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Woodward-Clyde Consultants

August 9, 1990
89C2857

Mr. James Frye
Olin Corporation
Lower River Road
P.O. Box 248
Charleston, TN 37310

Re: Revised Procedure
Reconstruction of Slurry Wall
Cap and North Embankment Slope
Pine and Tuscarora Remediation
Site, Niagara Falls, New York

Dear Jim:

Enclosed are the recommended revised construction procedures and the results of slope stability analyses for the reconstruction of the slurry wall cap and north embankment at the Pine and Tuscarora Remediation Site. The revised reconstruction procedure involves moving the toe of the north embankment slope to the south side of the slurry wall and making the slope inclination 2.5 horizontal to 1 vertical. Slope stability calculations are also enclosed for the 2.5:1 slope.

We have also completed the FEMA flood routing for the revised design and the report has been sent to FEMA. The new routing study indicates that the revised design should meet FEMA's requirements. A telephone call from FEMA's consultant has indicated preliminary acceptance.

Should you or any other party involved in this project have any questions, please do not hesitate to contact us.

Very truly yours,

WOODWARD-CLYDE CONSULTANTS

Richard T. Roe

Richard T. Roe, P.E.
Project Geotechnical Engineer

Frank S. Waller

Frank S. Waller, P.E.
Senior Managing Principal

RTR/FSW/mek
Enclosures
cc: G. Blaine Butaud - Olin
George Harbison - Olin
Alan Elia - Sevenson

RECEIVED
AUG 10 1990
Bureau of
Construction Services

RECOMMENDED CONSTRUCTION PROCEDURE

Reconstruction of Slurry Wall Cap
and North Embankment Slope
Pine and Tuscarora Remediation Site
Niagara Falls, New York

RECOMMENDED CONSTRUCTION PROCEDURE
RECONSTRUCTION OF SLURRY WALL CAP AND NORTH EMBANKMENT SLOPE
PINE AND TUSCARORA REMEDIATION SITE
NIAGARA FALLS, NEW YORK

The following construction procedure is recommended for the reconstruction of the slurry wall cap and the north embankment slope of the Pine and Tuscarora Remediation Site. This area experienced a slope stability problem two days after the 2-foot clay cap was constructed over the slurry wall. Measurements have shown that the slurry wall has widened from the 3-foot constructed (and design) width to approximately 5 to 5.5 feet. Probing has shown that the slurry wall is intact, with the soil-bentonite backfill slumping as the embankment slope between the slurry wall and the adjacent creek channel moved laterally (due to the imposed pressure of the slurry wall and construction equipment operating adjacent to and over the slurry wall). The redesign of this area calls for a flattened 2.5:1 slope (versus an original 2:1 design slope) plus a reduced height of slurry wall (approximately 7 feet versus 10 feet). Also, the toe of the slope intersects a bench at elevation 567 at a distance of 4 feet south of the slurry wall, minimizing future vertical loads on the top of the slurry wall. An 8-inch-thick reinforced concrete slab is to be poured over the existing slurry wall to bridge over the 5- to 5.5-foot-wide slurry wall without imposing any additional vertical load on the slurry wall. A detailed cross section of the redesign is shown on the attached Figure 1. Details for the construction procedure are presented below.

RECONSTRUCTION OF SLURRY WALL CAP

The 8-inch-thick, reinforced concrete slab shall be installed over the existing slurry wall as follows:

1. Excavate below the existing grade to approximately elevation 565 as shown on the attached Figure 1. The excavation should extend from 4 feet south of the slurry wall to 4 feet north of the slurry wall, with a 6-inch slope across the excavation from south to north. This excavation should be made with light equipment (equivalent to a Komatsu Model PC80 backhoe) located on the remediation site (south) side of the excavation area. Widening or reshaping of the existing bench and use of mats for spreading the equipment loads will likely be required. In no case should additional vertical loads be imposed on top of the slurry wall.

2. Place an 80-mil textured high density polyethylene (HDPE) geomembrane extending from beyond the south edge of the excavation to approximately 6 inches inside of the south

side of the slurry wall. At this location, install a 24-inch section of the 80-mil textured HDPE membrane that will be pushed into the existing slurry wall.

3. Place the ground stabilization fabric in the excavation on the south side of the slurry wall and then place the 12-foot-wide steel decking for the underslab form.

4. Place the steel reinforcing for the concrete slab and pour the slab in 50-foot maximum sections.

5. Wet (damp) cure the concrete slab surface for a minimum of 3 days to preclude shrinkage cracking.

CONSTRUCTION OF THE REMAINDER OF THE NORTH EMBANKMENT SLOPE

Since the subsoils in the north slope area adjacent to the slurry wall are in a weakened condition due to the previous slope movements, it is essential that heavy equipment be kept out of this area. The materials handling work should be accomplished with a light backhoe or Gradall located a minimum of 5 feet south of the existing slurry wall or on mats if light equipment must be operated on the bench between the slurry wall and the creek channel. No loaded trucks or heavy earthmoving equipment shall be operated on the lower bench (approximately elevation 566). The recommended procedure to be followed for completing the north embankment slope is as follows:

1. Since much of the cap adjacent to the steep excavation along the northern edge of the site has been disturbed by the heavy construction equipment operating in that area, at least the upper 1 to 2 feet of edge material should be removed. This material should be replaced to eliminate cracks in the subgrade so that the 2-foot clay cap can be placed on a continuous, stable subgrade.

2. Shape the north embankment slope as shown on Figure 1 to provide for an 18-inch-thick rip-rap section on the 2.5:1 slope and a 12-inch-thick rip-rap section on the bench at elevation 566 to 567. Place an 80-mil textured HDPE geomembrane liner on the shaped slope extending into an anchor trench to be constructed 5 feet south of the top of slope. This 80-mil geomembrane is to be welded to the section of 80-mil geomembrane placed beneath the concrete slab.

