Charles Gibson Site #932063

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July 9, 1990
89C2857

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

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

Dear Jim:

Enclosed are the recommended 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 reconstruction involves flattening the embankment (channel) slope in the area experiencing slope movement from a 2:1 to a 4:1 slope. Slope stability calculations are also enclosed for the 4:1 slope.

We have also completed the FEMA flood routing for the revised design and the report is in preparation. The new routing study gave the following results:

1. The revised "original" creek 100-year flood levels are lower than the former "original" levels at all cross sections by tenths or hundredths of 1 foot.

2. The new creek (with 4:1 slope) floodway run (i.e. 100-year) elevations are lower than the former floodway levels by tenths or hundredths of 1 foot.

3. The 1.00 foot maximum rise criteria has not been exceeded.

4. The above conditions incorporate the two transition cross sections requested by FEMA, at the upstream and downstream property boundaries, and an additional station for the flattened 4:1 slope at project station 1+00 (HEC-2 station 7880) plus the additional channel length of 80 feet.

These are the data that were requested by FEMA, and these results should meet their technical requirements.

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

Frank S. Waller

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

FSW/mek
Enclosures

cc: G. Blaine Butaud - Olin
George Harbison - Olin
Alan Elia - Sevenson
Richard T. Roe
Majed Khoury
Paul Dutko

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 4:1 slope (versus an original 2:1 design slope) plus a reduced height of slurry wall (approximately 6 feet versus 10 feet). A cement-bentonite fill layer, 2 feet thick, is to be poured over the existing slurry wall to provide a stronger material that can bridge over the 5- to 5.5-foot-wide slurry wall, but still be somewhat pliable so that it may strain with the embankment without developing large cracks. 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 cement-bentonite fill slurry wall cap should be constructed in short segments of not more than 50 feet in length (along the slurry wall) at one time. The purpose of this staged construction is to avoid digging a trench the full length of the reconstruction zone which would reduce the factor of safety on the existing steep excavation slope on the cap side of the slurry wall. Also, this will allow completing all work associated with the cement-bentonite fill zone within a given section the same day. In fact, only sections that can be excavated and backfilled with the cement-bentonite fill the same day should be excavated at any one time. The specific procedures recommended for the slurry wall cap reconstruction are as follows:

1. Excavate below the existing grade to elevation 564 as shown on the attached Figure 1. The excavation should extend from 5 feet south of the slurry wall to the point where the 4:1 embankment slope (beneath the 18-inch-thick rip-rap zone) crosses elevation 564. Typically, this will be approximately 9 to 10 feet north of the channel side of the existing (widened) slurry wall. 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.

2. Place an 80-mil textured high density polyethylene (HDPE) geomembrane extending from the north edge of the excavation to approximately 6 inches inside of the south side of the slurry wall. At this location, provide a 12-inch section of the 80-mil textured HDPE membrane that will be pushed into the existing slurry wall.

3. Construct a clay fill "form" at the northern edge of the cement-bentonite layer, 5 feet north of the north edge of the slurry wall.

4. Pour approximately 4 inches of cement-bentonite fill and then lay a Tensar UX1500 (SR2-D) geogrid on top of the 4 inches of the cement-bentonite. Alternately, use similar techniques which will provide embedment of the Tensar grid with 4 inches of cement bentonite beneath it.

5. Place the remainder of the cement-bentonite fill to elevation 565.7.

6. Place a second Tensar UX1500 (SR2-D) geogrid layer over the cement-bentonite fill surface from the northern edge of the slurry wall (see Figure 1) and extend it all the way back to the southern edge of the excavation at elevation 566. Where the geogrid is over the cement-bentonite fill, place an additional 4 inches of cement-bentonite fill.

7. Allow the cement-bentonite slurry wall to cure for a minimum of 7 days before placing the clay fill over the cement-bentonite fill layer. Keep the surface of the cement-bentonite fill covered with polyethylene and the surface of the area moistened to prevent cracking of the cement-bentonite fill as it cures.

8. Construct the adjacent section of the cement-bentonite a minimum of one day later.

9. The cement-bentonite fill mix should be as follows:

   <u>Bentonite:Water Ratio</u>

   B:W = 0.045 (approx. 75 lb bentonite/cubic yard water)
   Use 90 BBL (barrel) yield bentonite (no additives)

   <u>Cement:Water Ratio</u>

   C:W = 0.25 (approx. 4-1/2 bags cement/cubic yard slurry)
   Use Type II cement

   Mix should have 7 day strength of about 12 to 15 psi and 7 day permeability of about 5 x 10^{-6} cm/sec. Permeability would be expected to decrease slightly with time to approximately 3 x 10^{-6} cm/sec, and strength at 28 days increases to approximately 20 psi.

   Values are <u>approximate</u> and should be verified by laboratory testing with trial mixes using proposed site constituents. A minimal quantity of fluidifier may also be needed to reduce the viscosity during mixing and pumping. Laboratory testing could be used also to estimate the quantity of fluidifier.

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. In fact, no equipment should be driven directly over the slurry wall and the cement-bentonite fill. Only hand-operated equipment should be used directly over the cement-bentonite fill. 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, approximately the southern edge of the cement-bentonite fill. The recommended procedure to be followed for completing the north embankment slope is as follows:

1. Place a 6-inch lift of clay fill over the cement-bentonite fill and existing subgrade, and compact with hand-operated equipment such as a Model P-24EC Rammax walk-behind compactor. The clay fill should be placed and graded with a Gradall or light backhoe with a smooth-edged materials-handling bucket. Alternately, a low ground pressure (LGP) light bulldozer, such as a Komatsu Model D-37P or Caterpillar Model D-3 LGP, could be used to spread the clay fill, provided the bulldozer does not operate directly over the cement-bentonite fill, remaining at least 3 to 5 feet from its southern edge. Place the clay fill up to elevation 568, forming the 4:1 slope as shown on Figure 1. The clay fill should be placed wet, of optimum moisture content and compacted to achieve 90 percent of the maximum standard Proctor density (ASTM D 698).

