SCI-823-0.00

PID No. 77366

S.R. 823 Ramp A (NB) Over

Ohio River Road (CR-503)

Structure Type Study Submittal

Prepared for:
Ohio Department of Transportation
District 9
650 Eastern Ave.
Chillicothe, Ohio 45601

October 31, 2006

Prepared by:

TranSystems®

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BRIDGE TYPE STUDY NARRATIVE

1. Introduction

TranSystems is providing engineering services to the Ohio Department of Transportation for the design of new overpass structures that will carry the proposed S.R. 823 ramps over Ohio River Road at the U.S. 52 interchange. This bridge type study will address the overpass structure on Ramp A, which carries traffic from westbound U.S. 52 to northbound S.R. 823. As requested by the Scope of Services, a Structure Type Study report is to be submitted before any plan development. The purpose of this report is to investigate various span arrangements and superstructure and substructure types in order to determine the most appropriate and economical structure type that will meet the project requirements. An initial Structure Type Study report dated 7/15/2005 was submitted to the Department and comments, dated 9/20/2005, were in turn received by Transystems. However, since these dates, the overall project has experienced a change in profile – the original project profile presented in the Preferred Alternative Verification Report (PAVR) submitted July 2005 has been altered in order to reduce the fill heights over culverts and to rebalance the earthwork along the entire project length. This revised project profile was approved 2/15/2006 by the Department.

Although the earthwork-based revisions altered most of the project profile, they did not affect the horizontal alignment and vertical profiles of Ramps A and B at the U.S. 52-S.R. 823 interchange. Ramp B was adjusted horizontally eastward to provide proper clearance for future railroad tracks along the Norfolk Southern Railway. This, in turn, forced an adjustment in the alignment of Ramp A. Furthermore, to compensate for the high skew at which Ramp A crosses over Ohio River Road, the profile of Ramp A was raised to provide additional superstructure depth and sufficient vertical clearance. Such action was needed to stiffen the girders and thus minimize the differential deflections (and distortions) that can arise amongst the girders of a highly skewed superstructure during deck pour and service life. The profile grade of Ramp A was therefore updated to a 900' vertical curve with PVI at Station 41+00.00, g_1 = 4.95% and g_2 = -0.88% (as compared to the original profile grade vertical curve which had a 989' length, PVI at Station 43+62.00, g_1 = 3.50% and g_2 = -0.50%). Due to this profile revision, the elevation of the proposed Ramp A overpass structure was lifted approximately 8' to 9' over that originally specified in the July 2005 PAVR. This causes an increase in the height of built-up embankments as well as proposed MSE walls. Furthermore, the 9/20/2005 ODOT comments to the original 7/15/2005 Structure Type Study point out construction and design related problems/limitations associated with structures on large skew. As a result of these large skew issues and the changes in alignment and profile, the bridge types for the proposed S.R. 823 Ramp A were reevaluated. This follow-up Structure Type Study presents the results of these reevaluations as alternative bridge types. Two (2) alternatives are evaluated in this study for construction of the proposed Ramp A overpass. Each alternative is evaluated with regard to estimated construction cost, projected maintenance costs, horizontal and vertical clearances, constructability, and maintenance of traffic. Discussion of these alternatives is presented later in this report.

2. Design Criteria

The proposed structure types are designed according to the current version of the Ohio Department of Transportation Bridge Design Manual and the 2002 AASHTO Standard Specifications for Highway Bridges, 17th Edition. Horizontal clearances (clear zone width and horizontal sight distance) and vertical clearances are based on the Ohio Department of Transportation Location and Design Manual, Volume One – Roadway Design.

3. Subsurface Conditions and Foundation Recommendation

DLZ Ohio, Inc. performed the subsurface exploration for the proposed Ramp A and prepared preliminary bridge foundation recommendations which were presented in Section 3 and Appendix E of the original 7/15/2005

Structure Type Study report. An updated Subsurface Exploration report, dated 10/16/2006, has since been prepared by DLZ Ohio, Inc. and is presented in Appendix E of this Type Study. In summary, DLZ recommends three possible solutions for supporting the Ramp A overpass abutments:

why pipe into bedrock? why not steel piles?

1) pipe piles placed in prebored holes 12 inches larger than the diameter of the pile and a minimum of 5' deep into bedrock;
2) drilled shafts socketed a minimum of 5' into competent bedrock; and,
3) spread footings bearing in fill

why worry about lateral load effects if steel piles are driven to bedrock?

Because the Ramp A overpass is located within a horizontal curve (see Sections 4 and 5 below), pipe piles or drilled shafts will be best suited to resist any lateral load effects due to this curvature. Furthermore, excessive uplift forces and lateral earth pressures are not anticipated at this site. Based on this information, DLZ's recommendations, and economics, TranSystems consequently believes pipe piles are the best foundation type for the abutments.

Preliminary MSE wall evaluations were performed by DLZ Ohio, Inc. as well and are presented in the Preliminary Subsurface Exploration report of Appendix E. These wall evaluations reveal that MSE walls can be used at the rear and forward abutment locations of Alternatives 1 and 2. DLZ anticipates that the MSE wall at the forward abutment will bear on or near bedrock whereas the MSE wall at the rear abutment will bear on either native soils or compacted granular fill (CMS Item 304) if loose, soft, or compressible soils are encountered at this location. Please refer to Appendix E for further information and details regarding MSE wall evaluations.

4. Roadway

The purpose of this project is to construct a new bypass state route – S.R. 823 – around the town of Portsmouth Ohio. The proposed alignment for S.R. 823 will carry two lanes of traffic, 15 plus miles in either direction, from an interchange with U.S. 52 just east of Portsmouth to another interchange with U.S. 23 north of Portsmouth in Valley Township. The proposed Ramp A bridge over Ohio River Road is part of the U.S. 52-S.R. 823 interchange and will carry northbound traffic from U.S. 52 to S.R. 823. Because this bridge is a ramp bridge, it will consist of one 16'-0" travel lane. The right and left shoulder widths on this bridge will be 8'-0" and 6'-0", respectively. The bridge deck will be 33'-0" out-to-out with 1'-6" right and left straight face deflector parapets (SBR-1-99). The baseline construction of Ramp A will also serve as the profile grade line and is located at the inside edge of pavement.

Because the proposed Ramp A bridge is positioned within a horizontal curve, its deck will be superelevated. The superelevation rate and layout are based on Figure 202-7E of the ODOT Location and Design Manual, Volume One – Roadway Design (using a degree of curve of 2º 15' and design speed of 60 mph) and Figure 205 of the ODOT Bridge Design Manual, respectively. Using these design references results in a superelevation rate of 0.056 ft/ft (5.6%) across the ramp travel lane. Furthermore, horizontal and vertical sight distances, in accordance with the design standards, have been provided over the proposed ramp bridge for all alternatives considered. The existing U.S. 52 and Ohio River Road will remain on their current horizontal and vertical alignments and their cross-sections, in the vicinity of Ramp A, will remain unchanged. Note that further discussion regarding the profile of the proposed ramp structure may be found in Section 5 of this report.

Vertical and Horizontal Design – As previously explained in this report, the vertical alignment of the ramp structure is dictated by vertical clearance over existing Ohio River Road. Ohio River Road is classified as an Urban Minor Arterial roadway. According to the ODOT Location and Design Manual, Volume One – Roadway Design, Figure 302-1E, a preferred vertical clearance of 17'-0" (minimum of 16'-6") must be provided over Ohio River Road. Each alternative considered for the proposed Ramp A overpass provides more than the preferred 17'-0" clearance.

Due to the existing and proposed conditions along both edges of Ohio River Road, a horizontal lateral clearance of 11'-8" minimum from edge of traveled way (i.e., edge of traveled lane) to face of obstruction should be maintained. This 11'-8" clearance applies to both edges of Ohio River Road and is derived from the following information:

1. The existing Ohio River Road is classified as an Urban Minor Arterial Street. Field inspection/evaluation of the site identified a posted speed limit of 45 mph. However, Figure 104-2E of the ODOT Location and Design Manual, Volume One – Roadway Design, recommends a design speed of 40-50 mph for an arterial street. Consequently, a *design speed of 50 mph is used.*;
2. Due to the "urban" conditions at this site, there are no ditches located off of Ohio River Road. In addition, it is intended that Type D barriers will be used/positioned off the sides of Ohio River Road. Using the arterial functional classification for Ohio River Road and a design speed of 50 mph in Figure 301-4E of the ODOT L & D Manual, Volume One (this figure is used to define lane and shoulder widths of urban roadways), *the minimum curbed shoulder width for Ohio River Road is 10' which is from edge of traveled lane to toe/face of barrier.* Note as well that Figure 302-1E of the ODOT L & D Manual, Volume One points out that for an arterial street, the horizontal lateral clearance under a bridge is a function of Figure 301-4E.
3. Footnote F of Figure 302-1E indicates that, if necessary, the 10' minimum curbed shoulder width may be reduced to 8'. This particular reduction, however, will not take place at the site in question – using a 10' width will ease any future widening of Ohio River Road.
4. According to Figure 302-1E, the horizontal lateral clearance for an arterial street under a new bridge is the sum of the curbed shoulder width (from Figure 301-4E) and barrier clearance. The barrier clearance is obtained from Figure 603-2E of the ODOT L & D Manual, Volume One. For the proposed Type D barriers that are to be used along the outside shoulder edges of Ohio River Road, the minimum barrier clearance is 20" which is also the width of a Type D barrier. Combining the 10' curbed shoulder width and the 20" minimum barrier clearance results in the 11'-8" minimum horizontal clearance.

Please note that for each alternative of the Ramp A overpass presented in this updated Structure Type Study report, the substructure and MSE wall layouts satisfy this 11'-8" clearance.

Drainage Design – The profile of the Ramp A overpass structure is on a positive grade and lies within a vertical curve whose high point is beyond the forward abutment (see Section 5 for profile information). Storm water runoff will drain from this high point towards the rear abutment. Superelevation due to horizontal curvature will also force drainage toward the right shoulder. However, the skew at which the Ramp A overpass crosses Ohio River Road makes it difficult to position scuppers in the bridge deck, especially along the right shoulder near the rear abutment – scuppers in the bridge deck will drain directly onto Ohio River Road (and its paved shoulders) and any drainage piping/plumbing connected to the bottom of the scuppers cannot be properly supported. Consequently, the collection of storm water runoff will be addressed off the bridge – catch basins will be positioned ahead of the forward abutment to prevent as much runoff as possible from draining onto the structure. Note that the type and layout of the drainage system will be investigated during the TS&L stage.

Utilities - No utilities will be placed on the bridge. However, lighting and ITS conduits will be provided if necessary.

Maintenance of Traffic - While the Ramp A overpass is under construction, traffic will be maintained on the existing Ohio River Road. It is anticipated that there will be limited closures during construction, primarily for beam setting.

5. Proposed Structure Configurations

Alignment & Profile: The proposed horizontal geometry for the Ramp A overpass structure is defined by a horizontal curve that is part of a spiral-curve-spiral alignment. The spiral portions of this alignment are well outside the overpass limits, so the parameters that define the pertinent horizontal curve portion are as follows: P.I. = Station 44+40.37, Δ (angle of intersection) = 34°25'56.25" Rt., D_c (degree of curve) = 2°15'00", R (radius) = 2546.48', T (tangent length) = 789.05', L (length of curve) = 1530.32', E (external distance) = 119.45', P.C. = Station 36+51.32, and P.T. = Station 51+81.64. The proposed profile for this same ramp structure is located on the inside edge of pavement which also serves as the baseline construction of Ramp A. This profile lies within a 900' vertical curve with P.V.I. at Station 41+00.00, g_1 = 4.95% and g_2 = -0.88%. The horizontal and vertical geometry for all alternatives considered are the same.

Several roadways are closely aligned in the proposed U.S. 52-S.R. 823 interchange. These are the existing U.S. 52, the existing Ohio River Road, the proposed Ramp A carrying traffic from westbound U.S. 52 to northbound S.R. 823, and the proposed Ramp B carrying traffic from southbound S.R. 823 to eastbound U.S. 52. The close proximity of these roadways and their differences in elevation at various locations warrant the use of MSE walls to satisfy both grading continuity and safe/proper embankment limits. MSE walls will be required not just along the roadway portions of Ramps A and B, but also at the abutments of the respective overpass structures. However, the proposed alignment of Ramp A causes the Ramp A bridge to cross Ohio River Road at a very high skew angle (approximately 70° left forward). It is known that a large skew can cause numerous construction problems for a bridge regardless of whether it is on a tangent alignment or within a horizontal curve (refer to ODOT BDM Section 302.2.7). In addition, MSE walls do not perform well and should not be utilized on structures with significant acute skews – ODOT stipulates that the maximum direction change for the face of an MSE wall should be 90 degrees (if possible) and that acute corners should be avoided (see Supplemental Specification 840). As a result of these criteria, substructure units, including MSE walls, for the Ramp A overpass structure should be oriented at skews of 30 degrees or less. Combining these lower skews with the 11'-8" minimum horizontal clearance along the shoulders of Ohio River Road results in the need of a long structure to overpass Ohio River Road. Because a pier cannot be placed on Ohio River Road, the resulting Ramp A overpass must consequently consist of a long, single span. The alternatives considered in this type study are essentially variations of this long, single span structure.

Structure: As per the Scope of Services, several bridge types and alternates were investigated as part of this type study. A total of two (2) alternatives were considered and are outlined in the following Structure Type Alternative Table:

STRUCTURE TYPE ALTERNATIVE TABLE		
Structure Type Alternative	**1**	**2**
Superstructure Type Description	125" web, Single-Span Steel Plate Girders A709 Grade 50W (dog-legged at splices)	122" web, Single-Span Steel Plate Girders A709 Grade 50W (dog-legged at splices)
Proposed Beam Spacing	3 Spaces @ 9'-2"±	3 Spaces @ 9'-2"±
No. of Spans	1	1
Abutment Type	**Rear and Fwd. Abut.**: Stub Type behind MSE wall	**Rear and Fwd. Abut.**: Stub Type behind MSE wall
No. of Piers	0	0
Pier Type	N/A	N/A
Substructure Orientation	30°00'00" LF (w/respect to Reference Line)	0°00'00" (w/respect to Reference Line)
Approx. Bridge Length	215'-8"	233'-5 3/16"
Approx. Structure Depth Slab Haunch Beam Total	 8.75" 2" 130.625" 141.375" (11.781')	 8.75" 2" 124.9375" 135.6875" (11.307')

Alternative Discussion:

Alternative 1

Span configuration: Alternative 1 is a long, single–span bridge with abutments located behind mechanically stabilized earth (MSE) walls. The abutments and MSE walls are oriented at a 30º00'00" left forward skew with respect to the reference line that runs from centerline of bearing to centerline of bearing. If MSE walls with turnback sections are not utilized along the Ramp A overpass, embankments with 2:1 spill-through-slopes will need to be constructed. Such embankments would encroach onto Ohio River Road and interfere with the Ramp B structure (see diagram in Appendix B). The use of MSE walls (with proper turnbacks) consequently permits proper embankment construction and allows grading and elevation requirements to be successfully coordinated amongst the proposed and existing roadways of the U.S. 52-S.R. 823 interchange. MSE walls are positioned to provide the 11'-8" minimum horizontal clearance required along the shoulders of Ohio River Road. Positioning the MSE walls in this manner and providing a minimum distance of 3'-6" between the back face of MSE wall panels and the centerline of front row of abutment piles (refer to ODOT BDM Section 204.6.2.1 and Fig. 331) results in an overall bridge length of 215'-8" from centerline of bearing to centerline of bearing. Note that this length is measured along the centerline of survey and construction of Ramp A.

Substructure:

I. Abutments: Due to the horizontal curvature, a conventional, or stub-type, abutment must be used at both the rear and forward abutments (refer to ODOT BDM Section 205.9). Turnback wingwalls will be used at the rear and forward locations and all abutment and wingwall details will follow ODOT Standard Drawing A-1-69.

From Section 3 earlier in this report, it was recommended that pipe piles be used to support the abutments. According to DLZ, these piles are to be placed in prebored holes 12" larger than the diameter of the pile and embedded in rock sockets that are 5' deep into bedrock. Precursory load analyses reveal that both the rear and forward abutments may be founded on 16" diameter pipe piles with a design capacity of 90 tons per pile and placed in 28" diameter sockets.

II. Piers: none.

Superstructure:

I. Girders and Deck: The superstructure for this alternative consists of 4-welded steel plate girders, Grade 50W, with 125" deep webs and an $8^{3}/_{4}$" thick deck (which includes a 1" monolithic wearing surface). The deck width is 30'-0" from toe-to-toe of parapet and has an overall width of 33'-0". Although this structure consists of a single-span, its length warrants the use of girder splices which allows for the fabrication of shorter, less costly girder segments. Such girder segments are easier to transport and will facilitate/simplify superstructure erection. Furthermore, the plate girders will be dog-legged to accommodate the horizontal curvature of the bridge. Dog-legging permits fabrication of straight girder segments which is easier and more economical than the fabrication of curved girder segments. The straight girder segments will be dog-legged at the splice points and placed parallel to one another between splices. Erection of the girders in this manner results in a center-to-center girder spacing of 9'-2"± (spacing between splice points actually varies from 9'-0 $^{3}/_{16}$"± to 9'-4 $^{7}/_{16}$"± - refer to the framing plan for Alternative 1). With such spacing, the 4-welded steel plate girders discussed above will satisfy the HS-25 (Case I) and Alternate Military Loading as well as a Future Wearing Surface loading of 60 psf.

The 125" web depth is needed to control the differential deflections between adjacent girder lines at coincident locations such as crossframes. Large skews and horizontal curvature – both of which are present on the Ramp A overpass – are significant sources of differential deflections in a superstructure and placement of deck concrete on a superstructure with such characteristics can induce, due to excessive differential deflections, web and flange distortion as well as out-of-plane bending of the girders (refer to ODOT BDM Section 302.2.7). Preliminary design efforts for the Ramp A overpass generated 4-plate girders, each comprised of a 105" deep web, 1.5" thick top flange, and 1.5" to 1.8125" thick bottom flange. Differential deflections between interior girders with these dimensions, however, exceeded the 0.5" limit specified in ODOT BDM 302.2.7. Consequently, the superstructure was stiffened to reduce these differential deflections. One option considered was adding an additional girder line (for a total of 5). This resulted in an undesirable solution – although a web deeper than 100" was required to limit differential deflections below 0.5", the problem with this solution was the required use of an uneconomic girder spacing of approximately 7', well below the recommended 9' minimum spacing of ODOT BDM 205.6. Other possible options/solutions included implementing the recommendations of AASHTO/NSBA Steel Bridge Collaboration document G 12.1-2003 "Guidelines for Design for Constructability" which addresses differential deflections through rotation checks at bearings and detailing/erecting steel for full dead load fit. In addition, TranSystems staff have investigated deck pours for a variety of skewed structures and designed temporary bracing to prevent girder rotation and crossframe overstressing. However, these solutions only address construction issues and do not address service life issues that may arise due to the large skews and/or horizontal curvature. Ultimately, from economic, structural, and serviceability

what is diff. deflection for 105" web? is it more than 1"?

standpoints, it was deemed that the best way to stiffen the superstructure was to increase the depth of the 4 original girder lines. Providing 4 girder lines, each comprised of a 125" deep web, a 25" x 2.75" top flange, and a 29" x 2.75"-2.875" bottom flange not only limited the differential deflections between adjacent girder lines to below 0.5", it also satisfied the 17'-0" preferred vertical clearance over Ohio River Road and provided sufficient structural capacity to support the appropriate design loads.

II. Expansion Devices and Bearings: Since there are no fixed bearings on this single-span structure, a preliminary evaluation of expansion devices involved using the overall bridge length of 215'-8" as the expansion length. Section 306.3.3 of the ODOT Bridge Design Manual and ODOT Standard Drawing EXJ-4-87 reveal that a 4" strip seal expansion joint can be used at both the rear and forward abutment. Note that this result is based on a simple preliminary evaluation of the bridge system and ignores, for now, the effects of horizontal curvature. In addition, a preliminary evaluation of bearings was performed. AASHTO Method A was used to identify laminated elastomeric bearings that can support the vertical reactions at the abutments, horizontal displacements due to thermal expansion/contraction, and rotational displacements due to applied dead and live loads. Consequently, laminated elastomeric bearings are recommended as the bearing type for Alternative 1.

The initial bridge construction cost for Alternative 1 is estimated to be $5,880,000 in year 2008 dollars. The present life cycle maintenance costs for this alternative are estimated to be $1,356,000, resulting in a total estimated ownership cost of $7,236,000 in year 2008 dollars.

Alternative 2

Span configuration: Alternative 2 was investigated in an effort to further reduce the skew of the Ramp A overpass. This alternative has a similar horizontal layout as Alternative 1, except that the rear and forward abutments and respective MSE walls are oriented at a 0°00'00" skew with respect to the reference line. Such a skew is ideal for a horizontally curved structure – it minimizes torsional effects and distortions (and thus differential deflections) on the I-shaped plate girders. However, this same skew, when utilized with the required 11'-8" horizontal clearance along the shoulders of Ohio River Road, will increase the length of the Ramp A overpass. The resulting centerline of bearing-to-centerline of bearing length for this alternative is 233'-5 $^{3}/_{16}$" (measured along the centerline of survey and construction of Ramp A).

