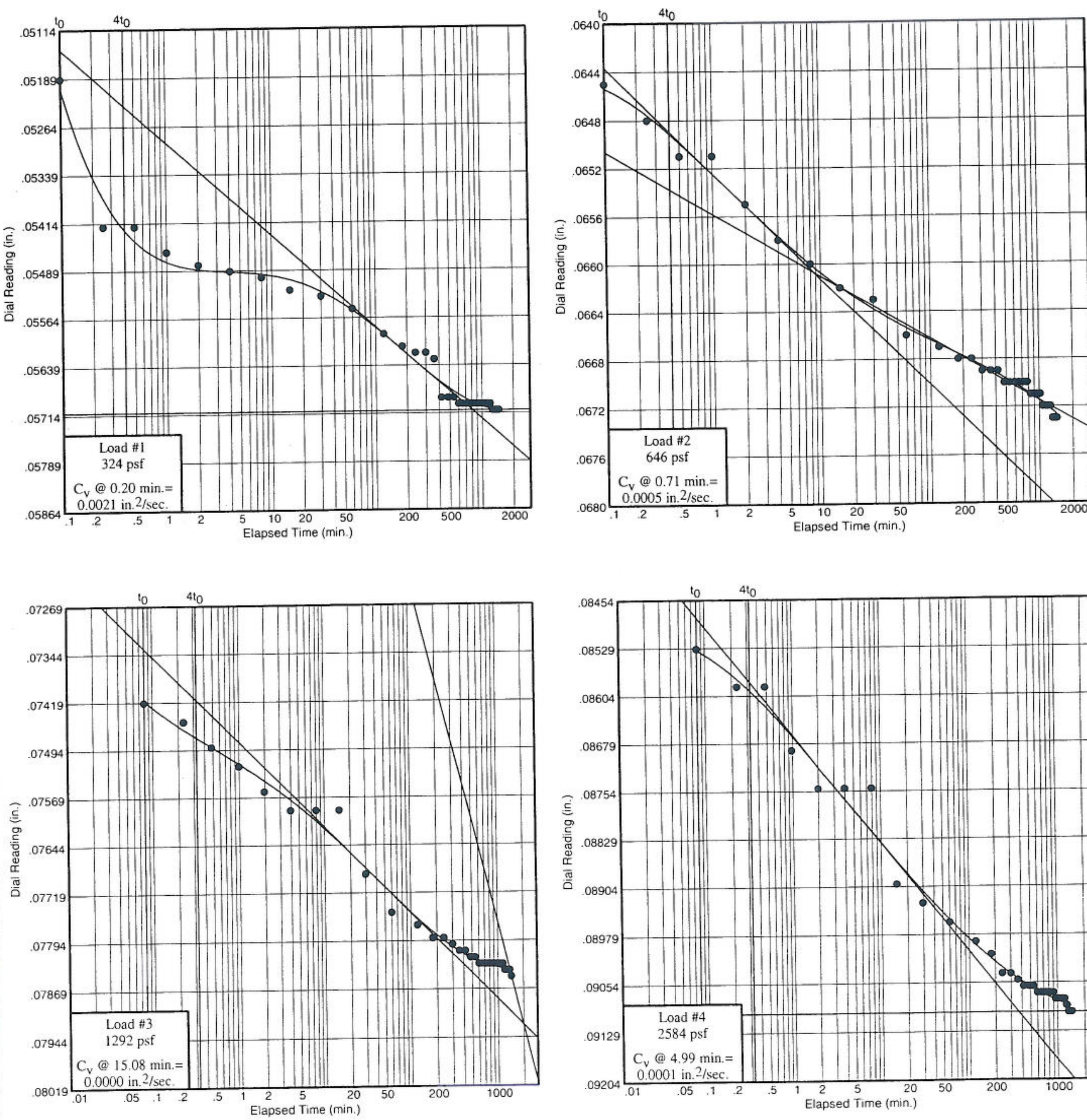
Remarks:

Figure

Dial Reading vs. Time

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

Source: B-54 | Sample No.: ST1 | Elev./Depth: 6.0



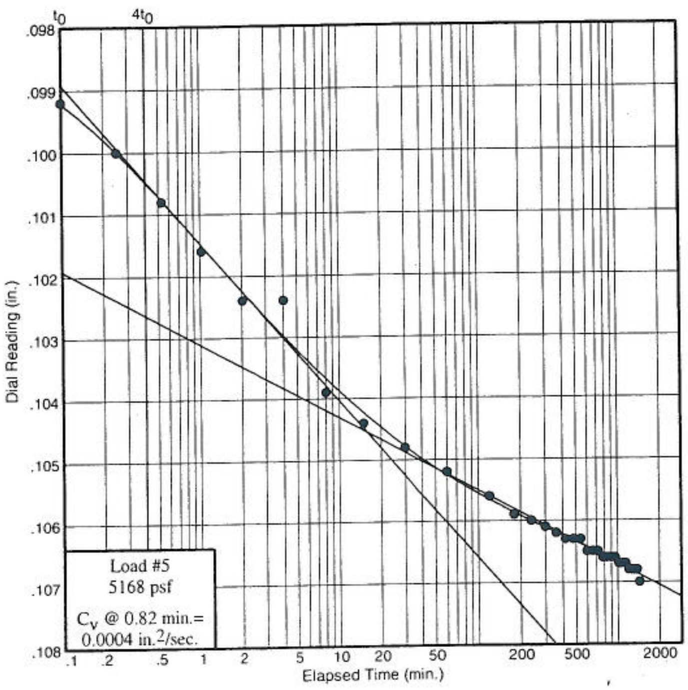
DLZ

Figure

Dial Reading vs. Time

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

Source: B-54 | Sample No.: ST1 | Elev./Depth: 6.0



DLZ

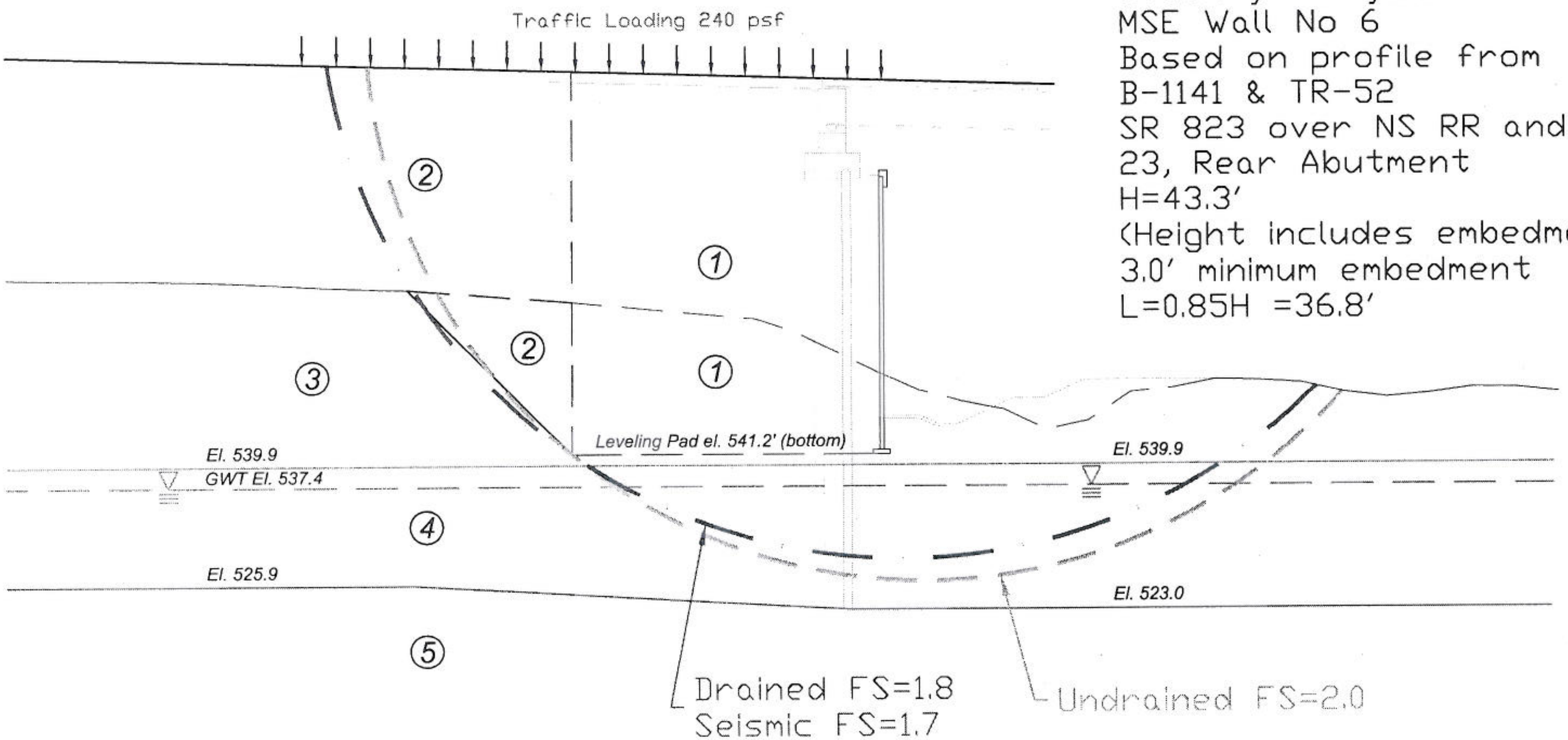
Figure

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	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-6b/A-4a	2000	0	0	29	120
Material 4	Loose-Med D	A-2-6	0	28	0	28	120
Material 5		Bedrock	3500	42	3500	42	140

Stability Analysis
MSE Wall No 6
Based on profile from
B-1141 & TR-52
SR 823 over NS RR and US
23, Rear Abutment
H=43.3'
(Height includes embedment)
3.0' minimum embedment
L=0.85H =36.8'



Stability Analyses performed using UTEXAS3 Version 1.201

SWT 11-5-07
SJK 11-3-07
pg. 1 of 23

US-23 Interchange
MSE Wall No. 6
Station 895+54

GLOBAL STABILITY ANALYSIS

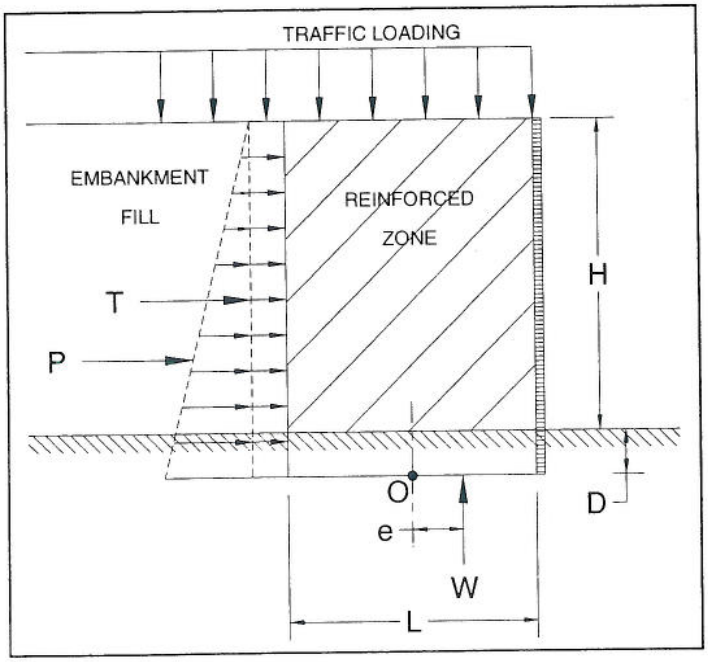
SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 10/30/07

M:\proj\0121\3070.03\Interchanges\US 23\embankment stability 11-1-07.dwg, 11/3/2007 8:57:11 AM, Xerox WorkCentre Pro 245 PS

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

* Shear strength values from A-2-6 layer at el. 537.4'

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	=	0	psf	Cohesion	Foundation soil
ϕ	=	* 28	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	=	36.805	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	=	43.3	ft	
H	=	40.3	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	14.433	ft	Moment arm
Γ Wt	=	21.65	ft	Moment arm
B'	=	30.71	ft	
γ'	=	57.6	pcf	
W_t		8,833	lb/ft of wall	Weight from traffic
W_{mse}	=	191,239	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 6,516 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 17,332 psf

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

q_{ALL} = 6,933 psf

Factor of Safety = **2.66** **OK**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 17,332 psf

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

q_{ALL} = 6,933 psf

Factor of Safety = **2.66** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	25.80	N_c	25.80
N_q	14.72	N_q	14.72
N_γ	16.72	N_γ	16.72

Eccentricity of Resultant Force

e = 3.05 ft

Kern

e < L/6 = 6.13 ft

DLZ	SUBJECT	Client	CH2M Hill	JOB NUMBER	0121-3070.03		
		Project	SCI-823 Portsmouth Bypass	SHEET NO.	3	OF	23
		Item	MSE Wall Stability	COMP. BY	SJK	DATE	11-3-07
		Rear Abutment, MSE Wall No 6		CHECKED BY	GWT	DATE	11-5-07

STABILITY OF MSE WALL

A-2-6 layer at el. 537.4

Assumptions:

1 Estimated height of embankment; H=40.3'
2 Sssume bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglected in resisting forces
5 Use shear strength values of granular layer

Wall Properties

H+D	=	43.3	feet
γ_{mse}	=	120	pcf
L	=	36.805	feet
L factor	=	0.85	
ϕ	=	30	deg

Foundational Soil Properties

c	=	0	psf	Cohesion
ϕ'	=	28	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

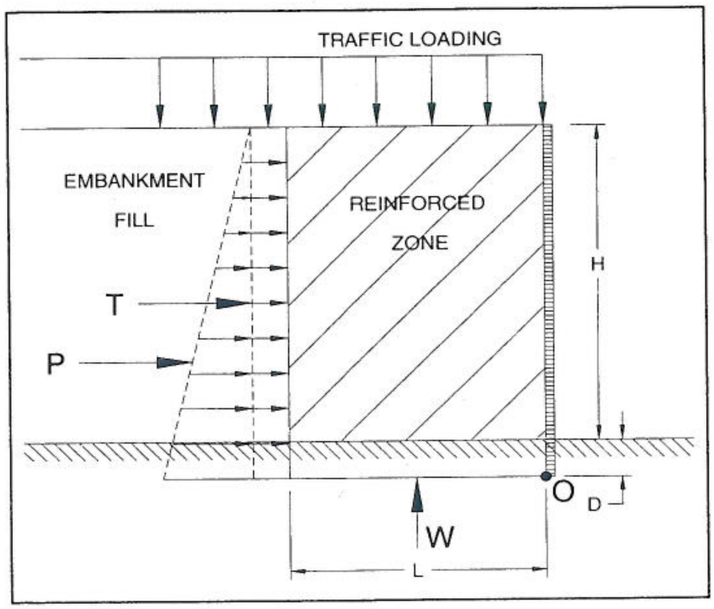
Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 40,552 lbs per foot of wall



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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ $\qquad \mu = 0.36$

P_r = 68,846 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 0 lbs per foot of wall

Use Drained Value

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

Resistance Against Sliding is **OK**

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 3,519,272 lb-ft $\qquad \Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 610,052 lb-ft $\qquad \Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated FS = **5.77** Required FS = 2.00

Resistance Against Overturning is **OK**

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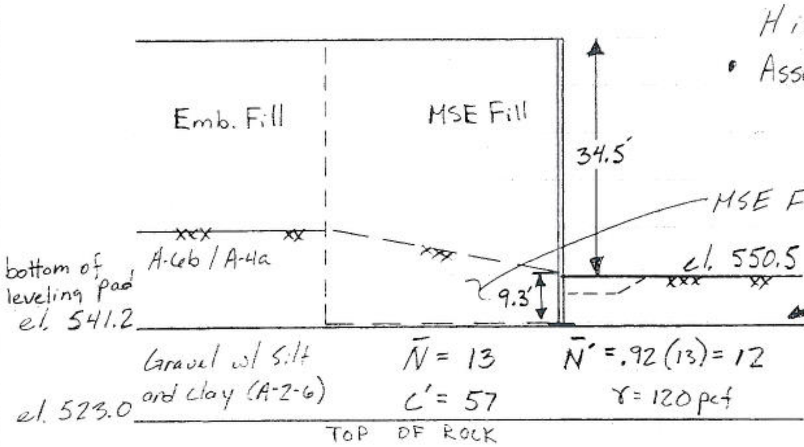
CLIENT	CH2M Hill	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	4 OF 23
SUBJECT	MSE Wall No. 6 - Settlement	COMP. BY	SJK DATE 10-30-0
	SR 823 over NS RR and US 23 - Rear Abutment	CHECKED BY	AWT DATE 11-8-07

Evaluate Settlement at Station 895+48 (Rear Abutment Location)
Profile based upon boring B-1141 and TR-52

Profile at Rear Abutment

- $H = 585 - 550.5 = 34.5'$ (At MSE wall face)
  H is measured above existing ground surface
- Assume groundwater table = el. 537.4'



MSE Fill (Assume Incompressible)

Neglect thin (12") A-6b/A-4a layer beneath Leveling Pad. Assume A-2-6 soil.

