TABLES

Table 1
Summary of Index Testing Results

Soil Type	Boring/Pit Number	Sample Depth (ft)	USCS Soil Classification	Moisture Content (%)	In-Situ Dry Density (pcf)	Specific Gravity	Atterberg Limits			Grain Size Distribution			Moisture/Density Relationship Standard		Jar Slake Index	Minimum Density (pcf)	Maximum Denisty (pcf)
							LL	PL	PI	% Finer 3/4"	% Finer #4	% Finer #200	Dry Density (pcf)	Moisture (%)			
Native Soils																	
Minturn Formation	GA-11B-20	32-34	SM	--	--	--	NP	NP	NP	85	51	17	--	--	--	--	--
	GA-11B-21	28	SC	--	--	--	28	15	13	98	88	49	--	--	--	--	--
Lincoln Porphyry	GA-11B-20	40-43	SC	--	--	--	28	18	10	90	63	21	--	--	--	--	--
	GA-11B-25	178	--	15.0	--	--	--	--	--	--	--	--	--	--	--	--	--
Glacial Till	GA-11B-24	132	GC	--	--	--	31	17	14	76	58	25	--	--	--	--	--
	GA-11B-25	104-106	--	7.8	--	--	--	--	--	--	--	--	129.5[1]	8.7	--	--	--
	GA-11B-23	249-251	--	6.9	--	--	--	--	--	--	--	--	--	--	--	--	--
Waste Rock Materials																	
Waste Rock	GA-11P-05	3-6	GP-GM	8.9	--	--	33	17	16	66	32	9	--	--	6	--	--
	GA-11B-25	16	GC	6.0	--	--	28	15	13	68	39	14	--	--	6	--	--
Tailing Materials																	
Tailing Beach	GA-11B-24	91-92	SM	--	--	--	NP	NP	NP	100	100	36	119.0[2]	10.1	--	73.1	120.7[3]
	GA-11B-25	43.5	SM	--	--	--	NP	NP	NP	100	100	26			--		
	GA-11B-25	55	SM	--	--	--	NP	NP	NP	100	100	25			--		
	GA-11B-25	42	--	26.9	97.8	--	--	--	--	--	--	--			--		
Tailing Fines	GA-11B-24	51.5-53.6	ML	31.9	--	2.75	NP	NP	NP	100	100	99	--	--	--	--	--
	GA-11B-24	71-73.2	ML	30.6	86	--	NP	NP	NP	100	100	97	--	--	--	--	--
	GA-11B-23	190-192	--	25.5	100.3	--	--	--	--	--	--	--	--	--	--	--	--

[1] Maximum dry density and corresponding moisture content determined by the standard Proctor method.
[2] Maximum dry density and corresponding moisture content determined by the modified Proctor method.
[3] Maximum dry density determined by compaction in a Proctor mold utilizing maximum compactive effort performed on soil at 2% below the optimum moisture content.

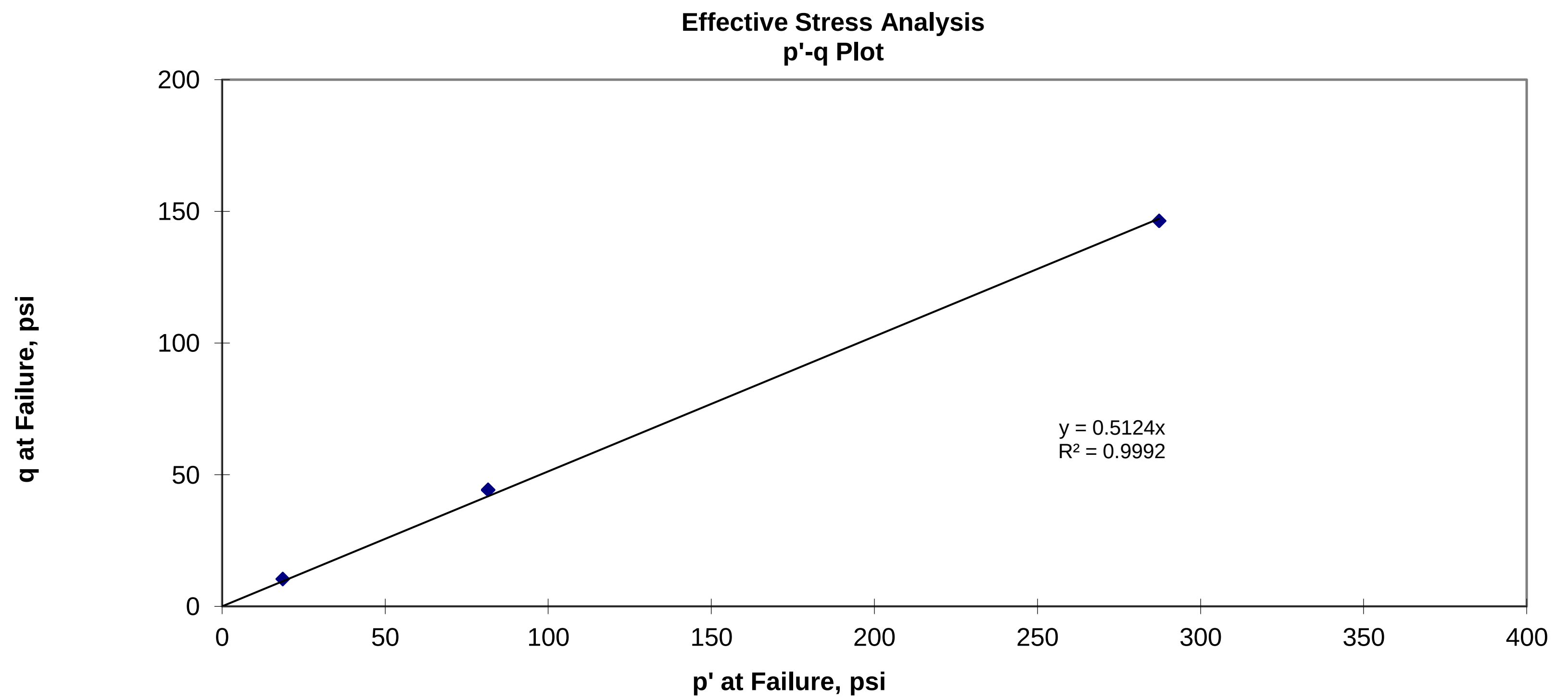
Table 2
Summary of Strength Testing Results

