Retaining Wall Type Study

Walls 1 & 2

SCI-823-0.00
PID No. 19415

Prepared for

Ohio Department of Transportation

March 2007

CH2MHILL

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1. Introduction

On July 14, 2005, CH2M HILL submitted Structure Type Studies for the Ramp B over Fairground Road, Ramp C over Fairground Road, and SR-823 over Fairground Road structures located at the proposed US 23/SR 823 Interchange. These three structures were designed to have both abutments supported behind a Mechanically Stabilized Embankment (MSE) wall due to not only the inexpensive nature of this type of wall construction, but also the reduced bridge costs, including life cycle maintenance costs. Subsequent ODOT review comments of the Structure Type Studies in August / September 2005 recognized the economic benefit of the recommended MSE wall abutments; however, ODOT Office of Structural Engineering (OSE) commented that *"The Design Consultant shall first determine that MSE wall supported abutments can be utilized at the proposed location prior to making any MSE wall recommendations during the Structure Type Study. Subsurface soil conditions are to be evaluated for expected settlements, differential settlements, allowable bearing capacities and global stability of the proposed MSE walls prior to submitting Structure Type Study to our office."*

All retaining wall justification and wall type studies were to be conducted by another consultant and coordinated with CH2M HILL's Structure Type Study effort. Since a Wall Type Study was not submitted, the aforementioned structures over Fairground Road have not been approved by OSE to-date. In December 2006, the Wall Type Study work was transferred to CH2M HILL. To assist ODOT OSE in performing a comprehensive review of this report, the revised Structure Type Studies for the structures over Fairground Road are submitted concurrently with this Wall Type Study.

In October 2006, the project's geotechnical consultant, DLZ, submitted a revised *"Subsurface Exploration and MSE Wall and Embankment Evaluations for Proposed US 23/SR 823 Interchange"* report, which included the design calculations requested by ODOT OSE. The report concluded that *"MSE walls can be safely constructed using staged construction and ground modification techniques at this interchange. However, due to the relatively poor subsurface conditions, the risk of detrimental differential settlement is greater when constructing the MSE walls using staged construction."* Due to concerns over the existing soil conditions at the proposed interchange location, additional ground improvement and/or wall alternatives were investigated in this Wall Type Study in conjunction with revising the original Structure Type Studies for this location. To determine the most economical solution, various bridge layouts and types were matched with these walls/ground improvement alternatives. For a summary of the wall/ground improvement alternatives presented by DLZ, see Appendix A. Included in Appendix A are the wall stability calculations, settlement calculations, and the results of laboratory testing. Also outlined within this document are the costs associated with the retaining walls and associated ground improvement techniques investigated. Furthermore, the associated cost of roadway embankment for each alternative is also included in this Wall Type Study.

2. Description of Bridge / Wall Alternatives Investigated

Five (5) bridge/wall alternatives were considered to determine the most economical, combined structural system:

1. Single span bridge behind MSE Walls constructed on soil that has been surcharged in stages;

2. Single span bridge behind MSE Walls utilizing deep soil mixing for ground improvement;
3. Three span bridge behind 2:1 spill-through slopes;
4. Single span bridge behind 2:1 spill-through slopes; and
5. Single span bridge behind pile-supported, reinforced CIP walls on soil that has been surcharged

The estimated initial bridge construction cost and projected life cycle maintenance cost of each bridge over Fairground Road is presented in its respective Structure Type Study revision. Included within this Wall Type Study are the estimated ground improvement and retaining wall costs for each alternative, as well as the associated roadway embankment costs.

3. Summary of Retaining Wall Unit Costs

Retaining wall unit costs were established for the three alternatives in which retaining walls would be required (Alternatives 1, 2, & 5). The cost of the associated ground improvement technique (either soil surcharging in stages or deep soil mixing) is included with each respective unit cost. See Appendix B for detailed calculations and sketches utilized to calculate the wall unit costs for each alternative.

Due to similar wall heights, ground improvement and wall costs are expected to be very similar between Wall 1 and Wall 2 for Alternative 1. For this reason, one wall unit cost was established for both walls. The unit cost for constructing MSE walls for Alternative 1 was calculated to be $150 per square foot of exposed wall face, which includes ground improvement costs.

For Alternative 2, the deep soil mixing will extend from a depth 5' below the existing ground down to bedrock; boring logs indicate that the top 5' of existing soil is suitable material and does not require additional ground improvement. The depth of bedrock at Wall 2 is approximately 11' lower than at Wall 1. For this reason, a different wall unit cost was established for each wall for Alternative 2. The unit cost for Alternative 2 was calculated to be $130 per square foot of exposed wall face for MSE Wall 1 and $180 per square foot of exposed wall face for MSE Wall 2. These unit prices include ground improvement costs.

The wall unit cost for Alternative 5 was determined by first calculating an average height of the CIP wall. This average height was based on the portions of the CIP wall that are not acting as bridge abutment seats; abutment portions of each wall were included with the cost of the respective bridge. The average height of both walls was approximately 20' above existing ground. For this reason, one wall unit cost was established for both walls. The CIP wall unit cost for Alternative 5 was calculated to be $165 per square foot of exposed wall face, which includes ground improvement costs.

Wall unit costs were then multiplied by the exposed square footage of each wall to yield a total wall cost. A comparison of the total wall costs is outlined in the table below.

Alternative	Wall 1 Cost	Wall 2 Cost	Total Wall Cost
Alt. 1	$780,000	$923,000	$1,703,000
Alt. 2	$676,000	$1,107,000	$1,783,000
Alt. 5	$587,000	$737,000	$1,324,000

4. Summary of Non-Retaining Wall Embankment Costs

In order to compare a longer bridge that has a spill-through slope with a shorter bridge supported behind retaining walls, it is important that the cost of additional roadway embankment required for the shorter bridge be accounted for in each alternative being considered. Included in the aforementioned wall unit costs are volumes of embankment that are required to build either the temporary MSE wall or the conventional, permanent MSE wall. It is anticipated that the embankment used during the staged surcharging will also be utilized when constructing the permanent retaining wall. The limits of embankment included with the wall costs are detailed in the typical wall sketches found in Appendix B. The boundary of embankment left to be quantified lies between the limits of embankment included with the wall costs and the ends of the approach slabs for the longest alternative, a three-span structure with spill-through slopes. In order to quantify the non-retaining wall embankment, cross-sections were cut across all three roadway alignments and the average end area method was used to calculate a volume of embankment. The non-retaining wall embankment quantities and costs are outlined in Appendix C. A comparison of non-retaining wall embankment costs is outlined in the table below.

Alternative	Non-Ret. Wall Embankment
Alt. 1	$228,000
Alt. 2	$338,000
Alt. 3	$342,000
Alt. 4	$491,000
Alt. 5	$375,000

5. Evaluation of Alternatives

Alternatives 1 and 5 both involve soil surcharging constructed in 10-foot stages to maintain adequate undrained bearing capacity. DLZ also recommends that settlement plates and piezometers be installed to monitor consolidation and pore pressures in clay layers. Wick drains are recommended in order to shorten the time duration required to achieve 90 percent consolidation. When the surcharge embankment is removed, it is anticipated that the foundation soils will rebound slightly before they consolidate again under the weight of the new wall and fill. Some differential settlement is expected, although it is anticipated that it will be within the accepted tolerances of the MSE wall.

Alternative 2 involves deep soil mixing, which would create a concrete/soil mass capable of providing suitable bearing for conventional MSE retaining walls. After the soil is treated, it is expected that the MSE wall will be able to be constructed with negligible settlement.

6. Recommended Alternative

Construction costs for each alternative have been developed for an identical length of wall improvement, equal to the length of the longest bridge alternative. Estimated construction costs for each alternative include all proposed structures and wall work between these limits. A cost matrix summary is included in Appendix E. Alternative 1 is approximately $22,000 cheaper than Alternative 2. Qualitatively, there are two distinct differences between Alternative 1 and Alternative 2: *construction time and construction risk*. The staged construction nature of Alternative 1 will add additional construction time to the schedule,

due to the need to consolidate the existing subsurface in stages prior to construction of the permanent MSE Walls; quantitatively speaking, the additional construction time is dependent upon the use of wick drains, and if used, to what extent. In addition, per geotechnical consultant, DLZ, the relatively poor subsurface conditions increase the risk of detrimental differential settlement when constructing the MSE walls using staged construction. Soil mixing ground improvement, as used in Alternative 2, would lower construction risk and future maintenance problems associated with MSE wall construction. As a result, based on low estimated total ownership costs and lower qualitative costs in construction time and construction risk, CH2M HILL **recommends that the single-span bridges behind MSE walls built with deep soil mixing ground improvement of ALTERNATIVE 2, be constructed for the three structures over Fairground Road.**

APPENDIX A

DLZ
ENGINEERS • ARCHITECTS • SCIENTISTS
PLANNERS • SURVEYORS

March 29, 2007

Mr. Rob Miller, AICP
Project Manager
CH2M Hill
5775 Perimeter Drive Suite 190
Dublin, Ohio 43017

Re: **SR 823 and US 23 Interchange – Fairgrounds Road Structures**
Preliminary Retaining Wall and Bridge Foundation Recommendations
Project SCI-823-0.00
DLZ Job No.: 0121-3070.03

Dear Mr. Miller:

This letter reports additional preliminary recommendations for the proposed retaining walls and bridge foundations at the SR 823 and Fairgrounds Road site. This document is an addendum to our report of Preliminary Subsurface Exploration and MSE retaining wall and Embankment Evaluations, dated October 4, 2006. Additionally, this document presents alternative wall types and ground improvement techniques that could be employed at this site. This document presents options for walls 1 and 2, adjacent to Fairgrounds Road only. Recommendations for other retaining walls at the interchange will be presented in separate documents.

It is anticipated that three proposed bridges will span existing Fairgrounds Road. It is understood that one structure each will be required for Ramp B, Ramp C, and Mainline SR 823.

The findings and recommendations presented in this document should be considered preliminary. After the structure and wall configurations have been finalized, additional borings will be necessary to finalize the structure and retaining wall recommendations.

Preliminary Abutment Retaining Wall Recommendations – Fairgrounds Road Structures

As outlined in the October 4, 2006 report, DLZ recommended that MSE walls, built using staged construction and wick drains, were the most economical solution for the walls at the proposed interchange. However, as stated in the report, the subsurface conditions at the site are marginal for MSE walls and there is a significant risk of detrimental settlement occurring over time. In addition, it is anticipated that the final wall borings may reveal subsurface conditions that are poorer than those encountered by the preliminary borings, resulting in excessive settlements that may preclude MSE walls from being used.

Representatives of CH2M Hill expressed concern about the shear strength selection of the foundation soils of this site. At the request of CH2M Hill, DLZ has elected to assume more conservative values to carry out the preliminary analyses and to develop design parameters. The assumed values were based upon soil conditions encountered in boring B-1133. It should be noted that an extensive testing program (including in-situ testing) will be executed for "approved" structure and wall configurations to more accurately determine the appropriate shear strengths for use in analyses and design.

Consequently, we have re-evaluated the subsurface conditions and have analyzed an MSE wall using the conditions encountered by boring B-1133. The revised analyses indicate that MSE walls could be built in approximately ten-foot stages while maintaining adequate undrained bearing capacity. Additionally, primary consolidation is estimated to be approximately 9 inches (at the wall face). Differential settlement is estimated to be greater than 1.0 percent, which is typically considered to be the maximum allowable differential settlement. In addition to primary consolidation, secondary compression settlement was evaluated, and was found to be less than 1 inch over 75 years (service life). Consequently, secondary compression settlement is not considered to be of significant concern at this site. The results of bearing capacity, MSE stability (sliding and overturning), and settlement calculations are attached. Also, the results of MSE and embankment global stability results are attached.

Based upon the risk associated with using conventional MSE walls at this site, even with staged construction, we offer the following preliminary alternative recommendations for the proposed abutment retaining walls at the Fairgrounds Road site.

Option 1
Preload with Temporary Geotextile/Fabric-faced Wall and Build Conventional MSE Wall

As stated previously, primary consolidation has been estimated to be approximately 9 inches at the proposed wall face. A preloading (surcharge) embankment could be constructed at the Fairgrounds Road site to consolidate any soft and compressible foundation soils. Fabric-faced walls may be built with vertical or nearly vertical slopes (1H:20V batter) to allow preloading of soils near the existing road. Preliminary analyses indicate that the surcharge load must be constructed in 10-foot stages to maintain adequate undrained bearing capacity. It is recommended that settlement plates and piezometers be installed to monitor consolidation and pore pressures in clay layers.

Based on the preliminary results of consolidation tests at the site, the time to 90 percent consolidation (without wick drains) has been estimated to be approximately 110 days. This duration can be shortened through the use of wick drains. Wick drain spacing and resulting consolidation times (90 percent consolidation) are presented in the table below.

Time Rate of Consolidation Estimates Walls 1 and 2

Wall Locations	t_{90} Without Wick Drains (days)	Spacing (ft)	t_{90} With Wick Drains (days)
SR-823 over Fairground Rd	110	5.0	30
		7.0	45
		9.0	60

Wick drain treatment areas should extend 10 feet beyond the limits of the retaining walls, and be advanced to the top of rock.

The surcharge embankment should remain in place until at least 90 percent of primary consolidation has occurred. Once the surcharge embankment has been removed, construction of the MSE wall may commence. The MSE walls should also be constructed in 10-foot stages to maintain adequate undrained stability. When the surcharge embankment is removed, it is anticipated that the foundation soils will rebound slightly before they consolidate again under the weight of the new MSE wall and fill. Settlement calculations using the recompression index for the fine-grained foundation soils indicate that the primary consolidation beneath the new MSE wall will be approximately 2 inches with differential settlement being approximately 0.4 percent.

Fill material should be selected that can be used for both the surcharge embankment and the conventional MSE wall backfill. Also, consideration must be given to the degradation of the geotextile fabric when exposed to UV light. The selected fabric must be able to withstand the planned exposure to UV light during the service of the temporary surcharge walls. If degradation due to UV exposure is of significant concern, a temporary cover such as shotcrete or a UV resistant fabric cover (exposed face only) should be considered.

Option 2
Deep Soil Mixing (Grouting) with Conventional MSE Retaining Walls

Soil mixing may also be considered to strengthen the foundation soils. The deep soil mixing would create a concrete/soil mass, which would provide suitable bearing for conventional MSE retaining walls. The treatment area should extend approximately 10 feet beyond the limits of the retaining wall fill, and the soil mixing should extend to the top of bedrock. After the soil is treated, the MSE wall can be constructed with negligible settlement. For preliminary cost estimating purposes, 80 percent replacement (mixing) should be assumed in the areas to be treated.

Option 3
Preload with Temporary Geotextile/Fabric-faced Wall and Build Pile-Supported, Reinforced Concrete Retaining Walls

Pile-supported walls could be considered for these locations. If the piles are driven to bedrock, the settlement of the walls founded on piles would be negligible. However, the embankments behind the walls would settle, resulting in potential distortion of the new retaining wall and differential settlement between the wall and the embankment fill. Consequently, to reduce this differential settlement, it is recommended that the foundation soils be surcharged and allowed to consolidate prior to constructing the walls. Fabric-faced walls may be used to surcharge the soils near the existing road. These walls should be built according to the recommendations outlined in Option 1 on page 2.

If Option 3 is used, piles should not be driven and construction on the wall should not begin until at least 90 percent consolidation has been achieved. Piles to support the walls should be driven to refusal on bedrock. Estimated pile tip elevations for the structures are provided on page 6.

The surcharge embankment may be removed prior to constructing the pile-supported retaining wall. Alternatively, consideration could be given to leaving the surcharge embankment in place. This may not be feasible due to the dimensions of the proposed retaining wall and the space required for construction. If left in place, the void space between the surcharge embankment and the reinforced concrete retaining wall should be filled with suitable material and compacted. If there is not sufficient space to properly compact a granular fill material, a flowable-fill material, such as a low-strength concrete, could be considered.

Other Options

Other ground improvement techniques such as controlled modulus columns (CMC) could be considered to stabilize the foundation soils prior to construction of the walls and embankments at the interchange. However, it is understood that ODOT personnel do not want to explore this technique at this time.

The use of vibro-compaction has been considered to improve soils at this site. Although vibro-compaction could improve shear strengths in granular layers, several concerns still exist that may preclude the use of this technique at this site. Some concerns are the potential settlement of nearby railroad tracks and the low undrained shear strength of clay (fine-grained) layers across the site. The fine-grained soils would not realize an appreciable increase in undrained shear strengths using this technique. Consequently, this technique is not recommended.

Preliminary Bridge Foundation Recommendations

In the area of the proposed structures, borings generally encountered bedrock at depths ranging from 13 to 21 feet below the ground surface. Bedrock encountered in the borings generally consisted of soft to medium hard Shale, which was highly to moderately weathered and moderately fractured.

It is recommended that driven H-piles be used to support the proposed structure. Pile tip elevations have been estimated for HP 12x53, 70-ton piles driven to refusal on bedrock. Other H-piles could also be considered to support the bridge abutments. For preliminary purposes, the pile tip elevations provided for the HP 12x53 piles are also considered to be representative of HP 10x42 and HP 14x73 piles. It is anticipated that the piles will penetrate one to two feet into the bedrock. Because of the tendency of some shales to relax, it is recommended that the contractor restrike the piles 24 hours after installation to ensure the allowable bearing capacity of the pile is met.

Typically, a minimum of 15 feet of embedment is required for bearing piles. The overburden thickness on this site ranges from approximately 13 to 21 feet. It is anticipated that some piles

will not achieve the required 15 feet of embedment. If this is of concern, the piles could be pre-bored and socketed five-feet into competent bedrock. Alternatively, drilled shafts could be considered for support of the abutments.

If lateral loading or uplift is a concern, consideration could be given to using drilled shafts to support the abutments. If significant uplift or lateral loading of the structure foundation is anticipated, DLZ should be notified so that we may revise our recommendations as necessary.

A table summarizing the site conditions and foundation recommendations (assumes single-span structures) is presented below.

*Summary of Foundation Recommendations, HP-12x53, 70 ton Driven Piles**

Structure	Element	Boring Number	Existing Ground Surface Elevation (Feet)	Estimated Pile Tip Elevation (Feet)
Mainline (Westbound) over Fairgrounds Road	Rear Abutment	B-1146	567.7	551.7
	Forward Abutment	B-1144	565.2	542.2
Mainline (Eastbound) over Fairgrounds Road	Rear Abutment	B-1145	567.3	551.3
	Forward Abutment	TR-55A	565.4	544.4
Ramp B over Fairgrounds Road	East Abutment	TR-58	567.1	550.6
	West Abutment	B-1113	566.8	545.8
Ramp C over Fairgrounds Road	East Abutment	TR-54	566.9	550.4
	West Abutment	B-1116	565.8	544.8

* *Cited pile tip elevations are considered representative of all H-piles being considered.*

Closing

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.

Sincerely,

DLZ OHIO, INC.

Steven J. Riedy
Geotechnical Engineer

Dorothy A. Adams, P.E. (SJR)
Dorothy A. Adams, P.E.
Senior Geotechnical Engineer

Attachments: MSE Wall Stability Calculations
Settlement Calculations
Results of Laboratory Testing

cc: File

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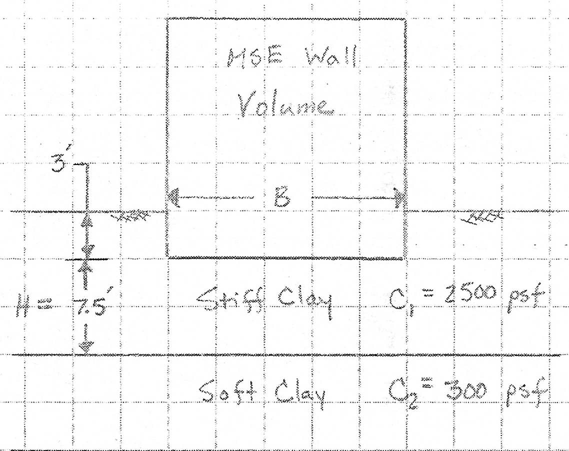
DLZ ENGINEERS • ARCHITECTS • SCIENTISTS PLANNERS • SURVEYORS

CLIENT: CH2M Hill / ODOT D-9 | PROJECT NO. 0121-3070.03
PROJECT: SCI-823 Portsmouth Bypass | SHEET NO. 1 OF 17
SUBJECT: Undrained Bearing Capacity Multi-Layered Analysis | COMP. BY SAK DATE 3-23-07
CHECKED BY GWT DATE 3-26-07

*Multi-layered bearing capacity

Reference: [AASHTO, Standard Spec. for Highway Bridges, 17th Ed.]



* Assume B = 30', L = 219' (Wall 1)
* Assume 3' Embedment of MSE Wall
* Assumes C_2 = 300 psf (conservative)

$q_{ULT} = C_1 N_m + q$ [4.4.7.1.1.7-1]

$N_m = (1/\beta_m + K \cdot S_c \cdot N_c) \leq S_c \cdot N_c$ [4.4.7.1.1.7-2]

β_m = Punching Index $\beta_m = BL / [2(B+L)H] = (30)(219) / [2(30+219)(7.5)] = 1.76$

$K = C_2 / C_1 = 300 / 2500 = 0.12$

S_c = Shape Factor $S_c = 1 + (B/L)(N_q/N_c)$ for other than continuous footings $(L < 5B)$

Since $L = 219' > 5B = 150$ → We may assume continuous footings
∴ $S_c = 1.0$

For Undrained Case $\phi = 0$ → $N_c = 5.14$

$N_m = (1/1.76) + (0.12)(5.14) = 1.18$

$q_{ULT} = C_1 \cdot N_m + q = (2500\ psf)(1.18) + (3')(120\ pcf) = 3310\ psf$

$q_{allow} = \frac{q_{ULT}}{F.S.} = \frac{3310\ psf}{2.5} = 1324\ psf$

DLZ

SUBJECT
Client: CH2M Hill | JOB NUMBER 0121-3070.03
Project: SCI-823 Portsmouth Bypass | SHEET NO. 2 OF 17
Item: MSE Wall Stability | COMP. BY SAK DATE 3-23-07
Fairgrounds Road Walls 1 & 2 | CHECKED BY DAA DATE 3-28-07
Based upon strengths from boring B-1133

STABILITY OF MSE WALL *(Using Pile Supported Abutments)*

Assumptions:

1. Estimated height of embankment; H=32'
2. It is assumed that the bridge is supported on piles
3. Ground water; Dw=0.0'
4. Traffic loading is neglected in resisting forces
5.

Wall Properties

H+D	=	35	feet
γ_{mse}	=	120	pcf
L	=	31.5	feet
L factor	=	0.90	
ϕ	=	30	deg

Foundational Soil Properties

c	=	2500	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

Dimensions X= 8.5 ft, Y= 10.0 ft

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 = 27,027 lbs per foot of wall

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

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

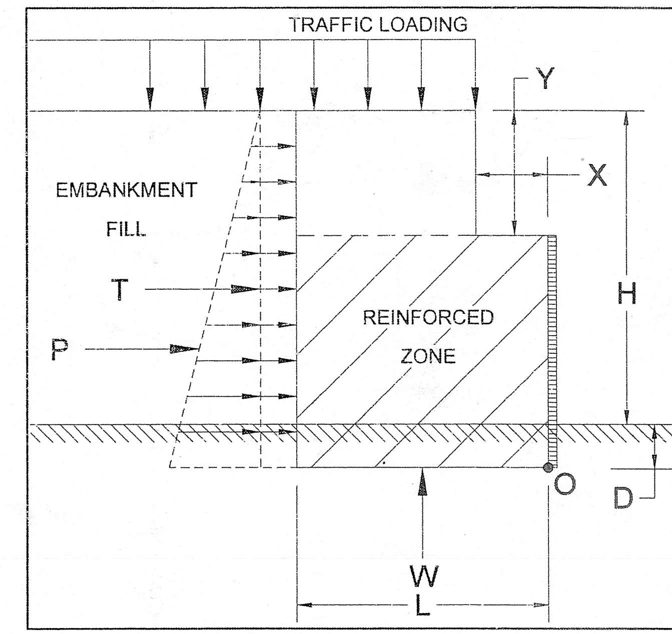
P_r = 45,177 lbs per foot of wall

USE THIS VALUE

$P_r = L(c)$ (Undrained)

P_r = 78,750 lbs per foot of wall

Use Drained Value



$FS = \frac{P_r}{P_a}$ Calculated FS = **1.67** 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

$\Sigma M_{resisting}$ = 2,040,375 lb-ft

$\Sigma M_{overturning}$ = 331,485 lb-ft

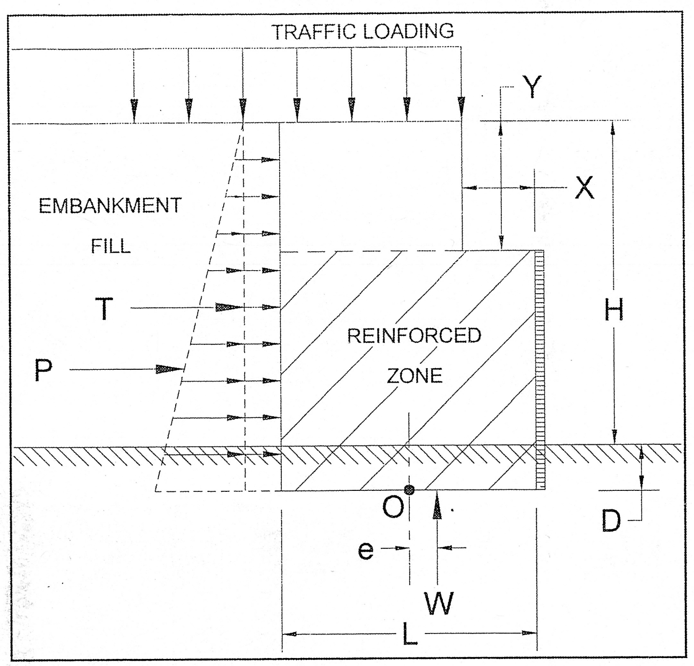
$\Sigma M_{resisting} = (L - X)Y\gamma\left(X + \left(\frac{L-X}{2}\right)\right) + L(H-Y)\gamma\left(\frac{L}{2}\right)$

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

$FS = \frac{\Sigma M_{resisting}}{\Sigma M_{overturning}}$ Calculated FS = **6.16** Required FS = 2.00 Resistance Against Overturning is **OK**

BEARING CAPACITY OF A MSE WALL *(Using Pile Supported Abutments)*

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	=	300	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

ωt	=	240	psf	traffic loading
L=B	=	31.5	ft	length of mse block
L factor	=	0.9		Length factor-range (0.7 - 1.0)
D	=	3	ft	embedment depth
Dw	=	0	ft	groundwater depth
H+D	=	35	ft	
H	=	32	ft	height of wall
Ka	=	0.33		
Γ Pa	=	11.667	ft	moment arm
Γ Wt	=	17.5	ft	moment arm
B'	=	28.52	ft	
γ'	=	57.6	pcf	
W_t	=	5,520	lb/ft of wall	
W_{mseA}	=	94,500	lb/ft of wall	
W_{mseB}	=	27,600	lb/ft of wall	

X = 8.5 ft
Y = 10.0 ft

Effective Bearing Pressure

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

σ_v = 4,475 psf

Ultimate undrained bearing capacity, q_{ult}

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

q_{ULT} = 1,715 psf

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

q_{ALL} = 686 psf

Factor of Safety = *0.38 **No Good**

*See multi-layered bearing capacity Analysis

Ultimate drained bearing capacity, q_{ult}

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

q_{ULT} = 18,726 psf

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

q_{ALL} = 7,490 psf

Factor of Safety = 4.18 **OK**

Bearing Capacity Factors for Equations

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

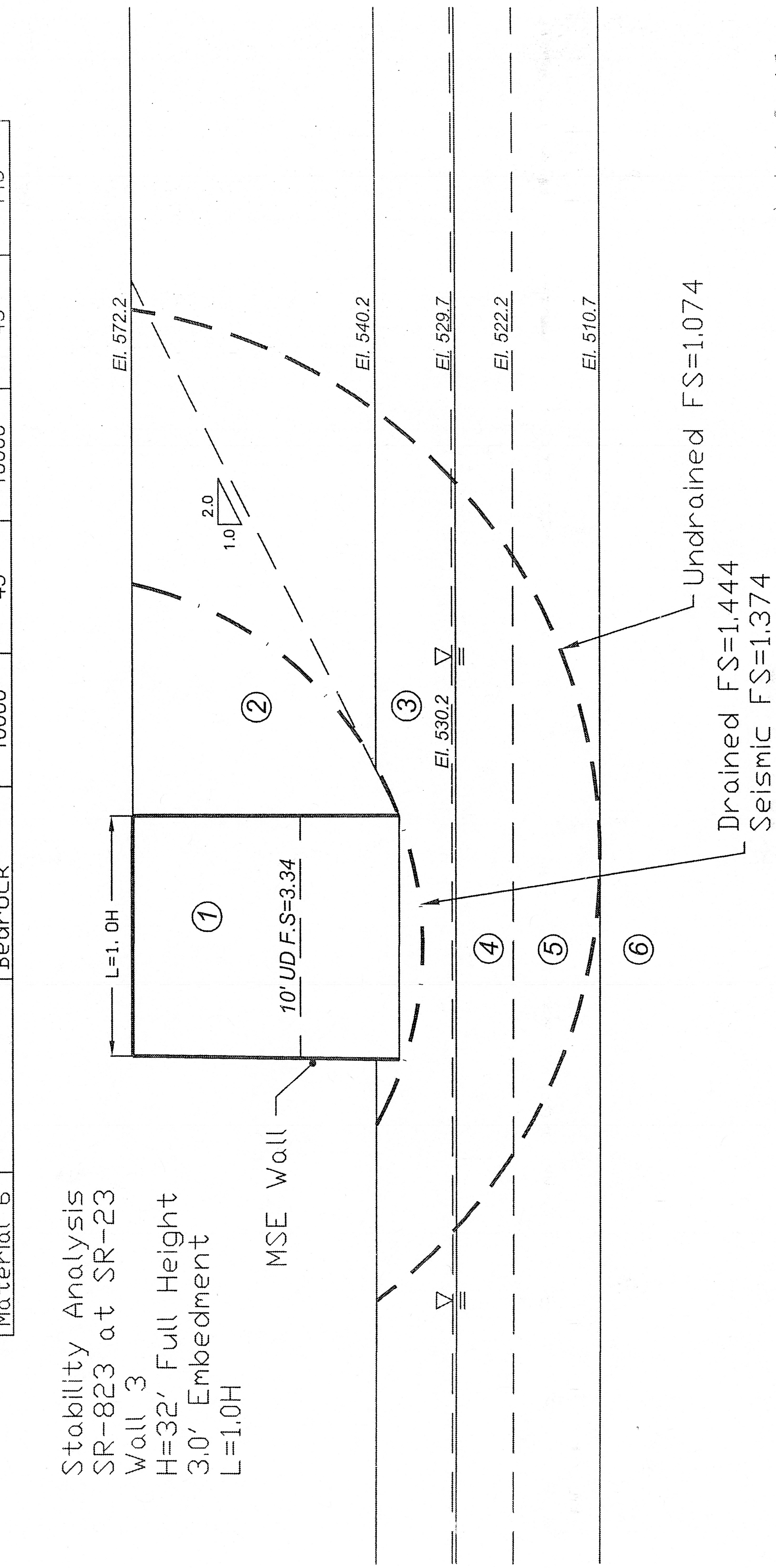
Eccentricity of Resultant Force

e = 1.49 ft

Kern

e < L/6 = 5.25 ft

Material	Consistency	Soil Type	Undrained C (psf)	Undrained φ (deg)	Drained C' (psf)	Drained φ' (deg)	γ (pcf)
Material 1	Lightweight	MSE Fill	10000	40	10000	40	30
Material 2	Compacted	Emb. Fill	0	30	0	30	120
Material 3	Very Stiff	Silt and Clay	2500	0	0	29	125
Material 4	Soft	Sandy Silt	300	0	0	29	120
Material 5	Soft	Silty Clay	300	0	0	29	125
Material 6		Bedrock	10000	45	10000	45	145