Estimated from FHWA-00-045 "Soils and Foundations Workshop"
*See Sample Calculation below.

Sample Calculation

Calculate equivalent parameters from C' for EMBANK inputs.

$\frac{1}{C'} = \frac{C_c}{1+e_o}$ 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 $C' = 57 \rightarrow \frac{2.0}{57} = 0.035$

$\delta_A = 0.1''$ @ toe of embankment / wall
$\delta_E = 1.2''$ @ ℄ of embankment / wall

- Differential Settlement, between pt A and pt B

$DS = \frac{(1.2'' - 0.1'')(1/12'')}{101.5'} = 0.0009$ or $.09\% < 1.0\%$

✓ $\boxed{OK}$

SWT 11-5-07

pg 5 of 23 SJK 11-3-07

SR 823 over NS RR and US 23 - Rear Abutment Settlement at MSE Wall Face

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	: 823RA.emb	Project Manager	: Nix
Date	: 10/30/07	Computed by	: sjr

Settlement for X-Direction

Embank. slope, x direc.	= 69.00 (ft)	Height of fill H	= 34.50 (ft)
y direc.	= 69.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 65.00 (ft)	p load/unit area	= 4140.00 (psf)
Embankment bottom width	= 203.00 (ft)	Foundation Elev.	= 550.50 (ft)
Ground Surface Elev.	= 550.50 (ft)		
Water table Elev.	= 537.40 (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.	9.3	-----	-----	-----	120.00	----	----
2	COMP.	18.2	0.035	0.035	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	9.10	536.65	1615.20	1615.20
3	9.10	527.55	2139.36	2139.36

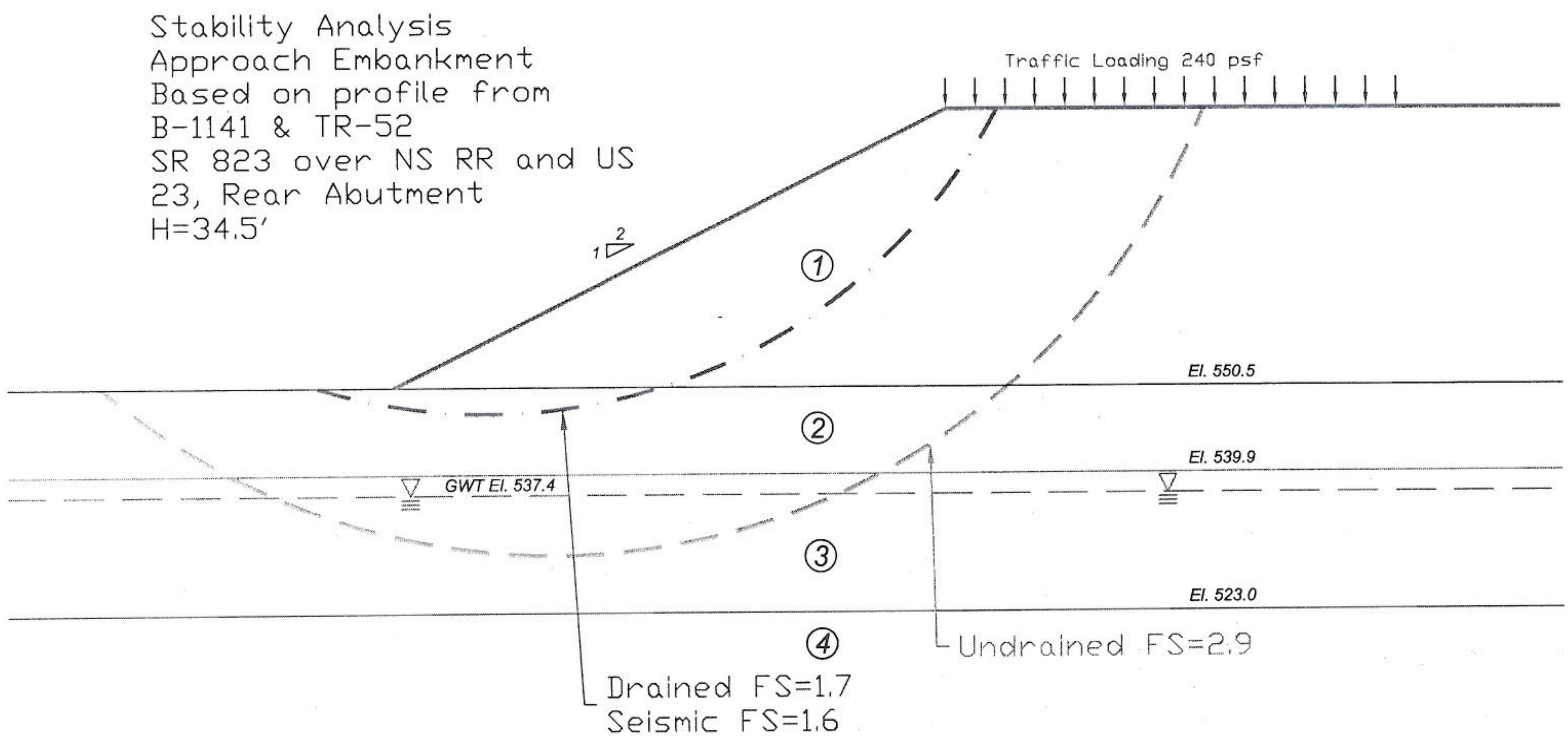
Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 20.30 Stress (psf)	Sett. (in.)	X = 40.60 Stress (psf)	Sett. (in.)	X = 60.90 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	128.26	0.06	625.78	0.27	1220.75	0.47	1787.10	0.62
3	209.53	0.08	650.21	0.22	1204.71	0.37	1714.48	0.49
		------		------		------		------
Pt. A @ toe		0.14		0.49		0.84		1.11

Layer	X = 81.20 Stress (psf)	Sett. (in.)	X = 101.50 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	2045.87	0.68	2066.79	0.68
3	1979.33	0.54	2026.65	0.55
		------		------
		1.22		1.24

Pt. B @ ℄ of embankment

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

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-4a	2000	0	0	29	120
Material 3	Loose-Med D	A-2-6	0	28	0	28	120
Material 4		Bedrock	3500	42	3500	42	140



US-23 Interchange
Approach Embankment
Station 895+17 to 895+54

GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 10/30/07

Stability Analyses performed using UTEXAS3 Version 1.201

EWT 11-5-07
pg 6 of 23 SJK 11-3-07

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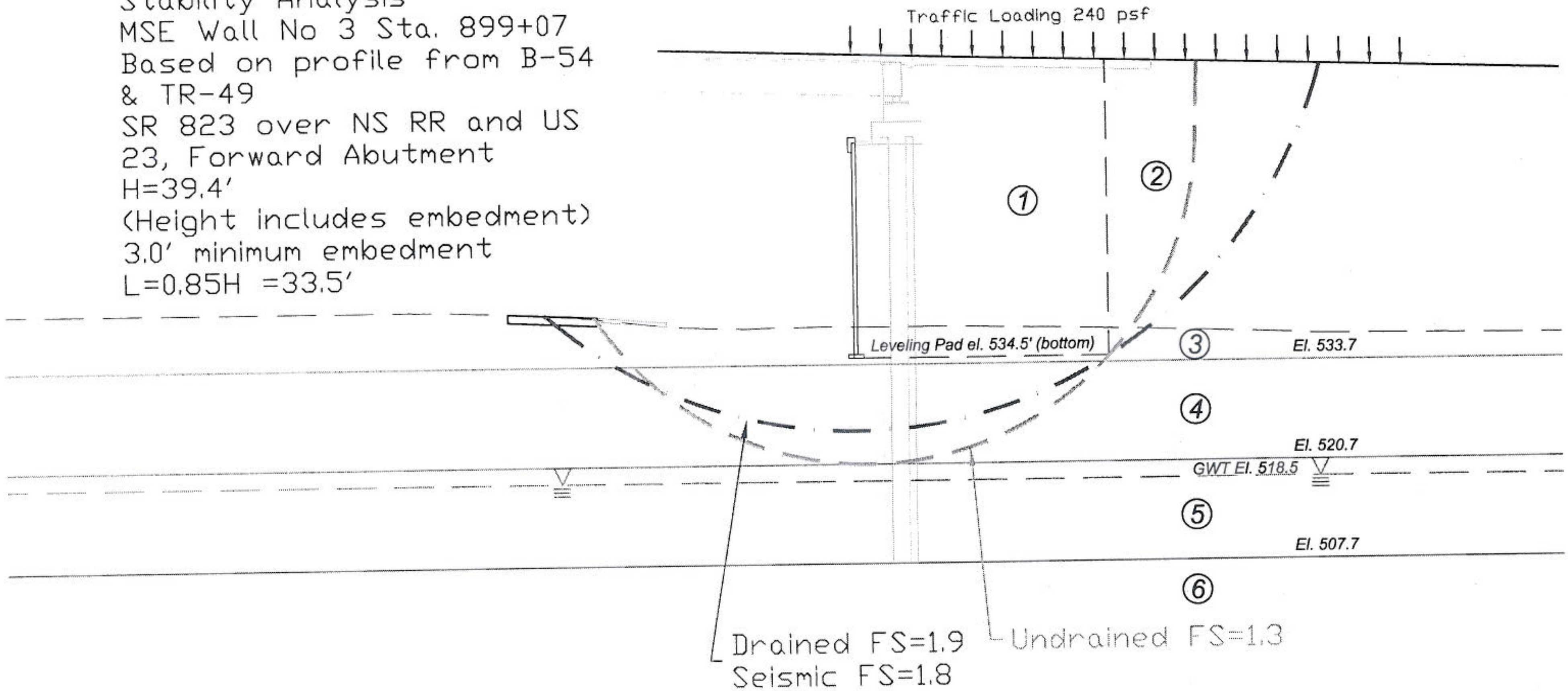
Material	Consistency	Soil Type	Undrained c (psf)	Undrained φ (deg)	Drained c′ (psf)	Drained φ′ (deg)	γ (pcf)
Material 1	Compacted	MSE Fill	0	34	0	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-6a/A-4a	2000	0	0	29	120
Material 4	Med. Stiff	A-6a/A-4a	900	0	0	28	120
Material 5	Med. Dense	Gravel	0	32	0	32	120
Material 6		Bedrock	3500	42	3500	42	140

Stability Analysis
MSE Wall No 3 Sta. 899+07
Based on profile from B-54 & TR-49
SR 823 over NS RR and US 23, Forward Abutment
H=39.4′
(Height includes embedment)
3.0′ minimum embedment
L=0.85H =33.5′



Stability Analyses performed using UTEXAS3 Version 1.201

pg. 7 of 23

EWT on 5-07
SJK 11-3-07

US-23 Interchange MSE Wall No. 3 Station 899+07			
GLOBAL STABILITY ANALYSIS SCI-823-0.00			
PROJECT NO. 0121-3070.03	CALC SJR		DATE 10/30/07

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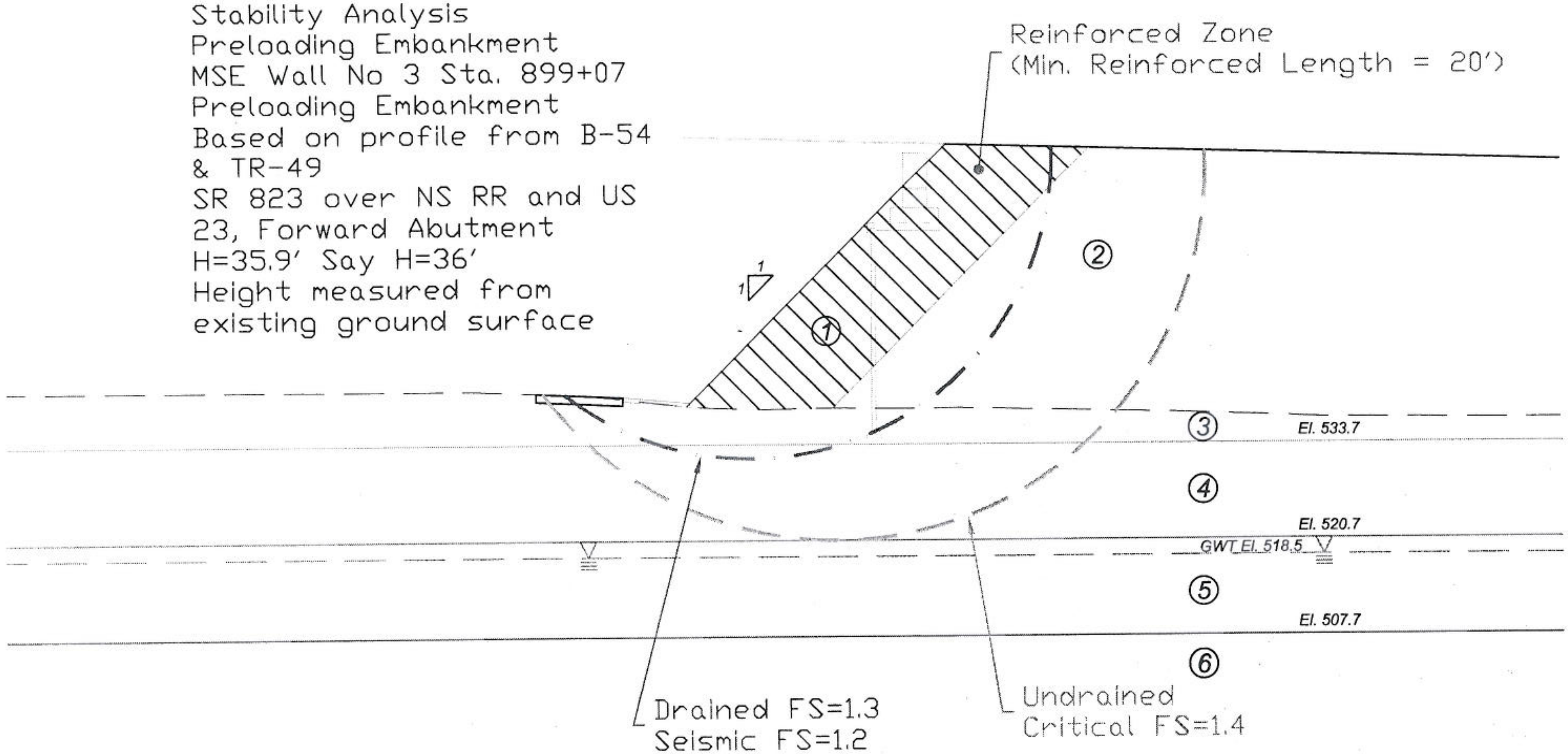
Material	Consistency	Soil Type	Undrained c (psf)	Undrained ϕ (deg)	Drained c′ (psf)	Drained ϕ′ (deg)	γ (pcf)
Material 1	Compacted	Reinforced Fill	2000*	34	2000*	34	120
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	V. Stiff	A-6a/A-4a	2000	0	0	29	120
Material 4	Med. Stiff	A-6a/A-4a	900	0	0	28	120
Material 5	Med. Dense	Gravel	0	32	0	32	120
Material 6		Bedrock	3500	42	3500	42	140

Stability Analysis
Preloading Embankment
MSE Wall No 3 Sta. 899+07
Preloading Embankment
Based on profile from B-54 & TR-49
SR 823 over NS RR and US 23, Forward Abutment
H=35.9′ Say H=36′
Height measured from existing ground surface