3. Place ground stabilization fabric (Supac 16NP) on the 2.5:1 slope from the anchor trench (at the top of the slope) to the edge of the section placed beneath the southern end of the concrete slab. Also place ground

stabilization fabric on the bench from the northern edge of the slurry wall (above the concrete slab) to the edge of the existing channel slope.

4. Weld the lower 40-mil geomembrane to the 80-mil geomembrane at the south edge of the anchor trench.

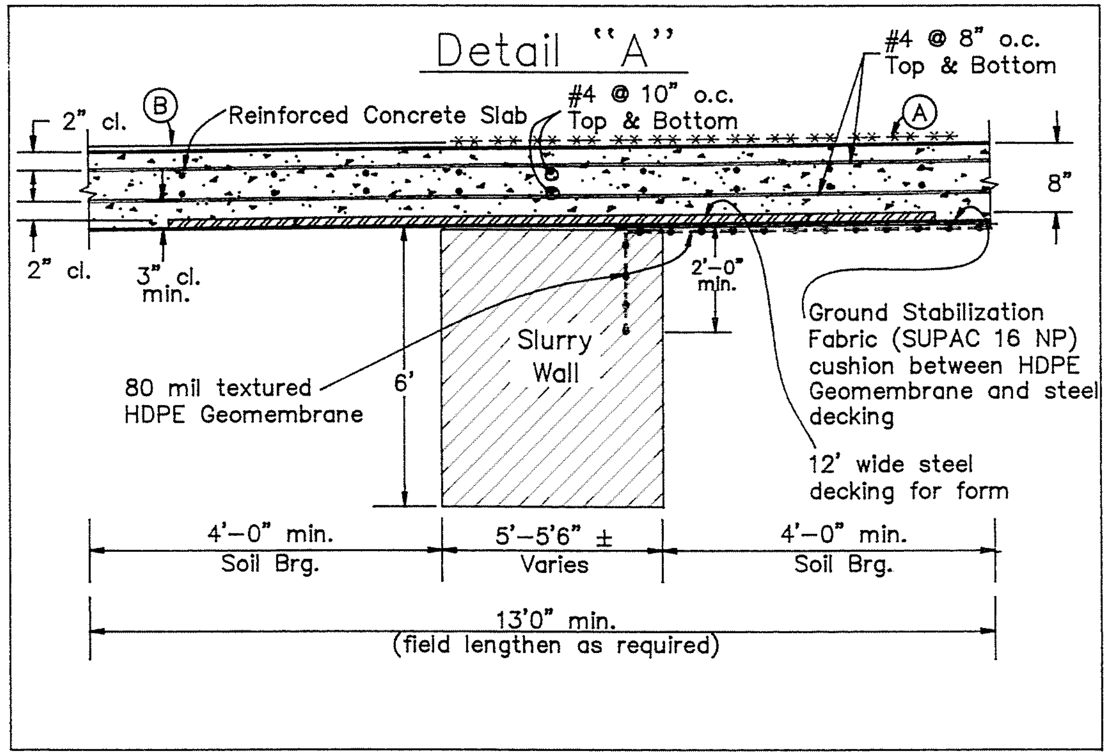
5. Place Mirafi Paragrid 100/255 geogrid on the 2.5:1 slope extending from the anchor trench 5 feet beyond (south of) the top of slope to the north edge of the slurry wall (above the concrete slab).

6. Carefully place the 12-inch layer of rip-rap on the bench at elevation 565 to 566 and the 18-inch layer up to the top of the 2.5:1 slope using a crane (with a clam-shell), backhoe, or Gradall and limiting the aggregate drop to 1 foot. No equipment traffic should be permitted on the rip-rap over the geomembrane liner.

7. Extend the 110-mil nonwoven geotextile and the upper 40-mil geomembrane to the edge of the rip-rap. Turn up the 60-mil nonwoven geotextile at the contact between the rip-rap and the select fill and topsoil to prevent migration of soil particles into the rip-rap.

8. Place compacted clay in the anchor trench at the top of slope and then place the 18 inches of structural fill and 6 inches of topsoil, as shown on Figure 1. Taper the thickness of the structural fill layer between the anchor trench and the top of slope to match the thickness of the rip-rap at the top of slope.

9. Seed the topsoil surface in accordance with the Project Specifications.



CONCRETE MIX

1. "Standard" ready mix for 4000 psi concrete (min.) high early strength (Type III)
   - A. Maximum water/cement ratio of .5
   - B. Maximum aggregate size 3/4"
   - C. Air entrainment – air content 6%
   - D. 6–1/2 bag mix
   - E. 3–inch slump (2"–4") check each truck in field
   - F. Cure – wet (damp) cure for 3 days minimum or use surface–applied curing compound as per manufacturer's recommendations.
   - G. Take 4 strength cylinders/slab pour – break 1st cylinder at 3 days, 2nd at 5th day, 3rd at 7th day, 4th (spare). Work can proceed on top of slab when concrete strength > 3000 psi (after minimum 3–day curing period)
2. Alternately precast concrete slabs, designed for 450 psf live load plus dead load of slab, may be used, provide membrane seal at all traverse joints

NOTE: Place transverse construction joints at 50' OC maximum. Joints shall be keyed (2" x 4" nominal key) with reinforcement continuous through the joint.