Stability Analysis
SR-823 at SR-23
Wall 3
H=32' Full Height
3.0' Embedment
L=1.0H

Sheet 4 of 17

Undrained, Drained and Seismic Analyses
Based on Boring B-1133

MSE GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 2/19/07

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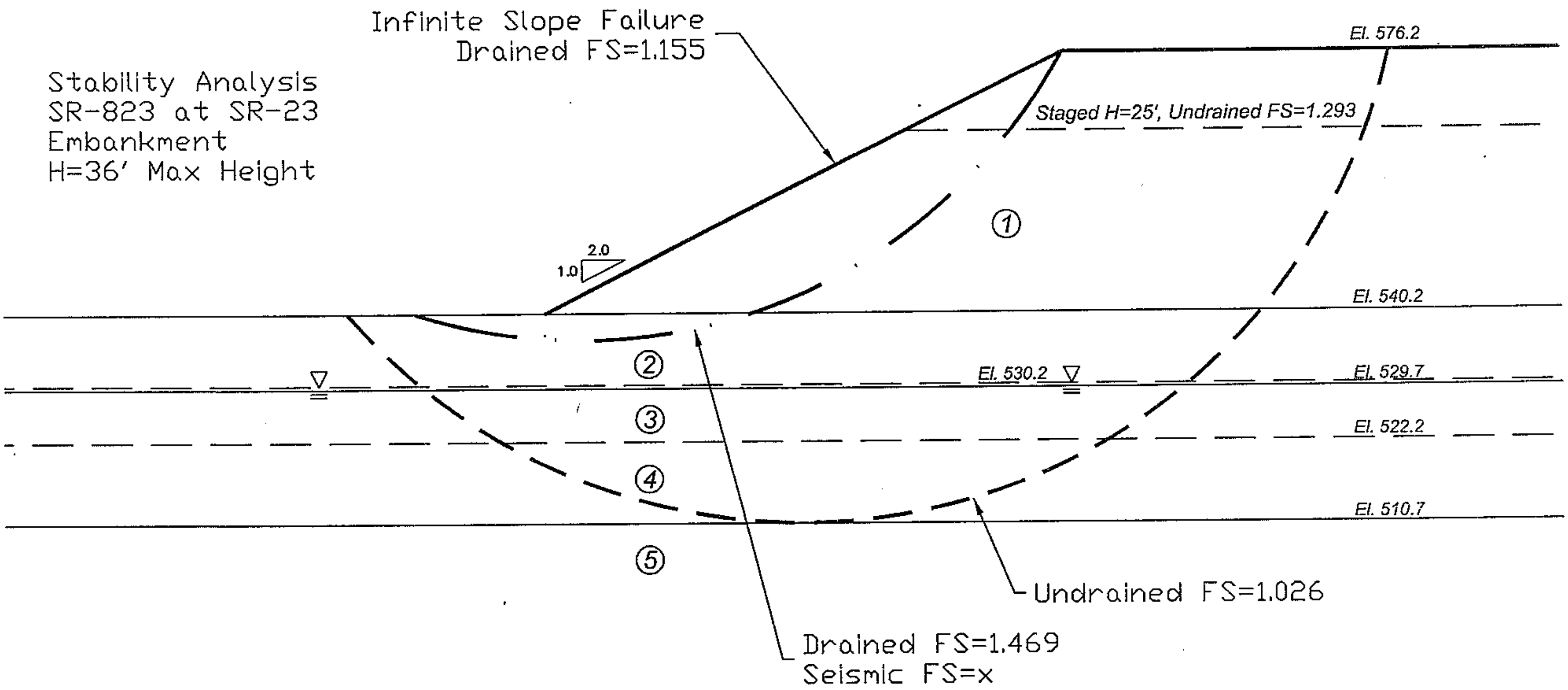
Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	Very Stiff	Silt and Clay	2500	0	0	29	125
Material 3	Soft	Sandy Silt	300	0	0	29	120
Material 4	Soft	Silty Clay	300	0	0	29	125
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
SR-823 at SR-23
Embankment
H=36' Max Height



Sheet 5 of 17

US-23 Interchange
Based on Boring B-1133
Embankment Stability & Staged Const.

EMBANKMENT GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 2/19/07

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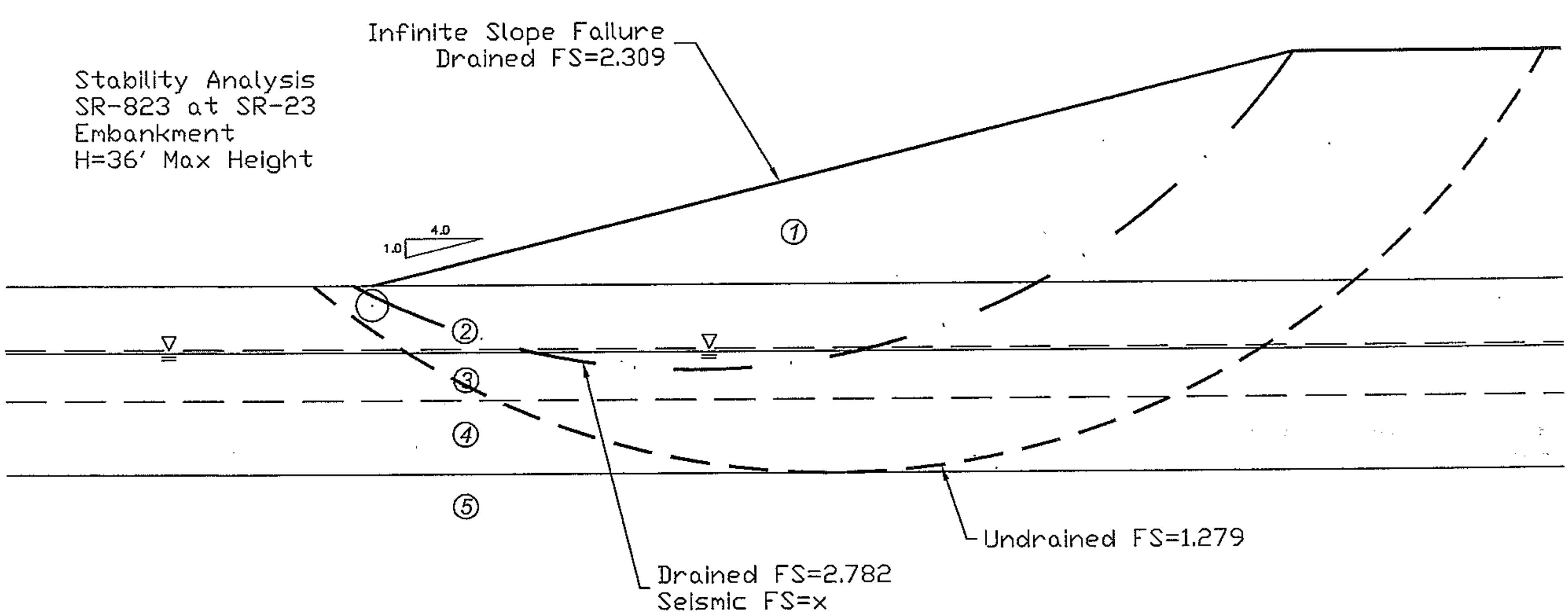
Material	Consistency	Soil Type	Undrained C (psf)	Undrained Φ (deg)	Drained C' (psf)	Drained Φ' (deg)	γ (pcf)
Material 1	Compacted	Emb. Fill	0	30	0	30	120
Material 2	Very Stiff	Silt and Clay	2500	0	0	29	125
Material 3	Soft	Sandy Silt	300	0	0	29	120
Material 4	Soft	Silty Clay	300	0	0	29	125
Material 5		Bedrock	10000	45	10000	45	145

Stability Analysis
SR-823 at SR-23
Embankment
H=36' Max Height



Sheet 6 of 17

US-23 Interchange
Based on Boring B-1133
Embankment Stability & Staged Const.

EMBANKMENT GLOBAL STABILITY ANALYSIS

SCI-823-0.00

PROJECT NO. 0121-3070.03 | CALC: SJR | DATE 2/19/07

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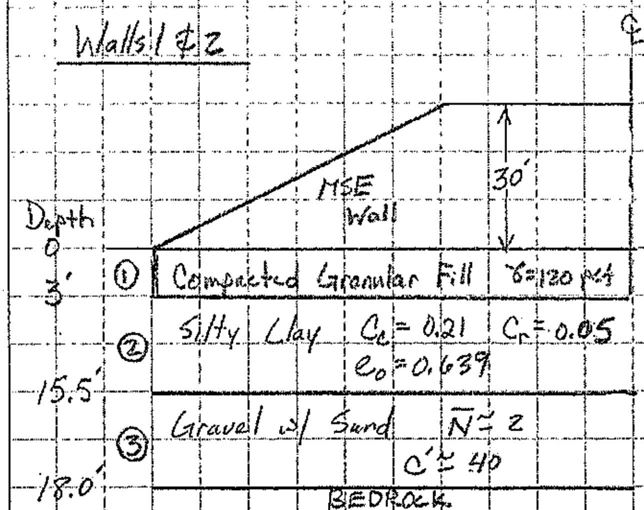
CLIENT	CH2 M Hill / ODOT D-9	PROJECT NO.	0121-3070.03
PROJECT	SCI-823 Portsmouth Bypass	SHEET NO.	7 OF 17
SUBJECT	US-23 Interchange Settlement of Wall at Fairgrounds Rd	COMP. BY	SJR DATE 2-28-07
		CHECKED BY	DAA DATE 3-1-07

Walls 1 & 2



Walls 1 & 2 *Assume H = 30'

Soil Profile based upon B-1113

Assume Incompressible

Consolidation Parameters Taken from tests on similar soils found in B-1108A - See attached

Ref [FHWA NHI-00-045] See Calculation Below

Layer 2

From Consolidation Tests, assume that soils are normally consolidated.

Layer 3

[Ref: FHWA NHI-00-045]

$N \cong N' \approx 2$ blows/ft $\longrightarrow C' \approx 40$

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

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

$\frac{1}{C'} = \frac{C_c}{1+e_o} \longrightarrow C' = \frac{2.0}{C_c} \longrightarrow \boxed{C_c = C_r = \frac{2}{C'}}$

When $C' = 40 \longrightarrow C_r = C_c \approx 0.05$ & $e_o = 1.0$

US-23 Walls 1 and 2 Initial Consolidation

Sheet 8 of 17

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name	: SCI-823	Client	: CH2M Hill
File Name	: 23-12	Project Manager	: P Nix
Date	: 2/28/10	Computed by	: SJR

Settlement for X-Direction

Embank. slope, x direc.	= 60.00 (ft)	Height of fill H	= 30.00 (ft)
y direc.	= 60.00 (ft)	Unit weight of fill	= 120.00 (pcf)
Embankment top width	= 120.00 (ft)	p load/unit area	= 3600.00 (psf)
Embankment bottom width	= 240.00 (ft)	Foundation Elev.	= 563.20 (ft)
Ground Surface Elev.	= 566.80 (ft)		
Water table Elev.	= 556.80 (ft)	Unit weight of Wat.	= 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	3.0	-----	-----	-----	120.00	----	----
2	COMP.	12.5	0.210	0.050	0.000	120.00	2.65	0.64
3	COMP.	2.5	0.050	0.050	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	5.65	560.38	771.00	771.00
3	6.25	554.42	1336.80	1336.80
4	2.50	550.05	1588.80	1588.80

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 12.00 Stress (psf)	Sett. (in.)	X = 24.00 Stress (psf)	Sett. (in.)	X = 36.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	16.58	0.08	374.97	1.49	745.97	2.55	1113.94	3.37
3	80.49	0.24	374.74	1.03	730.00	1.82	1088.95	2.49
4	122.11	0.02	389.67	0.07	730.70	0.12	1081.51	0.17
		0.35		2.60		4.49		6.02

Δ

Layer	X = 48.00 Stress (psf)	Sett. (in.)	X = 60.00 Stress (psf)	Sett. (in.)	X = 72.00 Stress (psf)	Sett. (in.)	X = 84.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	1478.45	4.04	1824.35	4.58	1840.10	4.60	1840.43	4.60
3	1442.18	3.05	1733.20	3.47	1802.29	3.56	1809.80	3.57
4	1421.25	0.21	1686.95	0.24	1780.00	0.24	1798.66	0.25
		7.30		8.28		8.40		8.42

δ_{max}

Sheet 9 of 17

US-23 Walls 1 and 2 Initial Consolidation

Layer	X = 96.00 Stress (psf)	Sett. (in.)	X = 108.00 Stress (psf)	Sett. (in.)	X = 120.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.			
2	1840.49	4.60	1840.51	4.60	1840.52	4.60
3	1811.50	3.57	1812.04	3.57	1812.18	3.57
4	1803.68	0.25	1805.38	0.25	1805.81	0.25
		8.42		8.42		8.42

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US-23 Walls 1 and 2 Consolidation after Surcharge

Sheet 10 of 17

ÚÄÄÄÄÄ ONE DIMENSIONAL SETTLEMENT ANALYSIS/Federal Highway Administration ÄÄÄÄÄ¿

INCREMENT OF STRESSES BENEATH THE END OF FILL CONDITION

Project Name : SCI-823 | Client : CH2M Hill
File Name : 23-12 | Project Manager : P Nix
Date : 2/28/10 | Computed by : SJR

Settlement for X-Direction

Embank. slope, x direc. = 60.00 (ft) | Height of fill H = 30.00 (ft)
y direc. = 60.00 (ft) | Unit weight of fill = 120.00 (pcf)
Embankment top width = 120.00 (ft) | p load/unit area = 3600.00 (psf)
Embankment bottom width = 240.00 (ft) | Foundation Elev. = 563.20 (ft)
Ground Surface Elev. = 566.80 (ft)
Water table Elev. = 556.80 (ft) | Unit weight of Wat. = 62.40 (pcf)

N§.	LAYER TYPE	THICK. (ft)	COEFFICIENT COMP.	RECOMP.	SWELL.	UNIT WEIGHT (pcf)	SPECIFIC GRAVITY	VOID RATIO
1	INCOMP.	3.0	-----	-----	-----	120.00	----	----
2	COMP.	12.5	0.210	0.050	0.000	120.00	2.65	0.64
3	COMP.	2.5	0.050	0.050	0.000	120.00	2.65	1.00

N§.	SUBLAYER THICK. (ft)	ELEV. (ft)	SOIL STRESSES INITIAL (psf)	MAX.PAST PRESS. (psf)
1	INCOMP.			
2	5.65	560.38	771.00	4713.89
3	6.25	554.42	1336.80	5375.00
4	2.50	550.05	1588.80	5861.11

Layer	X = 0.00 Stress (psf)	Sett. (in.)	X = 12.00 Stress (psf)	Sett. (in.)	X = 24.00 Stress (psf)	Sett. (in.)	X = 36.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	16.58	0.02	374.97	0.36	745.97	0.61	1113.94	0.80
3	80.49	0.06	374.74	0.25	730.00	0.43	1088.95	0.59
4	122.11	0.02	389.67	0.07	730.70	0.12	1081.51	0.17
		0.10		0.67		1.16		1.56

Layer	X = 48.00 Stress (psf)	Sett. (in.)	X = 60.00 Stress (psf)	Sett. (in.)	X = 72.00 Stress (psf)	Sett. (in.)	X = 84.00 Stress (psf)	Sett. (in.)
1	INCOMP.	INCOMP.	INCOMP.	INCOMP.				
2	1478.45	0.96	1824.35	1.09	1840.10	1.10	1840.43	1.10
3	1442.18	0.73	1733.20	0.83	1802.29	0.85	1809.80	0.85
4	1421.25	0.21	1686.95	0.24	1780.00	0.24	1798.66	0.25
		1.90		2.15		2.19		2.19