2. 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. Removing this material to elevation 568 would achieve a bench at the elevation of the clay fill (elevation 568) from which a crane, backhoe, or Gradall could operate to construct the slope protection over and north of the slurry wall.

3. Shape the north embankment slope as shown on Figure 1 to provide for an 18-inch-thick rip-rap section with a bench at elevation 566, and then extending up slope at an inclination of 4:1. Place an 80-mil textured HDPE geomembrane liner on the shaped slope. Place a small triangle of clay fill over top of the double geomembrane liner system at the toe of the slope to create the fill subgrade to elevation 564.5. Weld the lower 40-mil geomembrane to the 80-mil geomembrane at least 5 feet south of the top of slope.

4. Place ground stabilization fabric (Supac 16NP) on the 4:1 slope up to 2 feet beyond the upslope limit of the rip-rap. Lap the 60-mil geotextile, upper 40-mil HDPE geomembrane, and 110-mil geotextile over the 80-mil textured HDPE downslope to this location.

5. Place Mirafi Paragrid 100/255 geogrid on the 4:1 slope extending from 5 feet beyond (south of) the top of slope to 10 feet beyond the bottom of slope over the ground stabilization fabric).

6. Carefully place the 18-inch layer of rip-rap on the bench at elevation 566 and up the slope to elevation 571, 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. Above elevation 571, place the 18 inches of structural fill and 6 inches of topsoil, as shown on Figure 1.

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

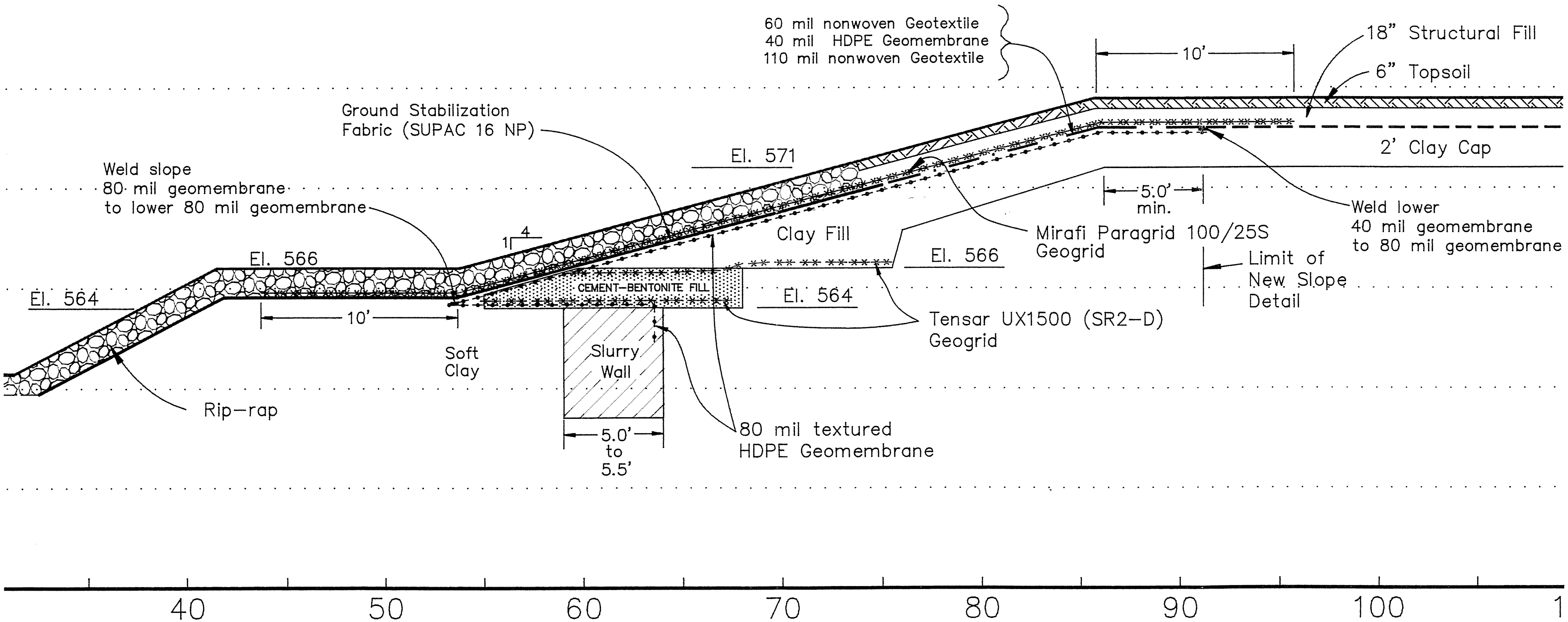

60 mil nonwoven Geotextile
40 mil HDPE Geomembrane
110 mil nonwoven Geotextile
10'
18" Structural Fill
6" Topsoil
Ground Stabilization
Fabric (SUPAC 16 NP)
El. 571
2' Clay Cap
Weld slope
80 mil geomembrane
to lower 80 mil geomembrane
5.0'
min.
Weld lower
40 mil geomembrane
to 80 mil geomembrane
1
4
Clay Fill
Mirafi Paragrid 100/25S
Geogrid
El. 566
El. 566
Limit of
New Slope
Detail
CEMENT-BENTONITE FILL
El. 564
El. 564
10'
Tensar UX1500 (SR2-D)
Geogrid
Soft
Clay
Slurry
Wall
Rip-rap
80 mil textured
HDPE Geomembrane
5.0'
to
5.5'
40
50
60
70
80
90
100
1