Soil Type	Boring/Pit Number	Sample Depth (ft)	Test Type	Peak		Residual	
				Cohesion (psi)	Friction Angle (degrees)	Cohesion (psi)	Friction Angle (degrees)
Native Materials							
Minturn Formation	GA-11B-21	28	Staged CU Triaxial	0	31	--	--
Lincoln Porphyry	GA-11B-20	40-43	Large Scale Direct Shear	16	38	18	35
Glacial Till	GA-11B-25 GA-11B-23	104-106 249-251	Large Scale Direct Shear	17	31	14	31
Waste Rock Materials							
Waste Rock	GA-11P-05	3-6	Large Scale Direct Shear	9	37	0	35
Waste Rock	GA-11B-25 GA-11B-23	16 38-40	Large Scale Direct Shear	6	37	0	36
Tailing Materials							
Tailing Fines	GA-11B-24	51.5-53.6	CU Triaxial	0	36	0	33

Table 3
Summary of Consolidation Testing Results

Soil Type	Boring/Pit Number	Sample Depth (ft)	Test Type	Initial		Final		Average Cv (ft^2/day)	Recompression Index C_r	Compression Index C_c	Swell Index C_s
				Void Ratio	Dry Density (pcf)	Void Ratio	Dry Density (pcf)				
Tailing Fines	GA-11B-24	51.5-53.6	One-Dimensional Consolidation	0.9	90.1	0.78	96.6	2.63	0.03	0.11	0.02

FIGURES



tan(ψ') =	**0.5124**	
a' =	**0**	**psi**
ϕ' =	**30.8**	**degrees**
c' =	**0**	**psi**

Golder Associates
Denver, Colorado

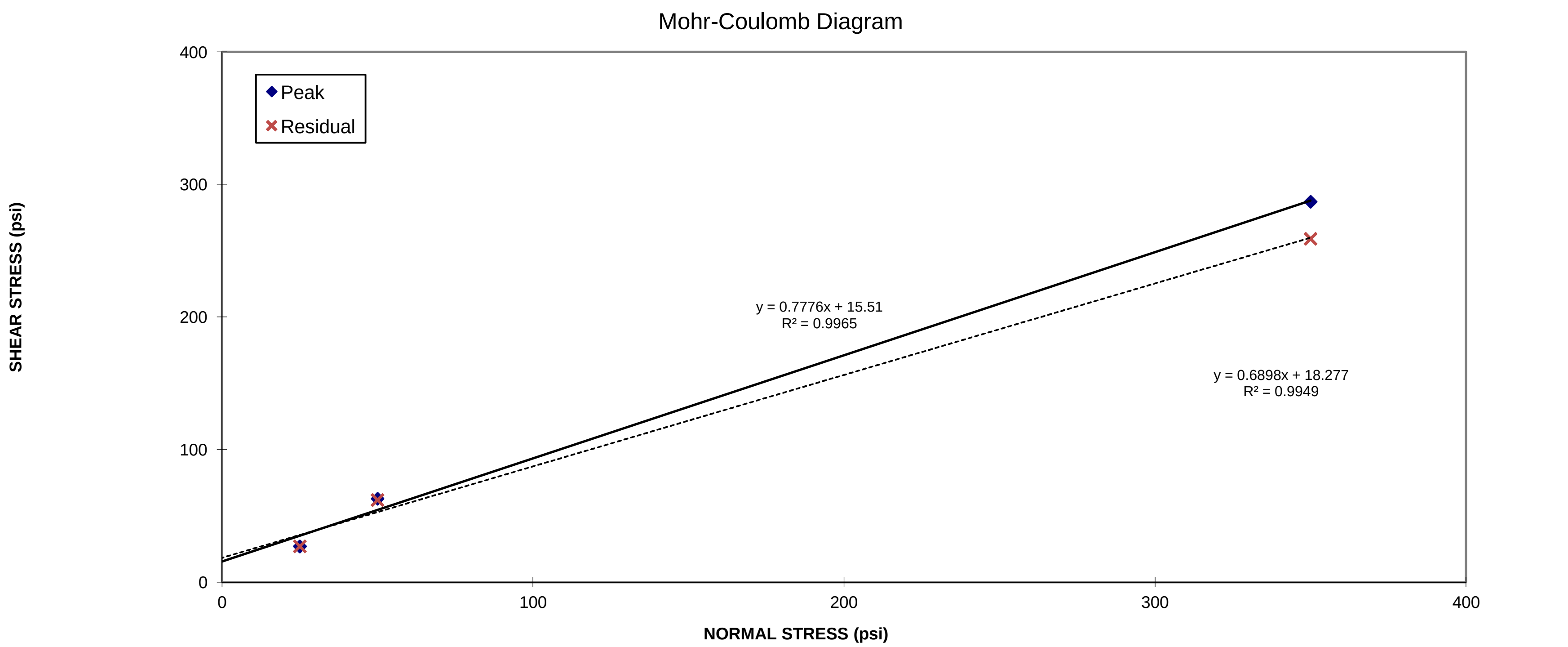
TITLE

Effective Stress Parameters for Staged Triaxial Test on Minturn Clay (GA-11B-21 @ 28 ft)

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	1



tan(ϕ')=	0.7776	
ϕ'=	37.9	degrees (peak)
C'=	16	psi (peak)
tan(ϕ')=	0.6898	
ϕ'=	34.6	degrees (residual)
C'=	18	psi (residual)