δ_{max}

Sheet 11 of 17

```
                    US-23 Walls 1 and 2 Consolidation after Surcharge
³          X =      96.00       X =    108.00       X =    120.00                     ³
³ Layer  Stress   Sett.      Stress  Sett.      Stress   Sett.                        ³
³         (psf)   (in.)       (psf)   (in.)      (psf)   (in.)                        ³
³                                                                                     ³
³   1   INCOMP.  INCOMP.   INCOMP.                                                    ³
³   2    1840.49   1.10     1840.51   1.10     1840.52   1.10                         ³
³   3    1811.50   0.85     1812.04   0.85     1812.18   0.85                         ³
³   4    1803.68   0.25     1805.38   0.25     1805.81   0.25                         ³
³                 ------             ------             ------                        ³
³                  2.19               2.19               2.19                         ³
³                                                                                     ³
³                                                                                     ³
³                                                                                     ³
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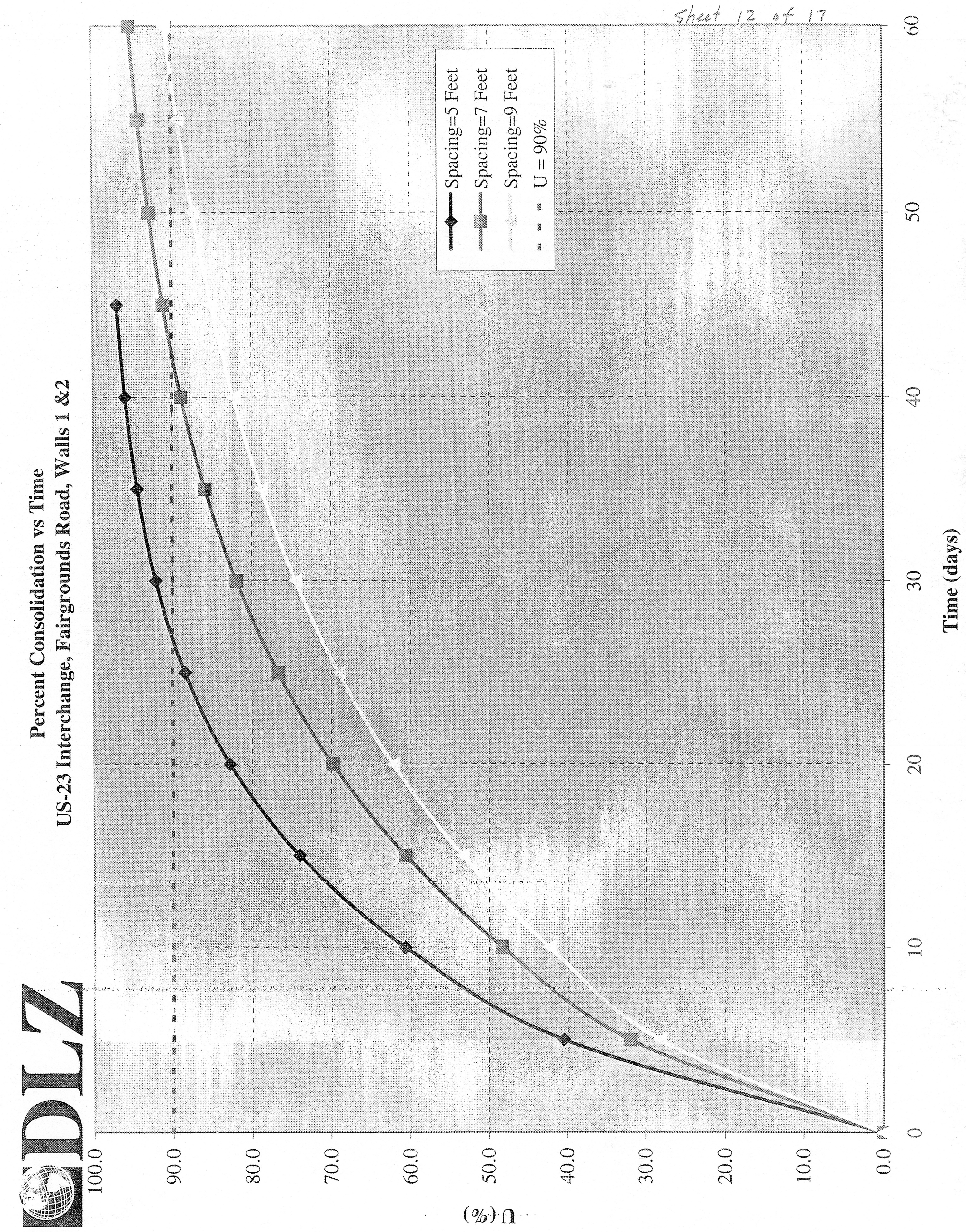

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
Fairgrounds Road Walls 1 & 2
Reference: FHWA-RD-86-168

Wick Drain Spacing 5.0 feet Use η = 10

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	5.25	0.30	6.25	8.4
5	0.0544	0.0384	0.25	0.20	40.5	3.4				
10	0.1088	0.0768	0.44	0.30	60.7	5.1				
15	0.1633	0.1152	0.58	0.39	74.0	6.2				
20	0.2177	0.1536	0.68	0.46	82.8	7.0				
25	0.2721	0.1920	0.76	0.52	88.5	7.4				
30	0.3265	0.2304	0.82	0.57	92.1	7.7				
35	0.3810	0.2688	0.86	0.61	94.4	7.9				
40	0.4354	0.3072	0.89	0.64	96.0	8.1				
45	0.4898	0.3456	0.91	0.67	97.0	8.1				

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
Fairgrounds Road Walls 1 & 2
Reference: FHWA-RD-86-168

Wick Drain Spacing 7.0 feet Use η = 10

t (days)	T_R	T_V	U_R	U_V	U_C	δ(inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	7.35	0.30	6.25	39
5	0.0278	0.0384	0.15	0.20	32.0	12.5				
10	0.0555	0.0768	0.26	0.30	48.4	18.9				
15	0.0833	0.1152	0.36	0.39	60.6	23.6				
20	0.1111	0.1536	0.44	0.46	69.8	27.2				
25	0.1388	0.1920	0.52	0.52	76.8	29.9				
30	0.1666	0.2304	0.58	0.57	81.9	32.0				
35	0.1944	0.2688	0.64	0.61	85.9	33.5				
40	0.2221	0.3072	0.69	0.64	88.8	34.6				
45	0.2499	0.3456	0.73	0.67	91.1	35.5				
50	0.2777	0.3840	0.77	0.69	92.9	36.2				
55	0.3054	0.4224	0.80	0.72	94.2	36.7				
60	0.3332	0.4608	0.82	0.73	95.3	37.2				
65	0.3610	0.4992	0.84	0.75	96.2	37.5				
70	0.3887	0.5376	0.86	0.77	96.9	37.8				
75	0.4165	0.5760	0.88	0.79	97.4	38.0				
80	0.4443	0.6144	0.89	0.80	97.9	38.2				
85	0.4720	0.6528	0.90	0.82	98.3	38.3				
90	0.4998	0.6912	0.91	0.84	98.6	38.4				

DLZ

Time Rate of Consolication of Foundation Soils with Wick Drians
Fairgrounds Road Walls 1 & 2
Reference: FHWA-RD-86-168

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

t (days)	T_R	T_V	U_R	U_V	U_C	δ (inches)	d_e	c_v	H_v	δ_{max}
0	0.0000	0.0000	0.00	0.00	0.0	0.0	9.45	0.30	6.25	39
5	0.0168	0.0384	0.10	0.20	28.2	11.0				
10	0.0336	0.0768	0.17	0.30	42.4	16.5				
15	0.0504	0.1152	0.24	0.39	53.5	20.9				
20	0.0672	0.1536	0.30	0.46	62.2	24.3				
25	0.0840	0.1920	0.36	0.52	69.1	27.0				
30	0.1008	0.2304	0.41	0.57	74.5	29.1				
35	0.1176	0.2688	0.46	0.61	78.8	30.7				
40	0.1344	0.3072	0.51	0.64	82.3	32.1				
45	0.1512	0.3456	0.55	0.67	85.0	33.2				
50	0.1680	0.3840	0.59	0.69	87.3	34.1				
55	0.1848	0.4224	0.62	0.72	89.2	34.8				
60	0.2016	0.4608	0.65	0.73	90.8	35.4				
65	0.2184	0.4992	0.68	0.75	92.1	35.9				
70	0.2352	0.5376	0.71	0.77	93.3	36.4				
75	0.2520	0.5760	0.73	0.79	94.3	36.8				
80	0.2687	0.6144	0.76	0.80	95.2	37.1				
85	0.2855	0.6528	0.78	0.82	96.0	37.4				
90	0.3023	0.6912	0.79	0.84	96.7	37.7				
95	0.3191	0.7296	0.81	0.86	97.3	37.9				
100	0.3359	0.7680	0.83	0.87	97.7	38.1				
105	0.3527	0.8064	0.84	0.89	98.1	38.3				
110	0.3695	0.8448	0.85	0.90	98.5	38.4				
115	0.3863	0.8832	0.86	0.91	98.7	38.5				
120	0.4031	0.9216	0.87	0.91	98.8	38.5				
125	0.4199	0.9600	0.88	0.91	98.9	38.6				
130	0.4367	0.9984	0.89	0.90	98.9	38.6				
135	0.4535	1.0368	0.89	0.88	98.8	38.5				
140	0.4703	1.0752	0.90	0.85	98.6	38.4				

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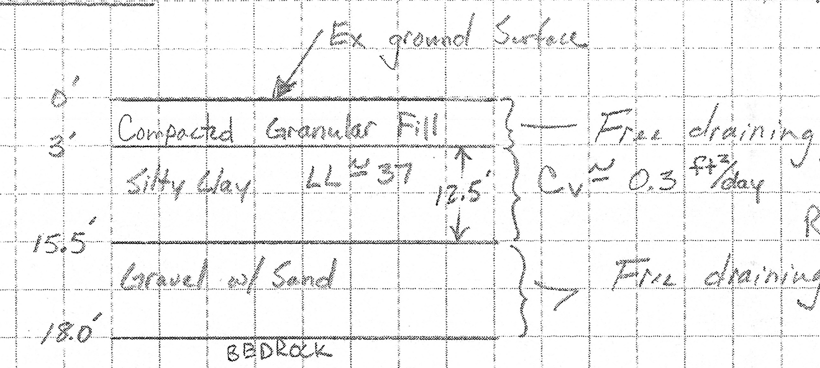
PLANNERS • SURVEYORS

CLIENT CH2M Hill / ODOT D-9
PROJECT SCI-823 Portsmouth Bypass
SUBJECT US-23 Interchange
Settlement of Wall at Fairgrounds Rd.

PROJECT NO. 0121-3070.03
SHEET NO. 16 OF 17
COMP. BY SJR DATE 2-28-07
CHECKED BY DAA DATE 3-1-07

Walls 1 & 2

*Soil Profile based upon boring B-1113



{Estimated as per FHWA based on LL}
Ref: FHWA HI-97-021, Also NAVFAC

Time-Rate of Consolidation

*Assume Double Drainage

$H_v = 12.5'/2 = 6.25'$

for $U = 90\% \rightarrow T_v = 0.848$

$t_{90} = \dfrac{T \cdot H_v^2}{C_v}$

$t_{90} = \dfrac{(0.848)(6.25')^2}{0.3\ ft^2/day} = 110.4$ days $\cong$ **110 days**

Differential Settlement

Prior to Surcharge: $DS = \dfrac{(2.60'' - 0.35'')(1.5ft/12'')}{12'}$

$\delta_{max} = 8.4''$

$DS = 0.016 = 1.6\% > 1.0\%$

After Surcharge: $DS = \dfrac{(0.67'' - 0.10'')(15ft/12'')}{12'}$

$\delta_{max} \cong 2.2''$

$DS = 0.004 = 0.4\% < 1.0\%$

Walls 1 & 2 – Secondary Compression Settlement

C_α, Secondary compression index measured from consolidation testing.

From boring B-1108 A, Sample P3

$C_\alpha \cong 0.003$ $e_p \cong 0.56$

$t = 75$ years (Service Life) $= 27{,}394$ days

$t_p = t_{95} = \frac{(1.13)(6.25)^2}{0.3\ ft^2/day} = 147$ days $H = 12.5'$

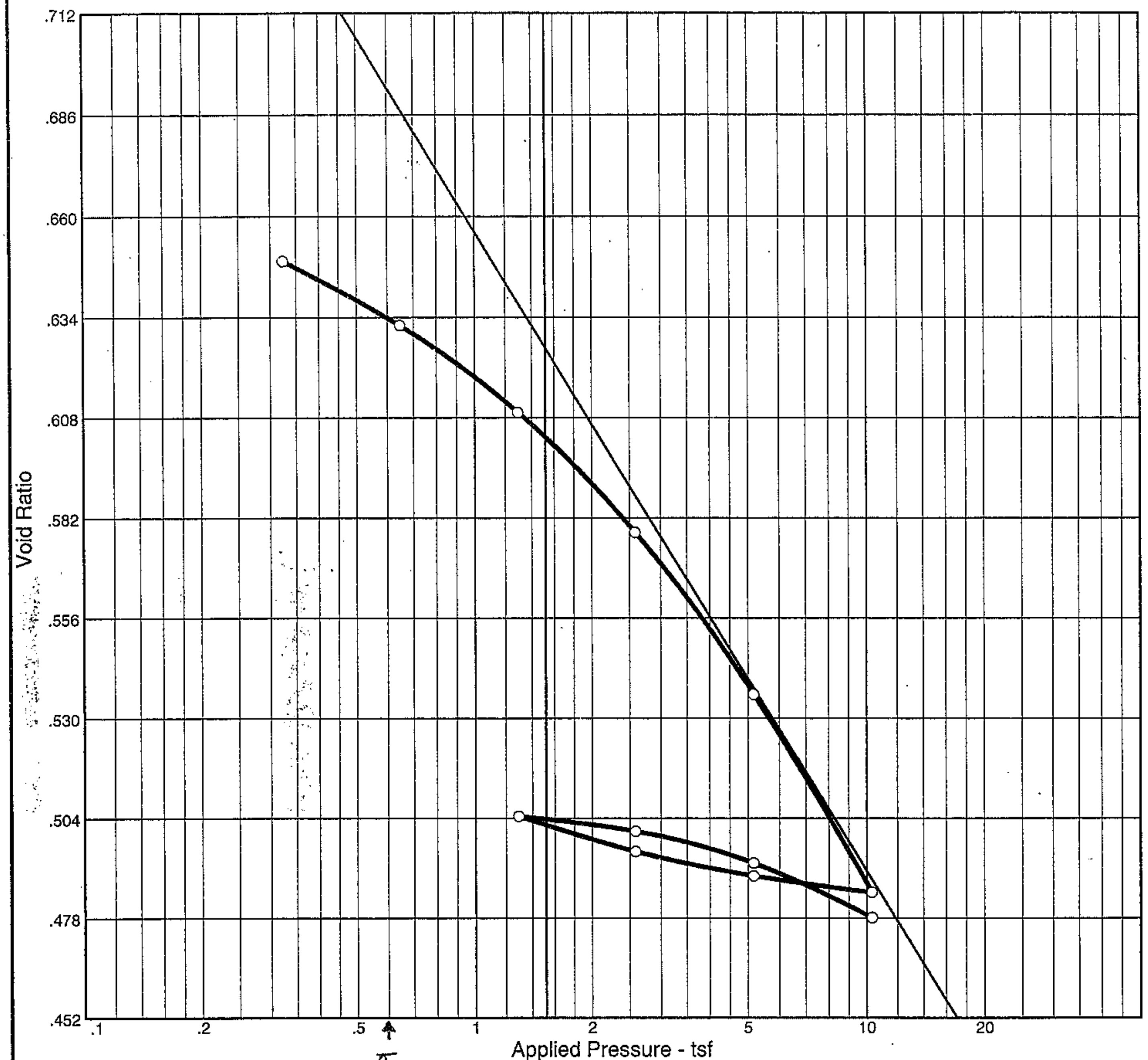
$$\delta_s = \frac{C_\alpha}{1+e_p} \cdot H \cdot \log\left(\frac{t}{t_p}\right)$$

$$\delta_s = \frac{0.003}{1+0.56}(12.5) \cdot \log\left(\frac{27{,}394}{147}\right) = 0.055\ ft$$

$\delta_s = 0.055\ ft = 0.7$ inches

* Secondary Compression at this site will be negligible

CONSOLIDATION TEST REPORT



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

MATERIAL DESCRIPTION

Lean clay, Silt and Clay (A-6a)