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 using field geometry for the existing conditions. Approximately two days after constructing the 2-foot clay cap over the slurry wall, settlement of the cap was observed together with a toe bulge in the middle of the channel. Also, longitudinal cracks along the edges of the slurry wall soon began to appear. After a few days, the toe bulge was observed to extend from about Station 1 to Station 3 of the new channel, and evidence of slumping and movement at the top of the 2-foot clay cap were apparent. It is obvious that the weight of the slurry wall backfill and the construction equipment used to place the clay cap had exceeded the shear strength of the underlying weak, soft clay layer below about elevation 561. Since there was evidence that movement was continuing at a very slow rate, Woodward-Clyde Consultants recommended that the top (approximately 6 feet) of the fill zone over the slurry wall be removed down to about elevation 565.5, forming a bench sloping from elevation 566 at the toe of excavation to 565 at the channel's edge. Removing the driving load of the fill from elevation 571 down to 565.5 (average) and the upper few feet of the slurry wall was judged to be adequate to mitigate the movement of the existing slope.

Attached are additional calculations used to reanalyze the slope conditions at the time of failure and also the recommended flattened 4:1 slope. Since the failure appears to have occurred by the lateral spreading of the 3-foot-wide slurry wall, a sliding wedge analysis was used to back-calculate the probable shear strength at the time of failure. Shown as "Case 1 - Conditions at Failure" on the attached calculations, the undrained shear strength, c, is estimated at the time of failure to have been approximately 285 psf. Since the slurry wall has spread by up to 2 to 2-1/2 feet, it is believed that the residual shear strength should be used for subsequent stability analyses.

Although the undrained shear strength tests performed on the soft natural clays in the laboratory showed no reduction in shear strength with large strains, for conservatism we propose to reduce the peak shear strength based on typical values found in the literature for similar clay materials. Conservatively, we have assumed that the residual friction angle could be as low as 12.5 degrees. Using this friction angle relative to the computed friction angle at the time of failure, estimated to be about 21 degrees (Figure 2), gives approximately a 60 percent reduction in strength for a residual condition. Applying the 60 percent reduction to the estimated peak shear strength of 285 psf yields a residual undrained shear strength of 171 psf. For the remediation design slope stability analyses, we have chosen to use a shear strength of 170 psf. More likely, the shear strength is expected to be a minimum of 200 psf and more probably in the 225 to 250 psf range. Thus, the factors of safety shown on the attached calculations are believed to be low by as much as 30 to 50 percent.

For the proposed remedial design, it is estimated that the factor of safety for a sliding wedge, similar to the failure that occurred previously, would be about 2.7 after excavating the slurry wall to elevation 564. Pouring the 2-foot-thick cement-bentonite fill layer over the slurry wall will temporarily reduce the factor of safety to about 1.6 until the slurry wall sets up. Once the cement-bentonite layer has set up, it is believed that the loading to the top of the slurry wall will be reduced substantially as the slurry wall continues to consolidate slightly and gain in strength with time.

Computer stability analyses were used to evaluate the slope stability of the overall slope for the following three conditions: (1) the 2:1 original design slope, (2) the flattened 4:1 slope, and (3) the construction condition with a crane sitting adjacent to the steep slope. The results of these analyses are shown on the attached Figures 3 through 5. For analysis purposes, parametric studies were performed using various strength parameters so that we could evaluate the impact of variable soil strengths on the factor of safety. The original design case (Figure 3), a 2:1 embankment slope, a soil strength of 800 psf in the materials above elevation 561 and a strength of 300 psf for the zone between elevation 561 and 552, yielded a factor of safety of 1.5. For these same soil strength conditions, the flattened 4:1 slope (Figure 3) will have a factor of safety of 1.8. Recognizing that the slope failure that has occurred along the north slope has resulted in a lower shear strength zone, we have reanalyzed the slope by reducing the shear strength in the soft layer between elevation 561 and 552 from 300 to 170 psf, consistent with the conservatively estimated residual shear strength previously discussed above for the wedge analysis. This change resulted in a computed factor of safety of 1.4. However, the true factor of safety is believed to be higher, because a major portion of the critical circle passes through this soft layer to the cap side of the slurry wall, where the reduced shear strength should not exist.

Recognizing that there has been some disturbance to the upper materials by the recent construction and embankment movements, we also evaluated the reduced shear strength for the materials above elevation 561. Using a shear strength of 650 and 550 psf for the stiff upper zone results in a factor of safety of 1.2 and 1.1, respectively. Again, we believe that these factors of safety are lower than actually exist because of the fact that the undrained shear strength of 170 psf for the entire Stratum 3 is overly conservative. Based upon the parametric studies that have been done, it is believed that the factor of safety for the flattened 4:1 slope will be at least 1.5.

A wedge analysis (Figure 4) was also run using a computer slope stability program. This analysis yielded similar, but slightly higher, results compared to the circular failure mode for both the 2:1 and 4:1 slopes as summarized below.

	Factor of Safety	
Slope	Circular Failure	Wedge Failure
2:1	1.2	1.4
4:1	1.5	1.6

An analysis (Figure 5) was also conducted to determine the factor of safety for the construction condition when the 2 feet of material is to be excavated along the slurry wall in 50-foot segments. Assuming the existing steep slopes and a crane load of 1,000 psf, a factor of safety of 1.2 was estimated. Although this factor of safety is low, it is a short-term construction condition and the analysis parameters are believed to be very conservatively estimated. Thus, the true factor of safety is expected to be substantially higher than the calculated factor of safety of 1.2.

