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STRUCTURAL ENGINEERING

FEB 29 2008

RECEIVED

October 5, 2007

Mr. Mike Lenett
Senior Bridge Engineer
TranSystems Corporation
720 East Pete Rose Way, Suite 360
Cincinnati, Ohio 45205

Re: Addendum to Report of Subsurface Exploration for SR 823 Bridge over Slocum Avenue (TR-248), SCI-823-0229 L & R, SCI-823-0.00 Portsmouth Bypass (PID #77366), dated September 6, 2007

Dear Mr. Lenett:

Per our teleconference dated September 24, 2007, this letter presents our response to your comments on the above-referenced report. Your comments are reiterated below in italic and followed by our response.

1. *Section 5.1, page 4 of the report states "Analyses indicate that the required pile capacities can be achieved by installing the piles to less than 12 inches (at Boring TR-36, right forward abutment) to approximately 17 feet (at Boring B-32, Pier 2) above the underlying bedrock. Given the size of the structure and the anticipated high lateral and uplift loads, considerations should be given to driving all piles to the top of rock." Based upon comments from ODOT's Office of Structural Engineering (OSE), it was our understanding that H-pile foundations bearing on bedrock were preferred to support the abutments and the piers of the proposed structures. Since the analyses indicated that friction piles could be used for the bridge foundations, a copy of the ODOT's comment should be included in the report for justification if the end-bearing piles were chosen for the bridge foundations in the final design.*

   A copy of letter from TranSystems to ODOT, dated November 20, 2006, is attached. Item #12 of the letter states that the abutment and piers be supported on H-piles (HP14X95) with a maximum capacity of 95 tons per pile. The estimated pile length should be 140 feet and 130 feet for the rear abutment and forward abutment, respectively. The estimated pile length should be 95 feet and 80 feet for the rear pier and forward pier respectively. Based upon the estimated pile lengths, the recommended H-piles would be founded on bedrock at the site.

2. *Section 5.1, page 5 of the report states "Due to the likelihood of piles being driven near the top of rock, it is recommended that reinforced pile points be used to protect the piles while driving." According to Section 202.2.3.2.a of the ODOT's Bridge Design Manual, pile points should not be used when the depth of*

overburden is more than 50 feet and the soils are cohesive in nature. According to the subsurface conditions at the site and the anticipated pile lengths, it appeared that the piles would penetrate more than 50 feet of cohesive soils. As a result, pile points should not be used. Please clarify your recommendation.

The boring information indicates that the overburden at the site was predominantly cohesive soils. However, granular soils consisting of sandy silt (A-4a), fine sand (A-3) or coarse and fine sand (A-3a) were sporadically encountered in the majority of the borings. In addition, layers of granular soils, between 8 and 20 feet thick, were mostly encountered immediately above the bedrock. Given the results of pile analyses, it is anticipated that the piles would penetrate through sporadic layers of granular soils, generally between 2 to 5 feet thick, embedded in the cohesive soils and end at a few feet into the granular soil layers immediately above the bedrock. If only a few feet of sporadic layers of granular soils were encountered, pile points may not be necessary when driving the piles. However, due to the size of the structure and the anticipated high lateral and uplift load, longer piles through the thick layers of granular soils above the bedrock may be necessary. Given the likelihood of piles being driven near the top of rock, it is therefore recommended that reinforced pile points be used to protect the piles while driving.

3. *Section 5.2, page 8 of the report states "Please note that a friction angle of 35 degrees was assumed for the 2H:1V spill-through slopes." This friction angle was higher than the friction angle of 30 degrees as recommended for general backfill in the ODOT's Bridge Design Manual. Please clarify.*

Given the anticipated amount of cut in the existing bedrock for the Portmouth project and the subsurface conditions in the overall project area, it is anticipated that the granular backfill to be used for the spill-through slopes would have higher than normal gravel contents, which will result in higher friction angle. DLZ discussed the possible use of higher friction angle for embankment evaluations with ODOT last year. With ODOT's concurrence, a friction angle of 35 degrees was used for the embankment evaluations in a report titled "Report of Subsurface Investigation for Embankments (Station 416+00 to 509+50), Project SCI-823.6.81, Phase 1 - Stage 1," dated November 29, 2006 (excerpt copy attached).

4. *A traffic load of 240 pounds per square foot was used in the MSE wall analyses. However, since the MSE wall would be located from the proposed bridge at a distance more than one-half the maximum wall height, traffic loads should not be considered.*

The stability analyses for the MSE wall were performed without a traffic load. The analyses indicate a slight increase in the factors of safety for overturning, sliding and bearing capacity. However, these increases do not change any of our original recommendations concerning the MSE wall. A copy of the stability analyses without a traffic load is attached.

5. *A 3.2:1 (H:V) backfill slope perpendicular to the highest wall section was used in the analysis. However, according to the preliminary wall design plans, the backfill slope perpendicular to the highest wall section would be level and the 3.2:1 (H:V) backfill slope would be at a wall section approximately 25 feet northeast of the highest wall section. Please clarify your assumptions made in the selection of wall section.*

   It is understood that the backfill slope perpendicular to the highest wall section will be level. However, since the backfill slope will vary along the wall alignment, any backfill slopes that are out of square with the highest wall section would be non-zero slopes. As a result, the highest wall section with a level backfill slope was not used for the analysis. Since the sloping backfill will create different loading conditions than the level backfill, the wall was analyzed using a critical wall section, which consisted of the highest wall height and a 3.2:1 (H:V) backfill slope.

This letter should be attached to the above-referenced September 6, 2007 subsurface investigation report and made a part thereof.

If you have any questions regarding this letter, please feel free to contact me at (614) 888-0040.

Sincerely,

DLZ, Ohio, Inc.

Eric W. Tse, P.E.
Senior Geotechnical Engineer

Attachments: TranSystems' November 20, 2006 letter to ODOT
Excerpt copy of DLZ's November 29, 2006 report
Stability analyses of MSE wall without traffic loads

M:\proj\0121\3070.03\Structures\Pershing and Slocum\Final\Addendum to 9-6-07 final report (10-5-07)

TranSystems

5747 Perimeter Drive
Suite 240
Columbus, OH 43017
Tel 614 336 8480
Fax 614 336 8540

www.transystems.com

November 20, 2006

Mr. Jawdat Siddiqi, PE
Office of Structural Engineering
Ohio Department of Transportation
1980 W. Broad Street
Columbus, Ohio 43223

SUBJECT: Structure Type Study Resubmission # 3
SR 823 over Slocum Avenue
SCI-823-0.00 Portsmouth Bypass
PID#19415

Dear Mr. Siddiqi:

Submitted for your review and approval is the revised site plan for SR 823 over Slocum Avenue, as requested by Jeff Crace in his October 2, 2006 review letter. Please find below a response to the 10/2/06 comments.

1. *We agree that the proposed superstructure can consist of three spans of prestressed concrete I-girders made composite with the deck. We agree that the substructures should consist of reinforced concrete T-type piers supported on piling and semi-integral abutments supported on piling.*

   Comment noted.

2. *We agree that MSE walls should not be utilized at this location due to the wall height (60 feet) and the subsurface conditions [low strength and large settlements (21")]. The unit cost of the MSE walls given in the cost analysis [high wall, >50', $85/ft^2$ (2005)] appears to be appropriate. The estimated cost for a average wall height [25'-35' is approximately $50/ft^2$ (2005)].*

   Comment noted.

3. *Relatively long structures (>200') on somewhat steep grades (>3%) have experienced high forces caused by movement toward the low end of the structure. Investigate utilizing fixed elastomeric bearings at the forward abutment (with semi-integral abutment details) along with the proposed fixed bearings at the forward pier. The flexibility of the pier and abutment should be enough to accommodate the expansion of the forward span (<1").*

Comment to be given consideration by the final design consultant however, our response follows. It is recommended that the final design first investigate resolving this force into the fixed pier and, if required, investigate adding resistance at the abutment. Resolving the horizontal force through the abutment requires consideration of the pile foundation stiffness. Discussions with OSE staff indicated that it is also important to check the superstructure to substructure connection and that it may be a weak point. We have investigated the horizontal force due to the self weight of the structure and found that it will add considerably to the longitudinal design forces at the fixed pier. The analysis used supports with stiffness in the longitudinal direction equivalent to preliminary bearing/substructure stiffness. It is recommended that the final design calculate and account for the force in a similar manner.

4. *Consider utilizing 3 equal spans due to the fact that the same beam design and strand arrangement will be utilized for all beams and this should result in a more economical design. The 0.7 to 0.8 span ratio, of end span to intermediate span, is a general statement that is intended for steel beams and girders. It appears that there is adequate lateral clearance from Slocum Avenue to accomplish this.*

   The attached site plan presents three equal spans. Fabricators indicated that detailing the same strand pattern for all of the beams allows them more flexibility within the casting beds. Consideration should be given to specifying the pour sequence in standard drawing PSID-1-99 to minimize cracking that could occur at the pier.

5. *Verify the bridge length (322.52'). Verify the beam length center to center of bearing. Does the bridge length take into account the distance between the centerlines of bearing at the piers? The span lengths shown in the Profile view on the Site Plan are shown as the centerline of bearing at the abutment to the centerline of the pier cap not the centerline of bearing for the beams.*

   The attached Site Plan more accurately indicates the spans are measured to the centerlines of the substructures.