Specific Gravity= 2.65

Project No. 0121- **Client:** TranSystems, Inc.
Project: SCI-823-0.00
Source: B-1108A **Sample No.:** P1 **Elev./Depth:** 10.0

DLZ

Remarks:
NC
$C_c = 0.17$
$C_r = 0.03$
$C_\alpha \approx 0.003$

Figure

Dial Reading vs. Time

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

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

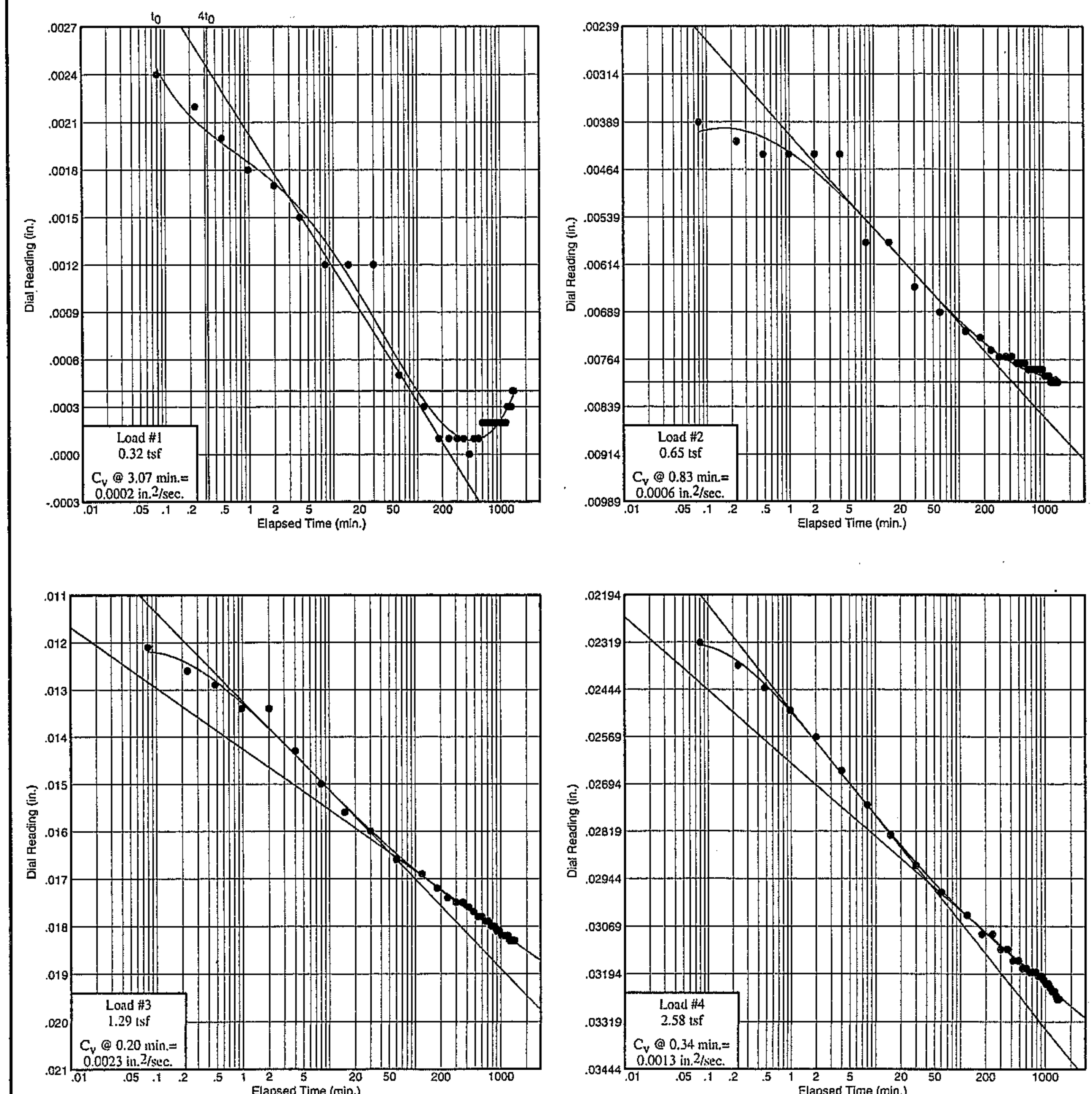


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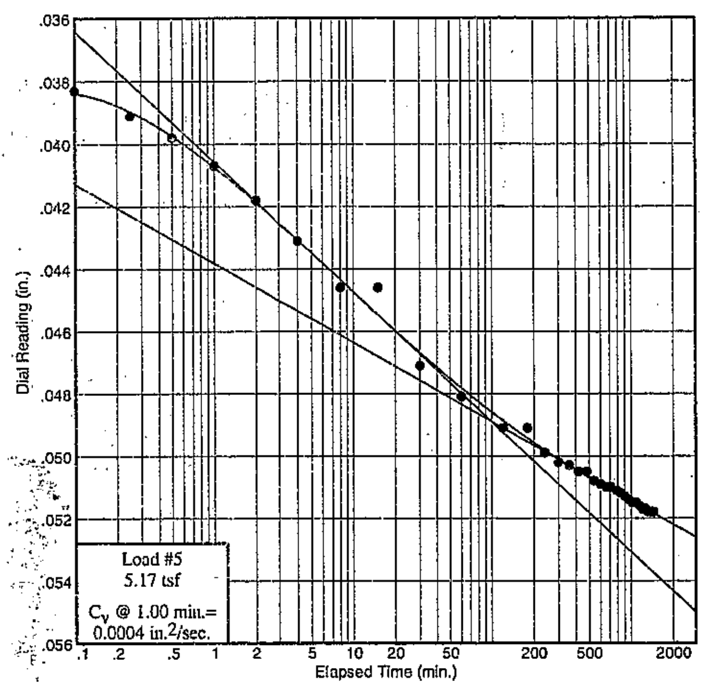
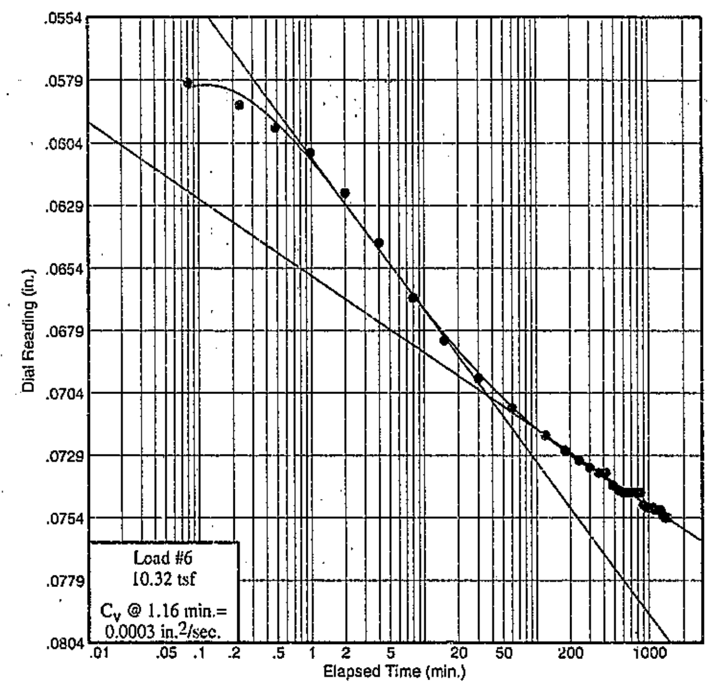
Figure

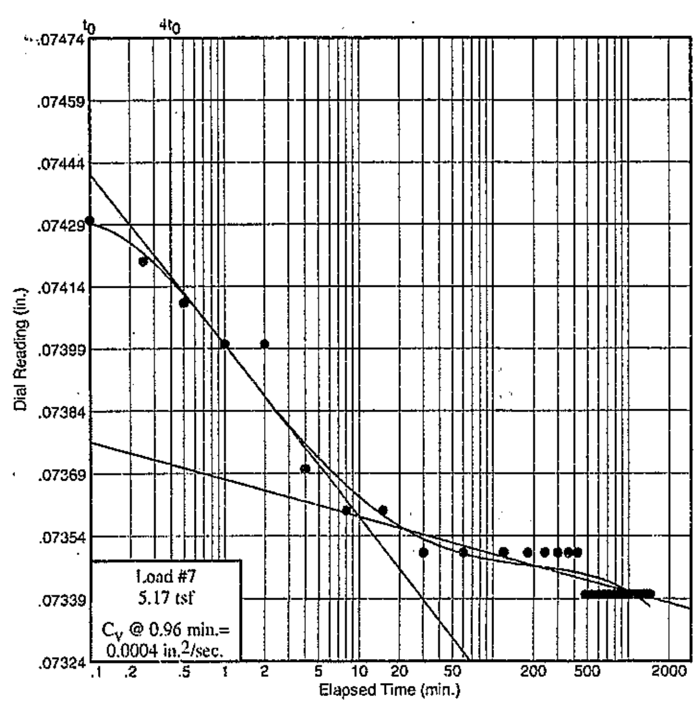
Dial Reading vs. Time

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

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







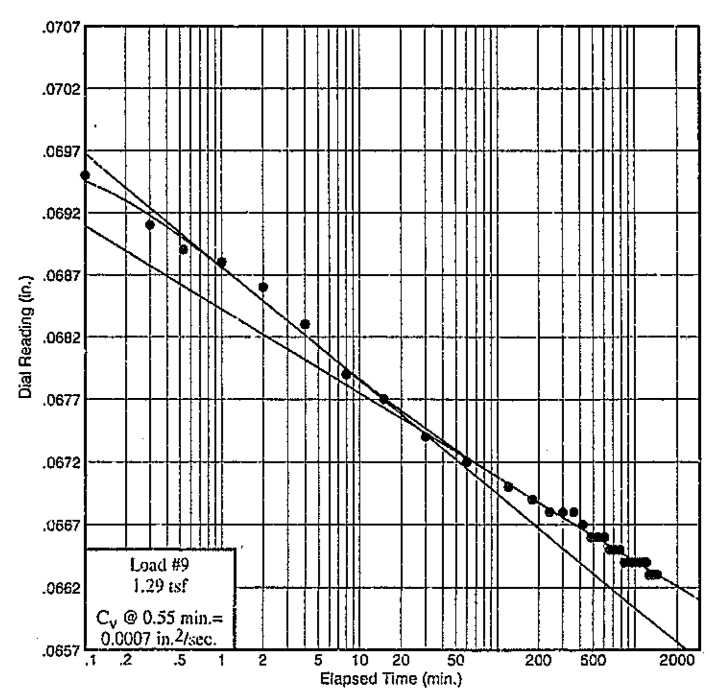


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Figure

Dial Reading vs. Time

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

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

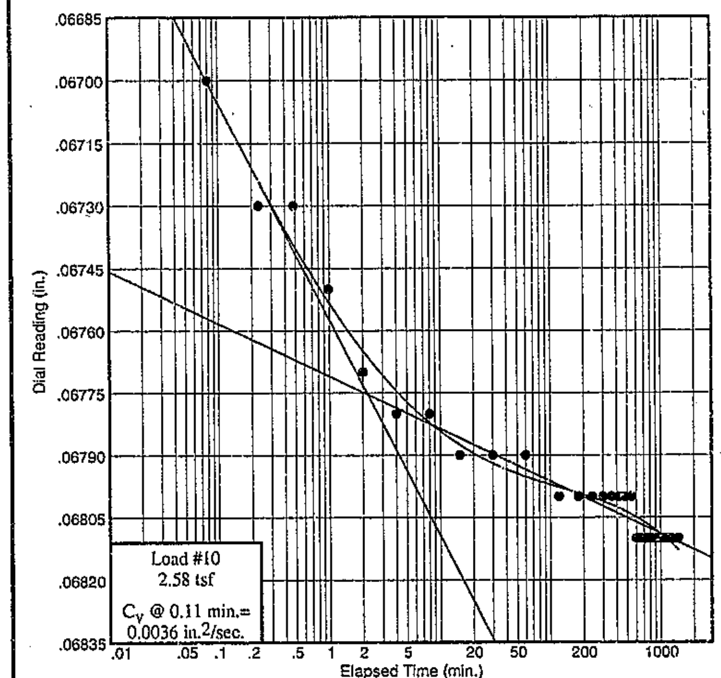

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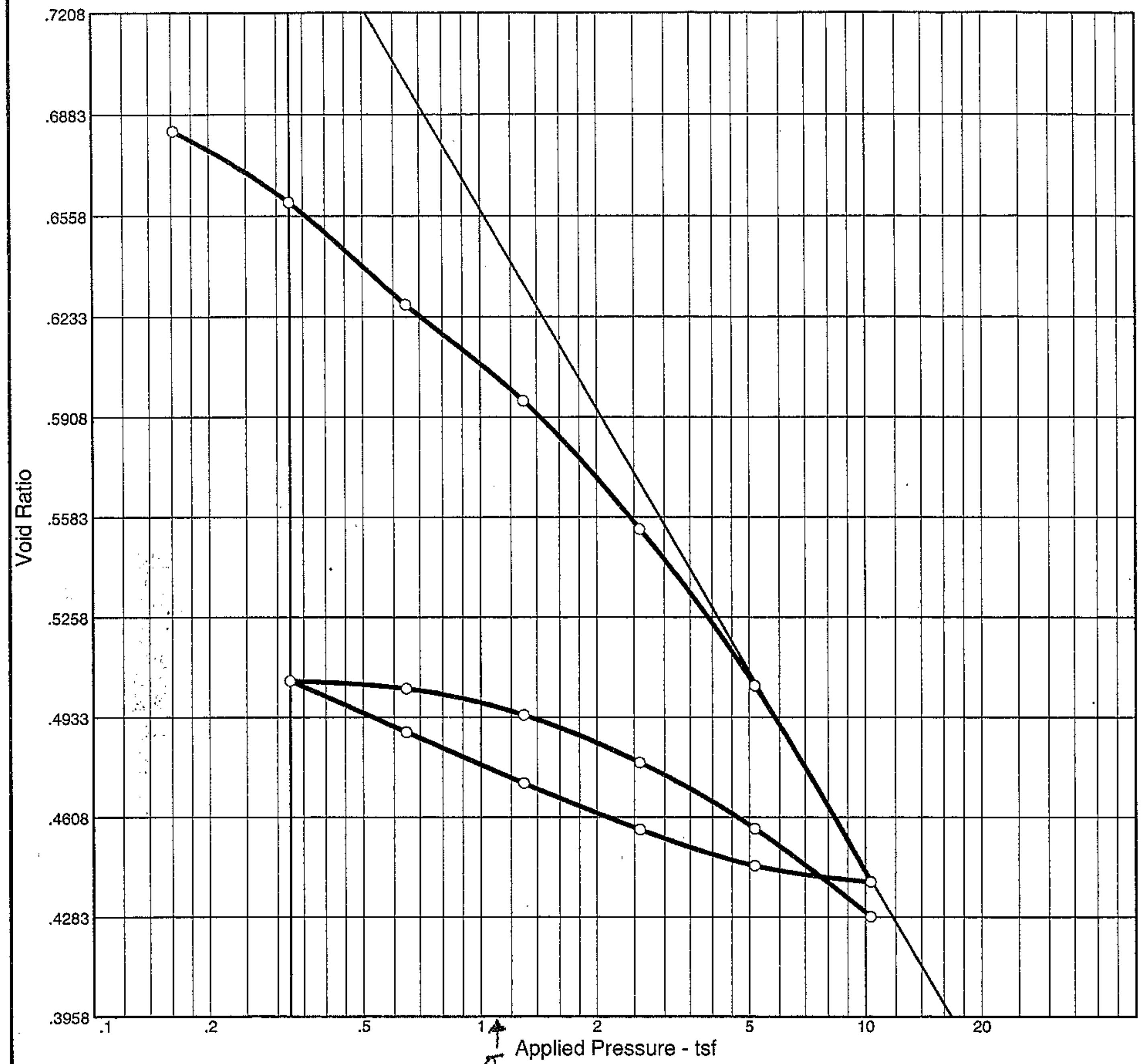
Figure

CONSOLIDATION TEST REPORT



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

MATERIAL DESCRIPTION

Lean clay   Silty Clay (A-6b)

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

Project: SCI-823-0.00

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

DLZ

Remarks:

NC

$C_c = 0.21$

$C_r = 0.05$

$C_\alpha = 0.003$

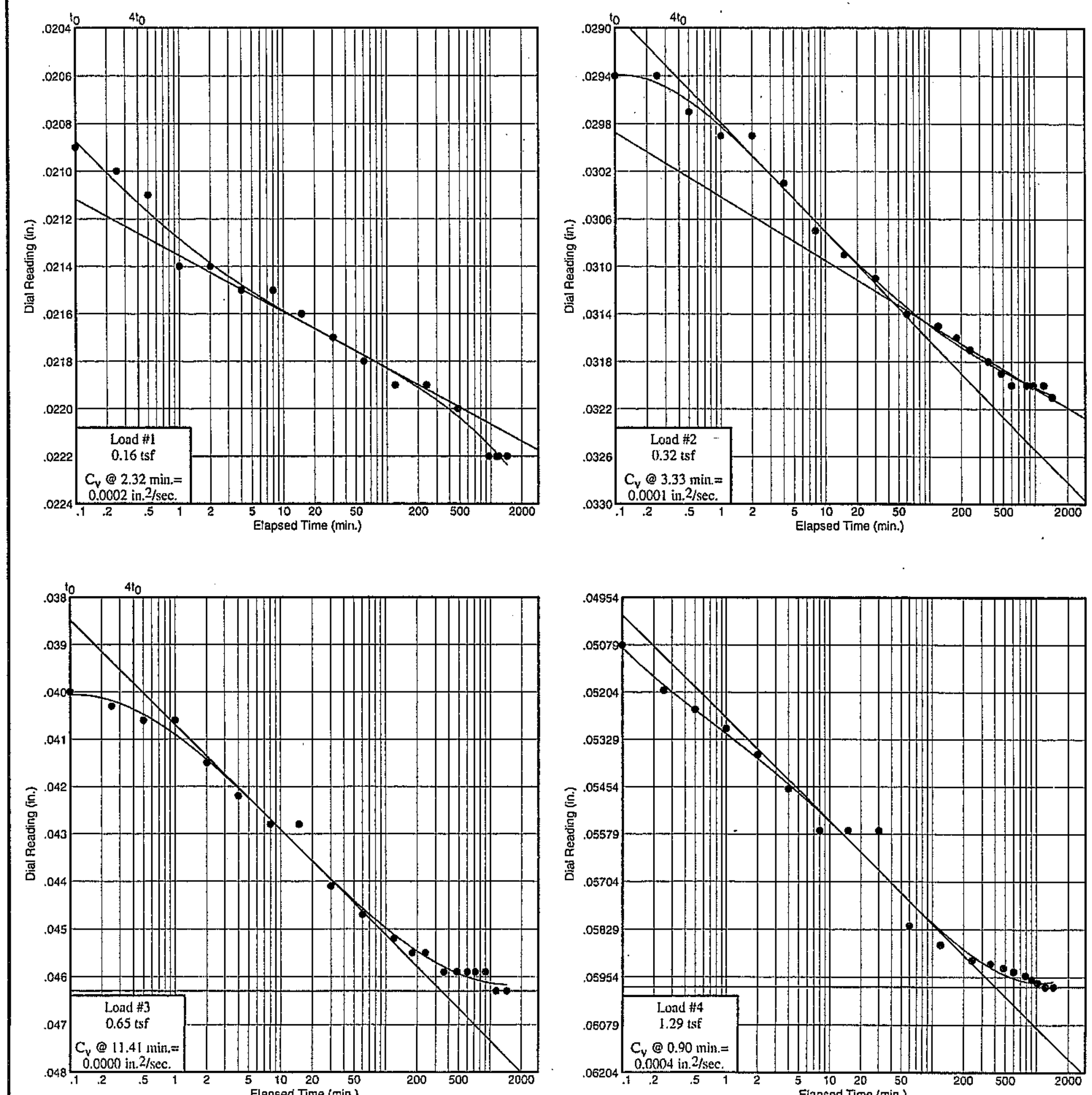
Figure

Dial Reading vs. Time

Project No.: 0121-3070.03

Project: SCI-823-0.00

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



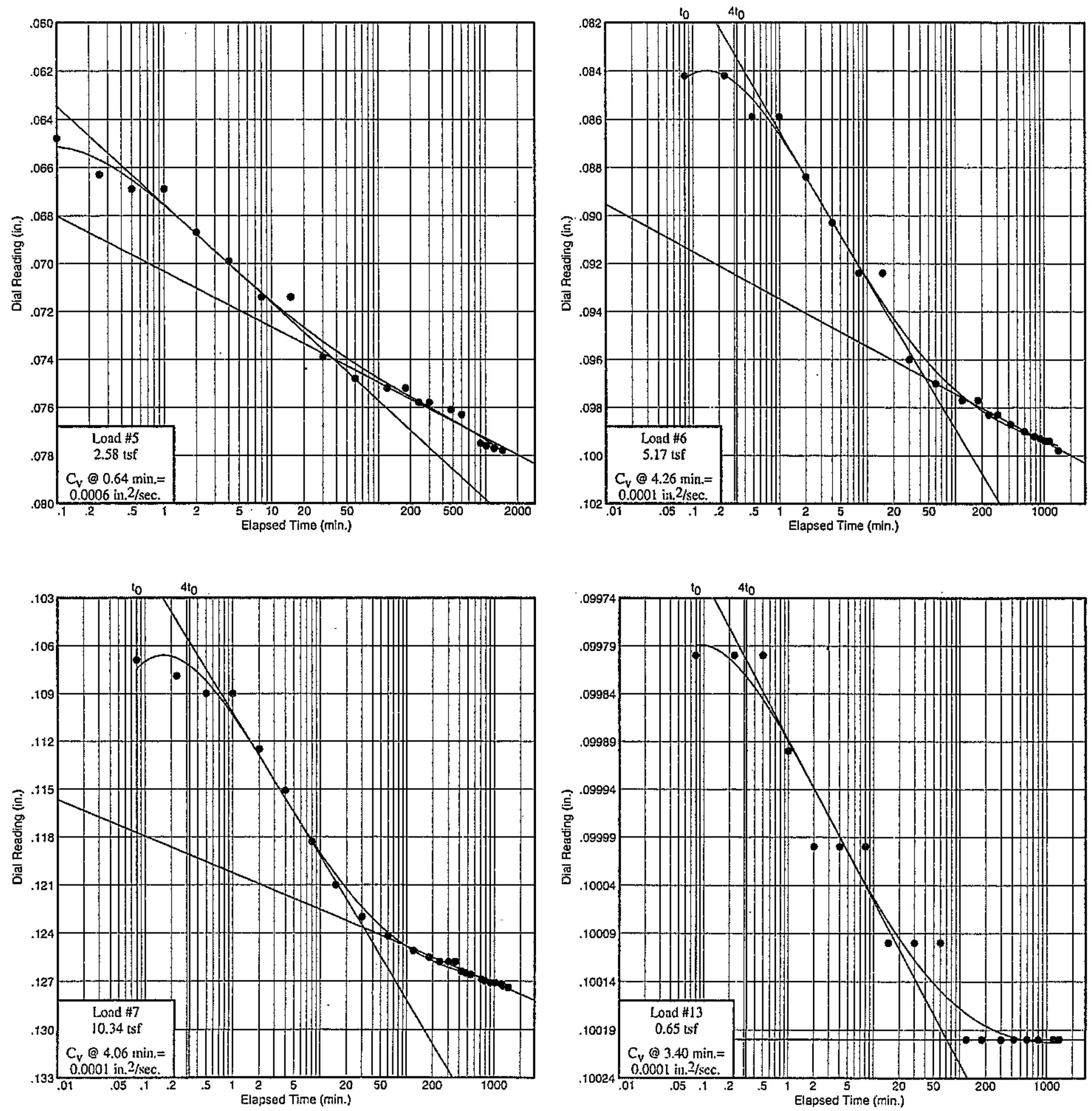
DLZ

Figure

Dial Reading vs. Time

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

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



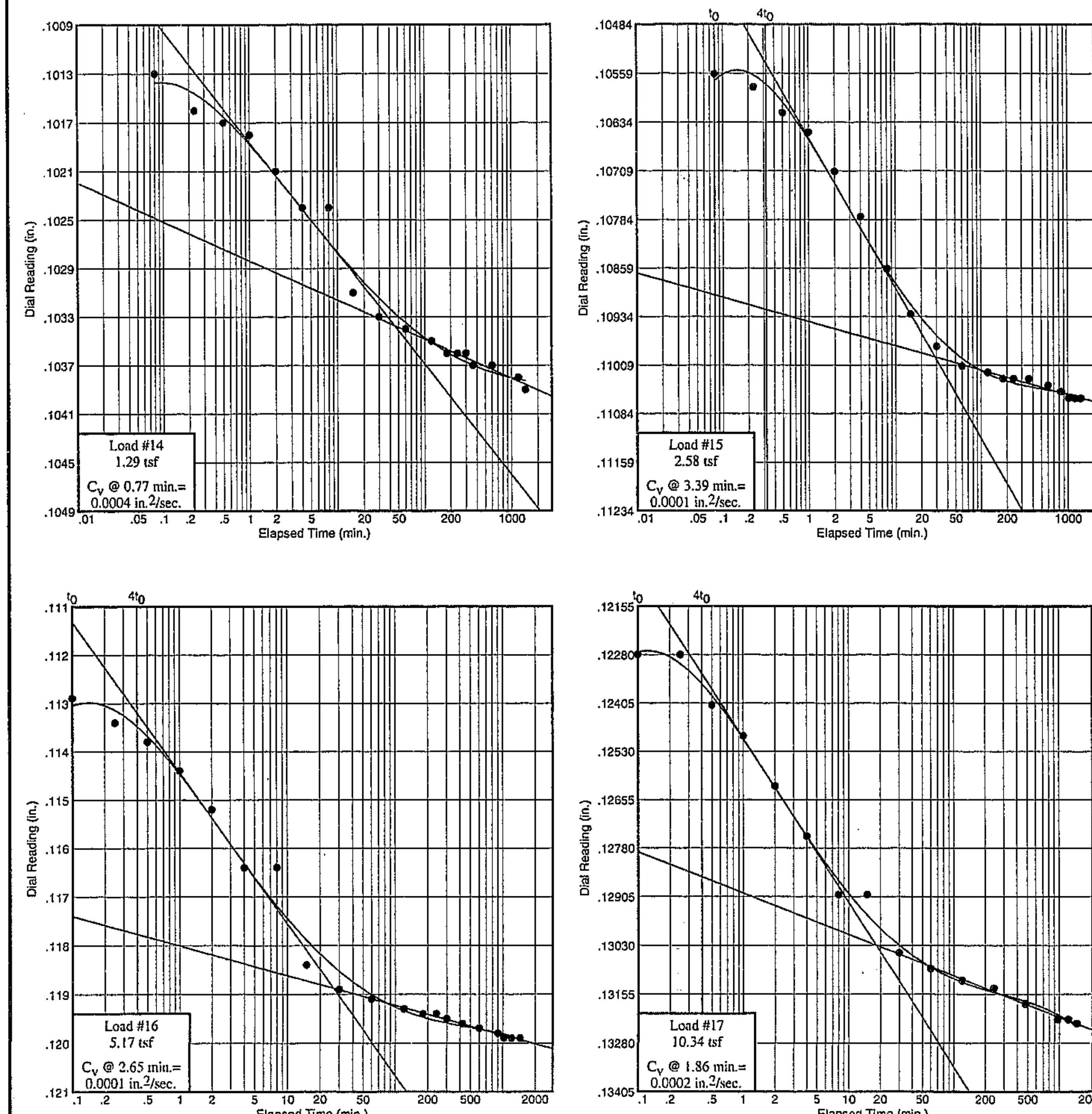
DLZ

Figure

Dial Reading vs. Time

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

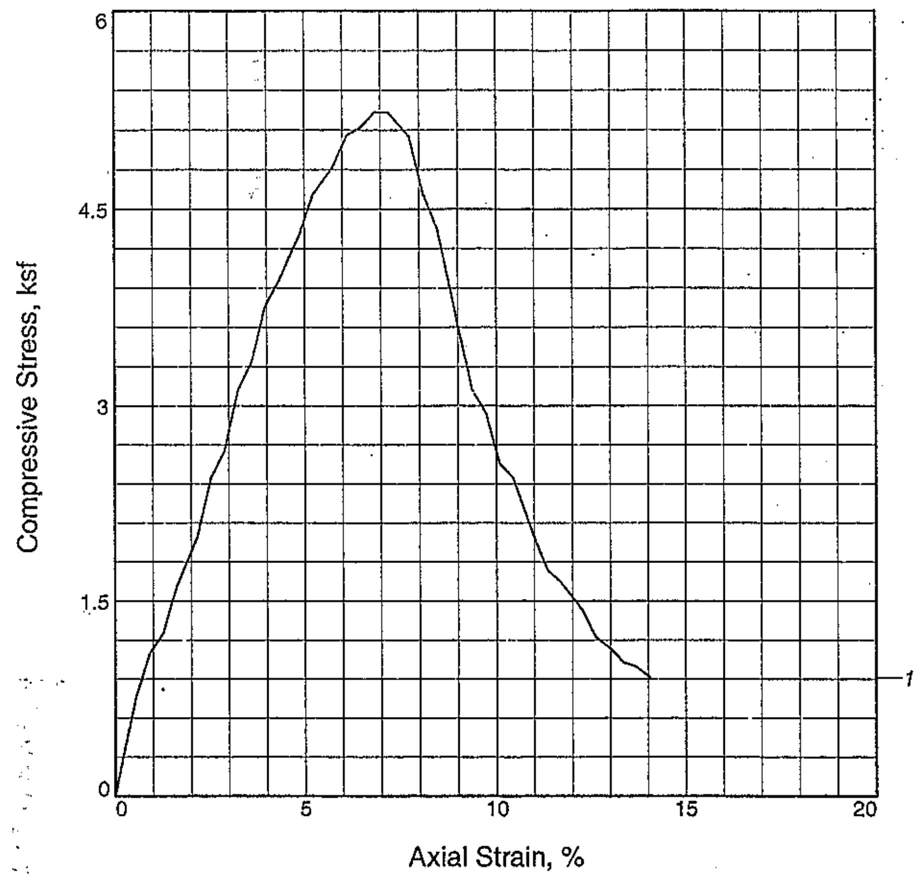
Source: B-1108A Sample No.: P3 Elev./Depth: 18.0



DLZ

Figure

UNCONFINED COMPRESSION TEST



Sample No.	1			
Unconfined strength, ksf	5.24			
Undrained shear strength, ksf	2.62			
Failure strain,	6.8			
Strain rate, in./min.	0.06			
Water content, %	22.4			
Wet density, pcf	126.5			
Dry density, pcf	103.4			
Saturation, %	93.1			
Void ratio	0.6602			
Specimen diameter, in.	2.83			
Specimen height, in.	5.55			
Height/diameter ratio	1.96			

Description: Moisture Content = 22.4%

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

Project No.: 0121-3070.03

Date: 08/16/06

Remarks:

Client: TranSystems, Inc.

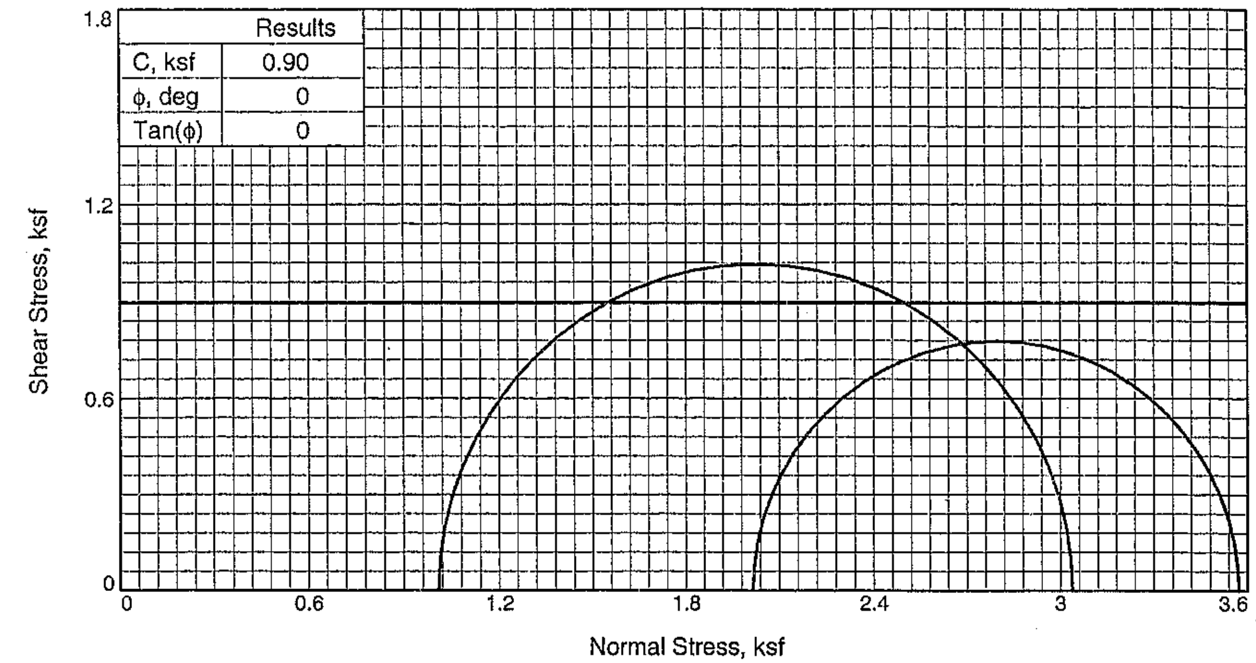
Project: SCI-823-0.00

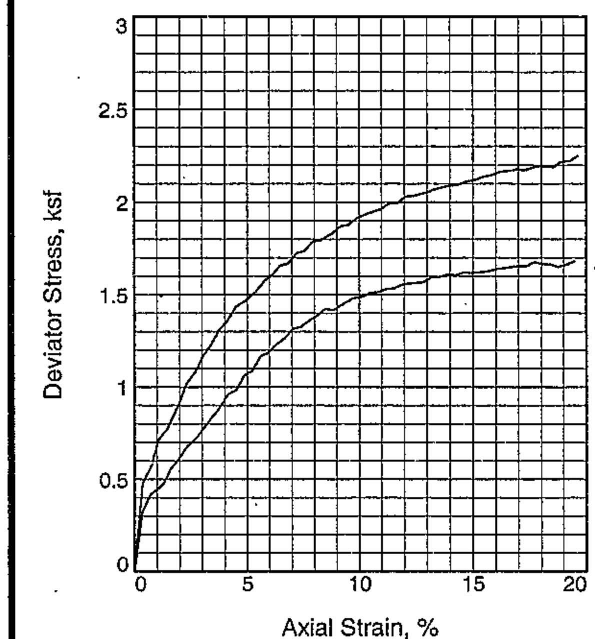
Source of Sample: B-1108A **Depth:** 10.0

Sample Number: P1

DLZ

Figure _______





	Sample No.	1	2
Initial	Water Content,	30.2	32.6
	Dry Density, pcf	95.2	89.5
	Saturation,	103.3	97.8
	Void Ratio	0.8041	0.9172
	Diameter, in.	2.83	2.84
	Height, in.	5.56	5.54
At Test	Water Content,	27.0	31.8
	Dry Density, pcf	95.2	89.5
	Saturation,	92.2	95.2
	Void Ratio	0.8041	0.9172
	Diameter, in.	2.83	2.84
	Height, in.	5.56	5.54
	Strain rate, in./min.	0.06	0.06
	Back Pressure, ksf	0.00	0.00
	Cell Pressure, ksf	1.01	2.02
	Fail. Stress, ksf	2.03	1.55
	Ult. Stress, ksf	2.03	1.55
	σ_1 Failure, ksf	3.04	3.57
	σ_3 Failure, ksf	1.01	2.02

Type of Test:
Unconsolidated Undrained

Sample Type: 3" Press Tube

Description: Lean clay with sand

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

Assumed Specific Gravity= 2.75

Remarks:

Client: TranSystems, Inc.

Project: SCI-823-0.00

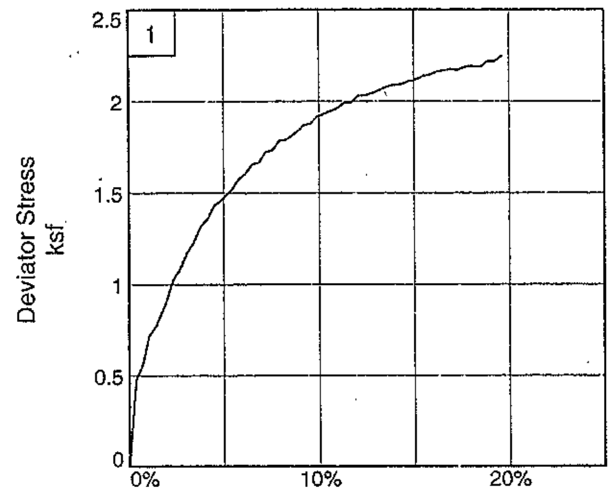
Source of Sample: B-1108A **Depth:** 14.0

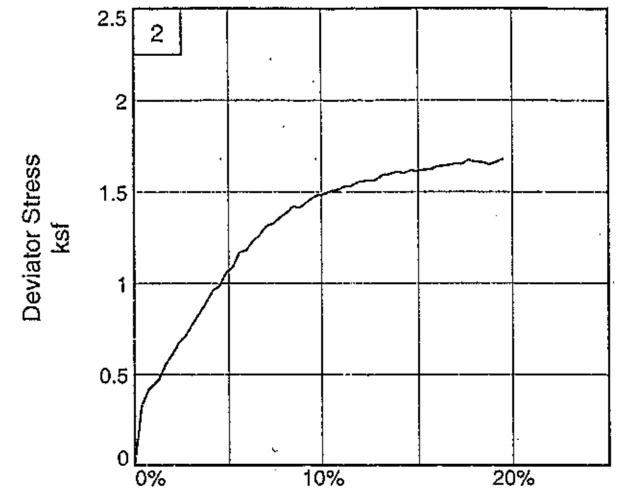
Sample Number: P2

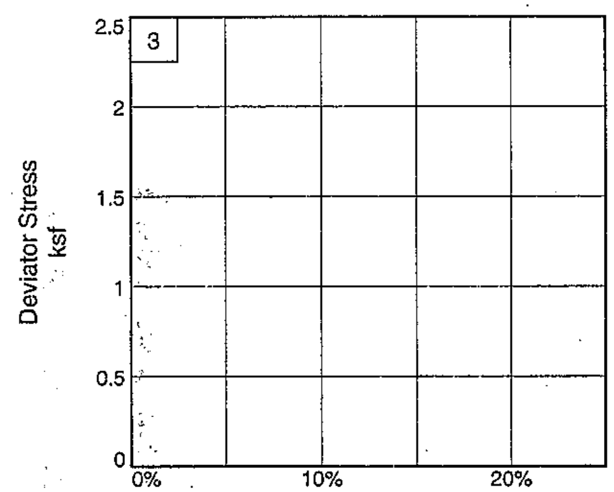
Proj. No.: 0121-3070.03 **Date:** 08/16/06

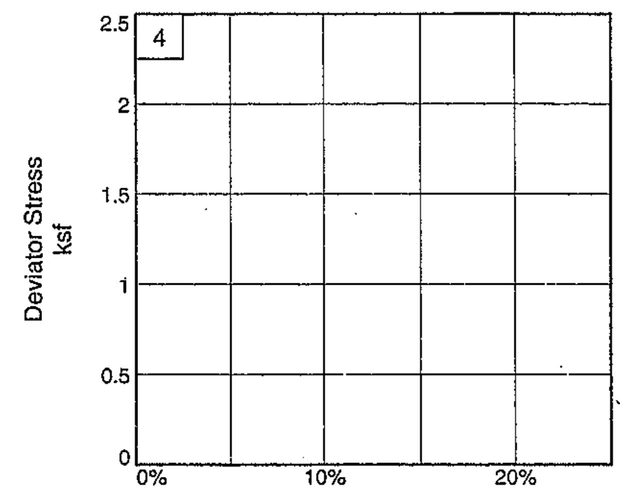
DLZ

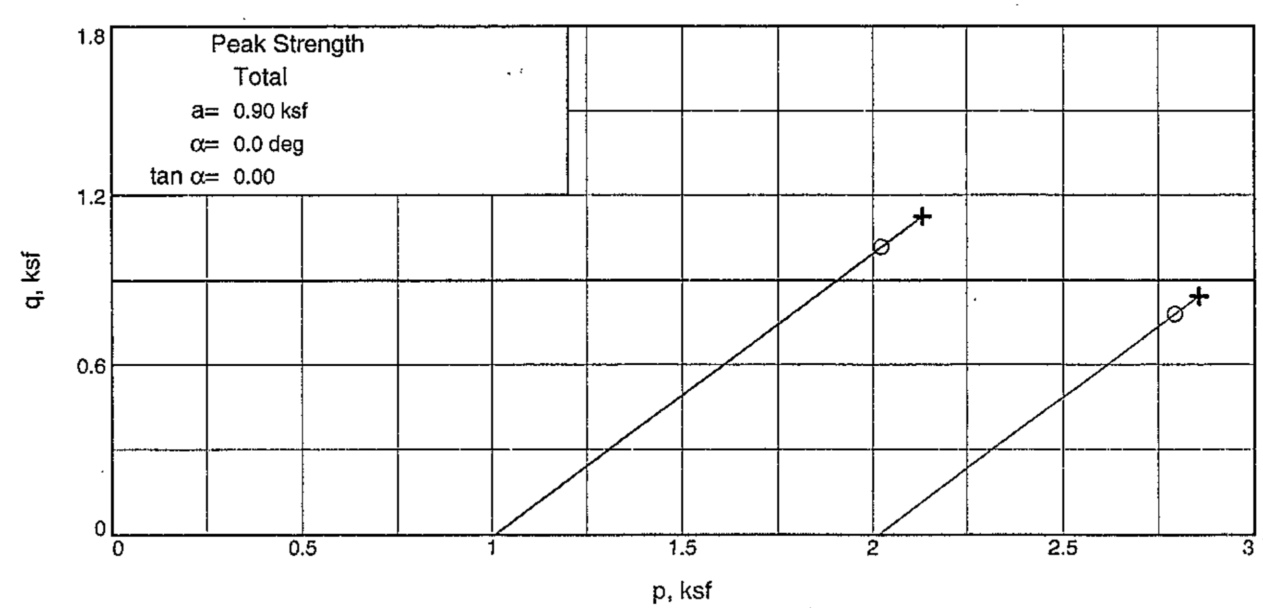
Figure _______







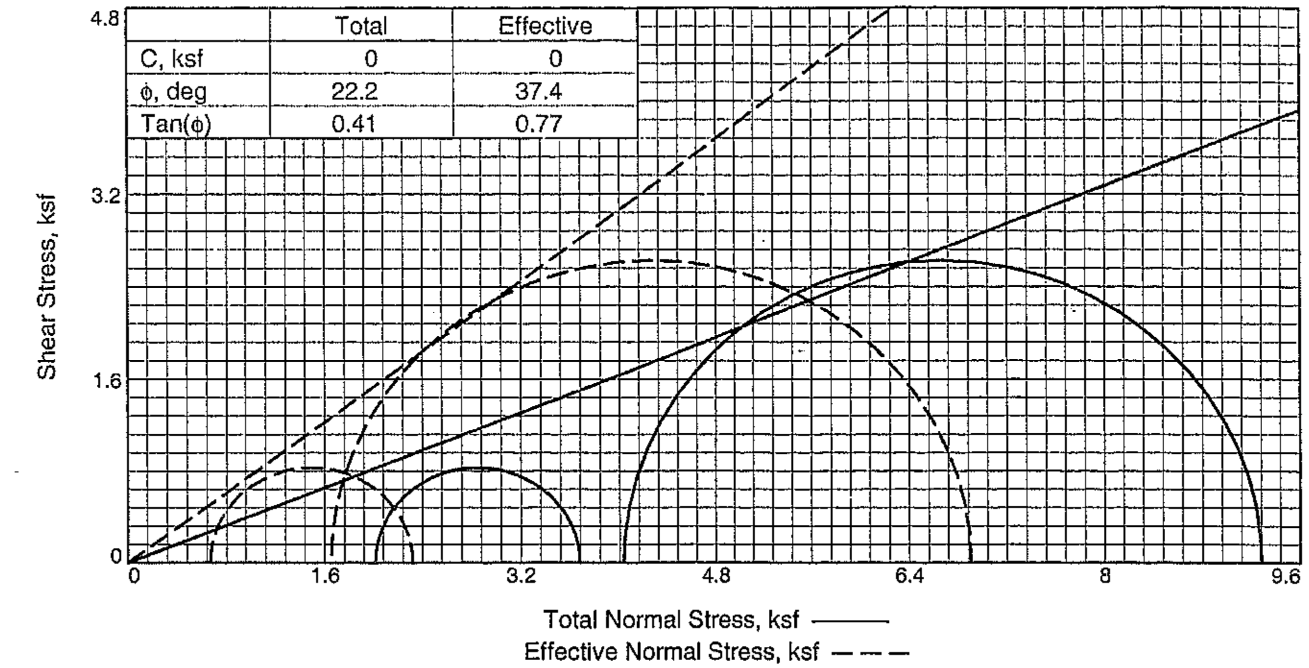




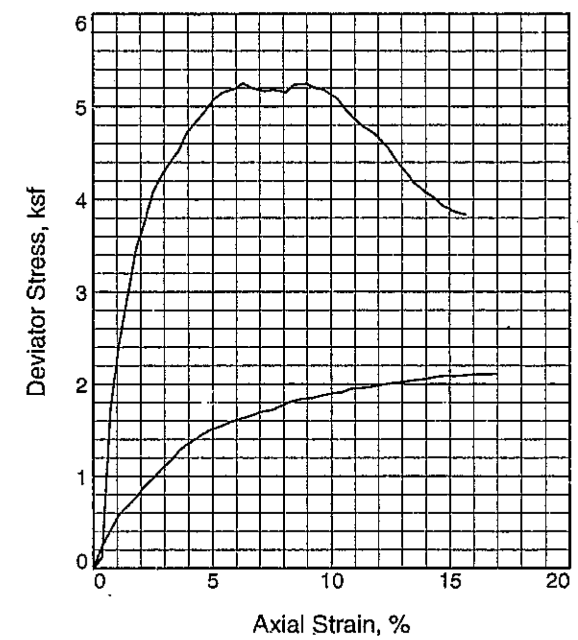
Stress Paths: o indicates peak + indicates end

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

DLZ, INC.



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



	Sample No.	1	2
Initial	Water Content,	28.4	29.1
	Dry Density, pcf	95.8	95.6
	Saturation,	98.7	100.4
	Void Ratio	0.7914	0.7964
	Diameter, in.	2.84	2.83
	Height, in.	5.56	5.56
At Test	Water Content,	26.3	25.7
	Dry Density, pcf	99.7	100.6
	Saturation,	100.0	100.0
	Void Ratio	0.7223	0.7068
	Diameter, in.	2.79	2.76
	Height, in.	5.56	5.56
	Strain rate, in./min.	0.06	0.06
	Back Pressure, ksf	8.06	8.06
	Cell Pressure, ksf	10.08	12.10
	Fail. Stress, ksf	1.66	5.25
	Total Pore Pr., ksf	9.42	10.45
	Ult. Stress, ksf	1.66	5.25
	Total Pore Pr., ksf	9.42	10.45
	$\bar{\sigma}_1$ Failure, ksf	2.32	6.90
	$\bar{\sigma}_3$ Failure, ksf	0.66	1.65

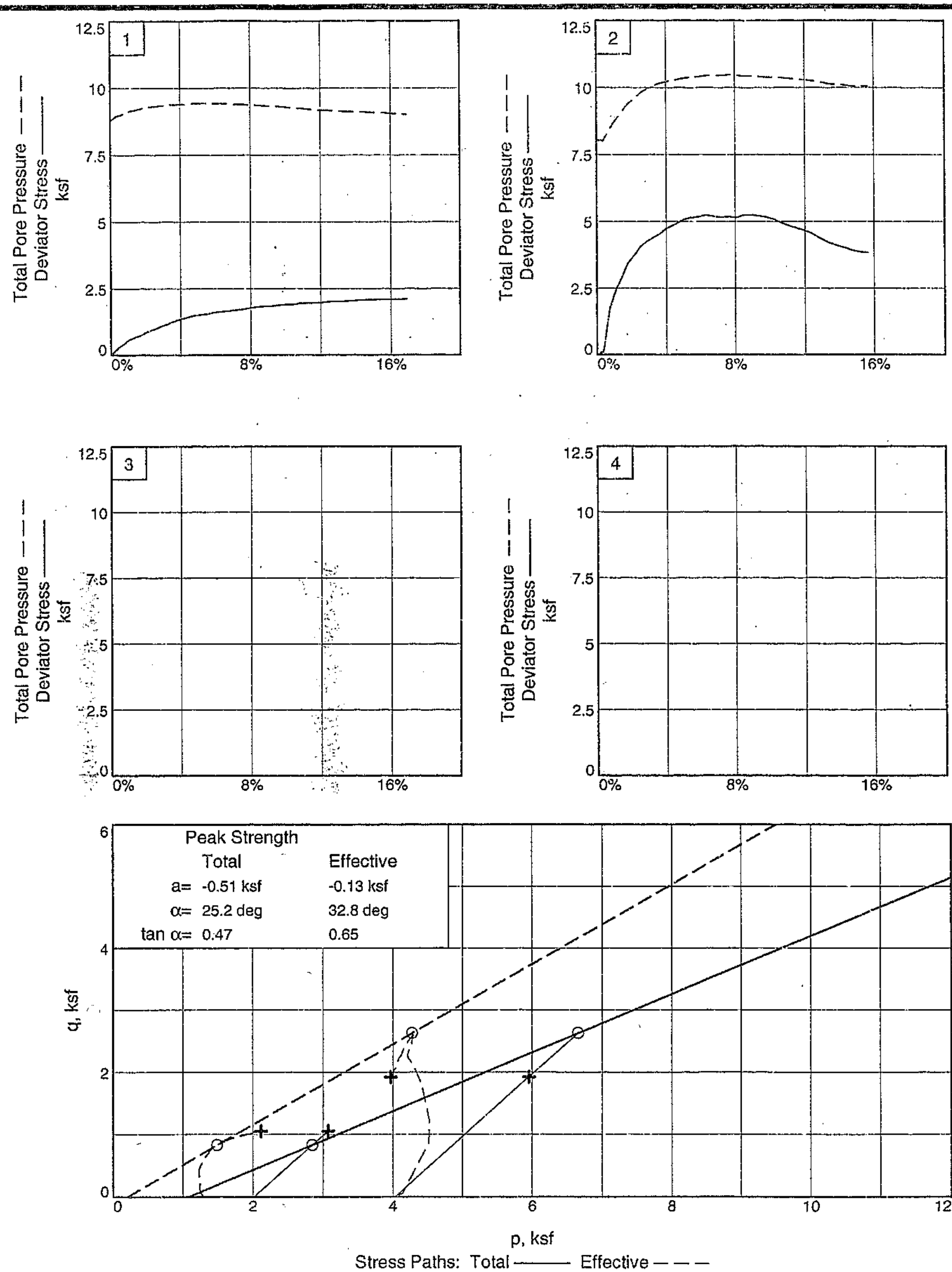
Type of Test:
CU with Pore Pressures
Sample Type: 3" Press TUbe
Description: Lean clay

LL= 38 **PL=** 19 **PI=** 19
Assumed Specific Gravity= 2.75
Remarks:

Figure ______

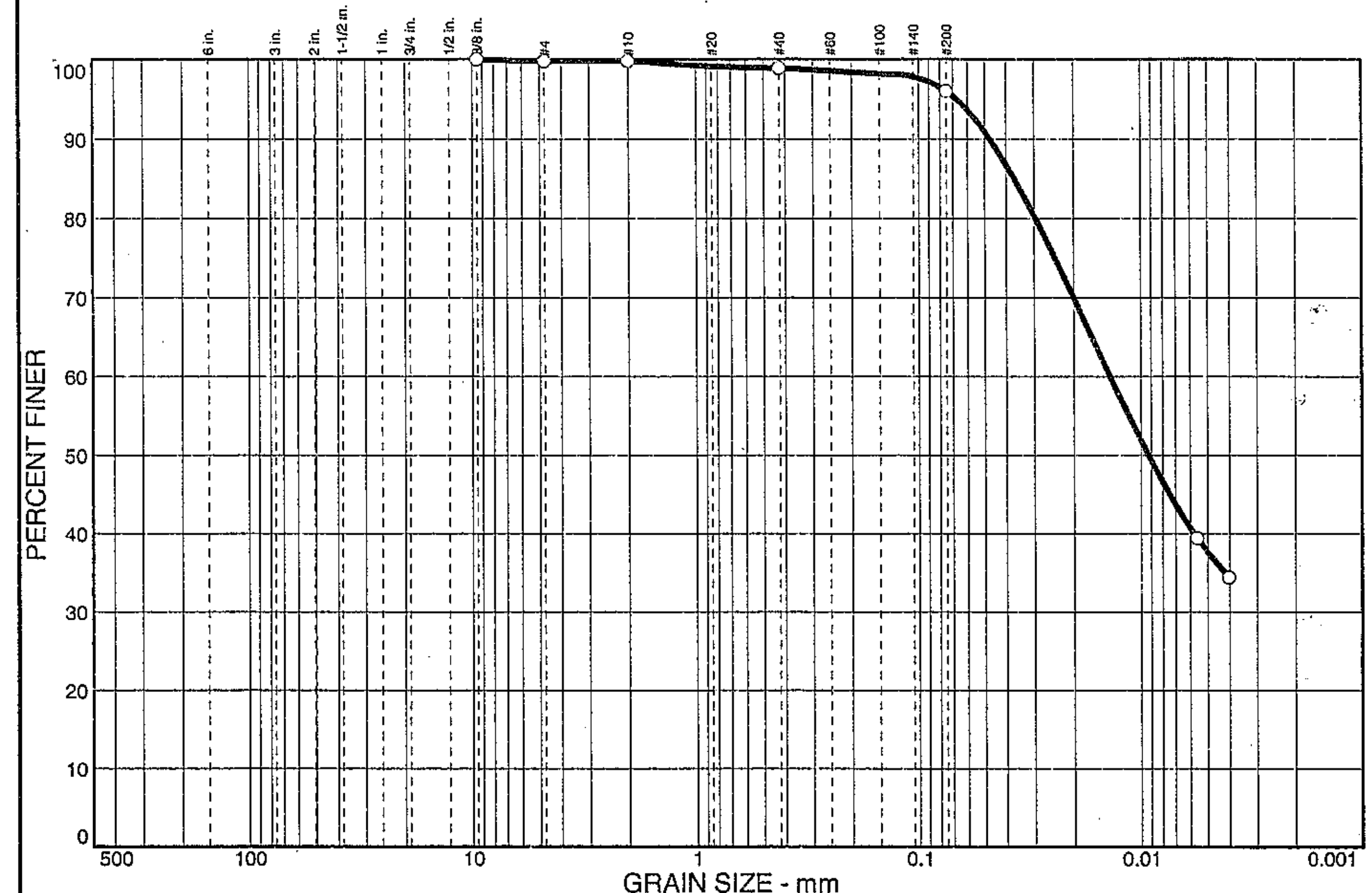
Client: TranSystems, Inc.
Project: SCI-823-0.00
Source of Sample: B-1108A **Depth:** 18.0
Sample Number: P3
Proj. No.: 0121-3070.03 **Date:** 08/16/06

DLZ



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

PARTICLE SIZE DISTRIBUTION TEST REPORT



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0.0	0.0	0.2	0.0	0.9	2.9	58.5	37.5

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.8		
#10	99.8		
#40	98.9		
#200	96.0		

* (no specification provided)

Soil Description

Lean clay,
Specific Gravity= 2.65

Atterberg Limits

PL= 21 LL= 36 PI= 15

Coefficients

D_{85}= 0.0370 D_{60}= 0.0138 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(15)

Remarks

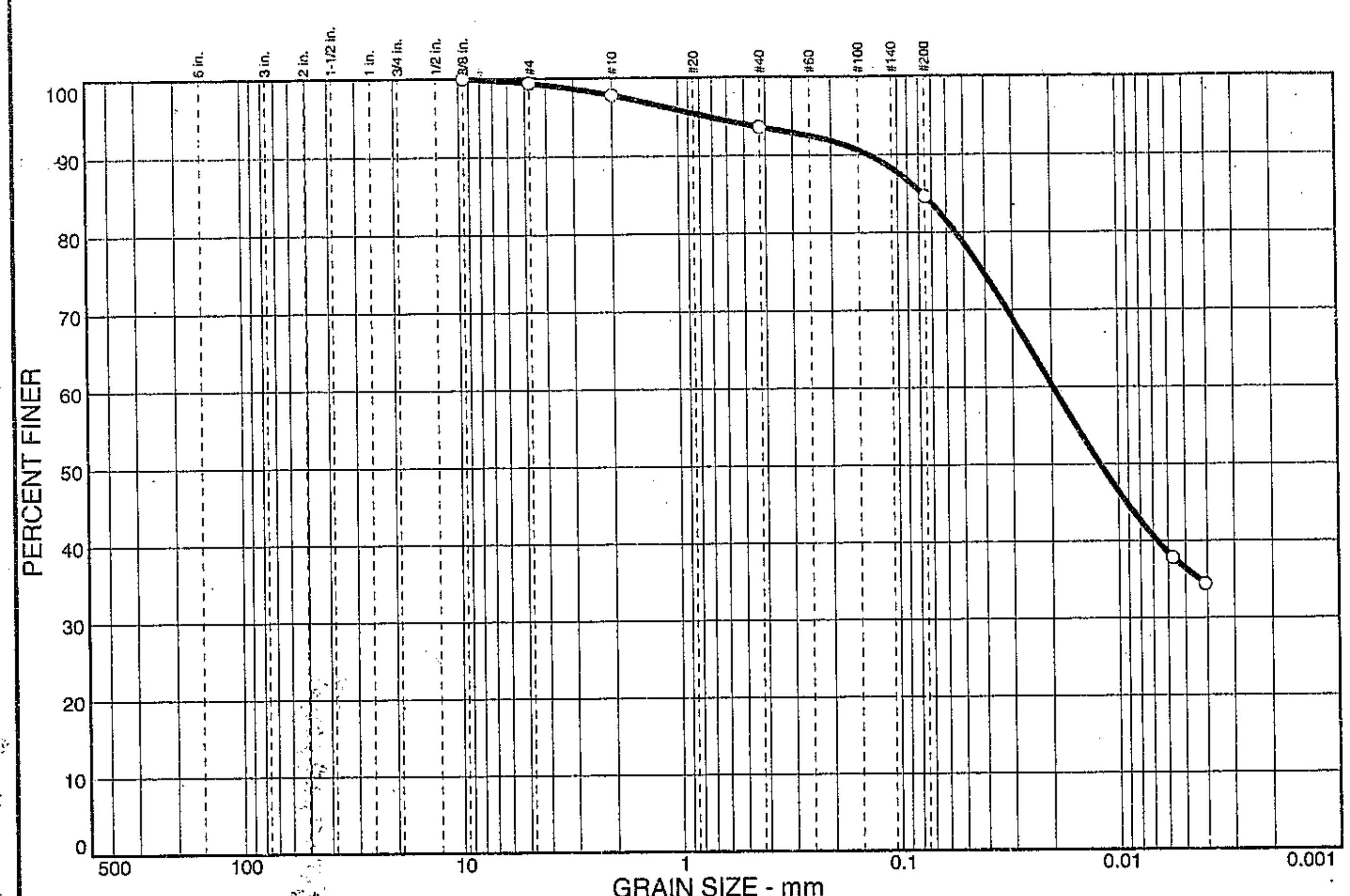
Moisture Content = 14.5%

Sample No.: P1 Source of Sample: B-1108A Date: 08/16/06
Location: Elev./Depth: 10.0

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	0.6	1.6	4.2	9.0	48.2	36.4

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.4		
#10	97.8		
#40	93.6		
#200	84.6		

* (no specification provided)