In summary, the slope stability analyses indicate that the flattened 4:1 slope will have an adequate design factor of safety. However, this is predicated on keeping construction equipment, other than light manually-guided equipment, off the top of the slurry wall.

By: FSW Date: 7/2/90 Subject: Sliding Wedge Analysis Sheet 1 of 6
Chkd. by: RDS Date: 7/6/90 Job No.: 89 C 2857
Pine & Tuscarora Site

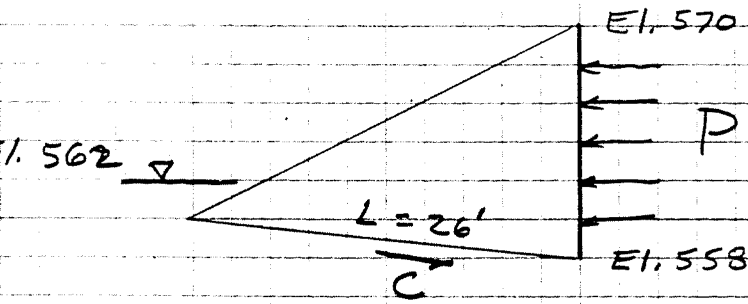
Case 1 - Conditions at Failure

Assumptions: (See Fig. 1)

- Slurry Wall not in place long enough to develop any strength and exerts a full fluid pressure ($\gamma = 110$ pcf)
- 2' Clay Cap and Geogrid placed over slurry wall berm (2' of loose clay) spans over the slurry wall
- $FS \leq 1.0$ at failure
- No additional stresses due to compaction of clay cap; this is probably not a correct assumption at the time of clay cap construction

Calculations

$$FS = \frac{cL}{P}$$



Calculate c at failue

$$c = \frac{P}{L}$$

$$= \frac{\frac{1}{2}\gamma H^2 - \frac{1}{2}\gamma_w h_w^2}{L} = \frac{1}{2} \times 110 \times 12^2 - \frac{1}{2} \times 62.4 \times 4^2$$

$$= \frac{7920 - 500}{26}$$

$$= 285 \text{ psf}$$ ✓ ← estimated peak cohesion at failure

Calculate Φ_u @ Failure (Refer to Fig. 2)

$$FS = W_T \tan \Phi_{f} / P \qquad \text{where } P = 7920 - 500 = 7420 \text{ lbs}$$

(Avg. above Φ_f)

$$W_1 = (5 \times 11.75 \times 126.5) - (5 \times 3.75 \times 62.4)$$

$$= 7{,}432 - 1172$$

$$= 6260 \text{ lbs}$$

$$W_2 = \left[21 \times \left(\frac{11.5+1}{2}\right) \times \overset{\text{Avg.}}{126.5}\right] - \left[21 \times \left(\frac{3.5+2}{2}\right) \times 62.4\right]$$

$$= 16{,}603 - 3{,}604$$

$$= 12{,}999 \text{ lbs}$$

$$W_T = 19{,}259 \text{ lbs}$$

$$\tan \Phi_f = \frac{7{,}420}{19{,}259} = 0.385$$

$$= 21^\circ$$

This Friction Angle is too high for a Residual Friction Angle that exists in the field following the slope failure. From literature, Φ_r for a clay w/ a Plasticity Index of 20 should be approximately $12^\circ \sim 13^\circ$; use $\Phi_r = 12.5^\circ$

$$\therefore \frac{\tan \Phi_r}{\tan \Phi_{peak}} = \frac{\tan 12.5^\circ}{\tan 21^\circ} = 0.58; \text{ use } 60\% \text{ strength}$$

For Remediation Design, assume

$$C_r = 0.60 \times 285 = 171 \text{ psf} \checkmark$$

<u>use 170 psf</u> (very Conservative)

Case 2 - Remediation Design
Flattened 4:1 Slope (See Fig. 3)

Assumptions:

- Slurry wall will be removed to El. 564, leaving a 6' high wall
- Construction will proceed before slurry gains any appreciable strength, ∴ use fluid pressure of 110 pcf
- Cement-bentonite wall could temporarily impose an added weight until it sets up
- After cement-bentonite wall sets up, it will span across the slurry wall w/o imposing substantial vertical loads
- Assume GWL @ El. 562
- Assume construction equipment will be kept off the top of the slurry to preclude additional stresses imposed by construction equipment

Calculations

For Slurry Wall @ El. 564

$$FS = \frac{C_r L_r}{P}$$

where: $P = \frac{1}{2}\gamma H^2 - \frac{1}{2}\gamma_w h_w^2$

$= \frac{1}{2} \times 110 \times 6^2 - \frac{1}{2} \times 62.4 \times 4^2$

$= 1980 - 499$

$= 1481$

$C_r = 170$ psf

$L_r = 26' - 2'$ (assumes 2' slurry wall movement)

