60095742 - TVA

Slope Stability Analyses

File Name	Material 1 Dike - Ash (& Dike B2)				Material 2 Ash - Sluiced			Material 3 Ash - (s_{up}=100 psf)			Material 4 Laminated Sensitive Silts and Ash Slimes 1			Material 5 Laminated Sensitive Silts and Ash Slimes 2			Material 6 Laminated Sensitive Silts and Ash Slimes 3			Material 7 Laminated Sensitive Silts and Ash Slimes 4			Material 8 Laminated Sensitive Silts and Ash Slimes 5			Material 9 Laminated Sensitive Silts and Ash Slimes 6			Material 10 Soft Clay			Material 11 Dike - Compacted Clay Fill			Material 12 Dike B1 and Swan Pond Road			Material 13 Rail Road Fill			Material 14 Alluvium - Clays and Silts			Material 15 Alluvium - Silty Sands and Silts			FOS
	γ_s	γ_m	ϕ	c	γ_s	ϕ	s_{up}/σ_v	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ_s	ϕ	c	γ	ϕ	c	γ	ϕ	c	γ	ϕ	p	γ	ϕ	p	M&P
Cell 2 Northwest Section																																																	
Stage 1 (Phi = 30 deg in sluiced ash)																																																	
Cell 2 NW Section Stage 1 Case 1	107	102	37	-	107	30	-	-	-	-	90	-	700	90	-	800	90	-	900	90	-	1100	90	-	1300	90	-	1500	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.32
Cell 2 NW Section Stage 1 Case 2	107	102	37	-	107	30	-	-	-	-	90	-	600	90	-	700	90	-	800	90	-	1000	90	-	1250	90	-	1400	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.24
Cell 2 NW Section Stage 1 Case 2 Circular	107	102	37	-	107	30	-	-	-	-	90	-	600	90	-	700	90	-	800	90	-	1000	90	-	1250	90	-	1400	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.60
Stage 2 (S_{up}/σ = 0.30 in sluiced ash)																																																	
Cell 2 NW Section Stage 2 Case 1	107	102	37	-	107	-	0.30	-	-	-	90	-	700	90	-	800	90	-	900	90	-	1100	90	-	1300	90	-	1500	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.03
Cell 2 NW Section Stage 2 Case 2	107	102	37	-	107	-	0.30	-	-	-	90	-	600	90	-	700	90	-	800	90	-	1000	90	-	1250	90	-	1400	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	0.97
Cell 2 NW Section Stage 2 Case 2 Circular	107	102	37	-	107	-	0.30	-	-	-	90	-	600	90	-	700	90	-	800	90	-	1000	90	-	1250	90	-	1400	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.02
Stage 3 (Surcharge = 1,600 psf)																																																	
Cell 2 NW Section Stage 3 Post-Failure Case 1	107	102	37	-	107	30	-	107	-	100	90	-	600	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	-	250	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	0.68
Cell 2 NW Section Stage 3 Post-Failure Case 2	107	102	37	-	107	30	-	107	-	100	90	-	600	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	0.96
Cell 2 NW Section Stage 3 Pre-Failure Case 1	107	102	37	-	107	30	-	-	-	-	90	-	600	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	-	250	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.24
Cell 2 NW Section Stage 3 Pre-Failure Case 2	107	102	37	-	107	30	-	-	-	-	90	-	600	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	-	600	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.72
Cell 2 NW Section Stage 3 Pre-Failure Case 1 Circular	107	102	37	-	107	30	-	-	-	-	90	-	600	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	-	250	120	15	600	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.50
Stage 4																																																	
Cell 2 NW Section - Stage 4	107	102	37	-	107	30	0.30	107	-	100	90	-	1200	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	0.0	1200	-	-	-	0.66
Cell 2 Southwest Section																																																	
Cell 2 Southwest Section Case 1	107	102	37	-	107	30	-	-	-	-	90	-	1100	90	-	1175	90	-	1250	90	-	1400	-	-	-	-	-	-	-	-	-	-	-	-	114	37	-	120	37	-	110	0.0	1200	130	30	600	1.49
Cell 2 Southwest Section Case 2	107	102	37	-	107	-	0.30	-	-	-	90	-	1100	90	-	1175	90	-	1250	90	-	1400	-	-	-	-	-	-	-	-	-	-	-	-	114	37	-	120	37	-	110	0.0	1200	130	30	600	1.09
Cell Phase 1 East Section																																																	
Cell Phase 1 East Section Case 1	107	102	37	-	107	30	-	107	-	100	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.56
Cell Phase 1 East Section Case 2	107	102	37	-	107	-	0.30	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	-	110	0.0	1200	130	30	600	1.01

Page 1 - 1

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CELL 2 NW SECTION SUMMARY
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER

M:\PROJECTS\60095742\DWG\slope_stability-figs\G60095742_4C_-CELL 2 NW SECTION INDEX.dwg; 6/12/2009 1:52:18 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 1 Case 1 (Average Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Drained Condition
Block Specified Failure Surface

Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

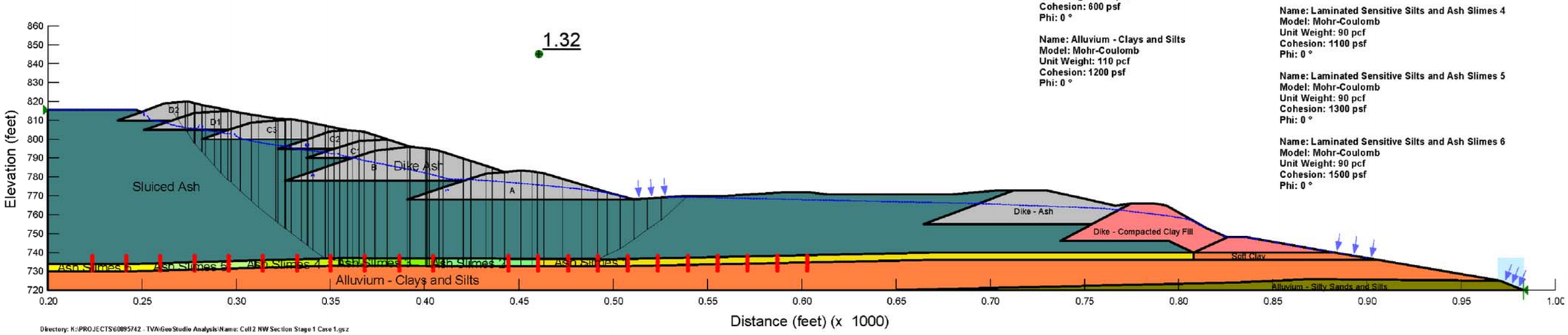
Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 900 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1100 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1300 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1500 psf
Phi: 0 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section Stage 1 Case 1.gsz

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CELL 2 NW SECTION STAGE 1 CASE 1-WEDGE
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_1

M:\PROJECTS\60095742\DWG\Seepage Figures\G60095742_4C_1-Cell 2 NW Section Stage 1 Case 1.dwg; 6/11/2009 5:11:43 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 1 Case 2 (Low Bound Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Drained Condition
Block Specified Failure Surface

Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

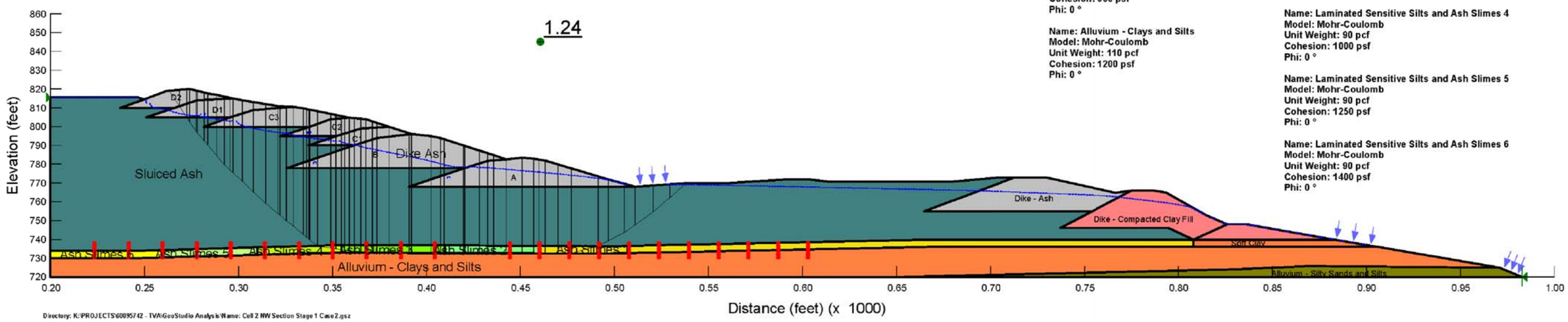
Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1000 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section Stage 1 Case2.gsz

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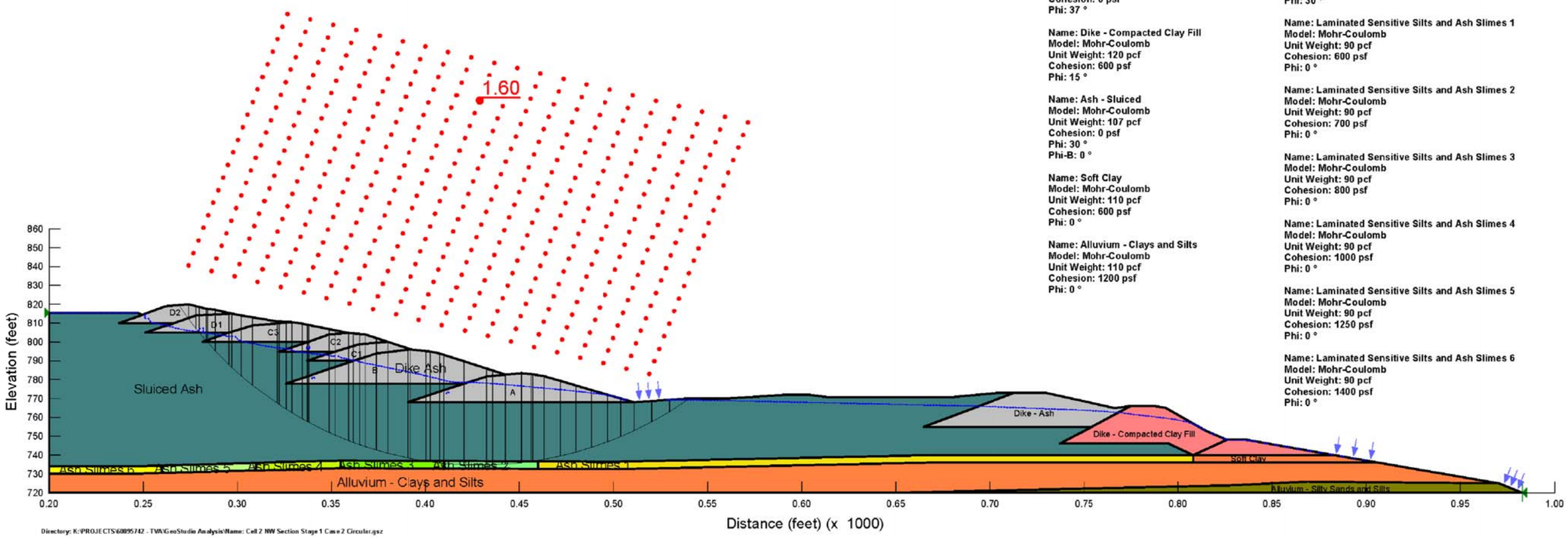
CELL 2 NW SECTION STAGE 1 CASE 2 - WEDGE
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009
PROJECT NUMBER	60095742	
FIGURE NUMBER	4C_2	

M:\PROJECTS\60095742\DWG\Seepage Figures\G60095742_4C_2-Cell 2 NW Section Stage 1 Case 2.dwg: 6/11/2009 5:17:15 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 1 Case 2 (Low Bound Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Drained Condition
Grid and Radius Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °
Phi-B: 0 °

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1000 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °

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CELL 2 NW SECTION STAGE 1 CASE 2 - CIRCULAR
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

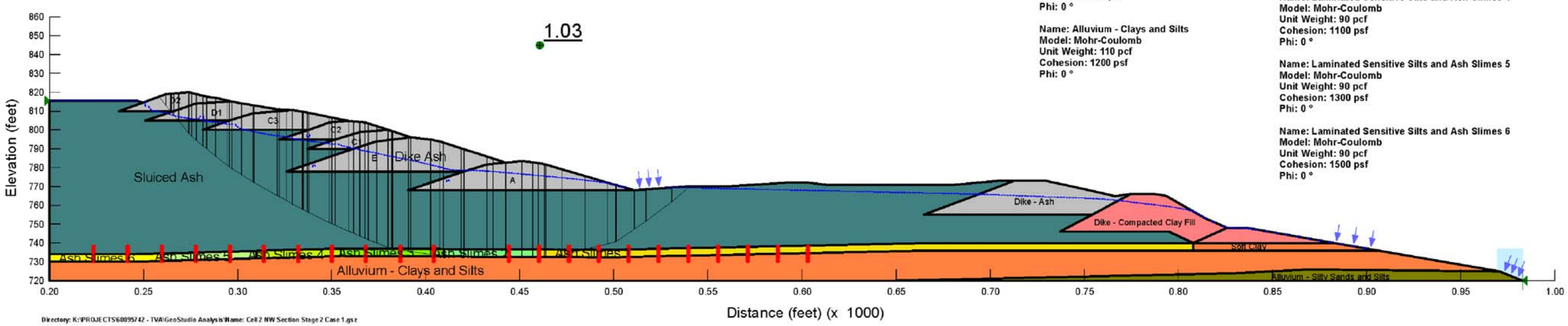
Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_3

M:\PROJECTS\60095742\DWG\Seepage Figures\G60095742_4C_3-Cell 2 NW Section Stage 1 Case 2 Circular.dwg: 6/11/2009 5:19:10 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 2 Case 1 (Average Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Undrained Condition
Block Specified Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 900 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1100 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1300 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1500 psf
Phi: 0 °

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CELL 2 NW SECTION STAGE 2 CASE 1 - WEDGE
SLOPE STABILITY ANALYSES
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DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_4

M:\PROJECTS\60095742\DWG\Seepage Figures\G60095742_4C_4-Cell 2 NW Section Stage 2 Case 1.dwg; 6/11/2009 5:23:58 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 2 Case 2 (Low Bound Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Undrained Condition
Block Specified Failure Surface

Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

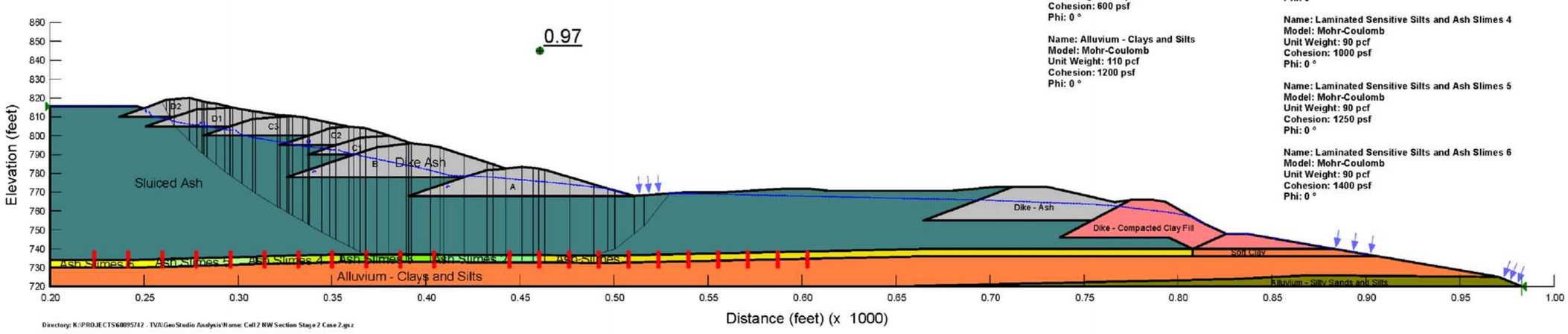
Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1000 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section Stage 2 Case 2.gsz

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CELL 2 NW SECTION STAGE 2 CASE 2 - WEDGE
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

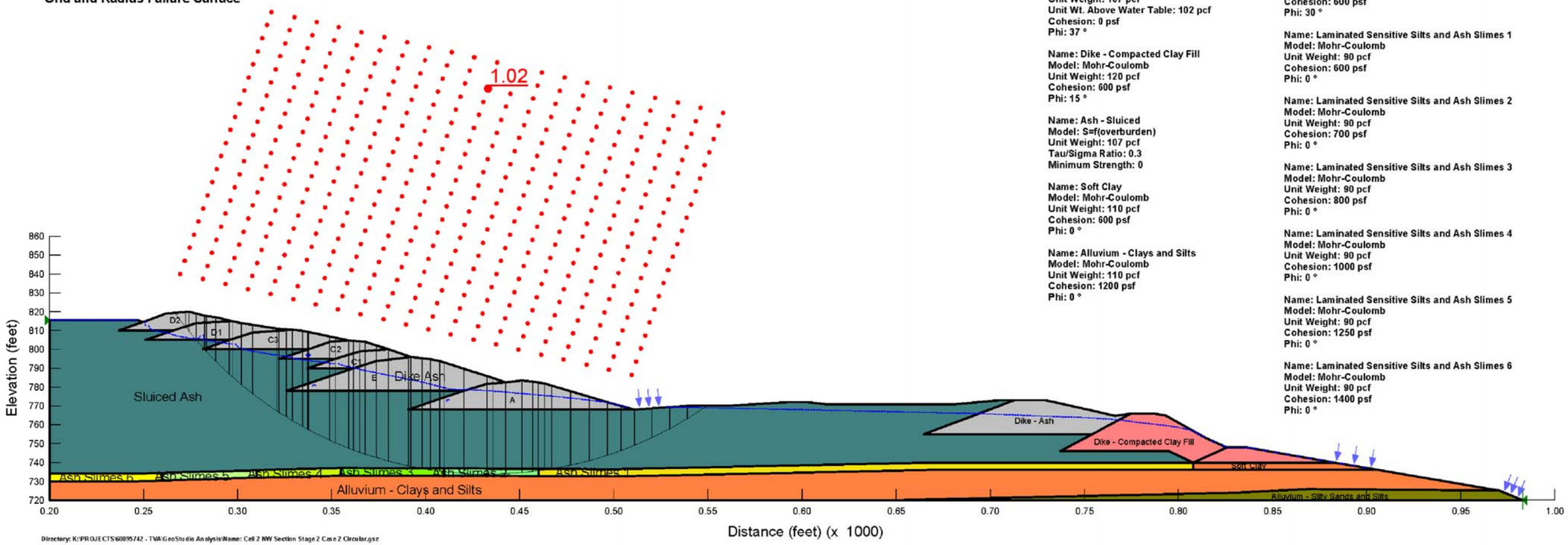
Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_5

M:\PROJECTS\60095742\DWG\Seepage Figures\G60095742_4C_5-Cell 2 NW Section Stage 2 Case 2.dwg: 6/11/2009 5:26:31 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Northwest Section Stage 2 Case 2 (Low Bound Strength of Ash Slimes)
Method: Morgenstern-Price
Analysis Sluiced Ash Undrained Condition
Grid and Radius Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Ash - Sluiced
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 700 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 800 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1000 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 5
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 6
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °

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CELL 2 NW SECTION STAGE 2 CASE 2 - CIRCULAR
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009
PROJECT NUMBER	60095742	
FIGURE NUMBER	4C_6	

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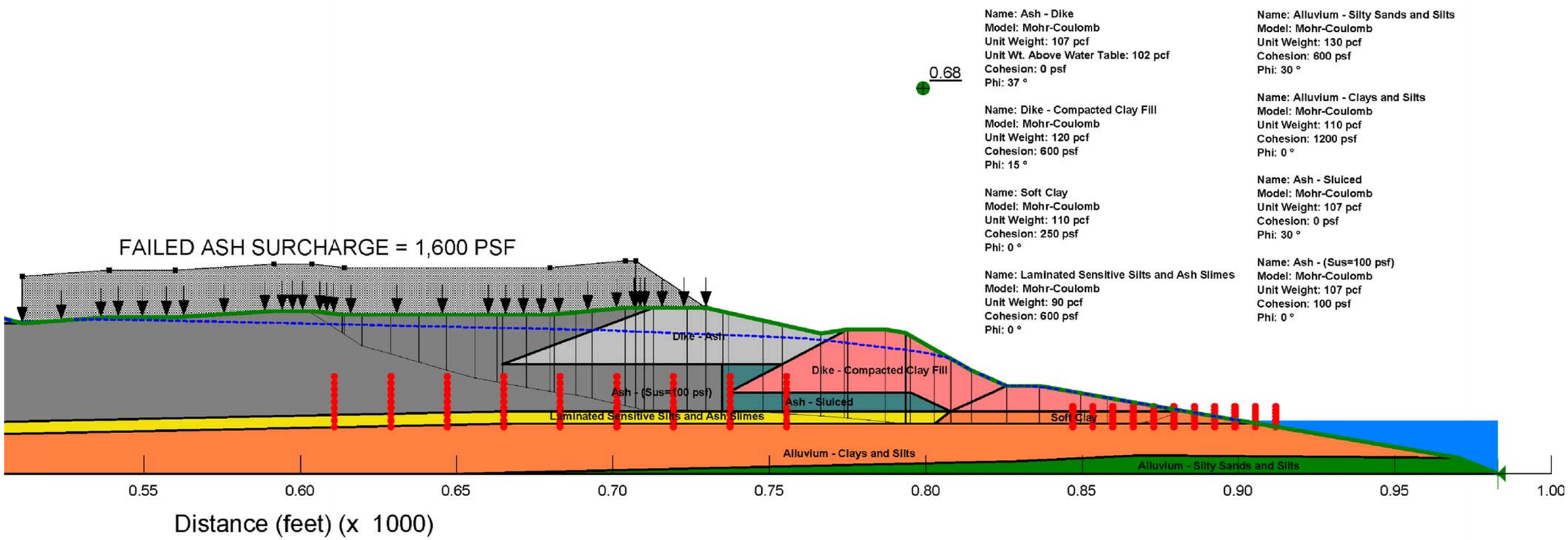
Title: TVA - Kingston RCA
Comments: Stability Analysis for Cell 2 Northwest Section Stage 3 Post-Failure Case 1
Method: Morgenstern-Price
Block Specified Failure Surface



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 3 Post-Failure.gsz

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CELL 2 NW SECTION STAGE 3 POST FAILURE CASE 1
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_7

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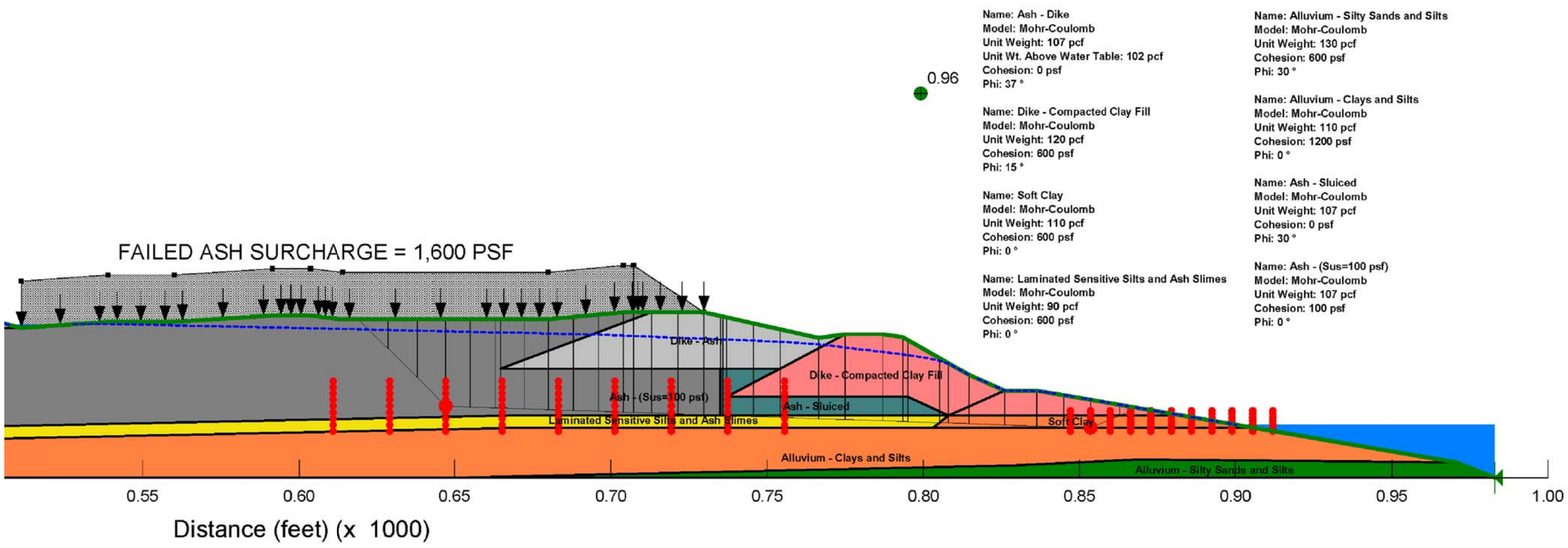
Title: TVA - Kingston RCA
Comments: Stability Analysis for Cell 2 Northwest Section Stage 3 Post-Failure Case 2
Method: Morgenstern-Price
Block Specified Failure Surface



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 3 Post-Failure Case 2.gsz

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CELL 2 NW SECTION STAGE 3 POST FAILURE CASE 2
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_8

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Title: TVA - Kingston RCA
Comments: Stability Analysis for Cell 2 Northwest Section Stage 3 Pre-Failure Case 1
Method: Morgenstern-Price
Block Specified Failure Surface

Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 250 psf
Phi: 0 °

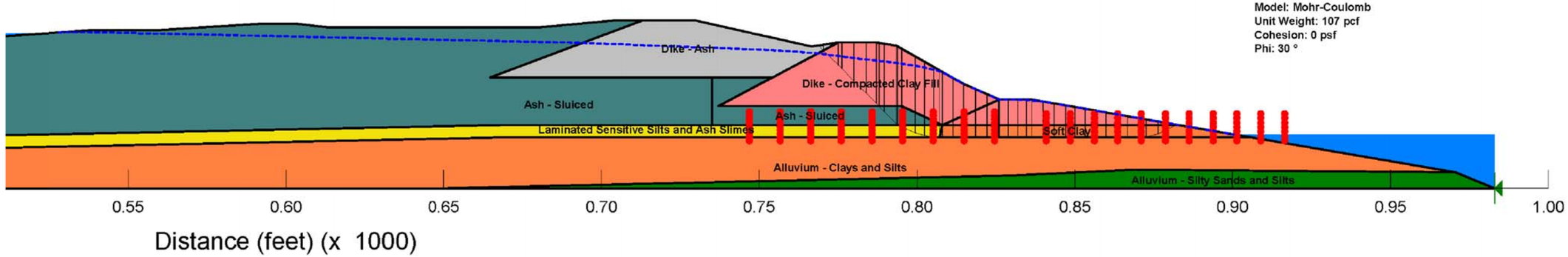
1.24

Name: Laminated Sensitive Silts and Ash Slimes
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 3 Pre-Failure Case 1.gsz

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CELL 2 NW SECTION STAGE 3 PRE FAILURE CASE 1
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

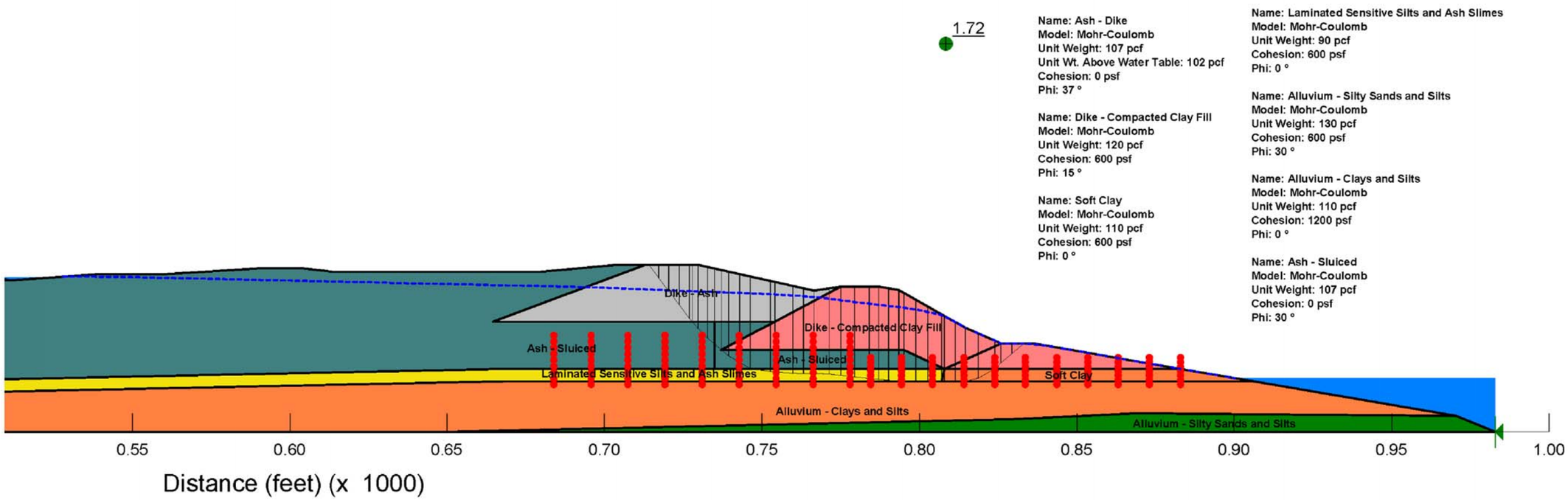
Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_9