Soil Description

Lean clay with sand

Atterberg Limits

PL= 19 LL= 38 PI= 19

Coefficients

D_{85}= 0.0775 D_{60}= 0.0198 D_{50}= 0.0121
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(16)

Remarks

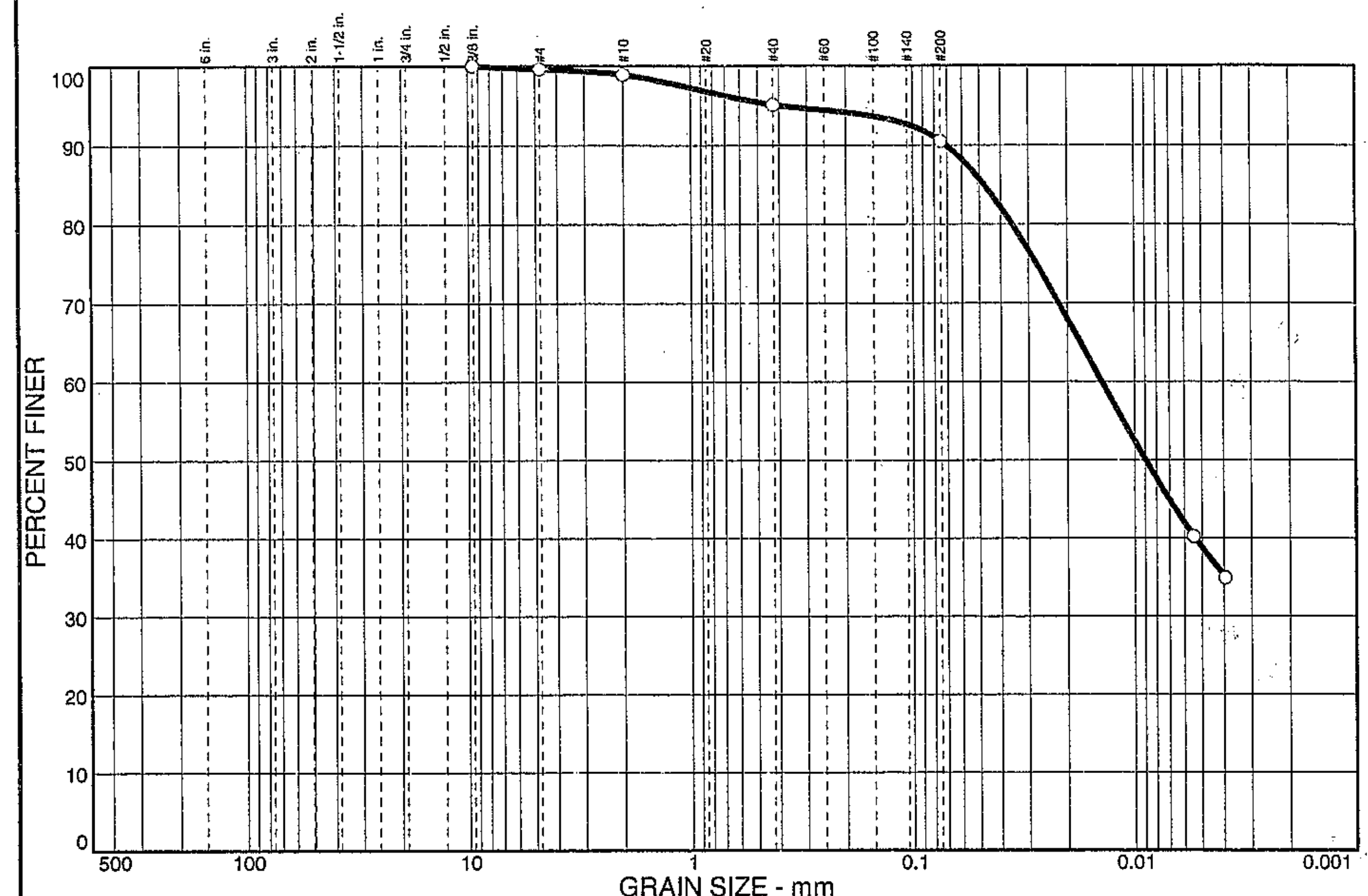
Moisture Content = 19.8%

Sample No.: P2 **Source of Sample:** B-1108A **Date:** 08/16/06
Location: **Elev./Depth:** 14.0

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	0.4	0.7	3.8	4.6	51.8	38.7

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
0.375 in.	100.0		
#4	99.6		
#10	98.9		
#40	95.1		
#200	90.5		

* (no specification provided)

Soil Description

Lean clay

Atterberg Limits

PL= 19 LL= 38 PI= 19

Coefficients

D_{85}= 0.0479 D_{60}= 0.0141 D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= CL AASHTO= A-6(17)

Remarks

Moisture Content = 24.0%

Sample No.: P3 **Source of Sample:** B-1108A **Date:** 08/16/06
Location: **Elev./Depth:** 18.0

DLZ

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

APPENDIX B

MSE Wall with Soil Surcharging & Temporary Fabric Faced Wall

Work Approach:

Calculate the unit cost ($/ft^2) of permanent MSE Wall for Walls 1 & 2 using the construction method of soil surcharging with temporary fabric faced MSE Walls built in phases. Because the wall heights are approximately equal assume unit cost of building Wall 1 is the same as the unit cost for building Wall 2. See attached typical wall sections for geometry. Assume sections are 1' long (into the page) in order to calculate exposed surface area of wall and associated quantities.

Item Unit Costs:

Temporary MSE Wall = $30 /ft^2 *unit cost includes cost of select granular within strap length area

Select Granular Embankment = $30 /cy

Item 203 Embankment = $12 /cy

Wick Drains = $1 /ft

Excavation = $10 /cy

Permanent MSE Wall = $85 /ft^2 *unit cost includes Item 203, select granular, wall panels & straps, and wall excavation below grade per Figure 330 in ODOT BDM

Exposed Surface Area of Permanent MSE Wall (final condition):

From Step 6
Height = 19.67 ft
Length = 1.00 ft

Area = 19.7 ft^2

Cost of Temporary MSE Wall:

From Step 3
Height = 30.00 ft
Length = 1.00 ft

Area of Wall Face = 30.0 ft^2

Unit Cost = $30 /ft^2

Volume Of Select Granular (included with this cost)
Height = 30.00 ft
Width = 30.00 ft
Length = 1.00 ft
Volume = 33.3 cy

Cost of Temporary MSE Wall = $900

Cost of Additional Select Granular Embankment:

From Step 3 - Select Granular that is not included with cost of Temporary Fabric Faced Wall (any select granular outside of strap length required for wall height)

Height = 30.00 ft
Width = 30.00 ft
Length = 1.00 ft

Volume = 16.7 cy (triangular wedge of select granular backfill)

Unit Cost = $30 /cy

Cost of Additional Select Granular Embankment = $500

CH2MHILL Alternative 1 Wall Unit Cost D. Stachler
March 26, 2007
Portsmouth By-Pass

Cost of Item 203 Embankment:

From Step 3

Height = 30.00 ft
Width = 30.00 ft
Length = 1.00 ft

Volume = 16.7 cy (triangular wedge of Item 203 Embankment)

Unit Cost = $12 /cy

Cost of Item 203 Embankment = $200

Cost of Wick Drains:

Assume wick drains are spaced at 7' c/c in order to get 90% consolidation @ 45 days. Account for wick drains that are located within the boundaries of the embankment being paid for with the MSE Wall. Assume that wick drains are located 10' past the face of the MSE Wall.

Depth of Wick Drains = 13.5 ft (approximate distance from existing ground to bedrock at Wall 1)

No. Wick Drains / 1' Wall Length = [(60'+10') / 7' Spa.] * (1' / 7' Spa.) = 1.429

Length = 19.3 ft

Unit Cost = $1 /ft

Cost of Wick Drains = $20

Cost of Excavating Temporary MSE Wall:

Volume to excavate equals total volume of select granular in place at Step 3.

Volume of Excavation = 50.0 cy

Unit Cost = $10 /cy

Cost of Excavating Temporary MSE Wall = $500

CH2MHILL

Alternative 1 Wall Unit Cost

D. Stachler
March 26, 2007
Portsmouth By-Pass

Cost of Building Permanent MSE Wall:

Wall Face Area = 19.7 ft^2

Unit Cost = \$85 /ft^2

Conventional MSE Wall Cost = \$1,672

Volume Of Select Granular needed for Permanent MSE Wall (See Wall Section Step 6):

(18.667'+3') * (21'-0.5') * 1' =	444.17 ft^3
(21'-3.5'-6') * 3' * 1' =	34.5 ft^3
(1/2) * (18.667'+3'+3')^2 * 1' =	304.23 ft^3
Total =	782.90 ft^3

Say 29.0 cy of Select Granular

Subtract out the cost associated with the select granular needed for the Permanent MSE Wall since the select granular will be purchased during construction of the Temporary MSE Wall.

Select Granular Volume = 29.0 cy

Unit Cost = \$30 /cy

Cost of Select Granular = \$870

1672 - 870 = 802

Permanent MSE Wall Cost = Conventional MSE Wall Cost - Cost of Select Granular

Cost of Permanent MSE Wall = \$802

Total Cost to Construct 19.7 ft^2 of Permanent MSE Wall = \$2,922

Wall Unit Cost for Alternative 1 = \$150 /ft^2

Exposed Area of Wall 1 = 5200 ft^2
Estimated Wall 1 Cost for Alternative 1 = \$780,000

Exposed Area of Wall 2 = 6150 ft^2
Estimated Wall 2 Cost for Alternative 1 = \$923,000

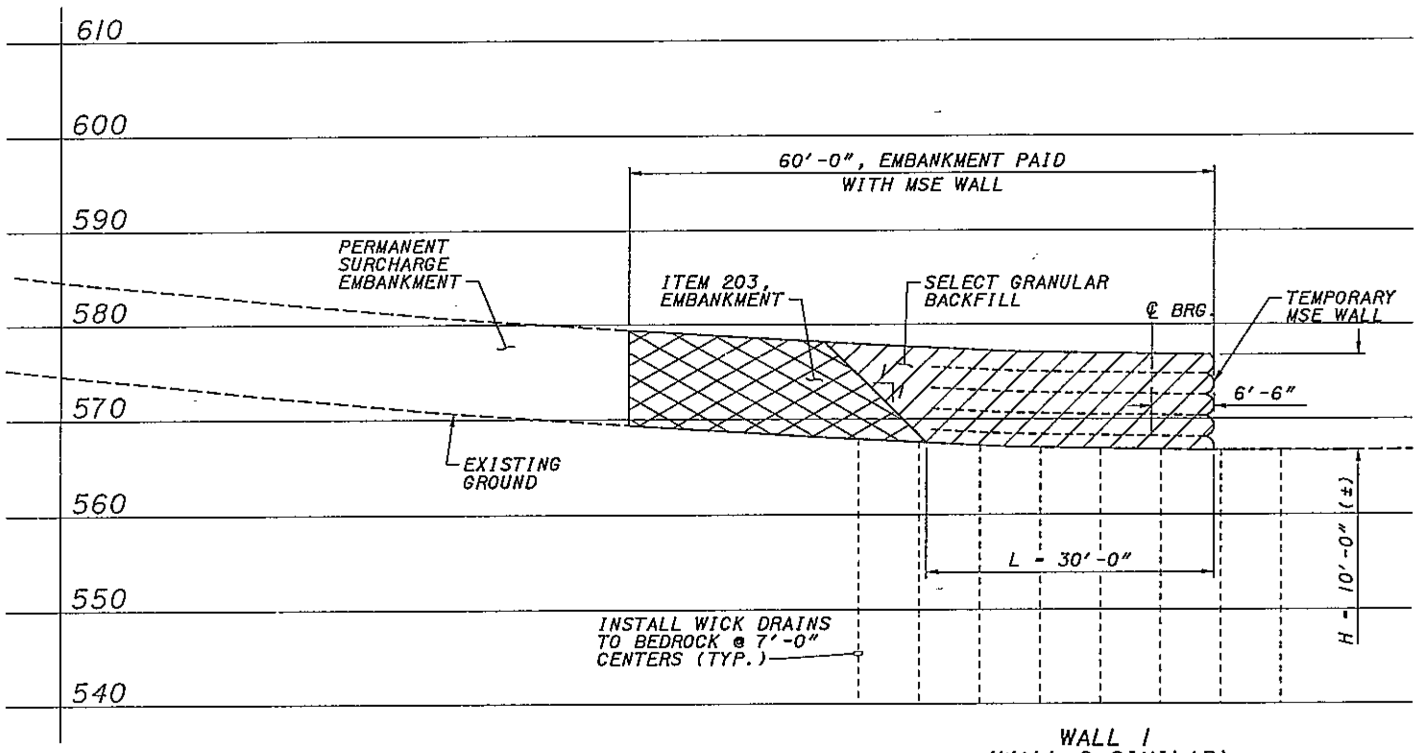
900
500
200
20
500
2120
802
2922

\$2922 / 19.7 = \$150/SF

150 x 5200 = \$780,000
150 x 6150 = 923,000

MSE WALL ALTERNATIVE 1: STEP 1
BUILD SURCHARGE AND TEMP. WALL

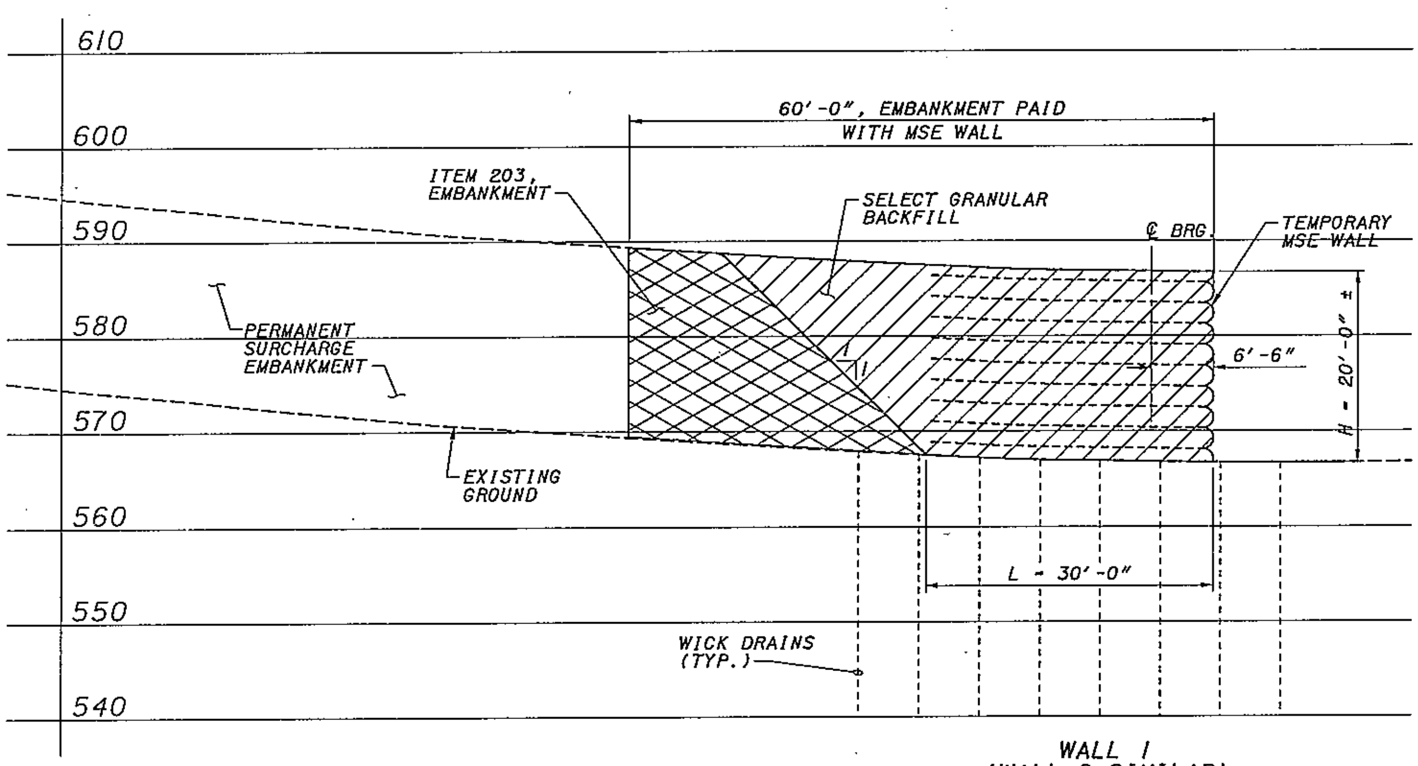
t_{90} = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.



WALL 1
(WALL 2 SIMILAR)

MSE WALL ALTERNATIVE 1: STEP 2
BUILD SURCHARGE AND TEMP. WALL

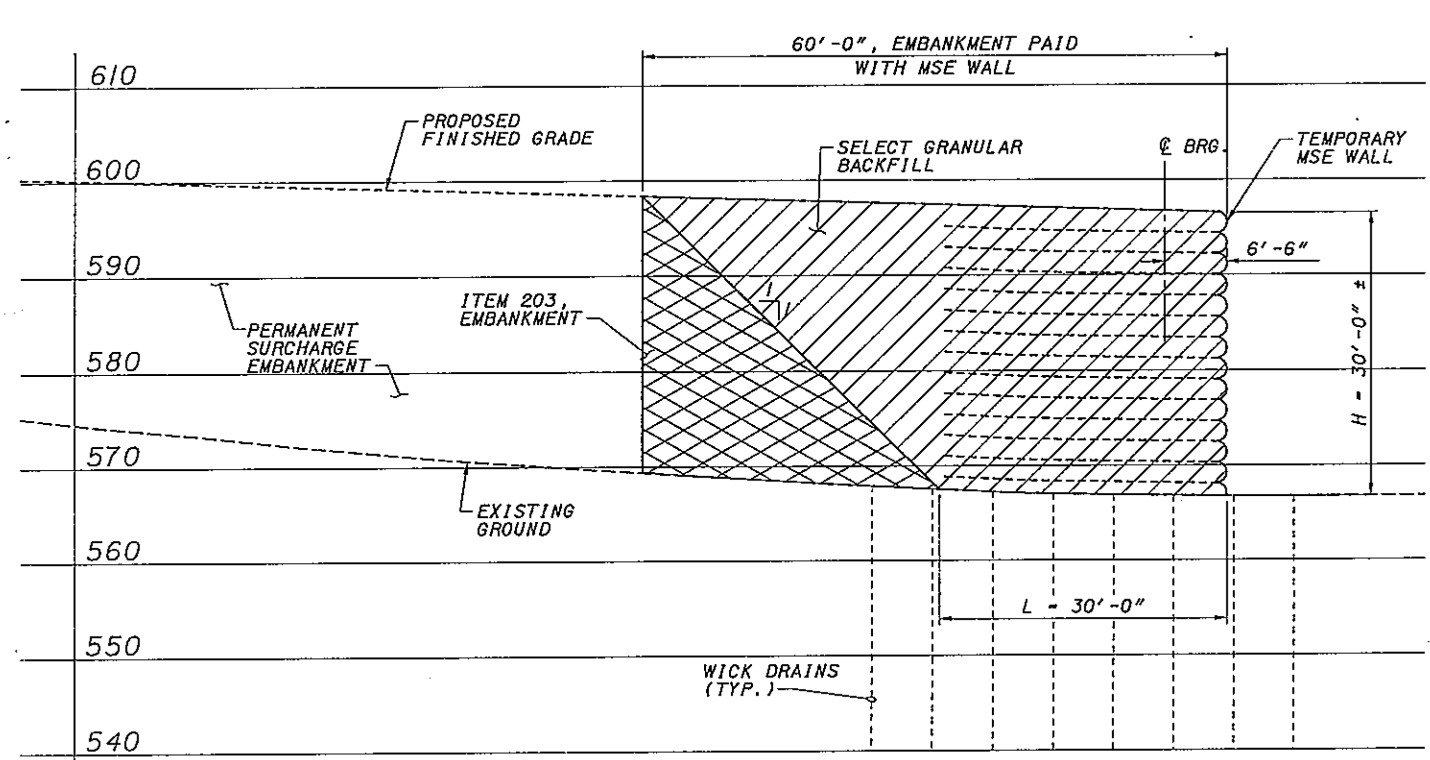
t_{90} = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.



WALL 1
(WALL 2 SIMILAR)

MSE WALL ALTERNATIVE 1: STEP 3
BUILD SURCHARGE AND TEMP. WALL

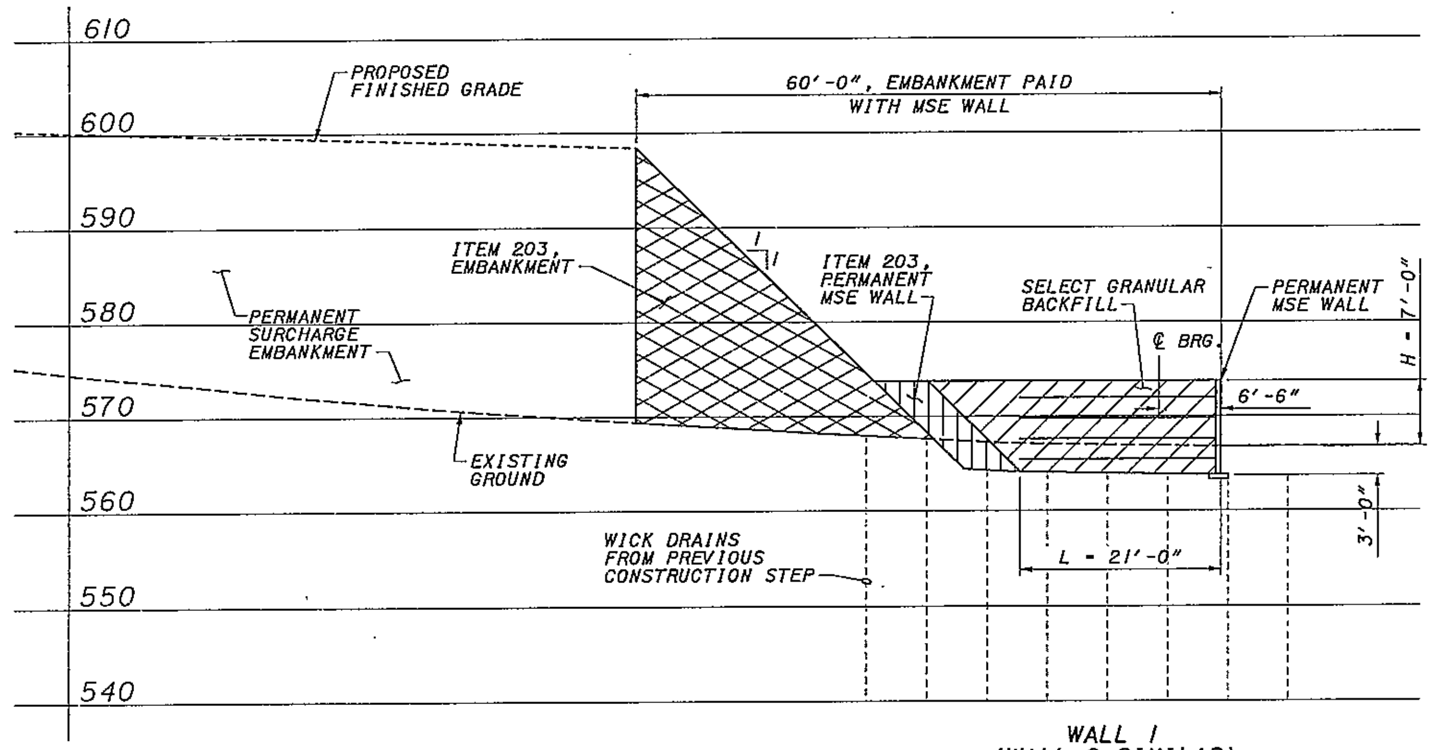
t_{90} = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.



WALL 1
(WALL 2 SIMILAR)

MSE WALL ALTERNATIVE 1: STEP 4
EXCAVATE SURCHARGE & REMOVE TEMP. WALL & BUILD 10 FT. LIFT OF PERMANENT MSE WALL

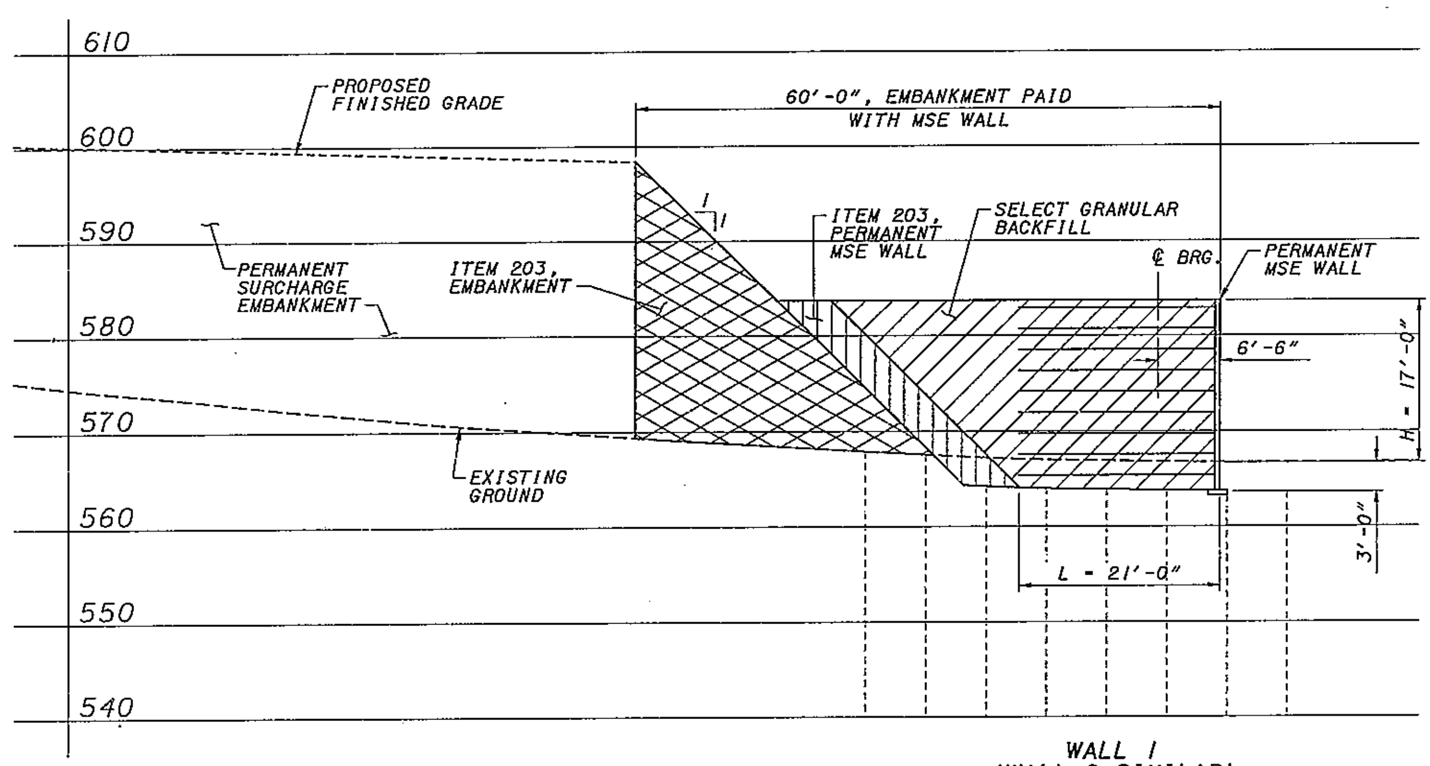
t90 = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.



WALL 1
(WALL 2 SIMILAR)

MSE WALL ALTERNATIVE 1: STEP 5
BUILD SECOND 10 FT. LIFT OF PERMANENT MSE WALL

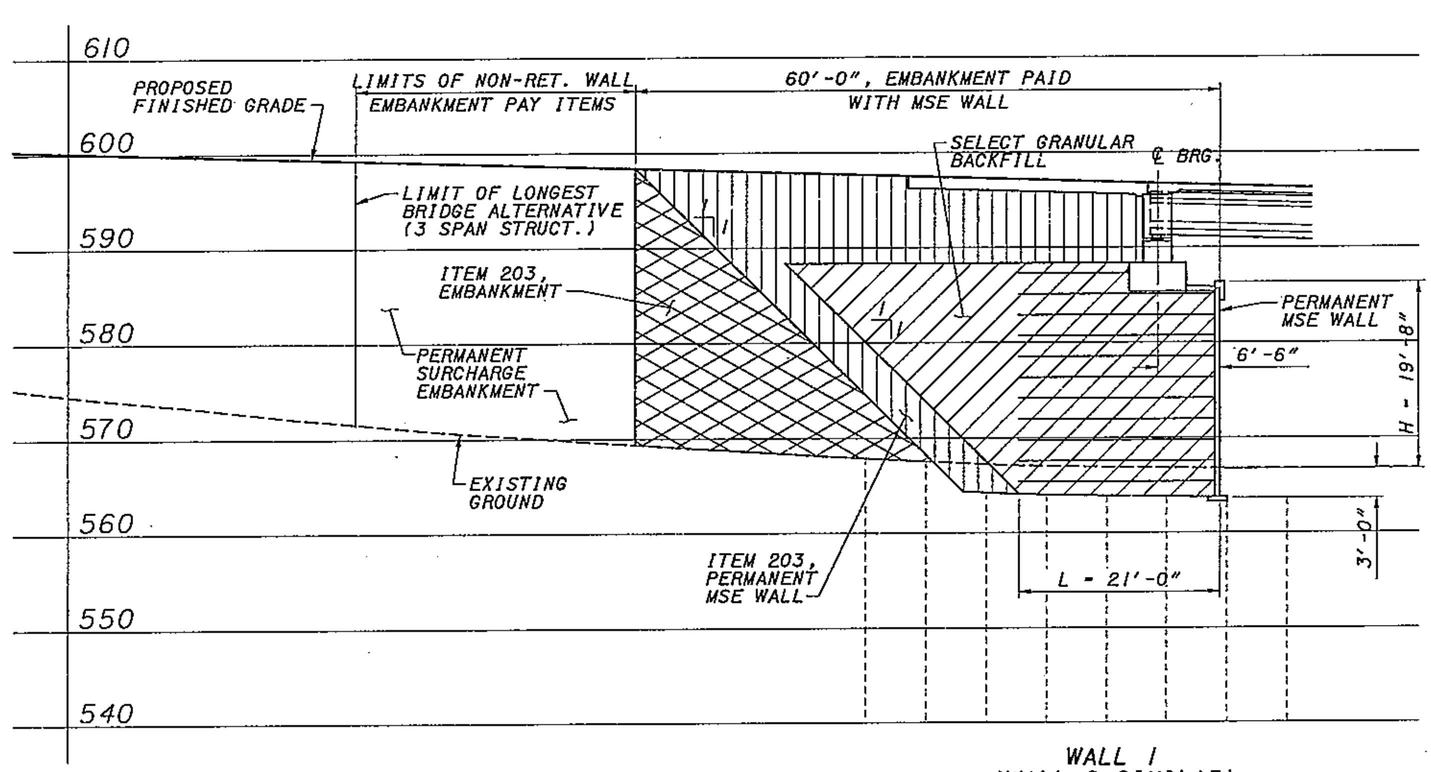