$$FS = \frac{170 \times 24}{1481} = \frac{4080}{1481} = 2.75 ✓$$

For Cement Bentonite Fill over Slurry Wall

Assume Cement-Bentonite Fill weighs 80pcf

$P = (\frac{1}{2} \times 80 \times 2^2) + (160 \times 6) + 1980 - 499$

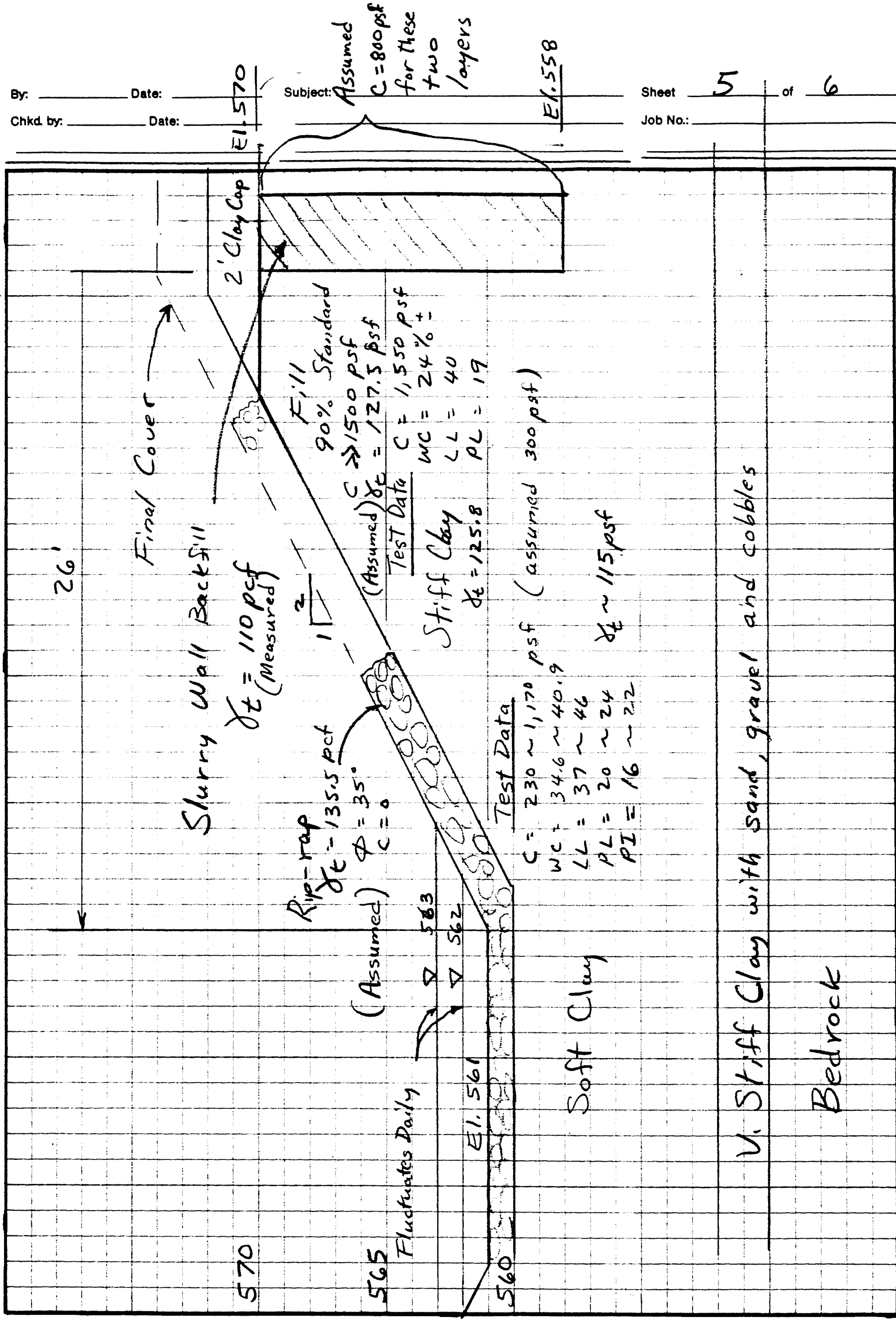
$= 160 + 960 + 1980 - 499$

$= 2601$