Stability Analyses performed using UTEXAS3 Version 1.201

US-23 Interchange
MSE Wall No. 3 (Preloading Embankment)
Station 899+07

GLOBAL STABILITY ANALYSIS

SCI-823-0.00

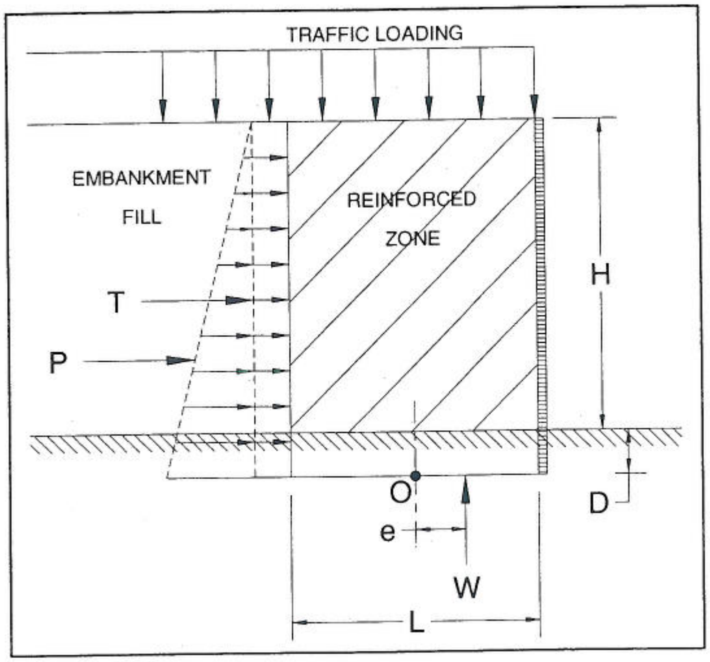
PROJECT NO. 0121-3070.03	CALC: SJR	DATE 10/30/07

11-5-07
SJK 11-3-07

pg 8 of 23

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

Using Initial Undrained Shear Strengths

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	=	33.49	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3.5	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	39.4	ft	
H	=	35.9	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	13.133	ft	Moment arm
Γ Wt	=	19.7	ft	Moment arm
B'	=	27.89	ft	
γ'	=	57.6	pcf	
W_t		8,038	lb/ft of wall	Weight from traffic
W_{mse}	=	158,341	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 5,966 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 4,828 psf

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

q_{ALL} = 1,931 psf

Factor of Safety = **0.81** * **No Good**

See preloading embankment

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 16,398 psf

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

q_{ALL} = 6,559 psf

Factor of Safety = **2.75** **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 = 2.80 ft

Kern

e < L/6 = 5.58 ft

DLZ

SUBJECT			
Client	CH2M Hill	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	10 OF 23
Item	MSE Wall Stability	COMP. BY	SMK DATE 11-3-07
MSE Wall No 3, US 23 Interchange		CHECKED BY	GWT DATE 11-5-07
SR 823 over NS RR and US 23, Forward Abutment			

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=36.4' OR 35.9

2 Sssume bridge is supported on deep foundations

3 Ground water; Dw=0.0'

4 Traffic loading is neglected in resisting forces

5 **Using Initial Undrained Shear Strengths**

Wall Properties			Foundational Soil Properties			
H+D =	39.4	feet	c =	900	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	28	deg	Friction angle
L =	33.49	feet	ω_T =	240	psf	Traffic loading
L factor =	0.85		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	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; $K_a = \tan^2(45 - \frac{\phi}{2})$ $\quad K_a = 0.33$

P_a = 33,857 lbs per foot of wall

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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ $\quad \mu = 0.36$

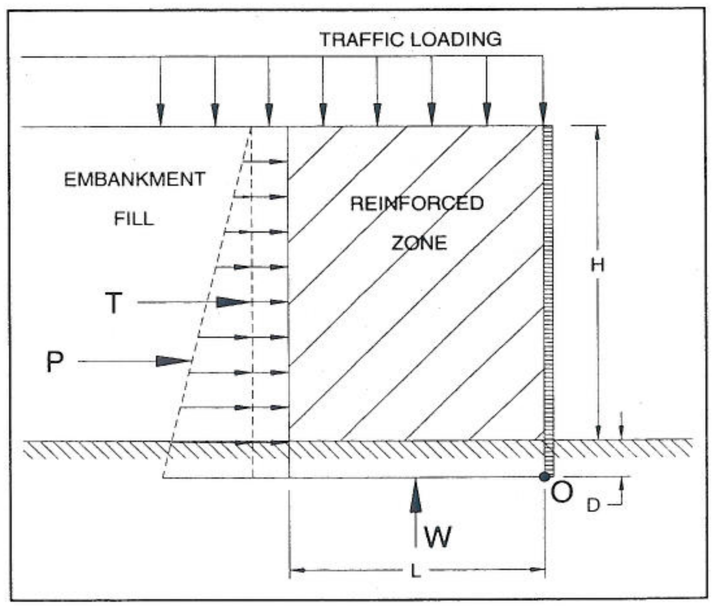
P_r = 57,003 lbs per foot of wall

Use Undrained Value

$$P_r = L(c)$$ (Undrained)

P_r = 30,141 lbs per foot of wall

USE THIS VALUE



$FS = \frac{P_r}{P_a}$ Calculated * FS = **0.89** Required FS = 1.50

*See Preloading Embankment

Resistance Against Sliding is **No Good**

Drained Sliding:
FS = 57,003 / 33,857 = 1.68 OK

RESISTANCE AGAINST OVERTURNING

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

$\Sigma M_{resisting}$ = 2,651,415 lb-ft $\quad$ $\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$

$\Sigma M_{overturning}$ = 465,149 lb-ft $\quad$ $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$$ Calculated FS = **5.70** Required FS = 2.00

Resistance Against Overturning is **OK**

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CLIENT: CH2M Hill
PROJECT: SCI-823 Portsmouth Bypass
SUBJECT: Special Construction
S.R 823 Sta. 899+07, MSE Wall No. 3

PROJECT NO. 0121-3070.03
SHEET NO. 11 OF 23
COMP. BY: [illegible] DATE: 10-30-07
CHECKED BY: SWT DATE: 11-5-07

A

Staged Construction was explored to deal with the low undrained bearing capacity. $q_{all} \simeq 1931$ psf * See page 9 *

Estimate increase in undrained shear strength under consolidating stress of ENTIRE fill height.

$H = 36.4'$, $\gamma = 120$ pcf, $\phi_{cu} = 17.2°$ (from CIU testing, B-54, ST3)

$\Delta\sigma_z = 4368$ psf   Assume $U = 100\%$ for checking purposes only.

$\Delta\sigma_z' = (H \cdot \gamma_{emb}) \cdot U$ ↑ This is a maximum theoretical increase

$C_u = C_{ui} + \Delta\sigma' \cdot \tan(\phi_{cu})$

$C_u = 900 + 4368 \cdot \tan(17.2°) = 900 + 1352 = 2252$ psf

Using $C = 2237$ psf $\longrightarrow$ $q_{all} \simeq 4{,}680$ psf   OR   FS = 2.0
* See page 13 *

∴ An adequate factor of safety against undrained bearing capacity failure cannot be provided by staged construction.

B

1). Construct preloading embankment and allow to consolidate to at least U = 90%.

1) Construct Preloading Embankment Using 1H:1V Slopes.
↳ place toe at station 898+82
↳ H = 36' (above existing ground surface
↳ time to 90% Consolidation = 81 days
$\bar{c} = 9°$

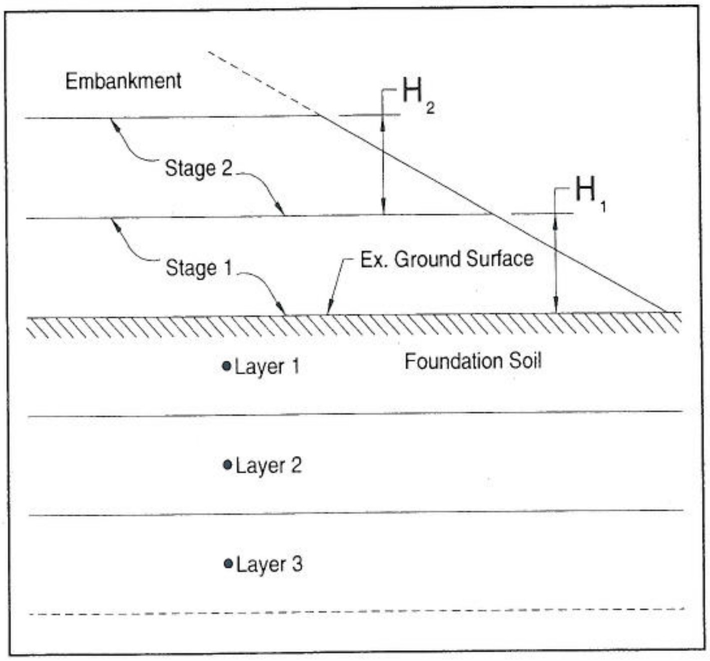
2) Remove preload after at least U = 90%.

3) Construct MSE retaining wall at forward ... (Wall No. 3)
↳ Drained Strengths will govern behavior *
MSE wall meets required minimum factors of safety.
↳ Refer to Drained bearing Capacity ... on pg 9 ...

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

Test to see if staged construction is possible

Use full height as consolidating pressure and calculate BC.

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

Stage 1 Embankment First Stage Embankment Height H_1= **36.0** Average Percent Consolidation U= **100%**

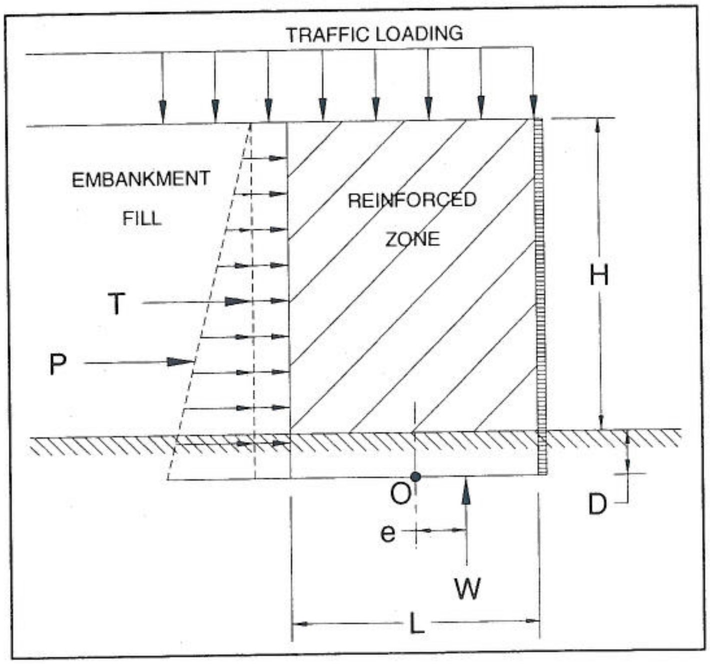
Depth	Soil Type	Initial Undrained Shear Strength, c_{ui} (psf)	$\Delta\sigma'$ (psf)	ϕ_{cu} (deg)	Δc_u (psf)	c_u (psf), *After Consolidation*	Percent Increase
0.8-13.8	A-6a/A-6b	900	4320	17.2	1337	2237	149%
						Not for use in	
						design.	

Stage 2 Embankment Second Stage Embankment Height H_2= **0.0** Average Percent Consolidation U= **0%**

Stage 3 Embankment Third Stage Embankment Height H_3= **0.0** Average Percent Consolidation U= **0%**

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

Test to show staged construction will not work (U=100%)

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	=	2237	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	=	33.49	ft	Length of MSE reinforcement
L factor	=	0.85		Length factor-range (0.7 - 1.0)
D	=	3.5	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	39.4	ft	
H	=	35.9	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	13.133	ft	Moment arm
Γ Wt	=	19.7	ft	Moment arm
B'	=	27.89	ft	
γ'	=	57.6	pcf	
W_t		8,038	lb/ft of wall	Weight from traffic
W_{mse}	=	158,341	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 5,966 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 11,700 psf

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

q_{ALL} = 4,680* psf

Factor of Safety = 1.96* **No Good**

* Value based on theoretical max. strength increase. Not for design.

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 16,398 psf

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

q_{ALL} = 6,559 psf

Factor of Safety = 2.75 **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 = 2.80 ft

Kern

e < L/6 = 5.58 ft

SR 823 over NS RR and US 23 - Forward Abutment Settlement Preloading Embankment

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	: 823FA1.emb	Project Manager	: Nix
Date	: 10/30/07	Computed by	: SJR

Settlement for X-Direction

Embank. slope, x direc.	= 72.00 (ft)	Height of fill H	= 36.00 (ft)
y direc.	= 72.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 65.00 (ft)	p load/unit area	= 4320.00 (psf)
Embankment bottom width	= 209.00 (ft)	Foundation Elev.	= 538.50 (ft)
Ground Surface Elev.	= 538.50 (ft)		
Water table Elev.	= 518.50 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	COMP.	4.8	0.170	0.030	0.000	120.00	2.65	0.62
2	COMP.	13.0	0.170	0.030	0.000	120.00	2.65	0.62
3	COMP.	13.0	0.031	0.031	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	4.80	536.10	288.00	2100.00
2	13.00	527.20	1356.00	2100.00
3	13.00	514.20	2647.68	2100.00

Y= 25.0' @ Proposed MSE Wall location

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 20.90 Stress (psf)	Sett. (in.)	X = 41.80 Stress (psf)	Sett. (in.)	X = 62.70 Stress (psf)	Sett. (in.)
1	18.55	0.03	1278.82	0.78	2542.56	1.70	2998.20	2.10
2	191.61	0.17	1208.77	1.97	2249.34	4.39	2802.53	5.41
3	353.29	0.37	1093.69	0.61	1891.69	0.81	2428.95	0.93
Pt. A		0.57		3.36		6.90		8.43

@ toe of wall location

Layer	X = 83.60 Stress (psf)	Sett. (in.)	X = 104.50 Stress (psf)	Sett. (in.)
1	3002.32	2.10	3002.44	2.10
2	2915.23	5.60	2924.58	5.61
3	2649.87	0.97	2692.34	0.98
		8.67		8.69

Pt. B
@ CL of wall location

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

GWT 11-5-07
SJK 11-3-07

SR 823 over NS RR and US 23 - Forward Abutment Settlement MSE Wall After Preloading

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	: 823FA1.emb	Project Manager	: Nix
Date	: 10/30/07	Computed by	: SJR

Settlement for X-Direction

Embank. slope, x direc.	= 72.80 (ft)	Height of fill H	= 36.00 (ft)
y direc.	= 72.80 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 65.00 (ft)	p load/unit area	= 4368.00 (psf)
Embankment bottom width	= 210.60 (ft)	Foundation Elev.	= 538.50 (ft)
Ground Surface Elev.	= 538.50 (ft)		
Water table Elev.	= 518.50 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	4.0	-----	-----	-----	120.00	----	----
2	COMP.	13.8	0.170	0.030	0.000	120.00	2.65	0.62
3	COMP.	13.0	0.031	0.031	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	13.80	527.60	1308.00	3888.00
3	13.00	514.20	2647.68	3888.00

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 21.00 Stress (psf)	Sett. (in.)	X = 42.00 Stress (psf)	Sett. (in.)	X = 63.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	99.88	0.10	642.45	0.53	1268.32	0.90	1878.42	1.19
3	221.67	0.08	675.07	0.24	1247.48	0.41	1780.69	0.54
Pt. C @ toe		0.18		0.77		1.31		1.73

Layer	X = 84.00 Stress (psf)	Sett. (in.)	X = 105.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.		
2	2175.52	1.30	2190.25	1.31
3	2075.50	0.61	2132.52	0.62
		1.91		1.93 Pt. D @ ℄

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

DLZ
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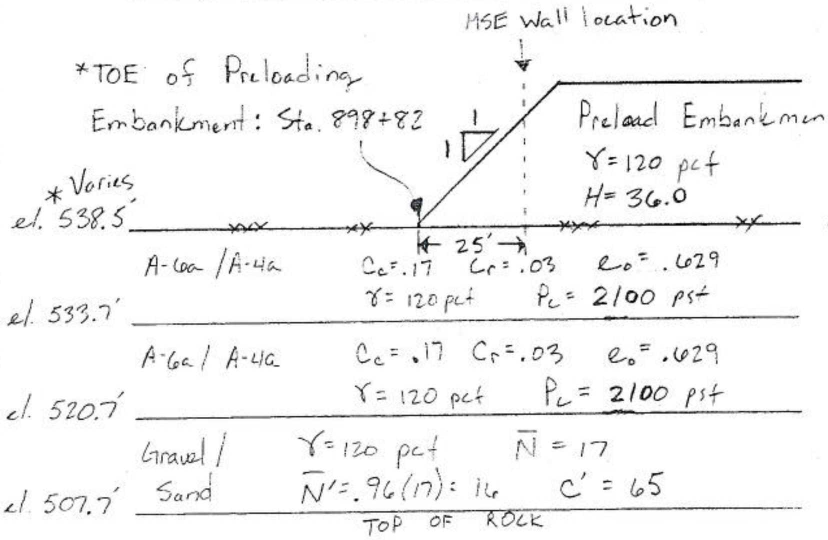
PLANNERS • SURVEYORS

CLIENT	CH2 M Hill	PROJECT NO.	0121 - 3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	16 OF 23
SUBJECT	MSE Wall No. 3 - Settlement	COMP. BY	SAK DATE 10-30-07
	SR 823 over NSRR and US 23 - Forward Abutment	CHECKED BY	GWT DATE 11-5-07

Evaluate Settlement at Station 899 + 13 (Forward Abutment Location)
Profile based upon borings B-54 and TR-49

* Settlement Under Preloading Embankment.



