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January 31, 2007

Mr. Jeff Ackerman
Project Manager
Burgess & Niple
100 West Erie Street
Painesville, OH 44077

Re: **Innerbelt Corridor Project, PID 77510 & 25795**
DLZ Job No.: 0422-1007.00

Dear Mr. Ackerman:

This letter reports supplemental information and recommendations regarding the proposed Mather Mansion retaining wall, located on the westbound side of I-90, between Chester Avenue and Euclid Avenue (Station 220+30 to 228+70). A mechanically stabilized earth (MSE) wall was previously planned for the Mather Mansion site. However, we understand that the wall type at this location has been changed to a cast-in-place (CIP) retaining wall. Information regarding pile capacity and global stability for the retaining wall is presented in the following paragraphs. Information regarding drilling procedures, logs of the preliminary borings, boring location plans and laboratory test results are presented in our previous correspondence dated November 27, 2006.

Mather Mansion CIP Retaining Wall

Preliminary plans indicate the proposed CIP wall between Chester and Euclid Avenues will range between approximately 22 and 31 feet above finished grade. The proposed wall generally parallels an existing bin wall. It is our understanding that the bin wall will remain and the new wall will retain fill placed between the walls. The proposed finished grade generally varies from two to seven feet below existing grade along this portion of I-90. Additionally, preliminary information indicates the wall is to be supported on 4 rows (3 battered, 1 vertical) of piles. Borings W-DLZ-7 through 9 were located along the proposed wall alignment and encountered generally loose to medium dense, silty, granular soils underlain by cohesive layers with consistency generally ranging from soft to stiff.

Preliminary analyses consisting of pile capacity and global stability have been performed for the proposed CIP wall. Soil parameters were selected for use in the analyses based on a combination of standard penetration test N-values, hand penetrometer readings, index testing, shear strength

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testing, typical values and past experience with similar soils. However, it should be noted that the material properties at different depths varied considerably between borings and, in some cases, within the individual borings over relatively small vertical intervals. Consequently, all of these analyses will need to be reevaluated once the final phase of exploration and testing is completed and the properties and vertical and lateral extent of the various layers are better defined.

The preliminary analyses indicate that issues may exist with regard to global stability. The analyses are discussed in more detail in the following paragraphs.

Global Stability

A section was developed for global stability analysis based on the results of the borings and the proposed wall section provided in the preliminary plans. Beneath the wall, the borings generally show granular soil over layers of medium stiff, stiff and soft cohesive soil. This section was analyzed for the end-of-construction (undrained strength) and long-term (drained strength) conditions.

The stability analysis for the end-of-construction condition indicates safety factors below the required minimum of 1.5 and even less than 1.0 for critical surfaces passing through the medium stiff and soft cohesive layers. The analysis is shown on attached Figure 1 of 2. The global stability analysis of the wall is very sensitive to the cohesion of the clay soils and geometry of these layers and will therefore need to be reevaluated once the final phase of exploration is performed and the subsurface conditions better defined.

There will be up to 7 feet of cut along the I-90 alignment in this area. Also, this proposed wall alignment is located in the "Innerbelt Trench" where the area was previously excavated to establish the existing I-90 alignment. Construction of the wall and placement of fill behind it will essentially refill a relatively small portion of the trench. Consequently, there is some question as to whether an undrained condition will actually develop due to construction of the proposed wall. We are continuing to analyze and research this unusual issue to establish the applicability of the global stability analysis for the undrained case.

Stability was also checked for long-term conditions (drained strengths). This analysis resulted in a critical failure surface passing just beneath the CIP wall. The factor of safety for this critical surface is 1.768 exceeding the required 1.5. Deeper failure surfaces had higher factors of safety. This analysis is shown on attached Figure 2 of 2.

Pile Foundations

It is our understanding that the proposed wall will be supported on multiple rows of piles. It further understood that the desired ultimate bearing capacity of the piles is 94 tons. Piles should be installed in accordance with ODOT Item 507, "Bearing Piles." Depending on the boring used for the pile capacity analysis, 16-inch diameter cast-in-place reinforced concrete piles of lengths ranging from 77 to 94 feet would be required to meet the desired ultimate bearing capacity of 94 tons. The pile capacity calculations are presented in Appendix I.

It is recommended that test piles be driven to indicate required pile lengths. The actual length of pile required to support the design working load should be established in the field using the dynamic pile driving capacity formula.