t90 = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.



WALL 1
(WALL 2 SIMILAR)

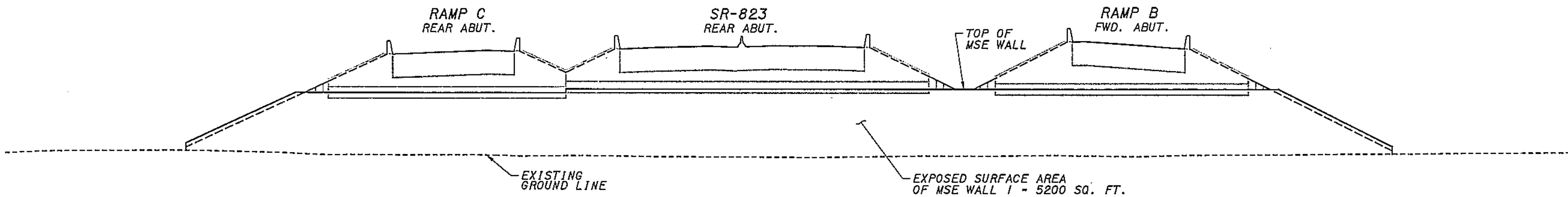
MSE WALL ALTERNATIVE 1: STEP 6
BUILD FINAL LIFT OF PERMANENT MSE WALL & CONSTRUCT BRIDGE

t90 = 110 DAYS W/O WICK DRAINS
= 45 DAYS W/ WICK DRAINS @ 7' SPA.

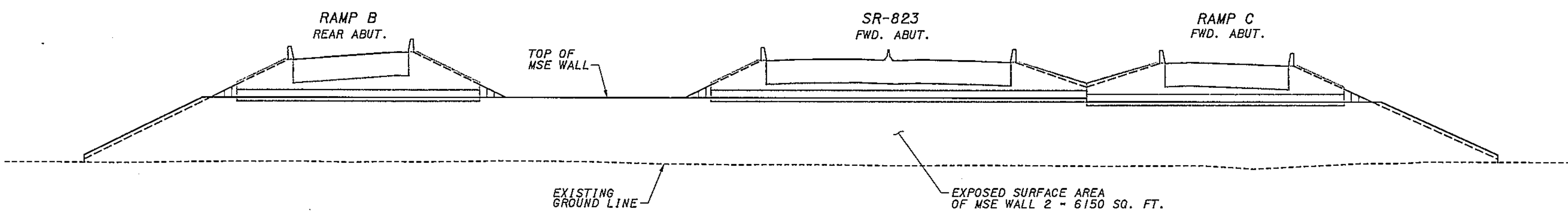


WALL 1
(WALL 2 SIMILAR)

MSE WALL SURFACE AREAS FOR ALTERNATIVES 1 & 2



WALL 1



WALL 2

MSE Wall with Deep Soil Mixing
Wall 1

Work Approach:

Calculate the unit cost ($/ft^2) of permanent MSE Wall for Walls 1 & 2 using the construction method of deep soil mixing ground remediation followed by construction of conventional MSE Walls. Cost of deep soil mixing will be dependent on depth of bedrock so must calculate independent unit costs for Wall 1 and Wall 2. See attached typical wall sections for geometry. Assume sections are 1' long (into the page) in order to calculate exposed surface area of wall and associated quantities.

Item Unit Costs:

Deep Soil Mixing = $80 /cy

Permanent MSE Wall = $85 /ft^2 — *unit cost includes Item 203, select granular, wall panels & straps, and wall excavation below grade per Figure 330 in ODOT BDM

Exposed Surface Area of Permanent MSE Wall 1:

Height = 19.67 ft
Length = 1.00 ft

Area = 19.7 ft^2

Approximate Bedrock Elevation:

Average Top of Rock Elevation for three borings taken in proximity of Wall 1 (TR-58, TR-56, & TR-54)

Boring No.	Top of Rock Elev.
TR-58 (Ramp B)	552.1
TR-56 (SR 823)	555.0
TR-54 (Ramp C)	551.9

Average Top of Rock at Wall 1 = 553.0

Cost of Deep Soil Mixing:

See Alternative 2 Wall 1 Section for limits of deep soil mixing - begin soil mixing 5' below existing ground and continue to depth of bedrock. Assume soil mixing extends 10' beyond face of MSE Wall and 10' beyond end of wall straps.

Volume of Soil within Soil Mixing Limits:
Height = 8.75 ft
Width = 41.0 ft
Length = 1.0 ft

Volume = 13.3 cy

Assume 80% of volume is soil-mixed

Volume of Soil Mixing = 10.6 cy

Unit Cost = $80 /cy

Cost of Soil Mixing = $850

Cost of Building Conventional MSE Wall:

Wall Face Area = 19.7 ft^2

Unit Cost = $85 /ft^2

Cost of Conventional MSE Wall = $1,672

Total Cost to Construct 19.7 ft^2 of Permanent MSE Wall = $2,522

Wall Unit Cost for Alternative 2 (Wall 1) = $130 /ft^2

Exposed Area of Wall 1 = 5200 ft^2
Estimated Wall 1 Cost for Alternative 2 = $676,000

MSE Wall with Deep Soil Mixing
Wall 2

Work Approach:

Calculate the unit cost ($/ft^2) of permanent MSE Wall for Walls 1 & 2 using the construction method of deep soil mixing ground remediation followed by construction of conventional MSE Walls. Cost of deep soil mixing will be dependent on depth of bedrock so must calculate independent unit costs for Wall 1 and Wall 2. See attached typical wall sections for geometry. Assume sections are 1' long (into the page) in order to calculate exposed surface area of wall and associated quantities.

Item Unit Costs:

Deep Soil Mixing = $80 /cy

Permanent MSE Wall = $85 /ft^2 *unit cost includes Item 203, select granular, wall panels & straps, and wall excavation below grade per Figure 330 in ODOT BDM

Exposed Surface Area of Permanent MSE Wall 2:

Height = 18.13 ft
Length = 1.00 ft

Area = 18.1 ft^2

Approximate Bedrock Elevation:

Average Top of Rock Elevation for three borings taken in proximity of Wall 2 (TR-59A, TR-55A, & TR-53A)

Boring No.	Top of Rock Elev.
TR-59A (Ramp B)	538.9
TR-55A (SR 823)	545.4
TR-53A (Ramp C)	542.8

Average Top of Rock at Wall 2 = 542.4

Cost of Deep Soil Mixing:

See Alternative 2 Wall 2 Section for limits of deep soil mixing - begin soil mixing 5' below existing ground and continue to depth of bedrock. Assume soil mixing extends 10' beyond face of MSE Wall and 10' beyond end of wall straps.

Volume of Soil within Soil Mixing Limits:
Height = 17.833 ft
Width = 40.0 ft
Length = 1.0 ft

Volume = 26.4 cy

Assume 80% of volume is soil-mixed

Volume of Soil Mixing = 21.1 cy

Unit Cost = $80 /cy

Cost of Soil Mixing = $1,691

CH2MHILL Alternative 2 Wall Unit Cost D. Stachler
March 26, 2007
Portsmouth By-Pass

Cost of Building Conventional MSE Wall:

Wall Face Area = 18.1 ft^2

Unit Cost = $85 /ft^2

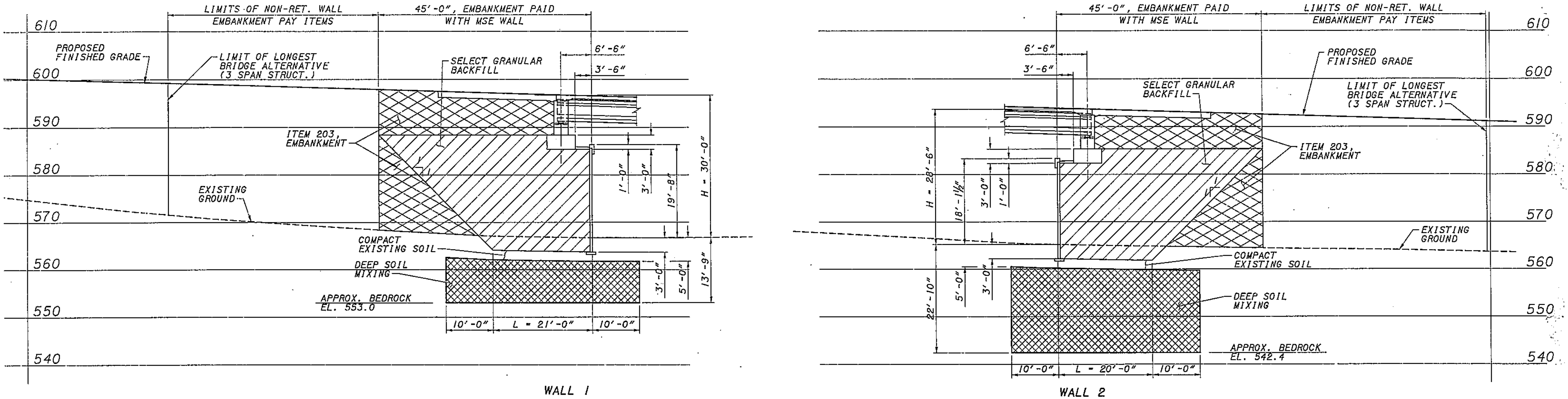
Cost of Conventional MSE Wall = $1,541

Total Cost to Construct 18.1 ft^2 of Permanent MSE Wall = $3,231

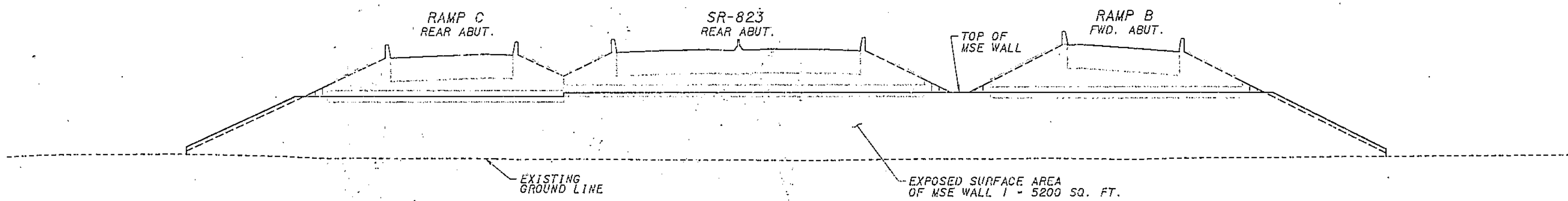
Wall Unit Cost for Alternative 2 (Wall 2) = $180 /ft^2

Exposed Area of Wall 2 = 6150 ft^2
Estimated Wall 2 Cost for Alternative 2 = $1,107,000

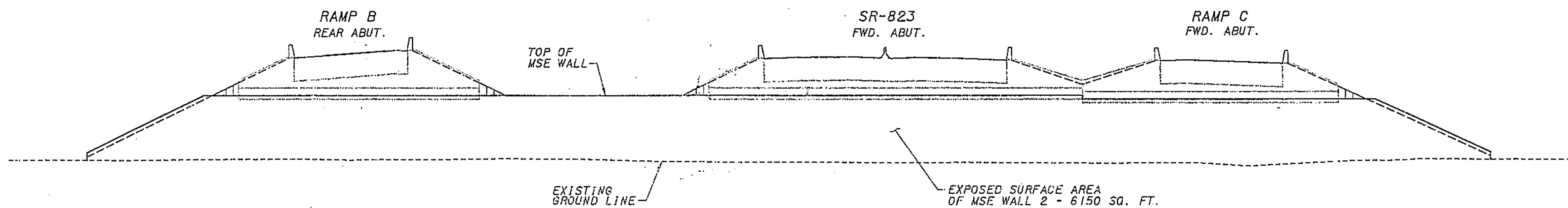
MSE WALL ALTERNATIVE 2:
DEEP SOIL MIXING WITH CONVENTIONAL MSE WALL



MSE WALL SURFACE AREAS FOR ALTERNATIVES 1 & 2



WALL 1



WALL 2

CIP Retaining Wall with Soil Surcharging & Temporary Fabric Faced Wall

Work Approach:

Calculate the unit cost ($/ft²) of permanent CIP Retaining Walls for Walls 1 & 2 using the construction method of soil surcharging with temporary fabric faced MSE Walls built in phases. Assume unit cost of building Wall 1 is the same as the unit cost for building Wall 2. See attached typical wall sections for geometry. Assume sections are 1' long (into the page) in order to calculate exposed surface area of wall and associated quantities.

Item Unit Costs:

Temporary MSE Wall = $30 /ft² *unit cost includes cost of select granular within strap length area

Select Granular Embankment = $30 /cy

Item 203 Embankment = $12 /cy

Wick Drains = $1 /ft

Excavation = $10 /cy

CIP Retaining Wall Concrete = $396 /cy

Reinforcing Steel = $0.81 /lb

Steel H-Piles $36.30 /ft

Exposed Surface Area of CIP Retaining Wall (final condition):

Height = 20.00 ft *Use an average wall height of 20'
Length = 1.00 ft

Area = 20.0 ft²

Cost of Temporary MSE Wall:

From Step 2
Height = 20.00 ft
Length = 1.00 ft

Area of Wall Face = 20.0 ft²

Unit Cost = $30 /ft²

Volume Of Select Granular (included with this cost)
Height = 20.00 ft
Width = 20.00 ft
Length = 1.00 ft
Volume = 14.8 cy

Cost of Temporary MSE Wall = $600

Cost of Additional Select Granular Embankment:

From Step 2 - Select Granular that is not included with cost of Temporary Fabric Faced Wall (any select granular outside of strap length required for wall height)