M:\PROJECTS\60095742\DWG\slope_stability-figs\G60095742_4C_9-CELL 2 NW SECTION STAGE 3 PRE FAILURE CASE1.dwg; 6/12/2009 1:33:03 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Stability Analysis for Cell 2 Northwest Section Stage 3 Pre-Failure Case 2
Method: Morgenstern-Price
Block Specified Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °

Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 3 Pre-Failure Case 2.gsz

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CELL 2 NW SECTION STAGE 3 PRE FAILURE CASE 2
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009
PROJECT NUMBER	60095742	
FIGURE NUMBER	4C_10	

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Title: TVA - Kingston RCA
Comments: Stability Analysis for Cell 2 Northwest Section Stage 3 Pre-Failure Case 1 Circular
Method: Morgenstern-Price
Block Specified Failure Surface

Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Dike - Compacted Clay Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 600 psf
Phi: 15 °

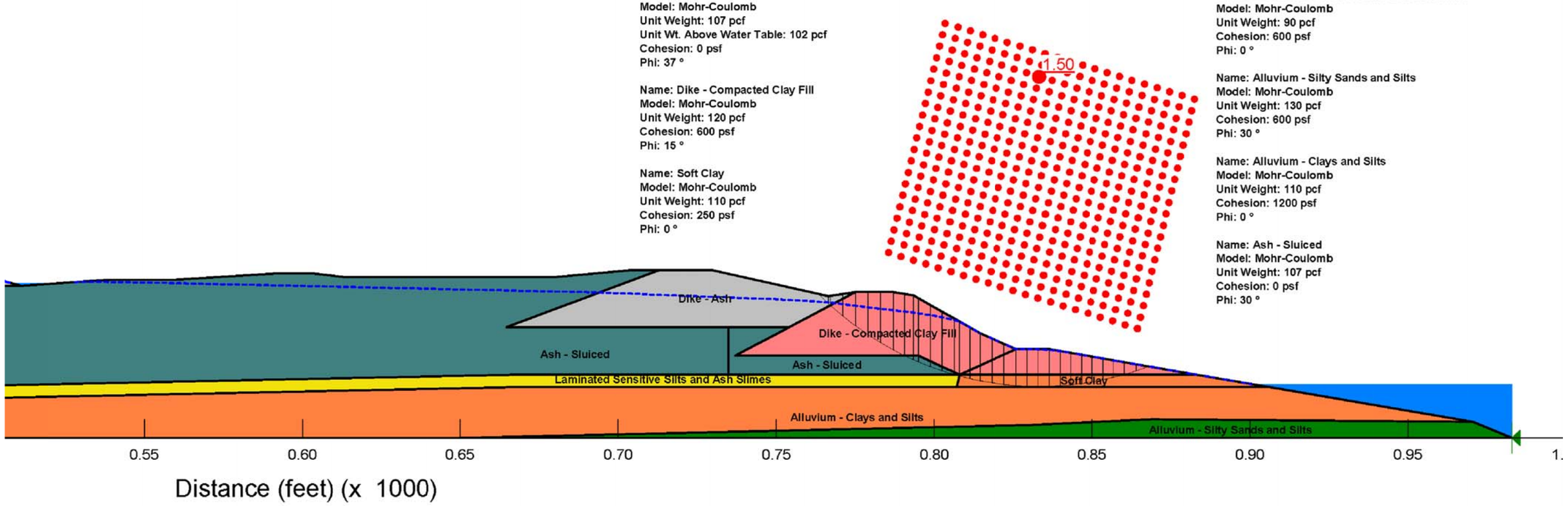
Name: Soft Clay
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 600 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 3 Pre-Failure Case 1 Circular.gsz

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CELL 2 NW SECTION STAGE 3 PRE FAILURE CIRCULAR
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_11

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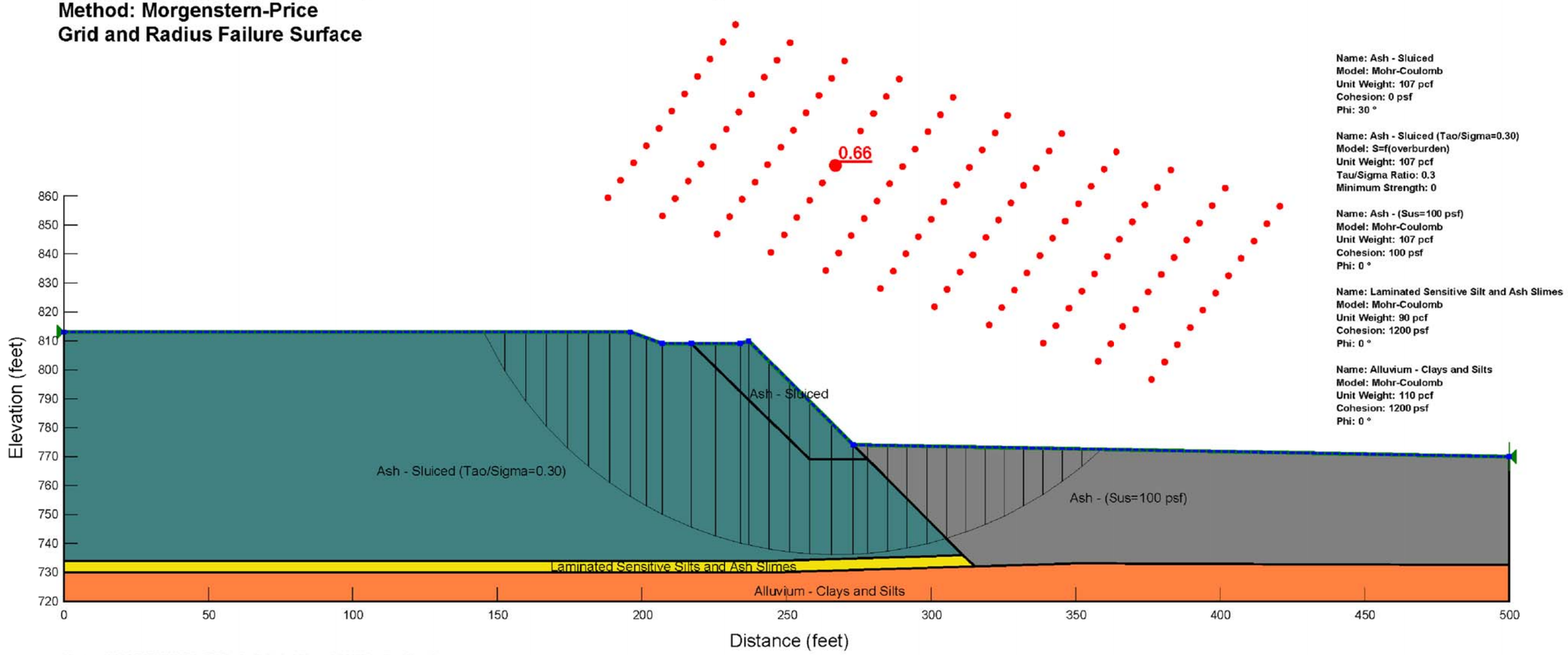
Title: TVA - Kingston RCA
Comments: Slope Stability Analysis For Cell 2 Northwest Section Stage 4
Method: Morgenstern-Price
Grid and Radius Failure Surface



Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °

Name: Ash - Sluiced (Tao/Sigma=0.30)
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Ash - (Sus=100 psf)
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 100 psf
Phi: 0 °

Name: Laminated Sensitive Silt and Ash Slimes
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 NW Section - Stage 4.gsz

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CELL 2 NW SECTION STAGE 4 - CIRCULAR
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_12

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Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Southwest Section Case 1
Method: Morgenstern-Price
Analysis Sluiced Ash Drained Condition
Block Specified Failure Surface

Name: Ash - Dike & Dike B2
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °

Name: Dike B1 & Swan Pond Road
Model: Mohr-Coulomb
Unit Weight: 114 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Railroad Shale Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

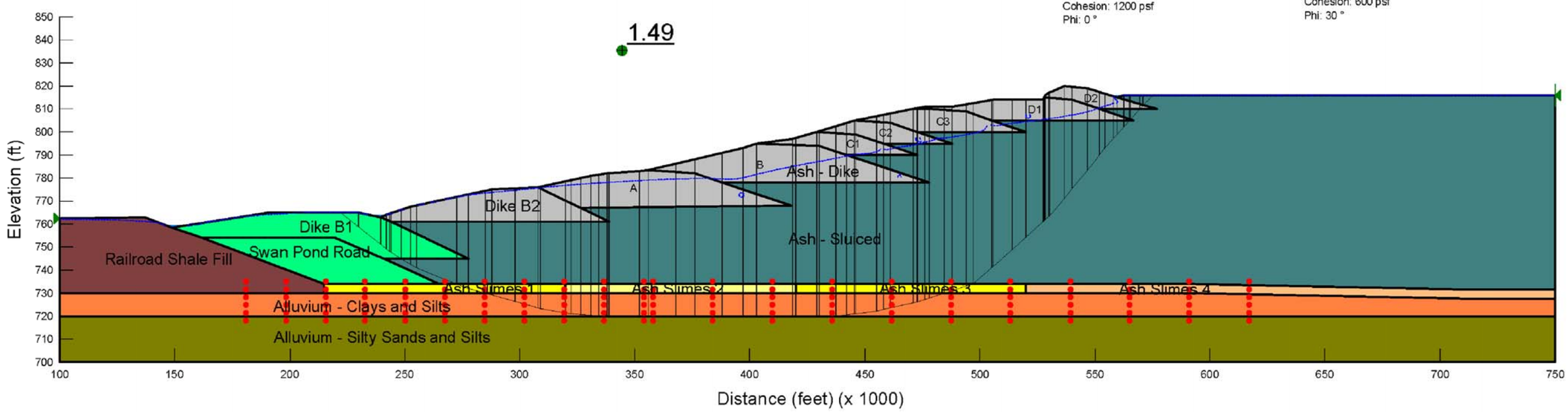
Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1100 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1175 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 SW Section Case 1.gsz

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CELL 2 SW SECTION CASE 1
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_13

M:\PROJECTS\60095742\DWG\Seepage Figures\VOL_4\G60095742_4C_13-Cell 2 SW Section Case 1.dwg: 6/12/2009 2:32:21 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Cell 2 Southwest Section Case 2
Method: Morgenstern-Price
Analysis Sluiced Ash Undrained Condition
Block Specified Failure Surface

Name: Ash - Dike & Dike B2
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Ash - Sluiced
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Dike B1 & Swan Pond Road
Model: Mohr-Coulomb
Unit Weight: 114 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Railroad Shale Fill
Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

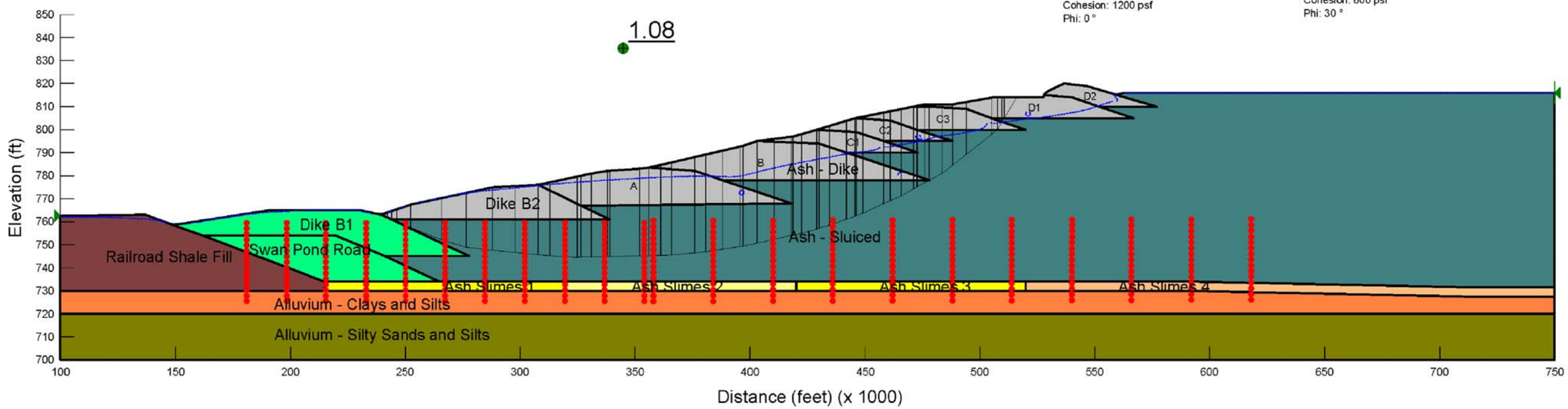
Name: Laminated Sensitive Silts and Ash Slimes 1
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1100 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 2
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1175 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 3
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1250 psf
Phi: 0 °

Name: Laminated Sensitive Silts and Ash Slimes 4
Model: Mohr-Coulomb
Unit Weight: 90 pcf
Cohesion: 1400 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °



Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Cell 2 SW Section Case 2_r.gsz

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CELL 2 SW SECTION CASE 2
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009
PROJECT NUMBER	60095742	
FIGURE NUMBER	4C_14	

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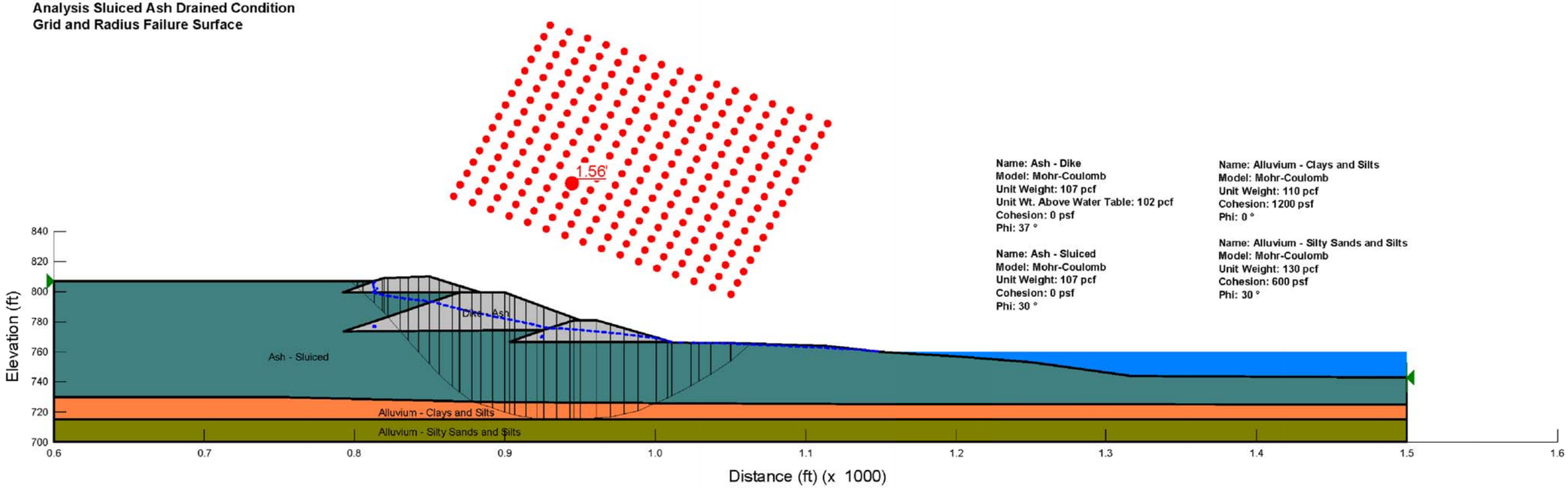
Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Phase 1 Emergency Dredge Cell East Section Case 1
Method: Morgenstern-Price
Analysis Sluiced Ash Drained Condition
Grid and Radius Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Ash - Sluiced
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Cohesion: 0 psf
Phi: 30 °

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Phase 1 Emergency Dredge Cell East Section Case 1_r1.gsz

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PHASE 1 EAST SECTION CASE 1
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

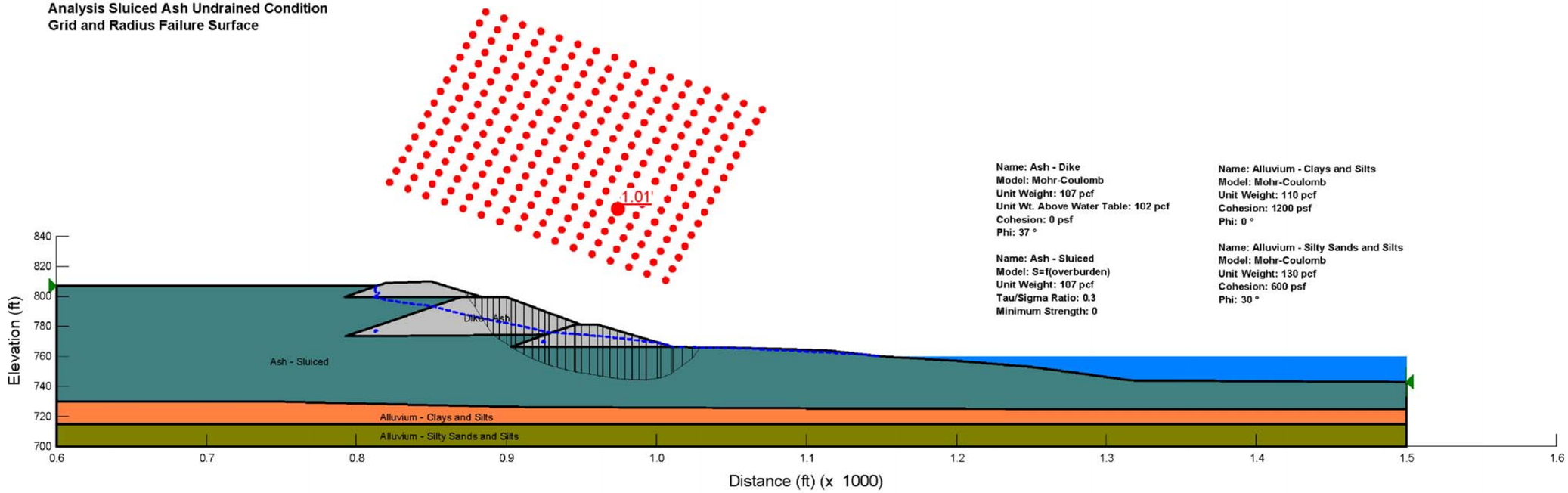
Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_15

M:\PROJECTS\60095742\DWG\slope_stability-figs\G60095742_4C_15-Phase 1 East Section Case 1.dwg; 6/12/2009 8:27:22 PM; BRAATZ, DAN T.; STS.stb

Title: TVA - Kingston RCA
Comments: Slope Stability Analysis for Phase 1 Emergency Dredge Cell East Section Case 2
Method: Morgenstern-Price
Analysis Sluiced Ash Undrained Condition
Grid and Radius Failure Surface



Name: Ash - Dike
Model: Mohr-Coulomb
Unit Weight: 107 pcf
Unit Wt. Above Water Table: 102 pcf
Cohesion: 0 psf
Phi: 37 °

Name: Ash - Sluiced
Model: S=f(overburden)
Unit Weight: 107 pcf
Tau/Sigma Ratio: 0.3
Minimum Strength: 0

Name: Alluvium - Clays and Silts
Model: Mohr-Coulomb
Unit Weight: 110 pcf
Cohesion: 1200 psf
Phi: 0 °

Name: Alluvium - Silty Sands and Silts
Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion: 600 psf
Phi: 30 °

Directory: K:\PROJECTS\60095742 - TVA\GeoStudio Analysis\Name: Phase 1 East Section Case 2_r.gsz

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PHASE 1 EAST SECTION CASE 2
SLOPE STABILITY ANALYSES
TVA-KINGSTON
DREDGE CELL FAILURE RCA
HARRIMAN, TENNESSEE

Analytics :	PCC	06/05/2009
Checked:	LWB	06/05/2009
Approved:	WHW	06/05/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_16

M:\PROJECTS\60095742\DWG\slope_stability-figs\G60095742_4C_16-Phase 1 East Section Case 2.dwg; 6/12/2009 9:47:54 PM; BRAATZ, DAN T.; STS.stb

STS | AECOM

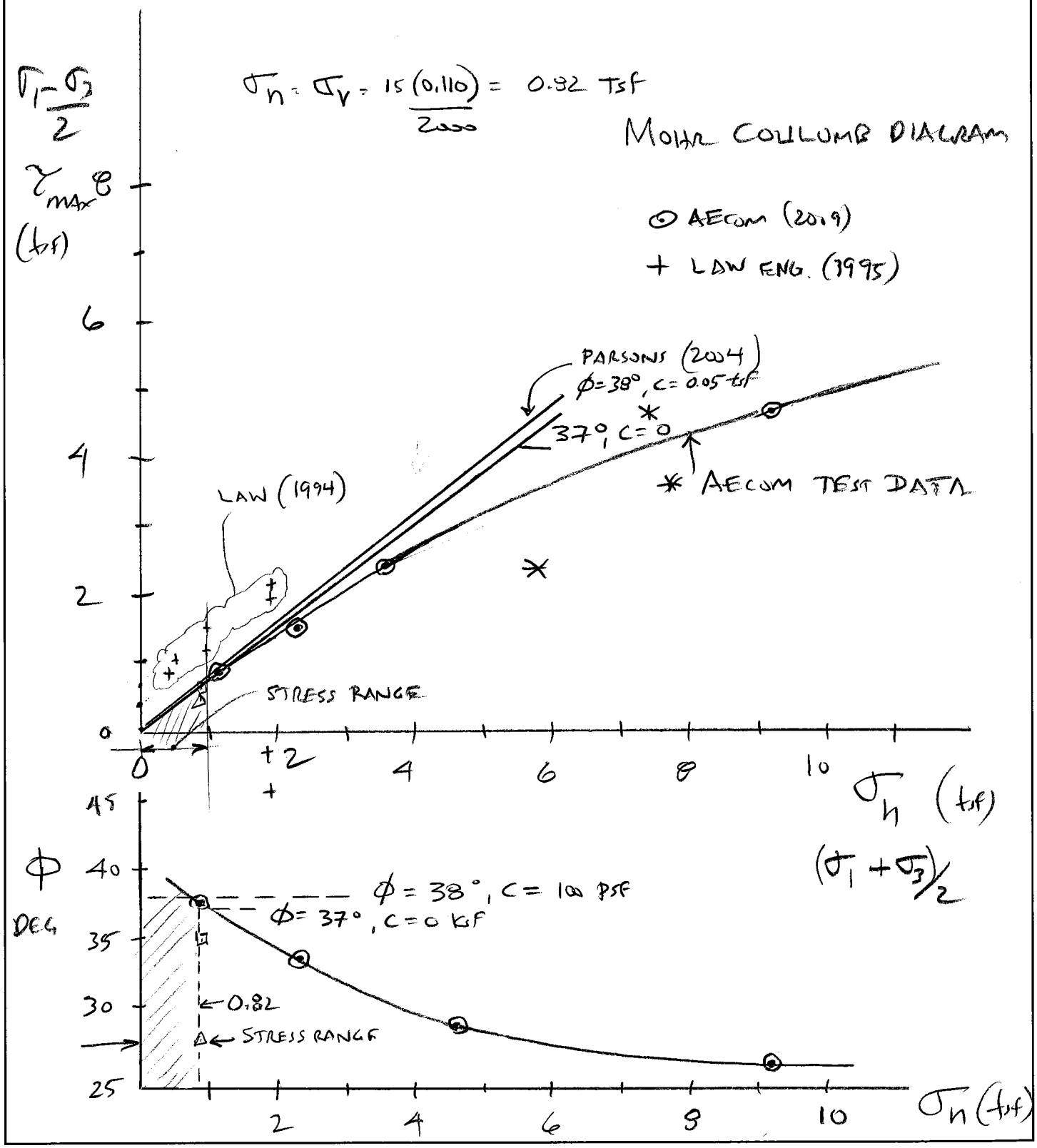
Calculation Sheet

Project	Subject					
KINGSTON	ASH STRENGTHS					
Originated By: [illegible]	Date: 4/28/09	Checked By: EED	Date: 6/2/09	STS Job No.: 60095742	Scale: NTS	Sheet No. 1 of 2

RECONSTITUTED FLYASH FROM 200 SERIES

$\sigma_n = \sigma_v = \frac{15(0.110)}{2000} = 0.82$ TSF

MOHR COULUMB DIAGRAM



STS | AECOM

Calculation Sheet

Project	KINGSTON			Subject	ASH DATA AECOM/LAW		
Originated By	Date 4/20/09	Checked By EED	Date 6/2/09	STS Job No. 60095742	Scale NTS	Sheet No. 2 Of 2	

DIRECT SHEAR TESTS BY AECOM

BORING/SAMPLE	USCS	τ_{max}	σ_n	ϕ	e_f	e_i	G_s	PI
09-102 ST 2A	ASH	0.451	0.86	27.6	0.65	0.69	2.31	NP
RECONSTITUTED ASH	ML	0.89	1.15	37.6	0.94	1.00	2.33	NP
" 200	"	4.66	9.21	26.8	1.21	1.37	2.33	NP
"	"	1.53	2.30	33.6	0.698	0.75	2.33	NP
"	"	2.47	4.61	27.8	1.58	1.74	2.33	NP

LAW ENGINEERING

	lost (top)						
CELL I	1.67 (0.82)	0.97	0.49	59°	0.94	2.29	NP
	2.37 (1.18)	1.94	0.92	51°	0.95	"	"
	3.99 2.0	3.88	1.94	46°	0.90	"	"
CELL III	2.11 (1.06)	0.97	(0.49)	65°	0.91	2.31	
	3.13 (1.57)	1.94	(0.97)	58°	0.93	2.31	
	4.4 (2.2)	3.88	(1.94)	48.6°	0.95	2.31	

Plotted, pg 1 of 2

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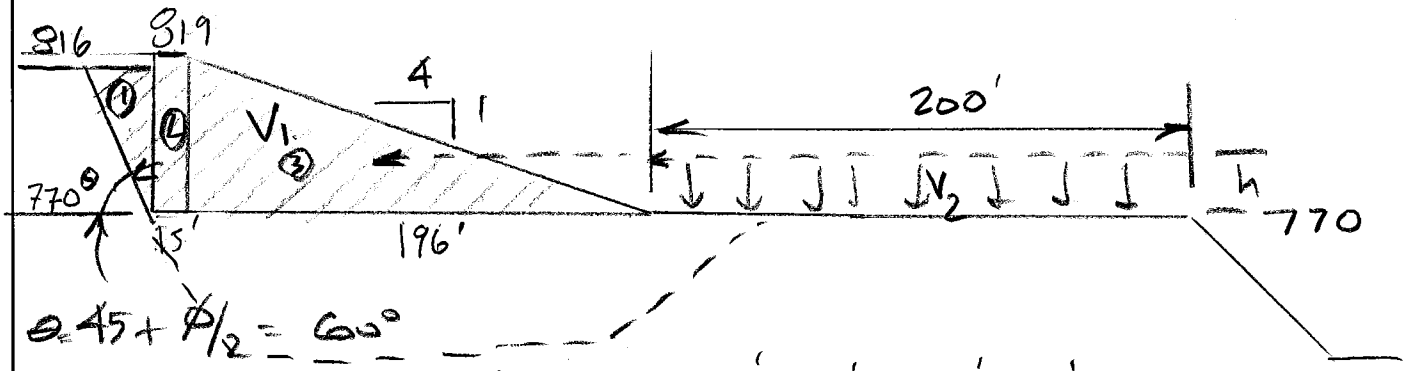
Calculation Sheet

Project	KINGSTON – RCA			Subject	DISPLACED VOLUME OF FILL		
Originated By	Date	Checked By	Date	AECOM Job No.	Scale		Sheet No. 1 Of 1
[illegible]	5/3/09	[illegible]	5/4/09	60095742	N.T.S.		