Substructure: Except for the orientation (skew), the substructure units and foundations used in Alternative 2 are identical to those in Alternative 1.

Superstructure: Alternative 2 is identical to Alternative 1 except for the skew and the use of 4-welded steel plate girders, Grade 50W, with 122" deep webs, 24" x 1.25" top flanges, and 32" x 1.25"-1.6875" bottom flanges. Girders with this depth provide sufficient capacity for a 233' single-span structure whereas girders with a web depth below 120" are insufficient. In accordance with ODOT BDM 306.3.3 and ODOT Standard Drawing EXJ-4-87, a 3" strip seal expansion joint can be used at both the rear and forward abutment (based on a simple preliminary evaluation of the bridge system). As with Alternative 1, laminated elastomeric bearings are recommended as the bearing type.

The initial bridge construction cost for Alternative 2 is estimated to be $5,730,000 in year 2008 dollars. The present life cycle maintenance costs for this alternative are estimated to be $1,466,000, resulting in a total estimated ownership cost of $7,196,000 in year 2008 dollars.

6. Recommendations:

Based upon the above information and discussions, TranSystems recommends **Structure Type Alternative 2**, which is a single-span structure comprised of A709 Grade 50W plate girders with 122" deep webs (girders are dog-legged at splice locations) and stub-type abutments behind MSE walls (see Appendix B for the Site Plan and Structure Details).

Alternative 2 is preferred, and thus recommended, based on the following items:

1. The lower skew angle minimizes/reduces the number of construction problems (such as girder distortions during deck pour) and thus simplifies construction;

2. The shorter web depth, and thus shorter superstructure, improves vertical clearance over Ohio River Road;

3. The shorter superstructure is less costly to erect/construct;

4. The lower skew angle will help reduce the number of serviceability issues (such as girder distortions, out-of-plane bending, etc.) that may arise over the life of the structure;

5. Alternative 2 has the lower total relative ownership cost of the two alternatives considered.

APPENDIX A
Cost Comparison Summary

TranSystems

SCI-823-0.00 - PORTSMOUTH BYPASS

RAMP A (NB) OVER OHIO RIVER ROAD - US 52 INTERCHANGE,

STRUCTURE TYPE STUDY

By: JRC
Checked: MSL

Date: 10/23/2006
Date: 10/30/2006

ALTERNATIVE COST SUMMARY

Alternative No.	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%)	Structure Contingency Cost (20%)	Total Alternative Cost	Superstructure Life Cycle Maintenance Cost	Total Relative Ownership Cost
1	1	215.66	215.66	4~ Welded Dog-legged Plate Girders	125" Web PG Grade 50W	**$1,444,000**	**$2,778,000**	**$675,500**	**$979,500**	**$5,880,000**	**$1,356,000**	**$7,236,000**
2	1	233.43	233.43	4~ Welded Dog-legged Plate Girders	122" Web PG Grade 50W	**$1,132,000**	**$2,983,000**	**$658,400**	**$954,700**	**$5,730,000**	**$1,466,000**	**$7,196,000**

NOTES:

1. Structure incidental cost allowance includes provision for structure excavation, porous backfill, sealing of concrete surfaces, structural steel painting, bearings, and crushed aggregate slope protection costs.
2. Estimated construction cost does not include existing structure removal (if any), which is common to all alternatives.

180,000
2.6%

40,000 0.5%

SCI-823-0.00 - PORTSMOUTH BYPASS
RAMP A (NB) OVER OHIO RIVER ROAD - US 52 INTERCHANGE,
STRUCTURE TYPE STUDY - STEEL GIRDER ALTERNATIVES- SUPERSTRUCTURE

By: JRC　　Date: #######
Checked: MSL　　Date: #######

SUPERSTRUCTURE

Alternative No.	Span Arrangement No. Spans	Lengths	Total Span Length (ft.)	Deck Length (ft.)	Deck Volume (cu. yd.)	Deck Concrete Cost	Deck Reinforcing Cost	Approach Slab Cost	Expansion Joint Cost	Proposed Stringer Section	Structural Steel Weight (Pounds)	Structural Steel Cost	Subtotal Superstructure Cost
1	1	215.66	215.66	218	282	$168,800	$70,800	$45,100	$26,200	125" Web PG Grade 50W	940,100	$1,132,700	**$1,444,000**
2	1	233.43	233.43	235.43	305	$182,600	$76,600	$45,100	$22,700	122" Web PG Grade 50W	664,800	$801,000	**$1,132,000**

COST SUPPORT CALCULATIONS

Deck Cross-Sectional Area:

Parapets:

	No.	Individual Area (sq. ft.)	Parapet Area (sq. ft.)
Parapets	2	4.26	8.52
Split Median Barriers	0	4.52	0.00

Slab:

	T (ft.)	W (ft.)	Slab Area	Haunch & Overhang Area	Total Concrete Area (sq. ft.)
Alt. 1	0.73	33.00	24.1	2.4	35.0
Alt. 2	0.73	33.00	24.1	2.4	35.0

Note: Deck width is out to out
10% of deck area allowed for haunches and overhangs.

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

	Year 2004	Annual Escalation	Year 2008
Deck	$491.00	3.5%	$563.00
Parapets	$615.00	3.5%	$706.00
Weighted Average =			$598.00

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

	Year 2004	Annual Escalation	Year 2008
Deck Reinforcing	$0.77	3.5%	$0.88

Structural Steel
Unit Costs ($/lb.):

	Cost Ratio	Year 2004	Annual Escalation	Year 2008	
Rolled Beams - Grade 50	n/a	$0.74	3.5%	$0.85	
Level 4 Plate Girders - Grade 50W	n/a	$1.05	3.5%	$1.20	Straight Girders
level 5 Plate Girders - Grade 50W	n/a	$1.20	3.5%	$1.38	Curved Girders

Reinforced Concrete Approach Slabs (T=17")
Unit Cost ($/sq. yd.):
Length = 30 ft.　　Width = 33 ft
Area = 110 sq. yd.

	Year 2005	Annual Escalation	Year 2008
Approach Slabs	$185.00	3.5%	$205.00

Expansion Joints
Unit Costs ($/Lin.Ft.):

	Cost Ratio	Year 2005	Annual Escalation	Year 2008
Modular Expansion Joints	1.00	$910.00	3.5%	$1,008.93
Strip Seal Expansion Joints	1.00	$310.00	3.5%	$343.70

SCI-823-0.00 - PORTSMOUTH BYPASS
RAMP A (NB) OVER OHIO RIVER ROAD - US 52 INTERCHANGE,
STRUCTURE TYPE STUDY - STEEL GIRDER ALTERNATIVES -SUBSTRUCTURE

By: JRC   Date: 10/25/2006
Checked: MSL   Date: 10/30/2006

SUBSTRUCTURE

Alternative No.	Span Arrangement No. Spans	Lengths	Framing Alternative	Proposed Stringer Section	Pier Concrete Cost	Pier Reinforcing Cost	Abutment Concrete Cost	Abutment Reinforcing Cost	Pile Foundation Cost	MSE Abutment & Wingwall Cost	Temporary Shoring Cost	Subtotal Substructure Cost
1	1	215.66	4~ Welded Dog-legged Plate Girders	125" Web PG Grade 50W	$0	$0	$70,500	$23,100	$49,500	$2,635,200	$0	**$2,778,000**
2	1	233.43	4~ Welded Dog-legged Plate Girders	122" Web PG Grade 50W	$0	$0	$58,900	$19,300	$49,500	$2,854,800	$0	**$2,983,000**

COST SUPPORT CALCULATIONS

Pier QC/QA Concrete, Class QSC1 Cost: (HP-Pile)

Component	Alt 1 Volume (cu. yd.)	Alt 2 Volume (cu. yd.)	Year 2004	Annual Escalation	Year 2008	Alt 1 Total Cost	Alt 2 Total Cost
Cap	0	0	$421.00	3.5%	$483.00	$0	$0
Columns	0	0	$421.00	3.5%	$483.00	$0	$0
Footings	0	0	$421.00	3.5%	$483.00	$0	$0
Total Cost						$0	$0

Abutment QC/QA Concrete, Class QSC1 Cost:

Alternate 1

Component	Volume (cu. yd.)	Year 2004	Annual Escalation	Year 2008	Total Cost
Abutment	146	$421.00	3.5%	$483.00	$70,500
Wingwalls	0	$421.00	3.5%	$483.00	$0

Alternate 2

Component	Volume (cu. yd.)	Year 2004	Annual Escalation	Year 2008	Total Cost
Abutment	122	$421.00	3.5%	$483.00	$58,900
Wingwalls	0	$421.00	3.5%	$483.00	$0

Epoxy Coated 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 2004	Annual Escalation	Year 2008
Pier	$0.77	3.5%	$0.88
Abutment	$0.77	3.5%	$0.88

MSE Abutment Unit Cost ($/sq. ft.):

	Total Area (sq. ft.)	Year 2005 Unit Cost	Annual Escalation	Year 2008
Alt. 1	43,200	$55.00	3.5%	$61.00
Alt. 2	46,800	$55.00	3.5%	$61.00

Note: MSE wingwall lengths include full length required for ramp

Pile Foundation Unit Cost ($/ft.): *16" Dia. Piles, Furnished & Prebored holes*

	Number of Piles		Total Furn. Length	Total Prebored Length
Alt. 1	24	SEE QUANTITIES CALCULATION	840	540
Alt. 2	24	SEE QUANTITIES CALCULATION	840	540

Pile Foundation Unit Cost ($/ft.):

	Year 2005 Unit Cost	Annual Escalation	Year 2008
Furnished	$39.00	3.5%	$43.20
Prebored	$22.00	3.5%	$24.40
Total			$67.60

Shaft Foundation Unit Cost ($/ft.): 60" Drilled Shaft

	Number of Shafts		Total Shaft Length
Alt. 1	0	SEE QUANTITIES CALCULATION	0

Shaft Foundation Unit Cost ($/ft.):

Unit Cost	Escalation	2008
$300.00	4.5%	$358.00

Cost of Shafts: $ -

Temporary Shoring and Support Unit Costs ($/sq. ft.):

	Temp. Shoring Area (sq. ft.)	Temp. Girder Support (lump sum)
Alt. 1	0	$ -
Alt. 2	0	$ -

	Year 2004 Unit Cost	Annual Escalation	Year 2008
Temporary Shoring	$22.50	3.5%	$25.80
Cofferdam	$32.00	3.5%	$36.70

SCI-823-0.00 - PORTSMOUTH BYPASS

RAMP A (NB) OVER OHIO RIVER ROAD - US 52 INTERCHANGE,

STRUCTURE TYPE STUDY - STEEL GIRDER ALTERNATIVES - QUANTITY CALCULATIONS

By: JRC
Checked: MSL

Date: ########
Date: ########

Pier Quantities Alternate 1 (HP-Piles Type Foundation)																
Pier Location	Length	Cap				Column					Footing					Total Volume
		Width	Depth	Area	Volume	Width	Height	Area	# Column	Volume	Width	Depth	Area	# Footing	Volume	
Pier 1	0	0	0	0.00	0	0	0	0.00	0	0	0	0	0	0	0	0
Pier 2	0	0	0	0.00	0						0	0	0	0	0	0
Pier 3	0	0	0	0.00	0						0	0	0	0	0	0
Pier 4	0	0	0	0.00	0						0	0	0	0	0	0
Pier 5	0	0	0	0.00	0						0	0	0	0	0	0
Pier 6	0	0	0	0.00	0						0	0	0	0	0	0
Pier 7	0	0	0	0	0	0	0	0.00	0	0	0	4	0	0	0	0
Total (Cu.Ft.)					**0**					**0**					**0**	**0**
Total (Cu.Yd.)					**0**					**0**					**0**	**0**

Pile Quantities Alternate 1							
Location	Total Piles	Top Elev.	Bot Elev.	Furnished Length	Total Furn. Length (Ft)	Prebored Length	Total Prebored Length (Ft)
Rear Abut.	12	571.5	535.0	40.0	480	33	396
Fwd. Abut.	12	577	548	30.0	360	12	144
Total	**24**				**840**	**45**	**540**

Pier Quantities Alternate 2 (HP-Piles Type Foundation)																
Pier Location	Length	Cap				Column					Footing					Total Volume
		Width	Depth	Area	Volume	Width	Height	Area	# Column	Volume	Width	Depth	Area	# Footing	Volume	
Pier 1	0	0	0	0.00	0	0	0	0.00	0	0	0	0	0	0	0	0
Pier 2	0	0	0	0.00	0	0	0	0.00	0	0	0	0	0	0	0	0
Pier 3	0	0	0	0.00	0	0	0	0.00	0	0	0	0	0	0	0	0
Pier 4	0	0	0	0.00	0						0	0	0	0	0	0
Pier 5	0	0	0	0.00	0						0	0	0	0	0	0
Pier 6	0	0	0	0	0	0	0	0.00	0	0	0	0	0	0	0	0
Pier 7																0
Total (Cu.Ft.)					**0**					**0**					**0**	**0**
Total (Cu.Yd.)					**0**					**0**					**0**	**0**

Pile Quantities Alternate 2							
Location	Total Piles	Top Elev.	Bot Elev.	Furnished Length	Total Furn. Length (Ft)	Prebored Length	Total Prebored Length (Ft)
Rear Abut.	12	571.5	535.0	40.0	480	33	396
Fwd. Abut.	12	577	548	30.0	360	12	144
Total	**24**				**840**	**45**	**540**

Abutment Quantities -Alternate 1																
Abut Location	Length (feet)	Backwall				Beam Seat					Footing					Total Volume
		Width	Depth	Area	Volume	Width	Height	Area		Volume	Width	Depth	Area	# Footing	Volume	
Rear Abut	39.1	1.75	12.5	21.88	855	4.75	2	9.50		371	6.75	3	20.3	1	792	2019
Fwd. Abut	37.2	1.75	12.5	21.88	814	4.75	2	9.50		353	6.75	3	20.3	1	753	1920
Total (Cu.Ft.)					**1669**					**725**					**1545**	**3939**
Total (Cu.Yd.)					**62**					**27**					**57**	**146**

Superstructure Steel Quantities - Alt 1				
Location	Wt.of girder	# Girders	Span Length	Total Weight
Span 1	1078	4	218	940016
Span 2	0	0	0	0
Span 3	0	0	0	0
Span 4	0	0	0	0
Span 5	0	0	0	0
Span 6	0	0	0	0
Span 7	0	0	0	0
Span 8	0	0	0	0
Total				940016

Superstructure Steel Quantities - Alt 2				
Location	Wt.of girder	# Girders	Span Length	Total Weight
Span 1	706	4	235	664770
Span 2	0	0	0	0
Span 3	0	0	0	0
Span 4	0	0	0	0
Span 5	0	0	0	0
Span 6	0	0	0	0
Span 7	0	0	0	0
Span 8	0	0	0	0
Total				664770

MSE Abutment Wall Quantities Alt. 1					
Abut Location		Wall			
		Height	Length	Area	Volume
Rear Abut		20	39	780	
RA Wing (L)		18	956	17208	
RA Wing (R)		18	1020	18360	
Fwd Abut		22	39	858	
FA Wing (L)		28	131	3668	
FA Wing (R)		28	82	2296	
Total (Sq.Ft.)				**43200**	

Temporary Cofferdams					
Location		Wall			
		Height	Length	width	Area
Pier 3		0	0	0	0
Pier 4		0	0	0	0
Pier 5		0	0	0	0
Total (Sq.Ft.)					**0**

48" Drilled Shafts												
Location	Load/girder	# Girders	Total Load	Subst Wt (kips)	Pile Cap.(Kips)	No. Piles	Increase Factor	Total Shafts	Top Elev.	Bot Elev.	Pile Length	Total Shaft Length (Feet)
Rear Abut.	0	0	0		0	0	1.1	0	0	0	0.0	0
Pier 1	0	0	0	0	0	0	1.25	0	0	0	0.0	0
Pier 2	0	0	0	0	0	0	1.25	0	0	0	0.0	0
Pier 3	0	0	0	0	0	0	1.1	0	0	0	2.0	0
Pier 4	0	0	0	0	0	0	1.1	0	0	0	2.0	0
Pier 5	0	0	0	0	0	0	1.1	0	0	0	2.0	0
Pier 6	0	0	0	0	0	0	1.25	0	0	0	0.0	0
Pier 7	0	0	0	0	0	0	1.25	0	0	0	0.0	0
Fwd. Abut.	0	0	0	0	0	0	1.25	0	0	0	0.0	0
Total								**0**				**0**

Abutment Quantities - Alternate 2																
Abut Location	Length (feet)	Backwall				Beam Seat					Footing					Total Volume
		Width	Depth	Area	Volume	Width	Height	Area		Volume	Width	Depth	Area	# Footing	Volume	
Rear Abut	33	1.75	12.5	21.88	722	4.25	2	8.50		281	6.5	3	19.5	1	644	1646
Fwd. Abut	33	1.75	12.5	21.88	722	4.25	2	8.50		281	6.5	3	19.5	1	644	1646
Total (Cu.Ft.)					**1444**					**561**					**1287**	**3292**
Total (Cu.Yd.)					**53**					**21**					**48**	**122**

MSE Abutment Wall Quantities -Alt. 2					
Abut Location		Wall			
		Height	Length	Area	
Abut Location		**Wall**			
		Height	Length	Area	Volume
Rear Abut		21	34	714	
RA Wing (L)		20	943	18860	
RA Wing (R)		20	1020	20400	
Fwd Abut		23	34	782	
FA Wing (L)		30	131	3930	
FA Wing (R)		30	70	2100	
Total (Sq.Ft.)				**46800**	

Temporary Cofferdams					
Location		Wall			
		Height	Length	width	Area
Pier 3		0	0	0	0
Pier 4		0	0	0	0
Pier 5		0	0	0	0
Total (Sq.Ft.)					**0**

By: JRC Date: 10/25/2006
Checked: MSL Date: 10/30/2006

LIFE CYCLE MAINTENANCE COST

Alt. No.	Span Arrangement No. Spans	Lengths	Framing Alternative	Structural Steel Painting: Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost	Sealing: Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost	Approach Pavement Resurfacing: Cost Per Cycle	Number of Maintenance Cycles	Total Life Cycle Cost
1	1	215.66	4~ Welded Dog-legged Plate Girders	$387,000	2	$774,000	$101,248	2	$202,495	$0	0	$0
2	1	233.43	4~ Welded Dog-legged Plate Girders	$421,600	2	$843,200	$109,640	2	$219,280	$0	0	$0

Alt. No.	Span Arrangement No. Spans	Lengths	Framing Alternative	Bridge Deck Overlay (5): Deck Demo & Chipping	Deck Overlay	Deck Joint Gland	Number of Maintenance Cycles	Total Life Cycle Cost	Bridge Redecking (5): Deck Concrete Cost (3)	Deck Reinforcing Cost (3)	Deck Joint Cost	Deck Removal Cost	Number of Maintenance Cycles	Total Life Cycle Cost	Superstructure Life Cycle Maintenance Cost (1)	Total Initial Construction Cost	Total Relative Ownership Cost
1	1	215.66	4~ Welded Dog-legged Plate Girders	$21,600	$26,200	$6,759	1	$54,559	$168,800	$70,800	$26,121	$58,900	1	$324,621	**$1,356,000**	**$5,880,000**	**$7,236,000**
2	1	233.43	4~ Welded Dog-legged Plate Girders	$23,400	$28,300	$5,869	1	$57,569	$182,600	$76,600	$22,684	$63,800	1	$345,684	**$1,466,000**	**$5,730,000**	**$7,196,000**

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.)
Alt.1	125	4	215.66	29.00	24,226	20%	29,100
Alt. 2	122	4	233.43	32.00	26,455	20%	31,700

Painting Cost per sq. ft.:

	Year 2005	Annual Escalation	Year 2008
Prep.	$6.75	3.5%	$7.48
Prime	$1.75	3.5%	$1.94
Intermed.	$1.75	3.5%	$1.94
Finish	$1.75	3.5%	$1.94
Total			$13.30

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

	H	V	Diag.	No.	Total
Bot. Flange	26			1	26.00
		8		2	16.00
Lower Fillets	9	9	12.73	2	25.46
Web		46		2	92.00
Upper Fillets	3	3	4.24	2	8.49
	11	2	11.18	2	22.36
Top Flange		4		2	8.00
Total Exposed Perimeter					198.31 in.

36" AASHTO Type 2

	H	V	Diag.	No.	Total
Bot. Flange	18			1	18.00
		6		2	12.00
Lower Fillets	6	6	8.49	2	16.98
Web		15		2	30.00
Upper Fillets	3	3	4.24	2	8.49
Top Flange		6		2	12.00
Total Exposed Perimeter					97.47 in.

MSE WALLS
Alt. 1 = 4800
Alt. 2 = 5200

PS Concrete Area:

	No. Stringers	Total Span Length (ft.)	Nominal Exposed Beam Area (sq. ft.)	Secondary Member Allowance	Total Exposed Concrete Area (sq. yd.)
Alt. 3	0	0.00	0	10%	0

Sealing Cost per sq. yd.:

	Year 2004	Annual Escalation	Year 2008
Epoxy-Urethane Sealer	$9.68	3.5%	$11.11

Bridge Redecking:
Bridge Deck Joint Cost per foot:

	Year 2005	Annual Escalation	Year 2008
Structural Expansion Joint Including Elastomeric Strip Seal	$310.00	3.5%	$343.70

	Bridge Width	No. Joints
Alt. 1	38.00	2
Alt. 2	33.00	2

(?)