Height = 20.00 ft
Width = 20.00 ft
Length = 1.00 ft

Volume = 7.4 cy (triangular wedge of select granular backfill)

Unit Cost = $30 /cy

Cost of Additional Select Granular Embankment = $230

Cost of Item 203 Embankment:

From Step 2

Height = 20.00 ft
Width = 20.00 ft
Length = 1.00 ft

Volume = 7.4 cy (triangular wedge of Item 203 Embankment)

Unit Cost = $12 /cy

Cost of Item 203 Embankment = $90

Cost of Wick Drains:

Assume wick drains are spaced at 7' c/c. Account for wick drains that are located within the boundaries of the embankment being paid for with the CIP Wall. Assume that wick drains are located 10' past the face of the CIP Wall.

Depth of Wick Drains = 13.5 ft (approximate distance from existing ground to bedrock at Wall 1)

No. Wick Drains / 1' Wall Length = [(40'+10') / 7' Spa.] * (1' / 7' Spa.) = 1.020

Length = 13.8 ft

Unit Cost = $1 /ft

Cost of Wick Drains = $20

Cost of Excavating Temporary MSE Wall:

Volume to excavate equals total volume of select granular in place at Step 2.

Volume of Excavation = 22.2 cy

Unit Cost = $10 /cy

Cost of Excavating Temporary MSE Wall = $230

Cost of Excavating for CIP Wall Footing:

Step 3
Height = 5.50 ft
Width = 26.00 ft
Length = 1.00 ft

Volume of Excavation = 5.3 cy

Unit Cost = $10 /cy

Cost of Excavating for CIP Wall Footing = $60

Cost of CIP Retaining Wall Concrete:

Wall Section Geometry: Use a weighted average wall height for approximate wall section geometry

Stem:			Footing:		
Height =	22.50	ft	Height =	3.00	ft
Width =	2.00	ft	Width =	15.00	ft
Length =	1.00	ft	Length =	1.00	ft
Stem Volume =	1.7	cy	Footing Volume =	1.7	cy

Total Volume of Wall Section = 3.3 cy

Unit Cost = $396 /cy

Cost of CIP Retaining Wall Concrete = $1,320

Cost of CIP Retaining Wall Reinforcing Steel:

Assume 125 lbs of reinforcing steel per cubic yard of concrete

Total Volume of Wall Section = 3.3 cy

Reinforcing Steel Wt. = 416.7 lbs

Unit Cost = $0.81 /lb

Cost of CIP Retaining Wall Reinforcing Steel = $340

Cost of Steel H-Piles:

Assume 20 foot pile length

Assume three rows of piles:
Front row of piles spaced @ 6 feet c/c
Middle row of piles spaced @ 6 feet c/c
Back row of piles spaced @ 8 feet c/c

Estimate Number of Piles Per foot of wall length:
(1' / 6' Spa Front Row) + (1' / 6' Spa Middle Row) + (1' / 8' Spa Back Row)
No. Piles = 0.458 piles / 1' of wall length

Length of Piles = 9.2 feet of piles / 1' of wall length

Unit Cost = $36.30 /ft

Cost of Steel H-Piles = $340

Total Cost to Construct 20 ft^2 of CIP Retaining Wall = $3,230

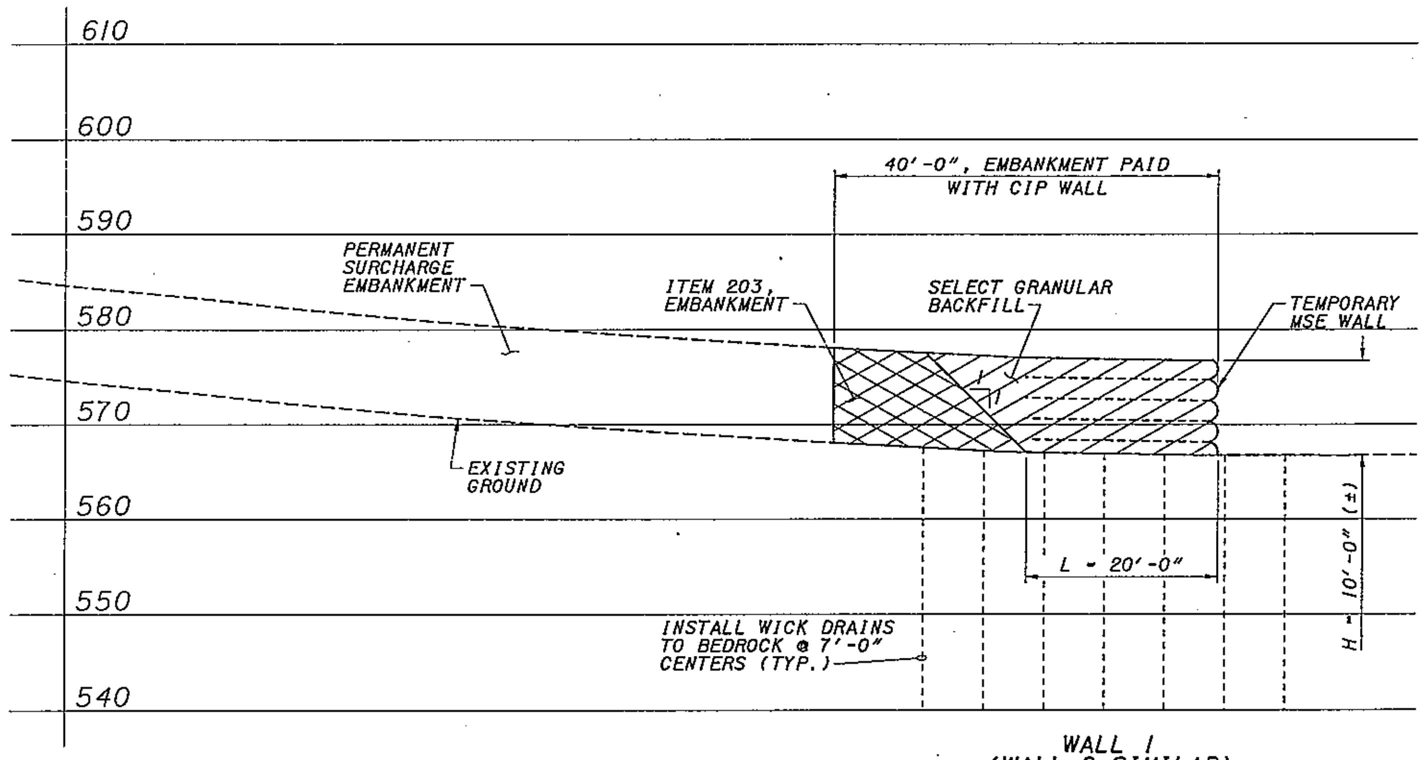
Wall Unit Cost for Alternative 5 = $165 /$ft^2$

Exposed Area of Wall 1 = 3555 ft^2
Estimated Wall 1 Cost for Alternative 5 = $587,000

Exposed Area of Wall 2 = 4465 ft^2
Estimated Wall 2 Cost for Alternative 5 = $737,000

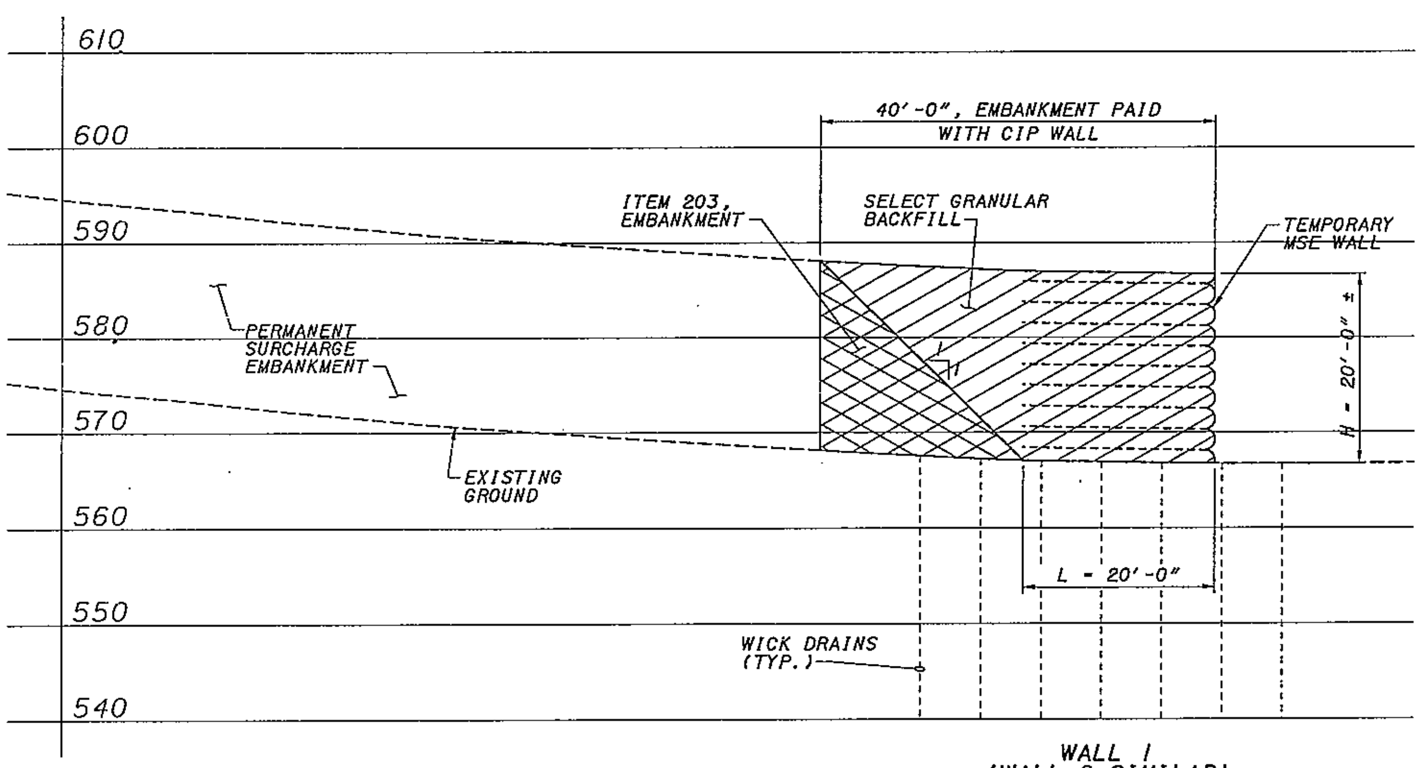
CIP RETAINING WALL ALTERNATIVE 5: STEP 1
BUILD SURCHARGE AND TEMP. WALL

t90 - 110 DAYS W/O WICK DRAINS
- 45 DAYS W/ WICK DRAINS @ 7' SPA.

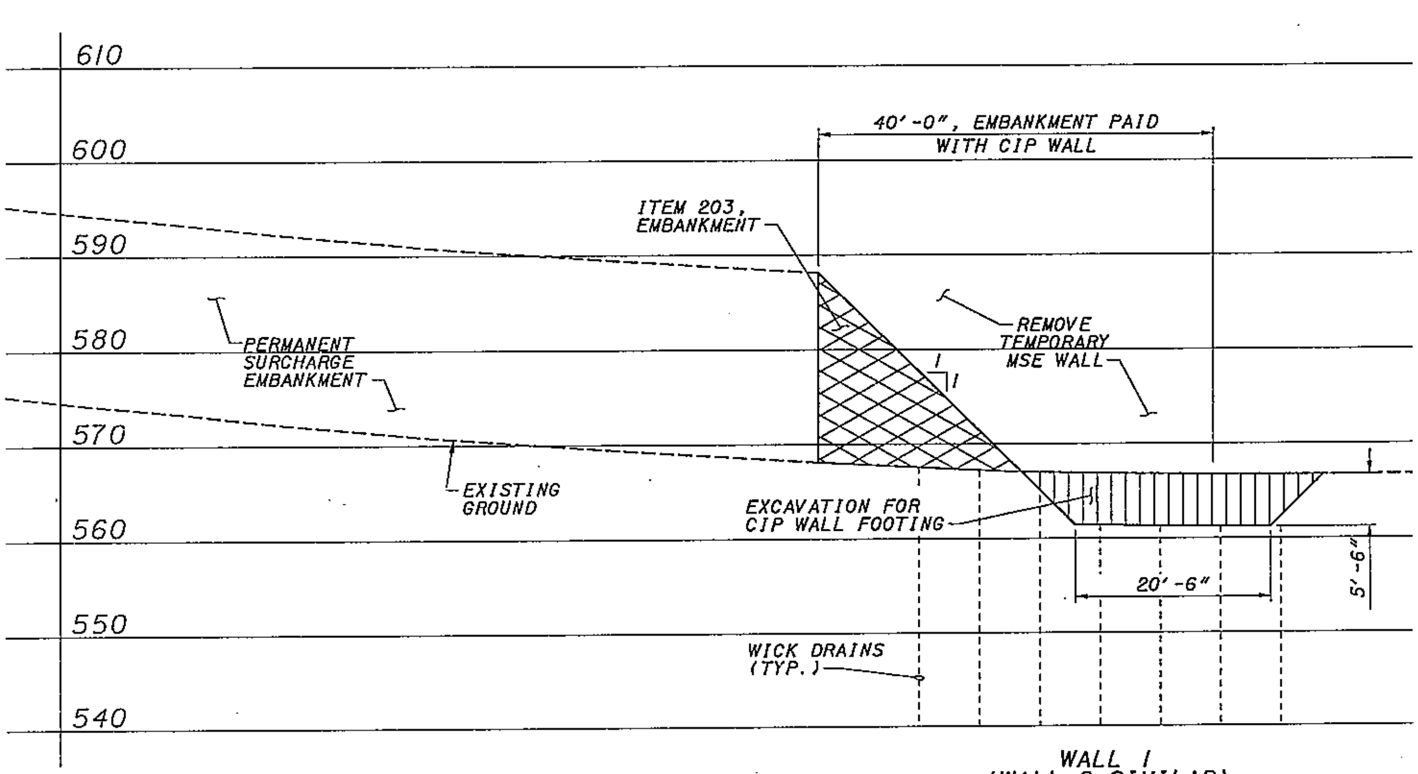


CIP RETAINING WALL ALTERNATIVE 5: STEP 2
BUILD SURCHARGE AND TEMP. WALL

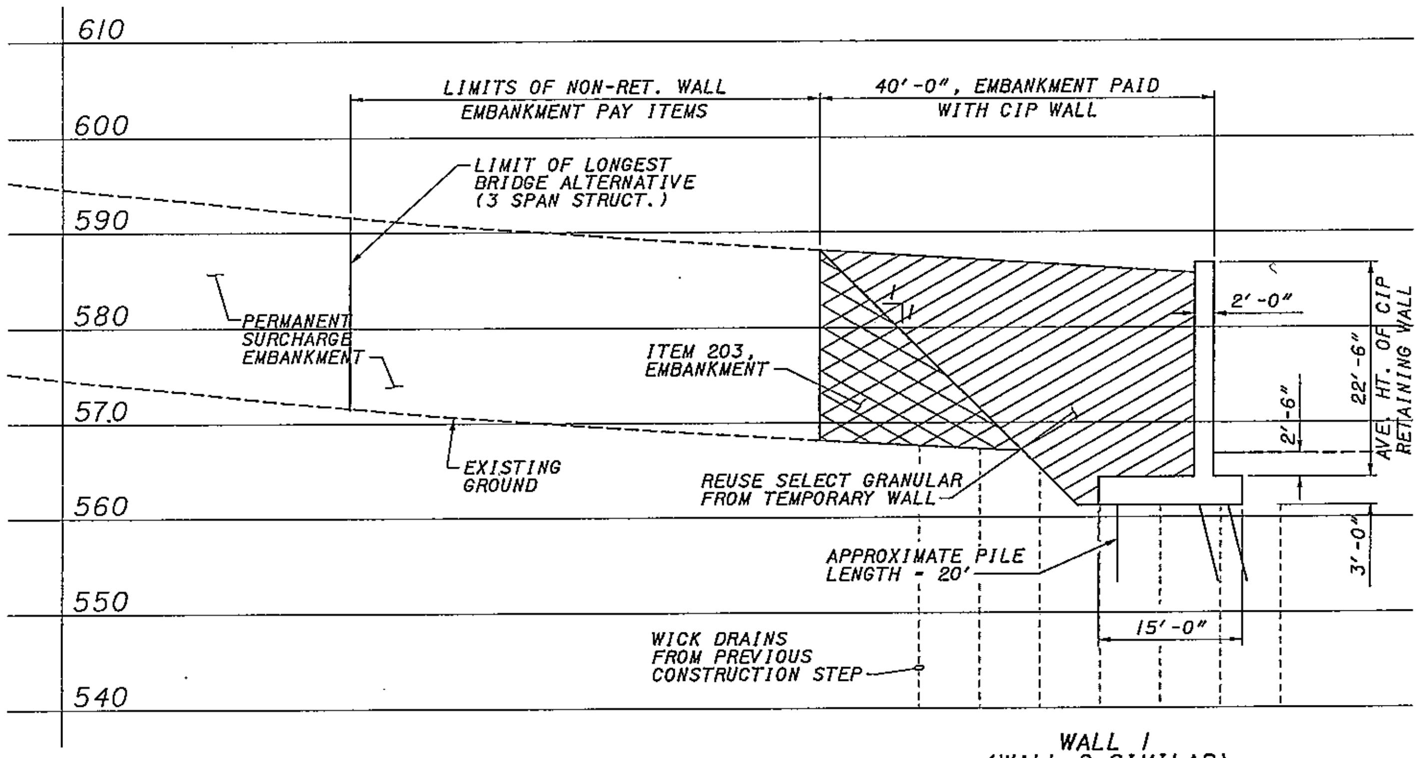
t90 - 110 DAYS W/O WICK DRAINS
- 45 DAYS W/ WICK DRAINS @ 7' SPA.



CIP RETAINING WALL ALTERNATIVE 5: STEP 3
EXCAVATE SURCHARGE & REMOVE TEMP. WALL

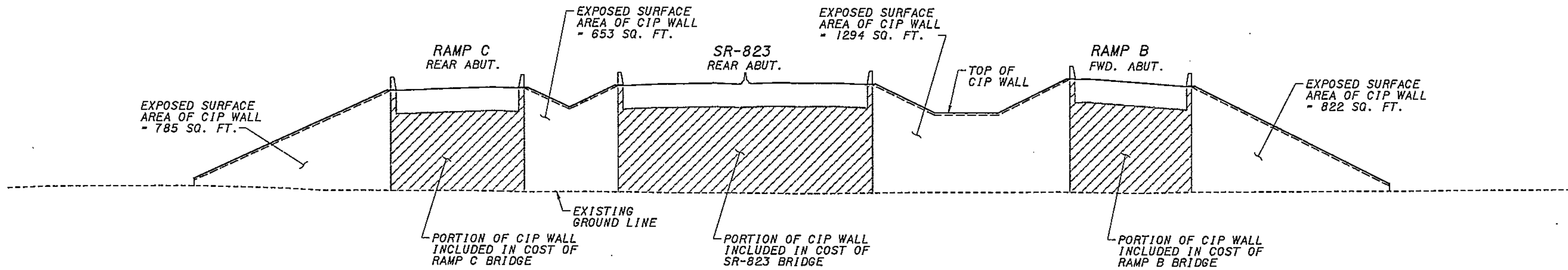


CIP RETAINING WALL ALTERNATIVE 5: STEP 4
CONSTRUCT CIP RETAINING WALL



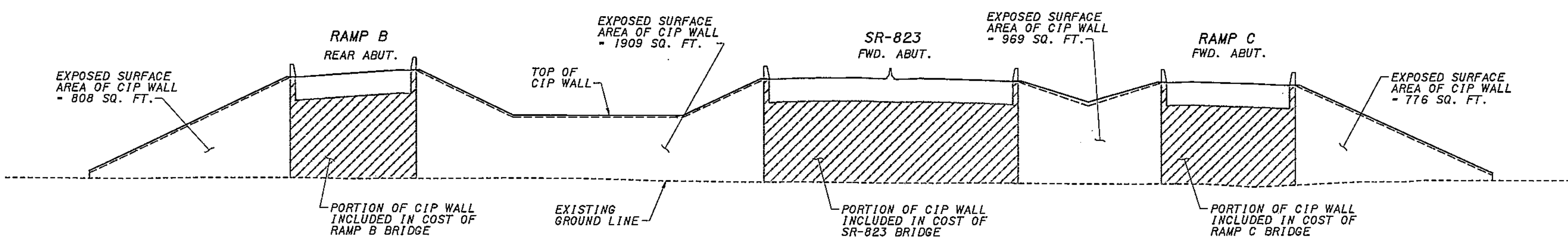
WALL 1
(WALL 2 SIMILAR)

CIP RETAINING WALL SURFACE AREAS FOR ALTERNATIVE 5



TOTAL EXPOSED SURFACE AREA
OF CIP RETAINING WALL 1
(NOT INCLUDED WITH BRIDGE COSTS)
= 3555 SQ. FT.

WALL 1



TOTAL EXPOSED SURFACE AREA
OF CIP RETAINING WALL 2
(NOT INCLUDED WITH BRIDGE COSTS)
= 4465 SQ. FT.

WALL 2

APPENDIX C

CH2MHILL

Non-Retaining Wall Embankment Cost

D. Stachler
March 29, 2007
Portsmouth By-Pass

Work Approach:
Calculate the cost of roadway embankment which is not already included in the cost of the retaining walls. Limits of Non-Retaining Wall Embankment Cost are set by limits of the approach slab for the longest bridge alternative (3-span structure) and by the limits of the embankment included with the cost of the retaining walls. For limits of embankment included with the retaining walls, see typical wall sections used for generating the unit cost of each wall alternative. See attached section cuts for embankment volume calculations.

Unit Cost:

Embankment = $12 /cy

Embankment Volume:

Alternative Number	Description	East Side of Fairground Road (Wall 1)		West Side of Fairground Road (Wall 2)	
		Volume (cy)	Cost	Volume (cy)	Cost
1	Single Span Bridges behind MSE Walls with Surcharging	7894	$95,000	11011	$133,000
2	Single Span Bridges behind MSE Walls with Deep Soil Mixing	11988	$144,000	16153	$194,000
3	Three Span Bridges behind Spill-Through Slopes	12495	$150,000	15929	$192,000
4	Single Span Bridges behind Spill-Through Slopes	17065	$205,000	23801	$286,000
5	Single Span Bridges with Pile Supported CIP Walls	13253	$160,000	17858	$215,000

EMBANKMENT QUANTITIES - ALTERNATIVE I
SECTION CUT LINES

DRAWN ON LEVEL 11

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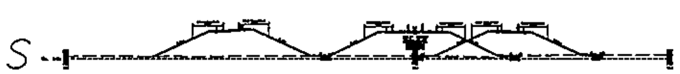
EMBANKMENT QUANTITIES FOR FAIRGROUND RD. BRIDGES

60' FROM FACE OF MSE WALL TO PAY LIMIT

MSE-WALL 2 SIDE

		END AREA	SR-823 STA.	VOLUME
3-SPAN APPR. SLAB LIMITS		8821 SF	893+54.53	
				11011 CY
END MSE WALL LIMITS		8781 SF	893+20.75	

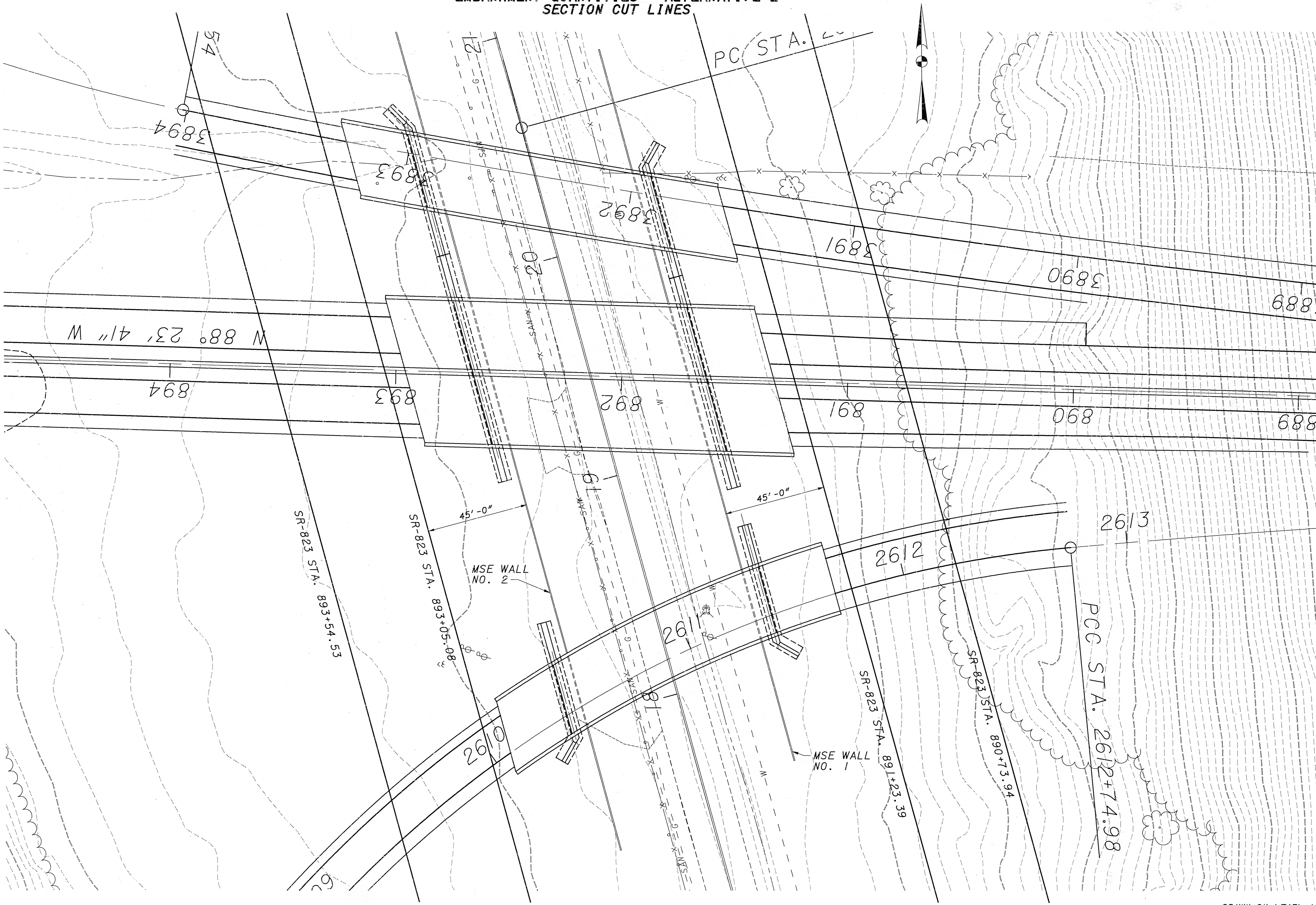
MSE WALL 1 SIDE

		END AREA	SR-823 STA.	VOLUME
BEGIN MSE WALL LIMITS		6654 SF	891+07.73	
				7894 CY
3-SPAN APPR. SLAB LIMITS		5961 SF	890+73.94	

ALTERNATIVE 1

18905 CY GRAND TOTAL

EMBANKMENT QUANTITIES - ALTERNATIVE 2
SECTION CUT LINES



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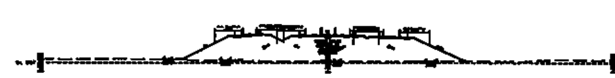
EMBANKMENT QUANTITIES FOR FAIRGROUND RD. BRIDGES

45′ FROM FACE OF MSE WALL TO PAY LIMIT

MSE WALL 2 SIDE

		END AREA	SR-823 STA.	VOLUME
3-SPAN APPR. SLAB LIMITS		8821 SF	893+54.53	
				16153 CY
END MSE WALL LIMITS		8818 SF	893+05.08	