* Assume Groundwater Table at el. 518.5'

Assume Parameters of underlying layer A-6a

Parameters taken from Consolidation Testing B-54, Sample ST1, A-6a Sample

* Use $C_c = C_r$ = .031 and e_o = 1.0 (EMBANK

* See Sample Calculation; pg 4

* Along Proposed Wall No. 3 Location: Preloading Embankment
1) At toe of embankment/wall, δ_A = 0.6"
2) At ℄ of embankment/wall, δ_B = 8.7"

Settlement Under MSE Wall (After Preloading) Wall No. 3

MSE; H = 36.0'
Bottom of MSE wall leveling pad, el. 534.5'
* Use Profile shown above.

* Along Proposed Wall No. 3 Location: MSE wall after preloading
1) At toe of embankment / wall, δ_C = 0.2"
2) At ℄ of embankment / wall, δ_D = 1.9"

* Differential Settlement, between pt C and pt D

$$\Delta S = \frac{(1.9" - 0.2")(1/12")}{105'} = 0.0013 = 0.1\% < 1.0\%$$

✓ OK

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CLIENT CH2M Hill
PROJECT SCI-823 Portsmouth Bypass
SUBJECT MSE Wall No 3 - Time-Rate of Settlement
SR 823 over NS RR and US 23 - Forward Abutment

PROJECT NO. 0121-3070.03
SHEET NO. 17 OF 23
COMP. BY SAK DATE 10-31-0
CHECKED BY SWT DATE 11-5-05

Assume bottom of MSE Wall Leveling Pad el. 534.5

534.5'			
533.7'	A-6a / A-4a	$\overline{LL} \simeq 29\%$	$c_v \simeq 0.5\ ft^2/day$
520.7'	A-6a / A-4a	$\overline{LL} \simeq 29\%$	$c_v \simeq 0.5\ ft^2/day$
507.7'	Gravel / Sand	Assume Free Draining	

C_v estimated from FHWA HI-97-021 "Subsurface Investigations", Fig 9-5 Taken from NAVFAC, DM-7.1, 1982

$$H_r = (534.5' - 520.7')/2 = 6.9'$$ (Assume Double Drainage)

Time to 90% Consolidation

$$t_{90} = \frac{T_v \cdot H_v^2}{C_v} = \frac{(0.848)(6.9')^2}{.50\ ft^2/day} = \underline{81\ days}$$

* To prevent downdrag on the piles, pile should not be driven until 0.4" or less of the consolidation settlement remains.

Total settlement of MSE wall after preloading, $\delta = 1.8"$
Of the 1.8", 1.2" is consolidation settlement.

$$\frac{0.4}{1.2} = 0.33 \qquad U = 1 - 0.33 = .67 \text{ OR } 67\%$$

Say $U = 70\%$ prior to driving piles

$$t_{70} = \frac{(.403)(6.9)^2}{0.5\ ft^2/day} = \underline{38}$$ Waiting period prior to driving piles.

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Time Rate of Consolication of Foundation Soils with Wick Drians
SR 823, Wall No. 3 Station 899+07
Reference: FHWA-RD-86-168

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	**0.50**	**6.9**	**1.2**
5	0.0907	0.0525	0.38	0.24	53.0	0.6				
10	0.1814	0.1050	0.61	0.37	75.6	0.9				
15	0.2721	0.1575	0.76	0.47	87.2	1.0				
20	0.3628	0.2100	0.85	0.54	93.0	1.1				
25	0.4535	0.2625	0.89	0.60	95.8	1.1				
30	0.5442	0.3151	0.92	0.65	97.3	1.2				
35	0.6349	0.3676	0.94	0.68	98.2	1.2				
40	0.7256	0.4201	0.96	0.71	98.9	1.2				
45	0.8163	0.4726	0.98	0.74	99.5	1.2				

Assumes a Triangular Grid Layout

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SJK 11-3-07

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Time Rate of Consolication of Foundation Soils with Wick Drians
SR 823, Wall No. 3 Station 899+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	**0.50**	**6.9**	**1.2**
5	0.0463	0.0525	0.22	0.24	41.1	0.5				
10	0.0926	0.1050	0.39	0.37	61.2	0.7				
15	0.1388	0.1575	0.52	0.47	74.2	0.9				
20	0.1851	0.2100	0.62	0.54	82.7	1.0				
25	0.2314	0.2625	0.70	0.60	88.2	1.1				
30	0.2777	0.3151	0.77	0.65	91.8	1.1				
35	0.3239	0.3676	0.82	0.68	94.2	1.1				
40	0.3702	0.4201	0.85	0.71	95.7	1.1				
45	0.4165	0.4726	0.88	0.74	96.8	1.2				
50	0.4628	0.5251	0.90	0.76	97.6	1.2				
55	0.5090	0.5776	0.91	0.79	98.2	1.2				
60	0.5553	0.6301	0.93	0.81	98.6	1.2				

Assumes a Triangular Grid Layout

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Time Rate of Consolication of Foundation Soils with Wick Drians
SR 823, Wall No. 3 Station 899+07
Reference: FHWA-RD-86-168

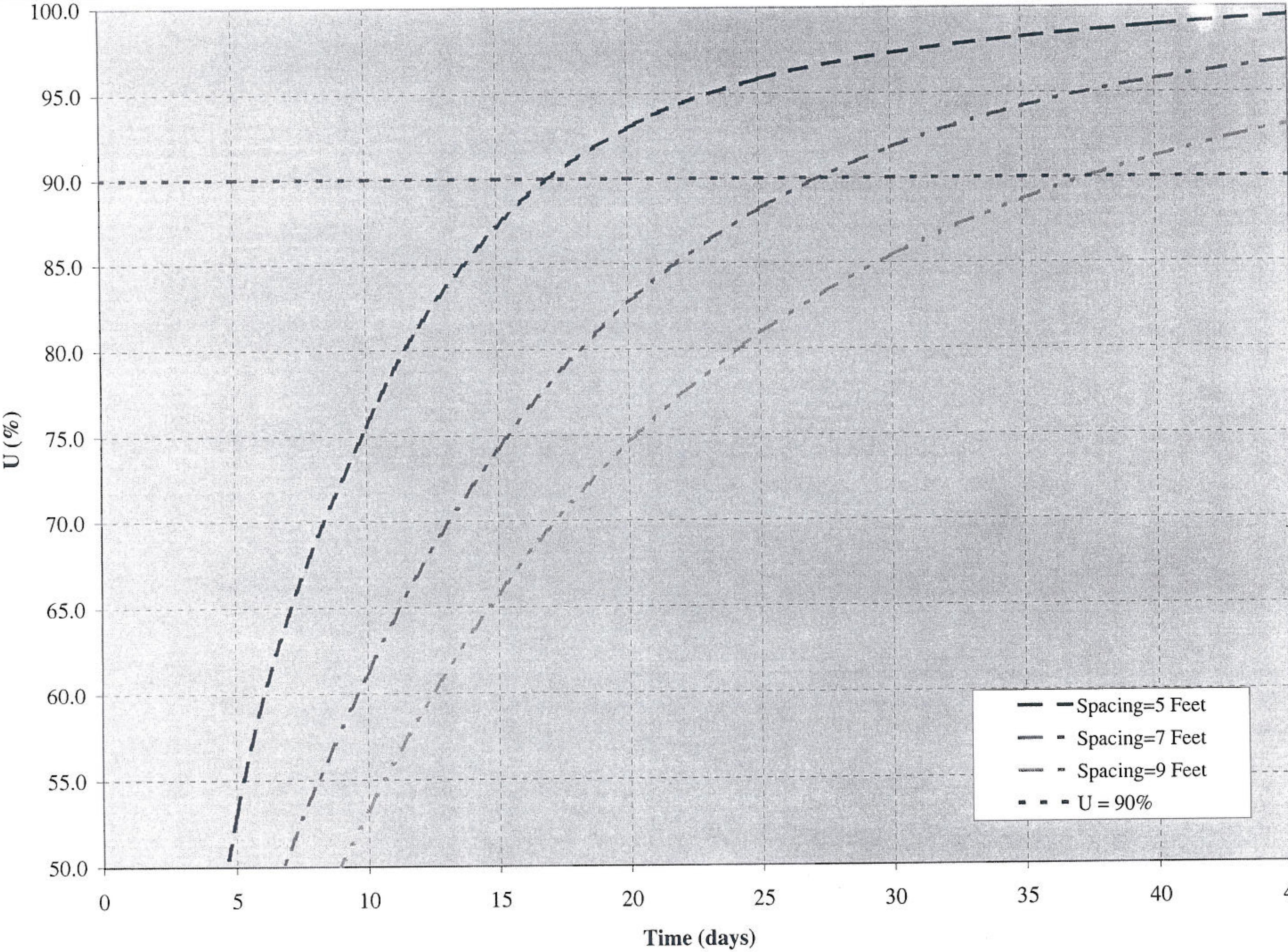
Wick Drain Spacing **9.0** feet Use $\eta = 10$

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	**0.50**	**6.9**	**1.2**
5	0.0280	0.0525	0.15	0.24	35.5	0.4				
10	0.0560	0.1050	0.26	0.37	53.2	0.6				
15	0.0840	0.1575	0.36	0.47	65.7	0.8				
20	0.1120	0.2100	0.44	0.54	74.6	0.9				
25	0.1400	0.2625	0.52	0.60	80.9	1.0				
30	0.1680	0.3151	0.59	0.65	85.4	1.0				
35	0.1960	0.3676	0.64	0.68	88.7	1.1				
40	0.2240	0.4201	0.69	0.71	91.2	1.1				
45	0.2520	0.4726	0.73	0.74	93.1	1.1				
50	0.2799	0.5251	0.77	0.76	94.6	1.1				
55	0.3079	0.5776	0.80	0.79	95.8	1.1				
60	0.3359	0.6301	0.83	0.81	96.7	1.2				
65	0.3639	0.6826	0.85	0.83	97.5	1.2				

Assumes a Triangular Grid Layout

DLZ

Percent Consolidation vs Time Using Prefabricated Vertical "Wick" Drains
US-23 Interchange, SR 823 over NS RR andUS 23, (Wall No 3)



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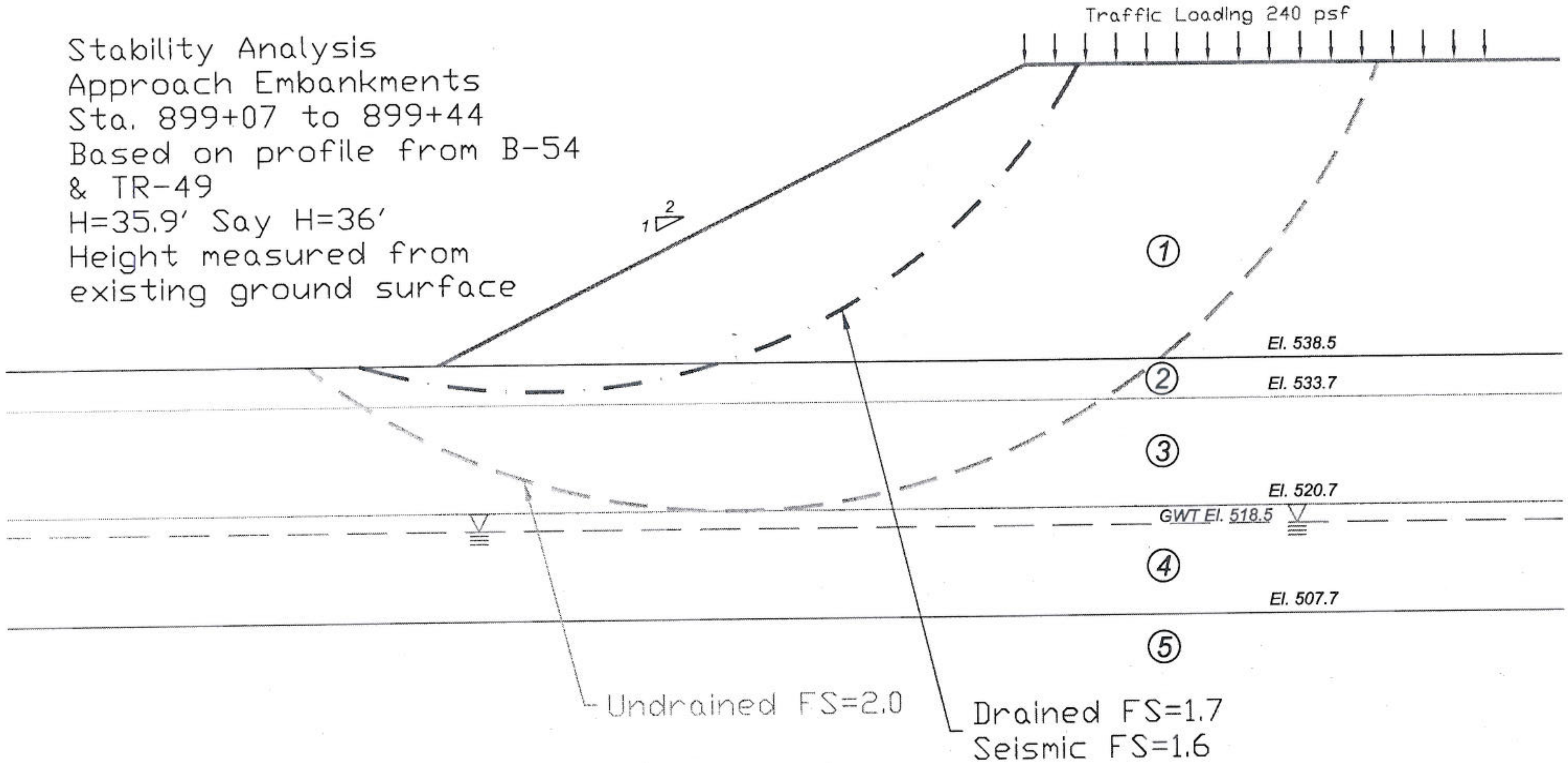
SWST 11-5-07

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-6a/A-4a	2000	0	0	29	120
Material 3	Med. Stiff	A-6a/A-4a	900	0	0	28	120
Material 4	Med. Dense	Gravel	0	32	0	32	120
Material 5		Bedrock	3500	42	3500	42	140

Stability Analysis
Approach Embankments
Sta. 899+07 to 899+44
Based on profile from B-54
& TR-49
H=35.9' Say H=36'
Height measured from
existing ground surface