Bridge Deck Removal Cost:

	Deck Area (3) (sq. ft.)	Year 2008	Deck Removal Cost
Alt. 1	7,117	$8.28	$58,900
Alt. 2	7,703	$8.28	$63,800

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

	Year 2004	Annual Escalation	Year 2008
Micro Silica Modified Concrete Overlay Using Hydrodemolition (1.25" thick)	$25.58	3.5%	$29.35
Surface Preparation Using Hydrodemolition	$22.85	3.5%	$26.22
Hand Chipping	$37.07	3.5%	$42.54

Bridge Deck MSC Overlay Cost per cu. yd.:

Micro Silica Modified Concrete Overlay (Variable Thickness), Material Only	$144.00	3.5%	$165.24

	Deck Area (3) (sq. ft.)	Deck Area (sq. yd.)	Hand Chipping (sq. yd.)	Variable Thickness Repair (cu. yd.)
Alt. 1	7,117	791	20	18
Alt. 2	7,703	856	21	19

Assume 25% of deck area requires removal to depth of 4.5" (3.25" additional removal).

Bridge Deck Joint Gland Replacement Cost per foot:

	Year 2004	Annual Escalation	Year 2008
Elastomeric Strip Seal Gland	$77.50	3.5%	$88.93

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 (2008 construction year) dollars.
2. See Superstructure Cost sheet.
3. See Alternative Cost Summary sheet.
4. Assume bridge deck overlay at Year 25 and bridge deck replacement at Year 50.
Assume superstructures are painted or sealed on a 25-year recurrence interval.
Assume complete bridge replacement at Year 75.
5. Life cycle maintenance cost differences are assumed to be predominately a function of superstructure maintenance costs. Consequently, substructure lifecycle maintenance costs are not included in this analysis.

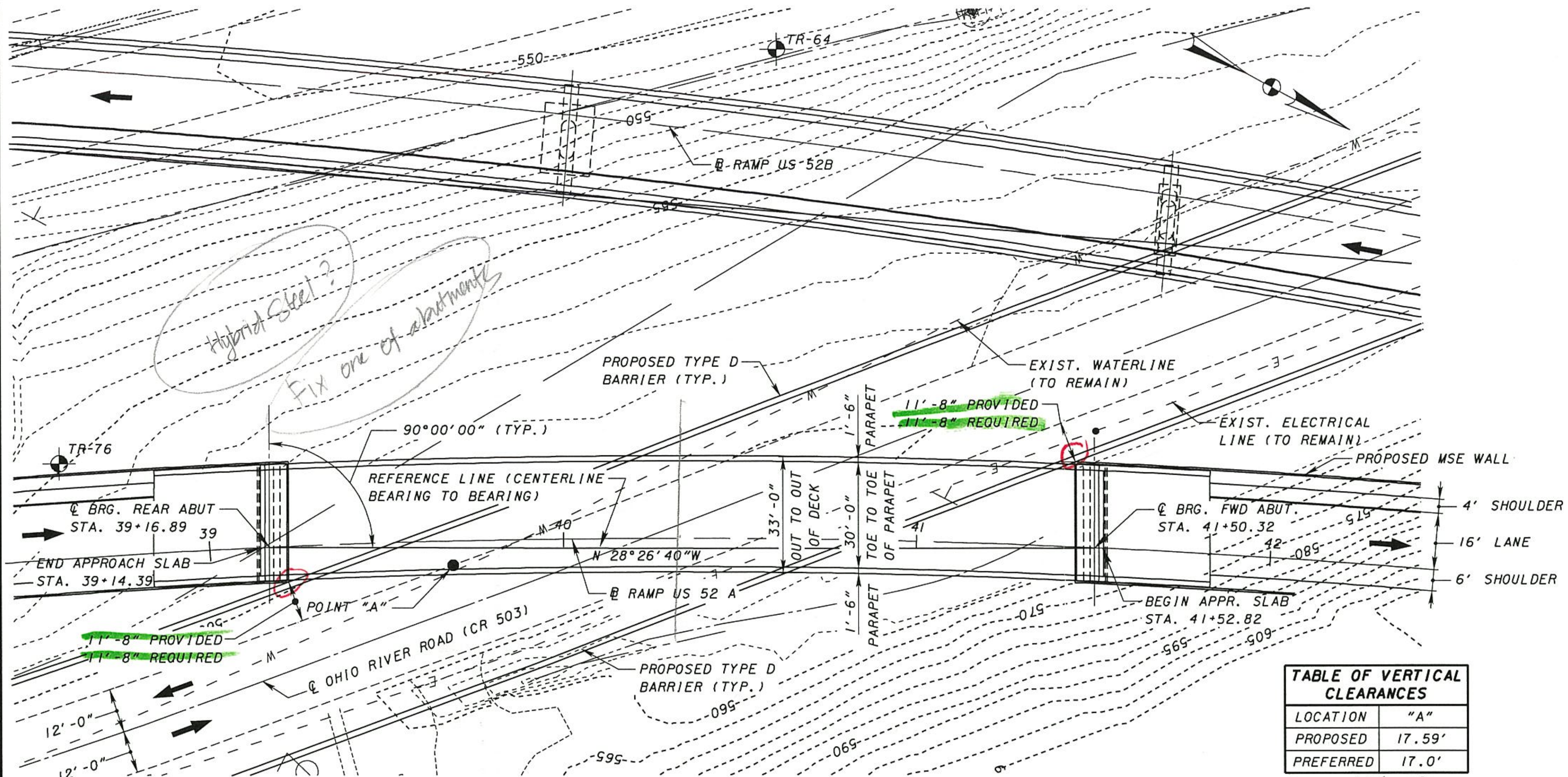
Approach Pavement Resurfacing:
Resurface Perpetual Asphalt Pavement:
Resurfacing Units Costs:

	Year 2004	Annual Escalation	Year 2008
Pavement Planing, Asphalt Concrete, per sq. yd. (Item 254)	$0.98	3.5%	$1.12
Asphalt Concrete Surface Course, per cu. yd.	$72.00	3.5%	$82.62

Asphalt Resurfacing Costs:

	Approach Roadway Length (ft.) (4)	Approach Roadway Width (ft.)	Resurfacing Area (sq. yd.)	Wearing Course Thickness (in.)	Wearing Course Volume (cu. yd.)
Alt. 1	0.0	33.0	0	1.50	0.0
Alt. 2	0.0	33.0	0	1.50	0.0

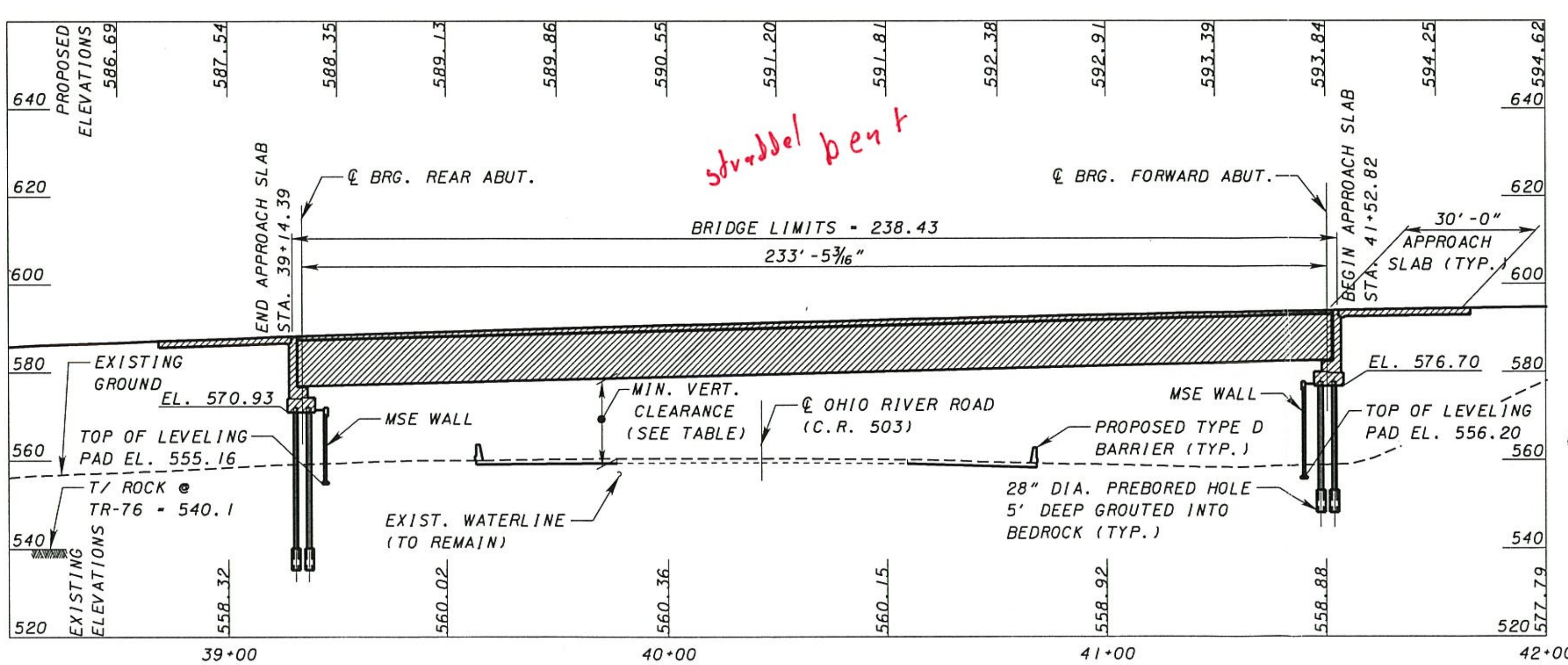
APPENDIX B
Preferred Alternative Site Plan and Details



PLAN

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	17.59'
PREFERRED	17.0'

Min. 16'-6"



ELEVATION ALONG ℄ RAMP US 52 A

HORIZONTAL CURVE DATA:

P.I. - STA. 44+40.37
Δ - 34°25'56.25'
Dc - 2°15'00"
R - 2546.48'
T - 789.05'
L - 1530.32'
E - 119.45'
e max - 0.056
P.C. - STA. 36+51.32
P.T. - STA. 51+81.64

900' VERT. CURVE DATA

P.V.I. - STA. 41+00.00
P.V.I. EL. - 599.46
G_1 - +4.95%
G_2 - -0.88%

ESTIMATED PILE LENGTHS
REAR ABUTMENT
FURNISHED LENGTH - 45'
DRIVEN LENGTH - 40'

ESTIMATED PILE LENGTHS
FORWARD ABUTMENT
FURNISHED LENGTH - 35'
DRIVEN LENGTH - 30'

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-62	43+02.06	93.34 LT.
TR-64	40+58.96	138.58 LT.
TR-76	38+59.24	27.09 LT.

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

TRAFFIC DATA

(ROUTE)
CURRENT YEAR ADT (2010) - 6,700
DESIGN YEAR ADT (2030) - 10,500
CURRENT YEAR ADTT (2010) - 938
DESIGN YEAR ADTT (2030) - 1,470

PROPOSED STRUCTURE

TYPE: SINGLE-SPAN, 122" WEB STEEL PLATE GIRDERS A709 GRADE 50W WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY STUB ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS

SPANS: 233'-5 3/16" C/C BEARINGS
ROADWAY: 30' TOE TO TOE OF PARAPETS
LOADING: HS-25 (CASE I) AND ALTERNATIVE MILITARY LOADING FWS=60 PSF
SKEW: 00°00'00" WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)
SUPERELEVATION: 0.056 FT/FT ACROSS LANE
ALIGNMENT: Dc - 2°15'00" CURVE TO THE RIGHT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)
LATITUDE:
LONGITUDE:

NOTES:

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

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED IN THE TS&L SUBMITTAL

FOUNDATION DATA:

ALL NEW PILES SHALL BE 16" DIA. CIP PILES AND HAVE A MAXIMUM CAPACITY OF 90 TONS

DESIGN AGENCY: TranSystems, 5747 PERIMETER DRIVE, SUITE 240, DUBLIN, OHIO 43017

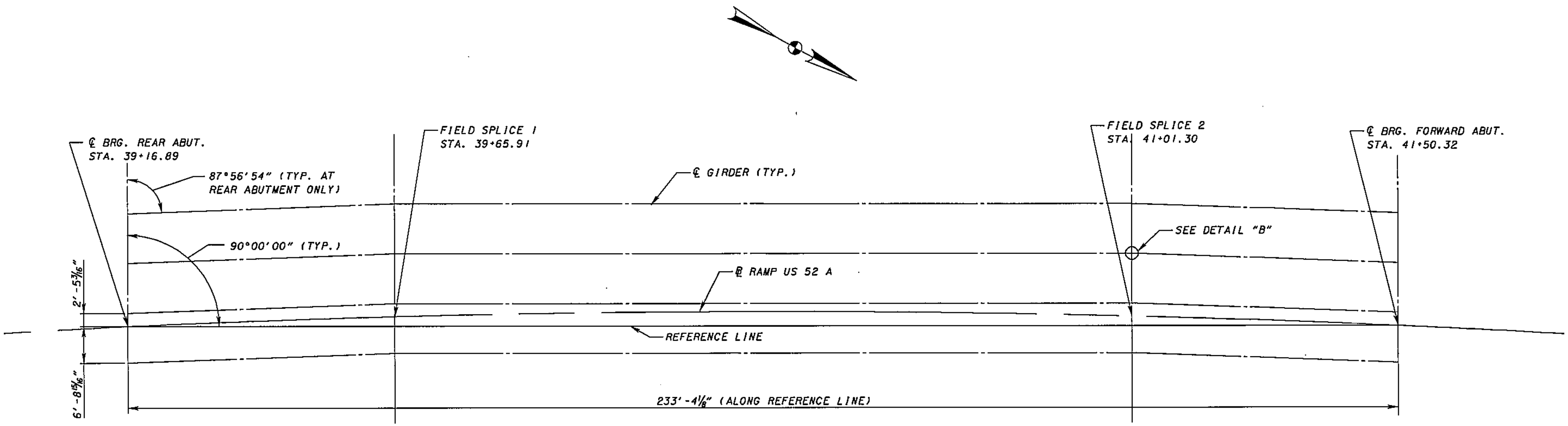
DESIGNED MSL | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED JRC | DATE 10/30/06 | STRUCTURE FILE NUMBER

SCIOTO COUNTY
STA. 39+14.39
STA. 41+52.82

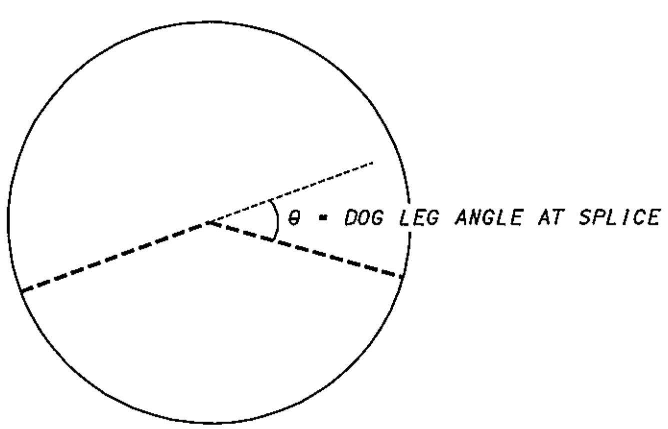
PRELIMINARY SITE PLAN - ALT. 2
BRIDGE US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 77366

1 / 4



FRAMING PLAN



DETAIL "B"

LOCATION	STATION	θ
℄ BRG. R. ABUT.	STA. 39+16.89	N/A
SPLICE 1	STA. 39+65.91	2.06°
SPLICE 2	STA. 41+01.30	2.06°
℄ BRG. FWD. ABUT.	STA. 41+50.32	N/A

FROM	TO	GIRDER LENGTH	GIRDER SPACING*
℄ BRG. R. ABUT	SPLICE 1	49.02′	3 SPACES @ 9′-2″± = 27.51′
SPLICE 1	SPLICE 2	135.37′	3 SPACES @ 9′-2″± = 27.53′
SPLICE 2	℄ BRG. FWD. ABUT	49.02′	3 SPACES @ 9′-2″± = 27.51′

* GIRDER SPACINGS ARE NORMAL TO GIRDER CENTERLINE

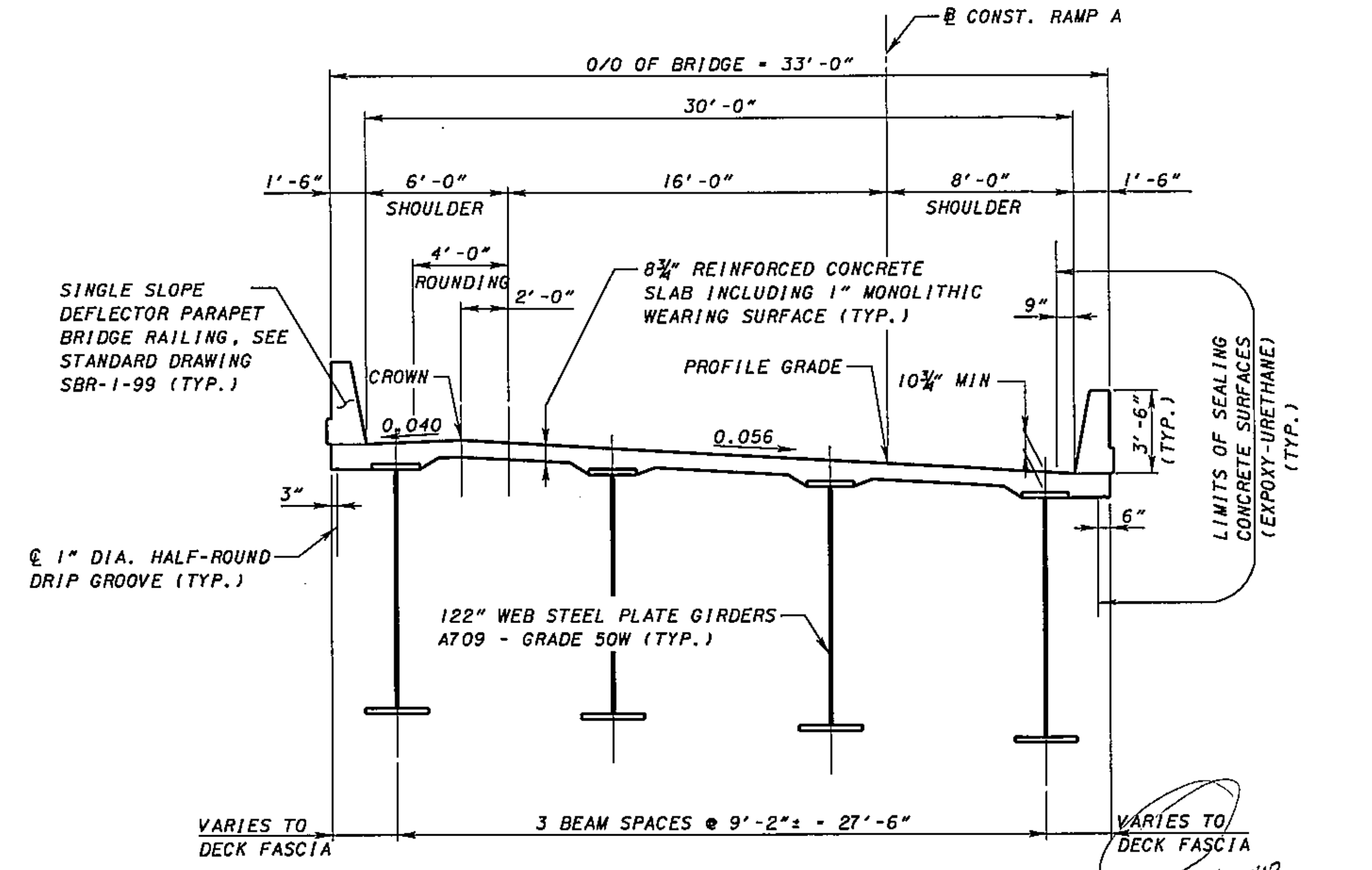
DESIGN AGENCY: TranSystems

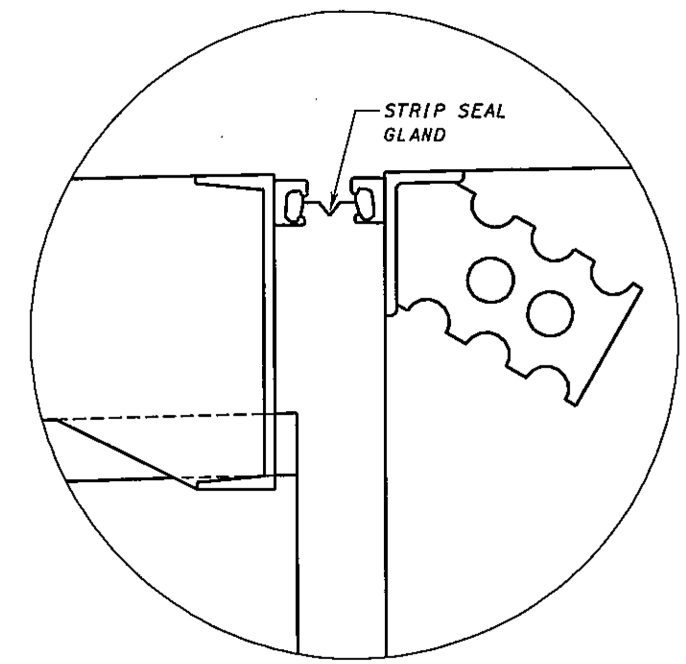
DESIGNED MSL | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED JRC | DATE 10/30/06 | STRUCTURE FILE NUMBER