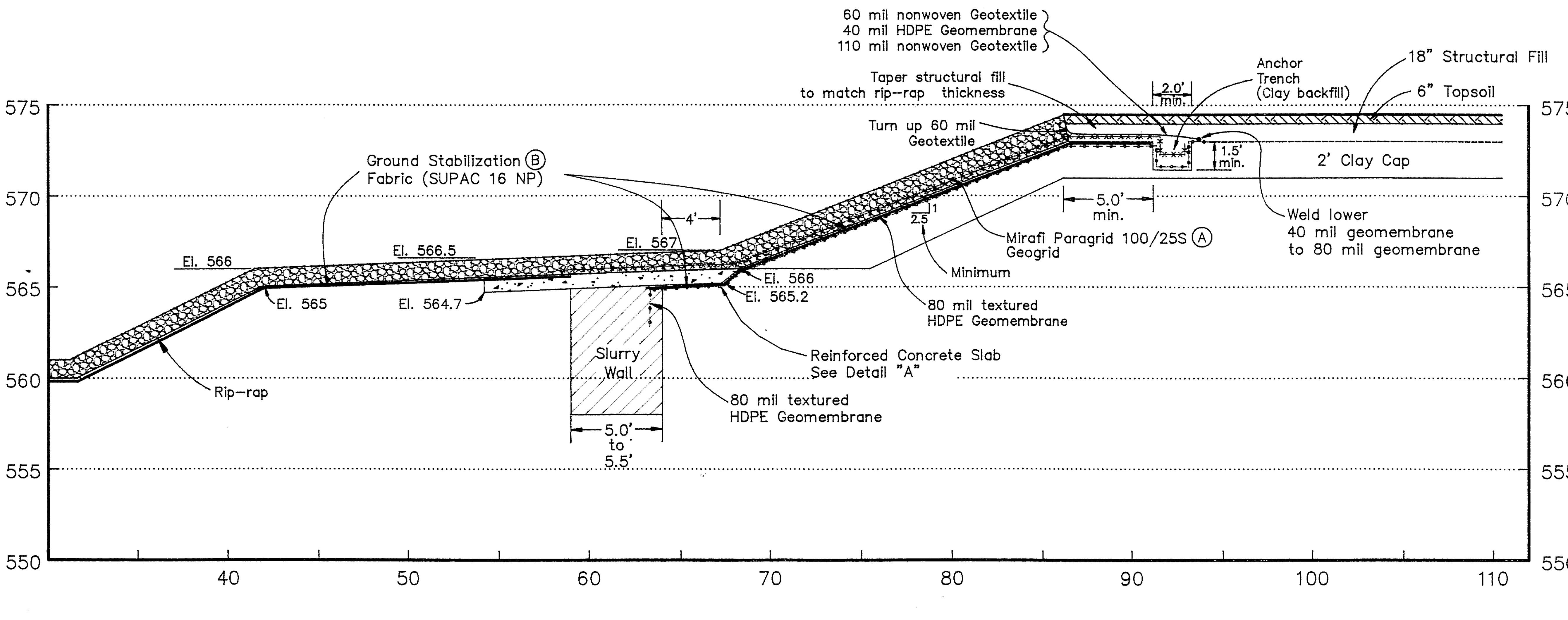


FIGURE 1

SLOPE STABILITY

Reconstruction of Slurry Wall Cap
and North Embankment Slope
Pine and Tuscarora Remediation Site
Niagara Falls, New York

SLOPE STABILITY
RECONSTRUCTION OF SLURRY WALL CAP AND NORTH EMBANKMENT SLOPE
PINE AND TUSCARORA REMEDIATION SITE
NIAGARA FALLS, NEW YORK

The slope stability for the north slope of the Pine and Tuscarora Site has been reanalyzed for the revised (2.5:1) slope condition, with the toe of slope being located south of the existing slurry wall. Three conditions have been studied:

1. Deep-seated failures assuming a soft layer extends from elevation 552 to 561 using both a circular arc and wedge failure surfaces

2. Shallow-seated failures assuming the failure surface is consistent with the bottom elevation of the slurry wall (elevation 558)

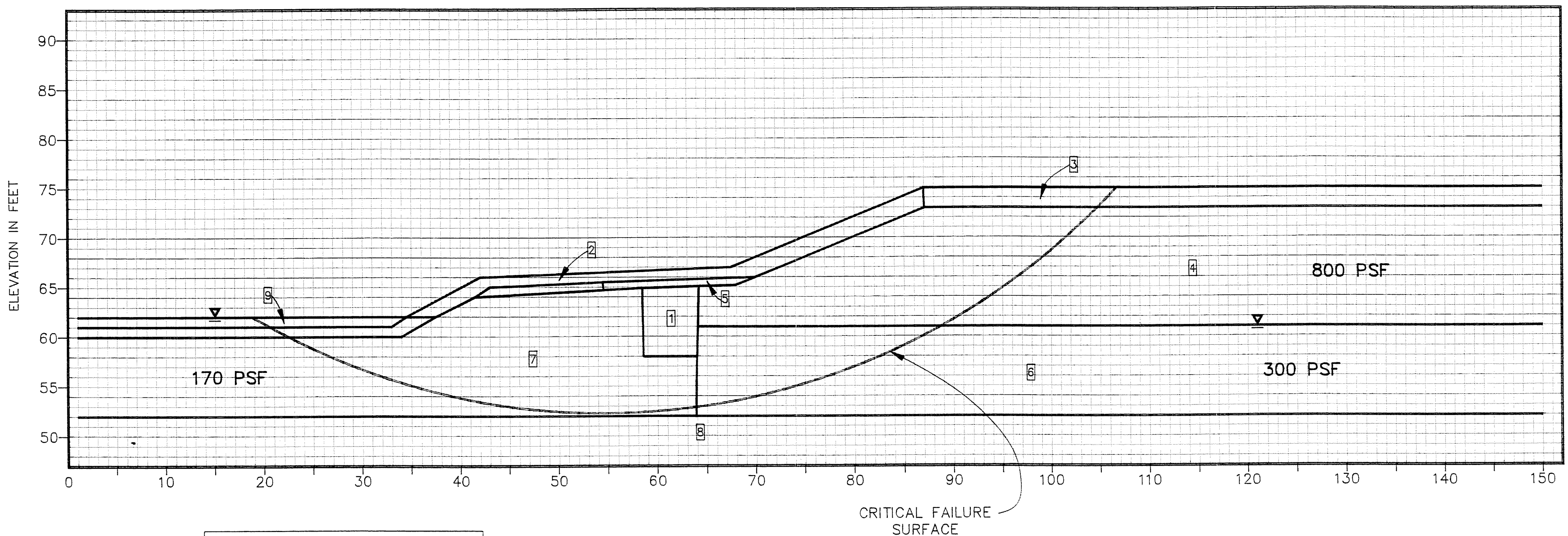
3. Hydraulic loading of the slurry wall with the failure surface extending from the bottom of the slurry wall to the near edge of the existing channel