Golder Associates — Denver, Colorado

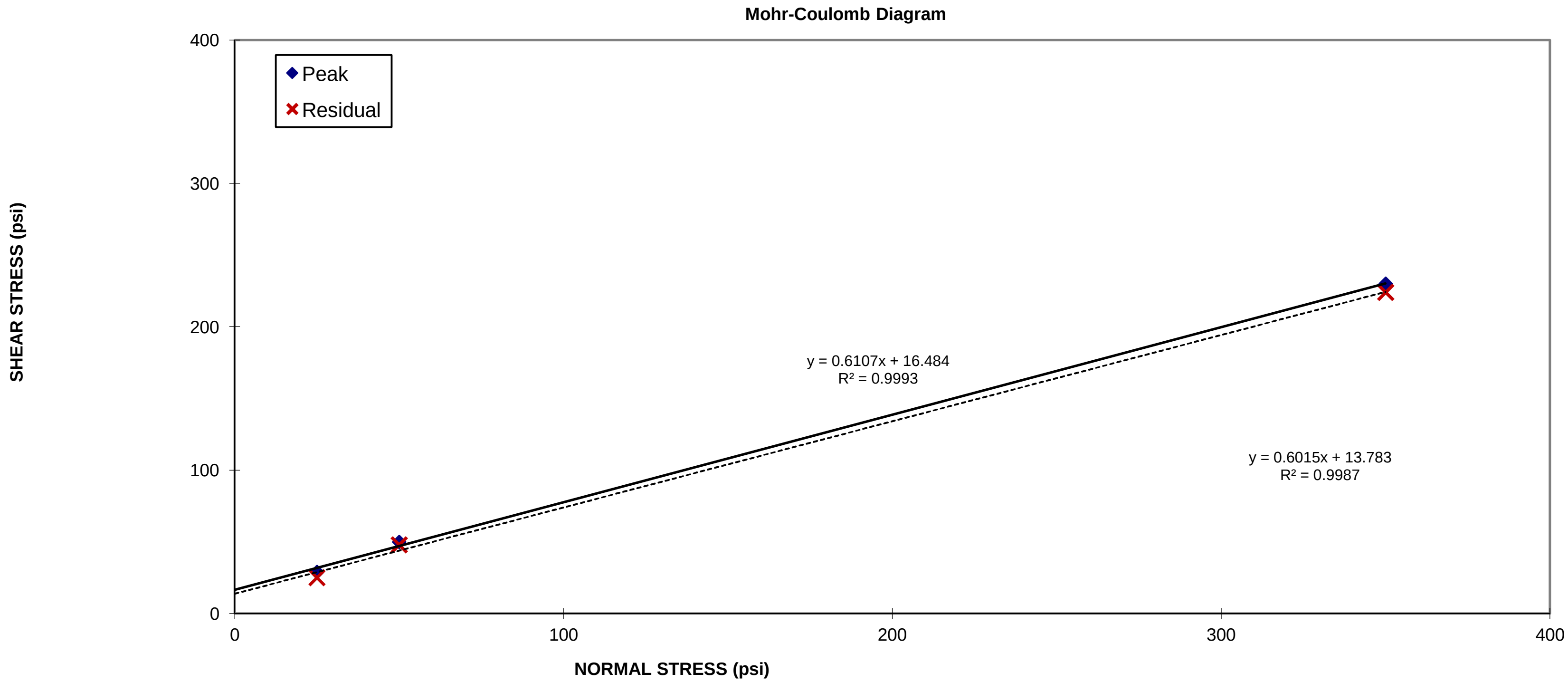
TITLE

Mohr-Coulomb Diagram for Large Scale Direct Shear on Lincoln Porphyry (GA-11B-20 @ 40-43 ft)

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	2



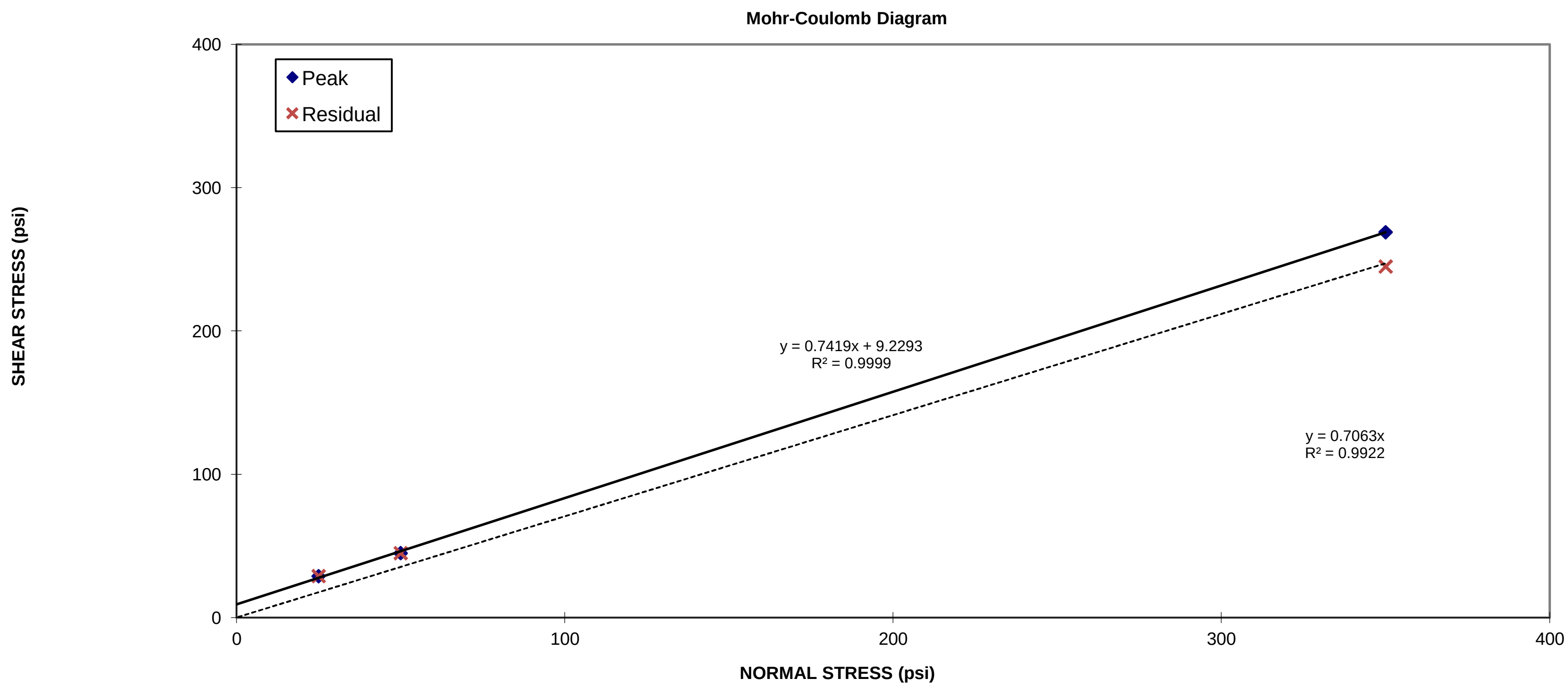
tan(ϕ')=	0.6107	
ϕ'=	31.4	degrees (peak)
C'=	16	psi (peak)
tan(ϕ')=	0.6015	
ϕ'=	31.0	degrees (residual)
C'=	14	psi (residual)

Golder Associates
Denver, Colorado

TITLE
Mohr-Coulomb Diagram for Large Scale Direct Shear on Glacial Till (GA-11B-25 @ 104-106 ft/GA-11B-23 @ 249-251 ft)