FRAMING PLAN - ALTERNATIVE 2
BRIDGE NO. US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 77366

2/4

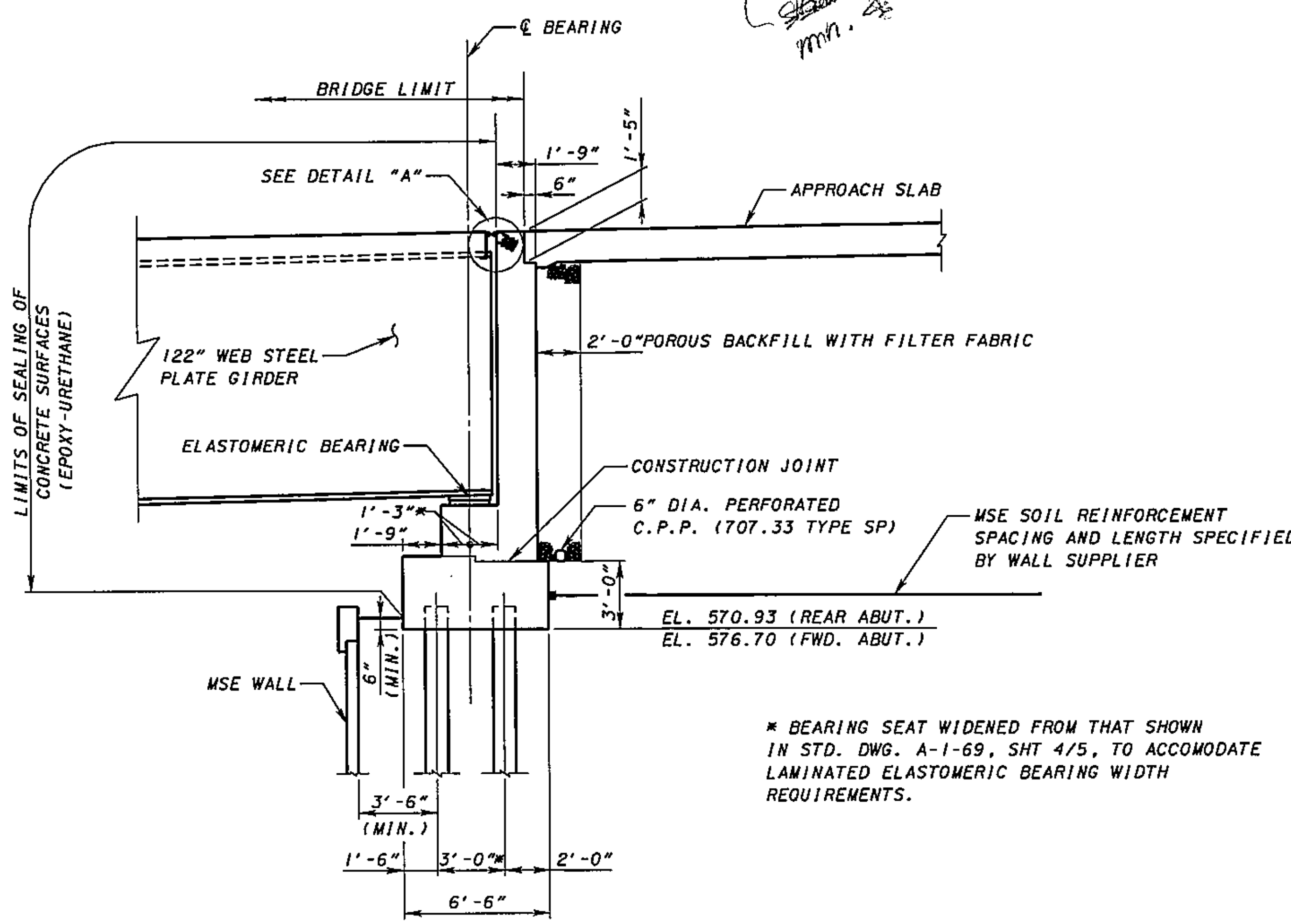




DETAIL "A"

REFER TO STANDARD DRAWING EXJ-4-87

SUPERSTRUCTURE DEPTH	
ITEM	122" WEB STEEL PLATE GIRDER A709 GRADE 50W
SLAB (INCLUDING WEARING SURFACE)	8¾"
HAUNCH (BOTTOM OF SLAB TO TOP OF FLANGE)	2"
GIRDER DEPTH	124.9375"
TOP OF WEARING SURFACE TO BOTTOM OF GIRDER FLANGE (INCH)	135.6875"
TOP OF WEARING SURFACE TO BOTTOM OF GIRDER FLANGE (FEET)	11.31'



* BEARING SEAT WIDENED FROM THAT SHOWN IN STD. DWG. A-1-69, SHT 4/5, TO ACCOMODATE LAMINATED ELASTOMERIC BEARING WIDTH REQUIREMENTS.

TYPICAL ABUTMENT SECTION

DESIGN AGENCY: TranSystems

DESIGNED	DRAWN	REVIEWED	DATE
MSL	MTN	JRC	10/30/06
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP			

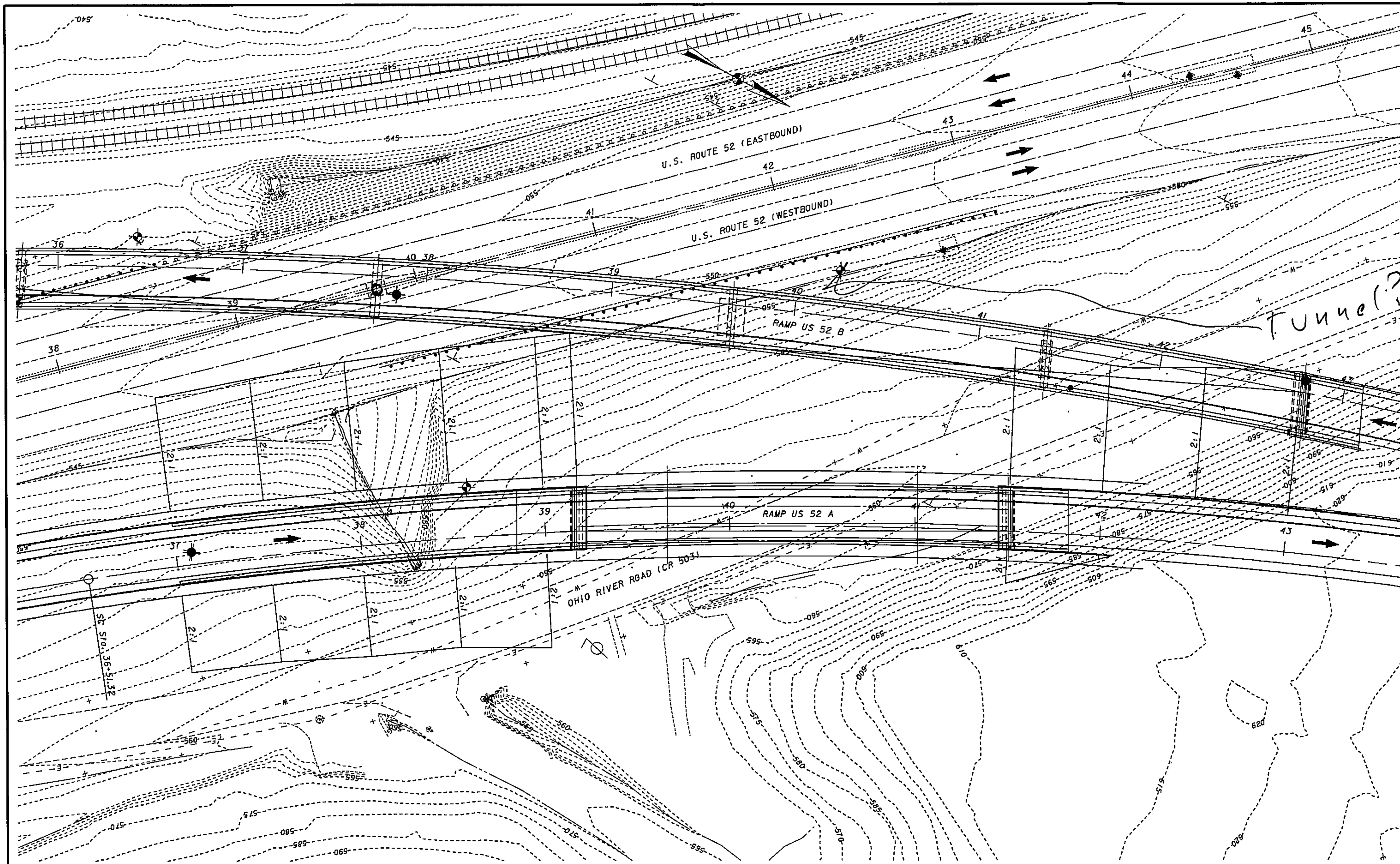
TYPICAL TRANSVERSE SECTION - ALTERNATIVE 2

BRIDGE NO. US-52-XXXX

US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00

PID 77366



THIS SHOWS THE LIMITS FOR 2:1 FILL AT RAMP A IF THERE WERE NO MSE WALLS ALONG THE SIDES OF THE RAMP. TOES OF 2:1 EMBANKMENTS ENCROACH/INTERFERE WITH U.S. 52 WESTBOUND AND OHIO RIVER ROAD.

DESIGN AGENCY TranSystems

DESIGNED	DRAWN	REVIEWED	DATE
MSL	MTN	JRC	10/30/06
CHECKED	REVISED	STRUCTURE FILE NUMBER	
PJP			

FILL LIMITS INVESTIGATION

BRIDGE NO. US-52-XXXX

US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00

PID 77366

4/4

APPENDIX C
Vertical Clearance Calculations

TranSystems

TranSystems

Made By	MSL	Date	10/31/06	Job No.	P403030064
Checked By	MTN	Date	10/31/06	Sheet No.	

VERTICAL CLEARANCE CALCULATIONS

Job Name *SCI-823-0.00* **Structure**

Description *S.R. 823 Ramp A OVER US 52* **PID #** *77366*

Alternative 1 - 4-125" Steel Plate Girders | ***Point Location:*** ***A***

Adjstment for Cross Slope

Comment	Grade		Offset (from PGL)	
Profile grade line to critical pt.:	-0.056	x	6.47	-0.36232
			Total Adjustment =	**-0.36**

Superstructure Depth

Comment	Depth (in)	Depth (ft)
Deck Thickness:	8.75	0.73
Haunch:	2	0.17
Girder or Beam Depth:	130.625	10.89
	141.375	11.79

Total Superstructure Depth (ft) = **11.79**

Vertical Clearance at Critical Point

Station @ Critical Point	=	**39+70.04**
Offset Location @ Critical Point	=	**6.47' Rt.**
Profile Grade Elevation at Critical Point	=	589.71
Adjustment for Cross Slopes to Beam CL	=	-0.36
Top of Deck Elevation @ Critical Point	=	**589.35**
Total Superstructure Depth	=	-11.79
Bottom of Beam Elevation @ Critical Point	=	**577.56**
Top of Pavement @ Critical Point	=	**560.40**
Actual Vertical Clearance	=	***17.16***
Preferred Vertical Clearance	=	*17.0*
Required Vertical Clearance	=	*16.5*

TranSystems

Made By	MSL	Date	10/31/06	Job No.	P403030064
Checked By	MTN	Date	10/31/06	Sheet No.	

VERTICAL CLEARANCE CALCULATIONS

Job Name SCI-823-0.00 Structure

Description S.R. 823 Ramp B OVER US 52 PID # 77366

Alternatives 2 - 4-122" Steel Plate Girders | **Point Location: A**

Adjstment for Cross Slope

Comment	Grade		Offset (from PGL)	
Profile grade line to critical pt.:	-0.056	x	6.84	-0.38304
			Total Adjustment =	**-0.38**

Superstructure Depth

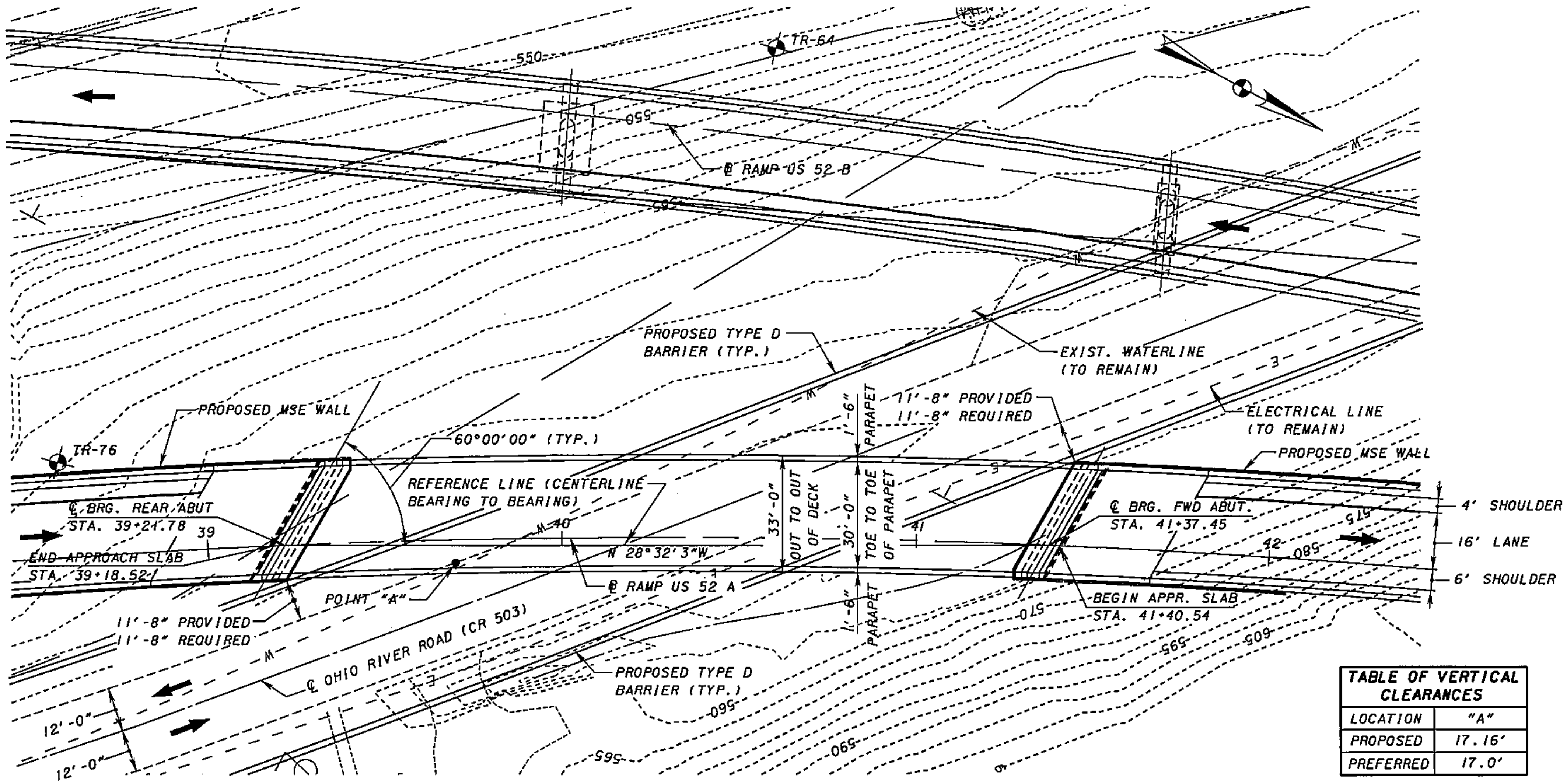
Comment	Depth (in)	Depth (ft)
Deck Thickness:	8.75	0.73
Haunch:	2	0.17
Girder or Beam Depth:	124.9375	10.41
	135.6875	11.31

Total Superstructure Depth (ft) = **11.31**

Vertical Clearance at Critical Point

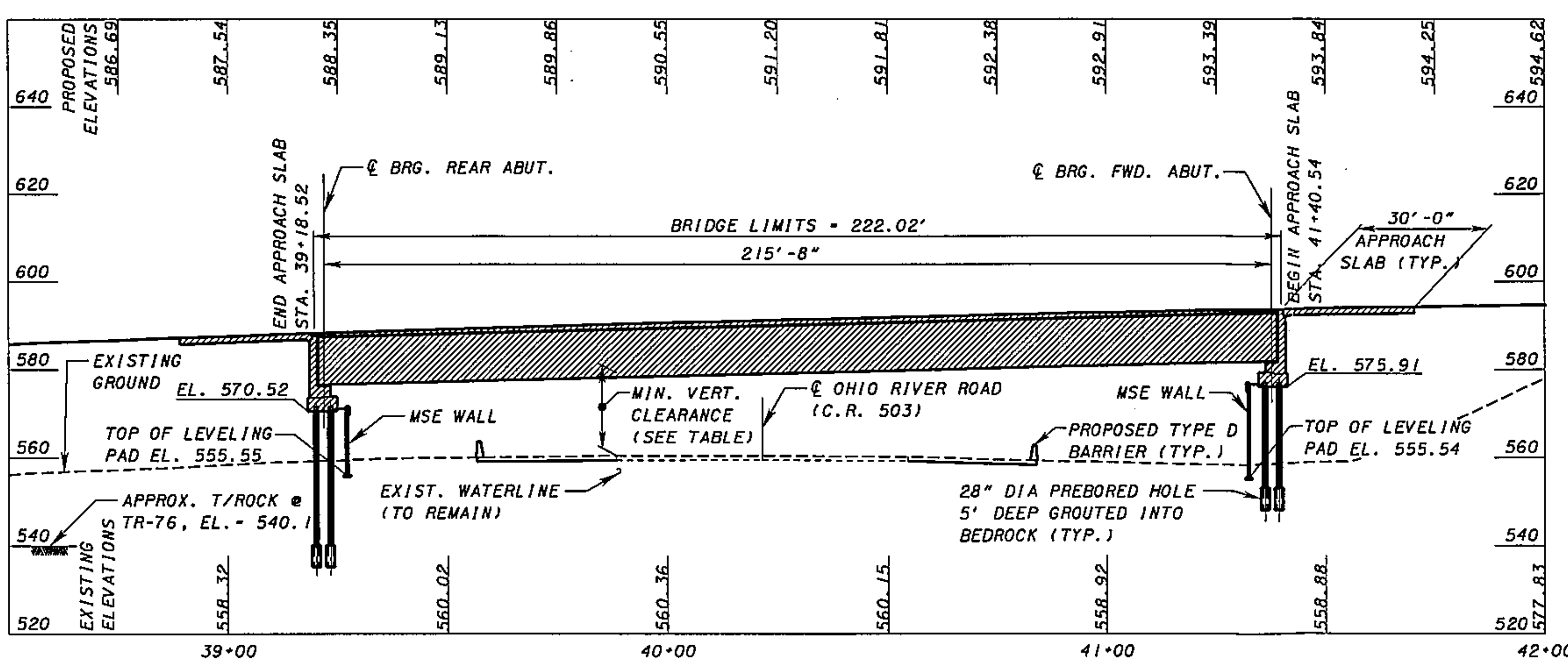
Station @ Critical Point	=	**39+68.95**
Offset Location @ Critical Point	=	**6.84' Rt.**
Profile Grade Elevation at Critical Point	=	589.68
Adjustment for Cross Slopes to Beam CL	=	-0.38
Top of Deck Elevation @ Critical Point	=	**589.30**
Total Superstructure Depth	=	-11.31
Bottom of Beam Elevation @ Critical Point	=	**577.99**
Top of Pavement @ Critical Point	=	**560.40**
Actual Vertical Clearance	=	***17.59***
Preferred Vertical Clearance	=	*17.0*
Required Vertical Clearance	=	*16.5*

APPENDIX D
Preliminary Structure Site Plan



PLAN

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	17.16'
PREFERRED	17.0'



ELEVATION ALONG ℄ US 52 A

HORIZONTAL CURVE DATA:

P.I. = STA. 44+40.37
Δ = 34°25'56.25'
Dc = 2°15'00"
R = 2546.48'
T = 789.05'
L = 1530.32'
E = 119.45'
e_{max} = 0.056
P.C. = STA. 36+51.32
P.T. = STA. 51+81.64