The various slopes analyzed are shown on the attached cross-sections and are summarized below:

Failure Condition	Factor of Safety
Deep-Seated	
Circular Arc	1.7
Wedge Failure	1.6
Shallow-Seated	
Wedge Failure at Elev. 558:	
Residual Shear Strength, C_r = 170 psf	2.1
Residual Shear Strength, C_r = 100 psf	2.0
Hydraulic Loading of Slurry Wall	
WCC Assumed Failure Surface:	
Residual Shear Strength, C_r = 170 psf	2.0
Residual Shear Strength, C_r = 135 psf	1.6
Residual Shear Strength, C_r = 100 psf	1.2

Failure Condition	Factor of Safety
NYSDEC Assumed Failure Surface:	
Residual Shear Strength, C_r = 170 psf	2.4
Residual Shear Strength, C_r = 135 psf	1.9
Residual Shear Strength, C_r = 100 psf	1.4
Residual Shear Strength, C_r = 90 psf	1.3
Residual Shear Strength, C_r = 80 psf	1.1

SOIL NO.	STRATUM	γ (pcf)	c (psf)	Φ (°)
1	SLURRY	110	0	0
2	RIP-RAP	135	0	0
3	STRUCTURAL FILL	130	300	0
4	COMPOSITE MATERIAL	127.5	800	0
5	REINFORCED CONCRETE	150	10,000	0
6	SOFT CLAY	115	300	0
7	SOFT CLAY (REMOLDED)	115	170	0
8	BEDROCK	150	10,000	0
9	WATER	62.4	0	0



RUN NO.: WCCF.170
PROGRAM USED: STABL5
FACTOR OF SAFETY: 1.676

CIRCULAR ANALYSIS

No. Surfaces: 100
No. Init. Points: 10
Initiation Limits: 10–35
Termination Limits: 85–120

Circle Center:

Xc = 54.7 ft
Yc = 123.2 ft
R = 70.9 ft

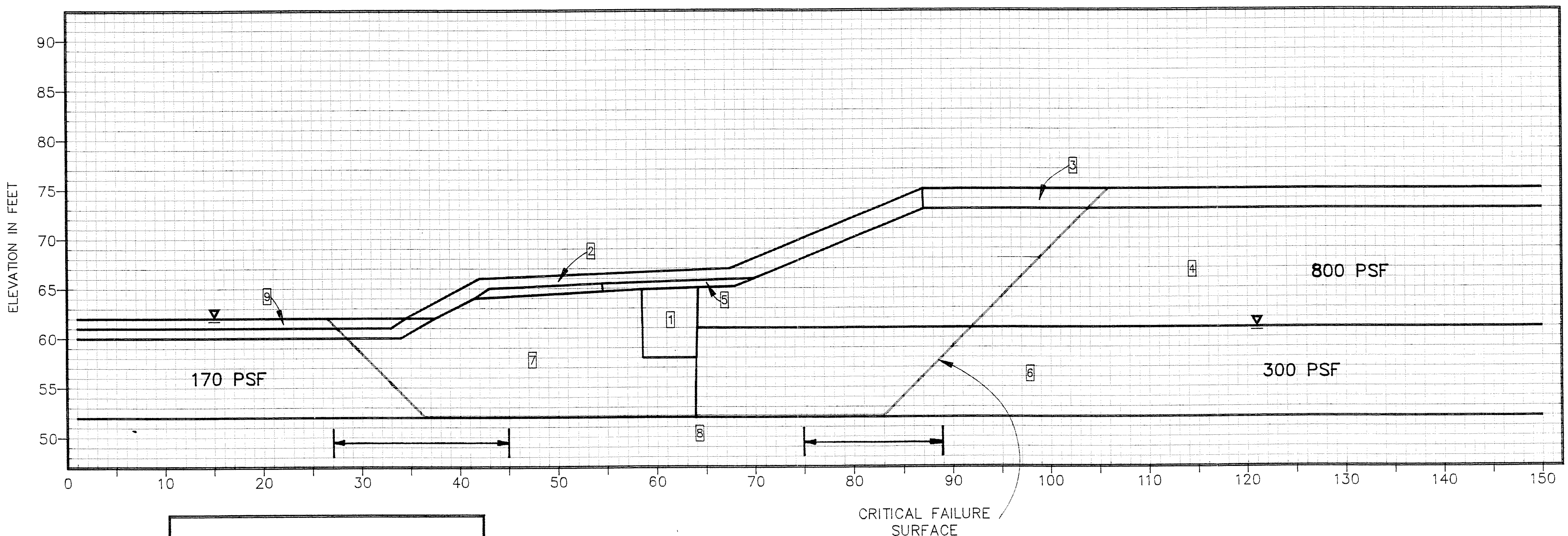
Rev. No.	Date	Type of Revision	Checked by:

DEEP-SEATED CIRCULAR ANALYSIS
PINE & TUSCARORA REMEDIATION SITE

Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857	Drawing No. 98570010	Date: 8/6/90
Drawn by: T.P.	Checked by: FSW	
Scale: 0 — 10 FEET		

SOIL NO.	STRATUM	γ (pcf)	c (psf)	Φ (°)
1	SLURRY	110	0	0
2	RIP-RAP	135	0	0
3	STRUCTURAL FILL	130	300	0
4	COMPOSITE MATERIAL	127.5	800	0
5	REINFORCED CONCRETE	150	10,000	0
6	SOFT CLAY	115	300	0
7	SOFT CLAY (REMOLDED)	115	170	0
8	BEDROCK	150	10,000	0
9	WATER	62.4	0	0