CLIENT/PROJECT
Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	3



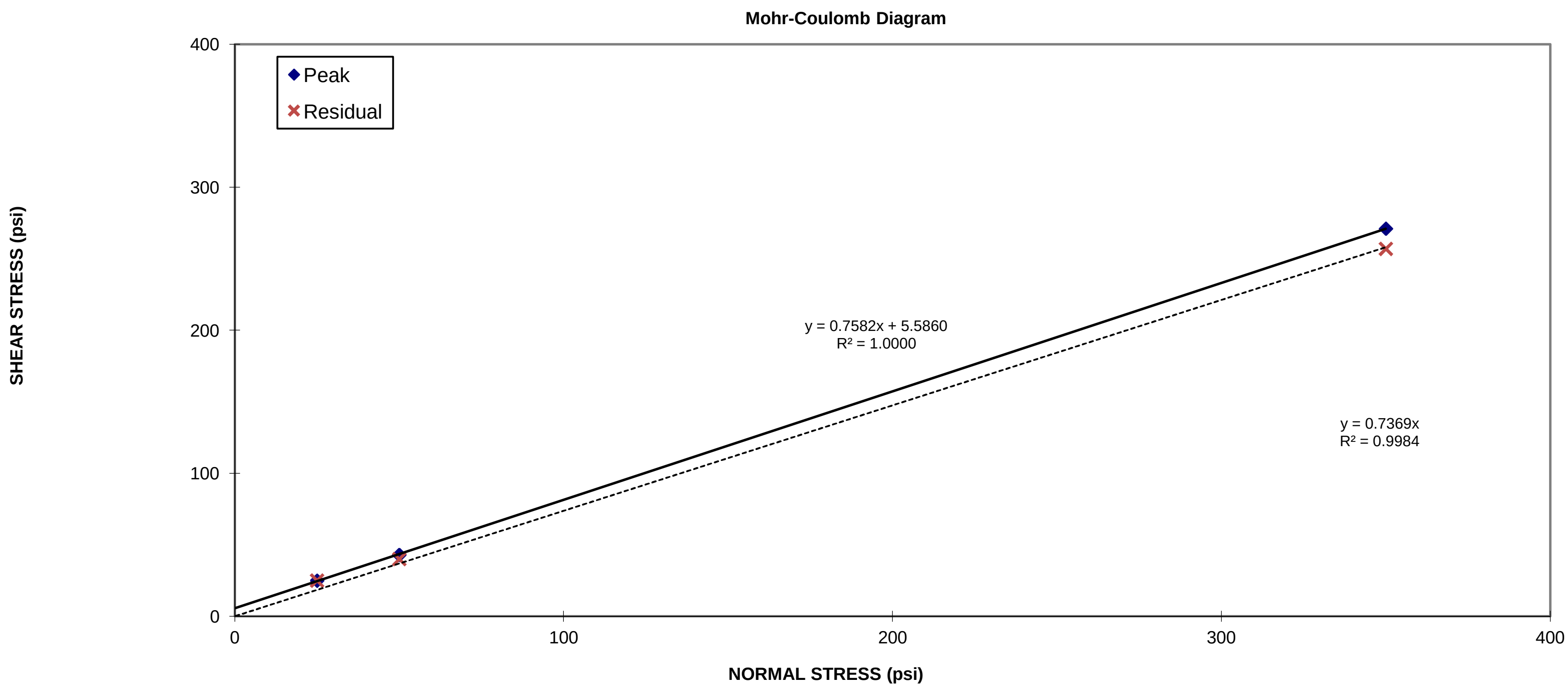
tan(ϕ')=	0.7419	
ϕ'=	36.6	degrees (peak)
C'=	9	psi (peak)
tan(ϕ')=	0.7063	
ϕ'=	35.2	degrees (residual)
C'=	0	psi (residual)

Golder Associates
Denver, Colorado

TITLE
Mohr-Coulomb Diagram for Large Scale Direct Shear on Waste Rock (GA-11P-05 @ 3-6 ft)

CLIENT/PROJECT
Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	4



tan(ϕ')=	0.7582	
ϕ'=	37.2	degrees (peak)
C'=	6	psi (peak)
tan(ϕ')=	0.7369	
ϕ'=	36.4	degrees (residual)
C'=	0	psi (residual)

Golder Associates
Denver, Colorado

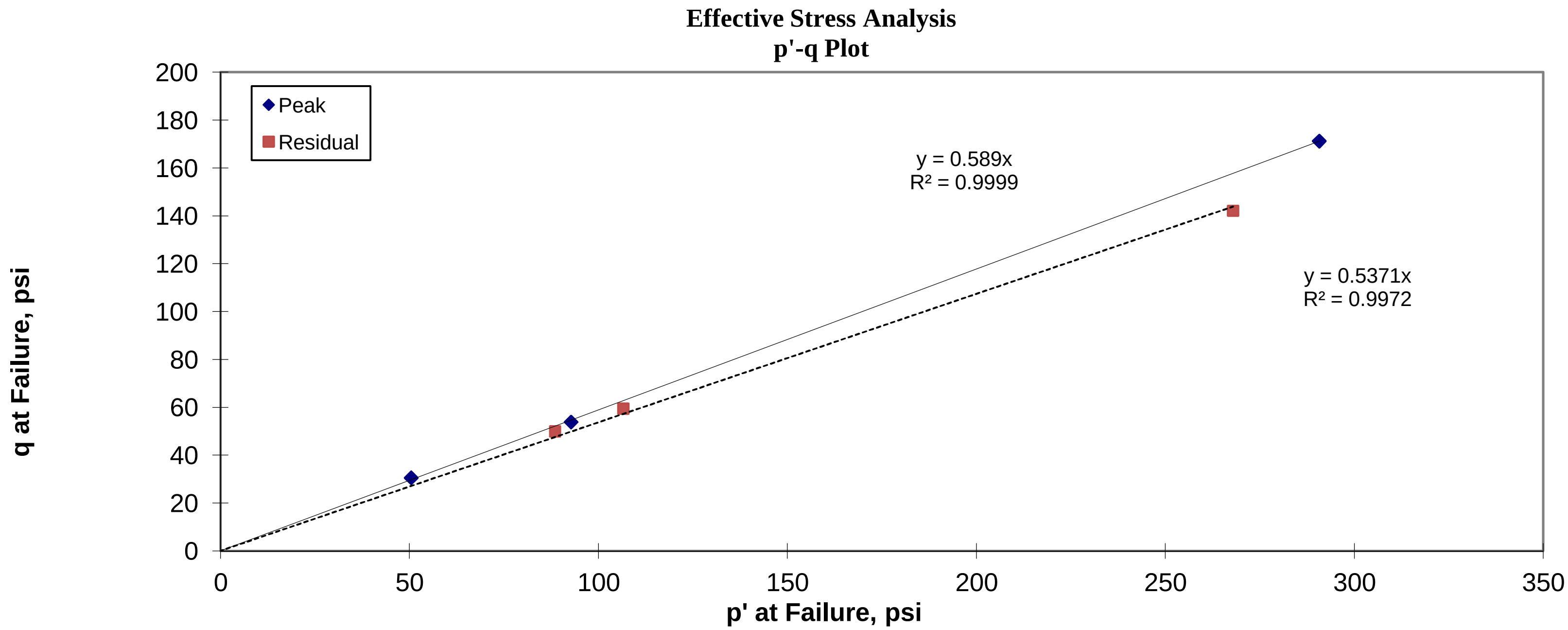
TITLE

Mohr-Coulomb Diagram for Large Scale Direct Shear on Waste Rock (GA-11B-25 @ 16 ft/ GA-11B-23 @ 38-40 ft)