900' VERT. CURVE DATA

P.V.I. = STA. 41+00.00
P.V.I. EL. = 599.46
G_1 = +4.95%
G_2 = -0.88%

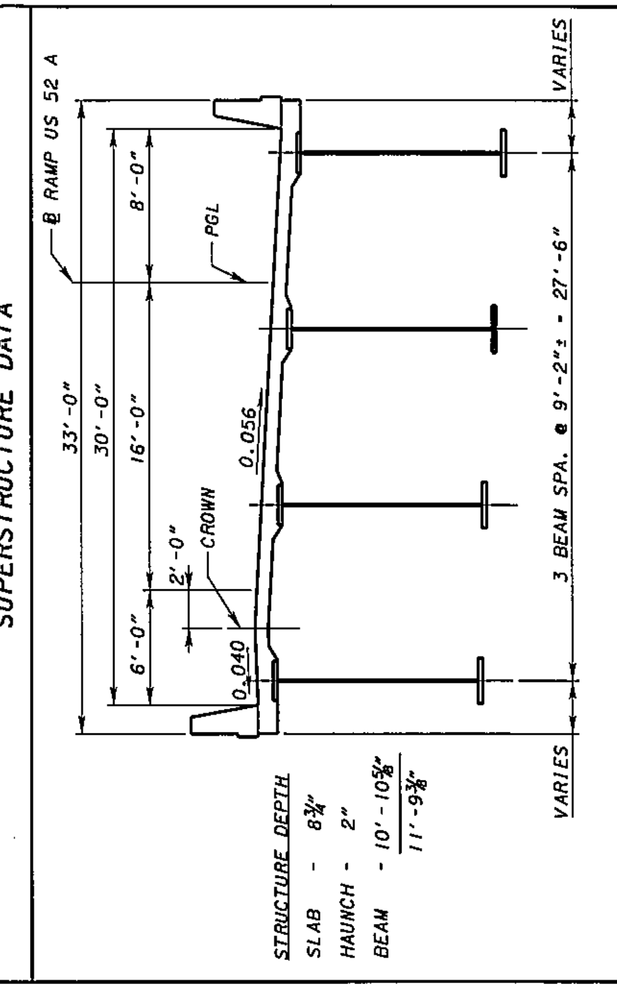
ESTIMATED PILE LENGTHS REAR ABUTMENT

FURNISHED LENGTH - 45'
DRIVEN LENGTH - 40'

ESTIMATED PILE LENGTHS FORWARD ABUTMENT

FURNISHED LENGTH - 35'
DRIVEN LENGTH - 30'

SUPERSTRUCTURE DATA



STRUCTURE DEPTH
SLAB - 8¾"
HAUNCH - 2"
BEAM - 10'-10⅝" / 11'-9⅞"

PROPOSED STRUCTURE

TYPE: SINGLE SPAN, 125" WEB STEEL PLATE GIRDERS A709 GRADE 50W WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY STUB ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS

SPANS: 215'-8" C/C BEARINGS
ROADWAY: 30' TOE TO TOE OF PARAPETS
LOADING: HS-25 (CASE I) AND ALTERNATIVE MILITARY LOADING FWS-60 PSF
SKEW: 30°00'00" LF WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)
SUPERELEVATION: 0.056 FT/FT ACROSS LANE
ALIGNMENT: Dc = 2°15'00" CURVE TO THE RIGHT
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)
LATITUDE:
LONGITUDE:

NOTES:

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

UTILITIES:

UTILITIES DISPOSITION WILL BE ADDRESSED IN THE TS&L SUBMITTAL

FOUNDATION DATA:

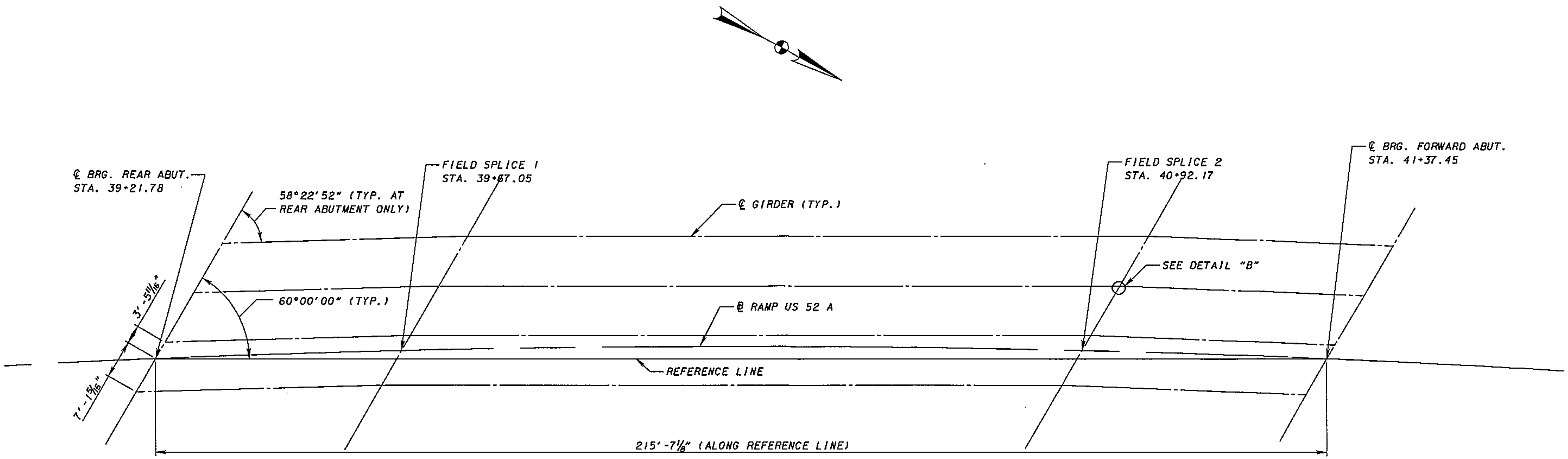
ALL NEW PILES SHALL BE 16" DIA. CIP PILES AND HAVE A MAXIMUM CAPACITY OF 90 TONS

DESIGN AGENCY
TranSystems
DESIGNED MSL | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED JRC | DATE 10/30/06 | STRUCTURE FILE NUMBER

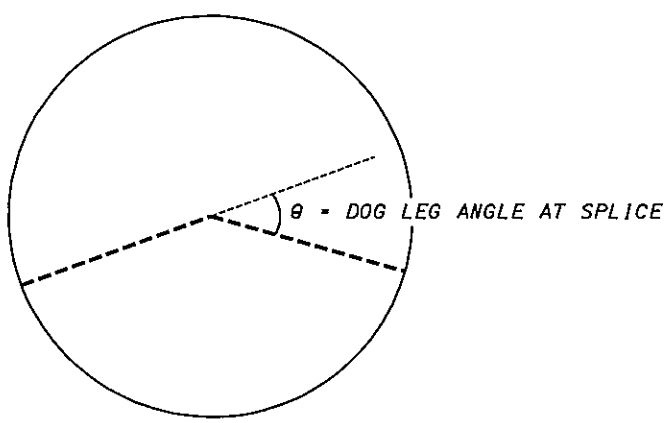
SCIOTO COUNTY
STA. 39+18.52
STA. 41+40.54

PRELIMINARY SITE PLAN - ALT. 1
BRIDGE NO. US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 77366



FRAMING PLAN



DETAIL "B"

LOCATION	STATION	θ
℄ BRG. R. ABUT.	STA. 39+21.78	N/A
SPLICE 1	STA. 39+67.05	1.62°
SPLICE 2	STA. 40+92.17	2.27°
℄ BRG. FWD. ABUT.	STA. 41+37.45	N/A

FROM	TO	GIRDER LENGTH	GIRDER SPACING*
℄ BRG. R. ABUT	SPLICE 1	45.12'	3 SPACES @ 9'-0 7/16"± = 27.04'
SPLICE 1	SPLICE 2	125.11'	3 SPACES @ 9'-2"± = 27.50'
SPLICE 2	℄ BRG. FWD. ABUT	45.12'	3 SPACES @ 9'-4 7/16"± = 28.11'

* GIRDER SPACINGS ARE NORMAL TO GIRDER CENTERLINE

DESIGN AGENCY TranSystems

DESIGNED MSL | CHECKED PJP | DRAWN MTN | REVISED | REVIEWED JRC | DATE 10/30/06 | STRUCTURE FILE NUMBER

FRAMING PLAN - ALTERNATIVE 1

BRIDGE NO. US-52-XXXX

US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00

PID 77366

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APPENDIX E
Preliminary Subsurface Exploration
Bridge and MSE Retaining Walls

Report of:

Preliminary Subsurface Exploration
Bridge and MSE Retaining Walls
US 52 Ramp A to Northbound SR 823
SCI-823-0.00 Portsmouth Bypass
Scioto County, Ohio

Prepared for:

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

Ohio Department of Transportation
District 9

DLZ Ohio, Inc.
6121 Huntley Road
Columbus, OH 43229
Phone: (614) 888-0040
Fax: (614) 436-0161

DLZ Job No. 0121-3070.03

October 16, 2006

Prepared by:

DLZ

REPORT

OF

PRELIMINARY SUBSURFACE EXPLORATION

FOR

BRIDGE AND MSE RETAINING WALLS

US 52 RAMP A TO NORTHBOUND SR 823

SCI-823-0.00 PORTSMOUTH BYPASS

SCIOTO COUNTY, OHIO

For:

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

By:

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

DLZ Job. No. 0121-3070.03

October 16, 2006

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REPORT
OF
PRELIMINARY SUBSURFACE EXPLORATION
FOR
BRIDGE AND MSE RETAINING WALLS
US 52 RAMP A TO NORTHBOUND SR 823
SCI-823-0.00 PORTSMOUTH BYPASS
SCIOTO COUNTY, OHIO

1.0 INTRODUCTION

This report includes the findings of the preliminary subsurface exploration, and the engineering evaluation of the foundation and mechanically stabilized earth (MSE) retaining walls for Ramp A of the US 52 interchange. The findings included in this report pertain to US 52 Ramp A to northbound SR 823 only. The findings of other structure evaluations will be submitted in separate documents.

The project consists in part of placing a bridge ramp structure for proposed US 52 over Ohio River Road (CR 503). The structure as planned, is a single-span structure using MSE walls to hold back the roadway embankments and contain the abutments.

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

The findings and recommendations presented in this report should be considered preliminary. After the bridge and ramp designs are refined, it will be necessary to drill additional borings in the area of the proposed structures in accordance with ODOT's specifications for subsurface investigations in order to finalize the MSE wall and foundation evaluations.

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

2.0 GENERAL PROJECT INFORMATION

It is understood that the plan location of the bridge structure for the proposed US 52 Ramp A over Ohio River Road (CR 503) has not changed from the approved location, as shown on the plan and profile drawing in Appendix I. It is understood that the MSE walls will be placed using two alternatives to hold back the roadway embankment for the proposed US 52 Ramp A. The first alternative (Alternative 1) involves placing the rear abutment of the MSE wall at station 39+20.60 and the forward abutment of the MSE wall at station 41+38.57. The second alternative (Alternative 2) involves placing the rear abutment of the MSE wall at station 39+16.89 and the forward abutment of the MSE wall at station 41+50.32. Through discussions with TranSystems, it is known that a structure using a pier may be considered to shorten the span lengths. Considering this, section 5.1.2 provides information for this element.

Based upon the structure plan and profile drawing, it is assumed that the maximum height of the embankment/MSE wall at the rear and forward abutments for both alternatives will be approximately 28.2 and 34.0 feet, respectively. Those heights are based upon the maximum difference between the proposed grade of US 52 Ramp A and the approximate existing grade at the site as indicated on the structure plan and profile drawing.

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

3.0 FIELD EXPLORATION

The field exploration consisted in part of two preliminary structural borings (TR-62 and TR-76). The borings were drilled between March 18 and 30, 2005. A boring plan is presented in Appendix I. Boring logs for borings TR-62 and TR-76 are presented in Appendix II. Information concerning the drilling procedures is also presented in Appendix II.

The boring locations were determined by representatives of DLZ. The surveyed locations and ground surface elevations of the borings (TR-62 and TR-76) were determined by representatives from Lockwood, Lanier, Mathias & Noland, Inc. (2LMN).

4.0 FINDINGS

4.1 Geology of the Site

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

The genesis of the soils varies across the site. Soils in the floodplain consist primarily of alluvium and alluvial terraces, generally composed of silty clay, coarse sand, gravel, and cobbles. Below approximately elevation 700, the soils on the hillsides are generally lacustrine deposits. Lacustrine soils in this area are commonly known as "Minford Silts" or the Minford Complex. These deposits were formed during the early to middle Pleistocene age when the northward flowing Teays River system was blocked by the southward advance of the Kansan aged ice sheets. As the glaciers advanced, the course of the Teays River was blocked south of Chillicothe and a large lake was formed from the impoundment of the waterways. As a result of the impoundment, vast quantities of sediments were deposited ranging from 10 to 80 feet in thickness, thinning towards the margins. Bedrock within the structure area is primarily sandstone of the Logan

Formation of Mississippian age. Bedrock of the Pennsylvanian Breathitt Formation can be found at the top of the slopes to typically above approximately elevation 770.

4.2 Subsurface Conditions

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

4.2.1 Soil Conditions

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

Boring TR-76 encountered 2 inches of topsoil underlain by natural cohesive soils. Borings TR-62 and TR-76 encountered natural cohesive soil deposits below the ground surface. The natural cohesive deposits consisted of very stiff to hard sandy silt (A-4a) and hard silt and clay (A-6a) and extended below the ground surface to approximate depths ranging between 3 and 8 feet, corresponding to approximate elevations between 547.1 and 556.1. The cohesive soil deposits in boring TR-76 were underlain by natural granular soils consisting of medium dense to dense sandy silt (A-4a) to an approximate depth of 8 feet, corresponding to elevation 540.1 where it was underlain by rock. In boring TR-62, rock was encountered below the cohesive soil deposit at an approximate elevation of 556.1.

4.2.2 Bedrock Conditions

In the area of the proposed structure, bedrock was encountered in both borings below the natural soil deposits. The bedrock consisted of medium hard to hard, slightly to highly weathered sandstone. The amount of rock recovered in each core was 100 percent. The rock quality designation (RQD) of the bedrock ranged between 64 and 78 percent with an average of 71 percent indicating fair to good rock.

Unconfined compressive strength of tested cores ranged between 10,794 and 11,036 pounds per square inch (psi). The tested cores correspond to samples at depths between 3.5 feet and 18.5 feet below the ground surface. A summary of the unconfined compressive strength of the tested cores is shown in Table 1, on the following page.

Table 1-Unconfined Compressive Strength Results

Boring	Depth (ft)	Elevation	Unconfined Compressive Strength (psi)
TR-62	9.3-9.7	549.0-549.8	10,794
TR-76	19.6-20.0	535.0-535.5	11,036

4.2.3 Groundwater Conditions

Seepage was not encountered in either of the borings drilled for this structure. There were also no measurable water levels in the borings prior to rock coring. Water was used during rock coring and masked any seepage zones that might exist in the rock. Measurable water levels were present in both borings upon the completion of coring (includes drill water) between approximate depths of 1.9 and 4.0 feet.

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

5.0 CONCLUSIONS AND RECOMMENDATIONS

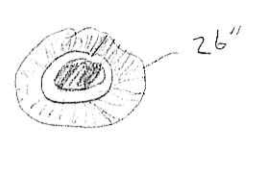
It is anticipated that the proposed ramp will be constructed as described in Sections 1 and 2 of this report. At this time, it is not known what foundation type would be used to support the abutments. Recommendations for spread footings, drilled shafts, and pipe piles are included for the support of the abutments. Additionally, through discussions with TranSystems Corporation, it is understood that an alternative is being evaluated which considers adding a pier to support the structure. Drilled shaft and spread footing foundation recommendations are also included for this pier. The site is well suited for the use of MSE walls to contain the abutments and hold back the roadway embankment. Recommendations for foundations and MSE walls are presented in the following sections.

5.1 Bridge Foundation Recommendations

5.1.1 Rear and Forward Abutments

No recommendations regarding pipe pile size! Why?

It is understood through previous communications with the ODOT Office of Structural Engineering (OSE) that pipe piles can be used to support the abutments. This foundation alternative includes supporting the abutments by steel pipe piles placed in prebored holes 12 inches larger than the diameter of the pile and 5 feet deep into bedrock. After installing the steel pipe pile in the prebored hole, grout or cement should be placed in the void area around the pile in the prebored hole prior to constructing the embankment granular fill (per OSE). Therefore, a pile sleeve may not be required for the installation of the piles. However, consideration should be given to the use of pile sleeves to mitigate down drag effects from compaction and to protect the pile during the embankment



and MSE wall construction. The allowable pile capacity, as per ODOT BDM 202.2.3.2.b, may be utilized in this configuration. Excessive lateral loading and uplift is not anticipated to be a concern at this site. However, if these forces are determined to be significant, longer socket lengths may be required.

Due to the relatively small rigidity of the steel pipe piles compared to drilled shafts, the steel pipe piles are anticipated to provide low lateral resistance to lateral earth pressures that can be induced in high embankment fills. Therefore, the prebored and socketed steel pipe pile foundation system may be a concern if significant lateral loads are present.

Drilled shafts may also be considered for the support of the abutments. Due to the large amount of embankment fill, it appears that drilled shafts socketed a minimum of 5 feet into competent rock will be well suited for the support of the proposed structural abutments. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 kips per square foot (ksf) or (40 tsf).

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

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 pounds per square foot (psf) may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance ignoring any end bearing.

Precautions should be taken to permit the shafts to be drilled and the concrete placed under relatively dry conditions. Although the borings did not encounter significant seepage, water could flow into the drilled shafts during installation particularly from seepage zones and wet zones not encountered in the borings that may be present in the rock or soil. It should be anticipated that materials across the site could vary considerably and temporary casing will be required during the drilling and concrete placement to seal out water seepage in the overburden and prevent cave-in. During simultaneous concrete placement and casing removal operations, sufficient concrete should be maintained inside the casing to offset the hydrostatic head of any groundwater. Extreme care must be exercised during concrete placement and removal of the casing so that soil intrusion is avoided.

Additionally, spread footings bearing in the MSE wall fill may also be considered to support the abutments. As per the Bridge Design Manual 204.6.2.1, an allowable bearing capacity of 4 kips per square foot (ksf) may be used to design the footings.

5.1.2 Piers

Spread footings can be constructed on the rock encountered by the borings to support the piers. Competent bedrock was generally encountered within two to three feet of the soil-rock interface. Spread footings bearing on competent bedrock may be designed using an allowable bearing capacity of 80 ksf (40 tsf).

Currently, lateral loading and uplift is not anticipated to be a concern at this site. However, if spread footings cannot be used at the piers, drilled shafts may be considered to support the piers. If drilled shafts are used to support the foundation of the piers, a minimum of 5-foot deep socket into competent rock is required. The drilled shafts should be straight (not belled) and may be designed based on an allowable bearing pressure of 80 kips per square foot (ksf) or (40 tsf).

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

If adequate capacity cannot be developed with reasonable shaft diameter, consideration should be given to the use of deeper rock sockets. Neglecting the upper two feet of the socket, allowable sidewall shear stress/adhesion of 7,500 pounds per square foot may be used. If deeper sockets are used, the shafts should be designed such that design loads are carried entirely by the socket resistance ignoring any end bearing.

Precautions should be taken to ensure appropriate drilled shaft construction practices are followed. See section 5.1.1 for more information.

Table 2, on the following page, summarizes the site conditions and foundation recommendations. It should be noted that the bedrock surface varies widely across the project area. The approximate bearing elevations presented below indicate the elevations at the boring locations only. Variations in the elevation at which competent bedrock is encountered should be anticipated.

Table 2-Summary of Foundation Recommendation

Structural Element	Boring	Existing Ground Surface Elevation (Feet)	Foundation Type	Approximate Bearing Elevation (Feet)	Allowable Bearing Capacity
Rear Abutment	TR-76	551.1	Pipe Piles	535.1 *	Pile Capacity^{+}
			Drilled Shafts	535.1 *	80 ksf^{++}
			Spread Footings	MSE Fill	4 ksf
Pier	TR-62/ TR-76	551.1-559.1	Spread Footings	535.1-548.1	80 ksf
			Drilled Shafts	535.1-548.1 *	80 ksf^{++}
Forward Abutment	TR-62	559.1	Pipe Piles	548.1 *	Pile Capacity^{+}
			Drilled Shafts	548.1 *	80 ksf^{++}
			Spread Footings	MSE Fill	4 ksf

* *Includes 5-foot socket into competent rock.*

$^{+}$ *Pile capacity should conform to ODOT BDM 202.2.3.2.*

$^{++}$ *End bearing capacity only.*

5.2 Mechanically Stabilized Earth (MSE) Retaining Wall Recommendations

It is understood that MSE walls would be used to construct the embankments and contain the abutments. Recommendations for the MSE wall are presented in the following sections. The MSE wall should be constructed per the recommendations presented in this report and in conformance with the manufacturer's specifications.

5.2.1 MSE Walls: General Information

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

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

Calculations for bearing capacity, sliding, and overturning as well as the results of the global stability analyses are attached. Other external and internal stability analyses are required for the design of an MSE wall, but are considered outside the scope of this report. The parameters required to perform the stability analyses are presented in Table 3, on the following page. In accordance with ODOT guidelines, a unit weight of 120 pcf and a friction angle of 34 degrees were selected for the backfill material in the reinforced zone. Similarly, the fill material used to construct the roadway embankments is assumed to have a unit weight of 120 pcf and a friction angle of 30 degrees. If the embankment fill material or backfill material for the reinforcing zone has properties significantly different from these values, DLZ should be informed so that the analyses may be revised as necessary.