6. *Can the overall bridge length be shortened by increasing the height of the breastwall (if a 5 foot high breastwall is utilized at each abutment the bridge length can be shortened by 20 feet)? At what point does the breastwall/abutment cost outweigh the savings in bridge length?*

   We have investigated shortening the superstructure by increasing the breastwall height on *SR 823 over Morris Lane-Blue Run Road (July 21, 2006)*. The construction cost analysis found that reducing the superstructure length 20' increased the construction and total ownership costs. The additional cost of the abutments and long piles, common at both structures, quickly offset the cost savings in the superstructure. This comment was discussed with OSE staff and it was generally agreed that it was not to be given additional consideration. The substructure/superstructure balance may be different with lighter steel superstructures and the higher painting cost.

7. *The result of comment numbers 5 and 6 may make it possible to decrease the size of the beam that is required.*

   Using equal spans allowed for the use of a 60" Modified AASHTO Type 4 beam. The preliminary analysis used 6ksi and 8ksi concrete strengths; similar to the 9/6/06 Type Study.

8. *When the alignment is finalized include the stationing portion of the bridge number in the Title Block.*

   The attached Site Plans include the bridge number.

9. *After the Bridge Number is determined the Structure File Number can be obtained by calling our office (Kathy Keller 752-9973).*

   The SFN will be included in the TS&L submittal.

10. *Include a detail (including the reinforcing) of the barriers in the center of the bridge in the Detail plans.*

    Comment to be given consideration by the final design consultant.

11. *Include the location (longitude and latitude) of the Structure in the Proposed Structure data block.*

    The attached Site Plans include the location of the structure.

12. *We agree that the abutments and piers should be supported on H-piles (HP14x95) with a maximum capacity of 95 tones per pile. The estimated pile length should be 140 feet and 130 feet for the rear abutment and forward abutment respectively. The estimated pile length should be 95 feet and 80 feet for the rear pier and forward pier respectively.*

    Comment to be given additional consideration upon completion of the final borings.

13. *Provide a note in the plans for any waiting period necessary prior to driving the piles.*

    The waiting period (based upon wick drain spacing) will be included in the Final Geotechnical Report along with other requirements associated with settlement.

14. *Once the final loads in the piles has been calculated the actual pile load should be included in the plans.*

    Comment to be given consideration by the final design consultant

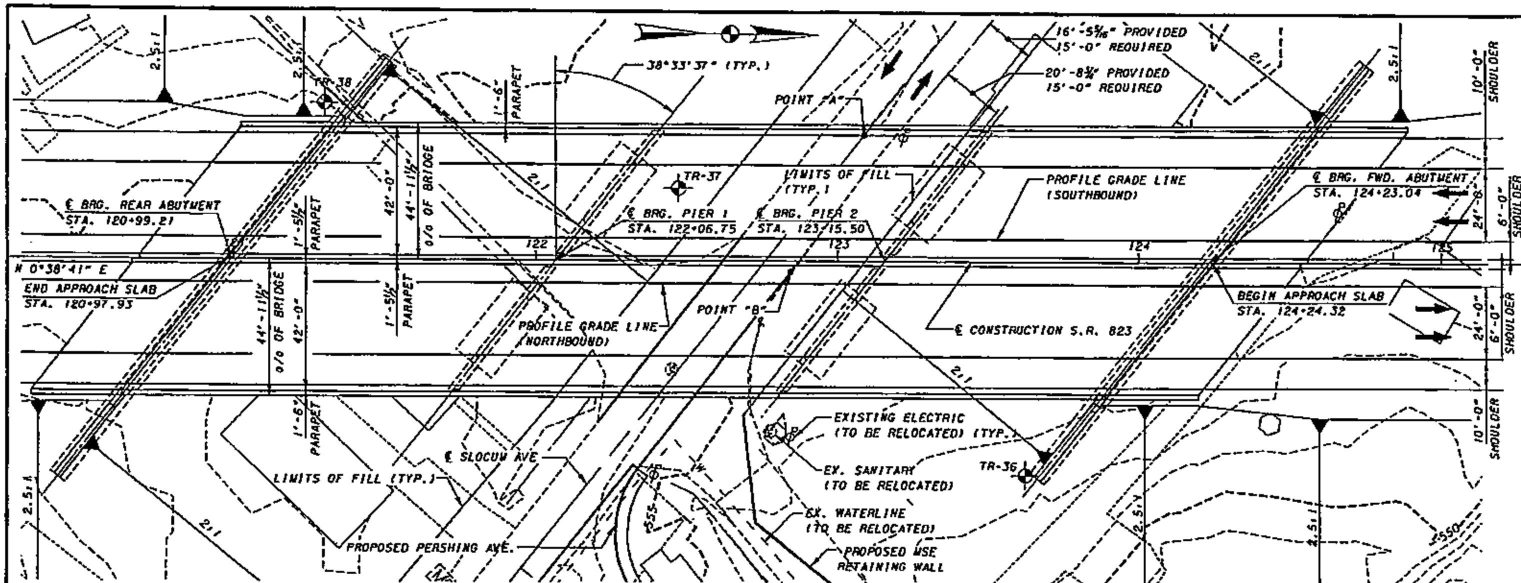
Alternative 1, a three span prestressed concrete I-girder made composite with the deck and supported on T-type piers and semi integral abutments, is recommended for further development. Furthermore, it is recommended that the span arrangement allow for all of the beams to be of equal length. Please don't hesitate to contact me or Jon Cox (513 621 1981), if there are any questions.

Sincerely,

Michael D. Weeks by JRC

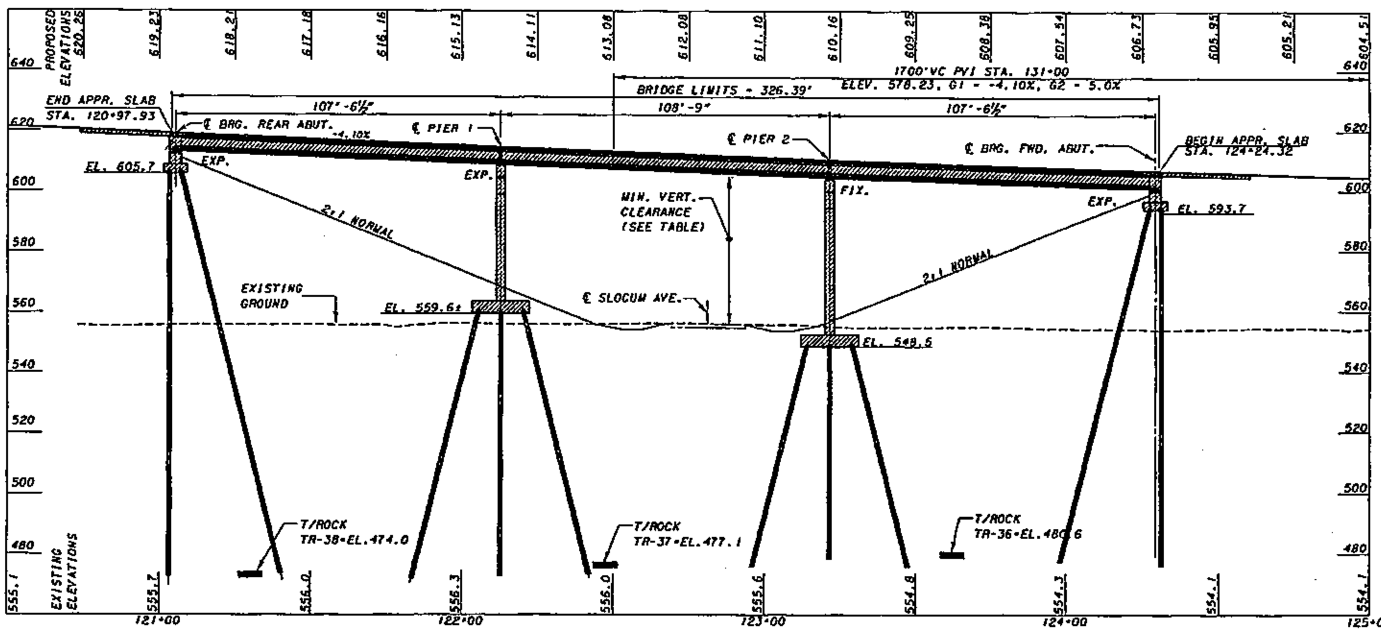
Michael D. Weeks, P.E., P.S.
Project Manager

Cc: D. Norris/J. Wetzel



PLAN

-◆- DENOTES BORING LOCATION



ELEVATION ALONG LEFT PROFILE GRADE S.R. 823

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.

TABLE OF VERTICAL CLEARANCES		
LOCATION	"A"	"B"
PROPOSED	48.35'	49.92'
PREFFERED	15.0'	15.0'

NOTES:

1. ALL SHEETS WITH PLAN DIMENSIONS ARE SHOWN HORIZONTAL.
2. EARTHWORK LIMITS SHOWN ARE APPROXIMATE. ACTUAL SLOPES SHALL CONFORM TO PLAN CROSS SECTIONS..

FOUNDATION DATA:

ALL NEW PILES SHALL BE HP 14x73 PILES AND HAVE A MAXIMUM CAPACITY OF 95 TONS PER PILE.