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	5



tan(ψ') =	**0.5890**	
a' =	**0**	**psi**
ϕ' =	**36.1**	**degrees (peak)**
c' =	**0**	**psi (peak)**
tan(ψ') =	**0.5371**	
a' =	**0**	**psi**
ϕ' =	**32.5**	**degrees (residual)**
c' =	**0**	**psi (residual)**

Golder Associates
Denver, Colorado

TITLE
Effective Stress Parameters for CU Triaxial Test on Tailing Fines (GA-11B-24 @ 51.5-53.6 ft and 71-73.2 ft)

CLIENT/PROJECT
Climax Molybdenum, OSF Design

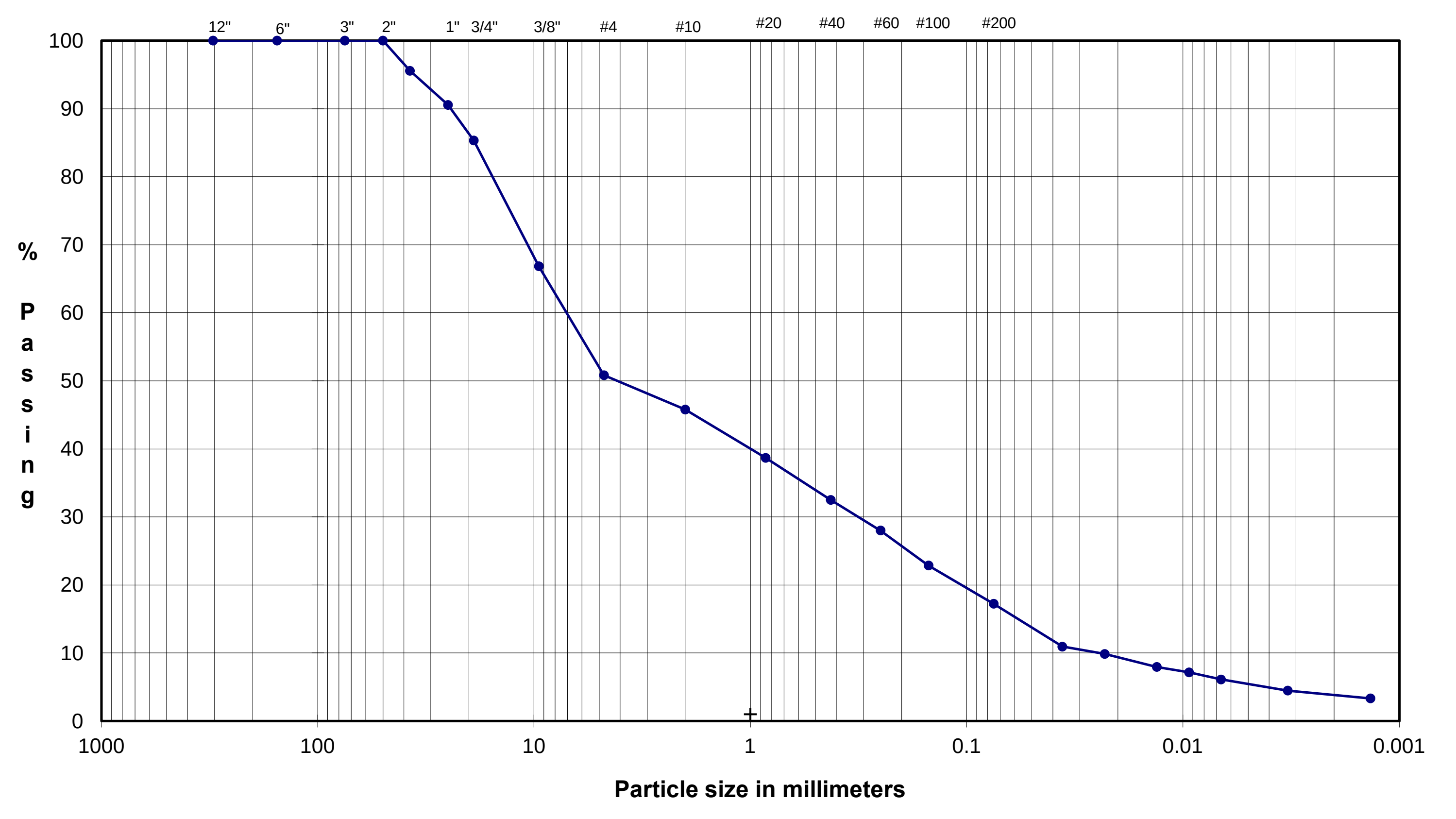
DRAWN	DAR	DATE	Jan-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	6

ATTACHMENT 1
INDEX TEST RESULTS

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-20, B20-2 Depth (ft): 32-34
TYPE: Bag



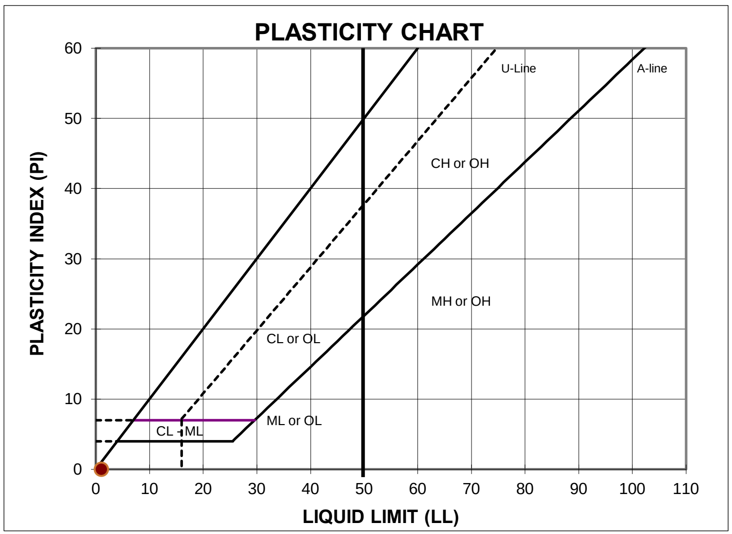
COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	95.6		
1.0"	25.0	90.5		
0.75"	19.0	85.3	Coarse Gravel	9.46
0.375"	9.5	66.8		
#4	4.8	50.8	Fine Gravel	39.72
#10	2.00	45.8	Coarse Sand	5.04
#20	0.85	38.7		
#40	0.43	32.5	Medium Sand	13.28
#60	0.25	28.0		
#100	0.15	22.9		
#200	0.075	17.2	Fine Sand	15.25