$\phi = 30°$



$\theta = 45 + \phi/2 = 60°$

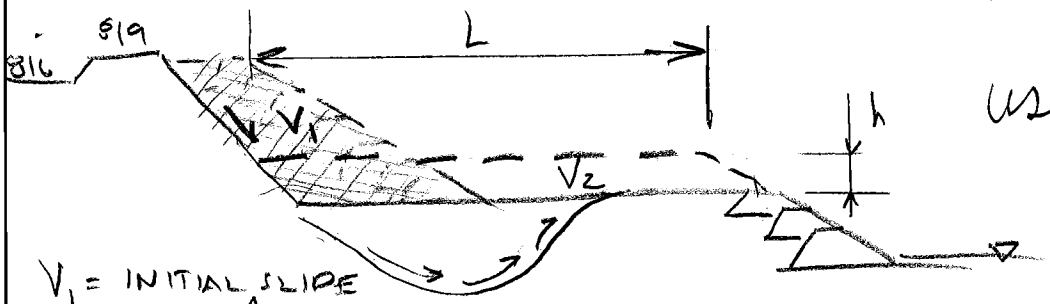
$$h(200') = \frac{196(49)}{2} + 15(49) + 46(26.6)/2$$

819'-770' 819'-770' 816'-770' 46'·cot 60°

$h = \text{VOLUME}/L$

$h = (4802 + 735 + 610)/200' = 31'$ MAX.

$h = (4802 + 735 + 610)/(200 + 196 + 15) = 15'$ MIN.



use 15'

V_1 = INITIAL SLIDE MASS ABOVE EL. 770

V_2 = DISPLACED VOLUME UNCHANGED SINCE FAILURE IS UNDRAINED

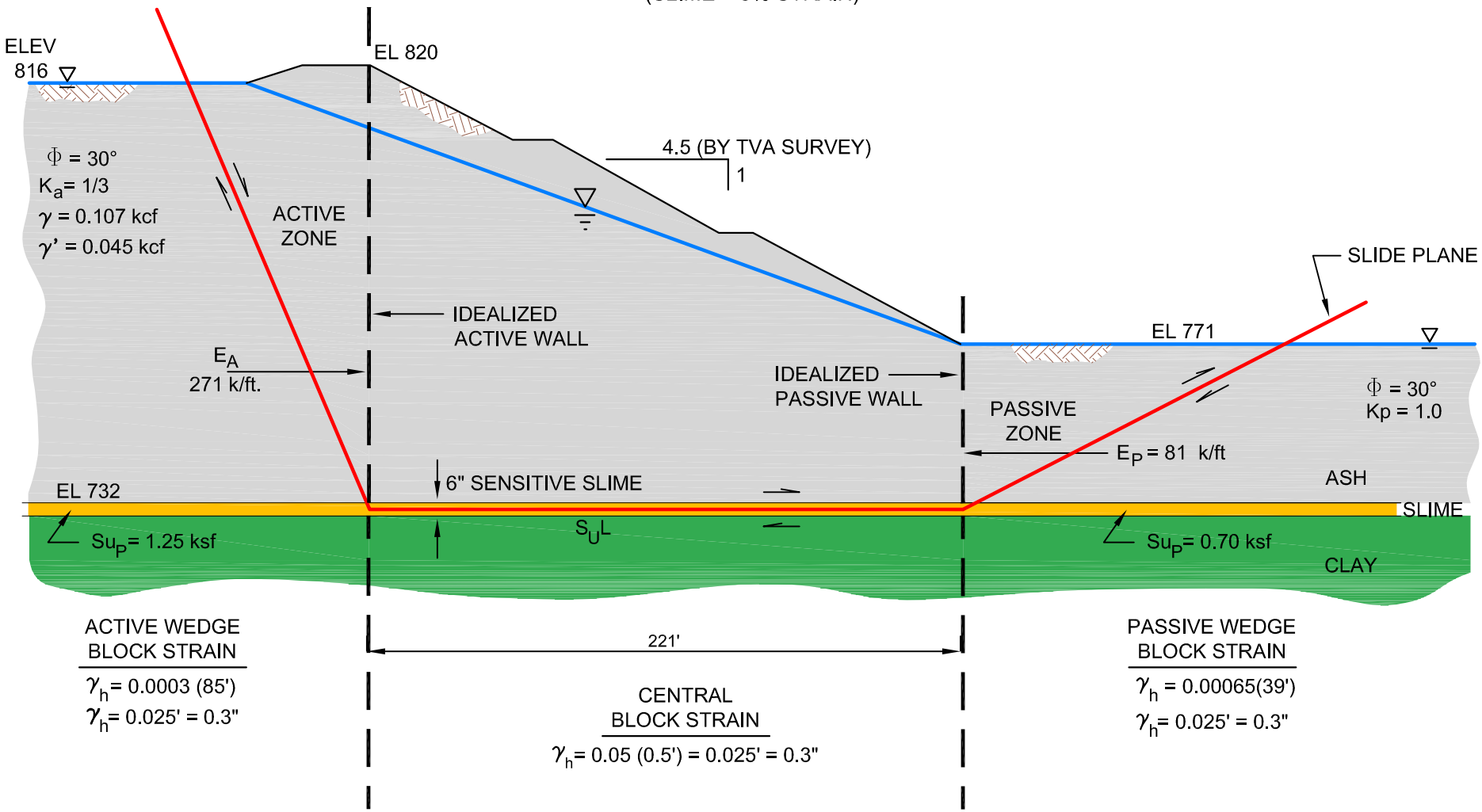
GIVEN: $V_1 = V_2$

$\sigma_v = h\,\gamma_{ASH}$ 15'(107 pcf) = 1,600 psf FOR SLOPE/W RUNS

CELL 2 NORTHWEST SECTION
WEDGE BLOCK STABILITY ANALYSIS

BASED ON DISPLACEMENT COMPATIBILITY
(SLIME = 5% STRAIN)

AECOM



ACTIVE WEDGE BLOCK STRAIN

γ_h= 0.0003 (85')

γ_h= 0.025' = 0.3"

CENTRAL BLOCK STRAIN

γ_h= 0.05 (0.5') = 0.025' = 0.3"

PASSIVE WEDGE BLOCK STRAIN

γ_h = 0.00065(39')

γ_h= 0.025' = 0.3"

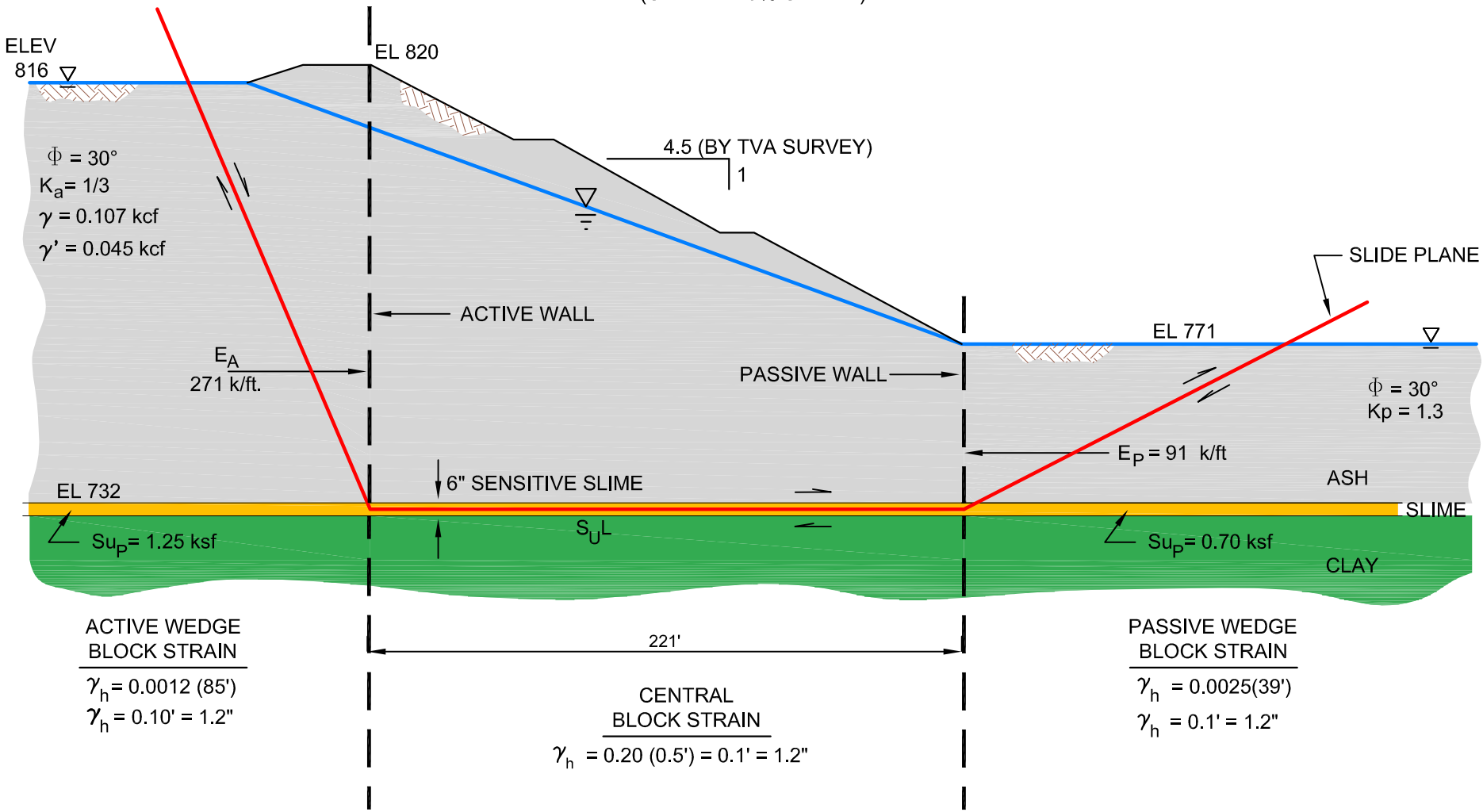
$$FS = \frac{S_u L}{E_A - E_P} = \frac{(1.25 + 0.7)/2\ (221')}{271 - 81} = \frac{215\ \text{k/ft.}}{190\ \text{k/ft.}} = 1.1,\ \textbf{FAILURE IMMINENT}$$

4C_17

CELL 2 NORTHWEST SECTION
WEDGE BLOCK STABILITY ANALYSIS

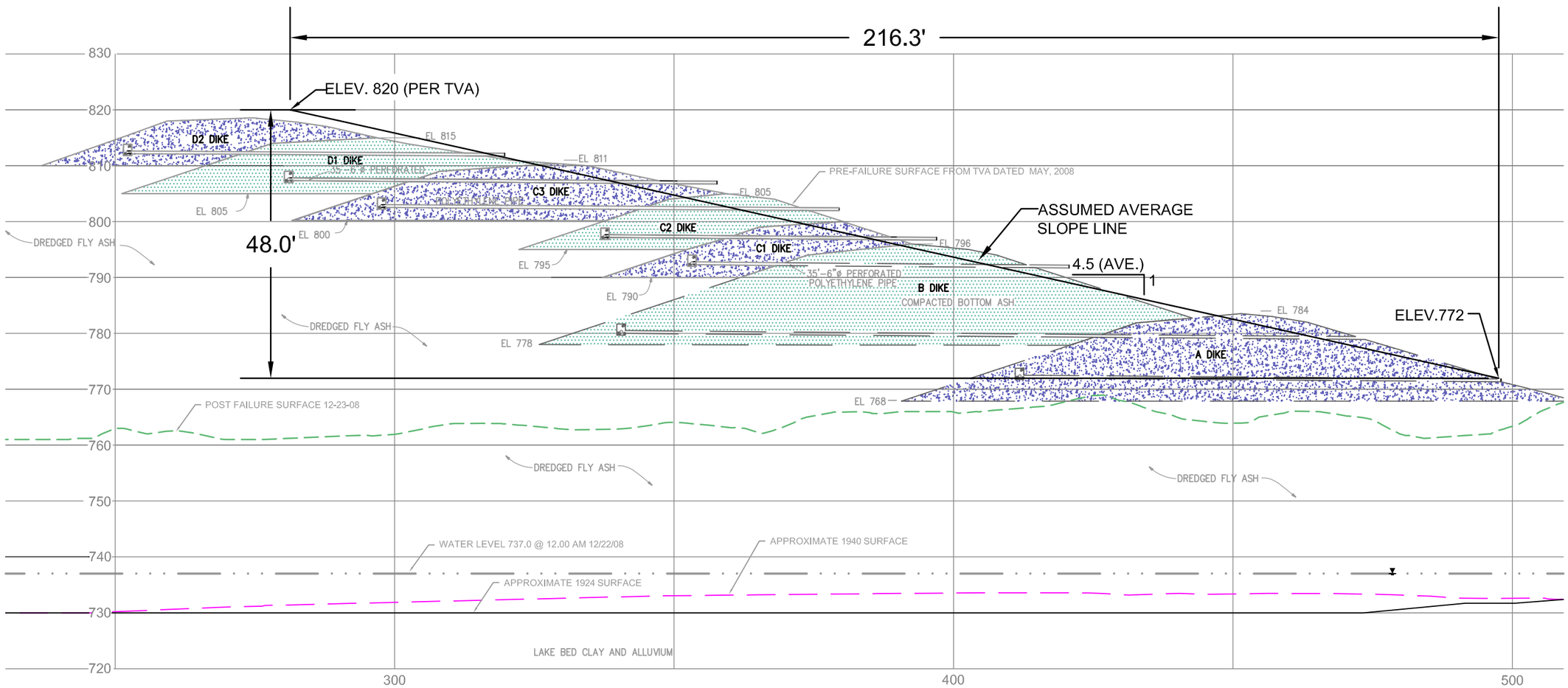
BASED ON DISPLACEMENT COMPATIBILITY
(SLIME = 20% STRAIN)

AECOM



$$FS = \frac{S_u L}{E_A - E_P} = \frac{0.75\ (1.25 + 0.7)/2\ (221')}{271 - 91} = \frac{162\ \text{k/ft.}}{180\ \text{k/ft.}} = 0.9,\ \textbf{FAILURE}$$

4C_18



AVERAGE SLOPE SECTION J-J

0 20'

1" = 20'

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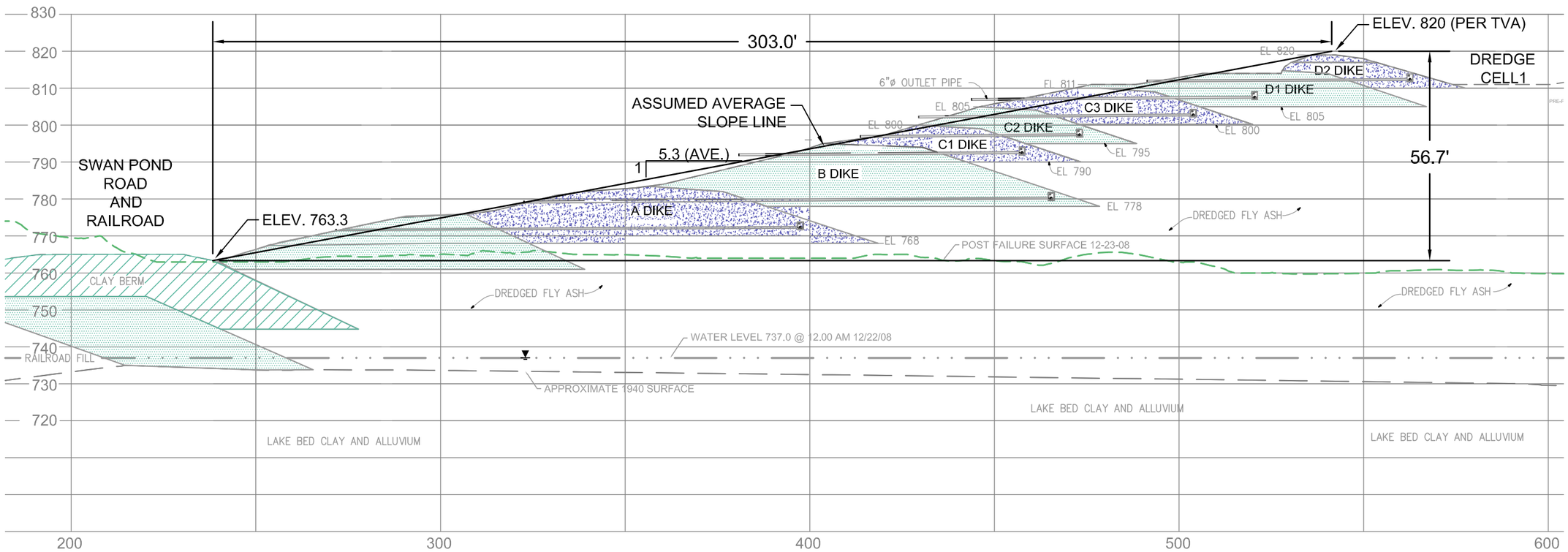
SECTION J- J CELL 2 NORTHWEST- WEDGE BLOCK ANALYSIS
ROOT CAUSE ANALYSIS
TVA KINGSTON DREDGE CELL FAILURE
ON DECEMBER 22, 2008
KINGSTON FOSSIL PLANT
HARRIMAN, TENNESSEE

Drawn :	DTB	6/12/2009
Checked:	LWB	6/12/2009
Approved:	WHW	6/12/2009
PROJECT NUMBER	60095742	
FIGURE NUMBER	4C_19	

M:\PROJECTS\60095742\DWG\G60095742-4C_19-SLOPE-J-J.dwg; 6/12/2009 2:05:33 PM; BRAATZ, DAN T.; STS.stb



AVERAGE SLOPE SECTION H-H

0 30'

1" = 30'

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SECTION H-H CELL 2 SOUTHWEST- WEDGE BLOCK ANALYSIS

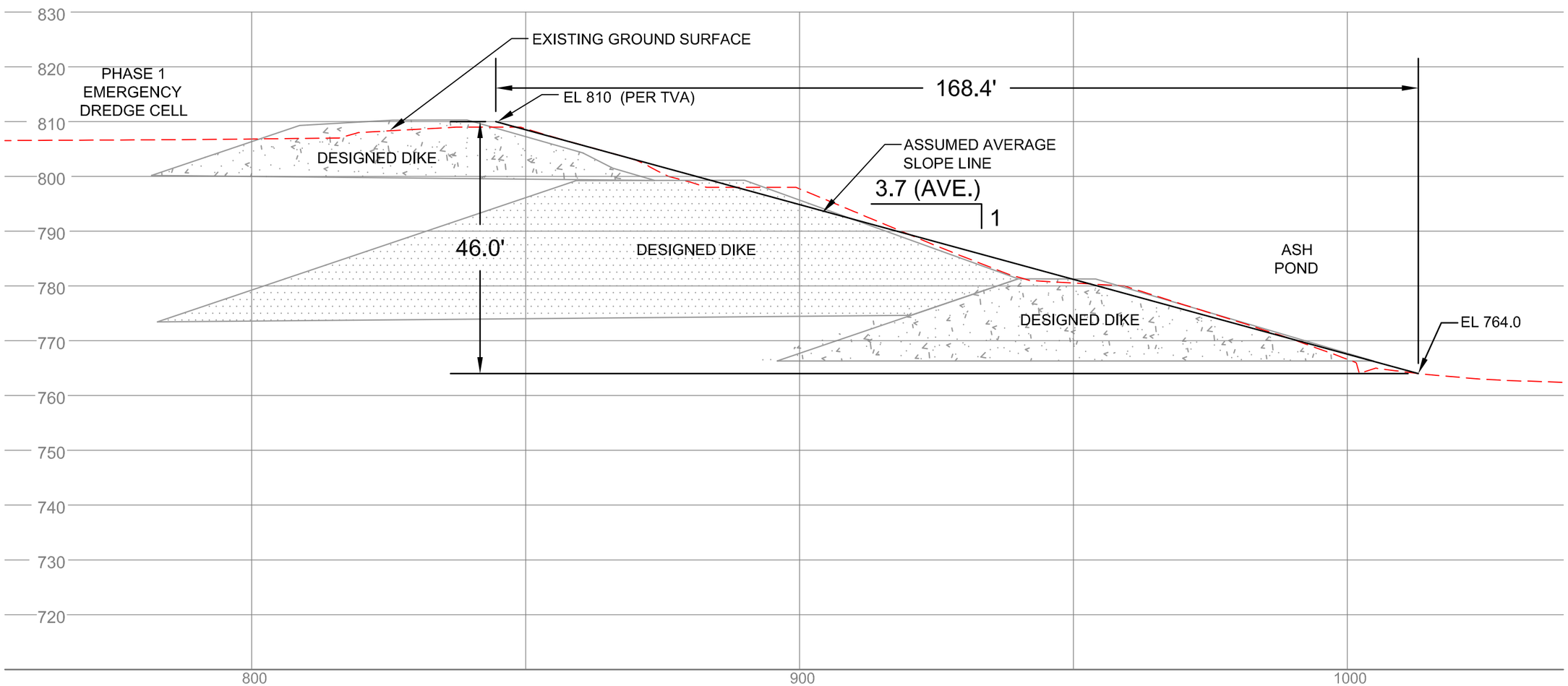
ROOT CAUSE ANALYSIS
TVA KINGSTON DREDGE CELL FAILURE
ON DECEMBER 22, 2008
KINGSTON FOSSIL PLANT
HARRIMAN, TENNESSEE

Drawn :	DTB	6/12/2009
Checked:	LWB	6/12/2009
Approved:	WHW	6/12/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_20

M:\PROJECTS\60095742\DWG\G60095742-4C_20-SLOPE-SECTION-H-H.dwg; 6/12/2009 2:08:51 PM; BRAATZ, DAN T.; STS.stb



AVERAGE SLOPE SECTION K-K

0 20'

1" = 20'

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PHASE 1 EMERGENCY CELL EAST - WEDGE BLOCK ANALYSIS
SECTION K-K CELL 1 EAST DIKE
ROOT CAUSE ANALYSIS
TVA KINGSTON DREDGE CELL FAILURE
ON DECEMBER 22, 2008
KINGSTON FOSSIL PLANT
HARRIMAN, TENNESSEE

Drawn :	DTB	6/12/2009
Checked:	LWB	6/12/2009
Approved:	WHW	6/12/2009

PROJECT NUMBER 60095742

FIGURE NUMBER 4C_21

M:\PROJECTS\60095742\DWG\G60095742-4C_21-SLOPE-SECTION-K-K-.dwg; 6/12/2009 2:12:32 PM; BRAATZ, DAN T.; STS.stb

Calculation Sheet

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
[illegible]	5/9/09	SSD	6/2/09	60095741	NTS	1 of 1

Project: KINGSTON RCA

Subject: WEDGE BLOCK STABILITY

WEDGE BLOCK CELL 2 NORTH STABILITY ANALYSIS

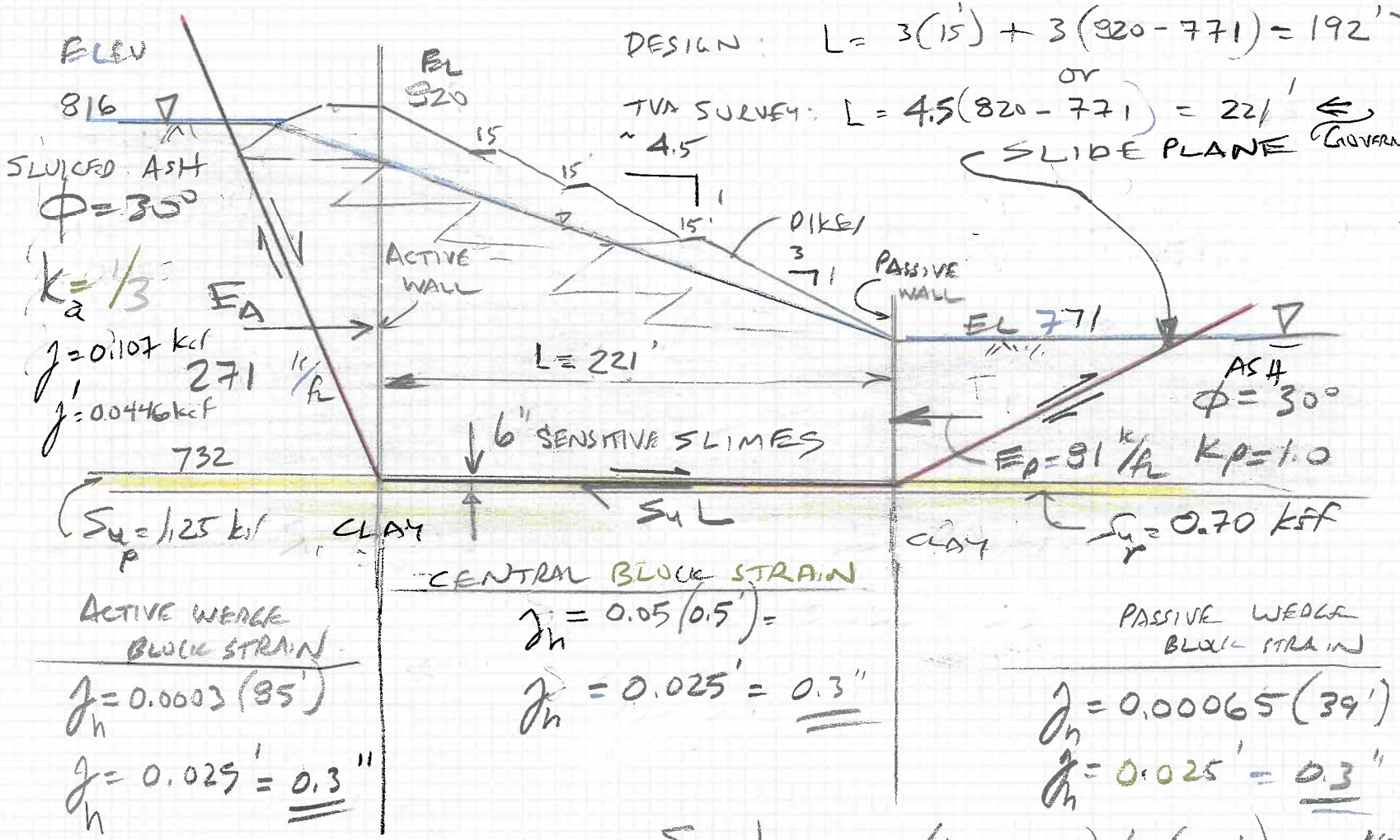
52 STRAIN COMPATIBILITY ACHIEVED

DESIGN: $L = 3(15') + 3(920 - 771) = 192'$

OR

TVA SURVEY: $L = 4.5(820 - 771) = 221'$ ⇐ GOVERNS

SLIDE PLANE



ACTIVE WEDGE BLOCK STRAIN

$\gamma_h = 0.0003(85')$

$\gamma_h = 0.025' = \underline{0.3''}$

CENTRAL BLOCK STRAIN

$\gamma_h = 0.05(0.5') =$

$\gamma_h = 0.025' = \underline{0.3''}$

PASSIVE WEDGE BLOCK STRAIN

$\gamma_h = 0.00065(39')$

$\gamma_h = 0.025' = \underline{0.3''}$

$$FS = \frac{S_u L}{E_A - E_p} = \frac{(1.25 + 0.7)/2\,(221')}{271 - 81} = \frac{215.5\ k/ft}{190\ k/ft} = \underline{1.1}$$

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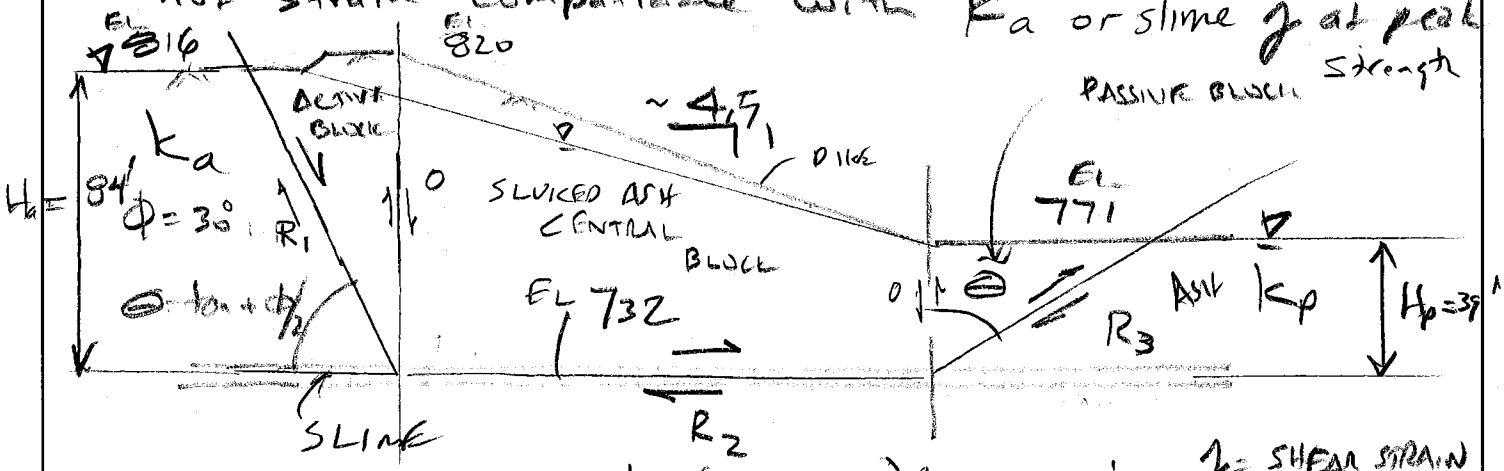
Calculation Sheet