MSE WALL 1 SIDE

		END AREA	SR-823 STA.	VOLUME
BEGIN MSE WALL LIMITS		7130 SF	891+23.39	
				11988 CY
3-SPAN APPR. SLAB LIMITS		5961 SF	890+73.94	

ALTERNATIVE 2

28141 CY GRAND TOTAL

EMBANKMENT QUANTITIES - ALTERNATIVE 3
SECTION CUT LINES

DRAWN ON LEVEL 13

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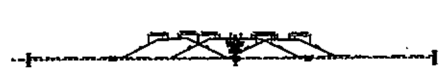
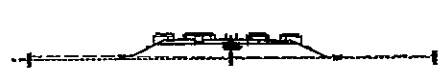
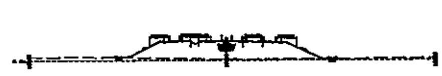
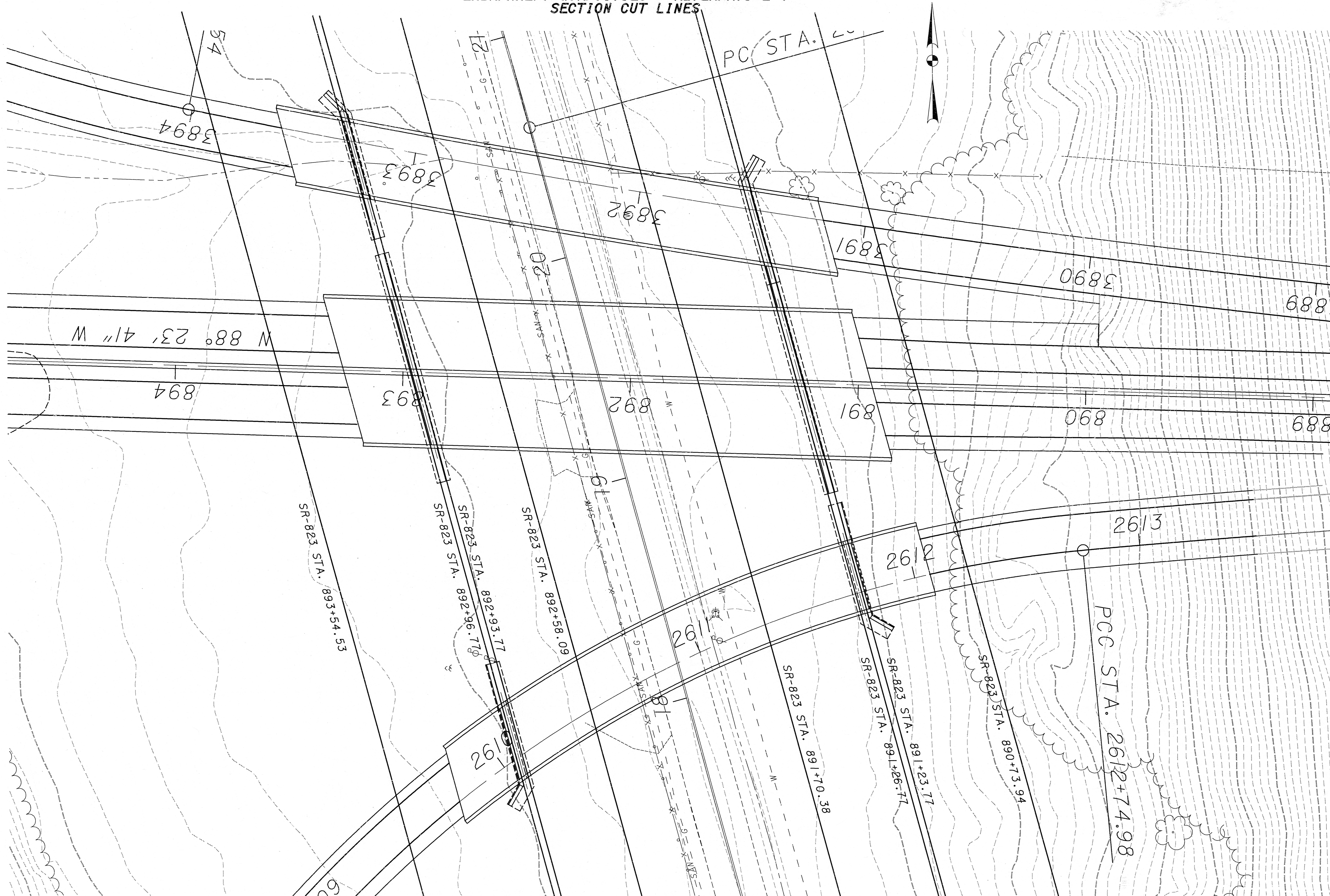
EMBANKMENT QUANTITIES FOR FAIRGROUND RD. BRIDGES

			END AREA	SR-823 STA.	VOLUME
WEST SIDE	END APPR SLAB		8821 SF	893+54.53	
					9637 CY
	BACK OF ABUT		8795 SF	893+24.99	
					897 CY
	TOP/ SPILL THROUGH SLOPE EL. 584.78		7348 SF	893+21.99	
					5395 CY
					15929 CY SUB TOTAL
	TOE/ SPILL THROUGH SLOPE		0 SF	892+82.34	
EAST SIDE	TOE/ SPILL THROUGH SLOPE		0 SF	891+55.04	
					4963 CY
	TOP/ SPILL THROUGH SLOPE EL. 591.34		5526 SF	891+06.54	
					671 CY
	BACK OF ABUT		6555 SF	891+03.54	
					6861 CY
					12495 CY SUB TOTAL
	BEGIN APPR SLAB		5961 SF	890+73.94	

ALTERNATIVE 3
3-SPAN W/ 2:1

28424 CY GRAND TOTAL

EMBANKMENT QUANTITIES - ALTERNATIVE 4
SECTION CUT LINES



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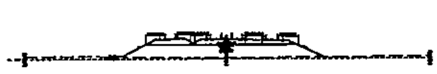
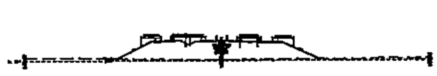
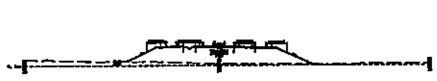
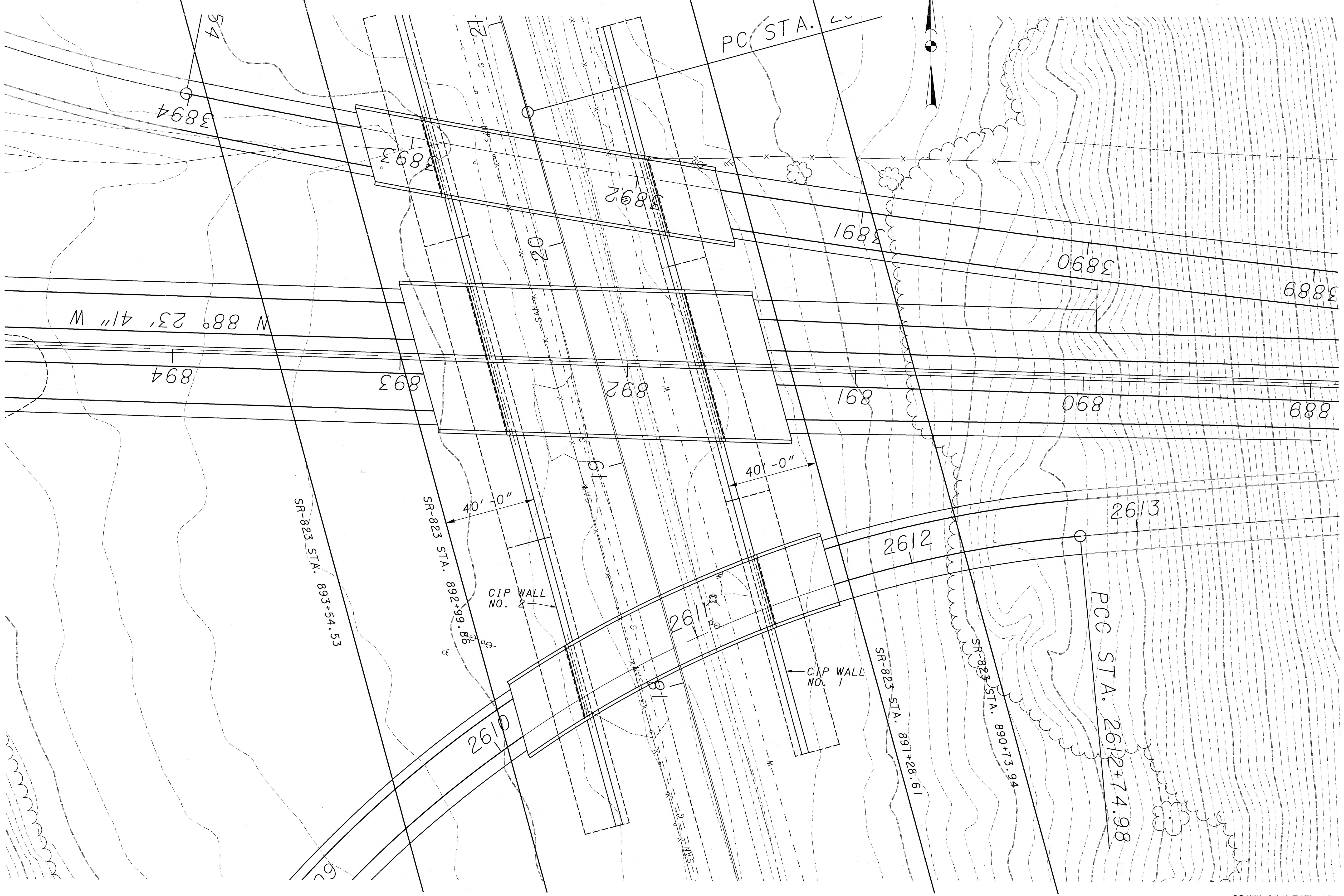
EMBANKMENT QUANTITIES FOR FAIRGROUND RD. BRIDGES

Side	Location	Section	END AREA	SR-823 STA..	VOLUME
WEST SIDE	END APPR SLAB		8821 SF	893+54.53	
					18645 CY
	BACK OF ABUT		8610 SF	892+96.77	
					841 CY
	TOP/ SPILL THROUGH SLOPE EL. 583.04		6531 SF	892+93.77	
					4315 CY
					23801 CY SUB TOTAL
	TOE/ SPILL THROUGH SLOPE		0 SF	892+58.09	
EAST SIDE	TOE/ SPILL THROUGH SLOPE		0 SF	891+70.38	
					4394 CY
	TOP/ SPILL THROUGH SLOPE EL. 588.50		5441 SF	891+26.77	
					692 CY
	BACK OF ABUT		7020 SF	891+23.77	
					11979 CY
					17065 CY SUB TOTAL
	BEGIN APPR SLAB		5961 SF	890+73.94	

ALTERNATIVE 4
1-SPAN W/ 2:1

40866 CY GRAND TOTAL

EMBANKMENT QUANTITIES - ALTERNATIVE 5
SECTION CUT LINES



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EMBANKMENT QUANTITIES FOR FAIRGROUND RD. BRIDGES

40′ FROM FACE OF CIP WALL TO PAY LIMIT

CIP WALL 2 SIDE

		END AREA	SR-823 STA.	VOLUME
3-SPAN APPR. SLAB LIMITS		8821 SF	893+54.53	
				17858 CY
END CIP WALL LIMITS		8818 SF	892+99.86	

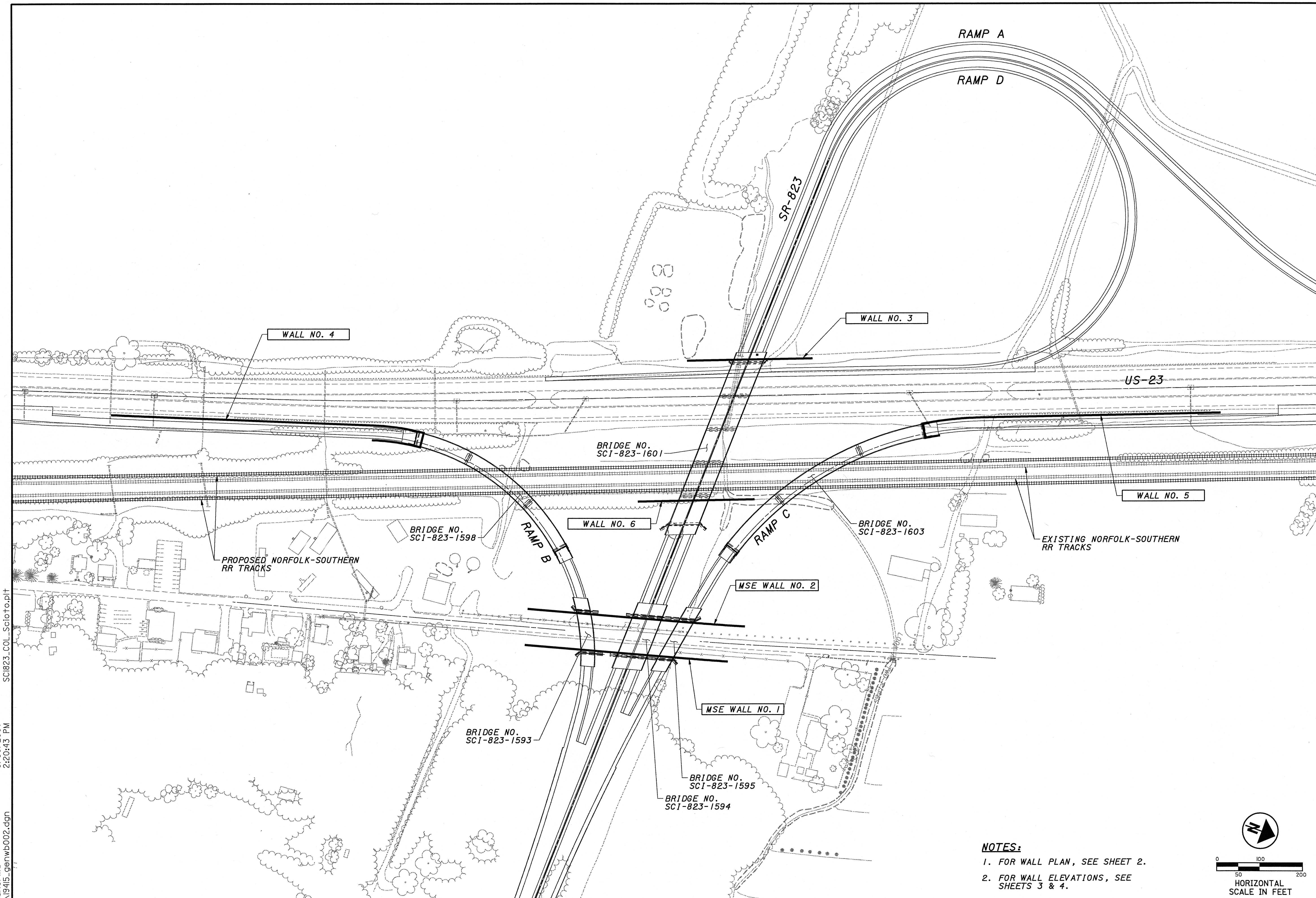
CIP WALL 1 SIDE

		END AREA	SR-823 STA.	VOLUME
BEGIN CIP WALL LIMITS		7130 SF	891+28.61	
				13253 CY
3-SPAN APPR. SLAB LIMITS		5961 SF	890+73.94	

ALTERNATIVE 5

31111 CY GRAND TOTAL

APPENDIX D



NOTES:

1. FOR WALL PLAN, SEE SHEET 2.
2. FOR WALL ELEVATIONS, SEE SHEETS 3 & 4.

RETAINING WALL KEY PLAN

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	03/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

SCI-823-0.00
PID 19415

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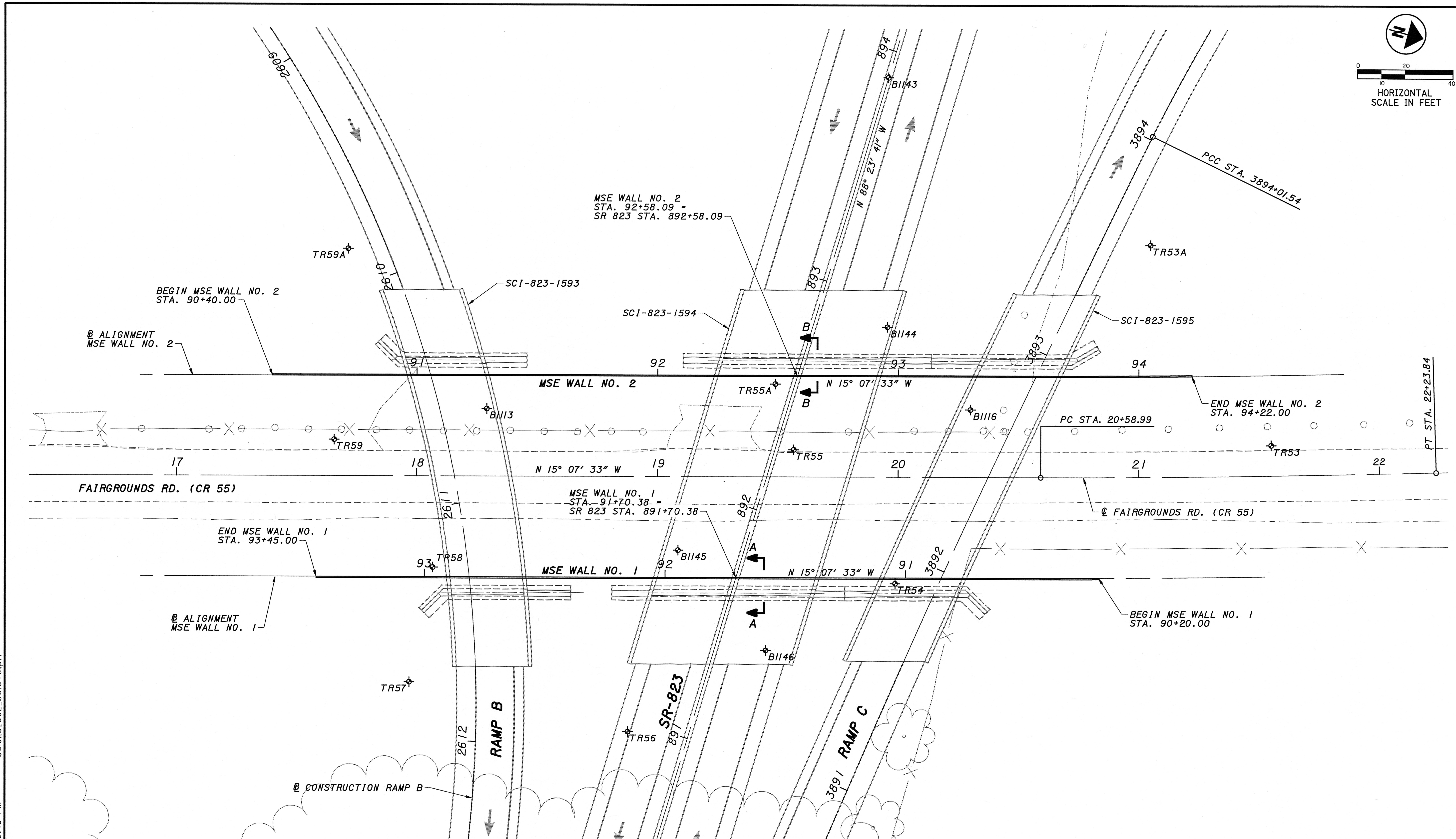

LEGEND

✧ INDICATES BORING LOCATION

NOTES:

1. WALL ALIGNMENTS ARE DEFINED ALONG THE EXPOSED FACE OF THE MSE WALL PANELS.
2. FOR ROADWAY ALIGNMENT HORIZONTAL CURVE DATA, SEE ROADWAY PLANS.
3. FOR SECTIONS A-A AND B-B, SEE SHEET 5.

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	03/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

MSE WALLS 1 & 2 PLAN VIEW

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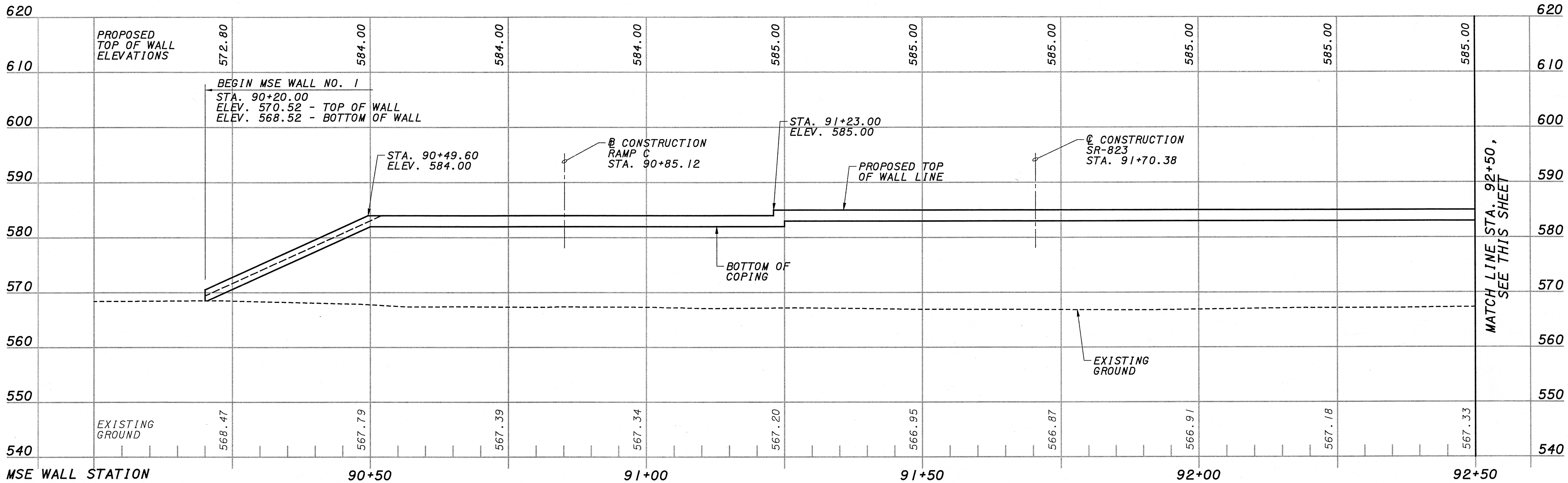

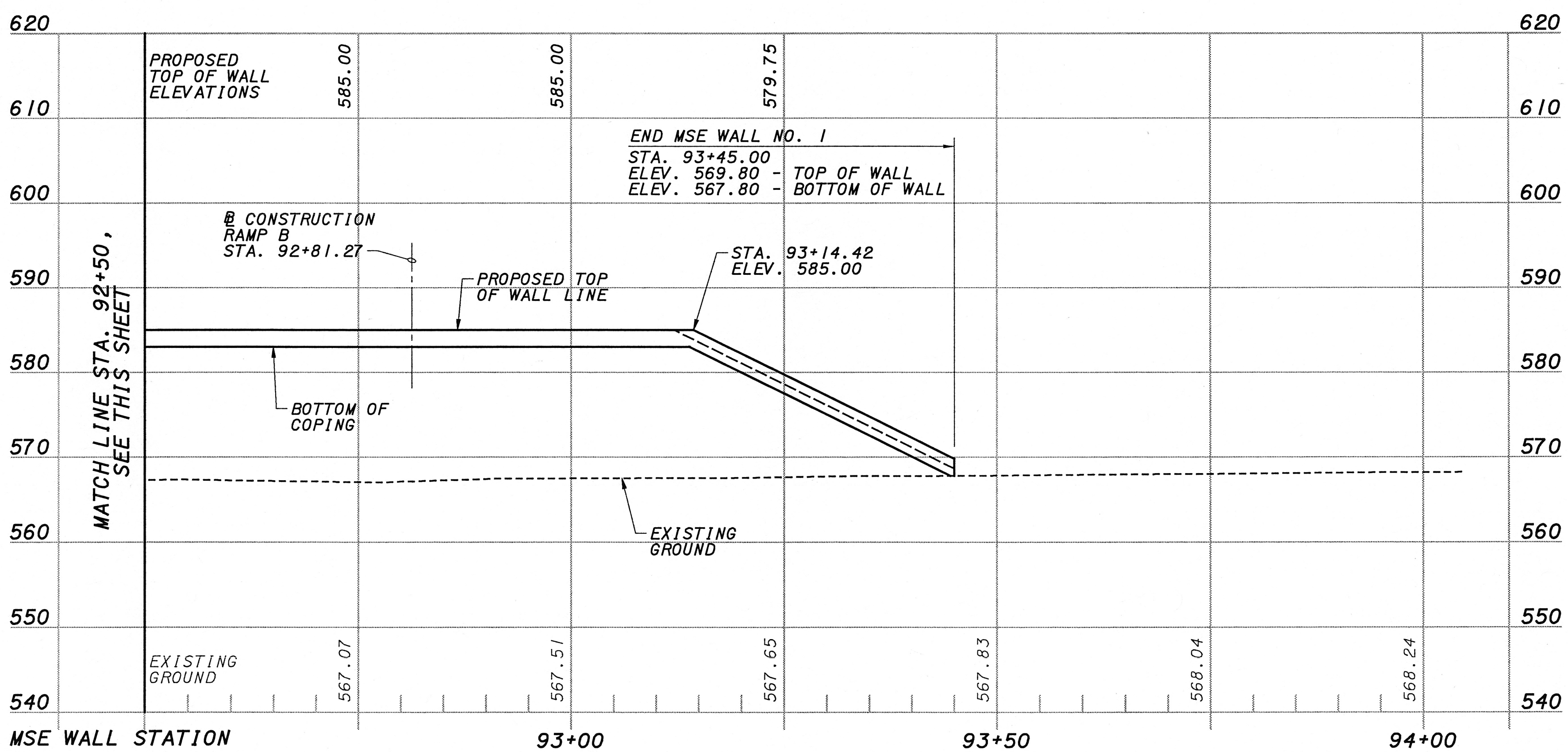

NOTES:

1. MSE WALL ELEVATIONS ARE SHOWN VIEWED FROM THE EXPOSED FACE OF THE WALL.
2. FOR DEFINITION OF MSE WALL TOP AND BOTTOM ELEVATIONS, SEE WALL SECTION ON SHEET 5.

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	03/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

WALL ELEVATION - MSE WALL NO. 1

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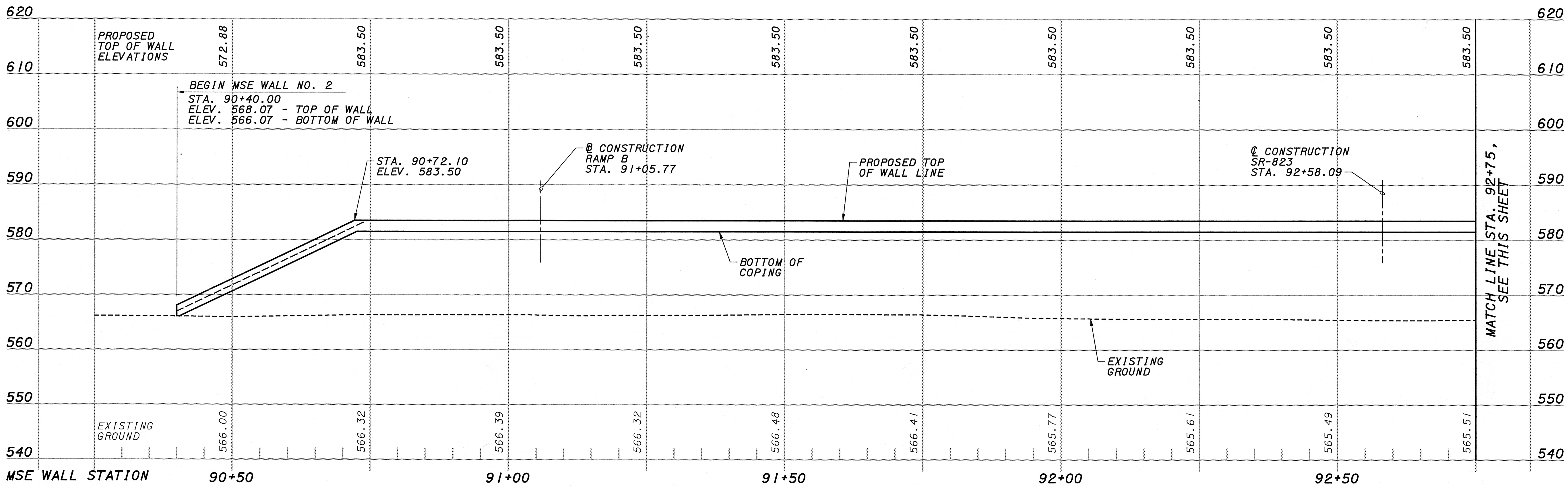

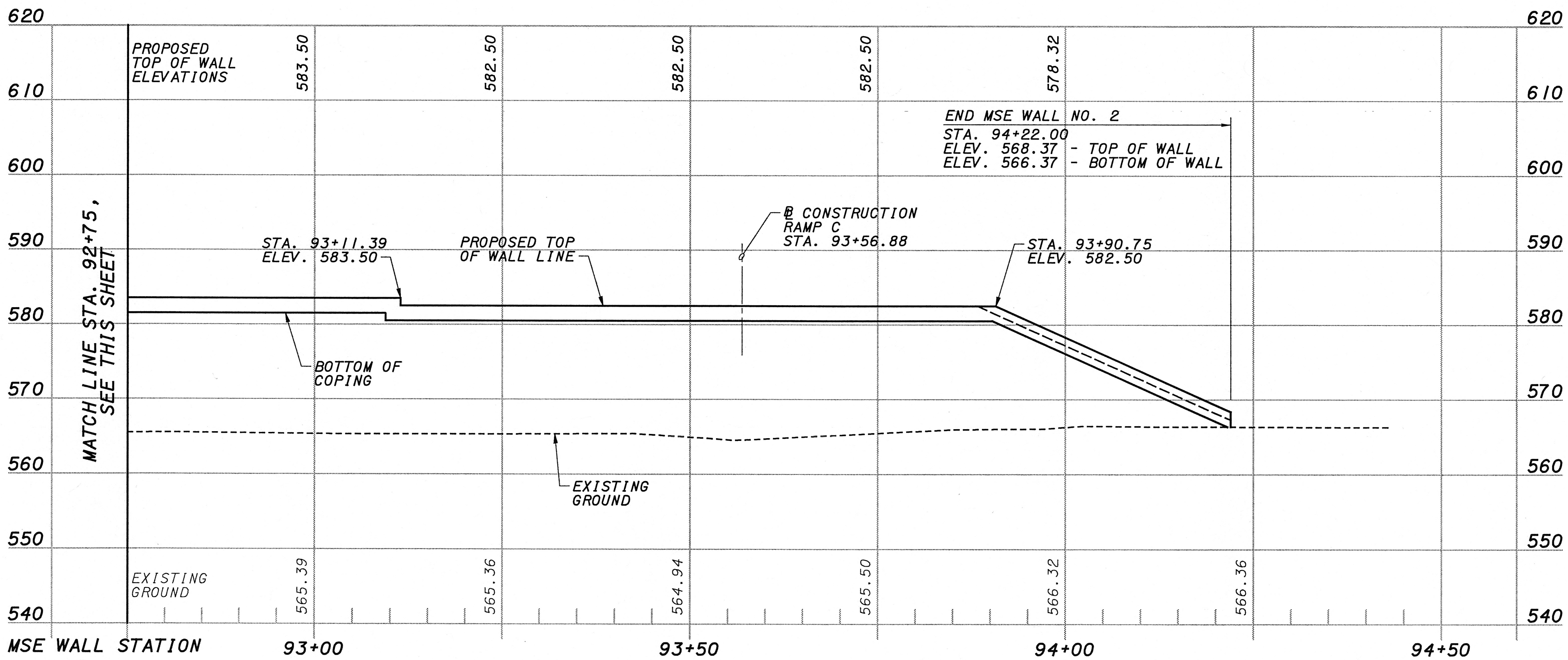

NOTES:

1. MSE WALL ELEVATIONS ARE SHOWN VIEWED FROM THE EXPOSED FACE OF THE WALL.
2. FOR DEFINITION OF MSE WALL TOP AND BOTTOM ELEVATIONS, SEE WALL SECTION ON SHEET 5.

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

DESIGNED DGS | DRAWN JBA | REVIEWED SCJ | DATE 03/07
CHECKED SKT | REVISED | STRUCTURE FILE NUMBER

WALL ELEVATION - MSE WALL NO. 2

SCI-823-0.00
PID 19415

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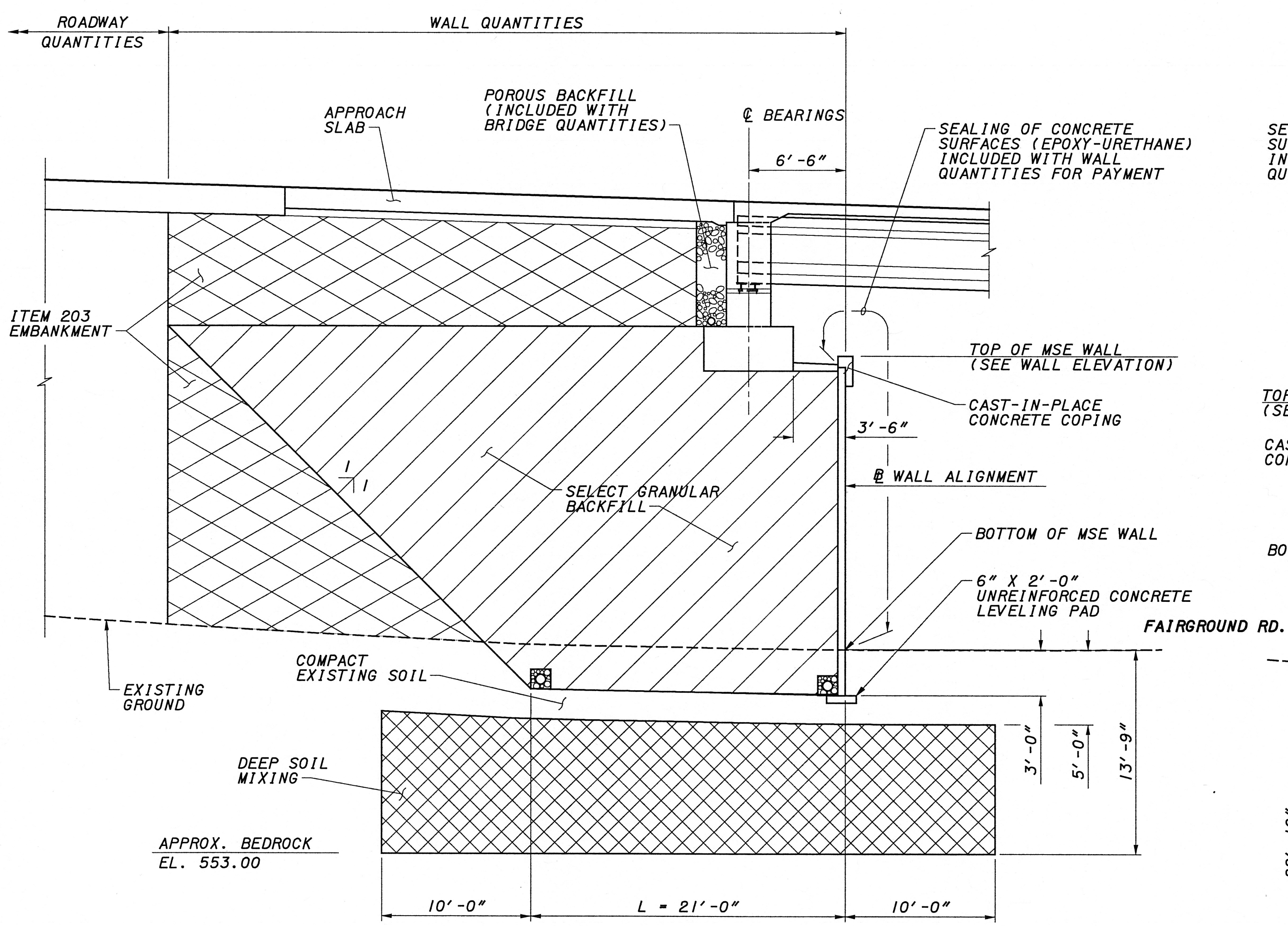
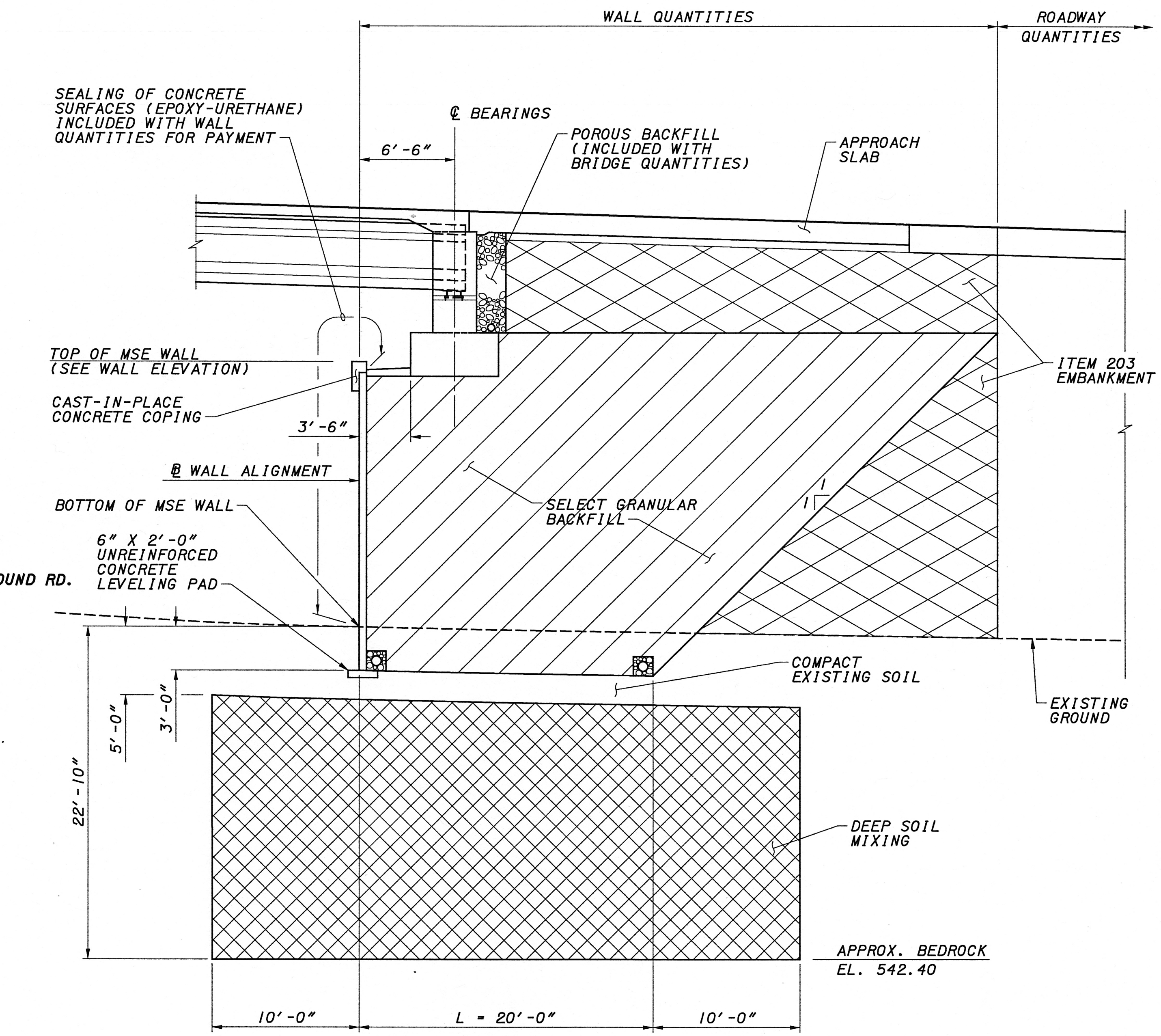


MSE WALL NO. 1
SECTION A-A
SEE SHEET 2 OF 5



MSE WALL NO. 2
SECTION B-B
SEE SHEET 2 OF 5

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

DESIGNED	DRAWN	REVIEWED	DATE
DGS	JBA	SCJ	03/07
CHECKED	REVISED	STRUCTURE FILE NUMBER	
SKT			

MSE WALL SECTIONS
MSE WALL NOS. 1 AND 2

SCI-823-0.00
PID 19415

APPENDIX E

ALTERNATIVE VS. COST MATRIX

		Alternative 1	Alternative 2	Alternative 3	Alternative 4	Alternative 5
		Single Span Bridges behind MSE Walls with Surcharging	Single Span Bridges behind MSE Walls with Deep Soil Mixing	Three Span Bridges behind Spill-Through Slopes	Single Span Bridges behind Spill-Through Slopes	Single Span Bridges with Pile Supported CIP Walls
Bridges	Ramp B over Fairgrounds Road	$756,000	$697,000	$1,829,000	$1,902,000	$941,000
	823 over Fairgrounds Road	$1,437,000	$1,379,000	$2,632,000	$3,486,000	$1,872,000
	Ramp C over Fairgrounds Road	$795,000	$744,000	$1,417,000	$1,865,000	$983,000
	Total Cost of Three Bridges	$2,988,000	$2,820,000	$5,878,000	$7,253,000	$3,796,000
Retaining Walls	MSE Wall 1 (East Side of Fairgrounds)	$780,000	$676,000	$0	$0	$0
	MSE Wall 2 (West Side of Fairgrounds)	$923,000	$1,107,000	$0	$0	$0
	CIP Wall 1 (East Side of Fairgrounds)	$0	$0	$0	$0	$587,000
	CIP Wall 2 (West Side of Fairgrounds)	$0	$0	$0	$0	$737,000
	Total Cost of Retaining Walls	$1,703,000	$1,783,000	$0	$0	$1,324,000
Non-Ret. Wall Embank.	Embankment at East Side of Fairgrounds	$95,000	$144,000	$150,000	$205,000	$160,000
	Embankment at West Side of Fairgrounds	$133,000	$194,000	$192,000	$286,000	$215,000
	Total Cost of Non-Ret. Wall Embankment	$228,000	$338,000	$342,000	$491,000	$375,000
TOTAL COST OF ALTERNATIVE		$4,919,000	$4,941,000	$6,220,000	$7,744,000	$5,495,000