Hydrometer Analysis

(mm)	%Finer		
0.036	10.9		
0.023	9.9		
0.013	8.0	Fines	
0.009	7.2	Silt or Clay	17.24
0.007	6.1		
0.003	4.5		
0.001	3.3		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	NP	NP	NP	2.70

DESCRIPTION: Red silty gravel with sand

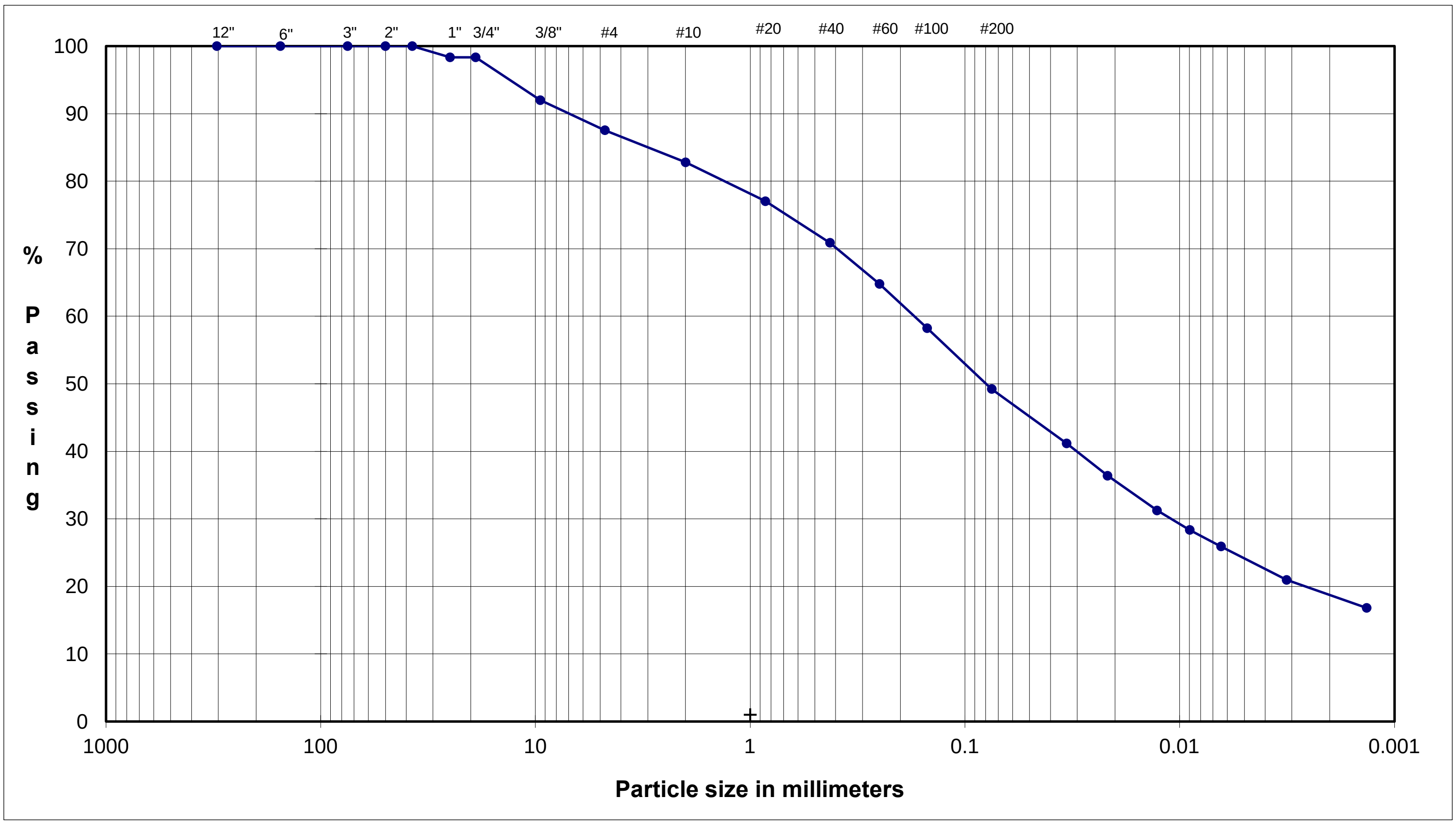
USCS: SM

TECH	BTC
DATE	11/15/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-21 (Sonic Core) Depth (ft): 28'
TYPE: Core



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

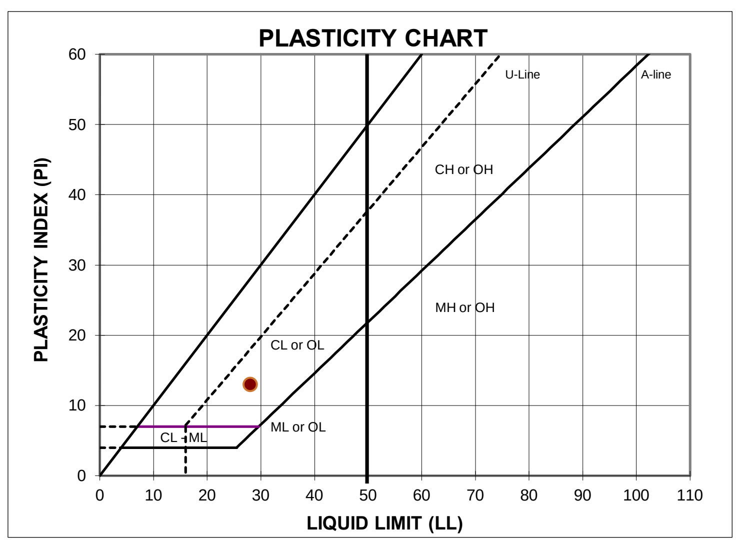
U.S. Standard Sieves Sizes and Numbers

Particle Size	(mm)	% Passing	Particle Size Classification	Particle Size Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	100.0		
1.0"	25.0	98.3		
0.75"	19.0	98.3	Coarse Gravel	1.66
0.375"	9.5	92.0		
#4	4.8	87.5	Fine Gravel	10.79
#10	2.00	82.8	Coarse Sand	4.76
#20	0.85	77.0		
#40	0.43	70.9	Medium Sand	11.91
#60	0.25	64.8		
#100	0.15	58.2		
#200	0.075	49.2	Fine Sand	21.65

Hydrometer Analysis

(mm)	%Finer		
0.034	41.2		
0.022	36.4		
0.013	31.2	Fines	
0.009	28.4	Silt or Clay	49.23
0.006	25.9		
0.003	21.0		
0.001	16.8		

PLASTICITY CHART



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	28	15	13	2.70

DESCRIPTION: Dry, reddish brown silt with small gravel.