US-23 Interchange
Approach Embankment
Station 899+07 to 899+44

GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03	CALC: SJR	DATE 10/30/07

Stability Analyses performed using UTEXAS3 Version 1.201

EWT 11-5-07
SJK 11-3-07

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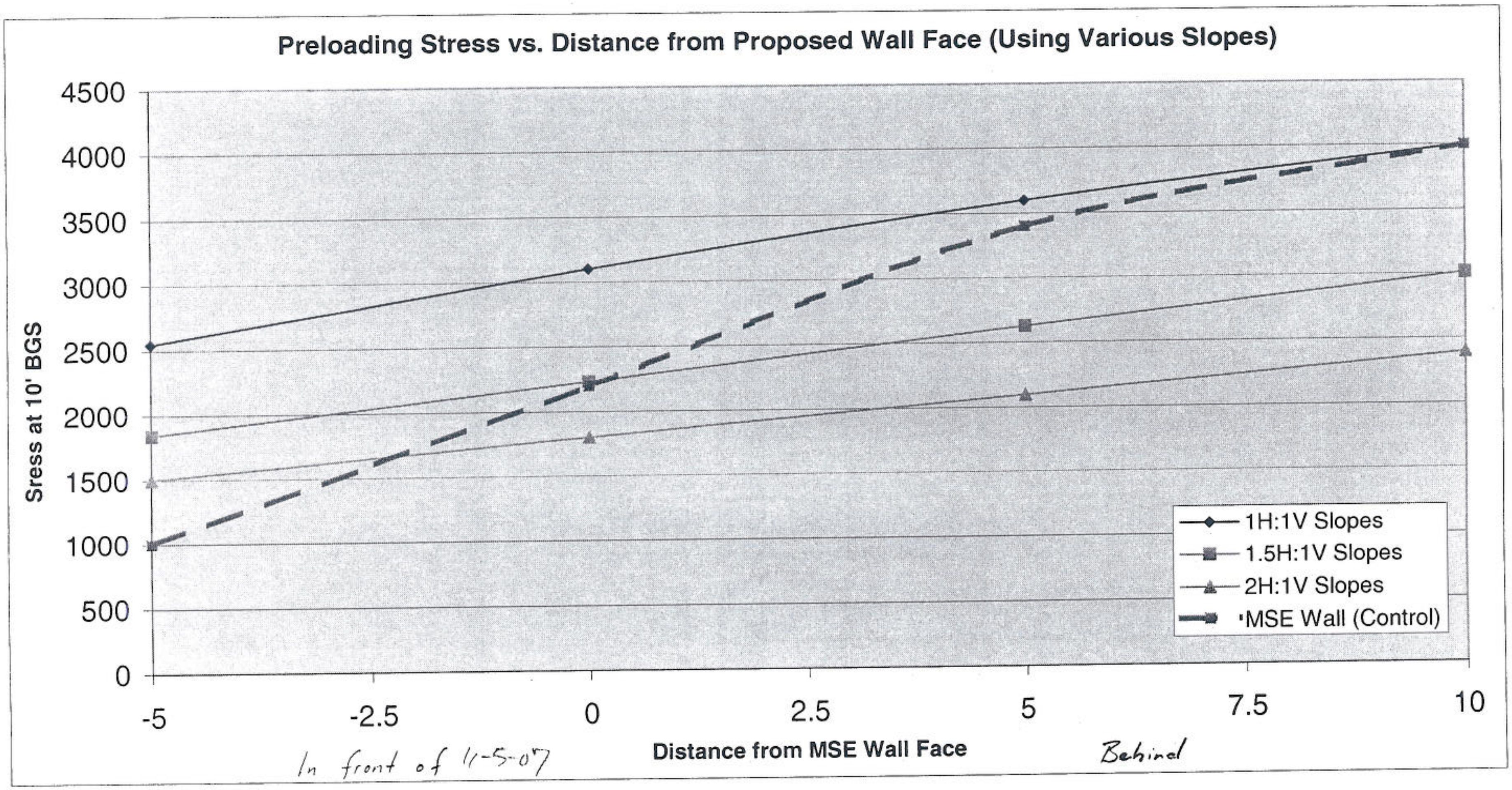
M:\proj\0121\3070.03\Interchanges\US 23\embankment stability 11-1-07.dwg, 11/3/2007 9:26:10 AM, Xerox WorkCentre Pro 245 PS

Preloading Embankments **MSE Wall No 3**

Stress Measured at Specified Locations at a Depth of 10' Below the Ground Surface

	Locations Relative to MSE Wall Location			
Slopes	**5' Front**	**Wall Face**	**5' Behind**	**10' Behind**
1H:1V slopes	2545	3100	3600	4000
1.5H:1V slopes	1840	2231	2625	3010
2H:1V slopes	1500	1800	2095	2395
MSE Wall (control)	1000	2200	3400	4000
	-5	0	5	10



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APPENDIX D

SCI-823-10.13

SR 823 OVER NORFOLK SOUTHERN RAILWAY AND US-23 / RAMP D

VERTICAL CLEARANCES

Filename: \\aries\proj\TranSystems\319861\19415\structures\Documents\Step 8 - Preliminary Design Report\Bridge Preliminary Design Reports\Bridge SCI823-1601C 823 over Railroad_US23\Report\[Vertical_Clearance.xls]Vertical Clearance

By: DGS Revised by: SCJ Date: 4/10/2007 Revised: 7/16/2007
Checked: JTC Date: 4/19/2007

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

52" Steel Plate Girder

PROFILE DATA - NORFOLK SOUTHERN RAILWAY

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	East	n/a	550.44
2	Top of Rail	West	n/a	548.96

PROFILE DATA - RAMP D

POINT	RAMP D LOCATION	EXISTING ELEV. @ US 23 EDGE OF PVMT.	DISTANCE ACROSS TAPER	PAVEMENT X-SLOPE	DISTANCE ACROSS SHLDR.	SHOULDER X-SLOPE	RAMP D - FINISHED GRADE @ POINT
3	RT. EDGE OF PVMT	540.91	10.12	-1.6%		-4.0%	540.75
4	RT. EDGE OF SHLDR.	540.91	10.18	-1.6%	8.00	-4.0%	540.43

PROFILE DATA - SR 823 MAINLINE

Linear: PVT Sta. 870+00.00 PVC Sta. 904+10.82
PVT Elev. 661.63 PVC Elev. 559.31
g -3.00%

Bridge Deck Cross Slope

Location	Pavement	
Center Shoulder	4.0%	7.5 Feet - Centerline Construction to Edge of Shoulder
Rdwy/Outside Shld.	-1.6%	22.25 Feet - Edge of Shoulder to Fascia Girder

Drop of Cross Slope from Centerline Construction to Fascia Girder: -0.0560

POINT	SR 823 MAINLINE LOCATION DESCRIPTION	STA.	OFF.*	SR 823 PG ELEV.	PAVEMENT X-SLOPE DROP	SR 823 - FINISHED GRADE @ POINT
1	RT. FASCIA GIRDER	896+15.98	22.25	583.15	-0.0160	582.80
2	RT. FASCIA GIRDER	896+60.03	22.25	581.83	-0.0160	581.48
3	RT. FASCIA GIRDER	898+86.11	22.25	575.05	-0.0160	574.69
4	RT. FASCIA GIRDER	898+94.86	22.25	574.79	-0.0160	574.43

* - Offset from Profile Grade Line

STRUCTURE DEPTH

Haunch + Max. Top Flange = 4.0 in

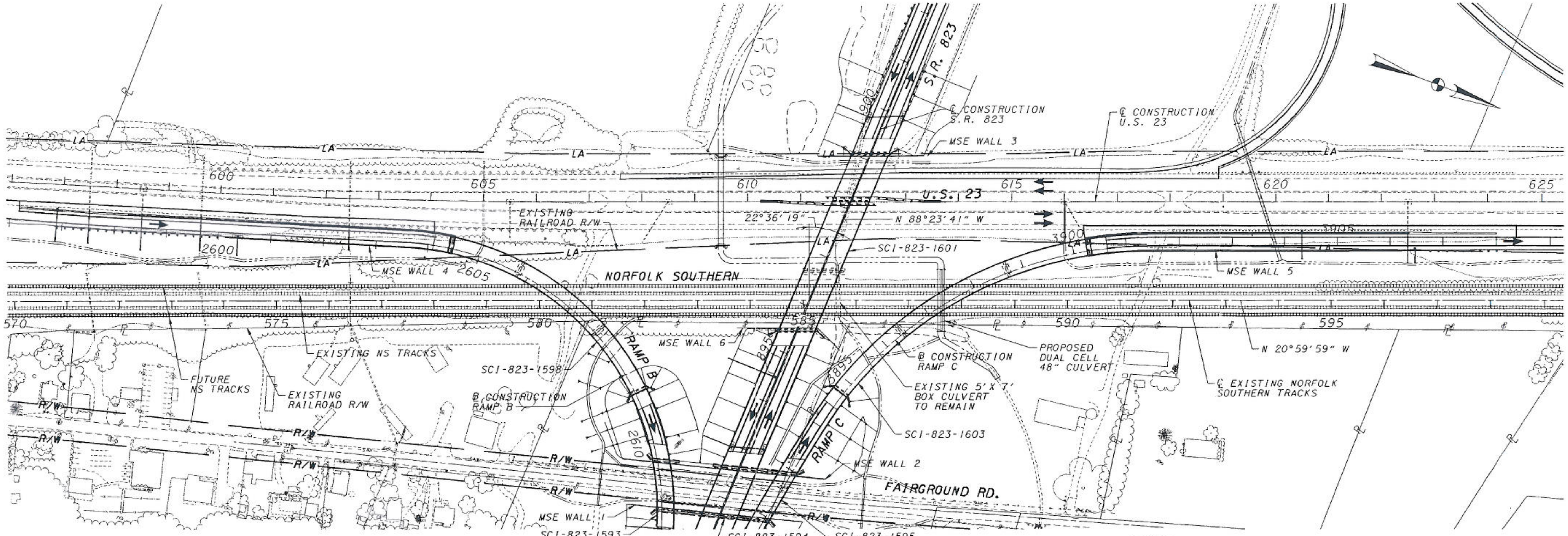
POINT	BEAM DESCRIPTION	Slab	Haunch	Top Flange	Web	Bot. Flange	Splice	Total	
1	Plate Girder (52" Web)	8.50	2.00	2.0	52	2.0	-	66.50	in
2	Plate Girder (52" Web)	8.50	2.00	2.0	52	2.0	2.0	68.50	in
3	Plate Girder (52" Web)	8.50	2.00	2.0	52	2.0	-	66.50	in
4	Plate Girder (52" Web)	8.50	2.00	2.0	52	2.0	-	66.50	in

VERTICAL CLEARANCE - SR 823 OVER NORFOLK SOUTHERN RAILWAY AND US-23 / RAMP D

POINT	LOCATION	SR 823 MAINLINE - FINISHED GRADE @ POINT	STRUCTURE DEPTH (in.)	BOT. GIRDER ELEVATION	RR / RAMP D - FINISHED GRADE @ POINT	VERTICAL CLEARANCE (ft.)		CHECK - MINIMUM REQUIRED VERT. CLEARANCE*
1	RT. FASCIA GIRDER	582.80	66.500	577.26	550.44	**26.82**	OK	23'-5 1/8"
2	RT. FASCIA GIRDER	581.48	68.500	575.77	548.96	**26.81**	OK	23'-6 1/2"
3	RT. FASCIA GIRDER	574.69	66.500	569.15	540.75	**28.40**	OK	MINIMUM VERT.
4	RT. FASCIA GIRDER	574.43	66.500	568.89	540.43	**28.46**	OK	CLR = 17'-0"

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

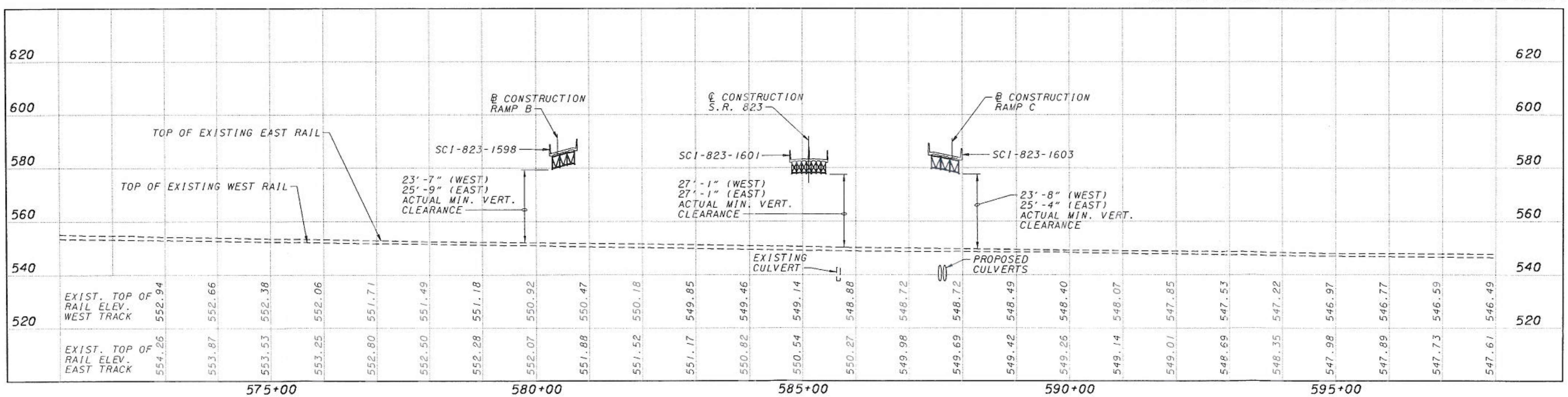
APPENDIX E



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′.



RAIL PROFILES

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			

SUPPLEMENTAL SITE PLAN

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

SCI-823-10.13
PID 79977

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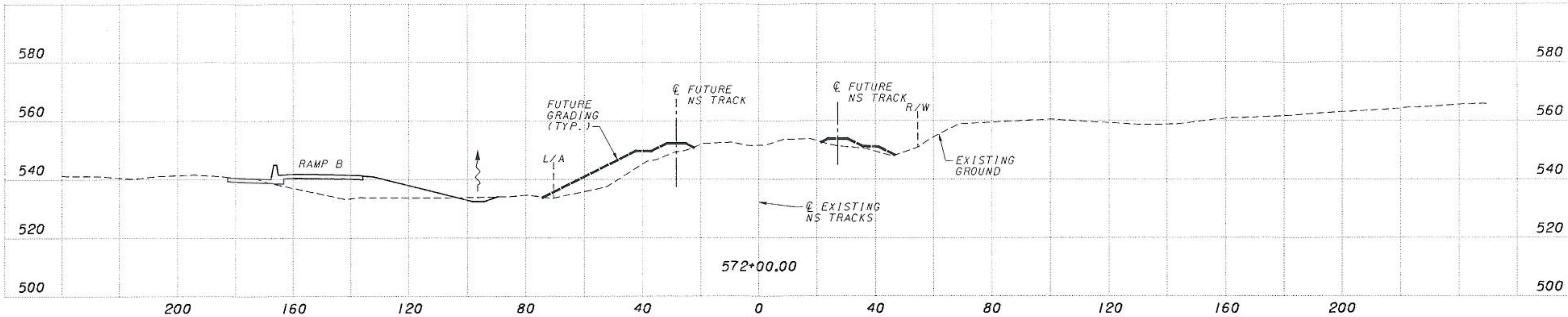