Table 3, Soil Parameters Used in The MSE Wall Stability Analyses

Zone	Soil Type	Unit Weight (pcf)	Strength Parameters			
			Undrained		Drained	
			c	φ	c'	φ'
Reinforced Fill	Compacted Granular Fill	120	0	34	0	34
Retained Soil	Compacted Embankment Fill	120	0	30	0	30
Foundation Soil (Rear and Forward Abutments) (Borings TR-62&76)	Stiff to Hard Sandy Silt (A-4a)	120	3500	0	0	29

5.2.2 MSE Wall Evaluations and Recommendations

The rear abutment location was analyzed at this structure location due to the relatively thick soil overburden cover over the rock. Due to the close proximity of the rock at the forward abutment, the foundation of the MSE wall is anticipated to be bearing on rock or near bedrock, and hence the bearing capacity and the sliding of the wall are not of concern. The proposed embankment in both alternatives is slightly higher at the forward abutment location than at the rear abutment. It should be noted that variations may be found in borings drilled for the final design that may change the results of the analyses. → based on boring TR-62

Analyses for the MSE walls bearing on the native soils at this location yielded factors of safety above the minimum recommended values for undrained and drained global stability, as well as stability (sliding and overturning) and undrained and drained bearing capacity. Consequently, it is recommended that MSE walls be built in this area using a minimum embedment of 3.5 feet, unless bedrock is encountered at a shallower depth. If founded on bedrock, no embedment into the rock is required.

Due to the currently proposed location of the forward abutment, significant rock excavation should be anticipated to accommodate the MSE wall reinforcing straps.

The stability analysis of the MSE wall was based on the assumption that the top 8 feet of the native soil along the MSE wall consists of natural cohesive deposits. The minimum embedment of the MSE wall in accordance to ODOT and AASHTO guidelines is 3.5 feet. If any loose, soft or compressible soils are encountered while excavating for the leveling pad, these soils should be removed and replaced with compacted granular fill. Any compacted granular fill below the leveling pad should be aggregate base conforming to CMS Item 304. In all cases, the thickness of the unreinforced concrete leveling pad shall not be less than 6 inches conforming to BDM Item 204. For stability, calculations have indicated that a minimum reinforcement length of 0.8H or 30.0 feet is required for stability of the proposed MSE wall at the forward abutment location. Similarly, a minimum reinforcement length of 0.8H or 25.4 feet is required for stability of the proposed MSE wall at the rear abutment.

The total maximum settlement of the MSE wall volume at the rear abutment was estimated to be approximately 4 inches at the centerline of the wall. Settlement was calculated using the computer program EMBANK, using the "end of fill" option to model the non-continuous embankments. Differential settlement at this location was estimated to be approximately 0.50 percent. MSE retaining walls are able to withstand relatively large amounts of differential settlement, typically up to 100 millimeters per 10 meters of wall length (1.0 percent). Settlement calculations are presented in Appendix IV. The MSE wall at the forward abutment will be founded at or near bedrock. Therefore, the settlement at the forward abutment location is assumed to be negligible.

Time-rate of settlement calculations will be presented in the final report based upon laboratory test results from samples collected in the final borings.

Table 4, below presents the MSE retaining wall parameters and results of analyses.

Table 4, MSE Retaining Wall Parameters and Analyses Results (Forward and Rear Abutments)

Compacted Granular Fill Foundation

Retained Soil (New Embankment) Unit Weight = 120 pcf Coefficient of Active Earth Pressure (K_a) = 0.33 (Based on $\Phi' = 30^{\circ}$)
Sliding along base of MSE wall Sliding Coefficient (μ)(0.67) = tan 29°(0.67) = 0.37 Use (μ)(0.67) = 0.35 as a maximum value as per AASHTO, BDM,303.4.1.1
Allowable Bearing Capacity – Undrained Condition q_{all} = 7,277 psf
Allowable Bearing Capacity – Drained Condition q_{all} = 6,976 psf
Global Stability Factor of Safety – Undrained Condition = 2.3 Factor of Safety – Drained Condition = 1.6 Factor of Safety – Drained Seismic Condition = 1.5
Estimated Settlement of MSE volume Maximum Total Settlement ≈ 4 inches Differential Settlement ≈ 0.50% < 1.0%
Full Height of MSE Wall (Maximum) = 37.5 feet (including embedment depth) Minimum Embedment Depth = 3.5+ feet Minimum Length of Reinforcement for External Stability = 30.0 feet (Forward Abutment) Minimum Length of Reinforcement for External Stability = 25.4 feet (Rear Abutment)

+ *Minimum embedment depth. No embedment in bedrock is required.*

5.3 MSE Wall Foundation Earthwork Recommendations

Excavations for the proposed MSE wall should be prepared in accordance with ODOT-CMS 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 (29 CFR 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. Backfill material used to establish planned grades may consist of nonfrost susceptible clean granular soil free of topsoil or organic material. Alternatively, the excavation may be backfilled with Ohio Department of Transportation (ODOT) Construction and Material Specifications (CMS) Item 304 and should be compacted in conformance to CMS 203.06 and 203.07.

5.4 Groundwater Considerations

Water seepage was not encountered in any of the borings. Groundwater was not noted prior to adding drill water. Representative final water levels could not be obtained due to the use of water during rock coring. Excavation for the pier foundation is expected to be limited to 15 feet or less. Foundation construction on the rock is expected to encounter only minor seepage. Excavations or shafts extending below ground level may encounter more significant seepage through fractured zones in the rock. The contractor should be prepared to deal with seepage and water flow 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 Riedy
Geotechnical Engineer

Wael Alkasawneh (SJR)

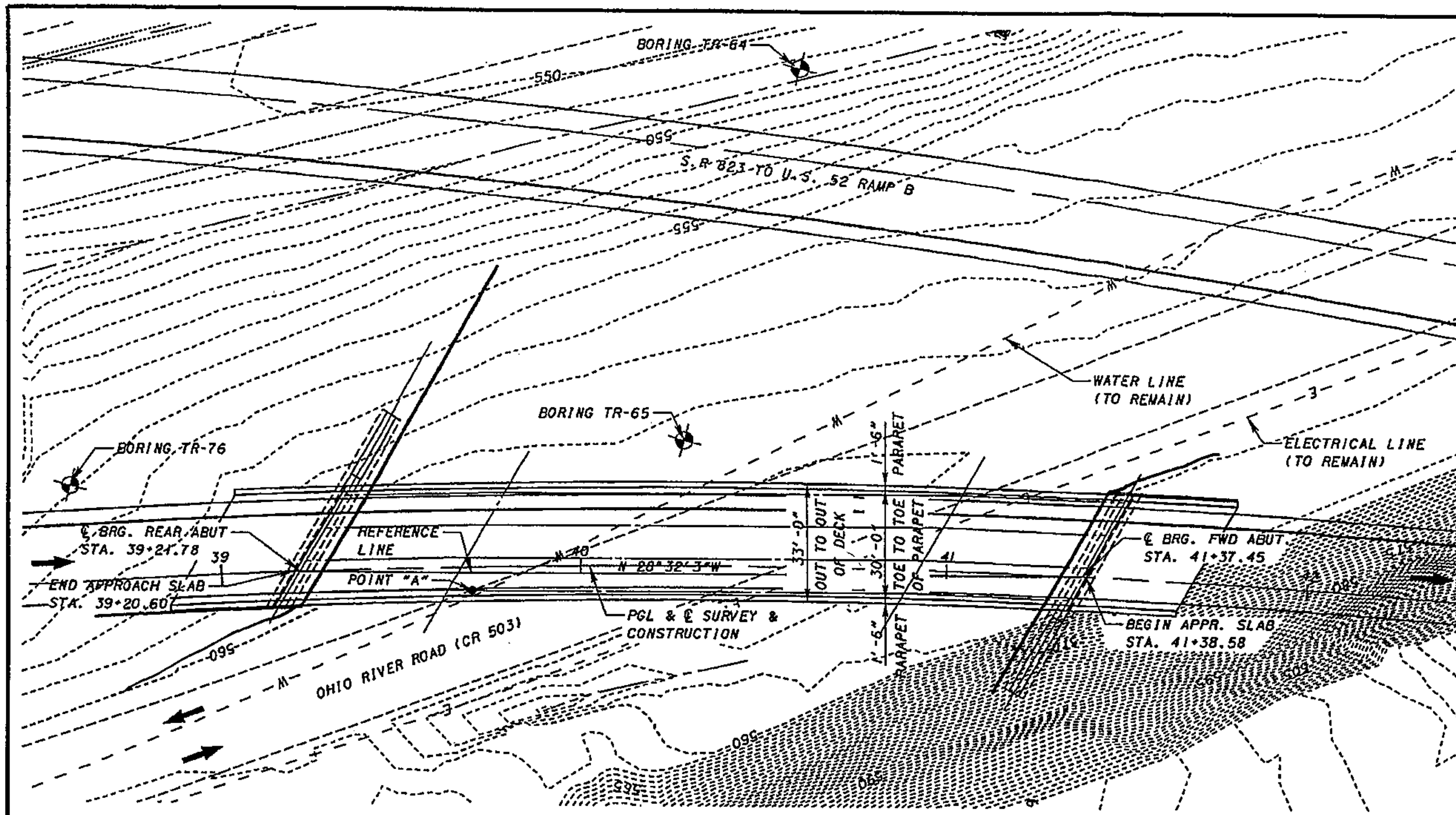
Wael Alkasawneh, P.E.
Geotechnical Engineer

sjr

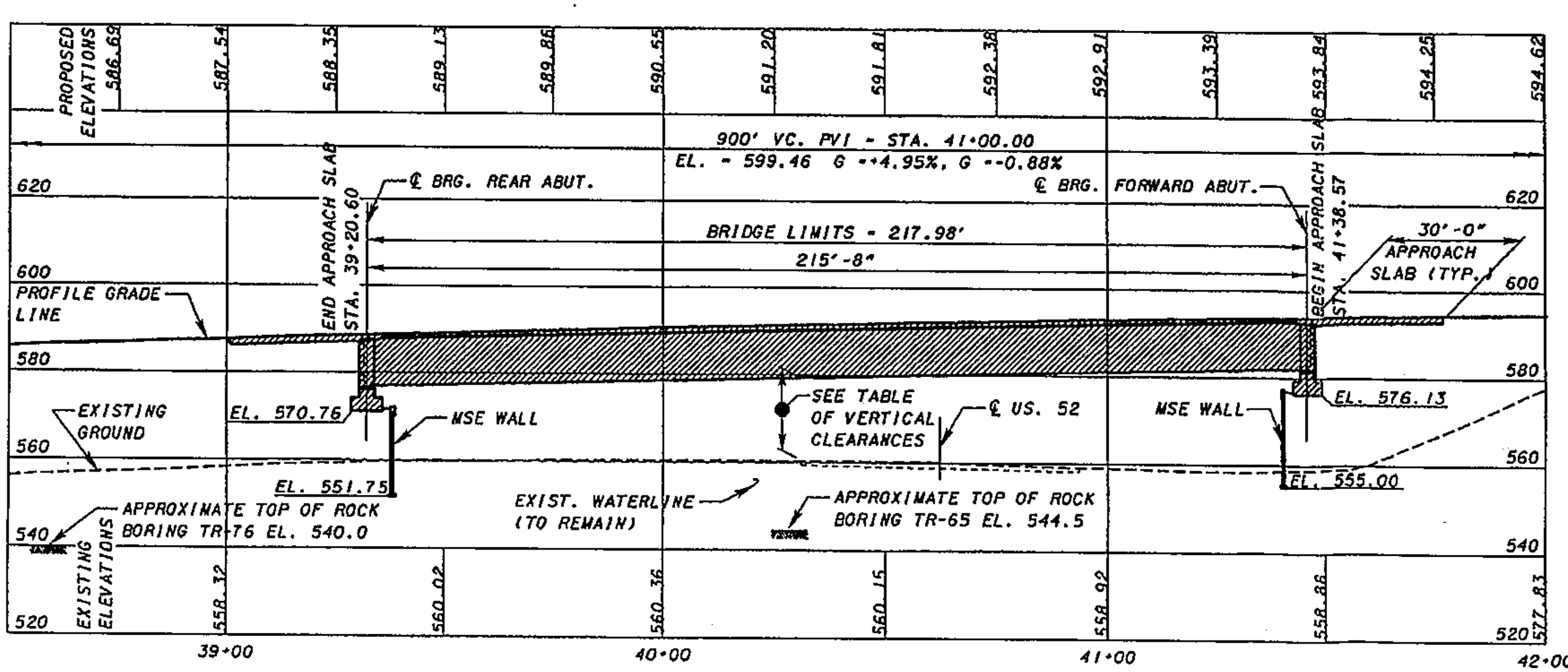
M:\proj\0121\3070.03\US 52\Ramp A\New after Transystem Final Plans_10_2_2006\US 52 Ohio River Road-Structure Report-RAMP A 10-16-2006 SJR.doc

APPENDIX I

Structure Plan and Profile Drawings - Two (2) -11"x17"
Boring Plan - 11"x17"



PLAN



PROFILE ALONG PROFILE GRADE LINE

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	17.16'
PREFERRED	17.0'

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	SIDE
REAR ABUT.	x	RT.
REAR ABUT.	x	LT.
FWD. ABUT.	x	RT.
FWD. ABUT.	x	LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.

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

TRAFFIC DATA

(ROUTE)
CURRENT YEAR ADT (2010) = 13,400
DESIGN YEAR ADT (2030) = 21,000
CURRENT YEAR ADTT (2010) = 1,876
DESIGN YEAR ADTT (2030) = 2,940

PROPOSED STRUCTURE

TYPE: SINGLE SPAN, 125" WEB STEEL PLATE GIRDERS A709 GRADE 50W WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY SEMI-INTEGRAL ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS
SPANS: 215'-8" C/C BEARINGS
ROADWAY: 30' TOE TO TOE OF PARAPETS
LOADING: HS-25 AND ALTERNATIVE MILITAR LOADING FWS-60 PSF
SKEW: 30°0'0" LF WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)
CROWN: 0.056 FT/FT ACROSS LANE
ALIGNMENT: Dc = 2°15'00"
WEARING SURFACE: MONOLITHIC CONCRETE
APPROACH SLABS: AS-1-81 (30' LONG)

LATITUDE:
LONGITUDE:

NOTES:

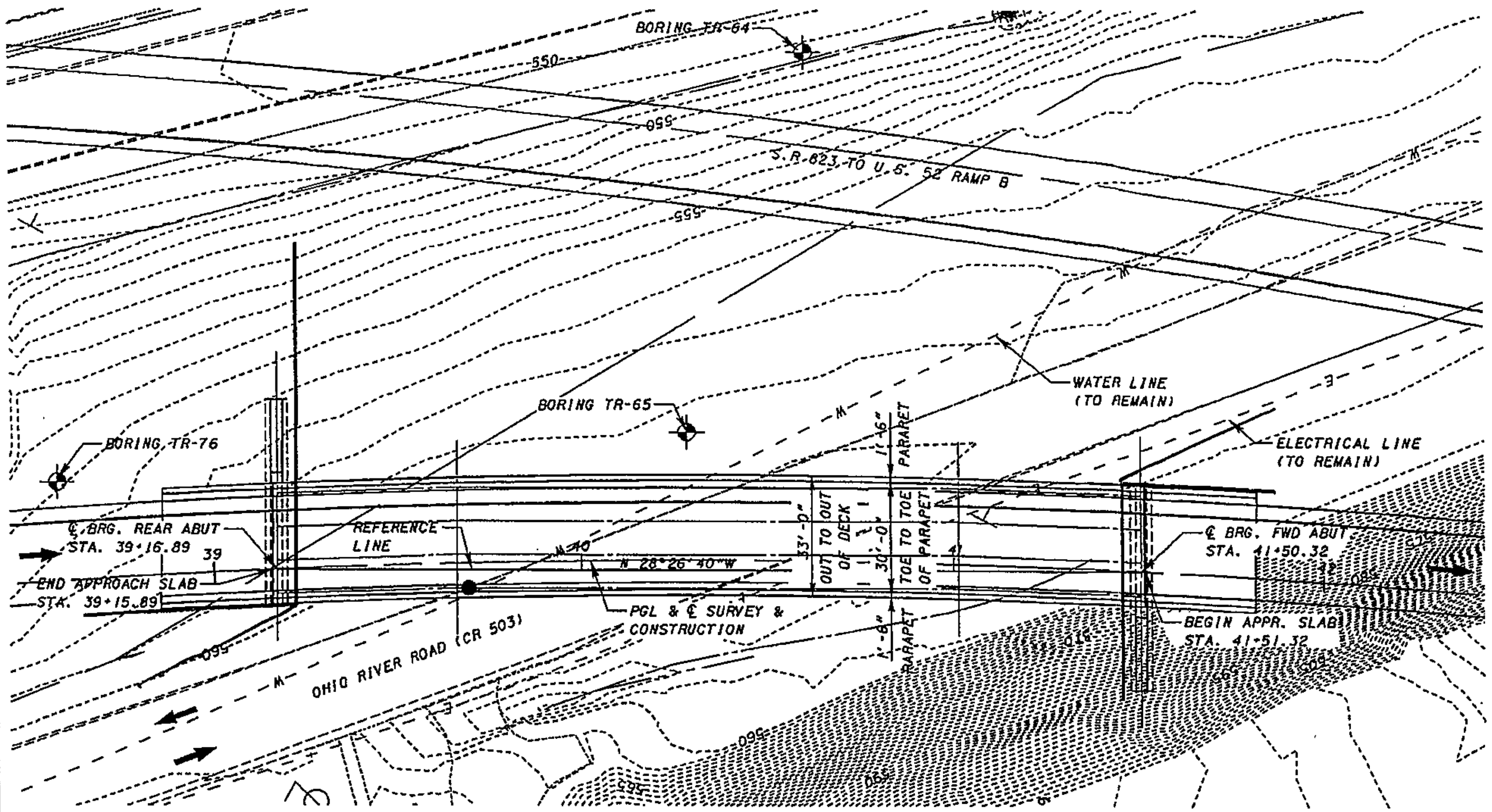
1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS.
3. THE PROPOSED PROFILE GRADE IS WITHIN BRIDGE LIMITS. SEE ROADWAY PLANS FOR PAVEMENT ELEVATIONS BEYOND BRIDGE LIMITS.

DESIGN AGENCY: TranSystems
DESIGNED WSL | CHECKED | DRAWN MTN | REVISED | REVIEWED | DATE | STRUCTURE FILE NUMBER

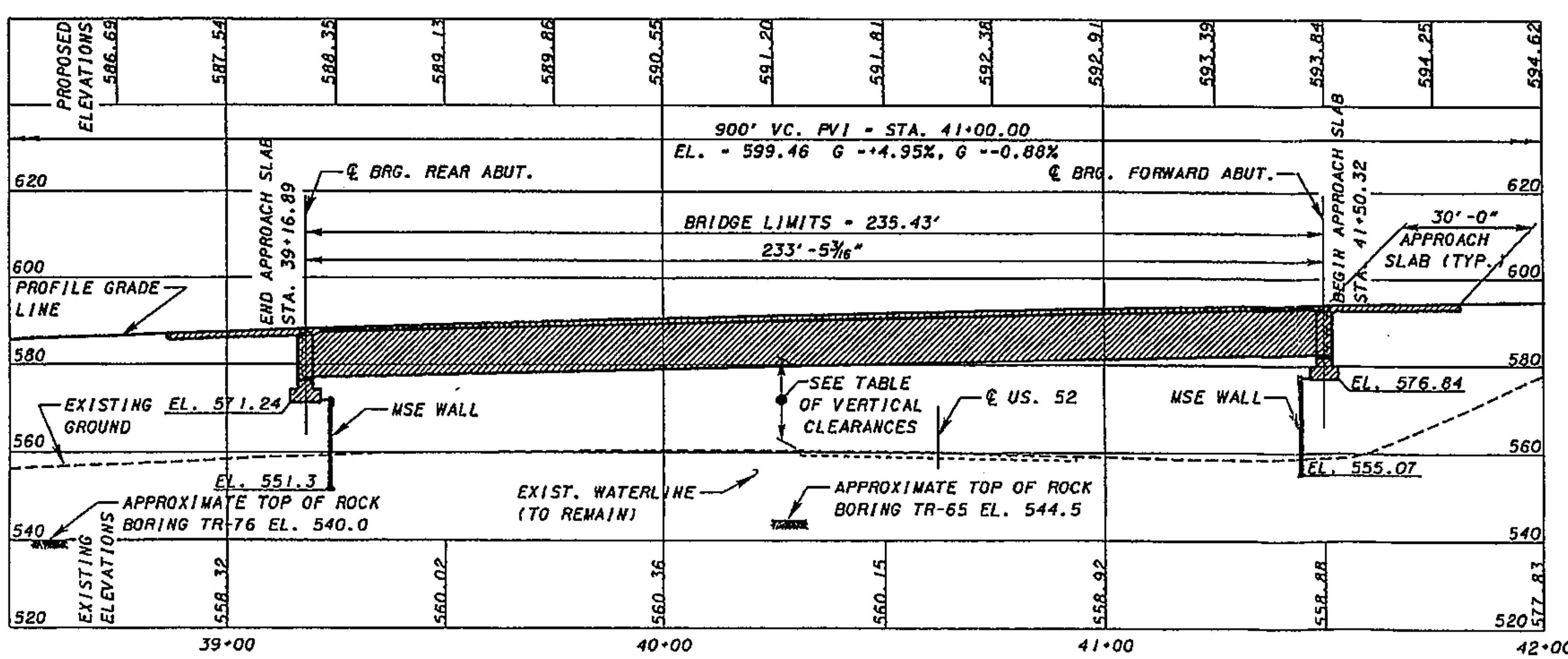
STA. 39+20.60
STA. 41+38.58

PRELIMINARY SITE PLAN - ALT. 1
BRIDGE NO. US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 19415



PLAN



PROFILE ALONG PROFILE GRADE LINE

FIRST GUARDRAIL POST OFF BRIDGE LOCATIONS		
LOCATION	STATION	SIDE
REAR ABUT.	x	RT.
REAR ABUT.	x	LT.
FWD. ABUT.	x	RT.
FWD. ABUT.	x	LT.