Project	Subject
KINGSTON RCA	WEDGE BLOCK STABILITY

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
[illegible]	5/7/09	EED	6/2/09	60095742	NTI	2 of 7

Review potential horizontal strains to develop k_a on driving block that pushes on central block founded on sensitive slime, and resisted by partially mobilized k_p resistance block. Full k_p is not strain compatible with k_a or slime γ at peak strength



$L = (820 - 771)4.5 = 221'$

γ = SHEAR STRAIN

$$FS = \frac{R_2}{R_1 - R_3}$$

SHEAR STRAIN GOAL

$\gamma_A = \gamma_C = \gamma_P$

$\phi = 30°$

$k_a = \frac{1 - \sin\phi}{1 + \sin\phi} = 1/3$ $(\gamma = 0.001)$, $k_p = 3$ FULL $(\gamma = 0.06)$

$k_o = 1 - \sin\phi = 1/2$

FULL k_p NOT STRAIN COMPATIBLE

BASED ON NAVFAC DM 7.2, FIG 1, PAGE 7.2-60

LOOSE COHESIONLESS: ROTATION γ/H TO ACHIEVE FAILURE STRAIN IN SENSITIVE SLIMES

ACTIVE	k_A (γ/H_A)	PASSIVE (γ/H_P)	k_p	$\Delta h/H$
0.0003	0.33	0.0065	1.0	5%
0.0012	0.28	0.0025	1.3	20%
0.004	0.25	0.06	3.0	∞

STS | AECOM

Calculation Sheet

CELL 2 NW

Project	Subject					
KINGSTON RCA	WEDGE BLOCK STABILITY					
Originated By: GHW	Date: 5/7/09	Checked By: EED	Date: 6/2/09	STS Job No.: 60095742	Scale: NTS	Sheet No. 3 of 7

CENTRAL BLOCK 6" THICK LAYER OF SENSITIVE SLIME

γ_h MAX SHEAR STRAIN = 5% × Δh = 0.05 × 6" = 0.3"

ACTIVE STRAIN (H_A = 84')

		Δh	K_A
$\frac{Y}{H} = \gamma_h$ =	0.0003 in/in × 84' × 12"/1' =	0.030"	1/3
		0.3"	1/3
$\frac{Y}{H} = \gamma_h$ =	0.0012 in/in × 84' × 12"/1' =	1.2"	<1/3
$\frac{Y}{H} = \gamma_h$ =	0.004 in/in × 84' × 12"/1' =	4.00"	<1/3

PASSIVE STRAIN (H = 39')

		Δh	K_p
$\frac{Y}{H} = \gamma_h$ =	0.00065 × 39' × 12" =	0.30"	0.9 use 1.0
$\frac{Y}{H} = \gamma_h$ =	0.0025 × 39' × 12"/1' =	1.17"	1.3
$\frac{Y}{H} = \gamma_h$ =	0.006 × 39' × 12"/1' =	2.8"	1.7
$\frac{Y}{H} = \gamma_h$ =	0.06 × 39' × 12"/1' =	28.1"	3.0

SAY 0.3" LATERAL MOVEMENT (ACTIVE & PASSIVE) TO MATCH CKU/DSS UNDRAINED SHEAR STRAIN

$K_A = 1/3$ $K_p \cong 1.0$

$\gamma' = 0.107 - 0.0624$ kcf

$\gamma' = 0.0446$ kcf

$\phi = 30°$ ASH

$\gamma = 0.107$ kcf

$\gamma' = 0.0446$ kcf

$\Delta h = 0.3" \sim 5/16"$

Calculation Sheet

Project	Subject	Originated By	Date	Checked By	Date	AECOM Job No.	Scale	Sheet No.
KINGSTON-RCA	WEDGE BLOCK STABILITY	[illegible]	5/3/09	[illegible]	5/4/09	60095742	NTS	4 of 7

CELL 2 NW

AECOM

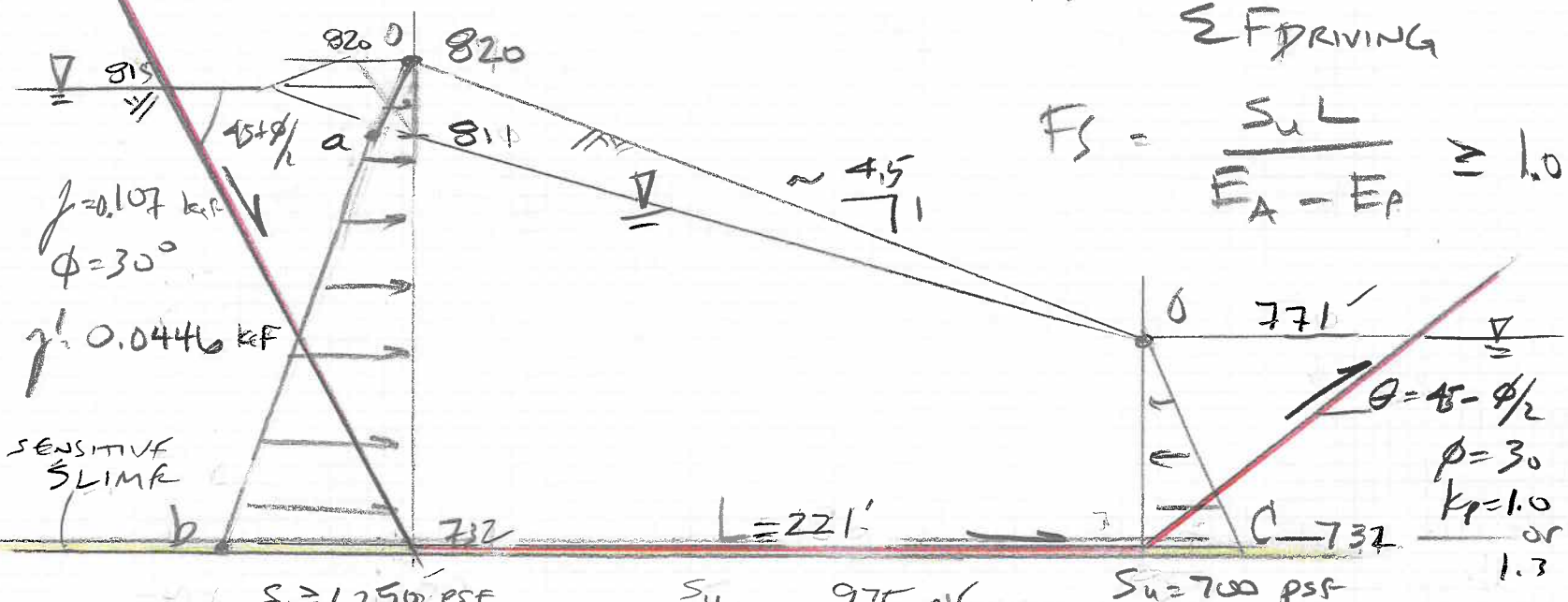
$\phi = 30°$
$K_a = 1/3$
$K_p = 1.0$

(DRAINED CONDITIONS IN ASH)

$\gamma_A \approx \gamma_P$ EQUAL SHEAR STRAINS IN ASH

$$FS = \frac{\Sigma F_{RESISTING}}{\Sigma F_{DRIVING}}$$

$$FS = \frac{S_u L}{E_A - E_P} \geq 1.0$$



$P_a = 9(0.333)(0.107) = 0.321$ (819)

$P_b = 0 + 1/3\,(84)(0.0446) + 84(0.0624) = 6.491$ (1.25, 5.242)

$P_c = 39(1.0)(0.0446)$ kcf $+ 39(0.0624)$ kcf

$E_A = \frac{9(0.321)}{2} + \frac{0.321 + 6.491}{2}(79)$

$E_A = 1.445 + 269.07 = 270.5$ k/ft

STRAIN CHECK

- $\epsilon = 5\%$ for $K_p = 1.0$: $E_p = 81.4$ kip/ft
- $\epsilon = 20\%$ for $K_p = 1.3$: $E_p = \frac{(39(1.3)(0.0446) + 2.43)39}{2}$

$E_p = 91.5$ kip/ft

$E_p = \frac{(1.74 + 2.43)(39)}{2}$

STS | AECOM

Calculation Sheet

CELL 2 NW

Project	KINGSTON RCA			Subject	WEDGE BLOCK STABILITY		
Originated By	Date 5/7/09	Checked By SED	Date 6/2/09	STS Job No. 60095742	Scale NTS		Sheet No. 5 Of 7

- COMPUTE $S_{u\ REQ'D}$

$L = 221'$

$K_A = 1/3 \quad K_P = 1.0$

$\gamma_A = \gamma_P$

$$FS = \frac{\Sigma F_{RESISTING}}{\Sigma F_{DRIVING}} = \frac{S_u L + E_P}{E_A}$$

$$FS = 1.0 = \frac{S_{u\ REQ'D} L + E_P}{E_A}$$

- SOLVE FOR $S_{u\ REQ'D}$

$$S_{req'd} = \frac{E_A - E_P}{L}, \text{ for } FS = 1.0$$

$$S_{req'd} = \frac{270.4\ k/ft - 81.4\ k/ft}{221\ ft} = \frac{189.1}{221} = 0.86\ ksf$$

for FS = 1.0

STATISTICAL ANALYSIS

09-503 1249 psf EL 812.5

09-108 / 09-109 / 09-605 / 09-104 = 700 psf (MEAN)

$S_{u\ AVE} = 1250 + 700 / 2 = 975\ psf = 1.025\ ksf$

CHECK FS (MEAN)

$$\therefore FS = \frac{0.975\ ksf\ (221')}{270.5 - 81.4} = 1.13 \sim 1.1$$

STS | AECOM

Calculation Sheet

CELL 2 NW

Project	KINGSTON RCA			Subject	WEDGE BLOCK STABILITY		
Originated By	Date 5/7/09	Checked By EED	Date 6/2/09	STS Job No. 60095742	Scale NTS	Sheet No. 6 Of 7	

- CHECK FS @ 5% MEAN FOR EL 732 SLIME LAYER

EL. 770 EL 812.5

$$S_u = \frac{700 \text{ psf} + 1250 \text{ psf}}{2} = 0.975 \text{ ksf}$$

$$FS = \frac{0.975(221)}{270.5 - 81.4} = 1.13 \sim 1.1$$

(1.1) FAILURE IMMINENT

- CHECK FS @ 20% SHEAR STRAIN (0.75).

$$S_u = \frac{[700 \text{ psf} + 1250](0.75)}{2} = 0.73 \text{ ksf}$$

$$FS = \frac{0.731(221)}{270.5 - 91.5} = 0.90 < 1.0$$

(0.9) FAILURE

Abramson et al. (1996), Pg. 16

"Peak strengths of the embankment and foundation soils cannot be mobilized simultaneously because of stress-strain incompatibility (Figure 1.7). Hence, a stability analysis using peak strengths of soils would overestimate the factor safety. Many engineers perform stability analyses using soil strengths that are smaller than the peak values to allow for possible progressive failure."

STS | AECOM

Calculation Sheet

CELL 2 NW

Project				Subject		
KINGSTON RCA				WEDGE BLOCK STABILITY		
Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
MAW	5/11/09	EED	6/2/09	60097542	NTS.	7 Of 7

TRY S_{us} AT 20% STRAIN ON UNDISTURBED DSS TESTS

PEAK S_{up} in Slimes	S_u 20% γ_h	DeGroot Test No.	% REDUCTION ON SLIMES
690	530	351	$\frac{530}{690} \times 100 \approx 77$
599	500	352	83
805	660	353	82
779	550	354	71
684	520	355	76
1060	880	356	83
985	700	357	71
NA	SHANSEP	358	OCR = 1
1106	800	359	72
1249	800	360	64

AVE $\frac{\Sigma 679}{9} = 75\%$

CHECK 20% STRAIN IN SLIMES

K

$\Delta h = 0.20 \times 6'' = 1.2''$ CENTRAL BLOCK

$\Delta h = 0.0012 \times 84' \times 12''/1' = 1.2''$ ACTIVE BLOCK $K_A = 1/3$

$\Delta h = 0.0025 \times 39' \times 12''/1' = 1.2''$ PASSIVE BLOCK $K_p = 1.3$

William Walton

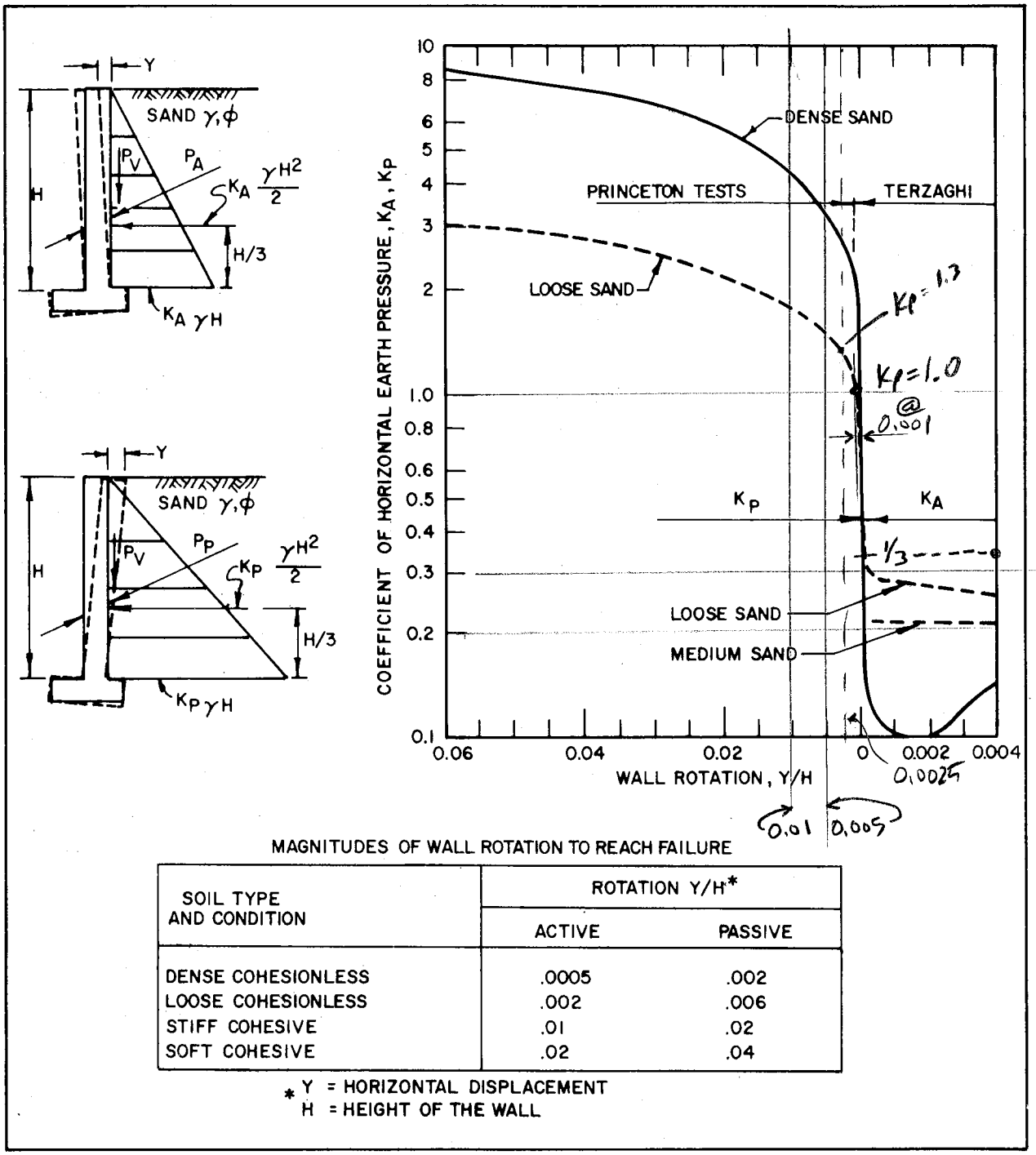
NAVFAC DM-7.2
MAY 1982

APPROVED FOR PUBLIC RELEASE

FOUNDATIONS AND EARTH STRUCTURES

DESIGN MANUAL 7.2

DEPARTMENT OF THE NAVY
NAVAL FACILITIES ENGINEERING COMMAND
200 STOVALL STREET
ALEXANDRIA, VA. 22332



MAGNITUDES OF WALL ROTATION TO REACH FAILURE

SOIL TYPE AND CONDITION	ROTATION Y/H*	
	ACTIVE	PASSIVE
DENSE COHESIONLESS	.0005	.002
LOOSE COHESIONLESS	.002	.006
STIFF COHESIVE	.01	.02
SOFT COHESIVE	.02	.04

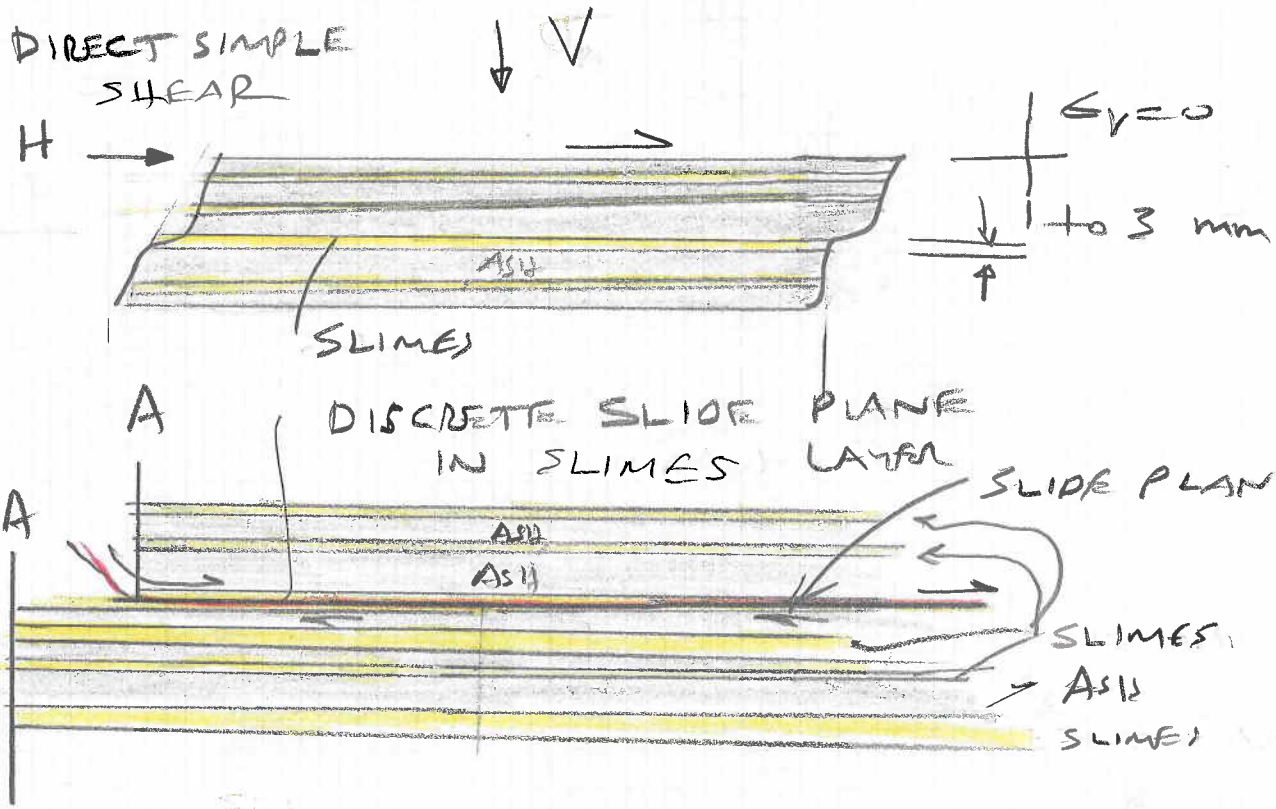
* Y = HORIZONTAL DISPLACEMENT
H = HEIGHT OF THE WALL

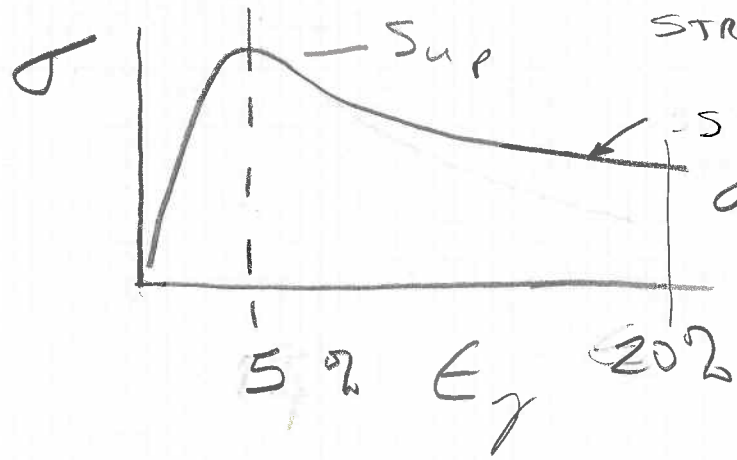
FIGURE 1
Effect of Wall Movement on Wall Pressures

AECOM

Calculation Sheet

Project	KINGSTON – RCA			Subject	SLIDING POTENTIAL IN SLIMES		
Originated By	Date 5/3/09	Checked By	Date 5/4/09	AECOM Job No.	Scale NTS	Sheet No. 1 Of 1	





$$\frac{25.4\,c}{1\,in} = \frac{0.075}{x\ inch}$$

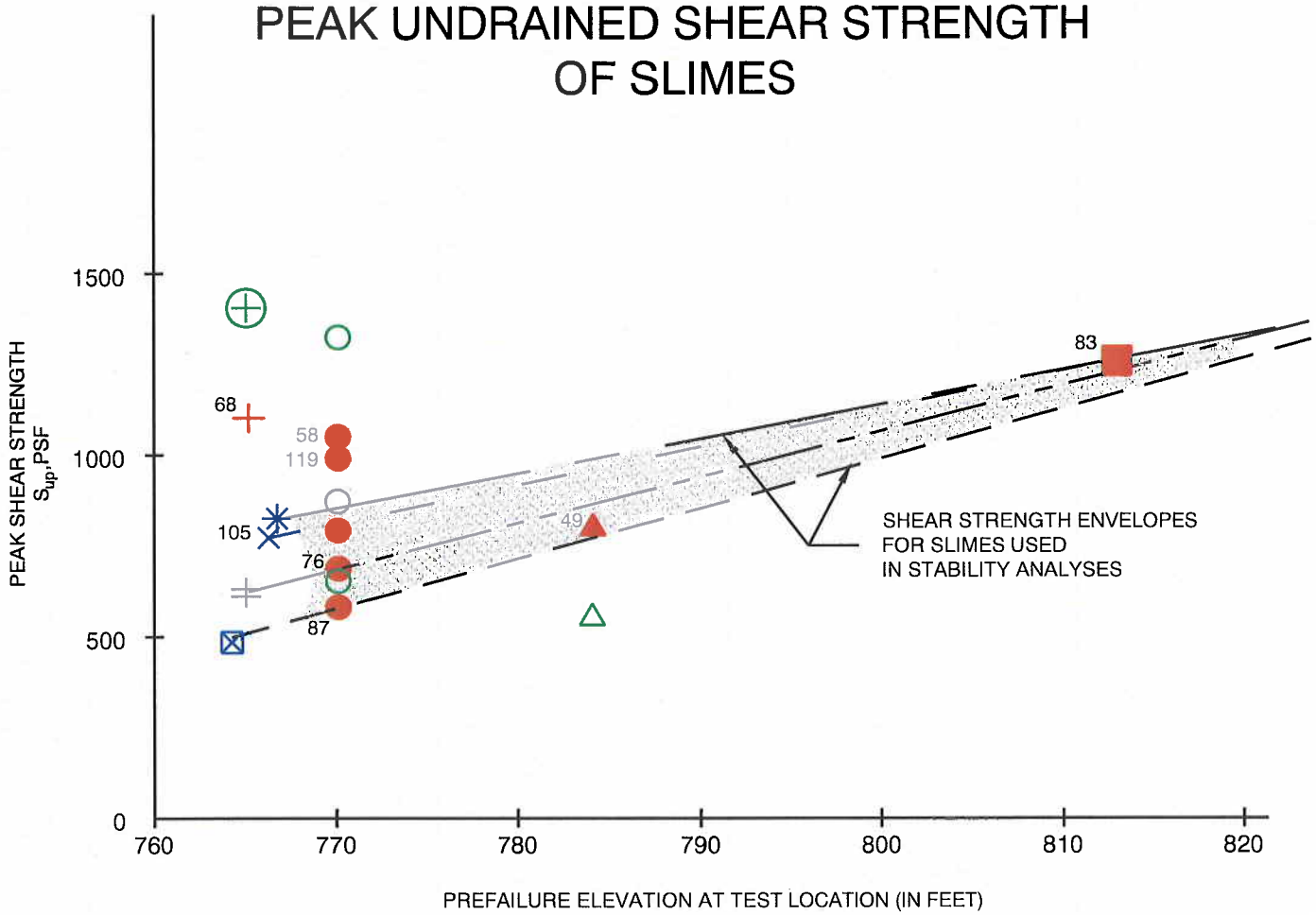
- 1 to 3 mm thick Layers

@ 5% ϵ $\Delta h = 0.05\,(3\ mm) = 0.075\ mm \sim 0.003$ mils

- 1" to 6 inch thick Layer

@ 5% ϵ $\Delta h = 0.05\,(6\ inch) = 0.3"$ @ 20% $\epsilon = .2(6") = 1.2"$

DREDGE CELL 2
PEAK UNDRAINED SHEAR STRENGTH OF SLIMES



BOREHOLE SERIES	DIRECT SIMPLE SHEAR CKU DSS	FIELD VANE SHEAR FVS
100	●	○
600	▲	△
400	+	⊕
500	■ (68)	

NATURAL WATER CONTENT

EL. 770 S_{up} LEGEND

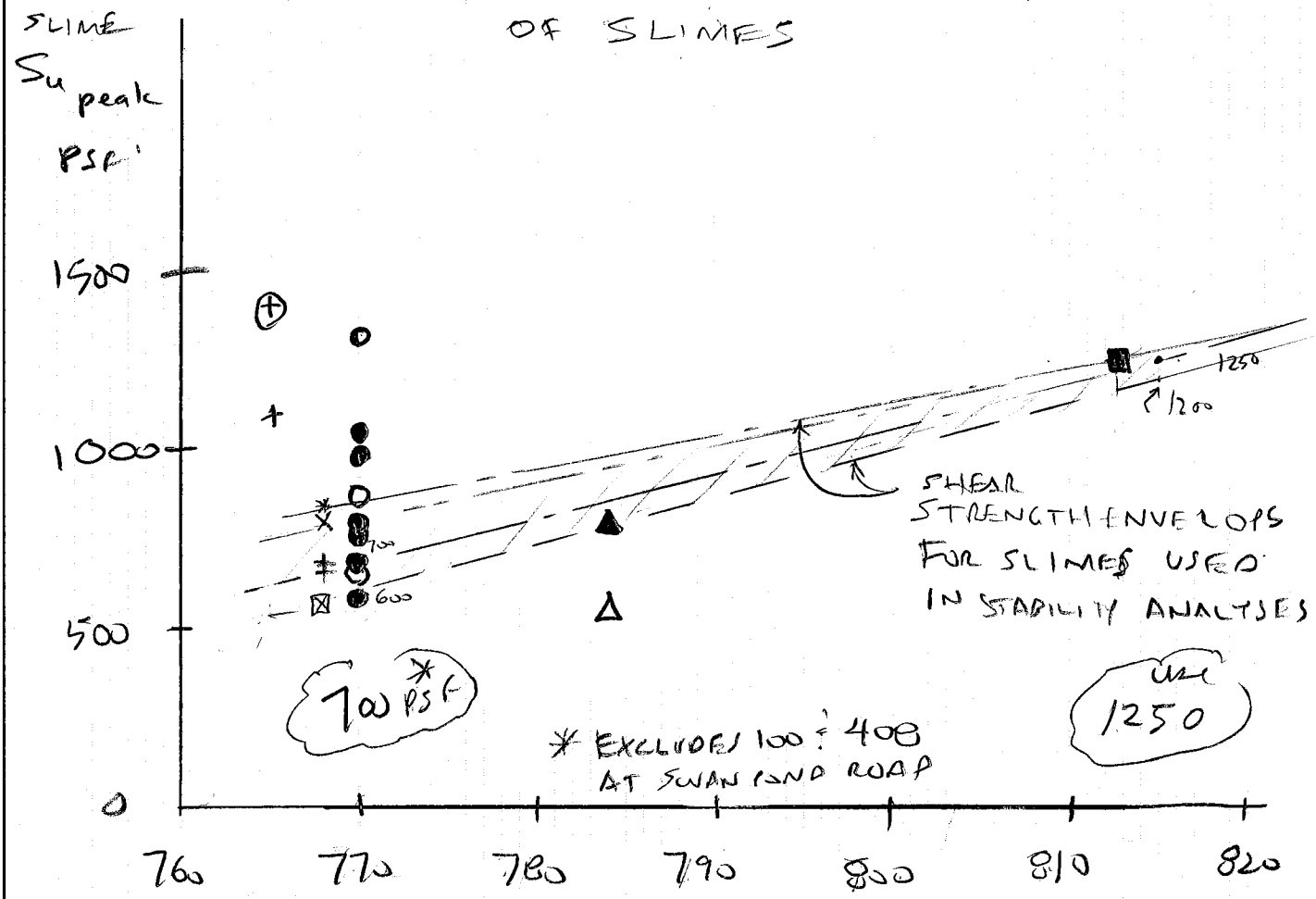
- ✳ GEOMETRIC MEAN OF DSS
- × P 50% PERCENTILE OF DSS
- ‡ P 33% PERCENTILE OF DSS
- ⊠ LOWER BOUND OF DSS