Lateral Resistance of Piles

The borings generally encountered loose to medium dense granular materials from the surface to approximately elevation 614 underlain by layers of soft to stiff cohesive soil. For lateral capacity of piles, the coefficient of lateral subgrade reaction, k_h, for granular soils and softer cohesive soils is usually assumed to increase linearly with depth as follows:

$$k_h = n_h (z/d) \qquad \text{(NAVFAC DM-7.2)}$$

where: n_h = coefficient of variation of lateral subgrade reaction
z = depth
d = pile diameter/width

For the loose/medium dense granular soil, it is recommended that a coefficient of variation of lateral subgrade reaction, n_h, of 10 tons per cubic foot be used. For the soft to medium stiff clays, it is recommended that a coefficient of variation of lateral subgrade reaction, n_h, of 3 tons per cubic foot be used.

Group effects should be considered when the pile spacing in the direction of loading is less than eight pile diameters.

Lateral Earth Pressure

The wall must be designed to resist lateral loads imposed by the soil, groundwater, and the surcharge effect of adjacent structures or equipment. The increase in lateral pressure for uniform surcharges is given as follows:

$$\Delta\sigma_h = K_a q_s$$

Where: $\Delta\sigma_h$ = increase in lateral earth pressure
K_a = active earth pressure coefficient
q_s = uniform surcharge loading

If traffic is expected to come within a distance of one-half the wall height from the wall, a uniform live load surcharge pressure of at least 240 psf should be used in the design.

The lateral earth pressure should be determined based on the properties of the soil within the zone defined by the wall and a line that rises from the base of the wall at an angle of $45°+\phi/2$ from the horizontal. The lateral earth pressure coefficients recommended for the materials encountered in the borings as well as values for select granular fill are presented in the following table. These values are based on a horizontal surface behind the wall.

Lateral Earth Pressure Coefficients

Type of Material	At Rest	Active	Passive
Select granular embankment material ($\phi = 34°$, $\gamma = 120$ pcf)	0.44	0.28	3.54
In-situ loose to medium dense granular soils ($\phi = 32°$, $\gamma = 125$ pcf)	0.47	0.31	3.25
In-situ soft to medium stiff silty clays ($\phi = 28°$, $\gamma = 125$ pcf)	0.53	0.36	2.77

At a minimum, a free-draining granular backfill should be placed against the wall for a distance of 2 to 3 feet from the wall to prevent the buildup of hydrostatic pressures. Unless there will be pavement immediately behing the wall, the top one-foot of backfill should consist of cohesive soil and be graded away from the wall to reduce the amount of surface water that infiltrates into the backfill.

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Closing Remarks

As stated previously, these analyses are preliminary for the proposed CIP wall and are very sensitive to the properties of the cohesive layers encountered beneath the upper sandy, silty granular soils. The strength parameters used in the analyses were conservatively selected based on a relatively few shear strength tests results and hand penetrometer readings on the splitspoon samples. These values varied considerably from boring to boring and, in some cases, between consecutive samples in the borings. It is therefore essential that these analyses be reevaluated once the final phase of exploration and testing is completed and the properties and geometry of these critical layers are better defined.

We appreciate the opportunity to be of service on this project. Please do not hesitate to call if you have any questions concerning the information presented herein or if you would like to discuss the preliminary findings and analyses in greater detail.

Sincerely,

DLZ OHIO, INC.