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

TRAFFIC DATA

S.R. 823
CURRENT YEAR ADT (2010) = 21,200
DESIGN YEAR ADT (2030) = 31,200
CURRENT YEAR ADTT (2010) = 2,968
DESIGN YEAR ADTT (2030) = 4,368

PROPOSED STRUCTURE

TYPE: 3 SPAN 60" TYPE 4 (MOD.) PRESTRESSED CONCRETE I-BEAM WITH COMPOSITE REINFORCED CONCRETE DECK SUPPORTED BY REINFORCED CONCRETE T-TYPE PIERS AND SEMI-INTEGRAL ABUTMENTS.

SPANS: 107'-6 1/2", 108'-9", 107'-6 1/2" C/C SUBSTRUCTURES

ROADWAY: 2 - 42'-0" T/T OF PARAPETS

LOADING: HS-25 AND ALTERNATE MILITARY LOADING, FWS-60 PSF

SKEW: 38°33'37" LF

CROWN: NORMAL 0.016 FT/FT

ALIGNMENT: TANGENT

WEARING SURFACE: MONOLITHIC CONCRETE

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

LATITUDE: 38°46'13" N

LONGITUDE: 82°52'36" W

DESIGN AGENCY: Trans Systems

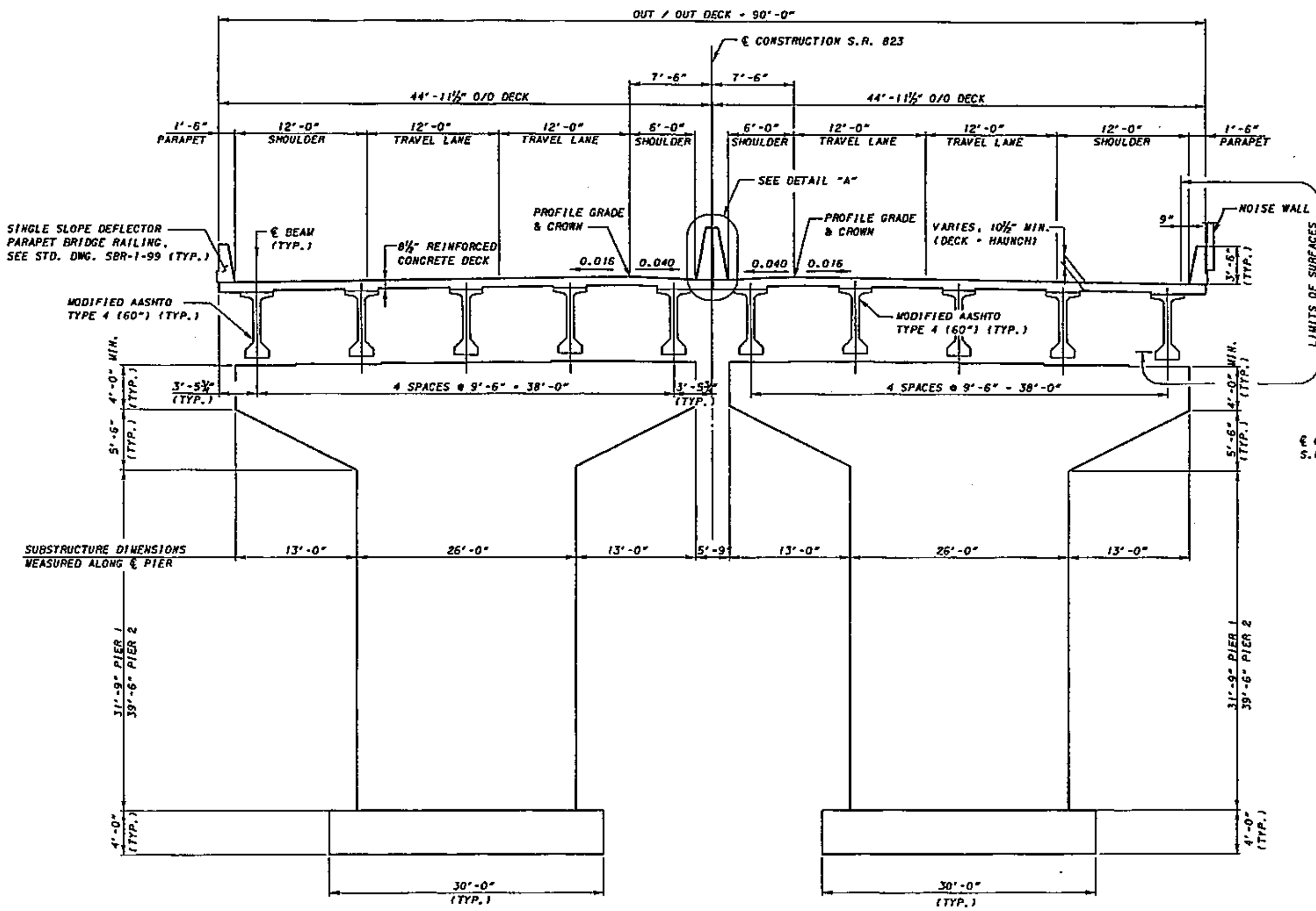
DESIGNED	CHECKED	DRAWN	REVISED	REVIEWED	DATE
PJP	MSL	AWB		JRC	11/20/06

SCIOTO COUNTY
STA. 120+97.93
STA. 124+24.32

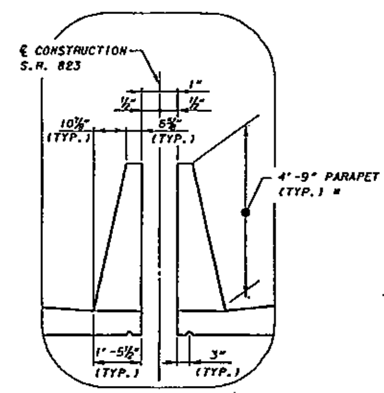
SITE PLAN - ALTERNATIVE 1
BRIDGE NO. SCI-823-0229 L&R
S.R. 823 OVER SLOCUM AVENUE (T.R. 248)

SCI-823-823-0.0
PID 77366

1 / 3



TYPICAL TRANSVERSE SECTION



DETAIL "A"
(NOT SHOWN TO SCALE)

* PARAPETS ARE SIMILAR TO TYPE A1 BARRIER FROM ROADWAY STANDARD DRAWING RM-4.3

DESIGNED	DRAWN	REVIEWED	DATE
PJP	CAS	JRC	08/31/06
CHECKED	REVISED	STRUCTURE FILE NUMBER	
MSL			

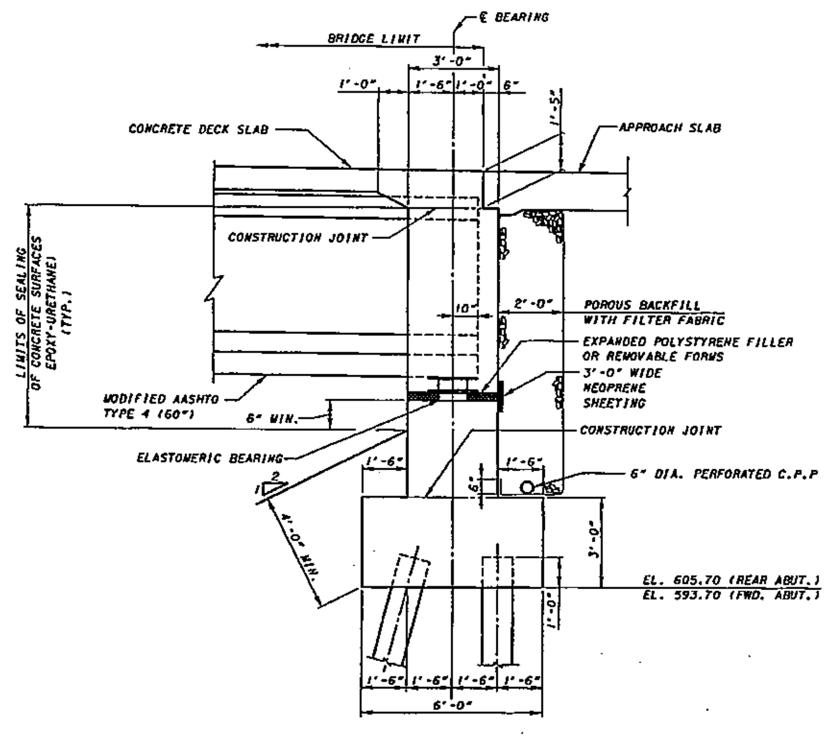
DESIGN AGENCY: Tran Systems

TRANSVERSE SECTION - ALTERNATIVE 1
BRIDGE NO. SCI-823-0229 L&R
S.R. 823 OVER SLOCUM AVENUE (T.R. 248)

SCI-823-0.00
PID 77366

2 / 3

SUPERSTRUCTURE DEPTH	
ITEM	60" MODIFIED AASHTO TYPE 4 BEAM
SLAB (INCLUDING WEARING SURFACE)	8.5"
HAUNCH (BOTTOM OF SLAB TO TOP OF FLANGE)	2"
GIRDER DEPTH	60"
TOP OF WEARING SURFACE TO BOTTOM OF GIRDER FLANGE (INCH)	70.5"
TOP OF WEARING SURFACE TO BOTTOM OF GIRDER FLANGE (FEET)	5.875'



ABUTMENT SECTION

DESIGNED PJP	DRAWN CAS	REVIEWED JRC	DATE 08/31/06
CHECKED MSL	REVISED	STRUCTURE FILE NUMBER	

TYPICAL ABUTMENT SECTION ALTERNATIVE I
BRIDGE NO. SCI-823-0229 L&R
S.R. 823 OVER SLOCUM AVENUE (T.R. 248)

SCI-823-0.00
PID 77366

3 / 3

REPORT

OF

SUBSURFACE INVESTIGATION

FOR

EMBANKMENTS (STATION 416+00 to 509+50)

PROJECT SCI-823-6.81

PHASE 1 – STAGE I

SCIOTO COUNTY, OHIO

For:

TranSystems Corporation
5747 Perimeter Drive
Suite 240
Dublin, Ohio 43017

Prepared By:

DLZ

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

DLZ Job. No. 0121-3070.03
PID No. 19415

November 29, 2006

5.4 Embankment Evaluations

5.4.1 Slope/Embankment Stability – State Route 823 Mainline

With the exception of the two interchange areas (presented under separate cover), slope/embankment stability is not considered to be a significant concern for most areas of the proposed State Route 823 mainline alignment. The following table outlines the station locations and approximate embankment heights for the proposed Phase 1 mainline embankments.