$$FS = \frac{4080}{2601} = 1.57$$

By: Date: Subject: Sheet 5 of 6

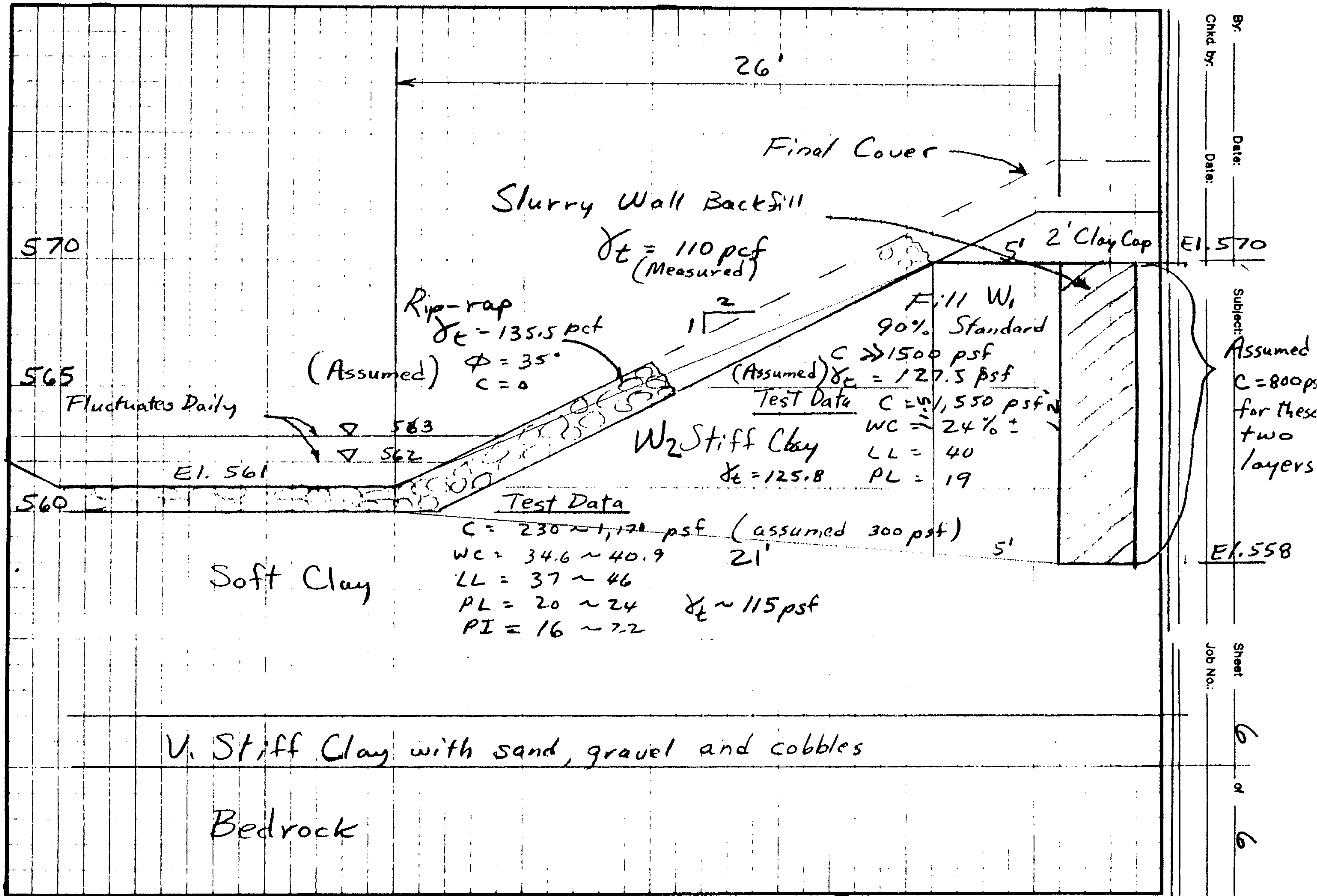
Chkd. by: Date: Job No.:



Scale: 1" = 5'

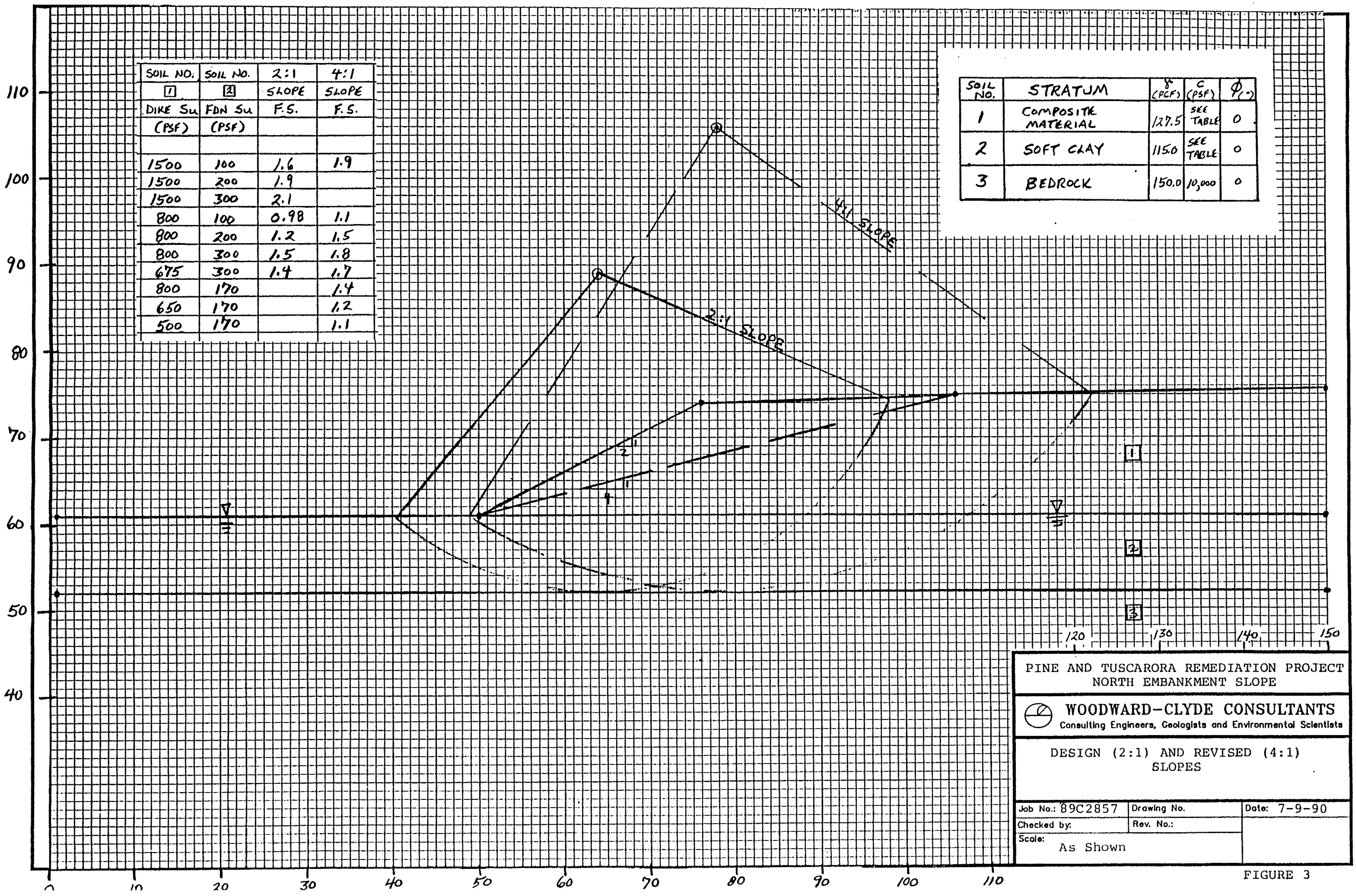
Fig. 1 - Conditions At Failure

By: ______ Date: ______ Subject: ______ Sheet 6 of 6
Chkd. by: ______ Date: ______ Job No.: ______