Calculation Sheet

Project	Subject
KINGSTON RCA	SLIMES UNDRAINED SHEAR STRENGTH

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
[illegible]	5/2/09	EED	6/2/09	60095746	NTS	2 of 4

PEAK UNDRAINED SHEAR STRENGTH OF SLIMES



BOREHOLE SERIES	CKU DSS	FVS
100	●	○
600	▲	△
400	+	⊕
500	■	

EL. 770 S_{u_p} LEGEND

*	GEOMETRIC MEAN
X	P50% PERCENTILE
#	P33% PERCENTILE
⊠	LOWER BOUND

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Calculation Sheet

Project	Subject
KINGSTON RCA	SLIMES UNDRAINED SHEAR STRENGTH

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
[illegible]	5/2/09	EED	6/2/09	60095742	NTS	3 of 4

Vane Shear Tests in SLIMES

BOREHOLE	ELEV	S_u V peak (DSS / VANE)	SENSITIVITY
09-100	731.27 to 731.7	1060 vs 1325 psf	ML 7.6
09-101	NONE	NO VANE	
09-102	NONE	NO VANE	
09-103	730.91 to 730.5	875 psf	CL –
09-104	736.28 to 736.8	LI = 6.1 800 psf	ML 2.1
09-105	734.73 to 735.15	900	ASH –
09-106	NONE	NO VANE	
09-107	NONE	NO VANE	
09-108	734.27 to 734.7	698 vs 650 psf	ML 13
09-109	736.27 to 736.7 735.9 to 735.52 734.27 to 734.7	675 psf 599 725 psf	A/H 13.5 ML CL 3.2
09-605	737.21 to 737.72	805 vs 550 psf	ML 5.5
09-502	NONE	NO VANE	
09-408	727.85 to 728.26	1110 vs 1425	ML/ASH 5.7

Table 5 Summary of Direct Simple Shear (DSS) specimen properties and test results

	Test No.	Boring Sample	Depth	w_n	γ_t	γ_d	σ'_{vo}	Laboratory Consolidation					Shear Data at Peak $(\tau_h)_{max} = s_u$				Figure Numbers	τ_h @ 20% psf
								σ'_{vc}	σ'_{vmax}	OCR	ε_{vmax}	ε_{vf}	γ	τ_h	τ_h/σ'_{vc}	σ'_v/σ'_{vc}		
	-	-	ft	%	pcf	pcf	psf	psf	psf	-	%	%	%	psf	-	-	-	
	1	2	3	4	5	5	6	7	8	9	10	11	12	13	14	15	16	
CL	G349	09-301B S6	87.5-90.0	22	126	103	4236	4174	4174	-	5.0	5.0	10.8	1190	0.285	0.484	5.4.1-5.4.5	NA CLAY
CL	G350	09-101B S5	38.0-40.5	23	123	100	2136	2103	4220	-	3.0	2.9	13.0	1840	0.874	1.409	5.4.6-5.4.10	NA CLAY
ML	G351	09-108B S2	23.5-26.0	76	93	53	1577	1591	2053	-	3.2	3.1	5.6	690	0.434	0.698	5.4.11-5.4.15	530 SILTE
ML	G352	09-109B S3	25.5-28.0	87	86	46	1483	1505	1891	-	3.9	3.9	4.8	599	0.398	0.625	5.4.16-5.4.20	500 "
ML	G353	09-605B S7	47.0-49.5	49	102	69	2356	2370	2904	-	3.8	3.8	5.2	805	0.340	0.624	5.4.21-5.4.25	660 "
ML	G354	09-103B S2	33.5-36.0	105	84	41	1935	1927	2580	-	3.6	3.4	5.1	779	0.404	0.672	5.4.26-5.4.30	550 "
ML	G355	09-103B S2	33.5-36.0	67	92	55	1935	1939	2592	-	4.2	4.1	5.7	684	0.353	0.652	5.4.31-5.4.35	520 "
ML	G356	09-100B S5	33.0-35.5	58	92	58	1958	1971	3436	-	3.2	3.1	12.2	1060	0.538	0.872	5.4.36-5.4.40	830 "
ML	G357	09-100B S6	35.5-38.0	119	83	38	2020	2024	3398	-	4.6	4.4	4.0	985	0.487	0.864	5.4.41-5.4.45	700 "
SHANSEP	G358	09-108B S2	23.5-26.0	104	84	41	1577	8288	8281	1.0†	15.1	15.1	4.0	1967	0.237	0.520	5.4.46-5.4.50	NA
ML	G359	09-408B S7	35.5-38.0	68	93	56	1707	1692	4193	-	2.4	2.3	5.1	1106	0.654	1.065	5.4.51-5.4.55	800 "
ML	G360	09-503B S3	35.5-38.0	83	83	46	3923	3937	4177	-	3.5	3.5	4.9	1249	0.317	0.563	5.4.56-5.4.60	800 "

Notes:
1. Notation given in Section xx

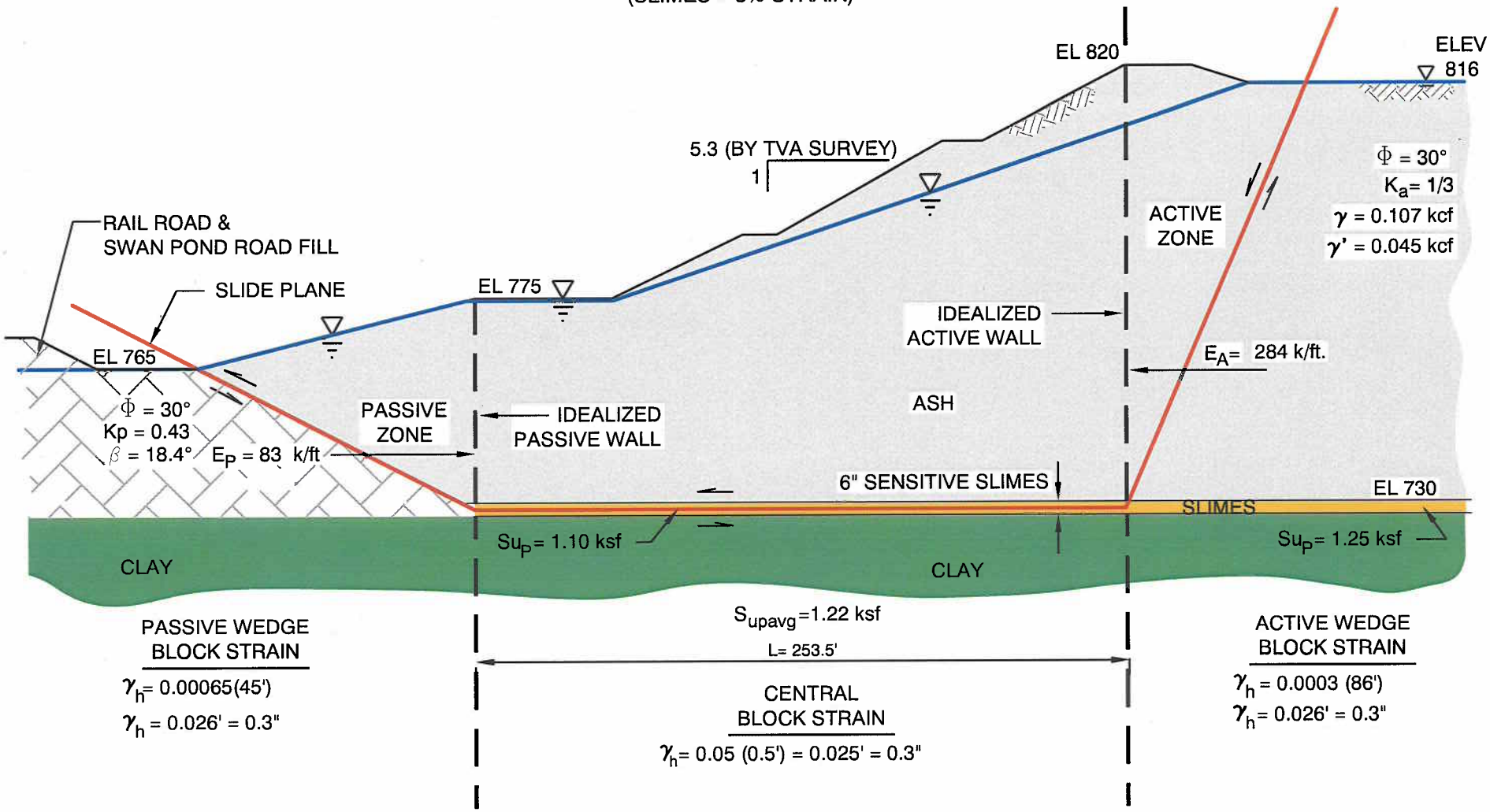
†SHANSEP OCR = 1 test

Draft – subject to change

CELL 2 SOUTHWEST SECTION
WEDGE BLOCK STABILITY ANALYSIS

BASED ON DISPLACEMENT COMPATIBILITY
(SLIMES = 5% STRAIN)

AECOM



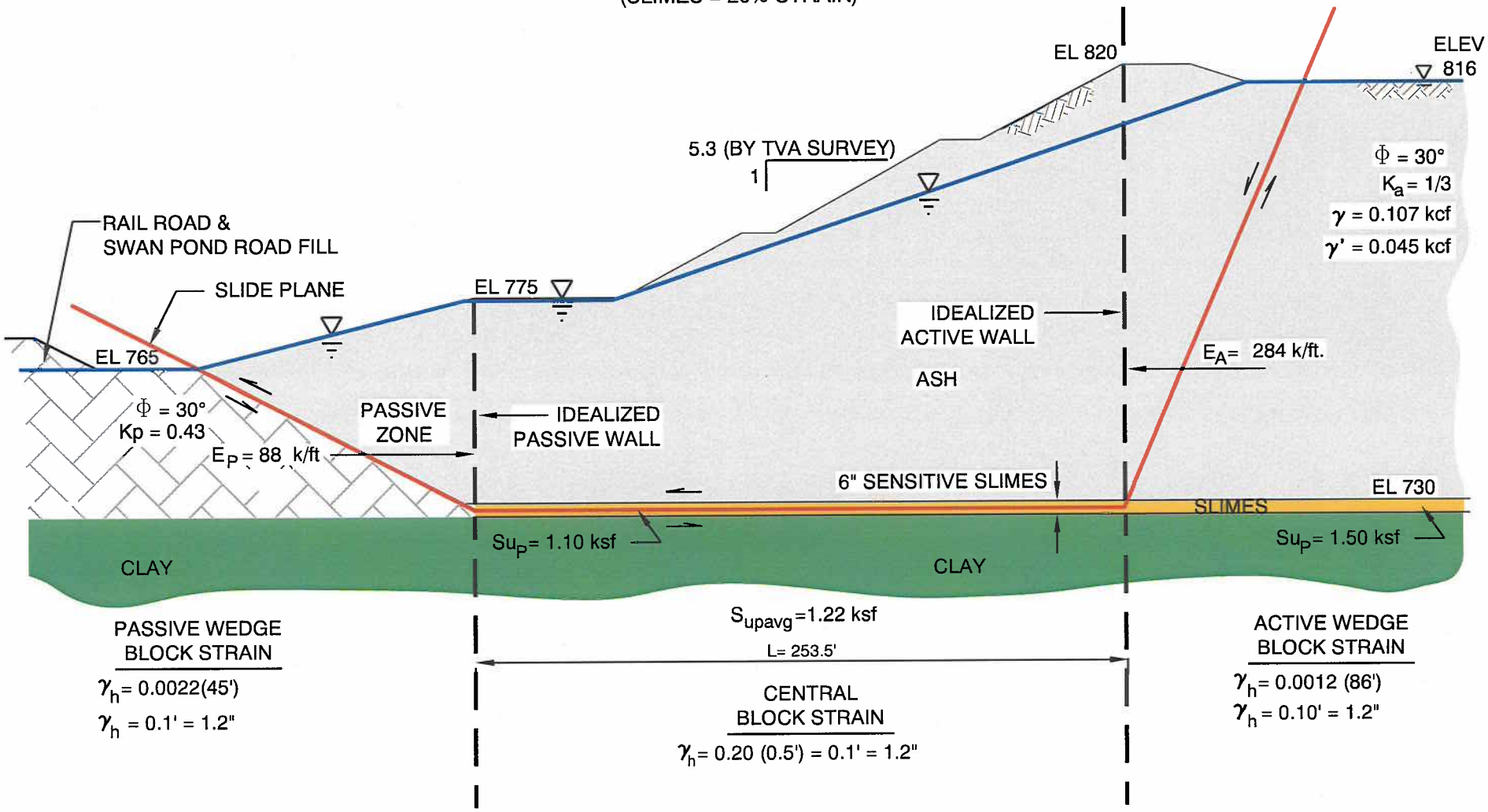
$$FS = \frac{S_{upavg}}{E_A - E_P} = \frac{1.22\ (253.5')}{284 - 83} = \frac{309\ k/ft.}{201\ k/ft.} = 1.5,\ \textbf{STABLE}$$

FIG. 1.6.3_17

AECOM

CELL 2 SOUTHWEST SECTION
WEDGE BLOCK STABILITY ANALYSIS

BASED ON DISPLACEMENT COMPATIBILITY
(SLIMES = 20% STRAIN)



$$FS = \frac{S_{upavg}}{E_A - E_P} = \frac{0.75\ (1.22)\ (253.5')}{284 - 88} = \frac{232\ k/ft.}{196\ k/ft.} = 1.2,\ \textbf{NO FAILURE}$$

(INCLUDES CREEP)

FIG. 1.6.3_18

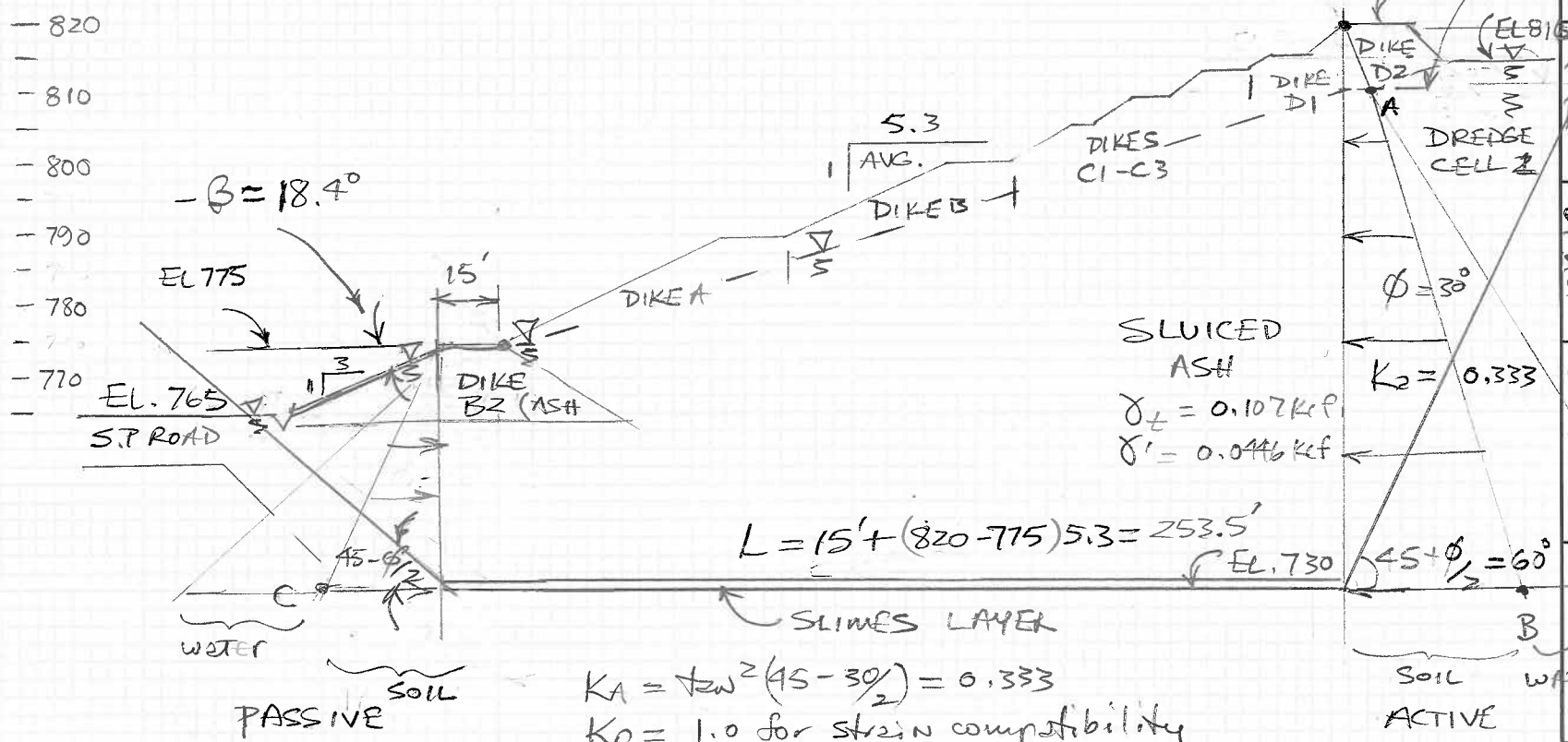
Calculation Sheet

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
SSD	6/12/09	RR	6/13/09	60075742	—	1 of 2

Project: KINGSTON RCA

Subject: WEDGE BLOCK - CELL 2 SW

DREDGE CELL 2 SW - WEDGE ANALYSIS



$K_A = \tan^2(45 - 30/2) = 0.333$

$K_p = 1.0$ for strain compatibility

5% Strain CASE { for $-\beta = -18.4°$; DM7.2 Fig 6; $K_p = 2.8/6.5 = \underline{0.43}$

$\beta/\phi = 0.61 \Rightarrow K_p = 2.8$; $\beta/\phi = 0$ (horiz.) $K_p = 6.5$

$$\text{Weighted } S_{u\,\text{Slimes}} \approx \frac{1500\text{psf}(18') + 1250(100) + 1175(100) + 1100(35.5)}{253.5'} = 1217 \text{ psf}$$

, Say 1215 psf

STS | AECOM

STS | AECOM

Calculation Sheet

Project	KINGSTON RCA			Subject	WEDGE BLOCK - CELL 2 SW		
Originated By	Date	Checked By	Date	STS Job No.	Scale		Sheet No.
SED	6/12/09	RCR	6/13/09	60095742	—		2 of 2

△ REV. 1 RCR 6/22/09

$p_A = \gamma_t \cdot H_1 \cdot K_2 = 0.107\ kcf \cdot (9')(0.333) \simeq 0.32\ ksf$

$p_B = \gamma' \cdot H_2 \cdot K_2 + \gamma_w \cdot H_2 (1.0) = 0.0446(\underbrace{816-730}_{86'})(0.383) + 0.0624(86')$

$= 1.28 + 5.37 = 6.65\ ksf$

△ REV. 1 RCR

$p_C = \gamma' H_3 \cdot K_p + \gamma_w H_3 = 0.0446(\underbrace{775-730}_{45'})(0.43) + 0.0624(45')$

$= 0.86 + 2.81 = 3.67\ ksf$

$E_A = p_A\left(\frac{H_1}{2}\right) + \frac{p_A + p_B}{2}\left(\frac{H_2}{2}\right) = 0.32\left(\frac{9'}{2}\right) + \underbrace{\frac{0.32+6.65}{2}}_{3.49}\underbrace{\left(\frac{811-730}{2}\right)}_{81'} =$

$= 1.44 + 282.3 = 283.7\ K/Lf$

$E_p = p_C\left(\frac{H_3}{2}\right) = 3.67\left(\frac{775-730}{2}\right) = 82.6\ K/Lf$

$FS_{5\%} = \frac{S_{u_{Avg}} \cdot L}{E_A - E_p} = \frac{1.215\ psf\ (253.5')}{283.7 - 82.6\ K/Lf} = \frac{308.0}{201.1} = 1.53 = \underline{1.5}$ (For 5% strain)

For 20% strain, from Cell 2 NW; $K_p = 1.3$ vs. 1.0 $\quad \frac{1.3}{1.0} = 1.3\times$
20% ε $\quad$ 5% ε

For 20% strain Cell 2 SW; use $K_p = 1.3(\underbrace{0.43}_{K_p,\ \varepsilon = 5\%}) = \underline{0.56}$

Use Avg $S_{u_{Steady\ State}} \sim 0.75\ S_u = 0.75(1.215\ ksf) = 0.91\ ksf$

$p_C = 0.0446(45')(0.56) + 0.0624(45') = 1.12 + 2.81 = 3.93\ ksf$

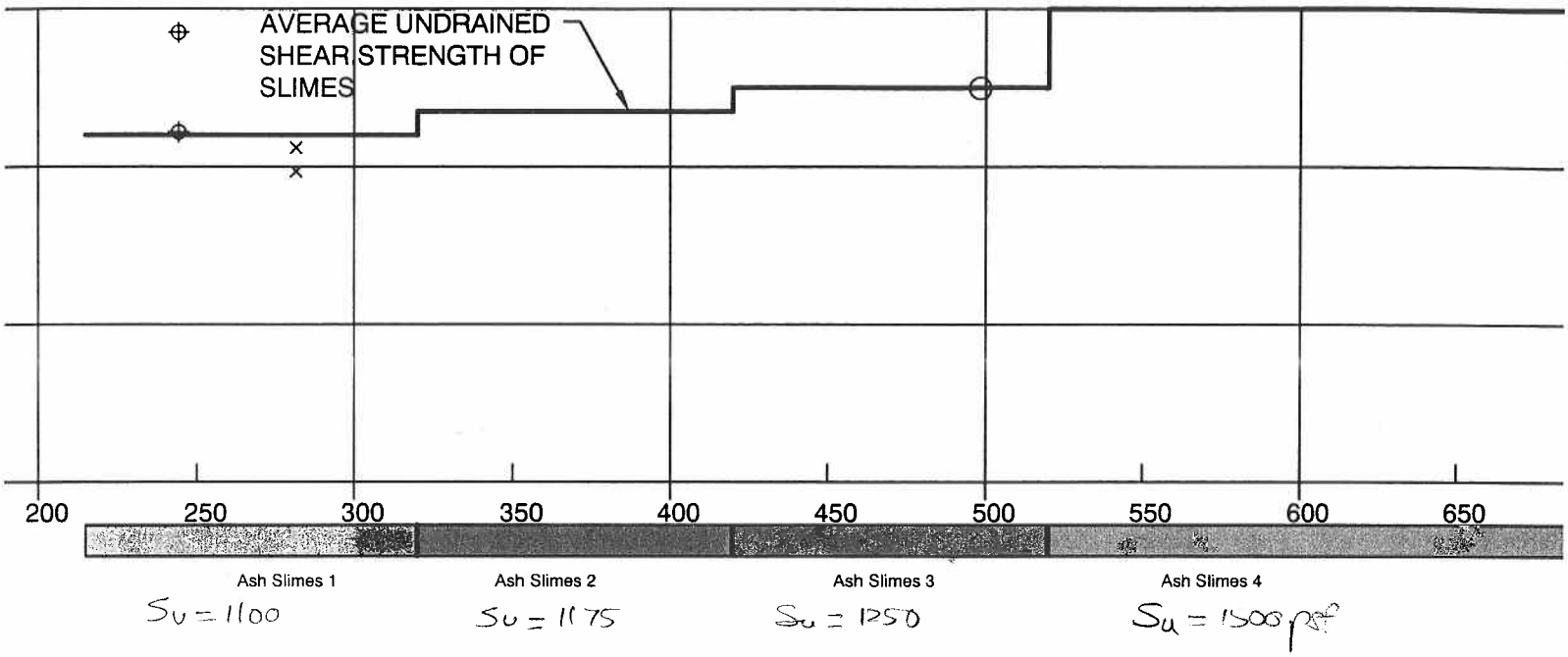
$E_p = 3.93\left(\frac{775-730}{2}\right) = 88.4\ K/Lf$

$FS_{20\%} = \frac{S_u \cdot L}{E_A - E_p} = \frac{0.91\ ksf\ (253.5)}{283.7 - 88.4\ K/Lf} = \frac{230.7}{195.3} = 1.18 \approx \underline{1.2}$

However for starting FS = 1.5 @ 5% ($1/1.5$ or 67% of S_u is used)

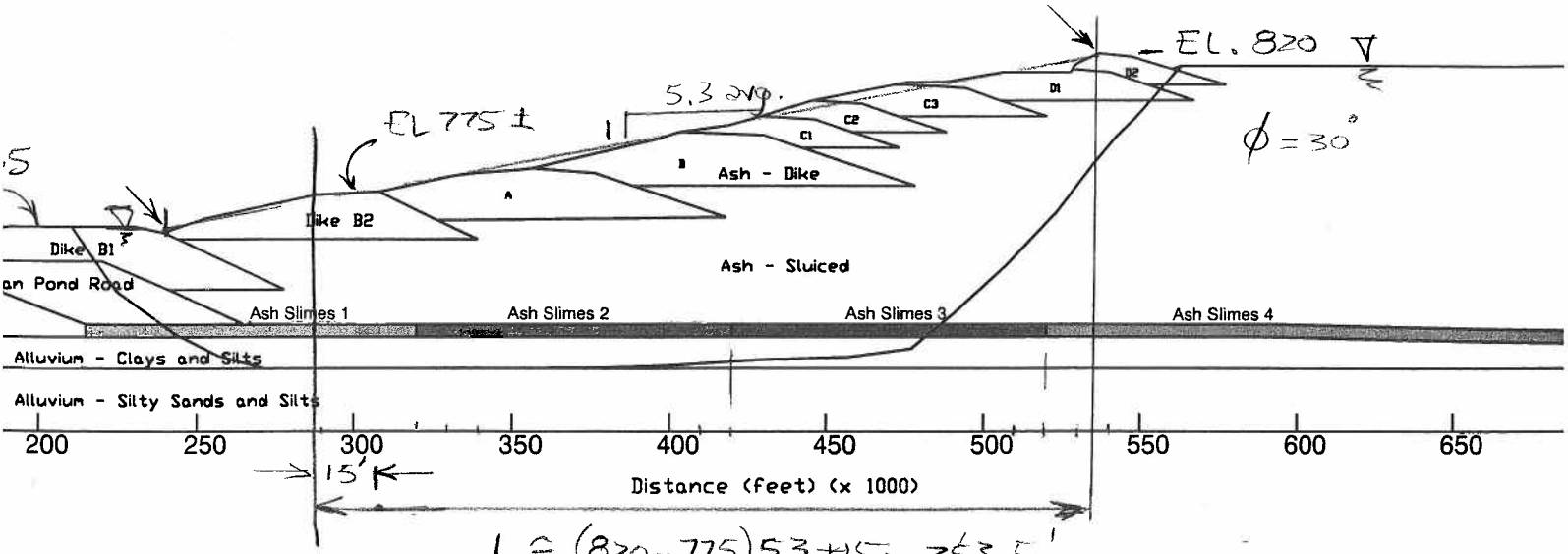
Creep test on slimes doesn't trigger until lateral strain ≈ 85%±

SLOPE STABILITY ANALYSIS FOR CELL 2 SW SECTION CASE 1 (SECTION ALONG SWAN POND ROAD)



EGEND:			
SS		VST	
	○	-	○
	⌖	1425	⌖
60	×	-	

CA Comments: Slope Stability Analysis for Cell 2 Southwest Section Case 1
'rice
rained Condition
Surface