Sidehill Fill / Fill Embankments (STA. 416+00 to 509+50)

Begin Station	End Station	Approximate Maximum Fill (ft.)
434+00	449+00	44.3
457+00	479+00	70.6
483+50	497+50	58.9
504+00	507+50	34.7

Soil parameters used for the stability and settlement analyses were based on laboratory test results (grain-size and plasticity), visual examination of the preserved samples, hand penetrometer readings, and typical values. Due to the consistency of the soils encountered in this area, undisturbed Shelby tube samples were not obtained for laboratory testing. Global stability analyses and settlement calculations are presented in Appendix C.

In accordance with ODOT guidelines a unit weight of 120 pcf was used for the embankment fill materials. Due to the nature of the project, it is anticipated that the embankment fill will consist of cohesionless material ranging in size from fine granular material to rock but will generally be rock fill from adjacent cuts. The friction angles of the anticipated backfill materials will likely range from 28 degrees to over 40 degrees. We would anticipate that more of the rock fill would exhibit friction angles in excess of 40 degrees, but we conservatively selected a friction angle of 35 degrees for the embankment fill with no cohesion.

The stability analyses were performed using UTEXAS3 Version 1.204, a slope stability computer program using variations of the method of slices. UTEXAS3 was developed by Dr. Stephen Wright at the University of Texas for the U.S. Army Corps of Engineers. The Simplified Bishop procedure was used for all of the analyses and only circular failure surfaces were considered. All of the procedures use an iterative approach to investigate many failure surfaces until a critical surface is found. The results of stability analyses are included in Appendix C.

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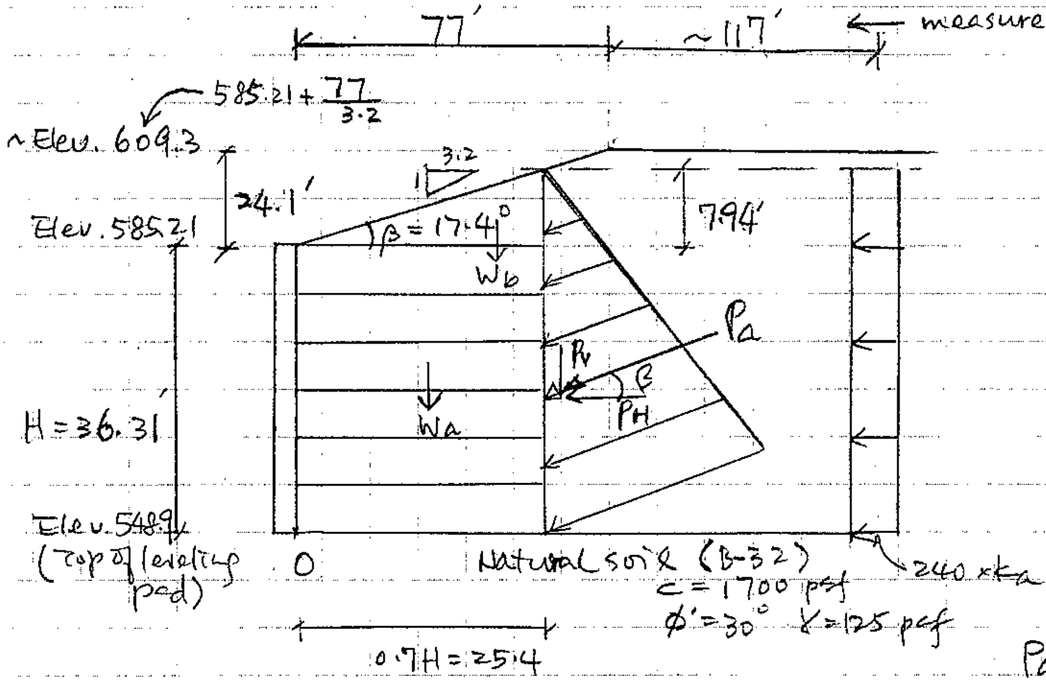
CLIENT Transystems Corps / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT MSE WALL over Slocum Ave

PROJECT NO. 0121-3070.03
SHEET NO. 1 OF 15
COMP. BY SWT DATE 10-4-07
CHECKED BY SJB DATE 10-4-07

Max wall height = 36.3' at Sta 31+03.11

β to wall $= \tan^{-1}\left(\frac{1}{3.2}\right) = 17.35°$, use 17.4°

the 1:3.2 slope was obtained by converting the 1:2.5 slope perpendicular to the wall.



Embankment Fill:
$\phi = 30°$
$\gamma = 120$ pcf

MSE Fill:
$\phi = 34°$
$\gamma = 120$ pcf

$H' = 36.31 + 7.94 = 44.25'$

$P_a = \frac{1}{2}\gamma H'^2 k_a$
$= \frac{1}{2}(120)(36.31+7.94)^2 k_a$
$= 49343$ lb/ft for $k_a = 0.42$

NAVFAC 7.2 Figure 3 Pg. 7.2-64

$$k_a = \left[\frac{\cos\phi}{1+\sqrt{\sin\phi(\sin\phi - \cos\phi\tan\beta)}}\right]^2$$

$\phi = 30°$, $\beta = 17.4°$

$k_a = 0.4189$ use 0.42

Try $L = 0.7H = 0.7 \times 36.31 = 25.417$ use 25.4'

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CLIENT: TranSystems Corps / ODOT D-9
PROJECT: SCI-823 Portsmouth ByPass over Slocum Ave
SUBJECT: MSE WALL

PROJECT NO. 0121-3070.03
SHEET NO. 2 OF 15
COMP. BY SWT DATE 10-4-07
CHECKED BY SJK DATE 10-4-07

A. Overturning

$$F.S. = \frac{\Sigma \text{ Resisting moments}}{\Sigma \text{ Overturning moments}}$$ (sum moments about "O")

$$= \frac{120 \times 25.4 \times 36.31 \times \left(\frac{25.4}{2}\right) + \frac{1}{2}(120) \times 25.4 \times 7.94 \times \frac{2}{3} \times 25.4 + P_v \times 25.4}{\left(P_h \times \frac{1}{3} \times 44.25\right)}$$

where $P_v = P_a \sin\beta = 49343 \times \sin 17.4^\circ = 14756$ lb/ft
$P_h = P_a \cos\beta = 49343 \cos 17.4^\circ = 47085$ lb/ft

$$F.S. = \frac{1985251}{694504} = 2.86 > 2.0$$ Good

B. Sliding

$$F.S. = \frac{\Sigma \text{ Resisting Forces } (P_r)}{\Sigma \text{ Driving Forces } (P_H)}$$ where P_r (drained) $= (W_a + W_b + P_v)\mu$

$\mu = \frac{2}{3}\tan\phi = 0.385$

P_r(drained) $= (120 \times 25.4 \times 36.1 + 120 \times \frac{1}{2} \times 25.4 \times 7.94 + 14756) \times 0.385$
$= 136889$ lb/ft $\times 0.385 = 52702$ lb/ft

P_r(undrained) $= cL = 1700 \times 25.4 = 43180$ lb/ft ← use this

$$F.S. = \frac{43180}{P_h} = \frac{43180}{47085}$$

$= 0.917 < 1.5$ No Good

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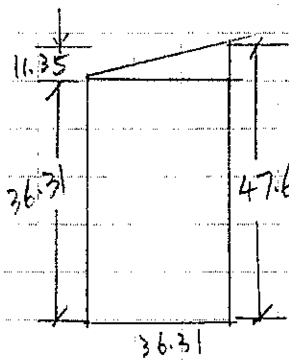
CLIENT	TranSystems Corps / ODOT D-9	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth By Pass	SHEET NO.	3 OF 15
SUBJECT	over Slocum Ave	COMP. BY	GWT DATE 10-4-07
	MSE wall	CHECKED BY	SJK DATE 10-4-07

Sliding (Cont'd)

Try $L = H = 36.31'$



P_r (drained) $= (120 \times 36.3 \times 36.3 + 120 \times \frac{1}{2} \times 36.3 \times 11.35 + P_v) \times 0.385$

Where $P_v = P_a \sin\beta$ & $P_a = \frac{1}{2}(120)(36.31 + 11.35)^2 K_a$
$= 57241$ lb/ft for $K_a = 0.42$