RUN NO.: WCDW.170
PROGRAM USED: STABL5
FACTOR OF SAFETY: 1.59

WEDGE ANALYSIS

Left Box: 27–45 (52.0, 0.1)
Right Box: 75–89
No. of Surfaces: 100

Fo Factor: $\frac{d}{L} = \frac{19.5}{82.5} = 0.236$

Fo = 1.106

F.S. = 1.435 x Fo = 1.59

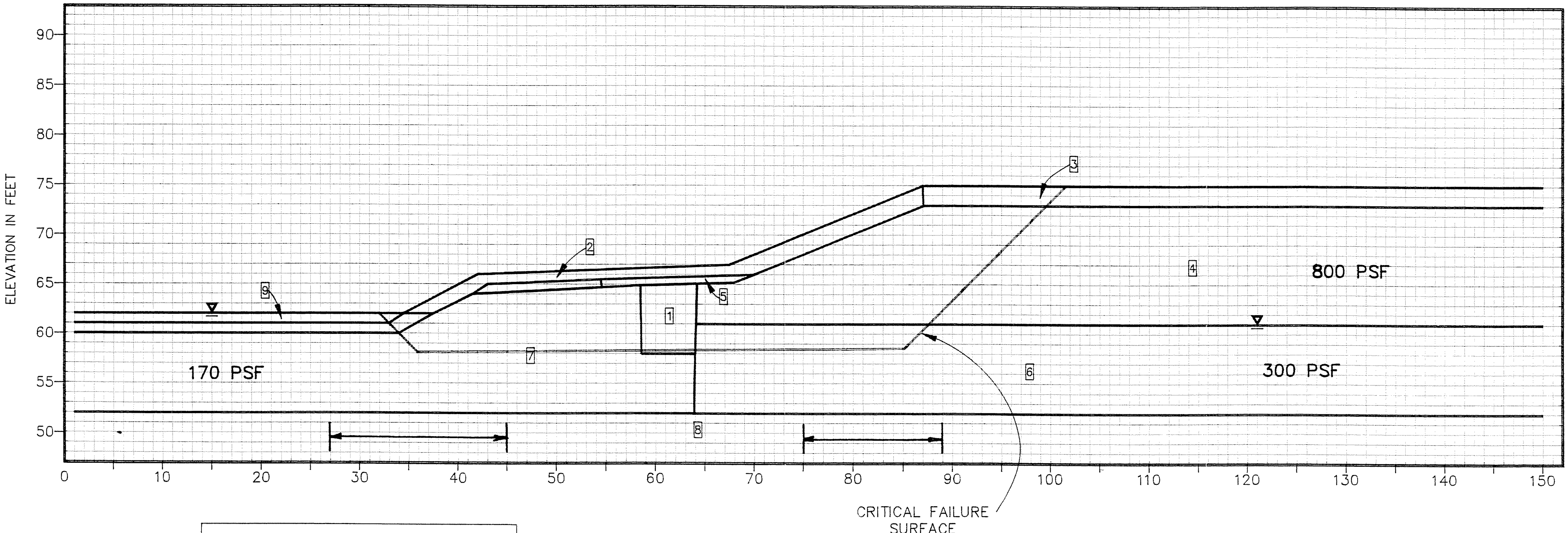
Rev. No.	Date	Type of Revision	Checked by:

DEEP SEATED WEDGE ANALYSIS
PINE & TUSCARORA REMEDIATION SITE

Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857 | Drawing No. 98570010 | Date: 8/6/90
Drawn by: T.P. | Checked by: FSW
Scale: 0 10 FEET

SOIL NO.	STRATUM	γ (pcf)	c (psf)	Φ (°)
1	SLURRY	110	0	0
2	RIP-RAP	135	0	0
3	STRUCTURAL FILL	130	300	0
4	COMPOSITE MATERIAL	127.5	800	0
5	REINFORCED CONCRETE	150	10,000	0
6	SOFT CLAY	115	300	0
7	SOFT CLAY (REMOLDED)	115	170	0
8	BEDROCK	150	10,000	0
9	WATER	62.4	0	0



RUN NO.: WCSW.170
PROGRAM USED: STABL5
FACTOR OF SAFETY: 2.13

WEDGE ANALYSIS

Left Box: 27–45 (59.0, 1.9)
Right Box: 75–89
No. of Surfaces: 100

Fo Factor: $\frac{d}{L} = \frac{13.5}{73} = 0.185$

Fo = 1.093

F.S. = 1.951 x Fo = 2.13

Rev. No.	Date	Type of Revision	Checked by:

SHALLOW-SEATED WEDGE ANALYSIS
PINE & TUSCARORA REMEDIATION SITE

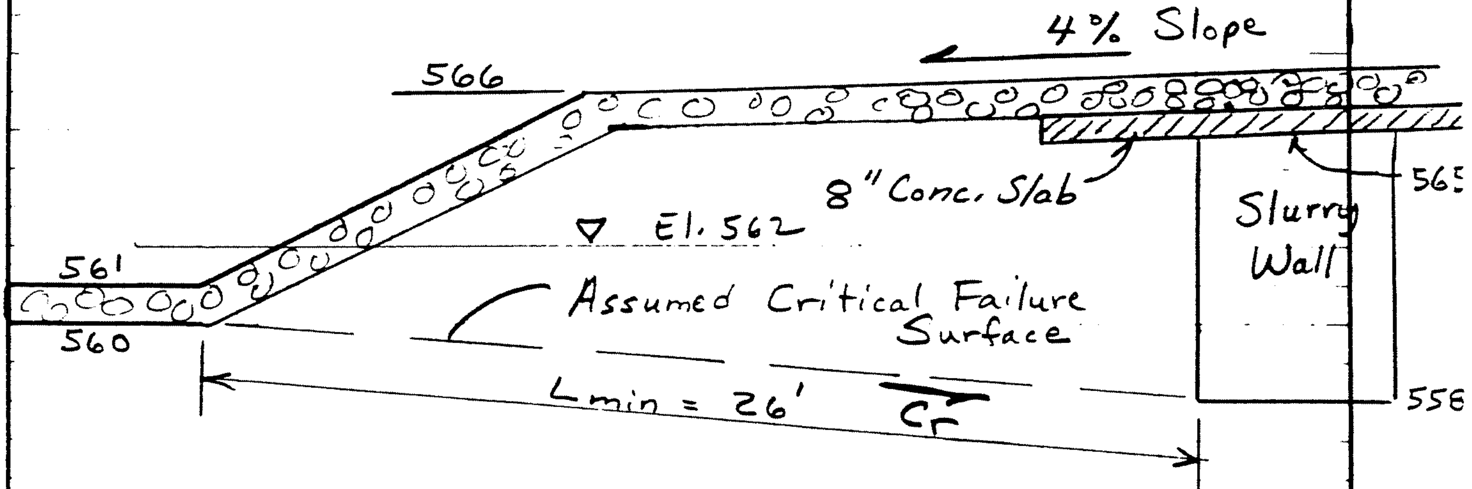
Woodward-Clyde Consultants
Consulting Engineers, Geologists and Environmental Scientists