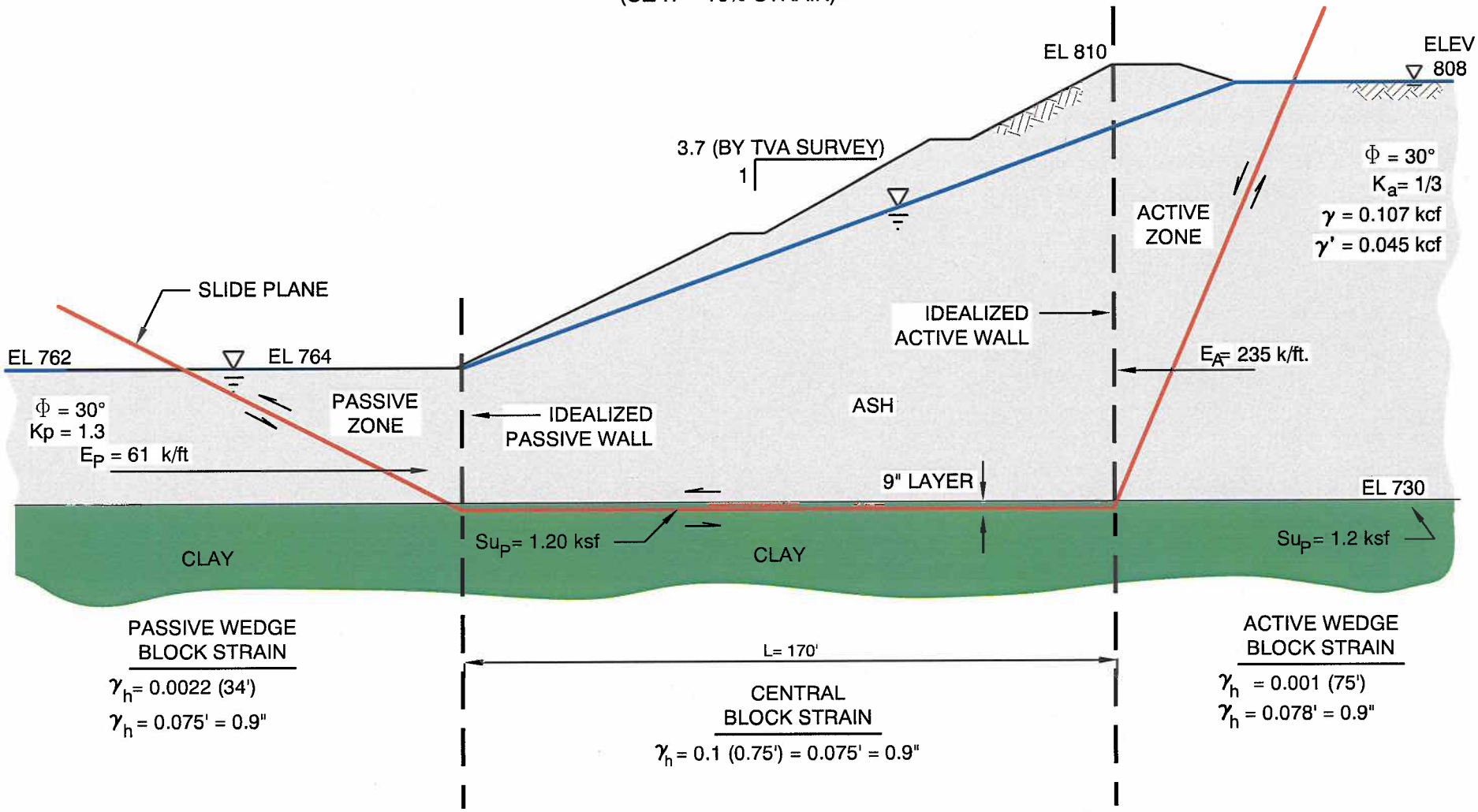
$L \cong (820-775)\,5.3 + 15 \cong 253.5'$

Weighted $S_{u_s} \cong \dfrac{1500\,psf\,(18') + 1250(100') + 1175(100') + 1100(35.5')}{253.5} = 1217\,psf$, say 1215 psf

PHASE 1 EAST EMERGENCY DREDGE CELL
WEDGE BLOCK STABILITY ANALYSIS

BASED ON DISPLACEMENT COMPATIBILITY
(CLAY = 10% STRAIN)

AECOM



$$FS = \frac{S_{upavg}}{E_A - E_P} = \frac{(1.2)\ (170')}{235 - 61} = \frac{204\ k/ft.}{174\ k/ft.} = 1.2, \textbf{ STABLE}$$

FIG. 1.6.3_19

Calculation Sheet

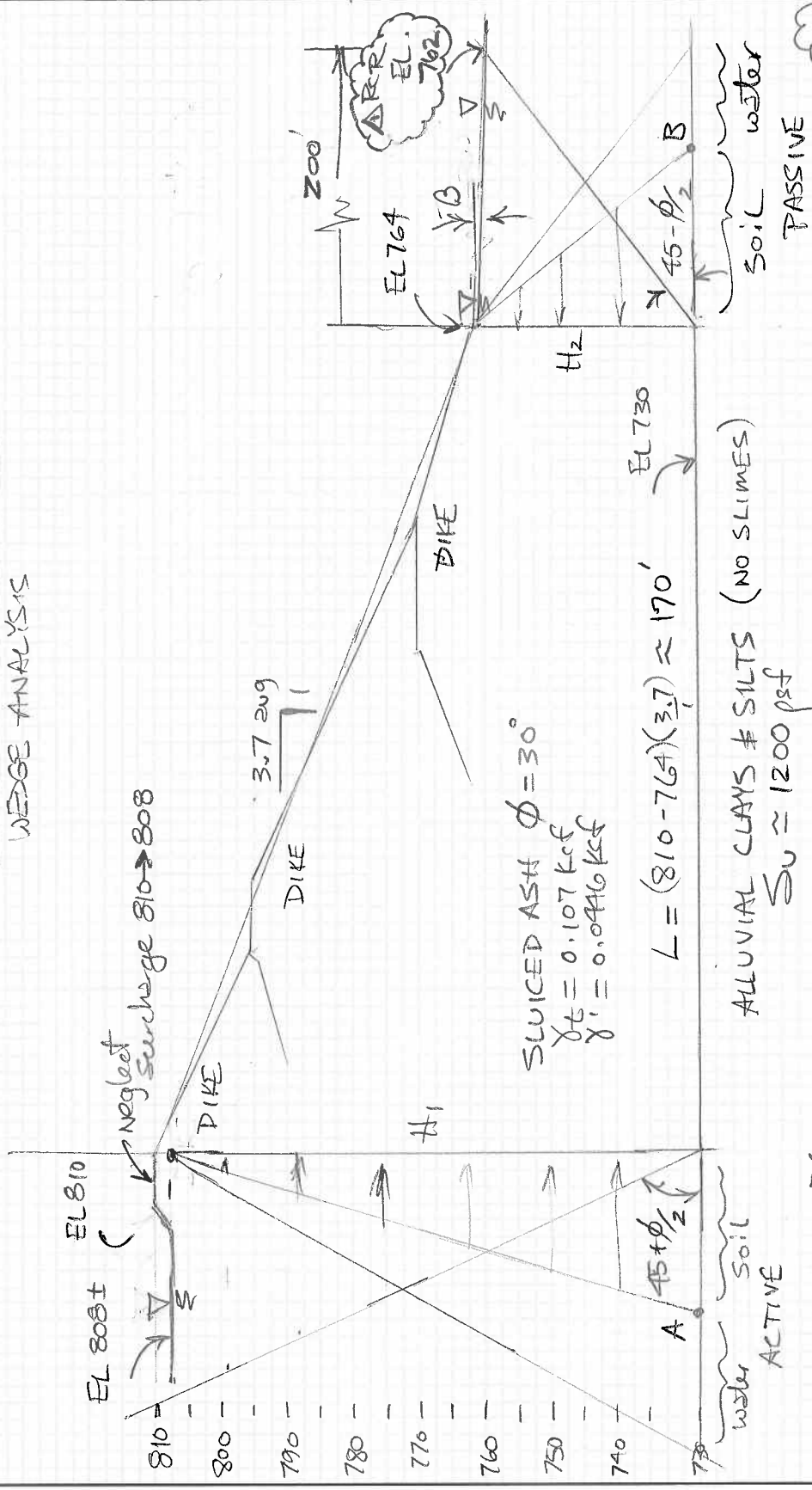
Project	Subject
KINGSTON RCA	WEDGE BLOCK – Ph. I Emerg. Cell (E)

Originated By	Date	Checked By	Date	STS Job No.	Scale	Sheet No.
ESD	6/12/09	RCR	6/13/09	60095742	—	1 of 2

△1 REV 1 RCR 6/22/09

PHASE I EMERGENCY DREDGE CELL (EAST)

WEDGE ANALYSIS



ASH: $K_a = \tan^2(45-30/2) = 0.333$

$K_p = 1/K_a = 3.0$, but use 1.3 for strain compatibility between active & passive wedges & along clay over length L

$\beta = \tan^{-1}\frac{764-760}{200'} = \sim 1.1°$ (Neglect reduction in P_p)

Native Clays peak S_u ~ 10% vs 5% ε

STS | AECOM

Calculation Sheet

Project	KINGSTON RCA			Subject	WEDGE-BLOCK - Ph I Emerg. Cell (E)	
Originated By ZED	Date 6/12/09	Checked By RCR	Date 6/13/09	STS Job No. 6009 S742	Scale	Sheet No. 2 of 2

△REV 1 RCR 6/22/09

$$p_A = \gamma' H_1 K_a + \gamma_w H_1 \cdot K_a$$

$$= 0.0446\ \text{kcf}\,(\underbrace{808-730}_{78'})(0.333) + 0.0624\ \text{kcf}\,(78')(1) = 1.16 + 4.87 = 6.03\ \text{ksf}$$

△REV 1 RCR

$$p_B = \gamma' H_2 K_p + \gamma_w H_2$$

$$= 0.0446\Big[\underbrace{\left(\frac{762+764}{2}\right)-730}_{33'}\Big]1.0 + 0.0624\,(34')(1.0) = 1.47 + 2.12 = 3.59\ \text{ksf}$$

$$E_A = p_A\left(\frac{H_1}{2}\right) = 6.03\ \text{ksf}\left(\frac{78'}{2}\right) = 235.2\ \text{K/Lf}$$

$$E_P = p_B\left(\frac{H_2}{2}\right) = 3.59\left(\frac{34'}{2}\right) = 61.0\ \text{K/Lf}$$

$$FS_{\varepsilon=5\%} = \frac{S_u L}{E_A - E_P} = \frac{1.200\ \text{ksf}\,(170')}{235.2 - 61.0} = \frac{204.0}{174.2} = \underline{1.17} = \underline{\underline{1.2}}$$

For ε = 20%. take no reduction in S_u for Alluvial clay due to overconsolidation. Also ε_{peak} ~ 10% for alluvial clay vs. 5% for slimes.

Therefore, from Cell 2 North calculation, can use at least $K_p = 1.3$ vs. 1.0 for ε = 5% strain.

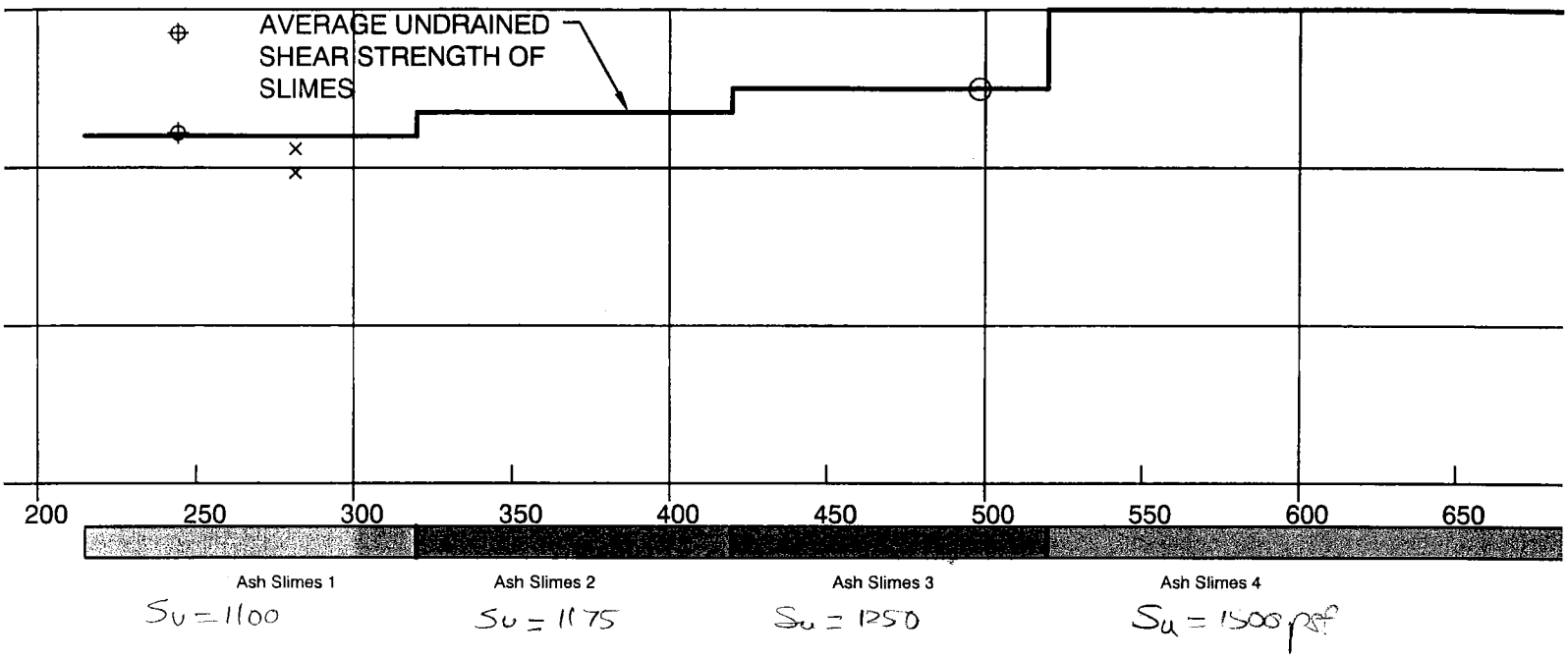
△REV 1 RCR

$$p_B = 0.0446(33')(1.3) + 0.0624(34')(1.0) = 1.91 + 2.12 \simeq 4.03\ \text{ksf}$$

$$E_P = (4.03\ \text{ksf})\left(\frac{34'}{2}\right) = 68.5\ \text{K/Lf}$$

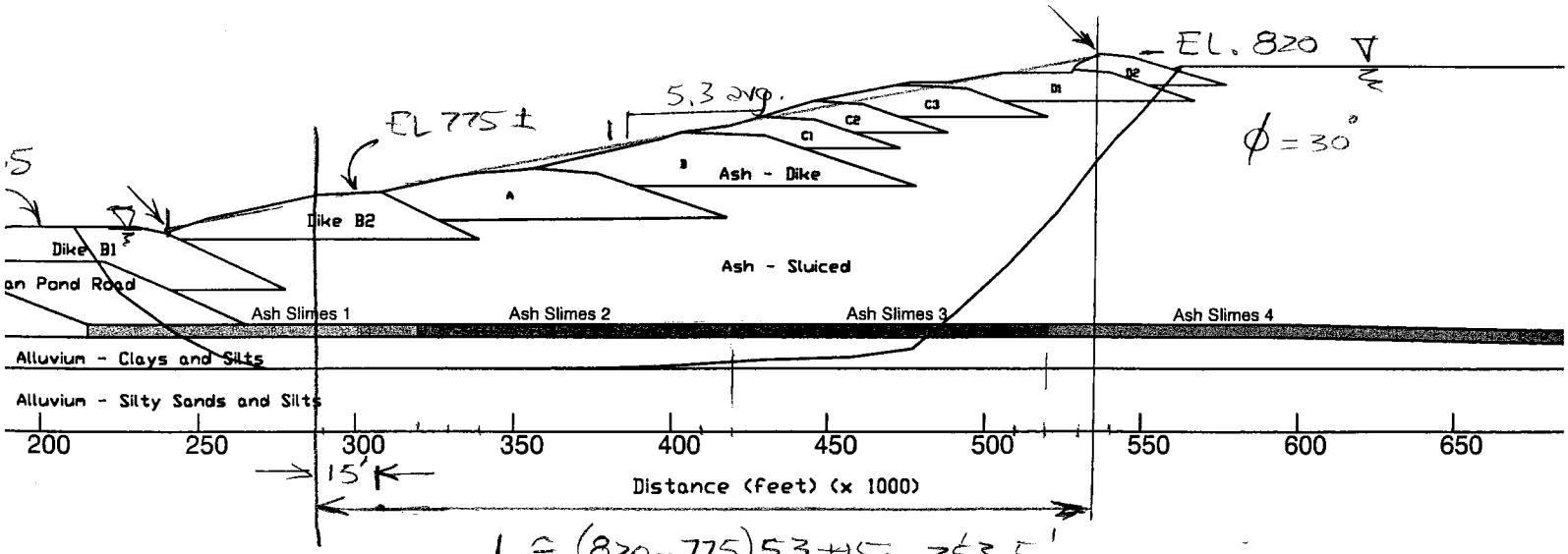
$$FS_{\varepsilon=20\%} = \frac{1.200(170')}{235.2 - 68.5} = \frac{204.0}{166.7} = \underline{1.22} \quad \text{use } \underline{\underline{FS = 1.2}}$$

SLOPE STABILITY ANALYSIS FOR CELL 2 SW SECTION CASE 1 (SECTION ALONG SWAN POND ROAD)



EGEND:			
SS		VST	
	○	-	○
	⊕	1425	⊕
60	×	-	

CA Comments: Slope Stability Analysis for Cell 2 Southwest Section Case 1
'rice
rained Condition
Surface



$L \cong (820-775)\,5.3 + 15 = 253.5'$

Weighted $S_{us} \cong \dfrac{1500\,psf\,(18') + 1250(100') + 1175(100') + 1100(35.5')}{253.5} = 1217\,psf$, say 1215 psf

FUNDAMENTALS OF

SOIL MECHANICS

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outlined in Section 9·18. The transition section at the inner portion of the shell is so much finer than the main portion of the shell that all heads are dissipated and practically all seepage forces occur within it. Therefore, no appreciable water pressures exist on the failure plane.

It may be noted that Fig. 18·4 (*a*) represents a special case of the general passive pressure condition illustrated in Fig. 17·12. This special case has a negative angle i and a zero value for angle α. Equation 17·17 could be used for a solution of this case. However, G. Gilboy (63) has given the following, more convenient equation for the hydraulic-fill dam that is just at the point of failure:

$$\sqrt{\frac{\gamma_c}{\gamma_d}} = \frac{(C - A)\sqrt{1 + B^2} + \sqrt{C - A}\sqrt{C - B}\sqrt{1 + A^2}}{(1 + C^2) - (C - A)(C - B)} \tag{18·2}$$

In this equation γ_c and γ_d are the unit weights of core and shell respectively, C and A are the cotangents of the outer slope angle and the core slope angle, respectively, and B equals cot ϕ. The notations are shown in Fig. 18·4 (*c*). This equation may be used to determine the value of γ_c which would just cause failure. The ratio between this value and the actual value of γ_c may then be obtained; it may be called the factor of safety with respect to core pressure.

Study of the force triangle of Fig. 18·4 (*b*) shows the beneficial effect of a steep core. If the core is flattened the force polygon becomes somewhat as shown by dotted lines, and on any trial failure plane it is seen that a greater obliquity of stress and a less stable condition result.

18·5 The Sliding-Block Analysis

A relatively simple and fairly common case, for which the use of the circular failure assumption is not reasonable, is that in which the critical surface, or a large portion of the critical surface, is approximately a horizontal plane. A good illustration is a dam with a relatively thin horizontal stratum of soil of low shearing resistance at a shallow depth below the surface of the foundation. Other examples are cases in which the foundation soils contain horizontal stratification and cases in which an over-

compacted earth fill overlies a foundation consisting of a clay with structure that is susceptible to disturbance.

Embankments of these types may be investigated by the procedure sometimes designated as the *sliding-block analysis*. A simple analysis of this type is shown in Fig. 18·5. It will be assumed that the horizontal plane OD in this figure is a surface of weakness within the foundation and that it is the critical surface. The analysis consists of the consideration of the forces acting on the mass represented by triangle AOD. The horizontal

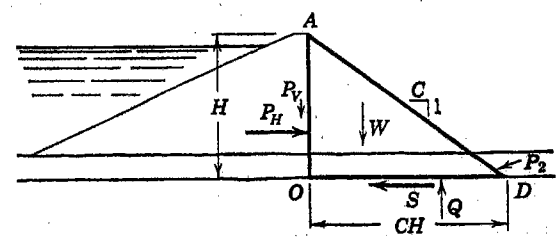

Fig. 18·5 Simple cross section used in the example of the sliding-block analysis.

forces are shown by heavy line vectors, since the analysis is based mainly on these forces. Vectors representing vertical forces are shown lightly and they will be considered in less detail. The horizontal force P_2, which is the passive, lateral resistance of the soil above point D, is also shown lightly, since it may be of inappreciable magnitude when the critical plane is at a shallow depth; it will be neglected in this example. Any failure tendency that results from weakness of the soil along line OD introduces lateral movement at all points within mass AOD, increasing the horizontal dimensions of the dam. This lateral bulging of the dam tends to reduce the lateral pressures on surface AO to active pressures, which have values that depend on the shearing characteristics of the dam and are independent of the strength of the underlying weak stratum. However, the thrust across AO may be the cause of failure of the mass AOD by sliding along the weak foundation plane OD.

Force P_H may be evaluated by active pressure considerations, and this thrust also must include any neutral force that acts across surface AO. The stability of the block AOD may be in-

vestigated by comparing thrust P_H with the available shearing resistance along surface OD. The average lateral pressure across AO is equal to P_H/H; it will be designated by p_a. The average shearing strength depends greatly on the degree to which consolidation takes place under the weight of the soil above surface OD. However, only the average, required shearing strength s_r will be considered in this simple analysis, and no attempt will be made to express the actual shearing strength in terms of the pressure. Force S must balance force P_H, whence

$$s_r CH = p_a H$$

in which C is the slope of the dam. Thus

$$s_r = \frac{p_a}{C} \tag{18·3}$$

Typical analyses of the sliding-block type are much more involved than the simple case used for illustration indicates. If the dam is composed of cohesive soil, the active pressure determinations may be difficult. If the weak stratum is of appreciable depth, trial failure surfaces at various depths must be considered. The consolidation conditions not only affect the shearing strengths in the weak stratum but they also may have a considerable effect on the location of the critical surface. The vertical surface through the crest of the dam is not necessarily the critical one, and other vertical surfaces nearer the toe of the dam must sometimes be considered.

An important type of sliding-block action that can occur near the toe of a dam will be cited. The overburden pressure at the base of the dam is much greater below the crest than it is nearer the toes. Therefore, overcompaction may exist near the toes when it does not occur below the crest. The swelling tendency that is inherent in overcompacted soil may lead to lateral pressures that approach passive pressure magnitudes. These pressures may cause bulging near the toe of the dam, and if the foundation soil is susceptible to remolding its shearing strength may be greatly decreased and a failure surface may develop. It is possible that the disturbance of structure may reduce the shearing strength in the foundation to such a low value that a

slide will occur. However, the bulging that occurs tends to relieve the lateral pressures and, if the remolded foundation soil is of sufficient strength to resist active lateral pressures, there will be no failure, but there may be noticeable bulging near the toe of the dam.

18·6 Roofing and Piping

When a masonry dam rests on soil, local irregularities always exist in the bearing pressures on the foundation because of the inevitable local variations in soil properties. Small zones exist over which pressures are much larger or much smaller than the average values thereabout, and at some points there may actually be small gaps between the masonry and the soil. These gaps or points of low pressure would of themselves cause no concern, but when water seeps through them they may form the starting points of channels of flow, and by backward erosion such channels may in time increase to appreciable lengths. Thus the resistance to seepage along seepage paths which are in contact with masonry may be much less than it is along other paths. This phenomenon is called *roofing*.

A similar condition may exist at the abutment of a masonry dam. To prevent the formation of flow channels where soil is in contact with a vertical masonry surface the soil should be well compacted and in tight contact with the masonry at all points. Cutoffs are often used to increase the lengths of flow paths and thus decrease the danger of such channels. A batter on a wall is desirable if adjacent soil is settling relative to the wall. If the wall is settling relative to the adjacent soil, a batter is undesirable, since it causes the contact pressure to decrease. The numerous difficulties and dangers that are inherent in any case in which seepage occurs at a contact between soil and masonry are an ever-present argument in favor of earth dams at sites where the foundations are of soil.

If erosion starts at the point of exit of a flow line that has passed below or around a dam, the erosion may progress backward, aided perhaps by the presence of a flow channel that is the result of roofing. The resulting condition is called *piping*. One of the worst characteristics of piping is that it needs to occur

SLOPE STABILITY AND STABILIZATION METHODS

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occur irrespective of the stability of the new overlying embankment material. Consideration of the internal stability of an embankment-foundation system, rather than just the embankment, may be necessary. A simple rule of thumb based on bearing capacity theory can be used to make a preliminary estimate of the factor of safety against circular arc failure for an embankment built over a clay foundation. The rule is (Cheney and Chassie, 1982)

$$\text{FOS} = \frac{6c}{\gamma_{\text{fill}} \times H_{\text{fill}}} \qquad \text{(Eq. 1-4)}$$

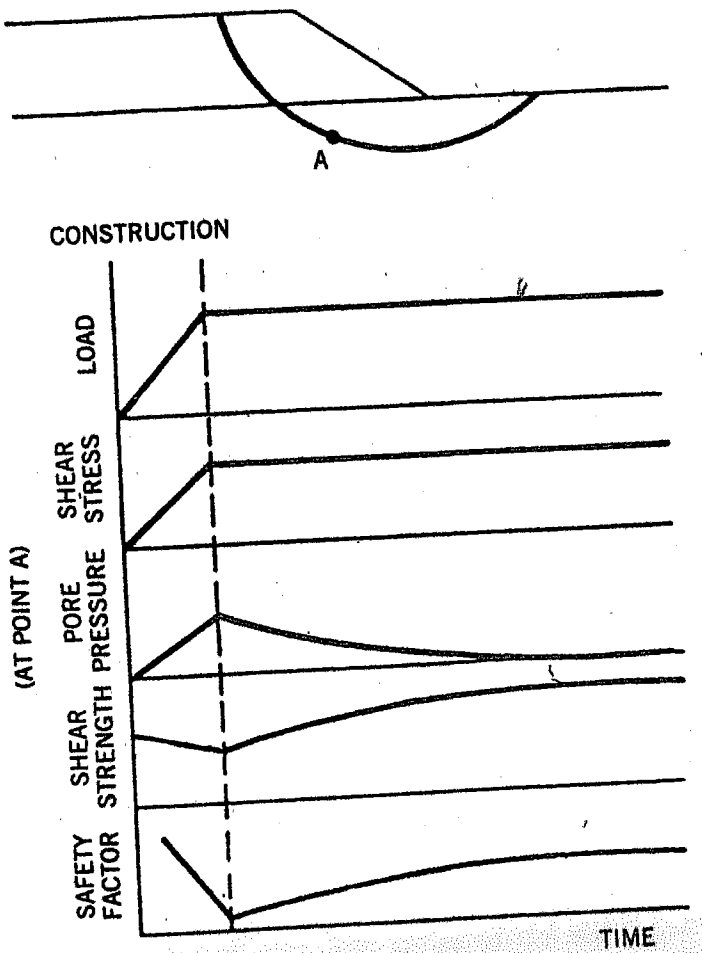

Figure 1.6 Stability conditions for an embankment slope over a clay foundation (from Bishop and Bjerrum, 1960, reproduced by permission of ASCE).

where FOS = factor of safety
c = cohesion of foundation clay (pounds per square foot)
γ_{fill} = unit weight of embankment fill (pounds per square foot)
H_{fill} = height of embankment fill (feet)

The factor of safety computed using this rule serves only as a rough preliminary estimate of the stability of an embankment over a clay foundation and should *not* be used for final design. The simple equation does not take into consideration factors such as fill strength, strain incompatibility between embankment fill and the underlying foundation soils, and fill slope angle. In addition, it does not identify the location of a critical failure surface. If the factor of safety using the rule-of-thumb equation is less than 2.5, a more sophisticated stability analysis is required (Cheney and Chassie, 1982).

Figure 1.6 shows the variations in safety factor, strength, pore pressures, load, and shear stresses with time for an embankment constructed over a clay deposit. Over time, the excess pore pressure in the clay foundation diminishes, the shear strength of the clay increases, and the factor of safety for slope failure increases.