BORING LOCATIONS		
BORING No.	STATION	OFFSET
TR-76	38+59.24	27.10' LT.
TR-65	40+28.51	35.06' LT.
B-X	xx+xx.xx	xx.xx' LT.
B-X	xx+xx.xx	xx.xx' LT.

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

TRAFFIC DATA

(ROUTE)
CURRENT YEAR ADT (2010) = 13,400
DESIGN YEAR ADT (2030) = 21,000
CURRENT YEAR ADTT (2010) = 1,876
DESIGN YEAR ADTT (2030) = 2,940

PROPOSED STRUCTURE

TYPE: SINGLE SPAN, 122" WEB STEEL PLATE GIRDERS A709 GRADE 50W WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY SEMI-INTEGRAL ABUTMENTS FOUNDED ON PILES AND MSE WALL EMBANKMENTS

SPANS: 215'-8" C/C BEARINGS

ROADWAY: 30' TOE TO TOE OF PARAPETS

LOADING: HS-25 AND ALTERNATIVE MILITAR LOADING FWS-60 PSF

SKEW: 90°0'0" WITH RESPECT TO THE REFERENCE LINE (ALSO SEE FRAMING PLAN)

CROWN: 0.056 FT/FT ACROSS LANE

ALIGNMENT: Dc = 2°15'00"

WEARING SURFACE: MONOLITHIC CONCRETE

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

LATITUDE:
LONGITUDE:

TABLE OF VERTICAL CLEARANCES	
LOCATION	"A"
PROPOSED	17.59'
PREFERRED	17.0'

NOTES:

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

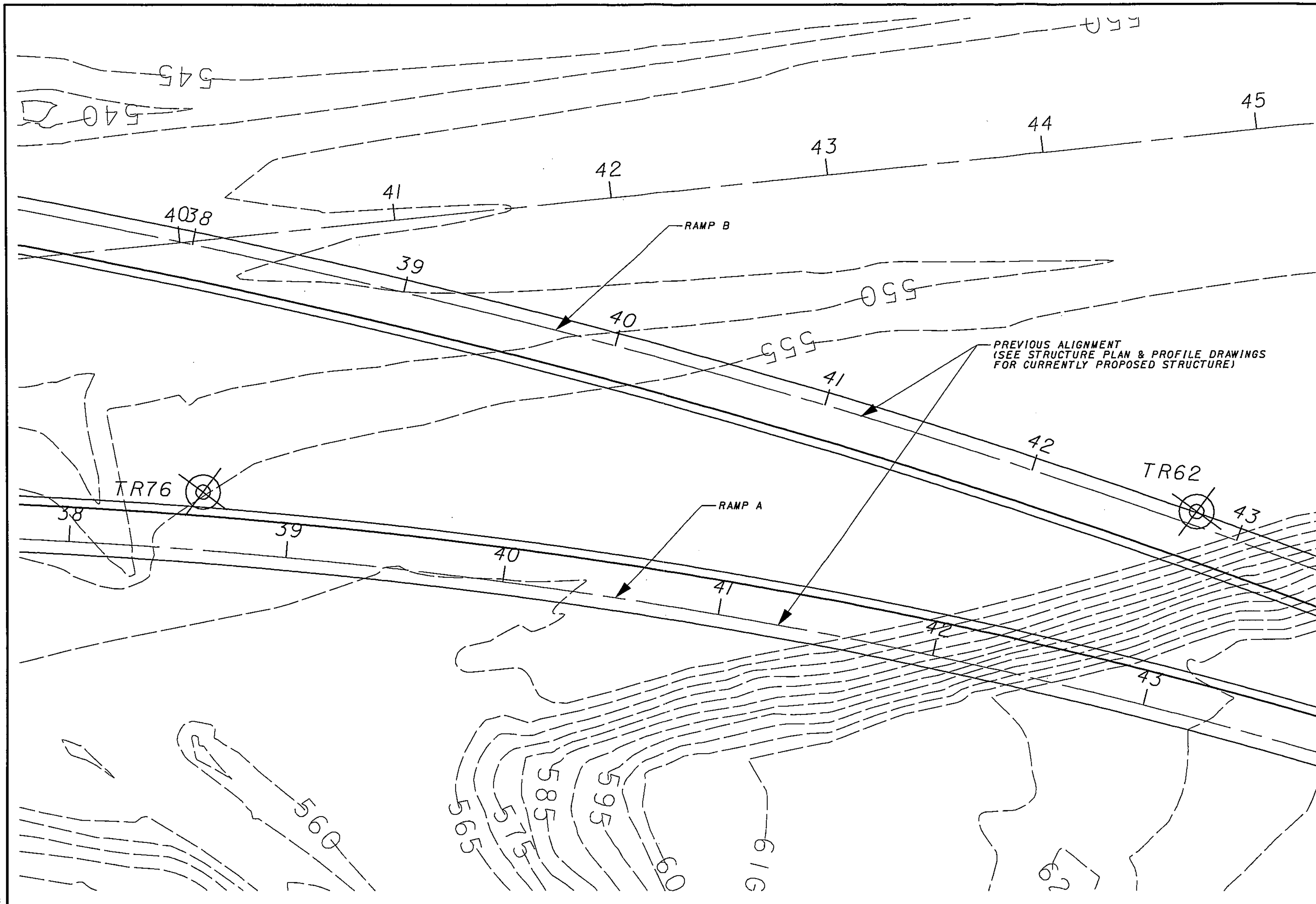
DESIGN AGENCY: TranSystems

DESIGNED MSL | CHECKED | DRAWN MTN | REVISED | REVIEWED | DATE | STRUCTURE FILE NUMBER

STA. 39+15.89
STA. 41+51.32

PRELIMINARY SITE PLAN - ALT. 2
BRIDGE US-52-XXXX
US-52 RAMP A TO NORTHBOUND S.R. 823

SCI-823-0.00
PID 19415



HORIZONTAL SCALE IN FEET

CALCULATED

CHECKED

US 52 RAMP A OVER OHIO RIVER ROAD

PRILIMINARY STRUCTURE BORING PLAN

SCI-823

APPENDIX II

General Information – Drilling Procedures and Logs of Borings
Legend – Boring Log Terminology
Boring Logs – Two (2) 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

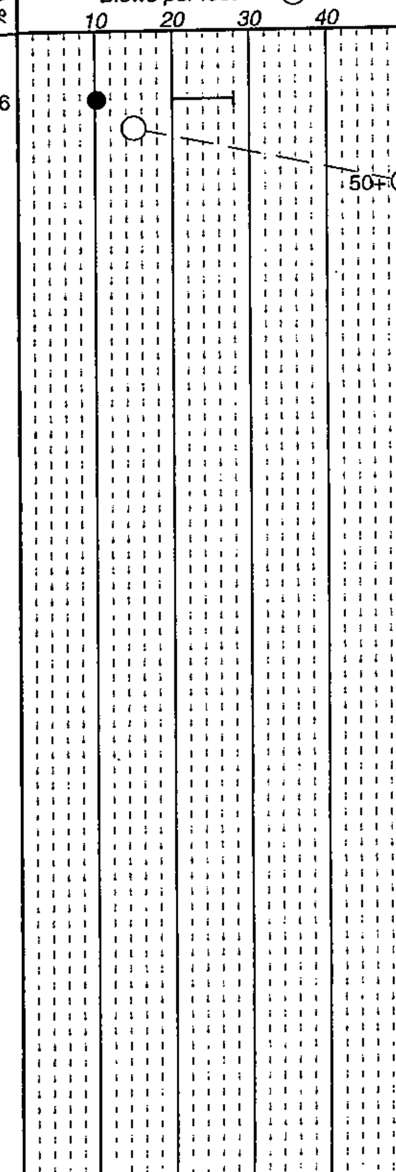
Client: TranSystems, Inc. | Project: SCI-823-0.00 | Job No. 0121-3070.03

LOG OF: Boring TR-62 | Location: US-52 Ramp A | Date Drilled: 3-18-05

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
0	559.1	8 9 6	14	1		3.5	Very stiff gray SANDY SILT (A-4a), little gravel, little clay; contains sandstone fragments; damp.	17	15	--	6	46	16
3.5	555.6	50/2	2	2			Soft to medium hard gray SANDSTONE; fine grained, highly weathered to decomposed, broken.						
6.0	553.1	Core 120"	Rec 120"	RQD 78%	R-1		Hard gray SANDSTONE; very fine to fine grained, slightly weathered, argillaceous, micaceous, thinly bedded, slightly fractured. @ 9.3', qu = 10,794 psi. @ 11.2' to 11.3' high angle fracture.						
16.0	543.1						Bottom of Boring - 16.0'						

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



Depth scale (ft): 0, 5, 10, 15, 20, 25, 30

FILE: 0121-3070-03 | 10/17/2006 2:43 PM]

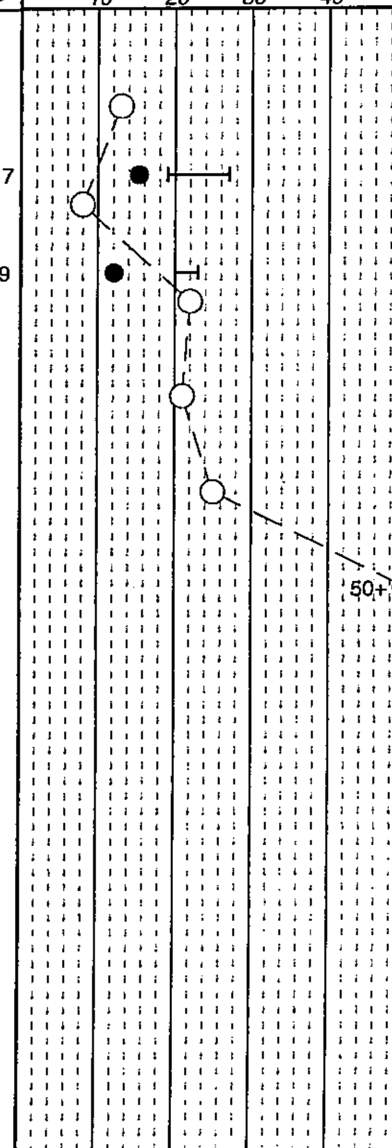
DLZ OHIO INC. * 6121 HUNTLEY ROAD, COLUMBUS, OHIO 43229 * (614)888-0040

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

LOG OF: Boring TR-76 | Location: As Per Plan | Date Drilled: 3/30/05

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

Depth (ft)	Elev. (ft)	Blows per 6"	Recovery (in)	Sample No. Drive	Sample No. Press / Core	Hand Penetrometer (tsf)	DESCRIPTION	% Aggregate	% C. Sand	% M. Sand	% F. Sand	% Silt	% Clay
	555.1												
0.2	554.9						Topsoil - 2"						
		4 6 7	12	1		4.5+	Hard brown SILT AND CLAY (A-6a), trace fine to coarse sand, trace gravel; damp.						
3.0	552.1	2 3 5	10	2		4.0	Very stiff to hard brown SANDY SILT (A-4a), trace to little clay, little gravel; damp.	15	10	--	12	47	17
		2 7 15	17	3				16	23	--	22	30	9
8.0	547.1	5 9 12	16	4			Medium dense brown SANDY SILT (A-4a), trace clay, little gravel; damp.						
		11 12 13	18	5			@ 11.0', contains sandstone fragments						
		1 6 50/5	17	6									
14.5	540.6						Soft to medium hard brown SANDSTONE; fine grained, highly weathered to decomposed, broken.						
16.1	539.0	Core 120"	Rec 120"	RQD 64%	R1		Hard brown SANDSTONE; very fine to fine grained, moderately to highly weathered, medium bedded, moderately fractured. @ 16.4'-16.8', 17.3', 18.4', filled fractures. @ 19.4' gray @ 20.9'-21.3', fractured. @ 19.6', qu = 11,036 psi.						
25.0	530.1						Bottom of Boring - 25.0'						



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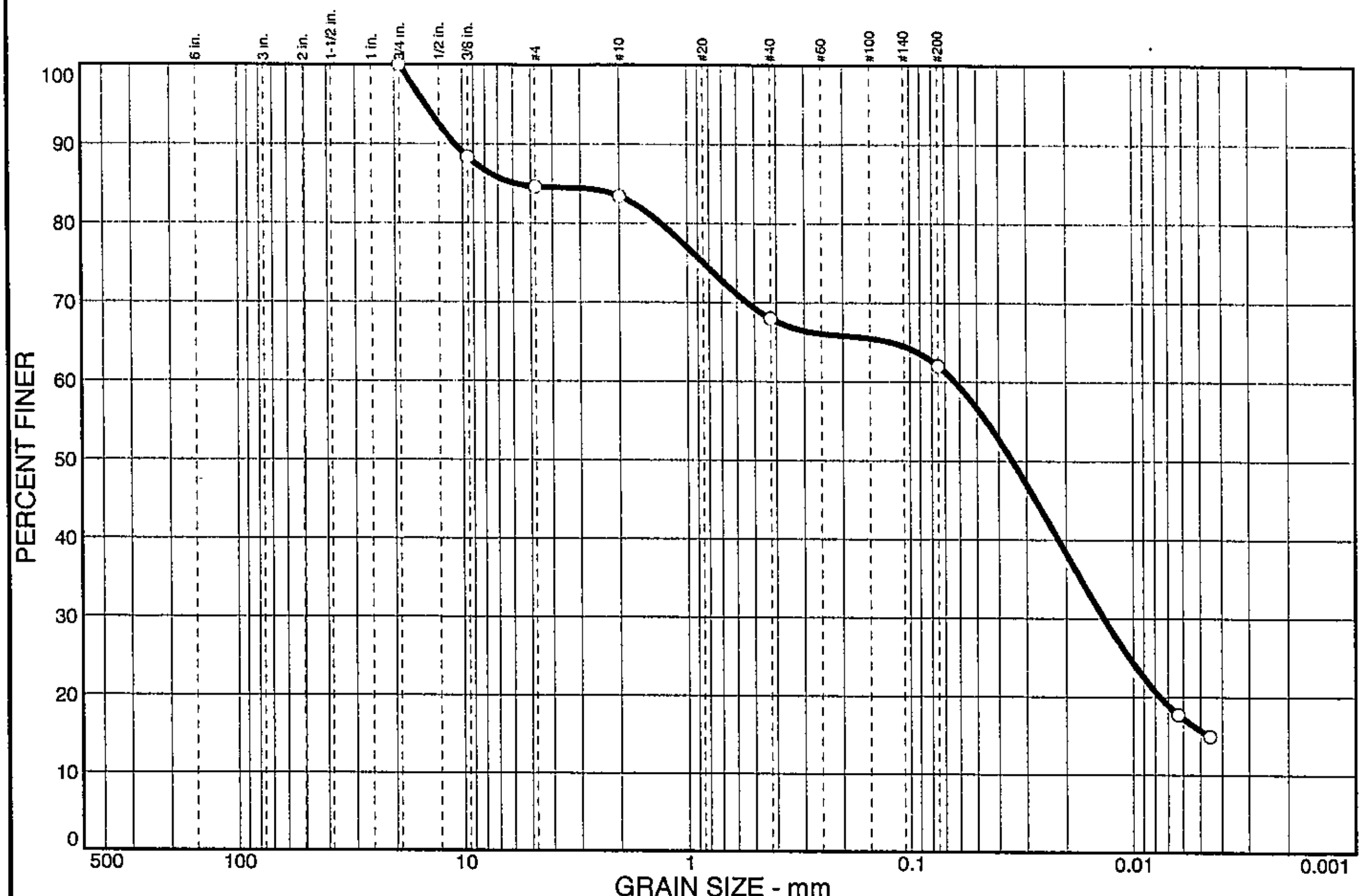
APPENDIX III

Laboratory Test Results

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	15.4	1.2	15.4	6.0	46.3	15.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	88.3		
#4	84.6		
#10	83.4		
#40	68.0		
#200	62.0		

* (no specification provided)

Soil Description

Sandy lean clay with gravel

Atterberg Limits

PL= 20 LL= 28 PI= 8

Coefficients

D_{85}= 5.86 D_{60}= 0.0631 D_{50}= 0.0350

D_{30}= 0.0136 D_{15}= 0.0046 D_{10}=

C_u= C_c=

Classification

USCS= CL AASHTO= A-4(3)

Remarks

Moisture Content= 10.2%

Sample No.: 1 **Source of Sample:** TR-62 **Date:** 4/12/05

Location: **Elev./Depth:** 1

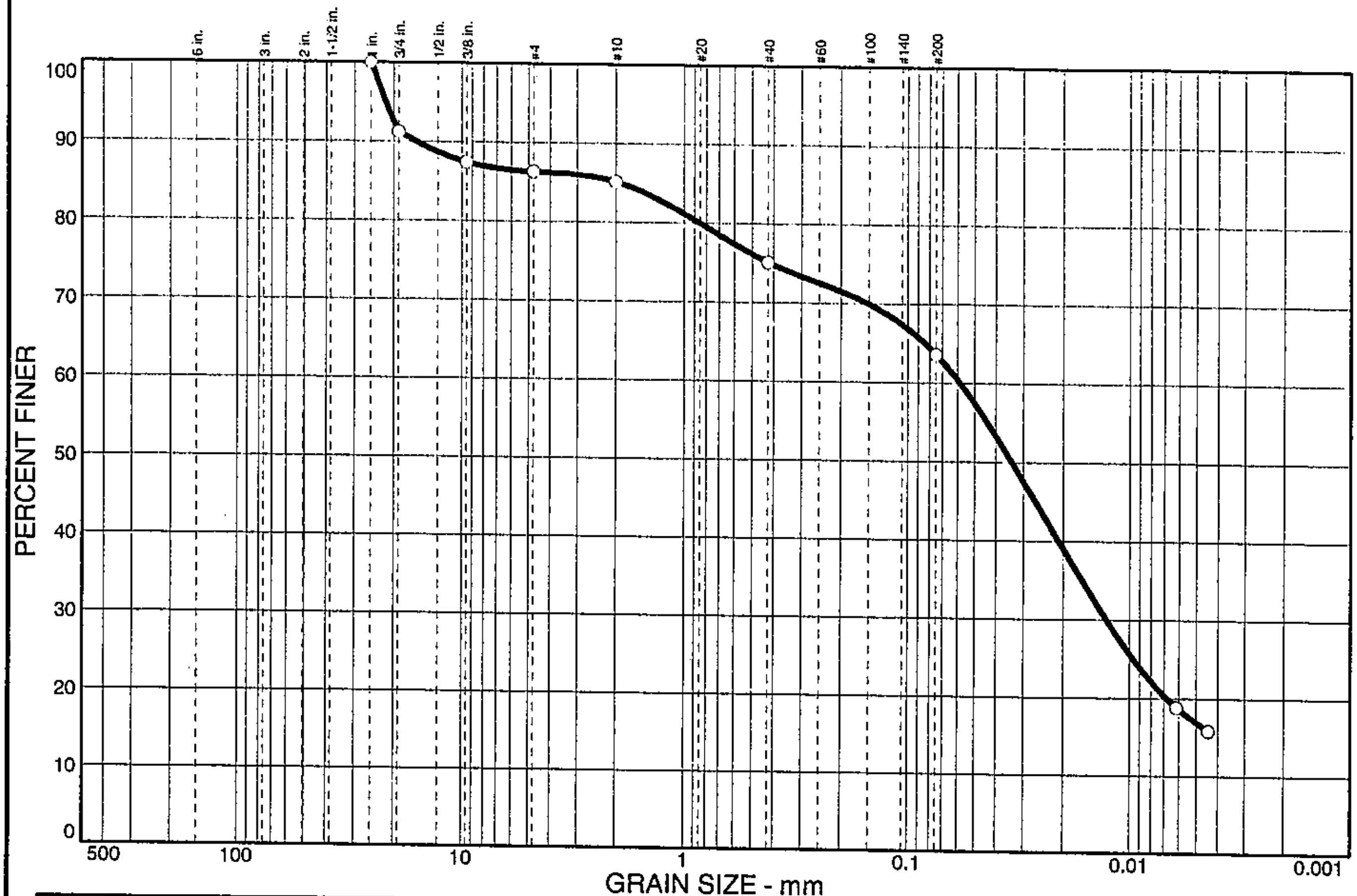
DLZ

Client: TranSystems, Inc.