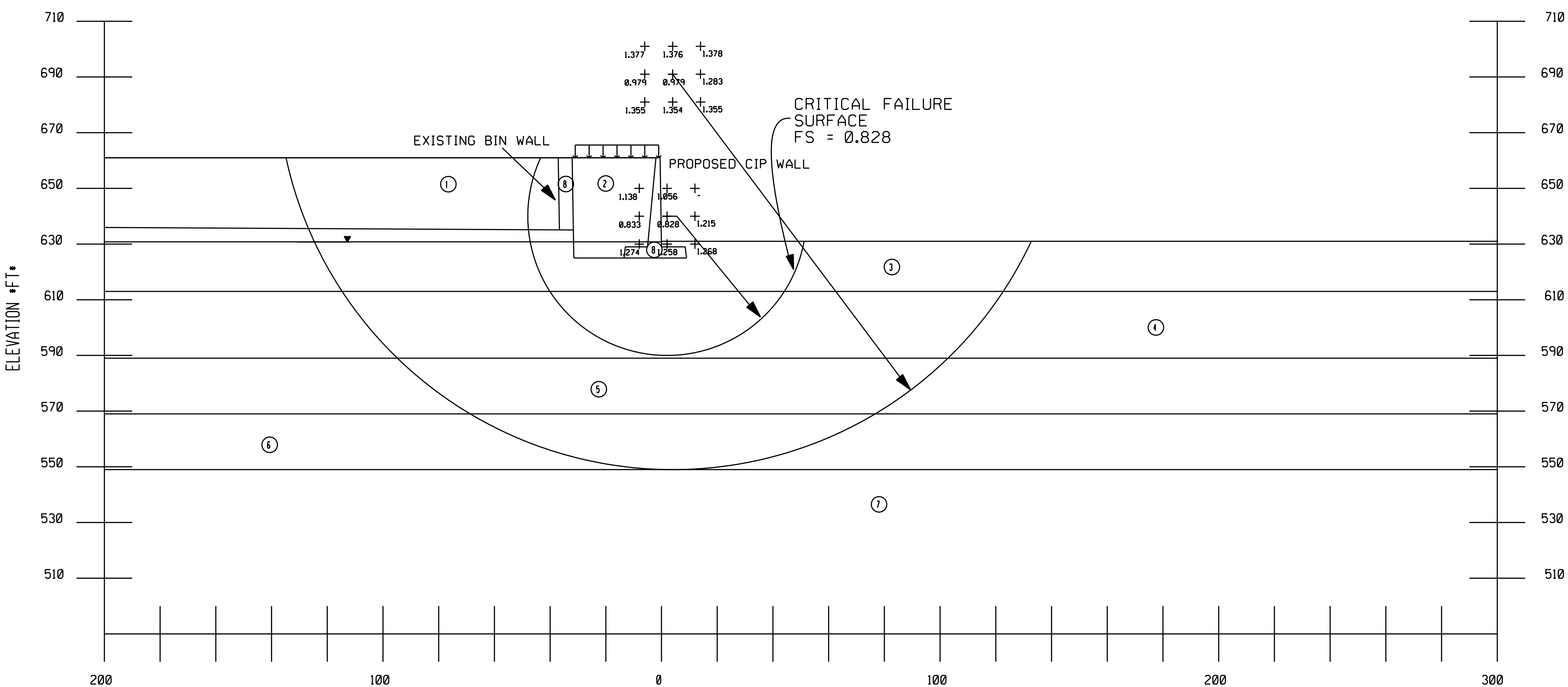
Richard Hessler
Geotechnical Engineer

Bryan Wilson, P.E.
Senior Geotechnical Engineer

Attachments: Figure 1 of 2 Slope Stability Analysis: Undrained
Figure 2 of 2 Slope Stability Analysis: Drained
Appendix I - Pile Calculation Sheets

cc: File

MATERIAL PROPERTIES			
MATERIAL	UNIT WT (pcf)	UNDRAINED STRENGTH	
		C(psf)	θ (deg)
(1) GRANULAR SOIL (ASSUMED)	125	·	30
(2) RETAINING WALL BACKFILL	125	·	30
(3) GRANULAR SOIL	130	·	32
(4) MEDIUM STIFF CLAY	125	750	·
(5) STIFF CLAY	125	1750	·
(6) SOFT CLAY	125	500	·
(7) STIFF CLAY	125	1250	·
(8) BIN WALL / CIP WALL	145	14500	60