$= 57241 \sin 17.4°$
$= 17117$ lb/ft

$\Rightarrow P_r$ (drained) $= 76985$ lb/ft

P_r (undrained) $= cL = 1700 \times 36.3 = 61710$ lb/ft ← use this

$$F.S. = \frac{61710}{P_h} = \frac{61710}{54622}$$

Where $P_h = P_a \cos\beta = 57241 \cos 17.4° = 54622$ lb/ft

$\Rightarrow F.S. = 1.13 < 1.5$ No Good

Based on the slope stability analysis, the MSE wall will need to be constructed in stages. The following is to check the F.S. against sliding based on stage construction

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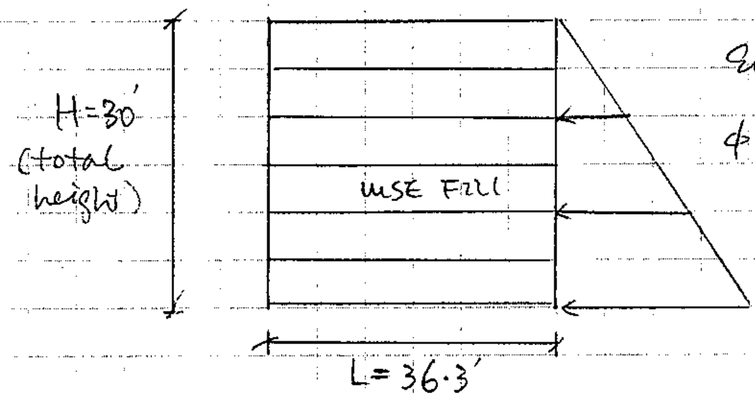
CLIENT	Transystems Corps / ODOT D-9	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth ByPass	SHEET NO.	4 OF 15
SUBJECT	over Slocum Ave MSE Wall	COMP. BY	EWT DATE 7-26-07
		CHECKED BY	SAK DATE 9-7-07

* 1st Stage Construction : Try H = 30' w/ flat backslope
and L = 36.3' (reinforcing length)
without traffic load

▸ Try H = 30' (total height including embedment depth)



(see attached calculations) for total height = 30'
- F.S. (undrained) for Bearing Capacity = 2.29 < 2.5 NG
- F.S. (undrained) for sliding = 2.86 > 1.5 ok

▸ Try H = 27' (total height including embedment depth) L=36.3'
(see Excel spreadsheets followed)
F.S. (undrained) for Bearing Capacity = 2.59 > 2.5 ok
F.S. (undrained) for sliding = 3.18 > 1.5 ok

* 2nd Stage Construction (Full Height = 36.3')
with increase in soil strength from c = 1700 psf
to c = 2636 psf (U=90%)

See previous page 2 for Overturning, F.S. = 2.86 > 2.0 Good
with c = 1700 psf & H=36.3'

Sliding

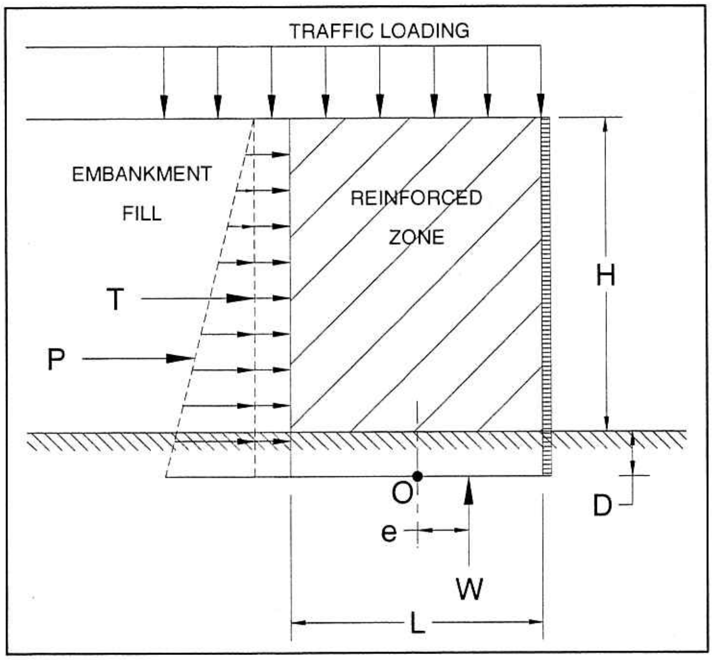
See previous page, P_r(drained) = 76985 lb/ft (for H = 36.3') for $\mu = \frac{2}{3}\tan\phi = \frac{2}{3}\tan 30° = 0.85$

& P_r(undrained) = cL = 2636 × 36.3
= 95687 lb/ft for c = 2636 psf

use P_r(drained) = 76985 lb/ft

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}	=	125	pcf	Unit weight	Foundation soil
c	=	1700	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	30	deg.	Friction ang.	Foundation soil

Loads and Parameters

L factor based on H=30 ft

ω_t	=	0	psf	Traffic loading
L=B	=	36.3	ft	Length of MSE reinforcement
L factor	=	1.21		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	30	ft	
H	=	27	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	10	ft	Moment arm
Γ Wt	=	15	ft	Moment arm
B'	=	33.58	ft	
γ'	=	62.6	pcf	
W_t		0	lb/ft of wall	Weight from traffic
W_{mse}	=	130,680	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 3,892 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 8,926 psf

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

q_{ALL} = 3,570 psf

Factor of Safety = **2.29** **No Good**

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 26,999 psf

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

q_{ALL} = 10,800 psf

Factor of Safety = **6.94** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	30.14
N_q	1.00	N_q	18.40
N_γ	0.00	N_γ	22.40

Eccentricity of Resultant Force

e = 1.36 ft

Kern

e < L/6 = 6.05 ft

DLZ

SUBJECT			
Client	ODOT9	JOB NUMBER	0121-3070.03
Project	SCI-823 Over Slocum Ave	SHEET NO.	6 OF 15
Item	MSE Wall Stability-1st Stage H=30'	COMP. BY	EWT DATE 07/27/07
		CHECKED BY	SAK DATE 9-7-07

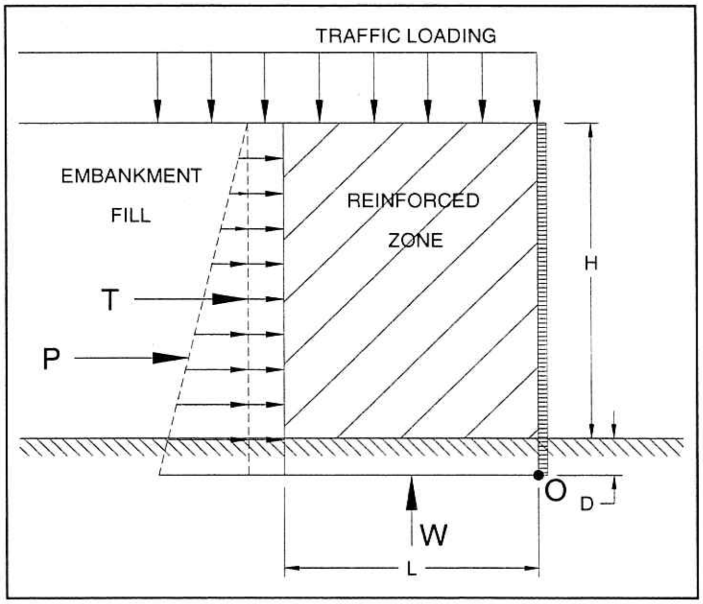
STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=30'
2 Ground water; Dw=0.0'
3 No traffic loads
4
5

Wall Properties			Foundational Soil Properties			
H+D =	30	feet	c =	1700	psf	Cohesion
γ_{mse} =	120	pcf	ϕ' =	30	deg	Friction angle
L =	36.3	feet	ω_T =	0	psf	Traffic loading
L factor =	1.21		Length factor-range (0.7 - 1.0)			
ϕ =	30	deg	Friction Angle of Embankment Fill			

RESISTANCE AGAINST SLIDING ALONG BASE



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

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

P_a = 17,820 lbs per foot of wall

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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.39

P_r = 50,965 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 61,710 lbs per foot of wall

Use Drained Value

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

Calculated FS = **2.86** 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

$\Sigma M_{resisting}$ = 2,371,842 lb-ft $\Sigma M_{resisting} = \gamma H L\left(\frac{L}{2}\right)$

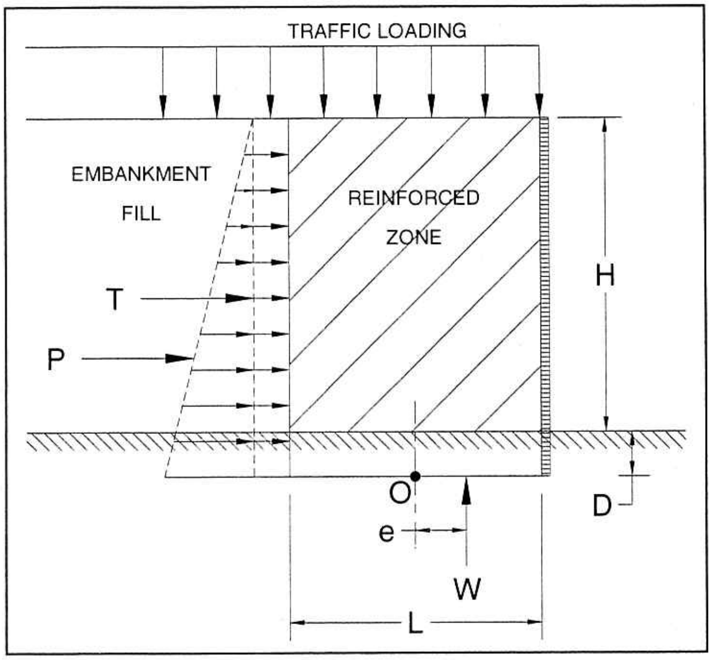
$\Sigma M_{overturning}$ = 178,200 lb-ft $\Sigma M_{overturning} = K_a\left[\frac{1}{2}\gamma H^2\left(\frac{H}{3}\right) + \omega_T H\left(\frac{H}{2}\right)\right]$