Scale: 1" = 5'

Fig. 2 - Conditions At Failure (Φ_u Analysis)



SOIL NO.	SOIL NO.	2:1	4:1
1	2	SLOPE	SLOPE
DIKE Su	FDN Su	F.S.	F.S.
(PSF)	(PSF)		
1500	100	1.6	1.9
1500	200	1.9	
1500	300	2.1	
800	100	0.98	1.1
800	200	1.2	1.5
800	300	1.5	1.8
675	300	1.4	1.7
800	170		1.4
650	170		1.2
500	170		1.1

SOIL NO.	STRATUM	γ (PCF)	C (PSF)	ϕ (°)
1	COMPOSITE MATERIAL	127.5	SEE TABLE	0
2	SOFT CLAY	115.0	SEE TABLE	0
3	BEDROCK	150.0	10,000	0

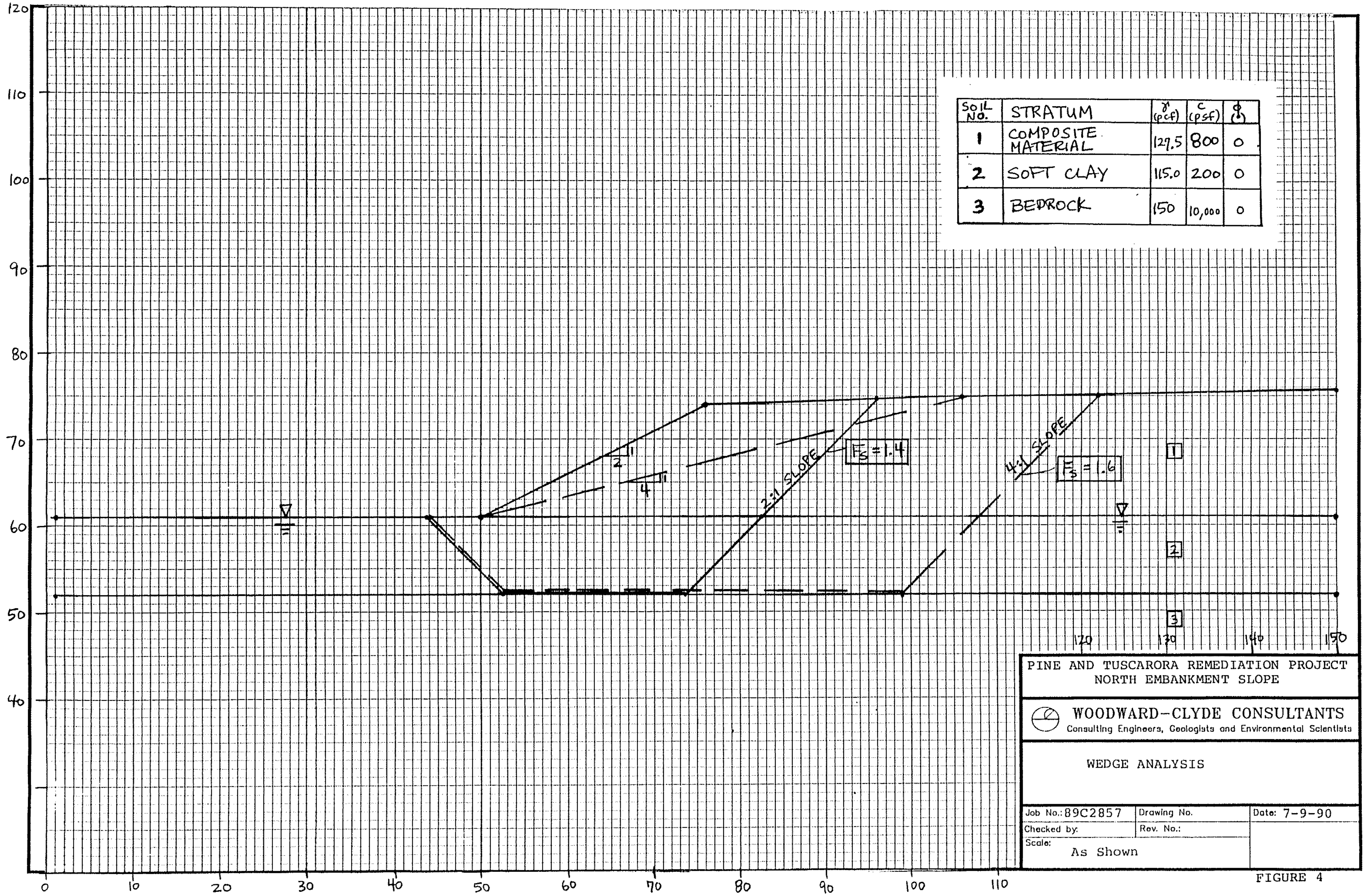
PINE AND TUSCARORA REMEDIATION PROJECT
NORTH EMBANKMENT SLOPE

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists

DESIGN (2:1) AND REVISED (4:1) SLOPES

Job No.: 89C2857 | Drawing No. | Date: 7-9-90
Checked by: | Rev. No.:
Scale: As Shown