Embankment fills over soft clay foundations are frequently stronger and stiffer than their foundations. This leads to the possibility that the embankment will crack as the foundation deforms and settles under its own weight and to the possibility of progressive failure because of stress–strain incompatibility between the embankment and its foundation. Design charts developed by Chirapuntu and Duncan (1977), using finite element method analyses, depict the effects of cracking and progressive failure on the stability of embankments on soft foundations. These charts may be used as a supplement to conventional stability analyses. The use of geosynthetic reinforcement in the fill may prevent the initiation of cracking and subsequent failure

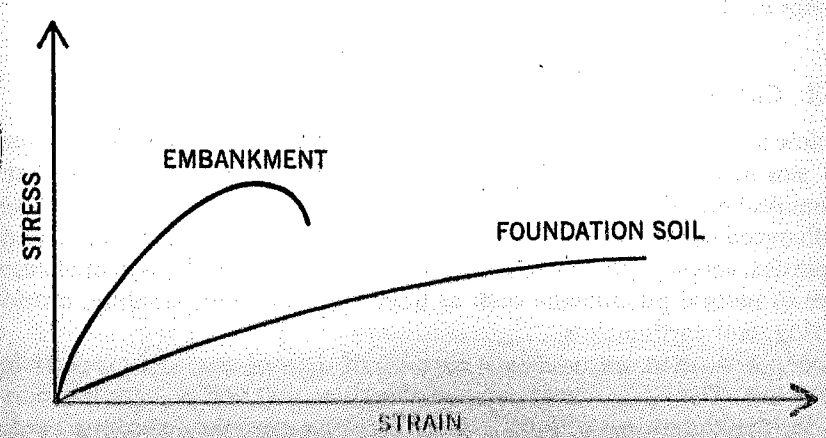

Figure 1.7 Example of stress–strain incompatibility.

This example illustrated the use of four methods that may be used to assign pore water pressures for slope stability analysis. Of these, three are suitable for problems where a program is used to determine the critical surface with the lowest FOS using an automatic search procedure. The analysis of the circular surface revealed that the FOSs were consistent for each of the methods employed and varied within a very small range of 1.294 to 1.301. This range is small enough to have been influenced by minor uncertainties concerning the manual development of a flow net and the determination of the piezometric surface for the circular failure surface. The small FOS range strongly suggests that a phreatic surface may be used to effectively model the pore water pressures within a slope. However, the engineer should be discouraged from using computer programs to search for a critical surface using a piezometric surface to simulate the variation in pore pressures. In such cases, the pore water pressure will be overestimated and an unrealistically low FOS value will be reported by the program.

6.4.4 Negative Pore Pressures

There may be cases where an engineer wishes to use *negative* pore pressures to take advantage of the apparent cohesive strength available due to suction within the soil in the slope. As recommended in the last chapter, the influence of suction should be included by increasing the *total cohesion* according to the measured values of matric suction within the slope. In some cases, actual negative pore pressures have been used in slope analysis to increase the shear strength of the soil. This method is not recommended, as it only affects the frictional component via the $(\sigma - u)\tan\phi$ term and may not generate reliable values of strength.

6.5 BLOCK ANALYSIS

A block analysis may be used to estimate the FOS against sliding in situations where the shearing strength of an embankment fill is greater than that of the foundation soils, as shown in Figure 6.11. In this case, it is necessary to investigate the stability along a surface of failure passing through the foundation of the embankment in addition to the usual studies for possible failure within the embankment itself.

A predominantly planar failure surface develops if the weak foundation layer is relatively thin. Stability may be analyzed by means of a sliding block shearing through the weak foundation layer. The block analysis is quick, simple and straightforward, and can be quickly performed by hand calculation.

For the analysis, the potential sliding block is divided into three parts (Figure 6.11): (1) an *active* wedge at the head of the slide, (2) a

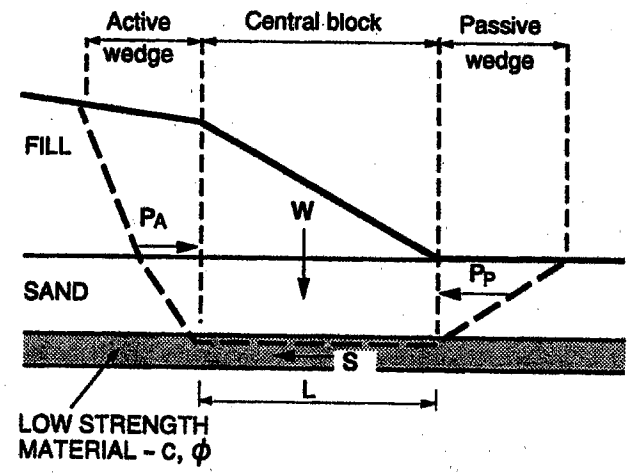

Figure 6.11 Sliding block analysis.

block, and (3) a *passive* wedge at the toe. The FOS would be computed by summing forces horizontally to give

$$\text{FOS} = \frac{\text{Horizontal resistance forces}}{\text{Horizontal driving forces}} = \frac{P_P + c'_m L + (W - u)\tan\phi'_m}{P_a} \qquad \text{(Eq. 6-5)}$$

where P_a = active force (driving)
P_p = passive force (resisting)
L = resisting force due to cohesion of clay
c'_m and ϕ'_m = strength parameters of the soil at the base of the central block, with effective weight $(W - U)$

The active and passive lateral earth pressures used in the block analysis are calculated using

$$\sigma_{A/P} = K_{A/P}\sigma'_v \mp 2c_m\sqrt{K_{A/P}} \qquad \text{(Eq. 6-6)}$$

where K_A = earth pressure coefficient
K_P = passive earth pressure coefficient
σ'_v = vertical effective stress
c_m = mobilized cohesion parameter

The earth pressure coefficients may be estimated using the Rankine

INTRODUCTORY

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SOIL MECHANICS AND FOUNDATIONS: GEOTECHNICAL ENGINEERING

FOURTH EDITION

MACMILLAN PUBLISHING CO., INC.
New York
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on sand (the angle of internal friction), the sand grain will slide down the slope. The steepest slope that a sand can attain, therefore, is equal to the angle of internal friction of the sand. Usually, this is near the minimum value of ϕ, since near the surface the sand is very likely to be in a loose condition. The *angle of repose* of sand as it forms a pile beneath a funnel from which it is poured is therefore about the same as the angle of internal friction of the sand in a loose condition.

STABILITY OF BLOCKS / In many cases wide seams of extremely soft strata or strata weakened by neutral stress causes failure to take place by translation or straight-line movement along planes of weakness instead of by rotation on a circular surface. This often occurs when earth dams are founded on stratified deposits because the horizontal force of the water acts parallel to the potential planes of failure.

The procedure is essentially one of trial and error. A possible failure plane (usually a weak stratum) is selected, and vertical boundaries *ab* and *cd* are arbitrarily assumed (Fig. 12.10). The wedge *abdc* is acted upon by its own weight, W, the force of active earth pressure P_A at the right, and the force of passive earth pressure P_P at the left. Shear along the failure plane $\overline{bd}$ resists the unbalanced forces acting on the block. If the total shear force is S, then the safety factor F_s is given by

$$F_s = \frac{S}{P_A - P_P} \tag{12:7}$$

Various possible failure planes should be tried, such as described by Lowe[12:15]; the one giving the minimum safety factor governs. The analysis includes the inconsistency of simultaneous mobilization of P_A and P_P, although much more movement is required to mobilize P_P. If the analysis finds a safety factor $F_s \leqq 1$ utilizing P_0 instead of P_P, the stability should be considered suspect. In the case of an earth dam, water pressure should be added to the effective active earth pressure causing movement. When thin, horizontal seams of cohesionless soil are present, the neutral stress must be computed in order to determine the effective strength and shearing resistance of the seam.

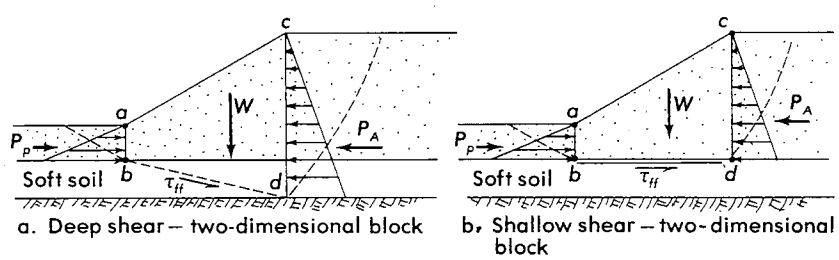

Figure 12.10 Sliding block analyses of a slope.

Stability problems in cracked soils or jointed rocks involve mult planes of weakness with different orientations. These cannot be analyzec the simple representation of a two-dimensional cross section, such as shc in Fig. 12.10. Instead, sliding takes place on two intersecting planes i direction parallel to their intersection (Fig. 12.11). The forces in suc three-dimensional system are most readily handled by vectors, as descril by Deere et al.[12:16] and Goodman.[12:17]

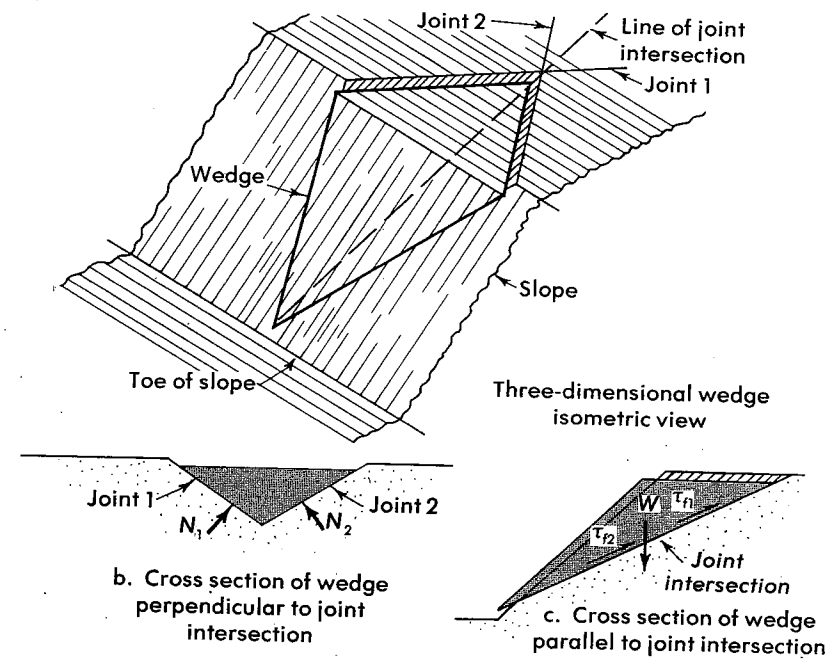

Figure 12.11 Wedge failure in jointed rock.

SHEAR STRENGTH FOR ANALYSIS / The shear strength c rock or soil varies greatly, depending on the environment and particularly o the degree of saturation, the effective stress and its change due to neutra stress variations, and the effects of progressive strain. Saturation destroy capillary tension and causes the buildup of neutral stresses under quickl applied loads. Therefore, if the environmental conditions indicate an possibility of saturation, the shear strength in that state is employed i analysis.

The effective stress depends on the rate of loading compared with th rate of drainage within the soil or rock. If the loads will be placed on the soi quickly, such as by rapid construction of a large embankment, the undrainec strength is used. If the embankment load is applied slowly, the undrainec strength should be safe, but the drained strength with proper consideratior

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Theoretical Soil Mechanics

From Theory to Practice in Soil Mechanics

THEORETICAL SOIL MECHANICS

By

KARL TERZAGHI

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(Fig. 50) for toe circles and $H/2$ for midpoint circles, regardless of the values of the theoretical maximum H_c (eq. 57(1)).

The effect of a tension crack (e_1f in Figure 50) on the stability of a slope is threefold. First of all the crack eliminates the resistance due to the co-

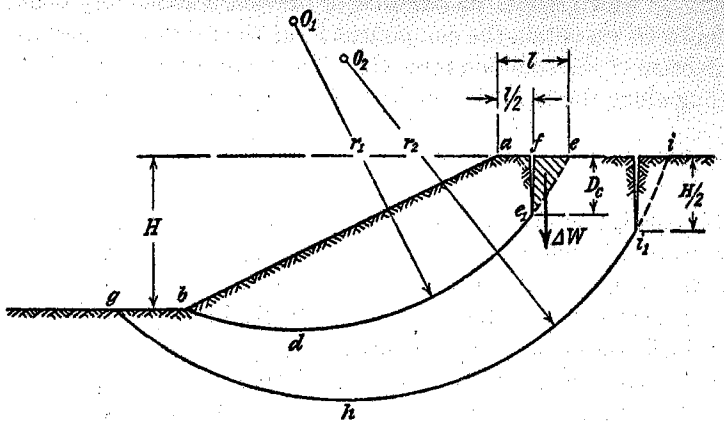

FIG. 50. Influence of tension cracks on stability of slope.

hesion along the arc e_1e. Hence, if c is the available cohesion per unit of area, the crack reduces the total cohesion per unit of length of the slope from $c \times \widehat{be}$ to $c \times \widehat{be_1}$. In order to produce the same effect, we can reduce the cohesion on $\widehat{be}$ from c to

$$c_a = c\,\frac{\widehat{be_1}}{\widehat{be}} \qquad [1]$$

without reducing the length of the arc. The second effect of the tension crack is to reduce the driving moment (weight of the mass of earth located above the surface of sliding $\times$ the lever arm with respect to the center of rotation) by the moment produced by the weight ΔW of the body efe_1 about O_1. The third effect is due to the hydrostatic pressure exerted by the surface water which accumulates in the cracks. This pressure increases the driving moment. However, computations have shown that the two last named effects practically compensate each other. Hence, if we replace the available cohesion c by the corrected cohesion, c_a (eq. 1), we can investigate the stability of the slope as if there were no cracks.

Applying the method of correction to a vertical slope on the assumption that the surface of sliding is plane, we find that the corrected cohesion c_a (eq. 1) is equal to one half of the available cohesion c. Replacing

c in equation 57(2a) by $c_a = c/2$, we get

$$H_c = 4\,\frac{c_a}{\gamma} = 2\,\frac{c}{\gamma} \qquad [2]$$

This equation takes the effect of an accumulation of surface water in the cracks into consideration. On the assumption that the cracks are not filled with surface water, we obtained

$$H_c' = 2.67\,\frac{c}{\gamma} \qquad 57(5)$$

63. Composite surfaces of sliding. If the base of a slope contains layers of clay which are very much softer than the other strata, the surface of sliding is likely to consist of several sections joining each other at obtuse angles. In such a case no continuous curve can be substituted for the entire surface of sliding without the risk of a grave error on the unsafe side. Furthermore no slide along a broken surface is conceivable without plastic flow within at least one section of the sliding mass, because the movement along a broken surface involves a radical deformation of the superimposed material. On account of these complications the problem of estimating the degree of stability of the fill cannot be solved without taking into consideration the forces which act in the interior of the mass of soil located above the potential surface of sliding. These forces are indicated in Figure 51 which represents a section through a sand dam with a clay core prior to filling the reservoir. The dam rests on a sand stratum which contains a thin bed of clay.

Even symmetrical dams slide toward one side only and the orientation of the slide with reference to the plane of symmetry depends on accidental factors. If the dam shown in Figure 51 fails, the slip will occur along a composite surface of sliding such as $abde$. In the right-hand part of the sliding mass, corresponding to the area d_1de, active failure is to be expected because within this part the earth is merely under the influence of its own weight. The central part b_1bdd_1 of the sliding mass will move under the influence of the active pressure on dd_1 along a surface of least resistance bd toward the left. On the left side of the toe of the dam, passive failure will occur within the thin layer of sand above the clay owing to the lateral thrust exerted by the advancing central part b_1bdd_1.

In order to investigate the conditions for the stability of the dam shown in Figure 51 we first determine the passive earth pressure P_P on several tentatively selected vertical sections in the vicinity of the left toe of the dam, such as the section bb_1. It is admissible to assume that

the pressure P_P acts in a horizontal direction, because the error due to this assumption is on the safe side. The central part b_1bdd_1 of the sliding mass cannot possibly advance toward the left unless it exerts on the vertical section bb_1 a horizontal pressure equal to or greater than the passive earth pressure P_P. Then we make a vertical section dd_1 through an arbitrarily selected point d on the left side of the clay core. The

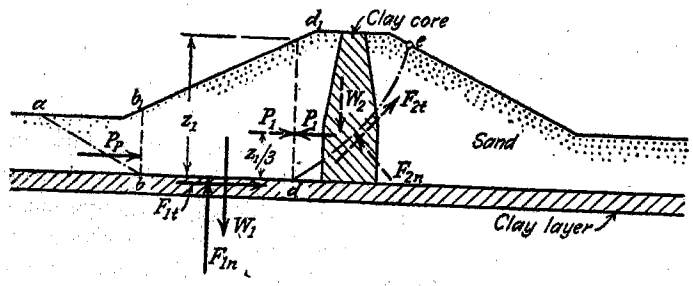

FIG. 51. Failure of fill along composite surface of sliding.

total resistance P_1 to a horizontal displacement of the body of soil bb_1d_1d with a weight W_1 is equal to the sum of P_P, the cohesion $c\,\overline{bd}$, and the frictional resistance F_{1t} against sliding along the base bd, or

$$P_1 = P_P + F_{1t} + c\,\overline{bd}$$

If the dam is on the verge of failure this resistance must be equal to or smaller than the active earth pressure exerted by the mass of earth located on the right-hand side of the section dd_1. Since the wedge d_1de can expand in a horizontal direction over its full height the distribution of the active earth pressure exerted by this wedge is hydrostatic. Therefore the point of application of the active earth pressure is located at an elevation $z_1/3$ above the surface of the clay. The shearing stresses along dd_1 can be neglected, because they increase the stability of the dam. The intensity of the active earth pressure on dd_1 can be determined by trial and error on the assumption that the surface of sliding, de, through d is an arc of a circle. The investigation must be repeated for different positions of point d (Fig. 51). The real surface of sliding satisfies the condition that the corresponding factor of safety G_s be a minimum.

64. Failure of fills by spreading. If the layer of soft clay shown in Figure 51 is located immediately beneath the base of the fill, the failure of the fill is likely to assume the character of a flow, whereby the fill spreads out laterally regardless of what the coefficient of internal friction of the fill material may be. The failure by spreading may either be

limited to the vicinity of the toes or else it may occur over the entire width of the base of the fill.

In order to determine the factor of safety of a fill with respect to a partial or a total failure by spreading it is necessary to investigate the intensity and the distribution of the shearing stresses on the base of the fill. These stresses depend to a certain degree on the state of stress in the fill. The following investigation is based on the assumption that the entire fill is on the verge of a complete failure by spreading. The shearing stresses obtained on the basis of this assumption represent the smallest stresses which are compatible with the conditions for the equilibrium of the fill.

65. Shearing stresses at the base of cohesionless fills. Rendulic (1938) has presented a simple method of determining the shearing stresses along the base of a cohesionless fill which is on the verge of active failure at every point. The shearing resistance along any section through the fill is assumed to be given by the equation

$$s = \sigma \tan \phi \qquad [1]$$

and the error due to assuming plane surfaces of sliding will be disregarded. Hence the earth pressure which acts in the fill can be determined by means of Coulomb's earth pressure theory.

Figure 52a shows a profile of the fill. The pressure on a vertical section ab produced by the fill material located on the right side of this section must be equal and opposite to the pressure exerted by the fill on the left side. This condition determines the angle δ between the resultant pressure on ab and its normal component. Since the angle δ is unknown it is advisable to use Engesser's graphical procedure (Art. 25) for determining the earth pressure on ab. According to this procedure, illustrated by Figure 21, plane sections are made through the foot b of the vertical section ab in Figure 52a, at different angles to the horizontal. The weights of the wedges located between ab and these sections are plotted on a vertical line (Fig. 52b) from point r in a downward direction. Thus for instance W_r (Fig. 52b) represents the weight of the wedge aba_r in Figure 52a, located on the right-hand side of ab. The slope of the base ba_r of this wedge has been arbitrarily selected. The wedge, with a weight W_r, is acted upon by the reaction F_r and by the earth pressure P_A. The reaction F_r acts at an angle ϕ to the normal on the inclined surface a_rb. F_r in Figure 52b has been traced through point t in Figure 52b parallel to F_r in Figure 52a. By repeating the same construction for wedges whose inclined base rises at different angles to the horizontal on the right-hand side of ab in

Soil Mechanics in Engineering Practice

Third Edition

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arrangement (Fig. 35.11). Inasmuch as the value of F determined in this manner refers to only one trial surface, the calculations must be repeated for other surfaces until the minimum value of F is found. Computer programs are available to expedite the calculations. However, to ensure that the results are reasonable and that no untenable assumption has been incorporated into the routine, a manual check by means of the foregoing procedure is advisable.

For most practical problems involving a noncircular surface of sliding, the assumption that ΔT_n is equal to zero leads to sufficiently accurate results. If the cross-section of the surface of sliding departs significantly from a circular shape, the use of Eq. 35.29 with $\Delta T_n = 0$ is preferable to the assumption of a circular cross-section and the use of Eq. 35.22. However, if greater refinement is justified, values of ΔT_n may be inserted in Eq. 35.29 and the factor of safety recalculated. The calculations are laborious and hardly practicable without a computer.

Because the interslice forces T and E depend on the deformation and the stress-deformation characteristics of the sliding mass, they cannot be evaluated by limit-equilibrium procedures, but values must be assigned to them arbitrarily. Any set of values T_n satisfying Eqs. 35.30 and 35.36 will ensure compliance with all conditions for equilibrium of the slide as a whole and for the horizontal and vertical equilibrium of each slice. However, not all such sets of values are reasonable or possible. For example, the values of T_n must not exceed the shearing strength of the soil along the vertical boundary of the corresponding slice under the influence of the normal force E_n. Moreover, tensile stresses should not occur across a significant portion of any vertical boundary between slices. These requirements form one basis for judging the reasonableness of a computer solution.

If the subsoil contains one or more thin exceptionally weak strata, the surface of sliding is likely to consist of three or more sections that do not merge smoothly one into another. In stability computations such a surface cannot be replaced by a continuous curve without the introduction of an error on the unsafe side.

Figure 35.12 represents a slope underlain by a thin layer of very soft clay with undrained shear strength s_u. If such a slope fails, the slip occurs along some composite surface *abcd*. In the right-hand part of the sliding mass, represented by the area *abf*, active failure must be expected because the earth stretches horizontally under the influence of its own weight. The central part *bce* moves to the left under the influence of the active pressure on *bf*. The left-hand part of the sliding mass *cde* exper-

Col. / Slice	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
	b	a	x	f	α	$\tan\alpha$	c	$\tan\phi'$	cb	ub	W_a	W_b	ΔT_n	$W_a + W_b + \Delta T_n$	(14) $- ub$	(15) $\tan\phi'$	(9) + (16)
1																	
2																	
:																	
n																	

Col. / Slice	18	19	20	21	22	23	24	25	26	27	28	29	30
	(17) $\cdot\, a$	$W_a + W_b$	(19) $\cdot\, x$	$ub \tan\phi'$	(21) $- cb$	(22) $\tan\alpha$	F_1	$\frac{(23)}{F_1}$	(14) + (25)	(26) $\cdot\, f$	m_α	$\frac{(18)}{m_\alpha}$	$\frac{(27)}{m_\alpha}$
1													
2													
:													
n													

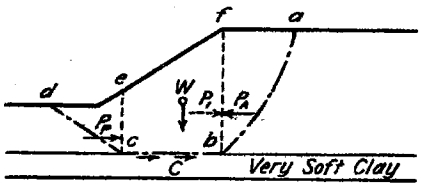
$\Sigma(20) =$ $\qquad$ $\Sigma(29) =$

$$F_2 = \frac{\Sigma(29)}{\Sigma(20) - \Sigma(30)}$$

$\Sigma(30) =$

Repeat steps 24 to 30 incl.

Figure 35.11 Tabular form for calculating factor of safety of slope by method of slices, if surface of sliding is not circular.



igure 35.12 Failure of slope underlain by thin layer of very ıft clay.

ıces passive failure due to the thrust of the advancing ıntral part *bcef*.

The first step in investigating the conditions for the ability of the slope is to compute the passive earth essure P_P of the soil located on the left side of a tenta- ely selected vertical section *ec* located near the toe of e slope. It is conservative to assume that P_P acts in e horizontal direction. The next step is to estimate the ısition of the right-hand boundary *b* of the horizontal ırt *cb* of the potential surface of sliding and to compute e active earth pressure P_A on a vertical section *fb* through The tendency for the mass *bcef* to move to the left is sisted by the passive earth pressure P_P and the total drained shear strength *C* along *bc*. If the slope is stable, e sum of these resisting forces must be greater than the tive earth pressure P_A, which is assumed to act in a rizontal direction. The factor of safety against sliding equal to the ratio between the sum of the resisting ces and the force P_A. The investigation must be repeated different positions of the points *c* and *b* until the face of least resistance to sliding is found that corres- nds to the least factor of safety.

oblems

1. A wide cut was made in a stratum of soft clay that had vel surface. The sides of the cut rose at 30° to the horizontal. drock was located at a depth of 12 m below the original und surface. When the cut reached a depth of 7.5 m, failure urred. If the unit weight of the clay was 19.2 kN/m³, what s its mobilized undrained shear strength? What was the racter of the surface of sliding? At what distance from the t of the slope did the surface of sliding intersect the bottom he excavation?

Ans. 24 kPa; midpoint circle; 5.25 m.

2. The rock surface referred to in problem 1 was located at epth of 9 m below the original ground surface. What were mobilized undrained shear strength of the clay and the racter of the surface of sliding?

Ans. 22 kPa; toe circle.

3. A cut is to be excavated in soft clay to a depth of 9 m. material has a unit weight of 18.3 kN/m³ and a s_u (UC) 4 kPa. A hard layer underlies the soft layer at a depth of n below the original ground surface. What is the slope angle hich failure is likely to occur?

Ans. $\beta = 69°$.

4. A trench with sides rising at 80° to the horizontal is excavated in a soft clay which weighs 19.2 kN/m³ and has an undrained shear strength of 12 kPa. To what depth can the excavation be carried before the sides cave in? At what distance from the upper edge of the slope will the surface of sliding intersect the ground surface?

Ans. 2.75 m; 2.46 m.

5. A bed of clay consists of three horizontal strata, each 4.5 m thick. The values for s_u for the upper, middle, and lower strata are, respectively, 30, 20, and 140 kPa. The unit weight is 18.4 kN/m³. A cut is excavated with side slopes of 1 (vertical) to 3 (horizontal) to a depth of 6 m. What is the factor of safety of the slope against failure?

Ans. 1.3.

6. To what depth can the trench in problem 4 be excavated without bracing if the soil is located above water table, and has a cohesion intercept *c* of 12 kPa and an angle of internal friction of 20°?

Ans. 4.25 m.

7. To what depth can a permanent trench in problem 4 be excavated without bracing if the clay bed is completely submerged and the effective cohesion intercept *c'* is 4 kPa and the angle of internal friction is 20°?

Ans. 2.96 m.

Selected Reading

An excellent overview of the methods of stability analysis and the assumptions on which they are based is given by D. Nash, "A Comparative Review of Limit Equilibrium, Methods of Stability Analysis," Chap. 2 "Slope Stability," Anderson, M. G. and K. S. Richards, Eds., Wiley, 1987. A similar comparison is found in Fredlund, D. G. and J. Krahn, "Comparison of Slope Stability Methods of Analysis," *Can. Geot. J.*, (1977), **14**, 429–439.

Lafleur, J. and G. Lefebvre (1980). "Groundwater regime associated with slope stability in Champlain clay deposits." *Can. Geotech. J., 17,* No. 1, pp. 44–53.

Charles, J.A. and M.M. Soares (1984). "The stability of slopes in soils with nonlinear envelopes." *Can. Geotech. J., 21,* No. 3, pp. 397–406.