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated FS = **13.31** 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}	=	125	pcf	Unit weight	Foundation soil
c	=	1700	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	30	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	0	psf	Traffic loading
L=B	=	36.288	ft	Length of MSE reinforcement
L factor	=	1.344		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	27	ft	
H	=	24	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	9	ft	Moment arm
Γ Wt	=	13.5	ft	Moment arm
B'	=	34.09	ft	
γ'	=	62.6	pcf	
W_t		0	lb/ft of wall	Weight from traffic
W_{mse}	=	117,573	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 3,449 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 8,926 psf

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

q_{ALL} = 3,570 psf

Factor of Safety = **2.59** **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} = 27,357 psf

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

q_{ALL} = 10,943 psf

Factor of Safety = **7.93** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	30.14
N_q	1.00	N_q	18.40
N_γ	0.00	N_γ	22.40

Eccentricity of Resultant Force

e = 1.10 ft

Kern

e < L/6 = 6.05 ft

DLZ

SUBJECT			
Client	ODOT9	JOB NUMBER	0121-3070.03
Project	SCI-823 Over Slocum Ave	SHEET NO.	8 OF 15
Item	MSE Wall Stability-1st Stage H=27'	COMP. BY	EWT DATE 07/27/07
		CHECKED BY	SJK DATE 9-7-07

STABILITY OF MSE WALL

Assumptions:

1 Estimated height of embankment; H=27'
2 Ground water; Dw=0.0'
3 No traffic loads
4
5

Wall Properties

H+D = 27 feet
γ_{mse} = 120 pcf
L = 36.288 feet
L factor = 1.34
ϕ = 30 deg

Foundational Soil Properties

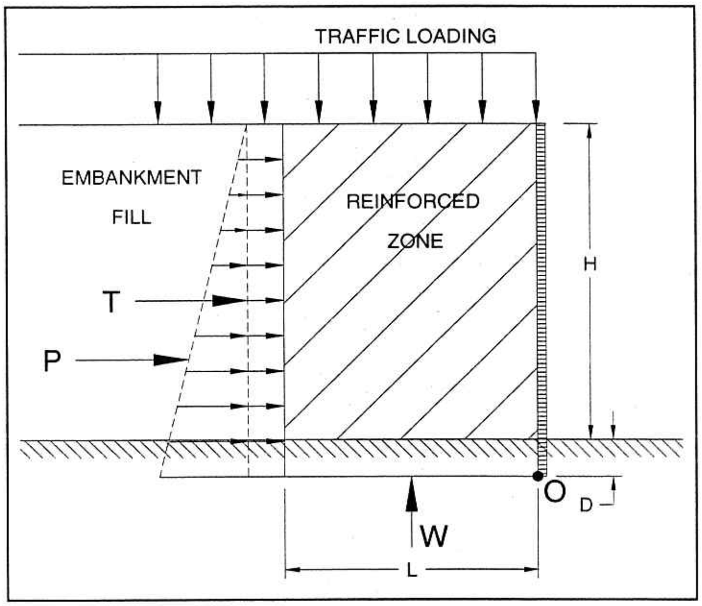
c = 1700 psf Cohesion
ϕ' = 30 deg Friction angle
ω_T = 0 psf Traffic loading
Length factor-range (0.7 - 1.0)
Friction Angle of Embankment Fill

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 = 14,434 lbs per foot of wall



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

where; $\mu = \left(\frac{2}{3}\right)\tan(\phi)$ μ = 0.39

P_r = 45,854 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 61,690 lbs per foot of wall

Use Drained Value

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

Calculated FS = **3.18** 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

$\Sigma M_{resisting}$ = 2,133,247 lb-ft $$\Sigma M_{resisting} = \gamma HL\left(\frac{L}{2}\right)$$

$\Sigma M_{overturning}$ = 129,908 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 = **16.42** Required FS = 2.00

Resistance Against Overturning is **OK**

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CLIENT: Transystems Corps / ODOT D-9 PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth ByPass SHEET NO. 9 OF 15
SUBJECT: over Slocum Ave MSE Wall COMP. BY QWT DATE 10-4-07
CHECKED BY SJK DATE 10-4-07

Continued from page 4 of this section:

$$F.S._{sliding} = \frac{76985}{54622 = P_h} = 1.41 < 1.5 \quad \underline{NG} \quad (H = 36.3')$$

use $u = \tan\phi$ instead of $u = \frac{2}{3}\tan\phi$

for non-continuous reinforcement

$$u = \tan\phi = 0.577$$

$$P_{r\,(drained)} = 76985 \times \frac{0.577}{0.385} = 115377 \text{ lb/ft}$$

$$P_{r\,(undrained)} = 95687 \text{ lb/ft} \quad \text{for } c = 2636 \text{ psf}$$

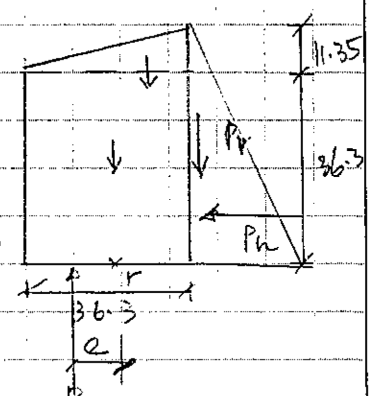
use P_r (undrained) $= 95687$ lb/ft

$$F.S._{sliding} = \frac{95687}{54622} = 1.75 > 1.5 \quad \underline{Good} \quad (H = 36.3')$$

Bearing Capacity

$$\Sigma F_y = 0 = (120 \times 36.3 \times 36.3) + (120 \times \tfrac{1}{2} \times 36.3 \times 11.35) + 17117 - R$$

$$\Rightarrow R = 199960 \text{ lb/ft}$$



$$\Sigma M_r = 0 = R \times e + \tfrac{1}{2}(120)(36.3)(11.35)\left(\frac{36.3}{2} - \frac{36.3}{3}\right) + 17117 \times \frac{36.3}{2} - 47085 \times \left(\frac{36.3 + 11.35}{3}\right)$$

$$\Rightarrow R \times e = 747867 - 149558 - 310674 = 287635$$

$$\Rightarrow e = 287635 / 199960 = 1.438 \text{ ft} < \frac{L}{6} = \frac{36.3}{6} = 6.05'$$

$P_v = 17117$ lb/ft

$P_h = P_a \cos\beta = 49343 \cos 17.4° = 47085$ lb/ft

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CLIENT	TranSystems Corps / ODOT9	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth By Pass	SHEET NO.	10 OF 15
SUBJECT	over Slocum Ave	COMP. BY	EWT DATE 10-4-07
	MSE wall	CHECKED BY	SJR DATE 10-6-07

$$\sigma_v = \frac{R}{L-(2\times e)} = \frac{199960}{36.3-(2\times 1.4387)} = 5982.5 \text{ psf}$$

say 5983 psf

$$q_{ult}\text{ (undrained)} = C N_c = 5.14(2636) = 13549 \text{ psf}$$

$$q_{all} = \frac{q_{ult}}{F.S.} = \frac{13549}{2.5} = 5420 < 5983 \quad \underline{NG}$$

(H = 36.3')

$$F.S. = \frac{13549}{5983} = 2.26 < 2.5 \quad \underline{NG}$$

(H = 36.3')

$$q_{ult}\text{ (drained)} = \frac{1}{2}\gamma B N_\gamma = \frac{1}{2}\times 125\times(36.3-2\times 1.438)\times 22.4$$

22.4 for $\phi = 30°$

$$= 46794 \text{ psf}$$

$$q_{all} = \frac{q_{ult}}{2.5} = \frac{46794}{2.5} = 18718 > 5983 \quad \underline{OK}$$

(H = 36.3')

$$F.S. = \frac{46794}{5983} = 7.82 \quad \underline{OK}$$

To achieve F.S. bearing capacity of 2.5, the required undrained shear strength is:

$$C = (2.5\times 5983)/5.14 = 2910 \text{ psf}$$

Need staged construction to improve undrained shear strength.