Project: SCI-823-0.00

Project No: 0121-3070.03 **Figure**

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	8.8	4.9	1.2	10.0	11.5	46.8	16.8

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1.00 in.	100.0		
0.75 in.	91.2		
0.375 in.	87.4		
#4	86.3		
#10	85.1		
#40	75.1		
#200	63.6		

* (no specification provided)

Soil Description

Sandy lean clay

Atterberg Limits

PL= 19 LL= 27 PI= 8

Coefficients

D_{85}= 1.95 D_{60}= 0.0586 D_{50}= 0.0340
D_{30}= 0.0128 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-4(3)

Remarks

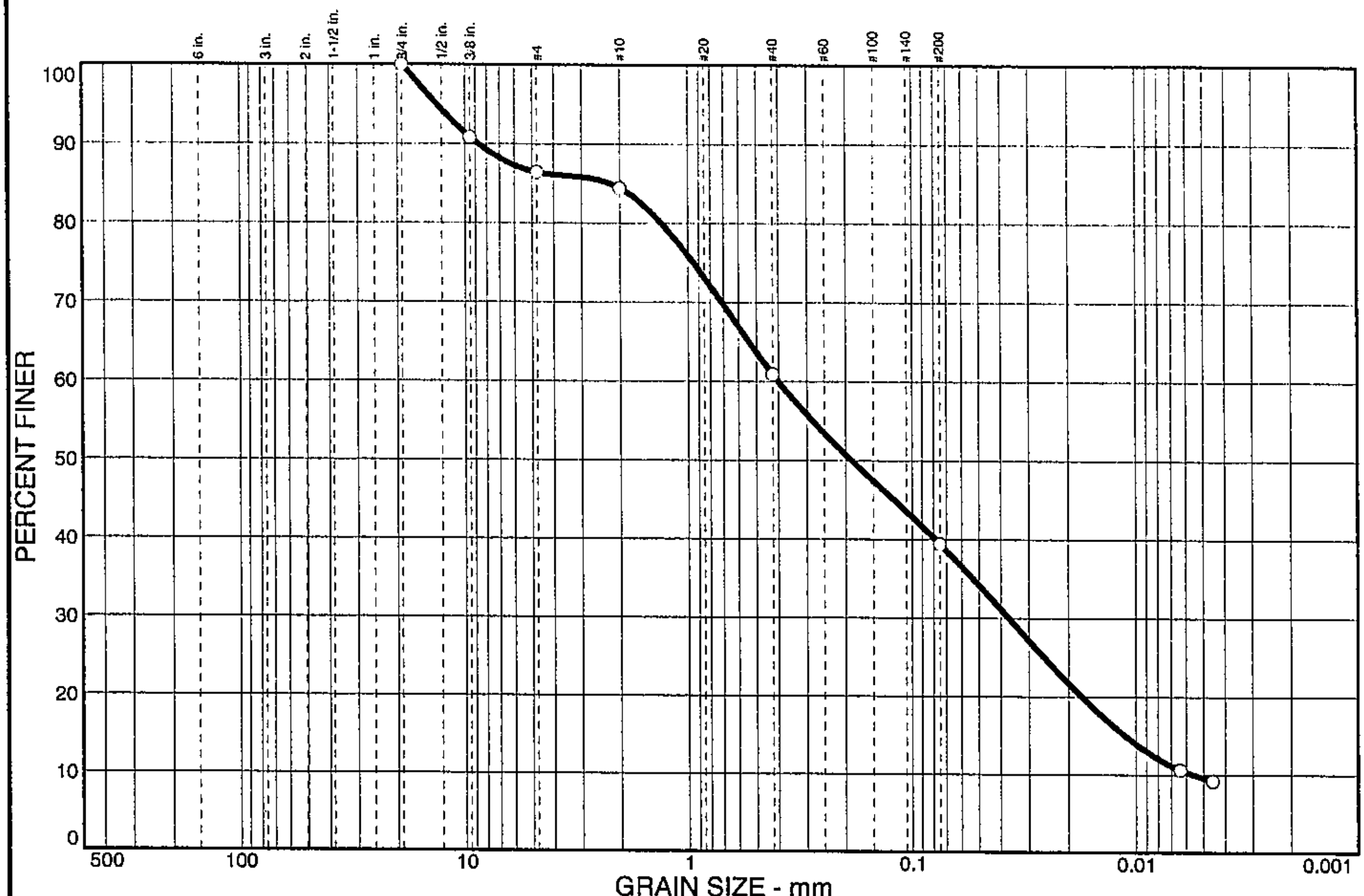
Moisture Content= 15.3%

Sample No.: 2 **Source of Sample:** TR-76 **Date:** 5/28/05
Location: **Elev./Depth:** 3.5

DLZ

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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	13.6	2.0	23.5	21.5	29.9	9.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.75 in.	100.0		
0.375 in.	90.8		
#4	86.4		
#10	84.4		
#40	60.9		
#200	39.4		

* (no specification provided)

Soil Description

Silty sand

Atterberg Limits

PL= 20 LL= 23 PI= 3

Coefficients

D_{85}= 2.22 D_{60}= 0.402 D_{50}= 0.189
D_{30}= 0.0371 D_{15}= 0.0113 D_{10}= 0.0056
C_u= 71.43 C_c= 0.61

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Moisture Content= 12.1%

Sample No.: 3 **Source of Sample:** TR-76 **Date:** 5/28/05
Location: **Elev./Depth:** 6.0

DLZ

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

Unconfined Compression of Rock Core Specimens

(ASTM D-2938)

DLZ Project No.: 0121-3070.03 | **Client: TranSystems**

Project Name: SCI-823-0.00 | **Date:** 10/12/2006

Boring	Run	Depth (ft.)	D_1	D_2	D_3	$D_{(ave)}$	L_1	L_2	L_3	$L_{(ave)}$	L/D	Volume (ft^3)	Mass (Gram)	Unit Wt.(pcf)	Load (lbs)	Strength (psi)
TR-62	1	9.3'-9.7'	1.989	1.987	1.991	1.989	4.691	4.696	4.677	4.688	2.357	0.0084284	590.26	154.40	33,550	**10,794**
			1.989	1.991	1.989											
TR-76	1	19.6'-20.0	1.986	1.985	1.982	1.985	4.485	4.478	4.466	4.476	2.255	0.0080101	509.31	140.18	34,140	**11,036**
			1.986	1.986	1.983											

DLZ

Engineers * Architects * Scientists

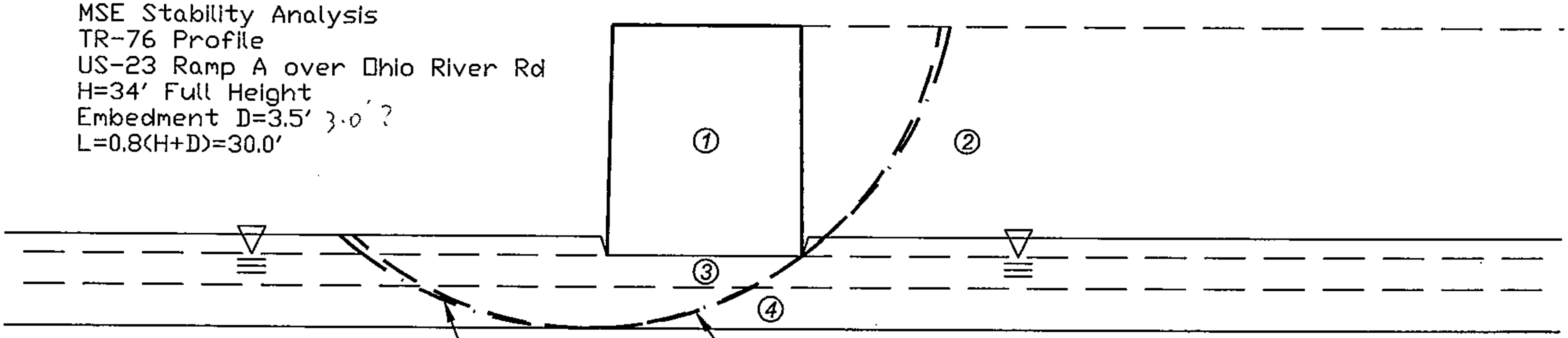
6121 Huntley Road * Columbus, Ohio * 43229-1003 * Phone: (614) 888-0576 * Fax (614) 888-6415

APPENDIX IV

MSE Wall Global Stability Analysis Results
MSE Wall Bearing Capacity and Stability Calculations
MSE Wall Settlement Calculations
Drilled Shaft – End Bearing and Side Resistance 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	Very Stiff	Sandy Silt	3500	0	0	(29) 32?	125
Material 4	M. Dense	Sandy Silt	0	29	0	29	125
Material 5		BEDROCK	10000	45	10000	45	145

MSE Stability Analysis
TR-76 Profile
US-23 Ramp A over Ohio River Rd
H=34' Full Height
Embedment D=3.5' 3.0'?
L=0.8(H+D)=30.0'



Sheet 1 of 7

US-52 Ramp A over Ohio River Road
BASED ON BORING B-76 PROFILE

MSE STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 10/12/06

DLZ

SUBJECT			
Client	TranSystems ODOT D-9	JOB NUMBER	0121-3070.03
Project	SCI-823 Portsmouth Bypass	SHEET NO.	2 OF 7
Item	MSE Wall Stability (Forward Abutment)	COMP. BY	SJR DATE 10/12/06
US-52 Ramp A-Northbound over Ohio River Road		CHECKED BY	gwsT DATE 10-17-06

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=34'
2 It is assumed that the bridge is supported on deep foundations
3 Ground water; Dw=0.0'
4 Traffic loading is neglacted in resisting forces
5

Wall Properties

H+D	=	38	feet
γ_{mse}	=	120	pcf
L	=	31	feet
L factor	=	0.80	
ϕ	=	30	deg

Foundational Soil Properties

c	=	3500	psf	Cohesion
ϕ'	=	29	deg	Friction angle
ω_T	=	240	psf	Traffic loading

Length factor-range (0.7 - 1.0)

Friction Angle of Embankment Fill

RESISTANCE AGAINST SLIDING ALONG BASE

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

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

P_a = 31,601 lbs per foot of wall

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

where; $\mu = \tan(\phi)$ 0.67μ = 0.37

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

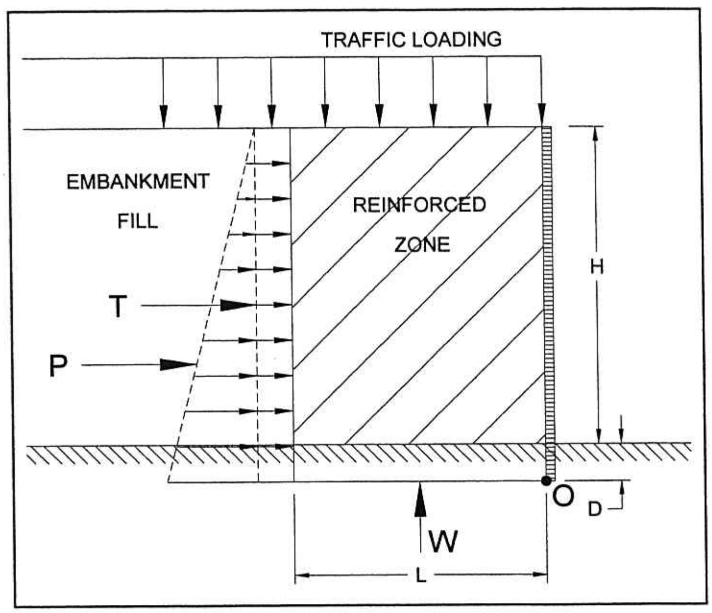
P_r = 49,476 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 108,500 lbs per foot of wall

Use Drained Value



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

Calculated: **FS = 1.57**

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

$\Sigma M_{resisting}$ = 2,191,080 lb-ft

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

$\Sigma M_{overturning}$ = 419,338 lb-ft

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

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

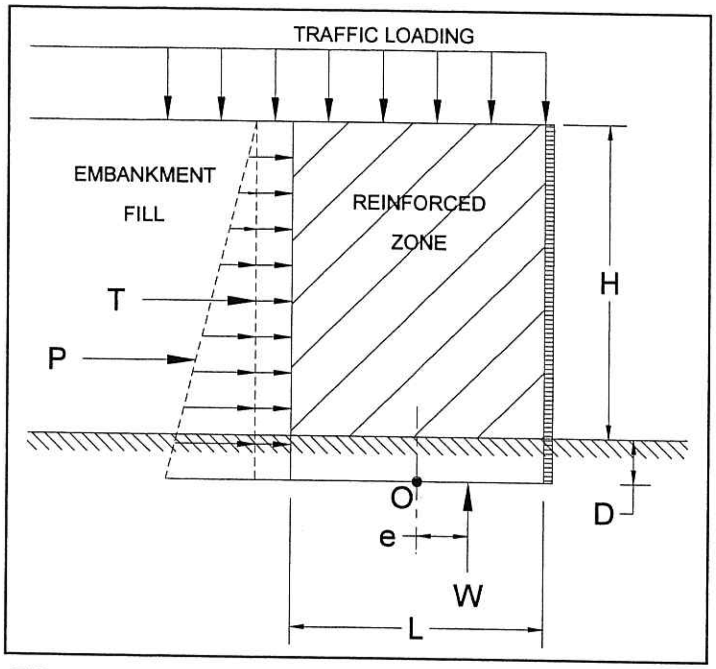
Calculated: **FS = 5.23**

Required: FS = 2.00

Resistance Against Overturning is **OK**

BEARING CAPACITY OF A MSE WALL

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



Soil Properties

γ_{EMB}	=	120	pcf	Unit weight	Embankment fill
ϕ'_{EMB}	=	30	deg.	Friction ang.	Embankment fill
γ_{FDN}	=	120	pcf	Unit weight	Foundation soil
c	=	3500	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	29	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	240	psf	Traffic loading
D	=	3.5	ft	Embedment depth (H/20<D<3.5)
Dw	=	0	ft	Groundwater depth
H	=	34	ft	Height of wall
H+D	=	38	ft	
L factor	=	0.8		Length factor-range (0.7 - 1.0)
L=B	=	31	ft	Length of MSE reinforcement
Ka	=	0.33		
Γ Pa	=	12.667	ft	Moment arm
Γ Wt	=	19	ft	Moment arm
B'	=	25.36	ft	
γ'	=	57.6	pcf	
W_t		7,440	lb/ft of wall	Weight from traffic
W_{mse}	=	141,360	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 5,868 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 18,192 psf

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

q_{ALL} = 7,277 psf

Factor of Safety = **3.10** **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} = 17,440 psf

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

q_{ALL} = 6,976 psf

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

Bearing Capacity Factors for Equations (AASHTO)

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

Eccentricity of Resultant Force

e = 2.82 ft

Kern

e < L/6 = 5.17 ft

US-52 Ramp A over Ohio River Road

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

Project Name : SCI-823 Portsmouth
File Name : US 52 Ramp A MSE
Date : 10/13/10
Client : TranSystems
Project Manager : Nix
Computed by : SJR
Checked by: EWT

Settlement for X-Direction

Embank. slope, x direc. = 54.00 (ft)
y direc. = 54.00 (ft)
Embankment top width = 30.00 (ft)
Embankment bottom width = 138.00 (ft)
Ground Surface Elev. = 555.00 (ft)
Water table Elev. = 552.00 (ft)
Height of fill H = 28.20 (ft)
Unit weight of fill = 120.00 (pcf)
p load/unit area = 3384.00 (psf)
Foundation Elev. = 555.00 (ft)
Unit weight of Wat. = 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	3.0	-----	-----	-----	120.00	----	----
2	COMP.	5.0	0.130	0.000	0.000	125.00	2.65	0.36
3	COMP.	6.4	0.035	0.000	0.000	125.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	5.00	549.50	516.50	516.50
3	6.40	543.80	873.32	873.32

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 6.90 Stress (psf)	Sett. (in.)	X = 13.80 Stress (psf)	Sett. (in.)	X = 20.70 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	49.46	0.23	222.76	0.89	433.39	1.52	646.81	2.02
3	107.40	0.07	253.81	0.15	445.73	0.24	651.47	0.33
		0.30		1.04		1.76		2.35

Settlement at Corner of Wall

Layer	X = 27.60 Stress (psf)	Sett. (in.)	X = 34.50 Stress (psf)	Sett. (in.)	X = 41.40 Stress (psf)	Sett. (in.)	X = 48.30 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	858.97	2.44	1071.39	2.80	1283.19	3.11	1487.51	3.38
3	860.58	0.40	1068.99	0.47	1272.10	0.52	1457.11	0.57
		2.84		3.26		3.63		3.95

Layer	X = 55.20 Stress (psf)	Sett. (in.)	X = 62.10 Stress (psf)	Sett. (in.)	X = 69.00 Stress (psf)	Sett. (in.)

US-52 Ramp A over Ohio River Road

```
3                                                                          3
3  1  INCOMP.  INCOMP.  INCOMP.                                            3
3  2  1638.09   3.56    1665.33   3.59   1668.23   3.59                    3
3  3  1588.10   0.60    1641.73   0.62   1653.45   0.62                    3
3                ------            ------            ------                3
3                 4.16              4.21              4.21                 3
3                                                                          3
3                                                                          3
3                                                                          3
ÀÄÄÄÄÄ Hit arrow keys to display next screen. <F8> Print. <F10> Main Menu ÄÄÄÄÄÙ
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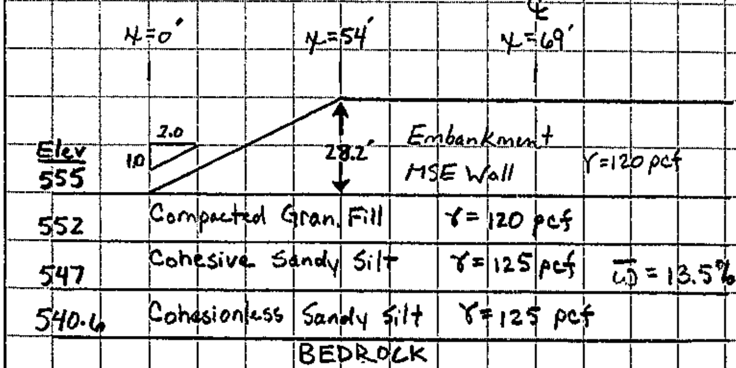
Settlement at Wall/Embankment ℄.

Most Critical Soil Profile is at boring TR-76 location.
At TR-76, maximum MSE Wall height is approximately 28.2'.



* Assume Soils are normally Consolidated

- Compacted Gran. Fill: Assume Incompressible
- Cohesive Sandy Silt: * $C_c = 0.13$ $e_o = 0.365$ [FHWA-NHI-00-045]
- Cohesionless Sandy Silt: * $\bar{N} \simeq 21$ $\bar{N}' \simeq 21$ → $C' = 57$ [FHWA]
  - → [$C_c = 0.035$, $e_o = 1.0$] - See Calculation Below

*Consolidation Parameters are estimated from FHWA NHI-00-045 for; cohesive soils based upon moisture and, cohesionless soils based upon an average SPT N-Values.

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

$$\frac{1}{C'} = \frac{C_c}{1+e_o}$$

Say $e_o = 1.0$ in this case.

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

When $C' = 57$, $C_c = \frac{2}{57} = 0.035$

From EMBANK $\delta_{c_{max}} = 4.21''$

Estimated Differential Settlement (DS)

$$DS = \frac{4.21'' - 0.30''\ (1/2'')}{69' - 0'} = 0.0050 = 0.50\ \%$$

* From lab testing rock core samples $q_u \simeq 10{,}794$ psi (TR-62)

End Bearing FHWA-IF-99-025 Eq. 11.6 $q_{max}(MPa) = 4.83\,[q_u(MPa)]^{0.51}$
For RQD between 70-100% and $q_u > 0.5$ MPa (5.2 tsf).

$q_u = 10{,}794$ psi $= 74.42$ MPa

[Eq. 11.6] $q_{max} = 4.83\,[q_u(MPa)]^{0.51}$

$q_{max} = 4.83\,[74.42\ MPa]^{0.51} = 43.50$ MPa

$q_{max} = 43.50$ MPa $= 6309$ psi $= 908$ ksf

$q_{allow} = \frac{q_{max}}{F.S.} = \frac{908\ ksf}{3.0} = 303$ ksf

* For this type of rock we typically use **$q_{allow} = 80$ ksf (40 tsf)**

Side Friction FHWA-IF-99-025 Eq. 11.24 $f_{max} = 0.65\,p_a\,[q_u/p_a]^{0.5} \le 0.65\,p_a\,[f'_c/p_a]^{0.5}$
- Assumes Smooth Rock Socket

* From lab testing rock core samples; $q_u \simeq 10{,}794$ psi $f'_c = 4{,}500$ psi

$f_{max} = 0.65\,p_a\,[q_u/p_a]^{0.5} \le 0.65\,p_a\,[f'_c/p_a]^{0.5}$

$f_{max} = 0.65\,(14.70\ psi)\left[\frac{10{,}794\ psi}{14.7\ psi}\right]^{0.5} \le 0.65\,(14.70\ psi)\left[\frac{4500\ psi}{14.70\ psi}\right]^{0.50}$

$f_{max} = 258.9$ psi ≤ 167 psi Use $f_{max} = 167$ psi

$f_{allow} = \frac{167\ psi}{3.0} = 55$ psi $= 7{,}920$ psf **Use $f_{allow} = 7{,}500$ psf**