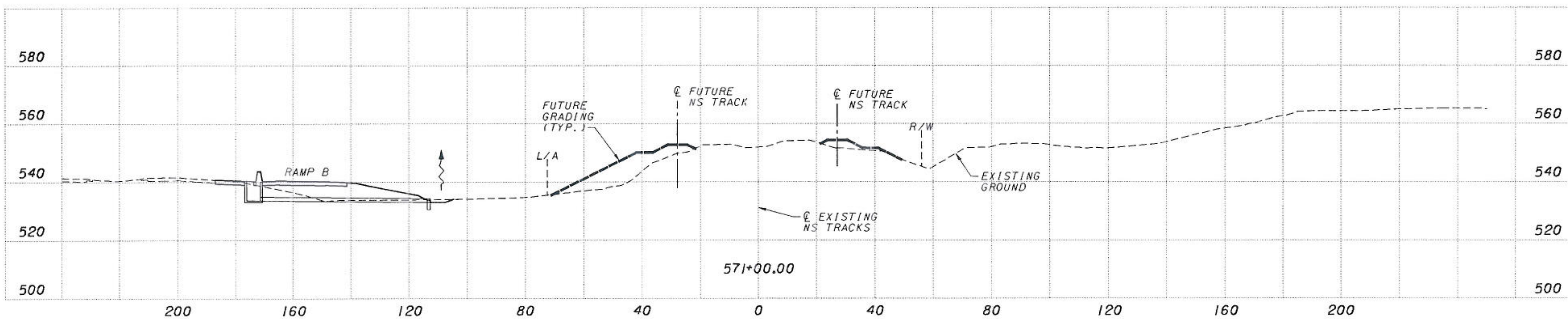

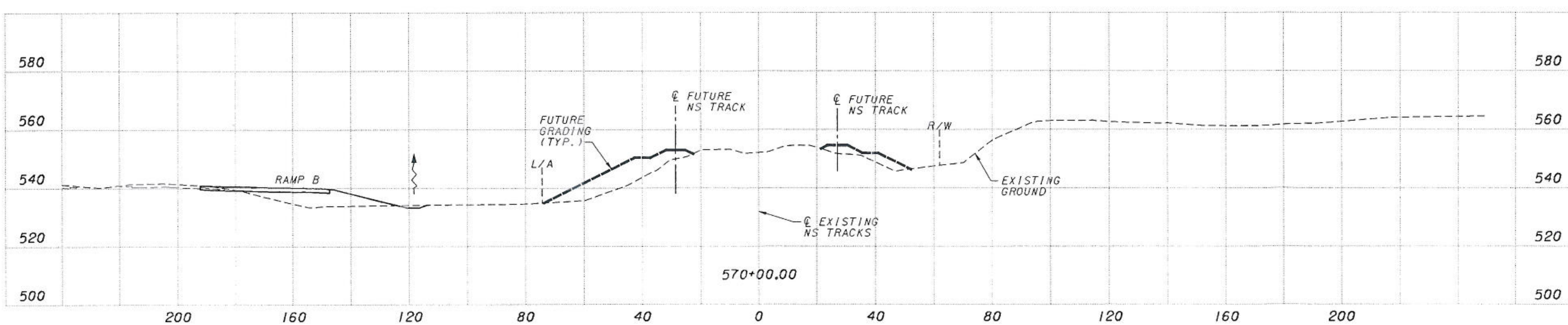

DESIGN AGENCY
CH2MHILL
5775 Perimeter Drive, Suite 190
Dublin, Ohio 43017

DESIGNED	DRAWN	REVIEWED	DATE
JBA	JBA	VKN	11/07
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SKT			

RAILROAD CROSS SECTIONS
570+00.00 TO 572+00.00

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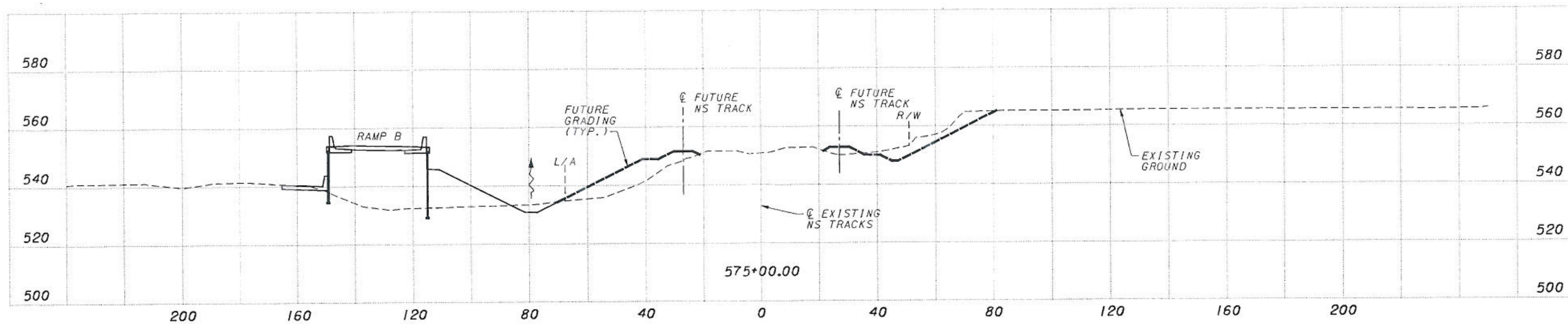

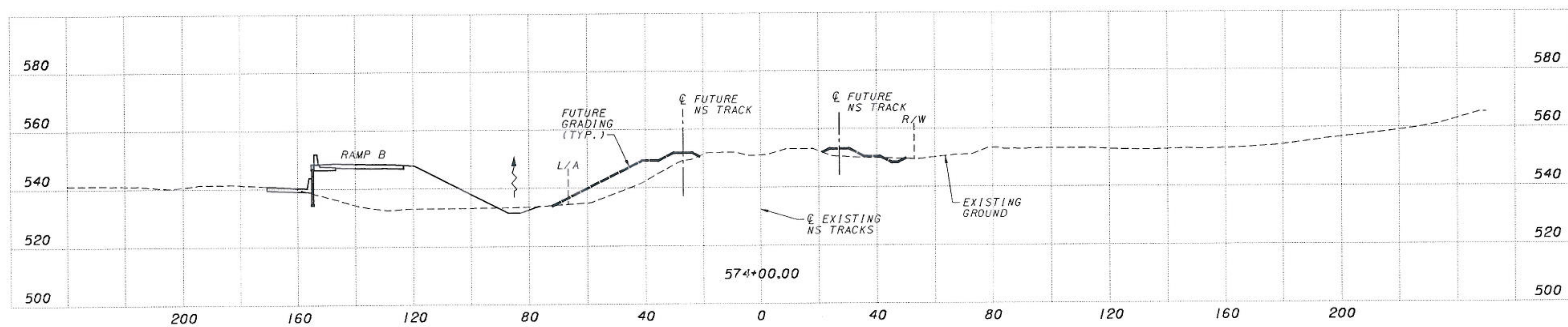

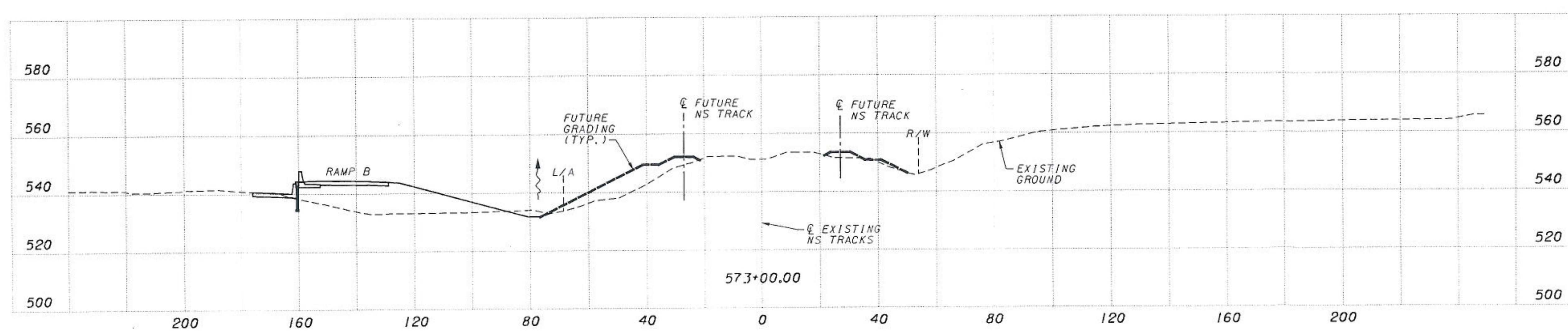

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RAILROAD CROSS SECTIONS
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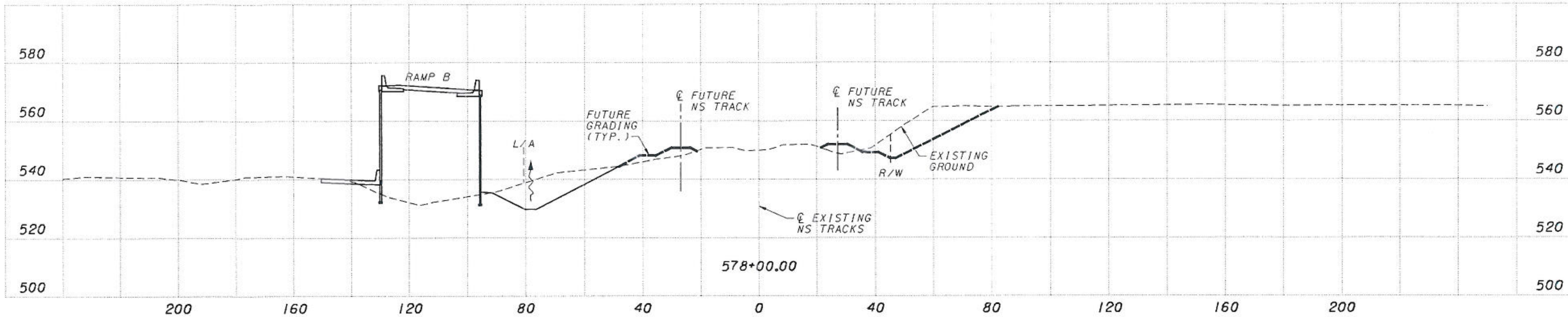

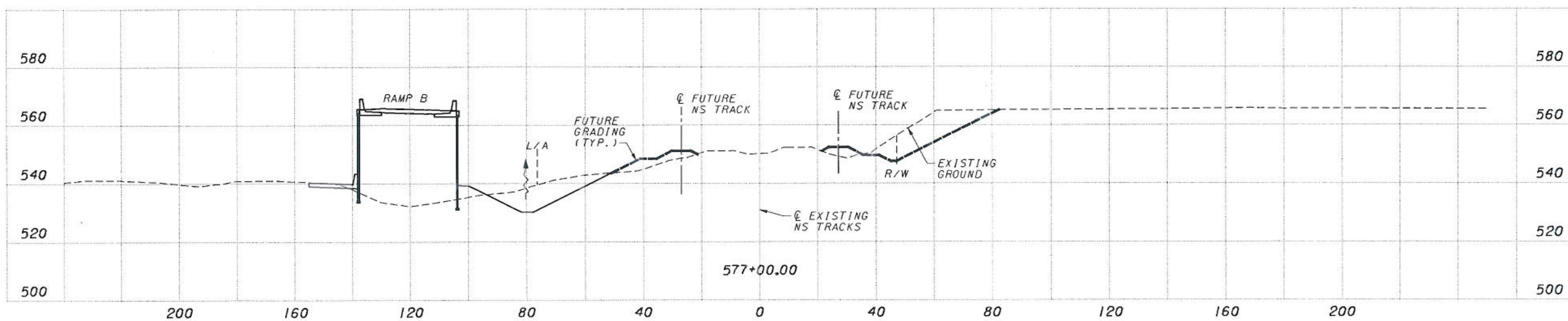

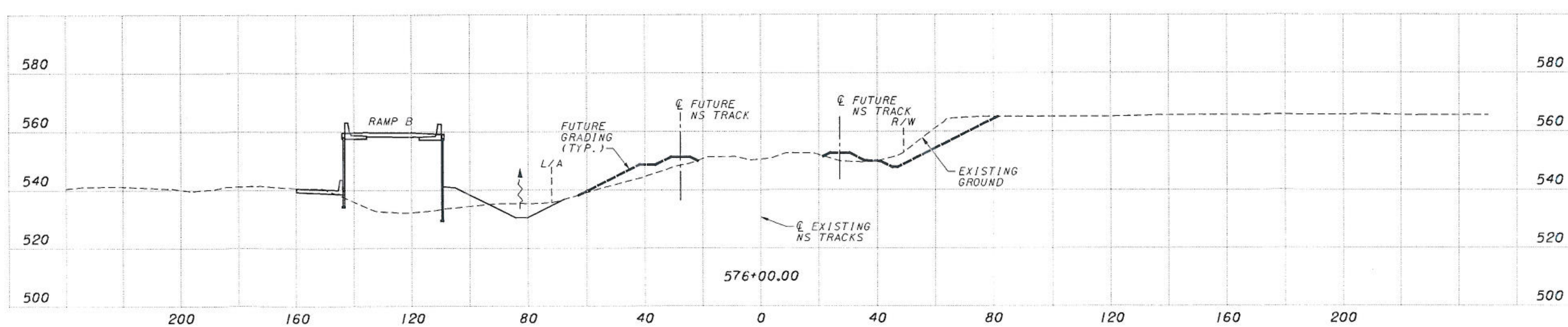

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RAILROAD CROSS SECTIONS
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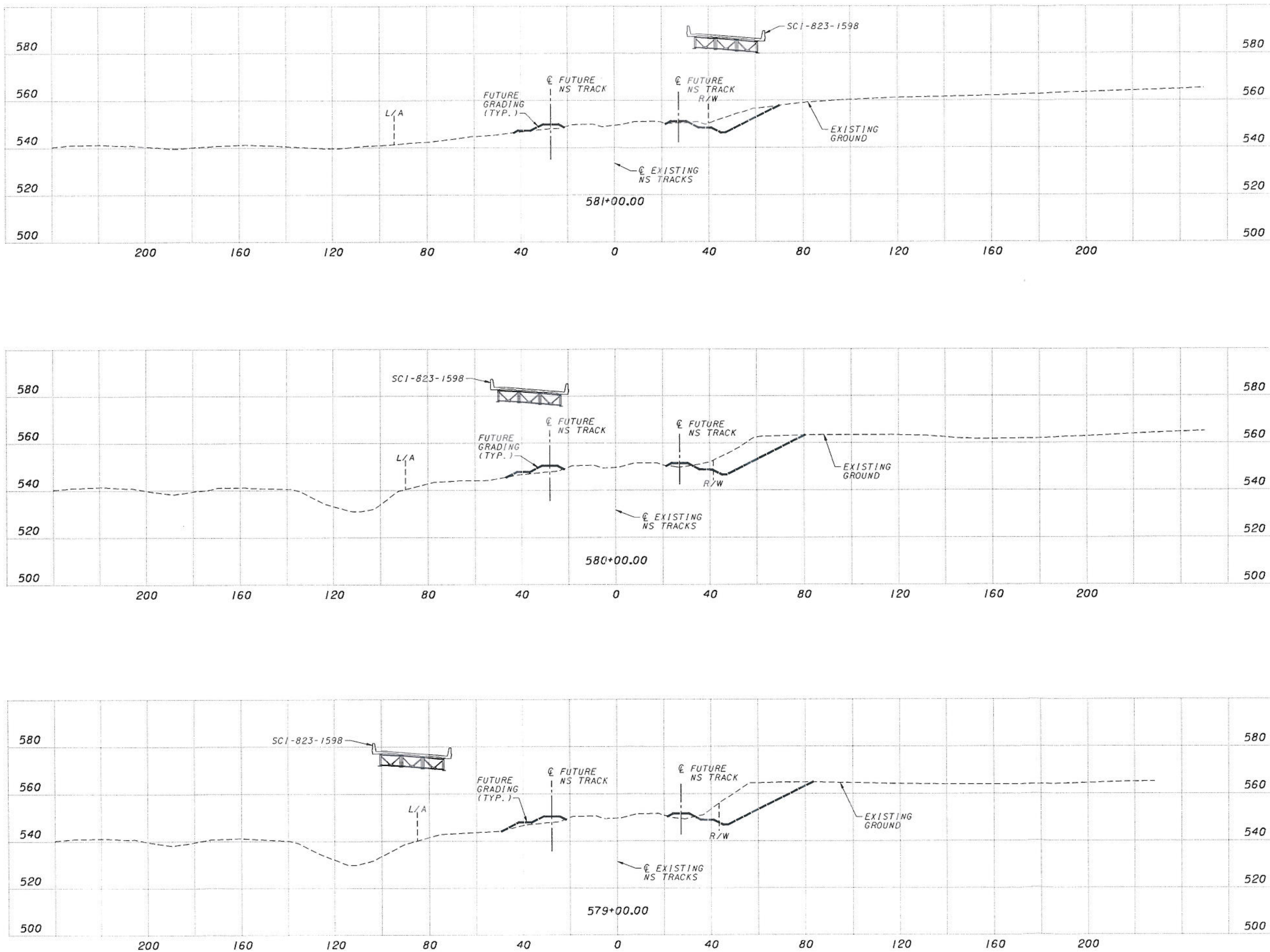

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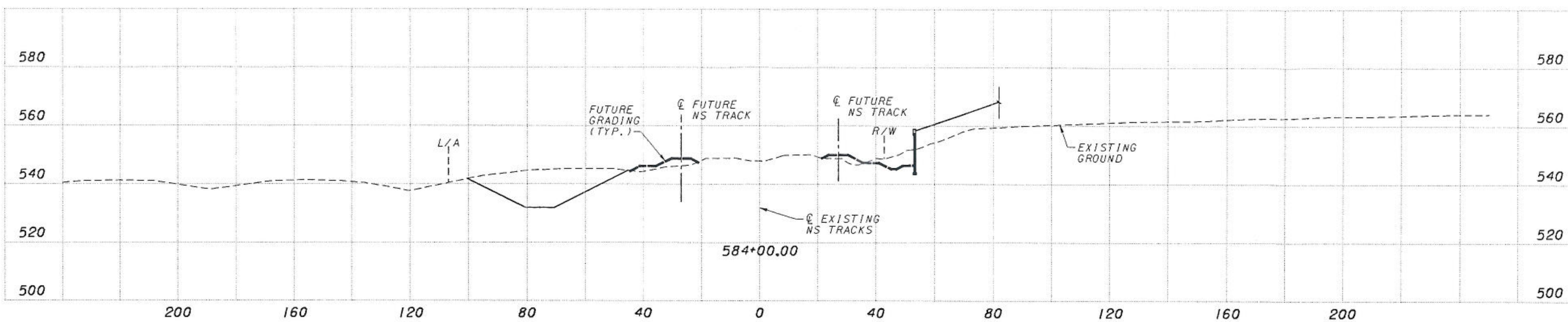