Job No.: 89C2857 | Drawing No. 98570010 | Date: 8/6/90
Drawn by: T.P. | Checked by: FSW
Scale: 0 10 FEET

WCC Assumed Failure Surface

Assumptions

- Slurry weight = 110 pcf fluid pressure
- Concrete Slab bridges over slurry wall
- C_r = 170 psf (also study 135 & 100 psf)



Calculations

$$FOS = \frac{C_r L}{P} \quad \text{where } P = \tfrac{1}{2}\gamma_{sw} H^2 - \tfrac{1}{2}\gamma_w H_w^2$$

For C_r = 170 psf
$$FOS = \frac{170 \times 26}{(\tfrac{1}{2} \times 110 \times 7^2) - (\tfrac{1}{2} \times 62.4 \times 4^2)} = \frac{4420}{2695 - 499} = \frac{4420}{2196} = 2.0$$

For C_r = 135 psf
$$FOS = \frac{135 \times 26}{2196} = \frac{3510}{2196} = 1.6$$

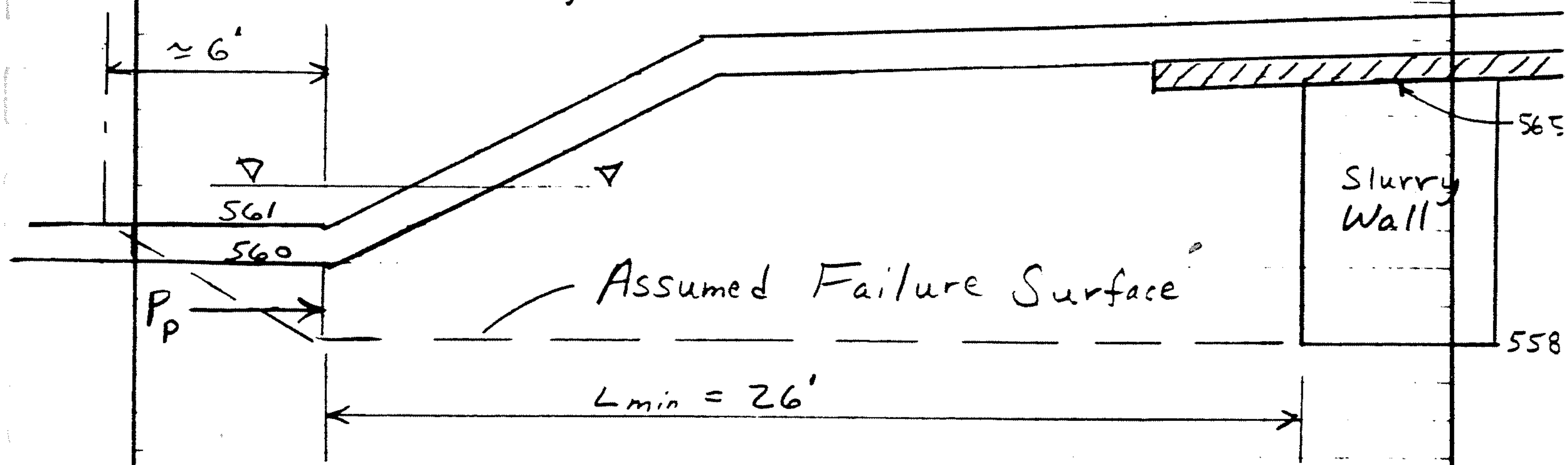
For C_r = 100 psf
$$FOS = \frac{100 \times 26}{2196} = \frac{2600}{2196} = 1.2$$

By: FSW Date: 8/6/90 Subject: Sliding Wedge Analysis Hydraulic Loading from Slurry Wall Sheet 2 of 3
Chkd. by: RDP Date: 8/8/90 Job No.: 89 C 2857

NYSDEC Assumed Failure Surface

Assumptions

- Driving Force, P_1 = Same = 2,196 lbs/ft
- P_p = Passive Toe Pressure
  $= \frac{1}{2}\gamma H^2 + 2cH \quad (\phi = 0)$ ✓
- Neglect rir-rap resistance



Calculations

$$P_p = \frac{1}{2}(115 - 62.4)\,2^2 + 2 \times C \times 2 \checkmark$$
$$= 105.2 + 4C \checkmark$$

For C_r = 170 psf, $P_p = 105.2 + 680 = 785.2$ ✓
$$\therefore FOS = \frac{(26 \times 170) + 785.2}{2196} = 2.4 \checkmark$$

For C_r = 135 psf, $P_p = 105.2 + 540 = 645.2$ ✓
$$\therefore FOS = \frac{(26 \times 135) + 645.2}{2196} = 1.9 \checkmark$$

For $C_r = 100$ psf, $P_p = 105.2 + 400 = 505.2$ ✓

$$\therefore FOS = \frac{(26 \times 100) + 505.2}{2196} = 1.4$$ ✓

For $C_r = 90$ psf, $P_p = 105.2 + 360 = 465.2$ ✓

$$\therefore FOS = \frac{(26 \times 90) + 465.2}{2196} = 1.3$$ ✓

For $C_r = 80$ psf, $P_p = 105.2 + 320 = 425.2$ ✓

$$\therefore FOS = \frac{(26 \times 80) + 425.2}{2196} = 1.1$$ ✓

Conclusion

Provided that no additional load is applied to the top of the slurry wall, the FOS against a "local" wedge failure is likely to be $\simeq 2.0$; assuming an ultra-conservative $C_r = 100$ psf (which WCC considers to be highly unlikely), the FOS is 1.2 for WCC failure surface and 1.4 for the NYSDEC failure surface (w/ passive toe wedge).