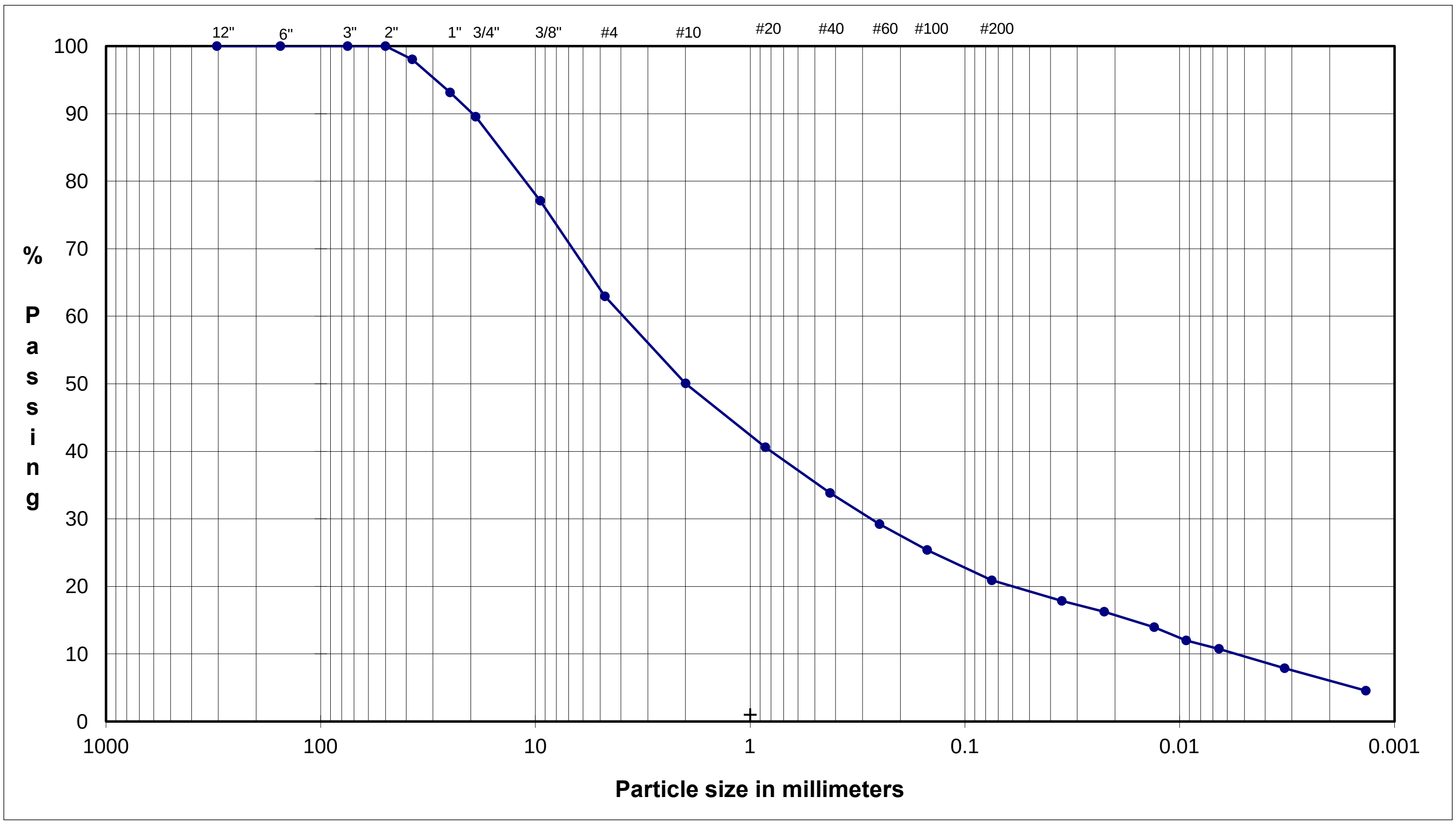
USCS: SC

TECH	BTC
DATE	12/21/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME:	Climax/Geotechnical Investigation		
SAMPLE ID:	GA-11B-20, B20-4	Depth (ft):	40-43
TYPE:	Bag		



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

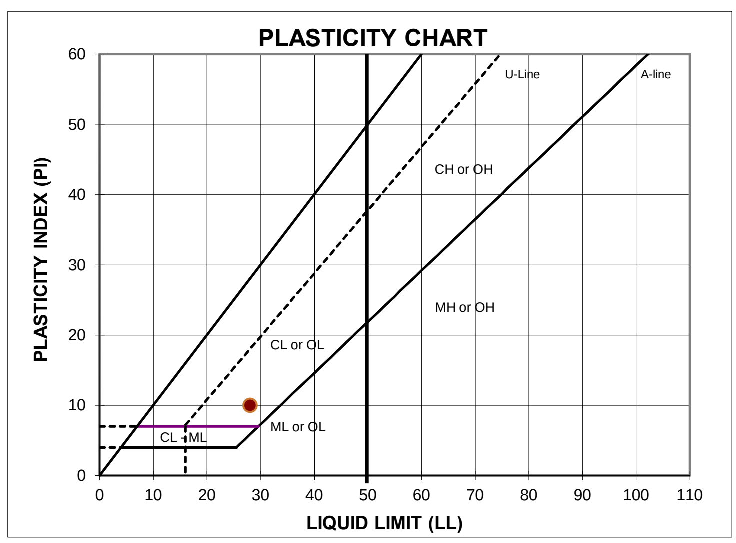
U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	98.0		
1.0"	25.0	93.1		
0.75"	19.0	89.6	Coarse Gravel	6.86
0.375"	9.5	77.1		
#4	4.8	63.0	Fine Gravel	30.19
#10	2.00	50.1	Coarse Sand	12.88
#20	0.85	40.6		
#40	0.43	33.8	Medium Sand	16.23
#60	0.25	29.2		
#100	0.15	25.4		
#200	0.075	20.9	Fine Sand	12.94