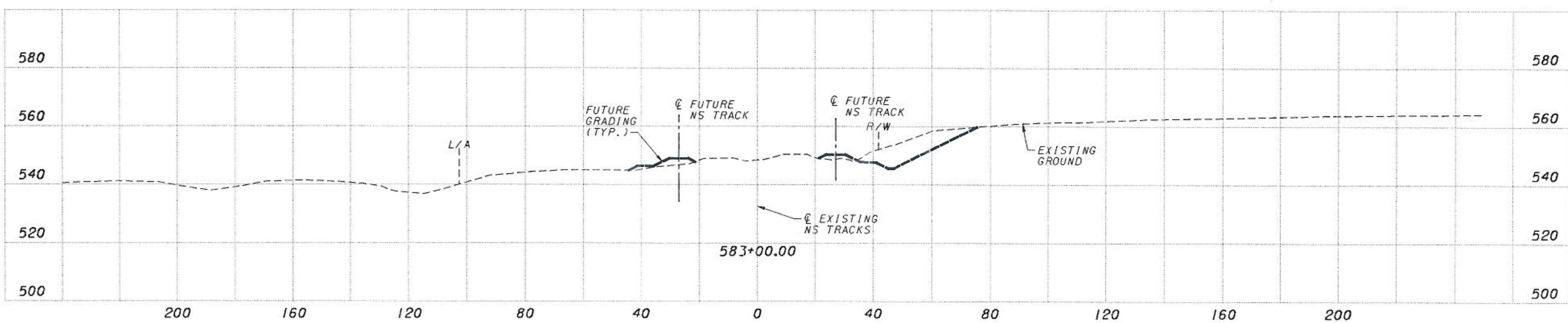

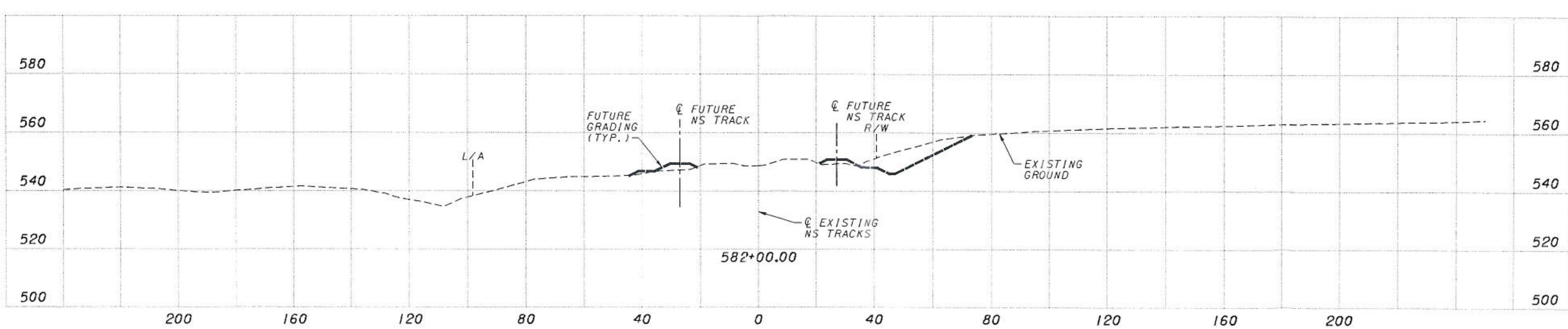

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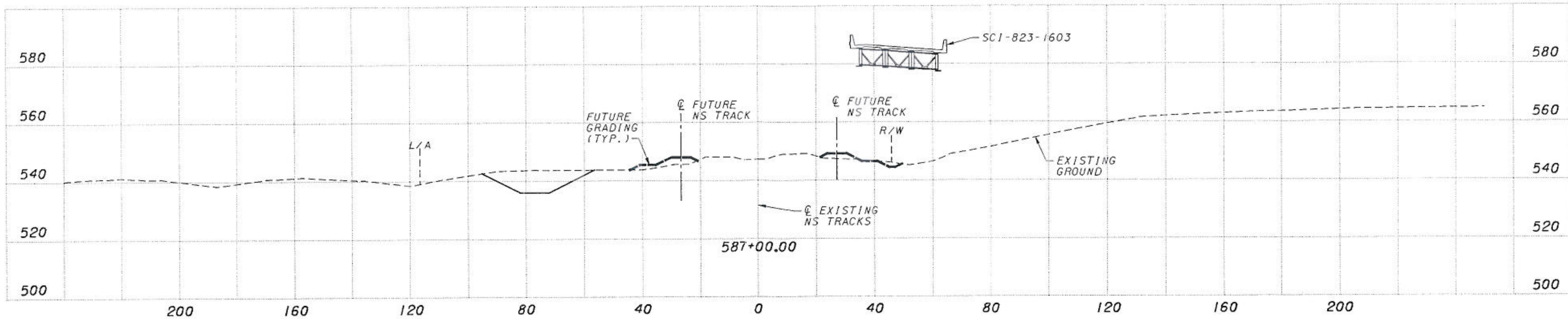

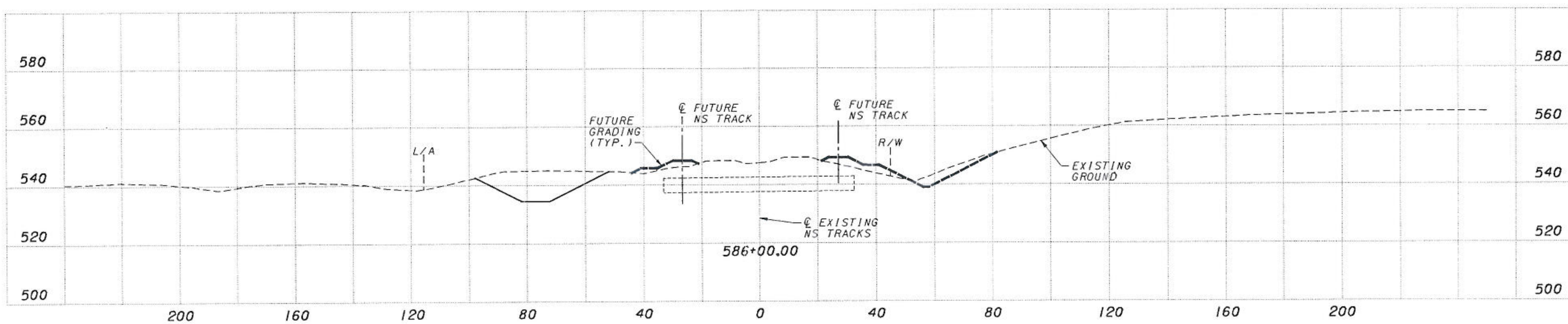

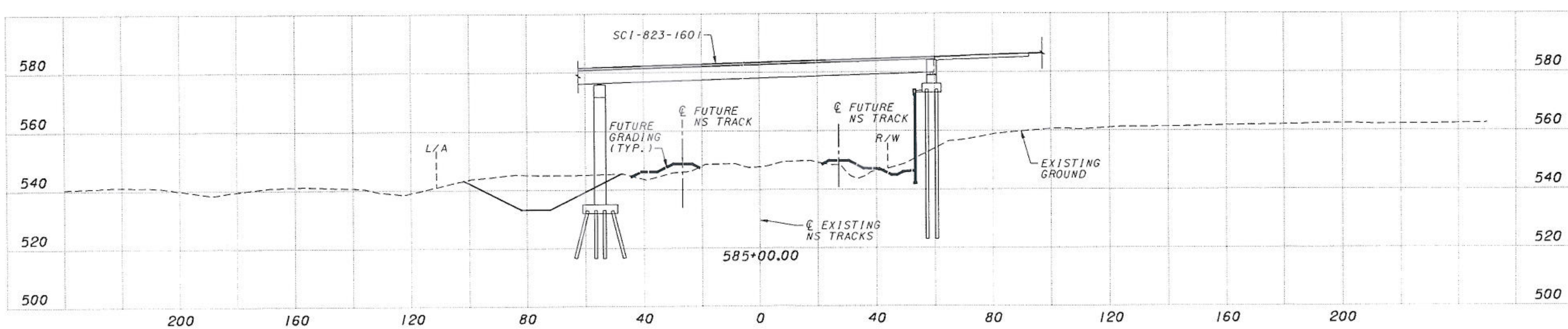

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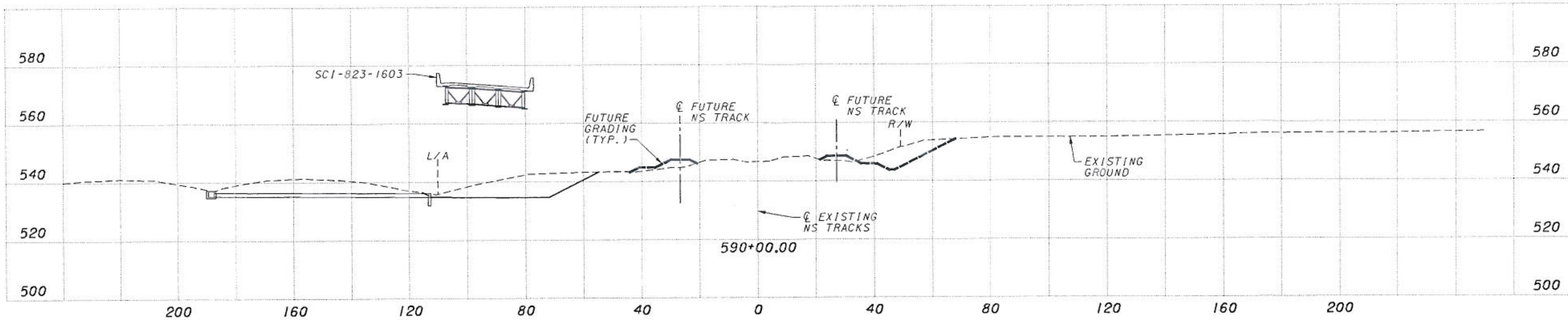

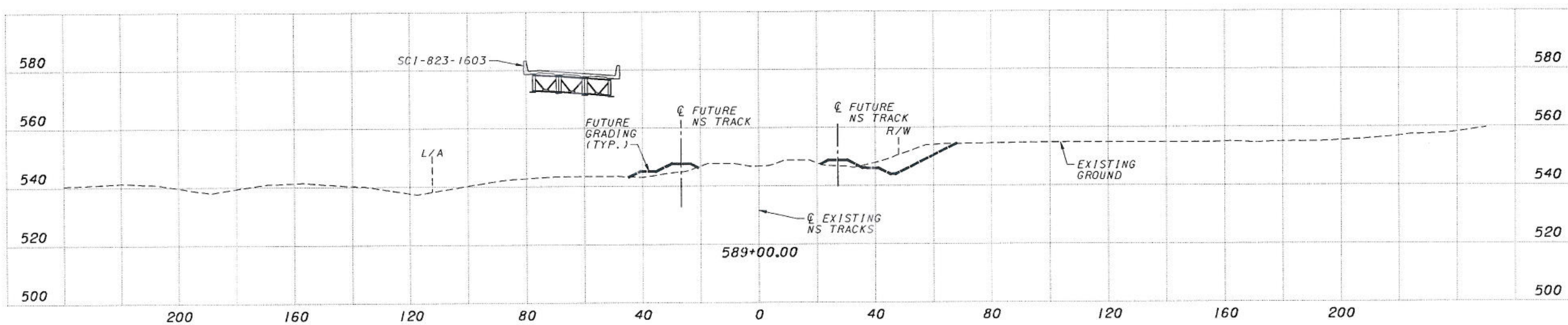

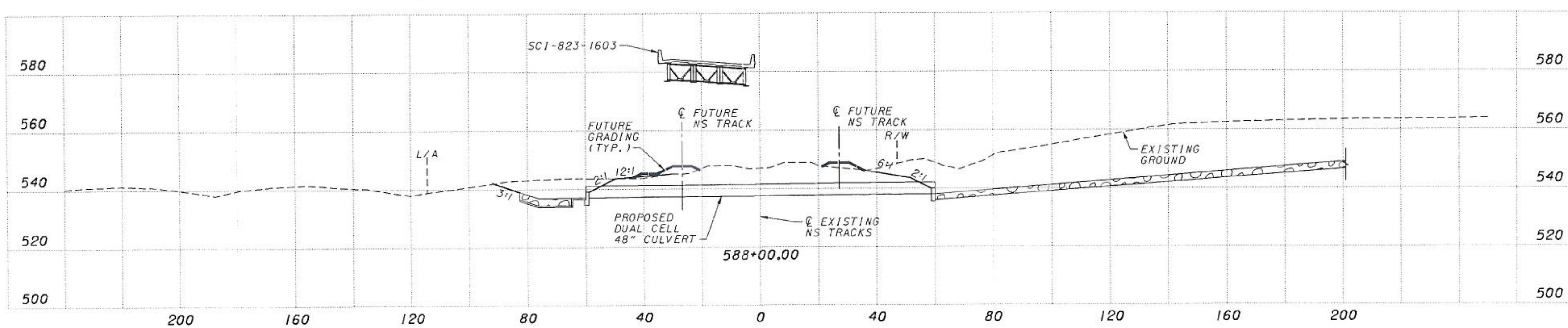

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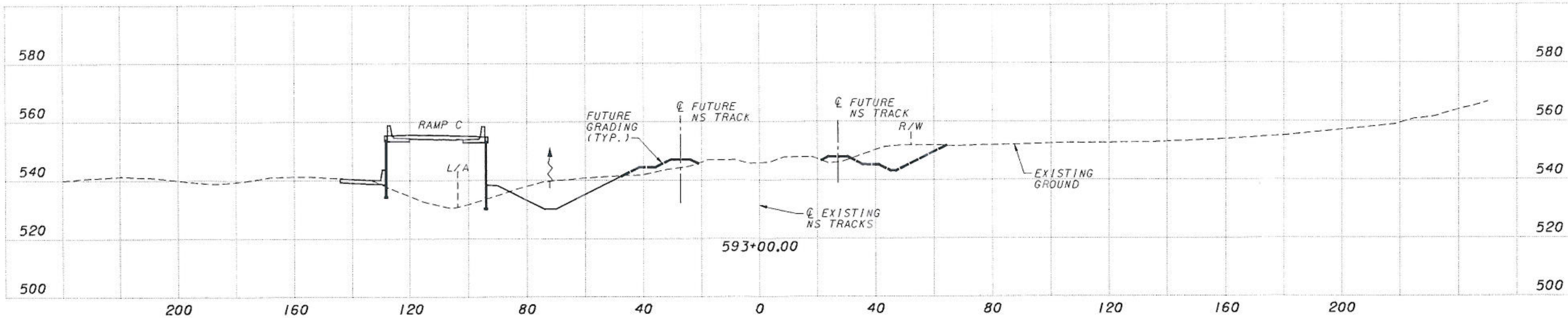