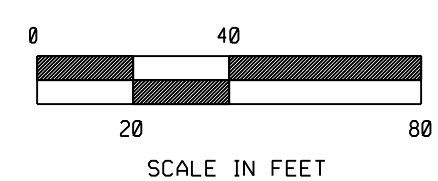


FIGURE 1: SLOPE STABILITY ANALYSIS - UNDRAINED
MATHER MANSION RETAINING WALL

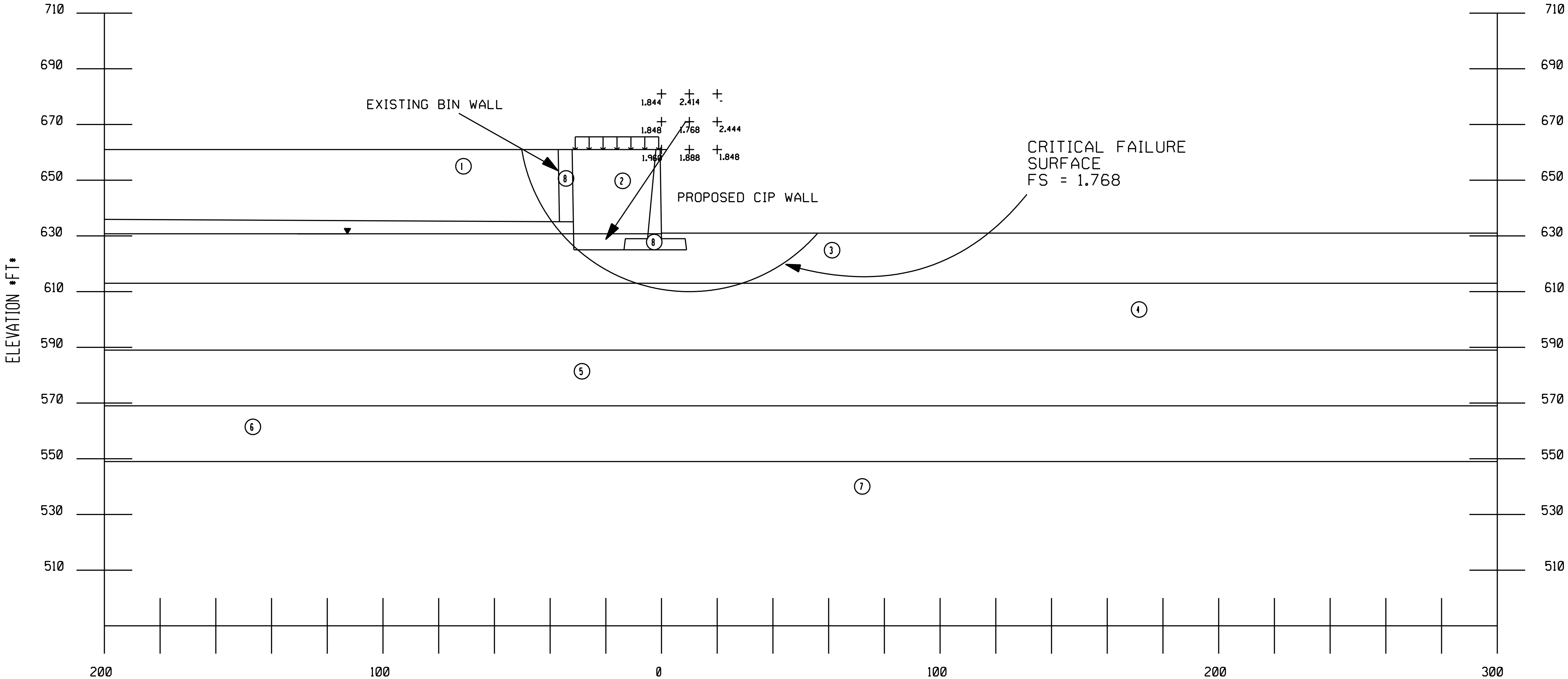
CUY-INNERBELT TRENCH

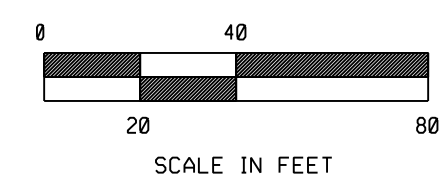
CALCULATED RJH
CHECKED BEW

1 / 2

MATERIAL	UNIT WT (pcf)	DRAINED STRENGTH C(psf)	DRAINED STRENGTH θ (deg)
(1) GRANULAR SOIL (ASSUMED)	125	-	32
(2) RETAINING WALL BACKFILL	130	-	30
(3) GRANULAR SOIL	130	-	32
(4) MEDIUM STIFF CLAY	125	-	28
(5) STIFF CLAY	125	-	28
(6) SOFT CLAY	125	-	28
(7) STIFF CLAY	125	-	28
(8) BIN WALL / CIP WALL	145	14500	60

MATERIAL PROPERTIES





CALCULATED RJH
CHECKED BEW

FIGURE 2: SLOPE STABILITY ANALYSIS - DRAINED
MATHER MANSION RETAINING WALL

CUY-INNERBELT TRENCH

2 / 2

APPENDIX I

Pile Calculation Sheets

Pile Capacity Calculations

FHWA Method (methods by Meyerhof, Nordlund and Thurman)

Location: Mathers Mansion
Retaining Wall (W-DLZ-8)

Pile type & size: 94 ton 16" dia. CIP

Pile Diameter 1.333 ft 16.0 in
Perimeter, Cd: 4.19 ft
Tip Area, Ap: 1.396 ft^2

Pile Length: 72.0 ft 236 ft

$Q_{ult} = Q_s + Q_p$

Sand: Q_s = .02 N' D Cd ;(N'<=50) $Q_p = Ap\ \alpha\ Pd\ N'_q$

Clay: Q_s = Ca Cd D Q_p = 9 Cu Ap

NOTE: Maximum effective stress reached at 20 x pile width and exclude top 5 ft of soil from calculation.