FIGURE 3



SOIL NO.	STRATUM	γ (pcf)	c (psf)	φ (°)
1	COMPOSITE MATERIAL	127.5	800	0
2	SOFT CLAY	115.0	200	0
3	BEDROCK	150	10,000	0

PINE AND TUSCARORA REMEDIATION PROJECT
NORTH EMBANKMENT SLOPE

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists

WEDGE ANALYSIS

Job No.: 89C2857	Drawing No.	Date: 7-9-90
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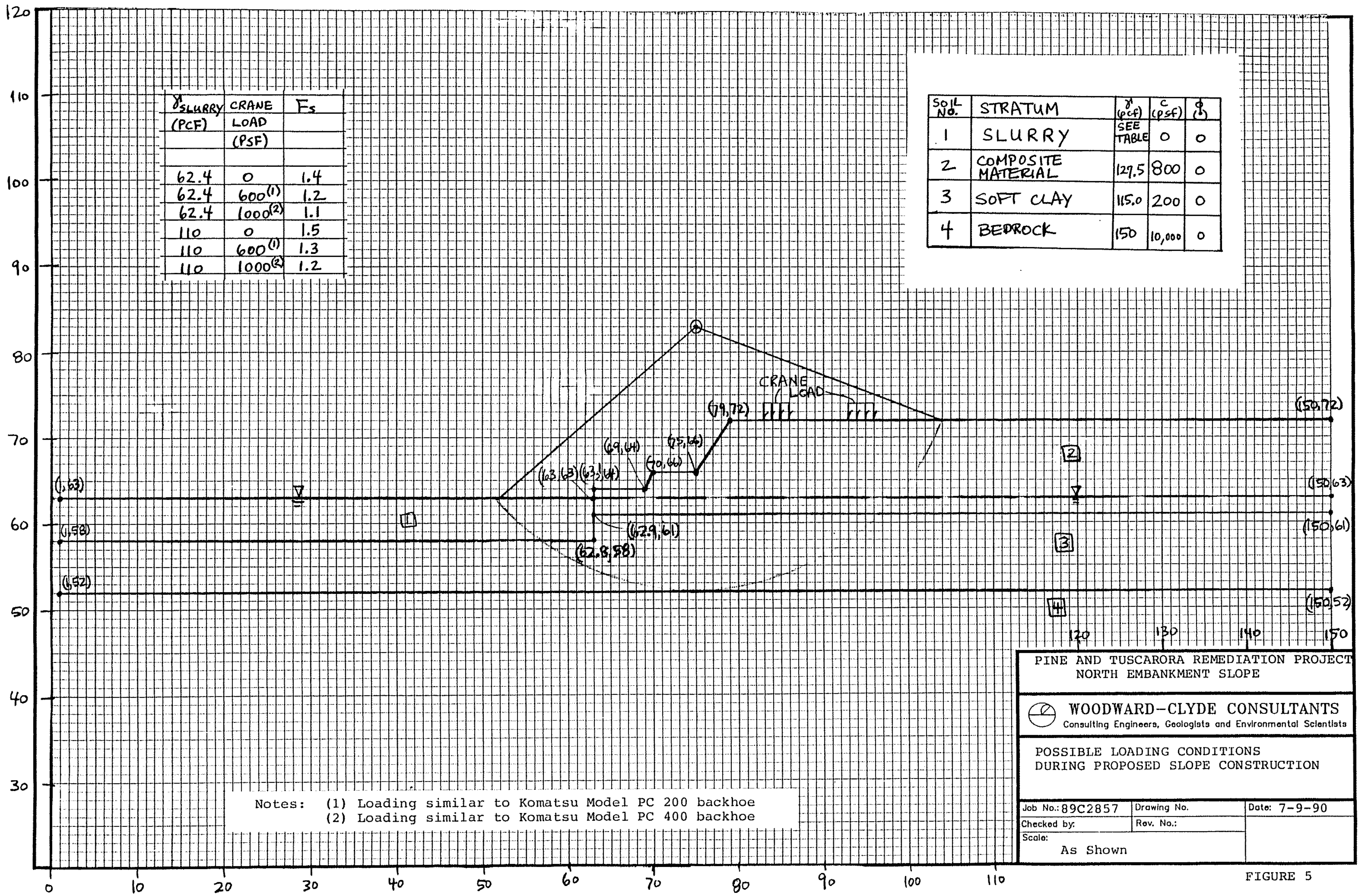
FIGURE 4



γ_{SLURRY} (PCF)	CRANE LOAD (PSF)	F_s
62.4	0	1.4
62.4	600 (1)	1.2
62.4	1000 (2)	1.1
110	0	1.5
110	600 (1)	1.3
110	1000 (2)	1.2

SOIL NO.	STRATUM	γ (pcf)	c (psf)	ϕ (°)
1	SLURRY	SEE TABLE	0	0
2	COMPOSITE MATERIAL	127.5	800	0
3	SOFT CLAY	115.0	200	0
4	BEDROCK	150	10,000	0

Notes: (1) Loading similar to Komatsu Model PC 200 backhoe
(2) Loading similar to Komatsu Model PC 400 backhoe

PINE AND TUSCARORA REMEDIATION PROJECT
NORTH EMBANKMENT SLOPE

WOODWARD-CLYDE CONSULTANTS
Consulting Engineers, Geologists and Environmental Scientists

POSSIBLE LOADING CONDITIONS
DURING PROPOSED SLOPE CONSTRUCTION

Job No.: 89C2857 | Drawing No. | Date: 7-9-90
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FIGURE 5