ARTICLE 36 STABILITY OF EARTH DAMS

36.1 Critical States for Design

The factor of safety of an earth dam with respect to a slope or foundation failure depends to a large extent on the porewater pressures. In a dam of given cross-section on a given foundation, the intensity and distribution of the porewater pressures vary with time between wide limits. For purposes of design it is convenient to distinguish among the porewater pressure conditions corresponding to three stages: during construction, and especially immediately after construction has been completed; after the reservoir has been full long enough to

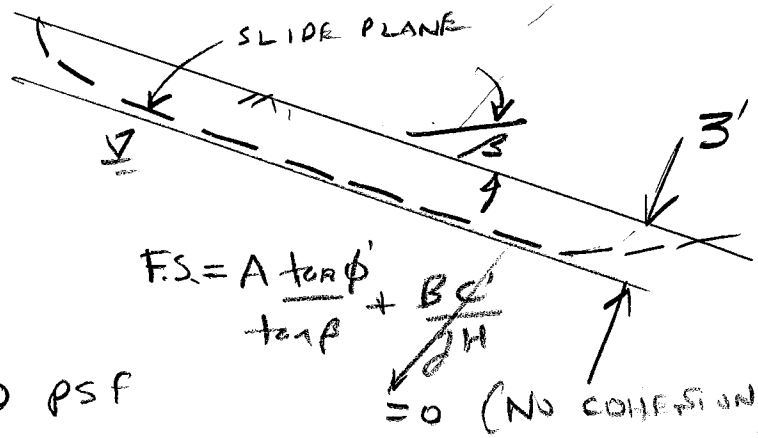
AECOM

Calculation Sheet

Project	KINGSTON – RCA			Subject	INFINITE SLOPE STABILITY		
Originated By	Date	Checked By	Date	AECOM Job No.	Scale		Sheet No.
WHW	4/29/09	LWD	5/4/09	60095742	NTS		1 Of 4

COMPUTE SLIDING STABILITY OF

COMPACTED ASH DIKE > 95% STD PROCTOR



$\gamma_T = 110$ pcf

$\gamma_{MOIST} = 100$ pcf

$\gamma_{WATER} = 62.4$ pcf

$\phi = 37°$, $C = 0$ psf

$F.S. = A\frac{\tan\phi'}{\tan\beta} + B\frac{c'}{\gamma H}$ $= 0$ (NO COHESION)

SLOPE 3H : 1V $\beta = 18.4°$

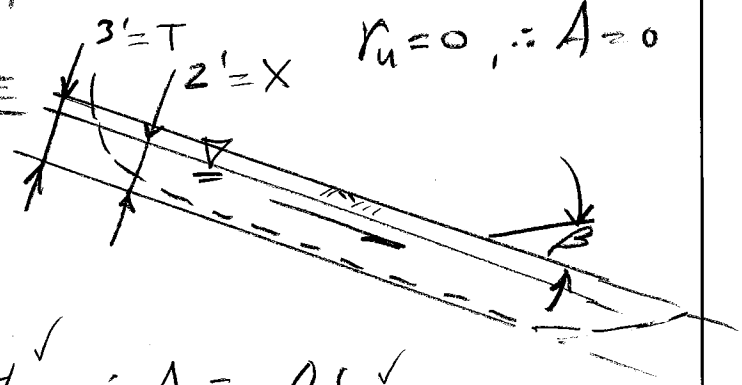
REF. USACE EM 1110-2-1902 31 OCT 03 FIGURE E-7

I. DRY SLOPE

$$FS = \frac{\tan\phi}{\tan\beta} = \frac{\tan 37°}{\tan 18.43°} = \frac{0.754}{0.333} = 2.26 = 2.3$$

$r_u = 0$, $\therefore A = 0$

II. SEEPAGE PARALLEL TO SLOPE



$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma_T}\cos^2\beta$$

$$r_u = \frac{2}{3}\left(\frac{62.4}{110}\right)\cos^2 18.4° = 0.34 \quad \therefore A = 0.6$$

$$b = \cot\beta = \frac{1}{\tan\beta} = 3.0 \quad \therefore B = 3.3$$

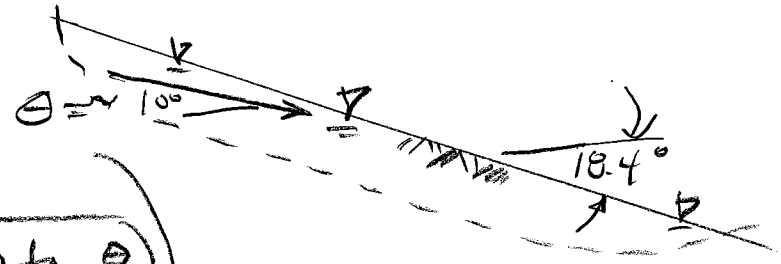
$$FS = A\frac{\tan\phi}{\tan\beta} + B\frac{c}{\gamma_T H} = 0.6\left(\frac{0.754}{0.333}\right) = 1.36 = 1.4$$

AECOM

Calculation Sheet

Project	KINGSTON			Subject	INFINITE SLOPE STABILITY		
Originated By	Date	Checked By	Date	AECOM Job No.	Scale	Sheet No.	
GMW	4/29/09	EED	6/2/09	60095742	NTS	2 Of 4	

III. SEEPAGE EMERGING FROM SLOPE



$$r_u = \frac{\gamma_w}{\gamma_T}\left(\frac{1}{1+(\tan\beta \tan\theta)}\right)$$

$$r_u = \frac{62.4}{110}\left(\frac{1}{1+\tan 18.4^\circ \tan 10^\circ}\right) = 0.54 \checkmark$$

$b = 3.0 \quad \therefore A = 0.38 \quad , C = 0$

$$FS = 0.38 \frac{\tan 37^\circ}{\tan 18.4^\circ} = 0.86 \cong 0.9 \checkmark$$

IV. WET SLOPE, SEEPAGE PARALLEL TO SLOPE

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta = \frac{1}{1}\left(\frac{62.4}{110}\right)\cos^2(18.4) = 0.51 \checkmark$$

$b = 3.0 \; ; \; A = 0.42$

$$FS = 0.42 \frac{\tan 37^\circ}{\tan 18.4^\circ} = 0.95 \cong 1.0 \checkmark$$

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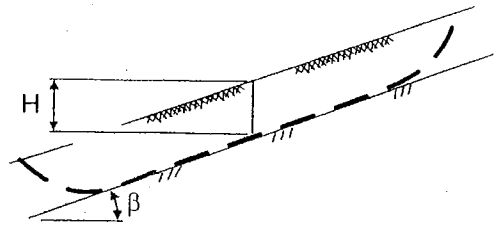
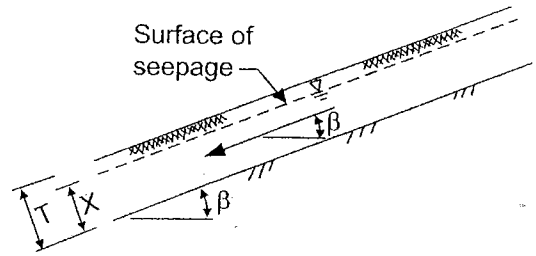
ENGINEERING AND DESIGN

Slope Stability

ENGINEER MANUAL





γ = total unit weight of soil

γ_w = unit weight of water

c' = cohesion intercept

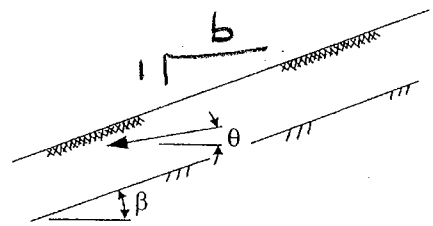
ϕ' = friction angle

r_u = pore pressure ratio = $u/\gamma H$

u = pore pressure at depth H

Seepage parallel to slope

$$r_u = \frac{X}{T}\frac{\gamma_w}{\gamma}\cos^2\beta$$

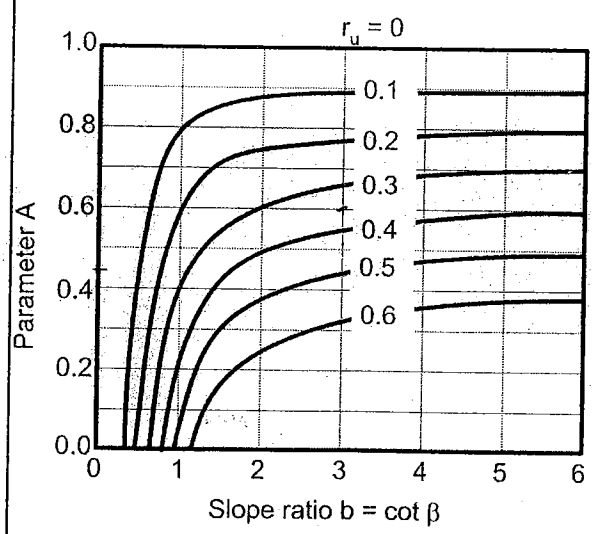


Seepage emerging from slope

$$r_u = \frac{\gamma_w}{\gamma}\frac{1}{1+\tan\beta\tan\theta}$$

Steps:

1. Determine r_u from measured pore pressure or formulas at right.
2. Determine A and B from charts below.
3. Calculate $F = A\dfrac{\tan\phi'}{\tan\beta} + B\dfrac{c'}{\gamma H}$



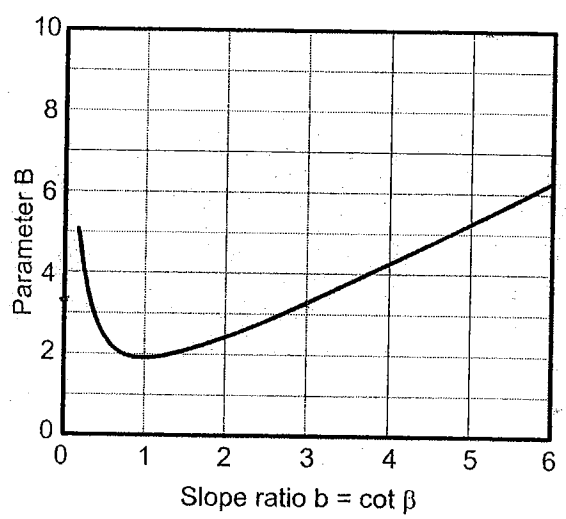


Figure E-7. Slope stability charts for infinite slopes (after Duncan, Buchianani, and DeWet 1987)

$\cot\beta = \frac{1}{\tan\beta}$

AECOM

Calculation Sheet

Project	Subject				
KINGSTON-RCA	FLOW SLIDE Analysis for Su or Sr				
Originated By: MAW	Date: 4/29/09	Checked By: LWB	Date: 5/4/09	AECOM Job No.: 60095742	Scale: NTS	Sheet No. 1 Of 4

INFINITE SLOPE ANALYSIS TO COMPUTE S_{us} or S_r

THE POST ASH FLOW UNDRAINED SHEAR STRENGTH

USING THE FORMULA

$$\frac{S_{us} \text{ or } S_r}{\sigma'_{vo}} = \frac{\gamma_T}{\gamma'} \cos\theta \sin\theta$$

from Jefferies & Been 2006, pg. 424

assumes ∇ @ surface after failure

θ slope of slide mass after failure

σ'_{vo} = Initial effective stress over slide plane $= \gamma' H$

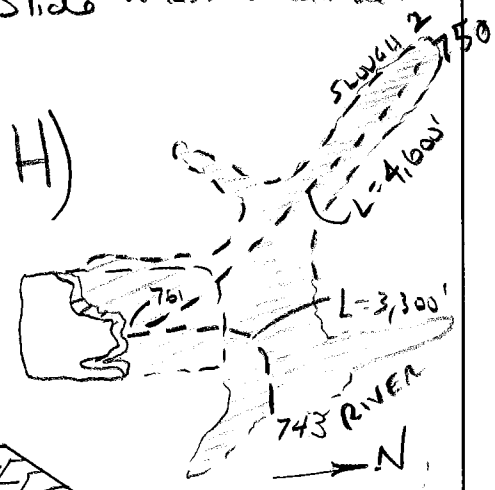
γ_T = Total Unit Weight of Slide Mass Material

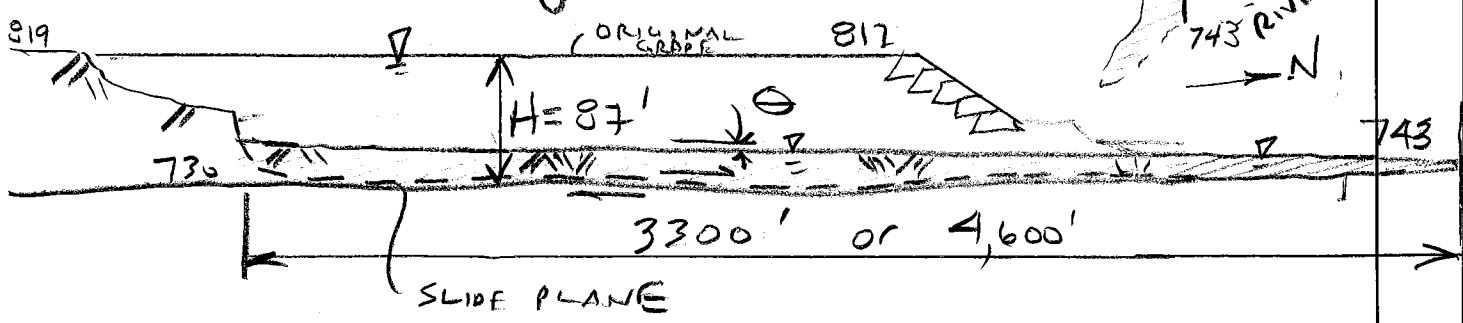
γ' = Submerged Unit Weight of Slide Mass Material

$\gamma' = \gamma_T - \gamma_w$

$$\therefore S_r = S_{us} = \frac{\gamma_T \cos\theta \sin\theta}{\gamma'}(\gamma' H)$$

$$S_r = S_{us} = \gamma_T H \cos\theta \sin\theta$$





AECOM

Calculation Sheet

Project	Kingston - RCA			Subject	FLOW SLIDE Analysis for Sus or Sn		
Originated By	Date	Checked By LWB	Date 5/4/09	AECOM Job No. 60095742	Scale NTS		Sheet No. 2 Of 4

REACH TO RIVER

$\theta = \tan^{-1}(18'/3{,}300') = 0.31°$ ✓

$S_r = S_{us} = 107\ pcf\ (87')\ \cos 0.31°\ \sin 0.31°$

$S_r = S_{us} \cong 50$ ✓ $psf < 100$ ✓ psf

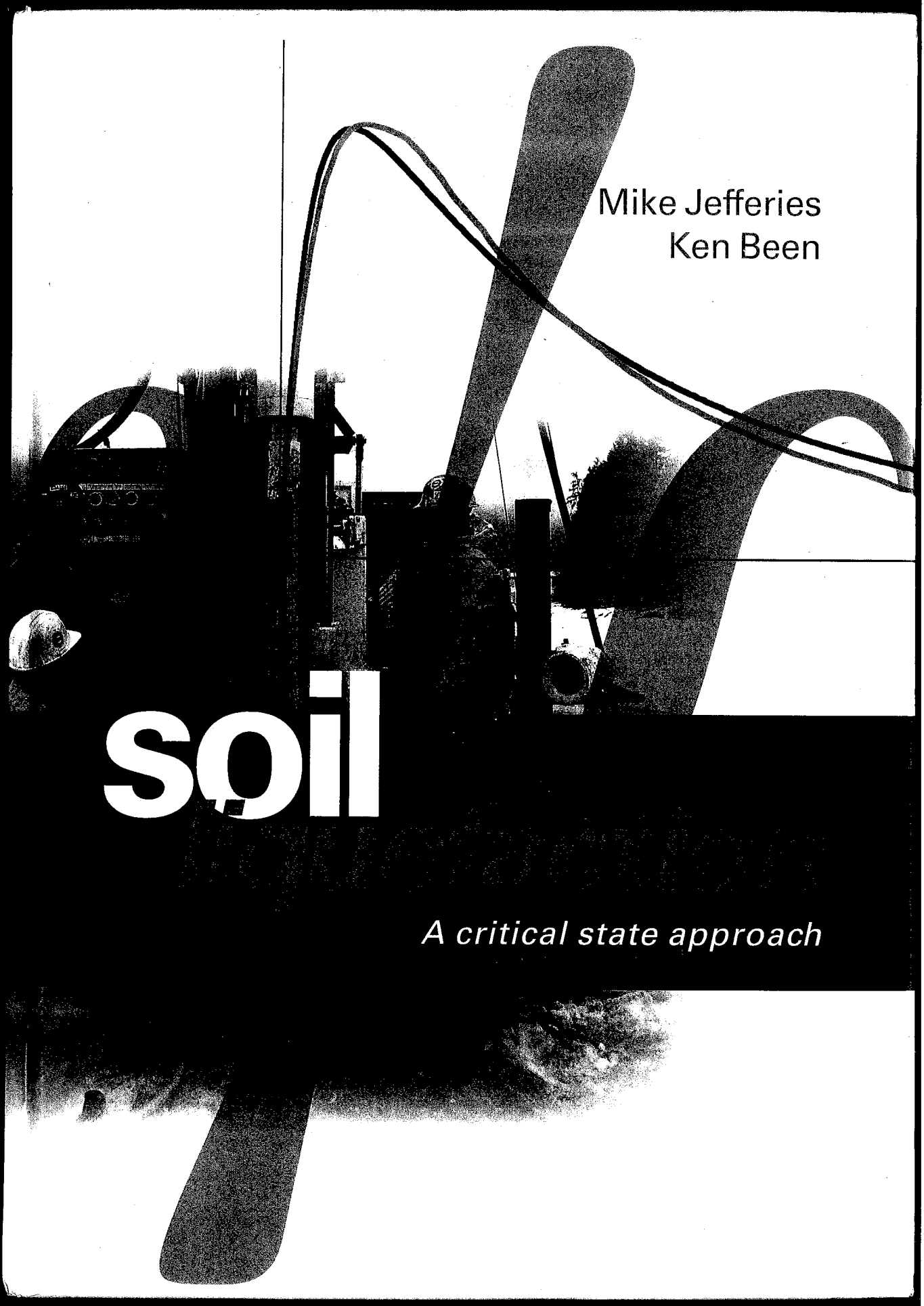
REACH TO SLOUGH 2

$\theta = \tan^{-1}(11'/4{,}600') = 0.14°$ ✓

$S_r = S_{us} = 107\ (87')\ \cos 0.14°\ \sin 0.14°$

$S_r = S_{us} = 23\ psf$ ✓ $< 100\ psf$

The S_{us} or S_r ranges from ~25 to 50 psf



Mike Jefferies
Ken Been

soil

A critical state approach

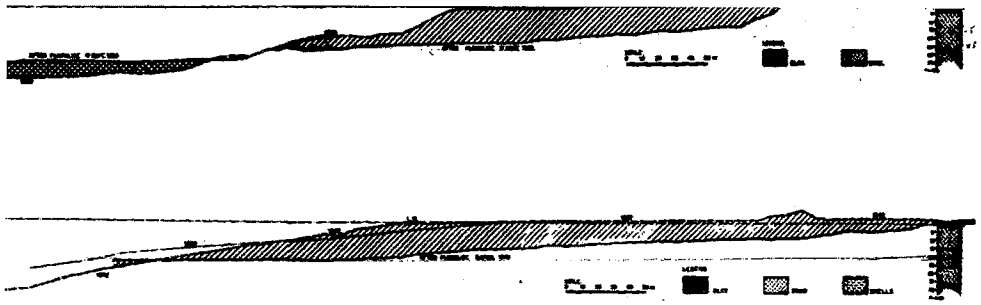


Figure F.2 Vlietpolder flowslide geometry (Koppejan et al., 1948)

Koppejan et al. distinguish between slope failures cause by toe erosion and true flow slides, both occurring in Zeeland. Flowslides were noted as causing unexpected sliding of a large portion of the foreshore and sometimes taking part of the flood prevention dyke with it. Zeeland flowslides are somewhat gradual with soil masses sliding downward and out at intervals of a few minutes, although these observation are only possible once the slide is well-established with the scarp visible above water level. The rearward regression rate is typically about 50 m/hour with the slide taking as much as a day from start to completion. An example of a flowslide geometry is shown on Figure F.2, this being from an 1889 slide at Vlietpolder involving nearly 1 million m^3 of soil. The steepest slope prior to failure was 27° while the post failure slope was about 4°.

The soils involved in the flowslides are predominantly fine uniform sand of the Older Holocene formation and with 90% of the gradation within the particle size range 70 microns to 200 microns. The *in situ* state can be judged from four CPT soundings presented by Koppejan et al. and which are reproduced on Figure F.3. The usual variability of tip resistance in sand is apparent as is the clear trend for increased penetration resistance with depth (which is arguably linear). Adopting the criterion that about the 80 – 90 percentile strength of soil controls its characteristic behaviour, the range of normalized penetration resistance that has been involved in flowslides is $30 < Q_k < 50$. These characteristic Q_k values are seemingly constant with depth.

Most interestingly, critical density tests were carried out on the Older Holocene sands. A critical porosity of 47.5% was quoted as differentiating between flow slides and non-flow failures. These critical density tests determine the volumetric strain caused by shear alone and are not the same as modern critical state tests in the US/UK usage (Chapter 2), with the Netherlands critical density being typically at around $\psi \approx -0.05$. It is therefore consistent that Koppejan et al. report a peak friction angle of $\phi = 37°$. Since the flowslides initiated in the foreshore, and the soil profile of apparently loose sand extends to substantial depth, the residual strength is estimated using infinite slope analysis. This type of analysis is documented in standard texts (e.g. Lambe and Whitman, 1968) and gives, assuming the water table is at ground surface (a reasonable assumption given it was a coastal foreshore that failed at low water):

pg 353

$$\frac{s}{\sigma'_{v0}} = \frac{\gamma_t}{\gamma'} \cos\theta \sin\theta \qquad [F.1]$$

Applying this equation and using the reported final slope of 4° gives an estimated strength ratio $s_r/\sigma'_{v0} \approx 0.13$ assuming no model uncertainty in the rigid-plastic infinite

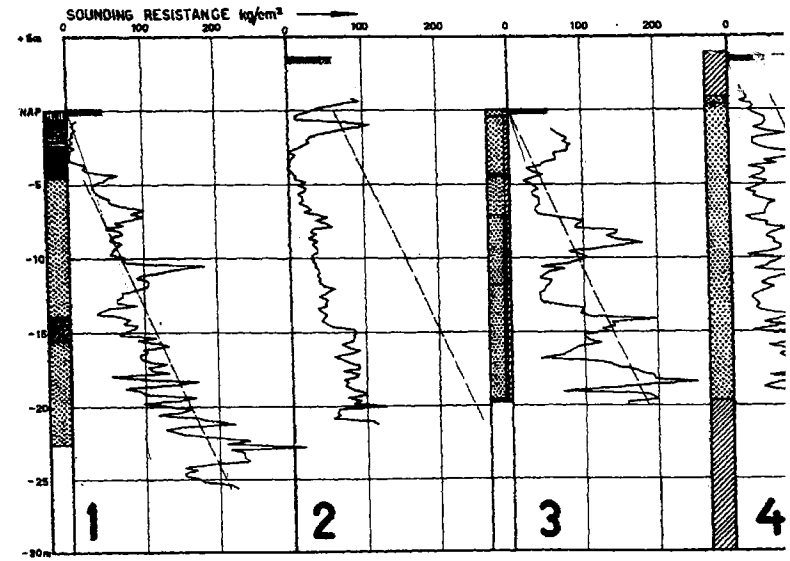


Figure F.3 Typical CPT soundings in flowslide material (Koppejan et

The last aspect to consider is the estimated CSL parameters and The sands involved in the slides are medium to fine sand and with only is not dissimilar to some of the dredged sands used in the Beaufort is and whose data has been presented at some length in Chapter 2. Look typical slope to the CSL would be about $\lambda_{10} \approx 0.06$ and $M_{tc} \approx 1.25$ (w with the reported $\phi = 37°$ because of the difference between the Dutc and the true critical state). The corresponding ranges for the CPT coe depth range of 5–20 m below ground surface are about $25 < k < 35$ ai range of CPT coefficients has been calculated using calibration chambe 4.19) at the low end and at the upper end using equation [4.12] w hardening parameter $H \approx 150$ and elastic rigidity of $I_r \approx 600$. Neither H controversial for a clean sand with only traces of silt (see the calibrat Chapter 3). Adopting $0.7 < K_0 < 0.9$ (this was normally consolidated ground), the estimated characteristic *in situ* state range is about $-0.09 <$ CPT data shown on Figure F.3 indicates some of the zones at the l spectrum are rather extensive.

F.2 1907: WACHUSETT DAM, NORTH DYKE (MASSACHUSE

Wachusett Dam is some 48 km from Boston and retains about 240 mil Even today, it remains an important water supply reservoir for the cit most important reservoir before 1939. However, an adjacent saddle da the North Dyke, failed with an upstream flow slide during first filling c 1907. What follows is based on Olson et al. (2000), this paper being comprehensive evaluations of residual strength back analyzed from a sta is found in a journal. As such, it is a good example of how such case h

Soil Mechanics

T. William Lambe • Robert V. Whitman

Massachusetts Institute of Technology

1969

John Wiley & Sons, Inc.

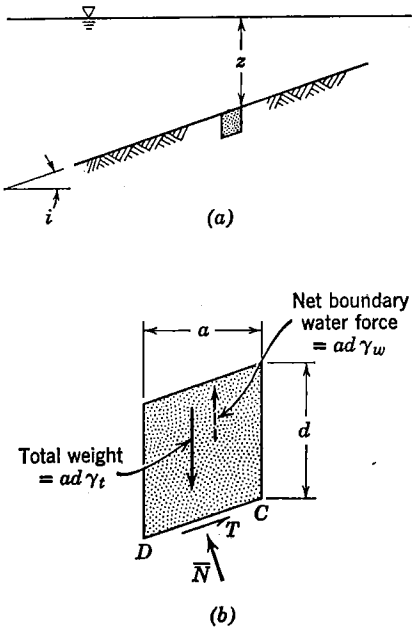
New York London Sydney Toronto

CD. The bar indicates that $\bar{N}$ is the resultant of the *effective* stresses.

3. The resultant of the shear stresses around the boundary. The shear stresses on the vertical side of the element must cancel each other, leaving only the resultant T of the shear stresses on CD. Since the pore water can carry no shear, T must be carried entirely by the mineral skeleton.
4. The resultant of the boundary pore pressures. Since there is no seepage, this force is buoyancy (as proved in Chapter 17), which is the volume of the element times the unit weight of the water.

Some readers may be puzzled by the fact that the resultant of the boundary pore pressures is vertical even when the top and bottom of the unit element are not horizontal. Figure 24.2 shows the actual boundary pore pressures and proves that the resultant is indeed vertical. Note that the pressures vary along the top and bottom of the element as well as along the sides of the element, and that the resultants against the two vertical sides are not equal. (In other words, buoyancy acts upward regardless of the shape of the submerged mass.)

Because the resultant of the boundary pore pressures is vertical, the final result of the analysis is the same as in Fig. 13.33 except that the buoyant unit weight replaces the total unit weight. The final result is that the slope is stable for $i \leq \bar{\phi}$, i.e., the maximum value of i is the strength angle $\bar{\phi}$ in terms of effective stresses. Thus the maximum stable slope angle is the same for a given sand whether the slope is completely dry or completely submerged under water.



By analysis of Fig. 13.33,

$$\bar{N} = ad\gamma_b \cos i$$
$$T = ad\gamma_b \sin i$$
$$\bar{\sigma} = \gamma_b d \cos^2 i$$
$$\tau = \gamma_b d \cos i \sin i$$

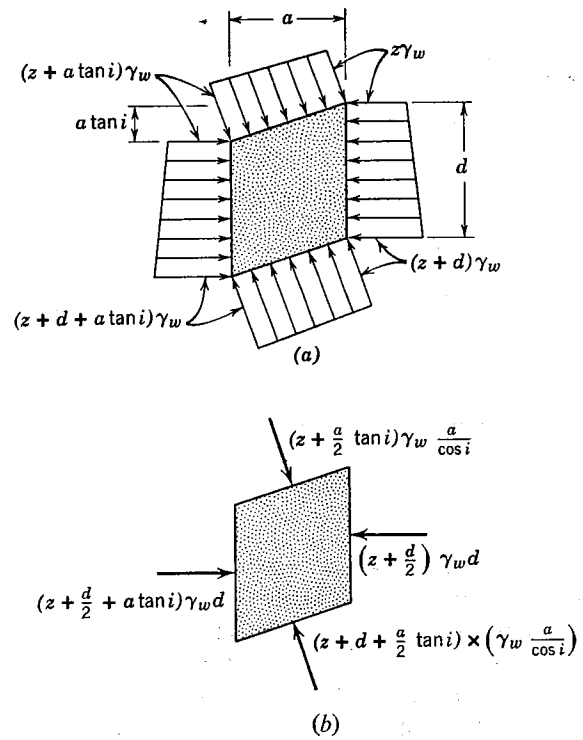
If full resistance is mobilized so that $\tau = \bar{\sigma} \tan \bar{\phi}$, then

$$i = \bar{\phi}$$

Fig. 24.1 Analysis of submerged infinite slope. (*a*) Submerged slope. (*b*) Analysis of equilibrium.

Seepage Parallel to Slope

Figure 24.3 shows the case of a slope with seepage parallel to the slope. The flow net consists of straight lines with the flow lines parallel to the slope and the equipotential lines perpendicular to the slope. Such a condition is often reached in the lower portions of natural slopes, as noted in Fig. 24.4.



Sum of horizontal forces:

$$\left(z + \frac{a}{2}\tan i\right)\gamma_w \frac{a}{\cos i}\sin i - \left(z + d + \frac{a}{2}\tan i\right)\gamma_w \frac{a}{\cos i}\sin i + \left(z + \frac{d}{2} + a\tan i\right)\gamma_w d - \left(z + \frac{d}{2}\right)\gamma_w d = -d\gamma_w \frac{a}{\cos i}\sin i + a\gamma_w d \tan i = 0$$

Sum of vertical forces:

$$\left(z + \frac{a}{2}\tan i\right)\gamma_w \frac{a}{\cos i}\cos i - \left(z + d + \frac{a}{2}\tan i\right)\gamma_w \frac{a}{\cos i}\cos i = ad\gamma_w$$

Fig. 24.2 Resultant boundary pore pressure for submerged slope. (*a*) Boundary pore pressures. (*b*) Resultants of boundary pore pressures.