Soil	Unit Wt pcf	Bottom of layer ft	N (for sand)	N' (for sand)	D ft	Ca (for clay) psf	Q_s tons	α (for sand)	Eff. stress, Pd psf	N'_q (for sand)	Cu (for clay) psf	Q_p tons	Q_{ult} tons
Ground Surface		637.0		---	---	---	---	---	0	---	---	---	---
Top of Pile	120	625.0		---	---	---	---	---	1,440	---	---	---	---
Sand	68	614.0	20	19	11.0		17.5	0.55	2,188	8		6.7	
Clay	120	605.2			8.8	490	9.0		3,244		500	3.1	
Clay	120	590.2			15.0	730	22.9		4,068		1,000	6.3	
Clay	120	565.2			25.0	490	25.7		4,068		500	3.1	
Clay	120	555.3			9.9	730	15.1		4,068		750	4.7	
Clay	120	540.3			2.3	490	2.4		4,068		250	1.6	94.2
Clay	120	537.0				730					1,000		

Ultimate Load 94 tons

Allowable Load **47 tons** Pile Tip 553.0 ft

Factor of Safety = 2.0

Pile Capacity Calculations

FHWA Method (methods by Meyerhof, Nordlund and Thurman)

Location: Mathers Mansion
Retaining Wall (W-DLZ-8)

Pile type & size: 94 ton 16" dia. CIP

Pile Diameter 1.333 ft 16.0 in
Perimeter, Cd: 4.19 ft
Tip Area, Ap: 1.396 ft^2

Pile Length: 77.0 ft 253 ft

$Q_{ult} = Q_s + Q_p$

Sand: Q_s = .02 N' D Cd ;(N'<=50) $Q_p = Ap\ \alpha\ Pd\ N'_q$

Clay: Q_s = Ca Cd D Q_p = 9 Cu Ap

NOTE: Maximum effective stress reached at 20 x pile width and exclude top 5 ft of soil from calculation.

Soil	Unit Wt pcf	Bottom of layer ft	N (for sand)	N' (for sand)	D ft	Ca (for clay) psf	Q_s tons	α (for sand)	Eff. stress, Pd psf	N'_q (for sand)	Cu (for clay) psf	Q_p tons	Q_{ult} tons
Ground Surface		635.0		---	---	---	---	---	0	---	---	---	---
Top of Pile	120	625.0		---	---	---	---	---	1,200	---	---	---	---
Sand	68	612.0	15	15	13.0		16.3	0.55	2,084	8		6.4	
Clay	58	598.2			13.8	730	21.1		2,877		1,000	6.3	
Clay	58	588.2			10.0	490	10.3		2,877		500	3.1	
Clay	58	568.3			19.9	795	33.1		2,877		1,500	9.4	
Clay	58	548.3			20.0	263	11.0		2,877		250	1.6	
Clay	58	535.0			0.3	750	0.5		2,877		1,250	7.9	100.2

Ultimate Load 100 tons

Allowable Load **50 tons** Pile Tip 548.0 ft

Factor of Safety = 2.0

Pile Capacity Calculations

FHWA Method (methods by Meyerhof, Nordlund and Thurman)

Location: Mathers Mansion
Retaining Wall (W-DLZ-8)

Pile type & size: 94 ton 16" dia. CIP

Pile Diameter 1.333 ft 16.0 in
Perimeter, Cd: 4.19 ft
Tip Area, Ap: 1.396 ft^2

Pile Length: 90.0 ft 295 ft

$Q_{ult} = Q_s + Q_p$

Sand: $Q_p = Ap\ \alpha\ Pd\ N'_q$

Clay: $Q_s = Ca\ Cd\ D$ $Q_p = 9\ Cu\ Ap$

NOTE: Maximum effective stress reached at 20 x pile width and exclude top 5 ft of soil from calculation.

Soil	Unit Wt pcf	Bottom of layer ft	N (for sand)	N' (for sand)	D ft	Ca (for clay) psf	Q_s tons	α (for sand)	Eff. stress, Pd psf	N'_q (for sand)	Cu (for clay) psf	Q_p tons	Q_{ult} tons
Ground Surface		635.0		---	---	---	---	---	0	---	---	---	---
Top of Pile	120	625.0		---	---	---	---	---	1,200	---	---	---	---
Sand	68	614.5	15	16	10.5		14.1	0.55	1,914	8		5.9	
Clay	120	598.2			16.3	490	16.7		3,854		500	3.1	
Clay	120	588.2			10.0	795	16.7		3,854		1,500	9.4	
Clay	120	573.2			15.0	490	15.4		3,854		500	3.1	
Clay	120	568.3			4.9	795	8.2		3,854		1,500	9.4	
Clay	120	535.0			33.3	260	18.1		3,854		250	1.6	90.7

Ultimate Load 91 tons

Allowable Load **45 tons** Pile Tip 535.0 ft

Factor of Safety = 2.0