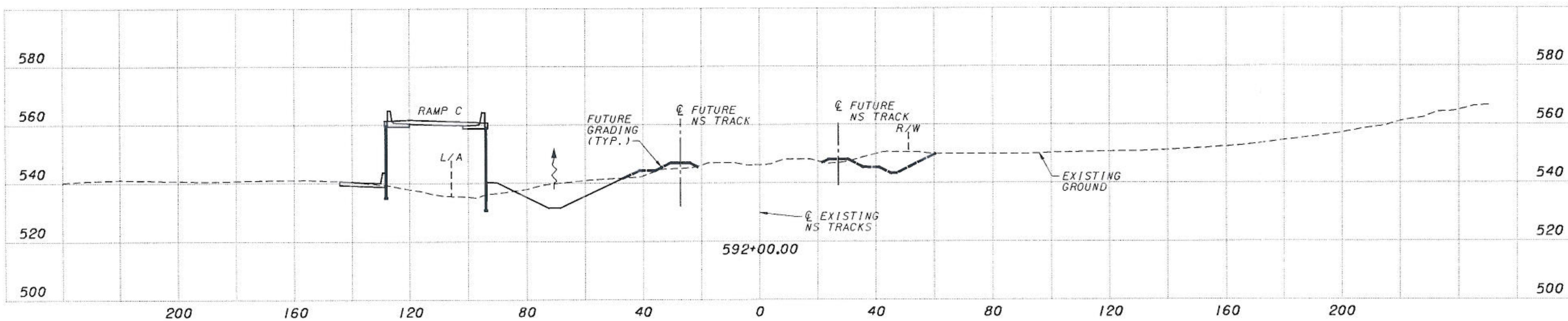

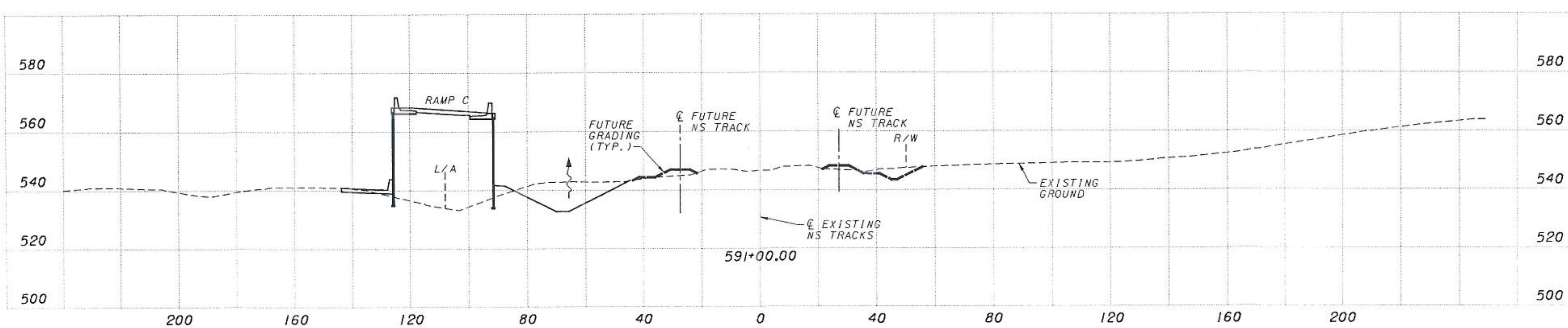

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RAILROAD CROSS SECTIONS
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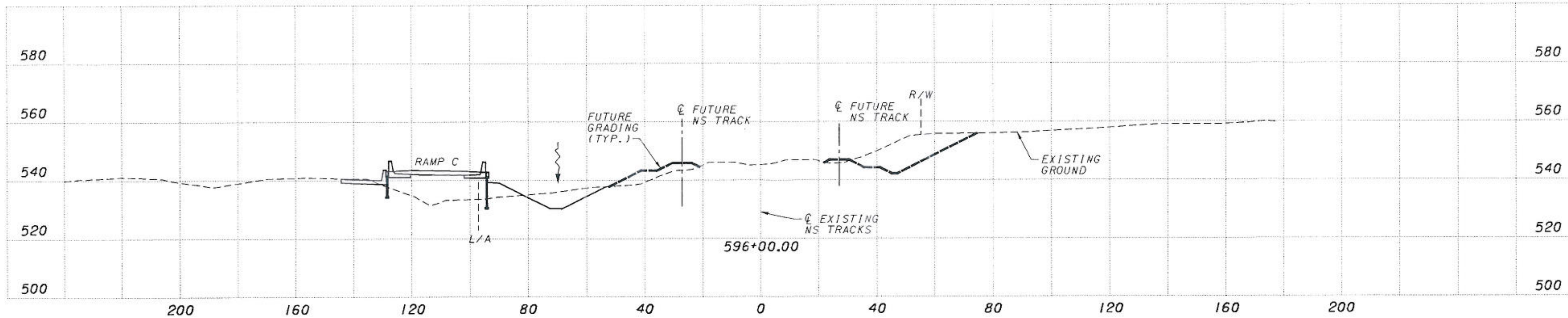

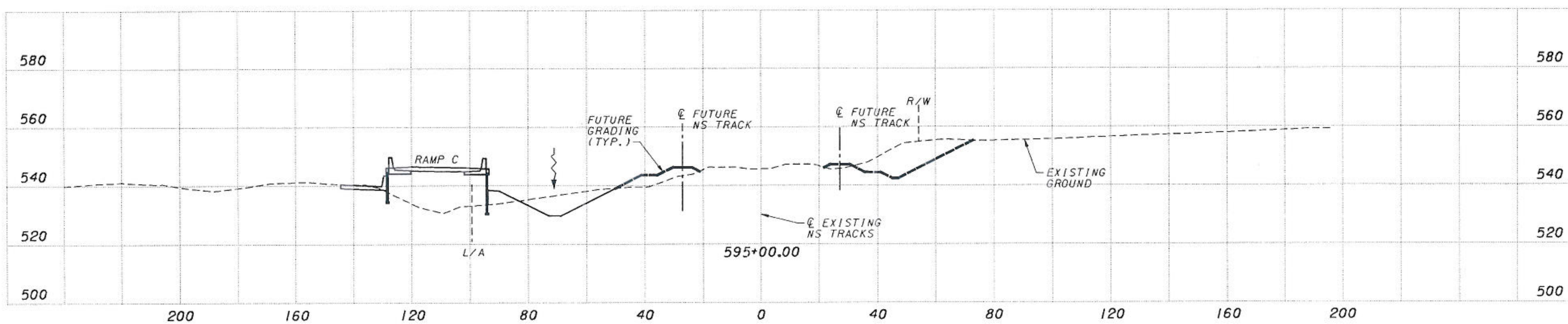

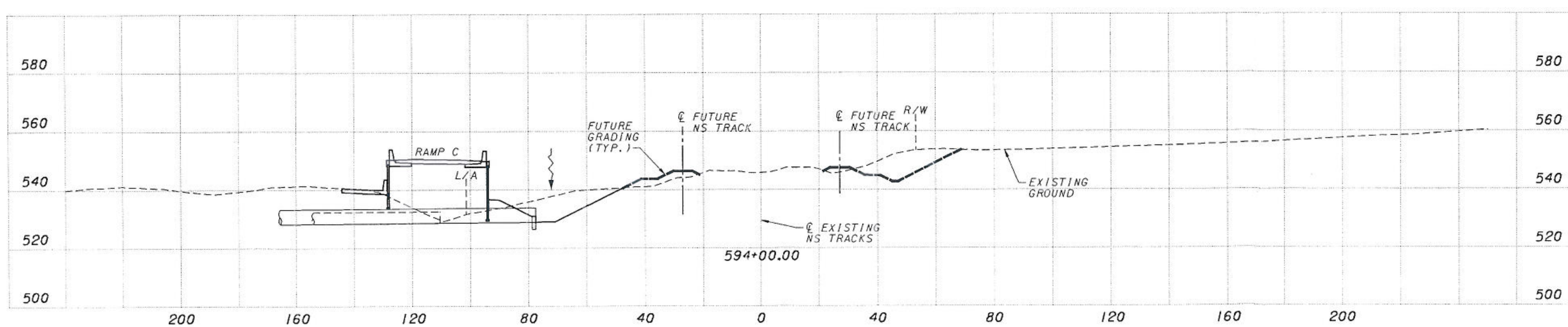

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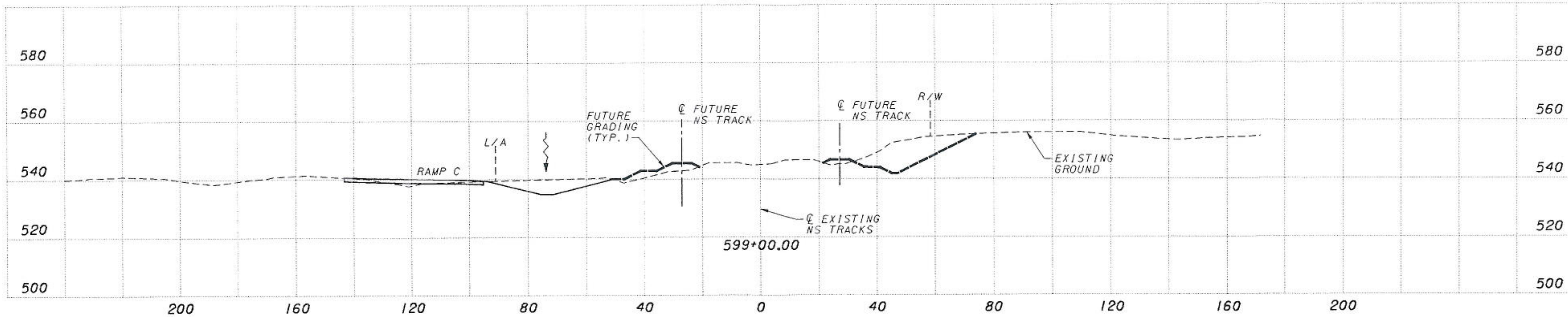

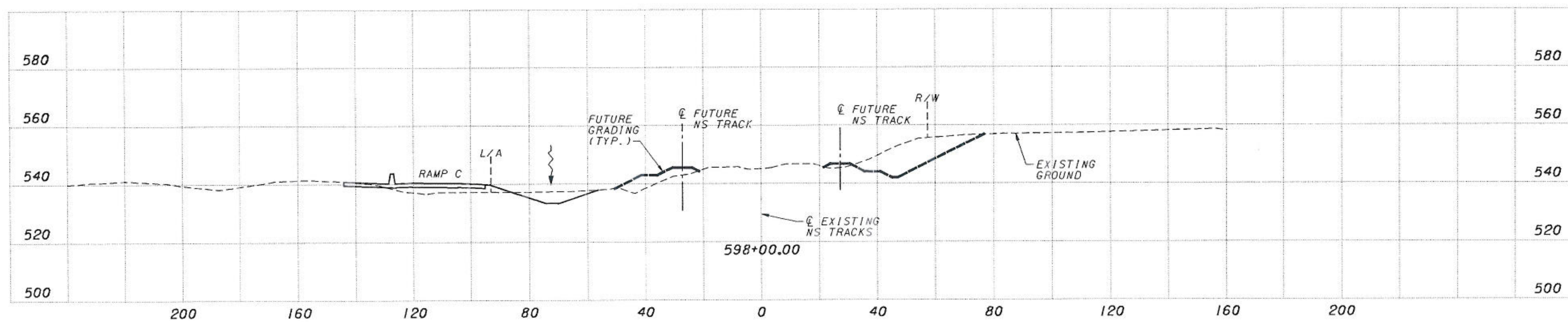

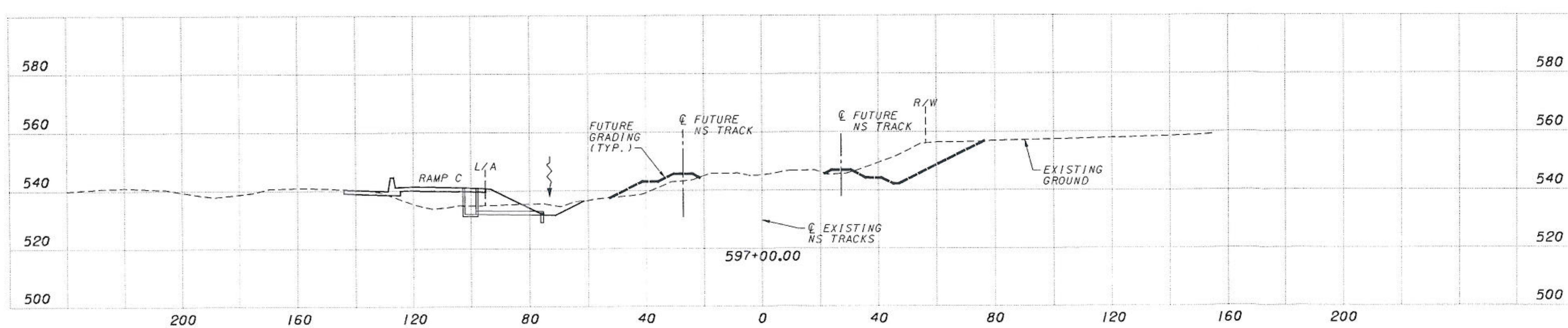

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RAILROAD CROSS SECTIONS
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APPENDIX F

inter-office communication

to: Jim Brushart, District 9 - District Deputy Director date: June 19, 2007
Attn: Tom Barnitz , District Production Administrator

from: Tim Keller, P.E., Administrator, Office of Structural Engineering by: Reza Zandi

subject: SCI-823-1601 over Norfolk Southern & U.S. 23, PID 7997, Revised Structure Type Study

We have reviewed the information furnished in the Revised Structure Type Study submittal prepared by Transystems Corporation for the above referenced bridge and offer the following comments:

1. The backwater from Scioto River will not cause any scour at the piers or abutment because the water will not have any significant velocity. A hydraulic analysis and scour analysis are not justified. The MSE walls do not require any special materials or construction procedures due to possible scour.

2. 100-year High Water Elevation shown on the Site Plan should be changed to 100-year Scioto River Backwater High Water Elevation.

3. There is no need to show Normal Water Elevation for a ditch.

4. Investigate if the culvert under SR 823 bridge can be brought into alignment with culverts under the RR. Similar to previous submission dated 07-05.

5. Top of the Pier 1 footing should be 3'-0" below the proposed ground elevation.

6. When the deck drainage is to flow off the bridge, provisions must be made to collect and carry away this run-off please refer to Bridge Design Manual (BDM) section 209.3.

7. Provide Structure File Number (SFN) in the title blocks of bridge plans.

8. PID number shown on Site Plan should be revised to 79977.

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

If you should have any questions regarding these comments, please contact our office.

TJK:JS: rz

c: Tim Keller, P.E., Office of Structural Engineering
Jawdat Siddiqi, P.E., Office of Structural Engineering

John Wetzel, P.E., District 9
Larry Wills, P.E., District 9
file

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 6/21/2007

Bridge SCI-823-1601: SR-823 over Norfolk Southern Railway & US 23

Rev: 11/26/2007
SKT

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.02

REVIEWER: ODOT OSE – Reza Zandi PHASE: Type Study

Reference Page/Sheet No.	Review Comment	Designer Response
	ODOT Comments	
General	1. The backwater from Scioto River will not cause any scour at the piers or abutment, because the water will not have any significant velocity. A hydraulic analysis and scour analysis are not justified. The MSE walls do not require any special materials or construction procedures due to possible scour.	Will comply.
Site Plan (1/3)	2. The 100-year High Water Elevation shown on the Site Plan should be changed to 100-year Scioto River Backwater High Water Elevation.	Will comply.
Site Plan (1/3)	3. There is no need to show Normal Water Elevation for a ditch.	Will comply.

Bridge SCI-823-1601: SR-823 over Norfolk Southern Railway & US 23

Rev: 11/26/2007
SKT

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.02

REVIEWER: ODOT OSE – Reza Zandi PHASE: Type Study

Site Plan (1/3)	4. Investigate if the culvert under the SR 823 Bridge can be brought into alignment with culverts under the RR. This is similar to the previous submission dated 07-05.	The drainage plan for the project was revised after the Type Study for this bridge was submitted. The proposed culvert under US 23 is now located at STA. 609+50, south of the proposed SCI-823-1601 structure. Due to insufficient hydraulic capacity of the existing railroad box culvert, an additional culvert is being proposed under the railroad at RR STA. 587+60. Flow from the two railroad culverts will lead to the proposed culvert under US 23. The location of the new proposed culvert under US 23 also addresses Review Comment #4 from the previous submission dated 07-05, by avoiding conflicts with proposed pier footings.

CH2MHILL

DESIGNER RESPONSE TO REVIEW COMMENTS

BY: SKT DATE: 6/21/2007

Bridge SCI-823-1601: SR-823 over Norfolk Southern Railway & US 23

Rev: 11/26/2007
SKT

PROJECT: **SCI-823-10.13: Portsmouth Bypass** PROJ. NO: 319861.08.02

REVIEWER: ODOT OSE – Reza Zandi PHASE: Type Study

Site Plan (1/3)	5. The Top of the Pier 1 footing should be 3'-0" below the proposed ground elevation.	The drainage plan for the project was revised after the Type Study for this bridge was submitted. The proposed culvert under US 23 at Sta. 611+00 has been relocated to Sta. 609+50±. This results in a ditch running parallel to Pier 1. The bottom of the ditch is at Elev. ±535.70'. BDM Section 303.3.2.2 states that the top of a footing should be a minimum of 1'-0" below the bottom of an adjacent ditch. If a 3'-0" thick footing and 30" of rip rap are used, then the bottom of footing elevation would have to be approximately 529.00' (535.70' – 2.50' - 1.00' – 3.00'), which is the bottom of footing elevation shown on the Site Plan. For this reason, CH2M HILL recommends that the bottom of footing elevation change to Elev. 529.00'. As the ditch design progresses, we will reevaluate the footing elevation to ensure that the top of footing is 1'-0" minimum below the bottom of the ditch.
Site Plan (1/3)	6. When the deck drainage is to flow off the bridge, provisions must be made to collect and carry away this run-off. Please refer to the Bridge Design Manual (BDM) section 209.3.	Will comply.
Site Plan (1/3)	7. Provide the Structure File Number (SFN) in the title blocks of the bridge plans.	Will comply. CH2M HILL has been notified that the Structure File Number for this bridge is **7306792**.
Site Plan (1/3)	8. The PID number shown on the Site Plan should be revised to 79977.	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]