If $c = 2910$; $q_{ult}\text{ (undrained)} = 5.14\times 2910 = 14957$ psf

$$q_{all} = \frac{q_{ult}}{F.S.} = \frac{14957}{2.5} = 5983 = 5983 \text{ req'd}$$

$\underline{OK}$

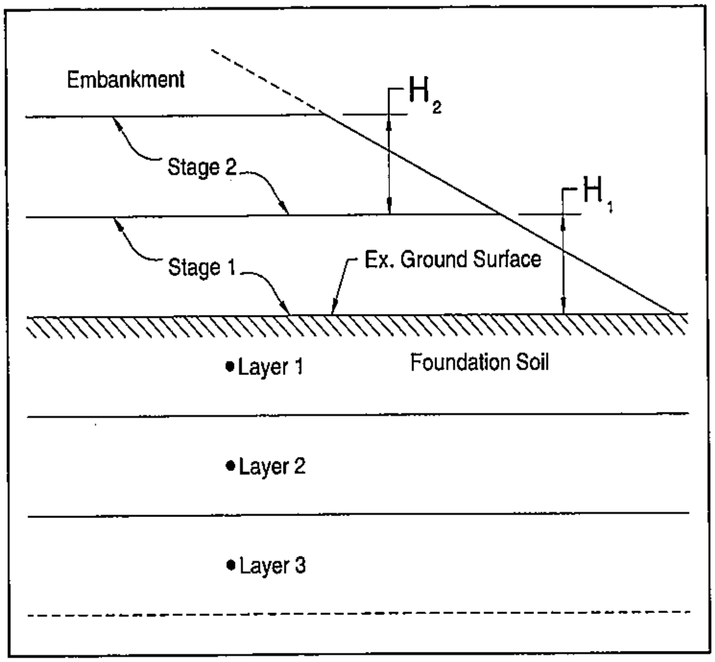
$$F.S. = \frac{14957}{5983} = 2.5 = 2.5$$

∴ req'd ok

Determine Increase in Undrained Shear Strength Due to Consolidation

Undrained Strength Analysis - Staged Construction

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



Increase in Undrained Shear Strength from consolidation

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

Where: c_{ui} Initial undrained shear strength, UU or q_u testing
ϕ_{cu} Determined from CIU testing
$\Delta\sigma'$ Effective stress increase due to embankment loading

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

Where: U Average degree of consolidation (%)
H_n Height of Embankment, Stage n (ft)

Embankment Fill

γ_{fill} **120** pcf

It is assumed that fill material is granular

Construction Option: 27'/9'

Stage 1 Embankment First Stage Embankment Height H_1= **27.0** Average Percent Consolidation U= **90%**

Depth	Soil Type	Initial Undrained Shear Strength, c_{ui} (psf)	$\Delta\sigma'$ (psf)	ϕ_{cu} (deg)	Δc_u (psf)	c_u (psf), *After Consolidation*	Percent Increase
	#1 Clay	1700	2916	17.8	936	2636	55%
	#2 Silt	1656	2916	17.0	892	2548	54%
	#3 Silty Clay	1125	2916	13.4	695	1820	62%

Stage 2 Embankment Second Stage Embankment Height H_2= **9.0** Average Percent Consolidation U= **80%**

	#1 Clay	2636	864	17.8	277	2913 > 2910	11%
	#2 Silt	2548	864	17.0	264	2812 OK	10%
	#3 Silty Clay	1820	864	13.4	206	2026	11%

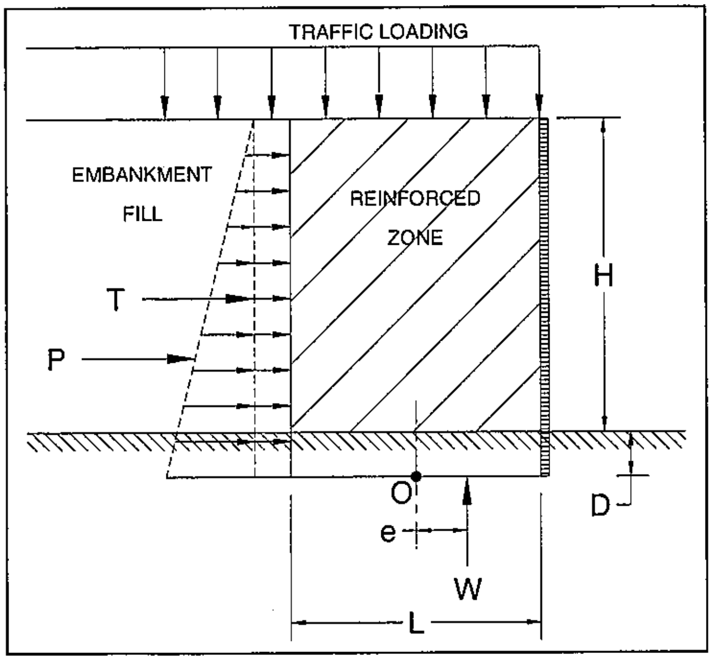
Stage 3 Embankment Third Stage Embankment Height H_3= Average Percent Consolidation U=

DLZ

SUBJECT			
Client	ODOT9	JOB NUMBER	0121-3070.03
Project	SCI-823 Over Slocum Ave	SHEET NO.	12 OF 15
Item	MSE Wall Bearing Capacity-1st Stage H=36.3"	COMP. BY	EWT DATE 7/27/07
	Flat backfill with increasd undrained shear strengtl	CHECKED BY	SJK DATE 9-7-07

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}	=	125	pcf	Unit weight	Foundation soil
c	=	2636	psf	Cohesion	Foundation soil
ϕ	=	0	deg.	Friction ang.	Foundation soil
c'	=	0	psf	Cohesion	Foundation soil
ϕ'	=	30	deg.	Friction ang.	Foundation soil

Loads and Parameters

ω_t	=	0	psf	Traffic loading
L=B	=	36.3	ft	Length of MSE reinforcement
L factor	=	1		Length factor-range (0.7 - 1.0)
D	=	3	ft	Embedment depth
Dw	=	0	ft	Groundwater depth
H+D	=	36.3	ft	
H	=	33.3	ft	Height of wall
Ka	=	0.33		
Γ Pa	=	12.1	ft	Moment arm
Γ Wt	=	18.15	ft	Moment arm
B'	=	32.30	ft	
γ'	=	62.6	pcf	
W_t		0	lb/ft of wall	Weight from traffic
W_{mse}	=	158,123	lb/ft of wall	Weight from MSE wall

Effective Bearing Pressure

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

σ_v = 4,895 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 13,737 psf

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

q_{ALL} = 5,495 psf

Factor of Safety = **2.81** **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} = 26,102 psf

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

q_{ALL} = 10,441 psf

Factor of Safety = **5.33** **OK**

Bearing Capacity Factors for Equations (AASHTO)

Undrained		Drained	
N_c	5.14	N_c	30.14
N_q	1.00	N_q	18.40
N_γ	0.00	N_γ	22.40

Eccentricity of Resultant Force

e = 2.00 ft

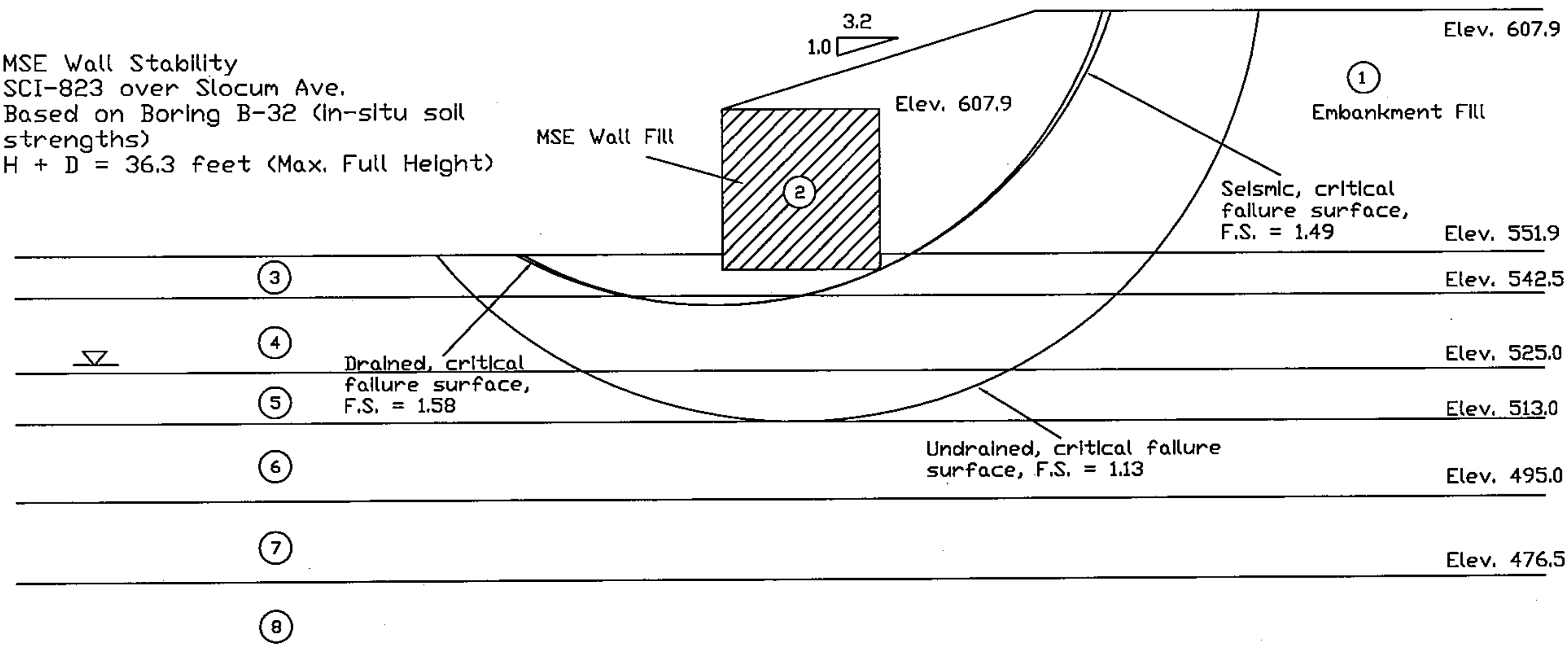
Kern

e < L/6 = 6.05 ft

13/15
EWT 10-4-07

Material	Consistency	Soil Type	Undrained C (psf)	Undrained φ (deg)	Drained C' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2		MSE Fill	0	34	0	34	120
Material 3	Stiff	Clay	1700	0	0	30	125
Material 4	Stiff	Silt	1656	0	0	30	125
Material 5	Stiff	Silty Clay	1125	0	0	28	120
Material 6	V. Stiff	Clay	2700	0	0	29	125
Material 7	M. Dense	C & F Sand	0	32	0	32	120
Material 8		Sandstone	5000	45	5000	45	150