RESPONSES TO NYSDEC LETTERS

Reconstruction of Slurry Wall Cap
and North Embankment Slope
Pine and Tuscarora Remediation Site
Niagara Falls, New York

RESPONSE TO NYSDEC COMMENTS DATED JULY 19, 1990
RECONSTRUCTION OF THE SLURRY WALL
CHARLES GIBSON SITE
NIAGARA COUNTY, NEW YORK

A. STABILITY ANALYSIS

1. Selection of Failure Surface

   a. The conditions at the suspension of construction were analyzed to back-calculate the shear strength (Su) of the soft clay layer "at failure." The critical surface is a plane beginning at the base of the slurry wall and exiting at the surface of clay at the toe of slope. This surface yields the maximum shear strength for soft clay at a FOS = 1.0 when the slope is postulated to be on the verge of failure. For this failure surface a value of Su = 285 psf was back-calculated. Any other surface would yield a lower value of Su as described in Comment 1 of the July 19, 1990 letter from the NYSDEC, but it would not likely represent the critical failure surface.

   b. As noted in the NYSDEC July 19 letter, a toe bulge exists in the center of the creek opposite the failure area. WCC believes that this toe bulge is most likely due to the buckling of the 12 inches of rip-rap placed over a geotextile filter fabric in the bottom of the creek channel. As the channel slope adjacent to the slurry wall moved laterally some 2 to 2.5 feet, clearly the rip-rap had to buckle. In the stability analysis, WCC chose to ignore this very limited resistance due to the low confining pressure of only 1 foot of rip-rap.

   c. As noted in (a) and (b) above, WCC purposely picked the selected failure surface to compute the <u>maximum</u> shear strength that would yield a FOS = 1.0. The NYSDEC chose a slightly different failure surface and back-calculated a shear strength of 230 psf. Actually, there is little difference between 230 and 285 psf when considering the uncertainties associated with the back-calculation. If the cohesion estimated by NYSDEC of 230 psf were assigned to the soft clay stratum, the computed FOS for the critical failure surface selected by WCC reduces to FOS = 0.8; obviously a failure would occur along the critical surface rather than the one selected by NYSDEC, which has a 25 percent higher FOS.

2. Strength Reduction

Admittedly, the transition from the undrained to the drained condition approach in our calculations is not well described. The intent of the drained (c = 0) analysis approach presented on page 2 of 6 of the calculations was simply to compute an "equivalent" friction angle (ϕ_f) that equates to the apparent "peak" shear strength at the time of failure. WCC recognized that the peak shear strength would not be applicable once the slope failure had occurred. For this reason, the residual shear strength approach was selected to provide a "lower bound value" of shear strength to be used for evaluating remedial alternatives.

To estimate the lower bound of the strength along the failure plane after the large slope movement, the residual friction angle (ϕ_r) of the clay was used. ϕ_r was estimated for a Plasticity Index of 20 percent, following equation (1) after Kanji & Wolle (1977).[1]

$$\phi_r = 46.4 \, (PI)^{-0.446} \tag{1}$$

This angle is independent of the structure of the clay and depends only on the mineralogy of the constituents.[1]

It may be noted that in the calculations on page 2 of 6 in WCC's report dated July 9, 1990, a reduction factor of 0.58 was not assumed but was rather calculated to show the consequence of assuming $\phi_r = 12.5°$ after failure had occurred. A shear strength of 170 psf was calculated corresponding to this residual friction angle.

3. Cement Bentonite Fill Layer

To alleviate the concern over the strain rate of the cement bentonite and the consolidation rate of the slurry wall, WCC proposed during a conference call with the NYSDEC the alternative of using a rigid reinforced

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[1]Kanji, M. A. and Wolle, C. M. (1977) "Residual Strength - New Testing and Microstructure," Proc. 9th International Conference on Soil Mechanics and Foundation Engineering, Tokyo, Japan, Vol. I, pp. 153-154.

concrete beam (slab). This alternative would provide arching over the trench so that there would be no additional load imposed on top of the slurry wall. Such a condition will substantially improve the stability of the slope in the affected area. Preliminarily, all parties believe that changing the cement bentonite fill layer to a concrete slab would be a preferred modification to the reconstruction design. Another advantage is that the concrete slab would be easier and quicker to construct in the field.

The revised design included with this submittal incorporates a concrete slab over the slurry wall in lieu of the cement-bentonite fill layer. This eliminates the concern raised by Comment No. 3.

4. Reconstruction of Slurry Wall Cap, Item 2

   The 12-inch section to be inserted into the slurry wall will run continuously along the entire length of the slope failure reconstruction. The 12-inch section is being increased to 24 inches as recommended by the NYSDEC.

5. This comment is no longer relevant since the cement-bentonite fill layer will not be used.

6. Construction of the Remainder of the North Embankment Slope, Item 3, Last Sentence.

   The detail for connecting the lower 40-mil geomembrane to the 80-mil geomembrane has been changed by the revised design. The upper edge of the 80-mil geomembrane is buried in an anchor trench 5 feet from the top of slope as shown on the revised Figure 1, and will be welded to the edge of the lower 40-mil geomembrane. The upper 40-mil geomembrane extends horizontally to the top edge of the rip-rap zone.

7. Ridge in Middle of Cayuga Creek

   This ridge has been modeled in the flood routing studies and has no measurable impact.

8. WCC has been advised verbally by Mr. Jerry Sparks of Dewberry and Davis, consultants to FEMA, that our HEC-2 flooding study results look fine. Mr. Sparks stated that Olin should hear from FEMA in about one month's time. Mr. Sparks had one question concerning a discrepancy between the distances upstream for cross-section A in the

original study and in WCC's Revised Existing Study. WCC pointed out that there was an apparent error in the original study. Mr. Sparks seemed satisfied with the answer, but thought we might need to send a letter to FEMA describing this change.