MSE Wall Stability
SCI-823 over Slocum Ave.
Based on Boring B-32 (in-situ soil strengths)
H + D = 36.3 feet (Max. Full Height)

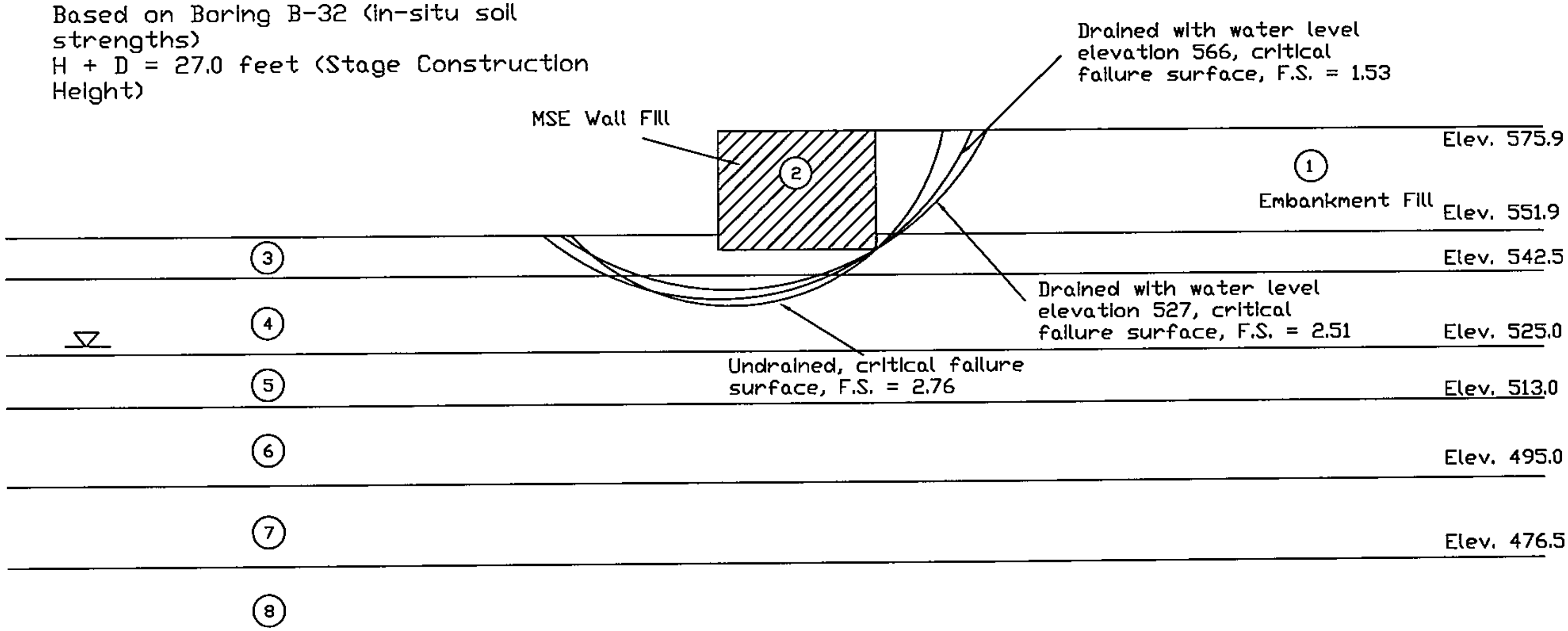


SCI-823 OVER SLOCUM AVENUE STABILITY ANALYSES BORING B-32 (IN-SITU SOIL STRENGTHS)		
MSE WALL STABILITY ANALYSES		
PROJECT NO. 0121-3070.03	CALC: EWT	DATE 07/30/07

14/15
EWT 9-7-07

Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2		MSE Fill	0	34	0	34	120
Material 3	Stiff	Clay	1700	0	0	30	125
Material 4	Stiff	Silt	1656	0	0	30	125
Material 5	Stiff	Silty Clay	1125	0	0	28	120
Material 6	V. Stiff	Clay	2700	0	0	29	125
Material 7	M. Dense	C & F Sand	0	32	0	32	120
Material 8		Sandstone	5000	45	5000	45	150

MSE Wall Stability - Stage Construction
SCI-823 over Slocum Ave.
Based on Boring B-32 (in-situ soil strengths)
H + D = 27.0 feet (Stage Construction Height)



SJK 9-7-07

SCI-823 OVER SLOCUM AVENUE
STAGE CONSTRUCTION
BORING B-32 (IN-SITU SOIL STRENGTHS)

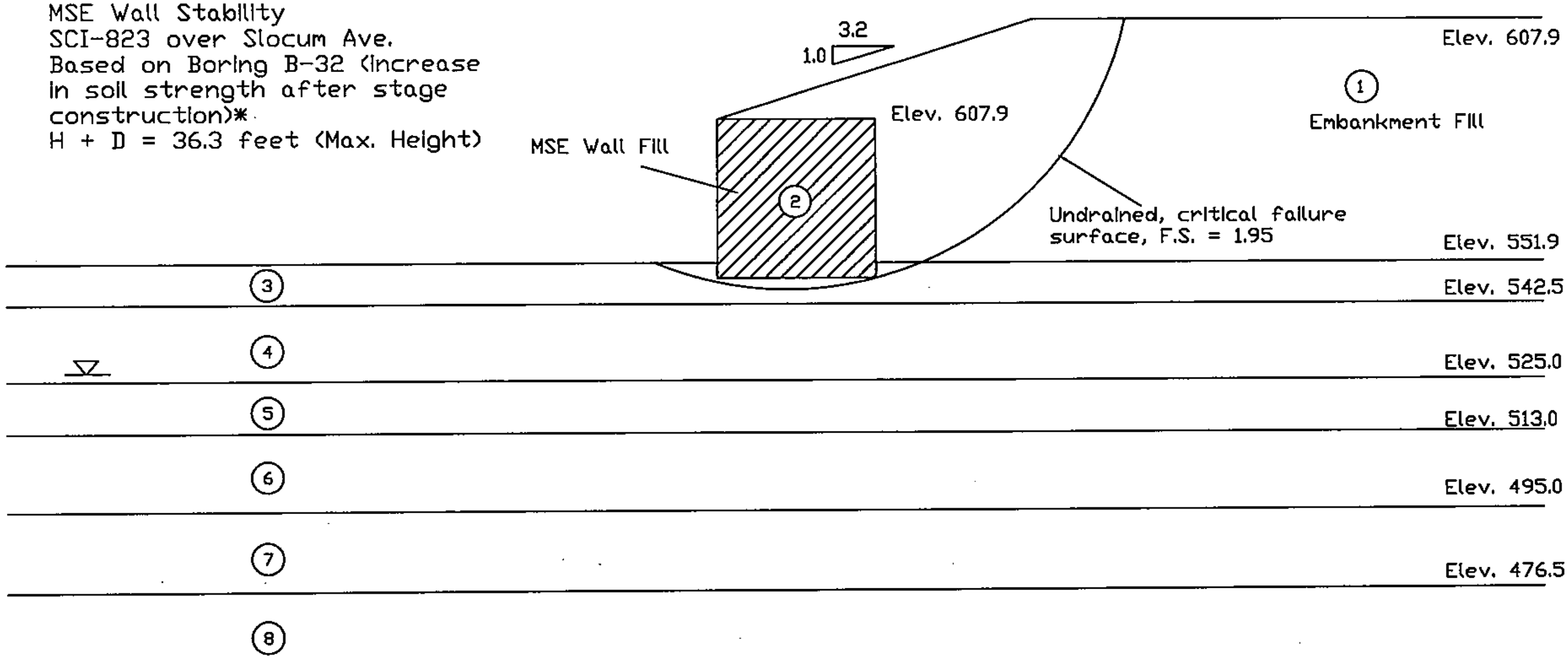
MSE WALL STABILITY ANALYSES

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

15/15
EWT 10-4-07

Material	Consistency	Soil Type	Undrained C (psf)	Undrained φ (deg)	Drained C' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2		MSE Fill	0	34	0	34	120
Material 3	Stiff	Clay	2636*	0	0	30	125
Material 4	Stiff	Silt	2548*	0	0	30	125
Material 5	Stiff	Silty Clay	1820*	0	0	28	120
Material 6	V. Stiff	Clay	2700	0	0	29	125
Material 7	M. Dense	C & F Sand	0	32	0	32	120
Material 8		Sandstone	5000	45	5000	45	150

MSE Wall Stability
SCI-823 over Slocum Ave.
Based on Boring B-32 (increase in soil strength after stage construction)*
H + D = 36.3 feet (Max. Height)



SCI-823 OVER SLOCUM AVENUE
STABILITY ANALYSES - BORING B-32
WITH INCREASE IN SOIL STRENGTHS

MSE WALL STABILITY ANALYSES

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