B. REMEDIAL DESIGN

The five provisions to be specified for the remedial design work are acceptable.

RESPONSE TO JAMES P. COLLINS' LETTER DATED 7/22/90

Regarding the likely failure surface, this was answered by our response to Comment No. 1 of the NYSDEC letter dated July 19, 1990. A failure mode that includes a passive toe wedge is not as critical as the failure surface which WCC assumed. The toe bulge observed in the creek channel is believed to be the buckling of the rip-rap layer rather than evidence that a deeper failure surface and passive wedge are included in the failure mass. The failure surface assumed by WCC has only a 4.4 degree inclination; thus, the WCC assumed failure mode is translation rather than rotation as was incorrectly stated in the Collins letter. It is possible that part of the bulge in the creek channel is a zone of soft clay in the channel bottom as postulated by Collins, considering that during construction the channel bottom was so soft that they reportedly had to mix some dry clay with the wet clay before the geotextile filter fabric could be placed.

With respect to the shear strength and factor of safety calculations presented on the top of page 2 of the Collins letter, WCC does not agree with these calculations. As noted in our response to the NYSDEC Comment No. 1, any calculated shear strength less than the maximum shear strength that will yield a FOS = 1 is less critical. For example, if the strength value of 247 psf computed by Collins were applied to the WCC assumed failure surface, a FOS of 0.87 is computed confirming that the WCC failure mode is more critical than that proposed by Collins. Furthermore, we disagree with application of the WCC calculated reduction factor of 0.58 (rounded to 0.60) to the less critical shear strength value. The "reduction factor" represents the ratio of the shearing resistance proposed for remediation (170 psf) to the back-calculated resistance (285 psf). If a shear strength of 247 psf instead of 285 psf were used, the reduction factor becomes 0.69 and c_r is 170 psf.

WCC does not consider rapid drawdown as a feasible loading condition. The embankment is being constructed of clay which has been tested to have coefficient of permeability values in the 10^{-7} cm/sec range. Obviously these soils are too impermeable to be affected by the saturation and subsequent excess pore pressure loading caused by rapid drawdown. The 12 inches of rip-rap is free-draining and also is unaffected by the rapid drawdown condition.

Collins' Recommendations

1. WCC considers that the computed residual shear strength of 170 psf is appropriate and represents a conservative "lower bound" value.

2. The redesign included with this submittal, which incorporates a reinforced concrete slab designed to span over the slurry wall, should alleviate this recommendation. As noted above, rapid drawdown is not relevant to this slope condition.

3. A flatter slope than 4:1 is not necessary at this site. WCC believes that the redesign has adequate factors of safety for both the end-of-construction and long-term conditions. Any future slope movements would most likely occur during or immediately after construction. If any detrimental movements occur, these would have to be remediated to the satisfaction of appropriate regulatory agencies.

Regarding the geomembrane cutoff recommendation, Olin has agreed to increase the depth of the cutoff from 1 foot to 2 feet. However, WCC and Olin believe that constructing a duplicate geomembrane cutoff is not necessary.

RESPONSES TO NYSDEC LETTER DATED JULY 26, 1990

Woodward-Clyde Consultants (WCC) offers the following responses to the additional comments presented in the NYSDEC letter dated July 26, written by Mr. A. Jeffrey Marachi, P.E. This letter raised concerns on two issues, selection of the failure surface and strength reduction factors.

SELECTION OF FAILURE SURFACE

We believe that the failure surface chosen by WCC is the critical failure surface, but it is not too different from the surface proposed by others. As part of this submittal, we have analyzed multiple failure surfaces, including the critical failure as computed by both a circular failure arc and a wedge analysis. We have also analyzed the wedge failure surface that the NYSDEC and their consultants seem to favor, a translation wedge failure through the bottom of the slurry wall. Thus, which failure surface to select is somewhat of a mute point since both have been analyzed for the revised design and they both have acceptable factors of safety.

SHEAR STRENGTH REDUCTIONS

The interpretation of the WCC approach and calculations submitted to the State is included as items 1-6 (page 2) of the NYSDEC July 26, 1990 letter. The interpretation is correct as we assumed $\phi_{cu} = \phi_r$ and obtained the residual strength of 170 psf. In our calculations we did not assume that excess pore pressures are present. However, it is assumed that pore pressures can be accommodated with an adequate end-of-construction safety of factor of at least 1.2. We recognized that the back-calculated equivalent friction angle was too high in comparison with the residual strength friction. Thus, we selected a residual friction angle of 12.5 degrees from established correlations with plasticity index. The design shear strength was calculated as 165 psf and rounded off to 170 psf. We believe that this strength represents a reasonable shearing resistance for slide remediation and have used this value in our analysis.

Recognizing NYSDEC's concerns with our strength reduction approach, we have included a parametric study in the revised calculations to determine the factor of safety as a function of the residual shear strength (C_r). These calculations indicate for the postulated failure surface, which passes through the bottom of the slurry wall and exits in the bottom of the channel as a wedge failure, would have a factor of safety of 1.3 for a residual shear strength as low as C_r = 90 psf. It is WCC's opinion that the residual shear strength is at least as high as this value and more likely in the range of 170 psf, which gives a factor of safety of 2.0 to 2.4, depending which postulated failure plane is used.

For additional conservatism, we have also assumed the soft soils on the channel side of the slurry wall to be completely remolded (i.e. C_r = 170 psf) and have allowed the computer to search for the critical deep failure circle and wedge analysis failure surface; the resulting factors of safety were computed to be 1.7 and 1.6, respectively. Clearly, this proposed model is ultra-conservative, as the wedge failure through the bottom of the slurry wall is far more likely. It is WCC's opinion that these calculations confirm that the proposed embankment can be constructed with an adequate end-of-construction (short-term) and long-term factor of safety against future slope failures.