Hydrometer Analysis

(mm)	%Finer		
0.035	17.9		
0.022	16.3		
0.013	14.0	Fines	
0.009	12.0	Silt or Clay	20.91
0.007	10.8		
0.003	7.9		
0.001	4.6		

PLASTICITY CHART



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	28	18	10	2.70

DESCRIPTION: Gray clayey sand with gravel

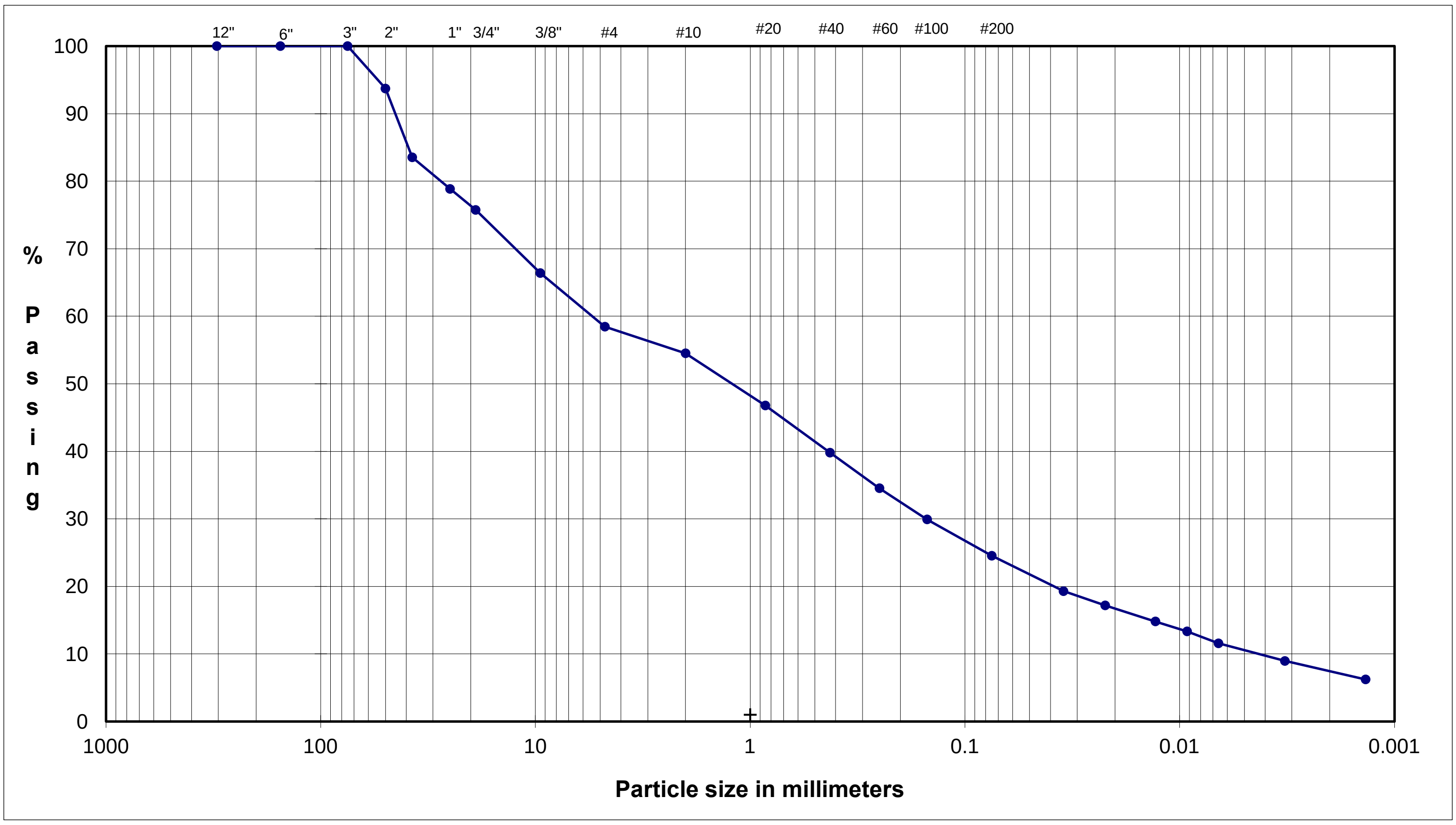
USCS: SC

TECH	BTC
DATE	11/15/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-24, B24-18 Depth (ft): 132
TYPE: Bag



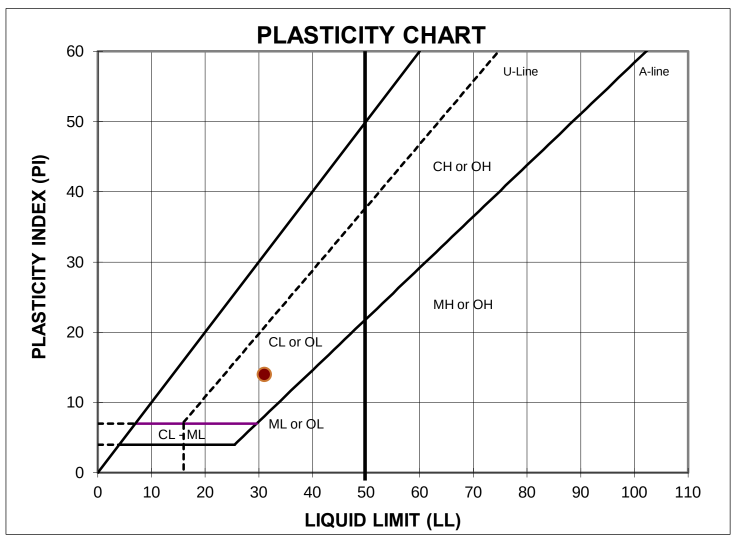
COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

Particle Size	(mm)	% Passing	Particle Size Classification	Particle Size Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	93.7		
1.5"	37.5	83.5		
1.0"	25.0	78.9		
0.75"	19.0	75.7	Coarse Gravel	21.15
0.375"	9.5	66.4		
#4	4.8	58.5	Fine Gravel	20.39
#10	2.00	54.5	Coarse Sand	3.94
#20	0.85	46.8		
#40	0.43	39.8	Medium Sand	14.71
#60	0.25	34.5		
#100	0.15	29.9		
#200	0.075	24.5	Fine Sand	15.26

Hydrometer Analysis

(mm)	%Finer		
0.035	19.3		
0.022	17.2		
0.013	14.8		
0.009	13.3	Fines Silt or Clay	24.54
0.007	11.6		
0.003	9.0		
0.001	6.2		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	31	17	14	2.70

DESCRIPTION: Red clayey gravel with sand

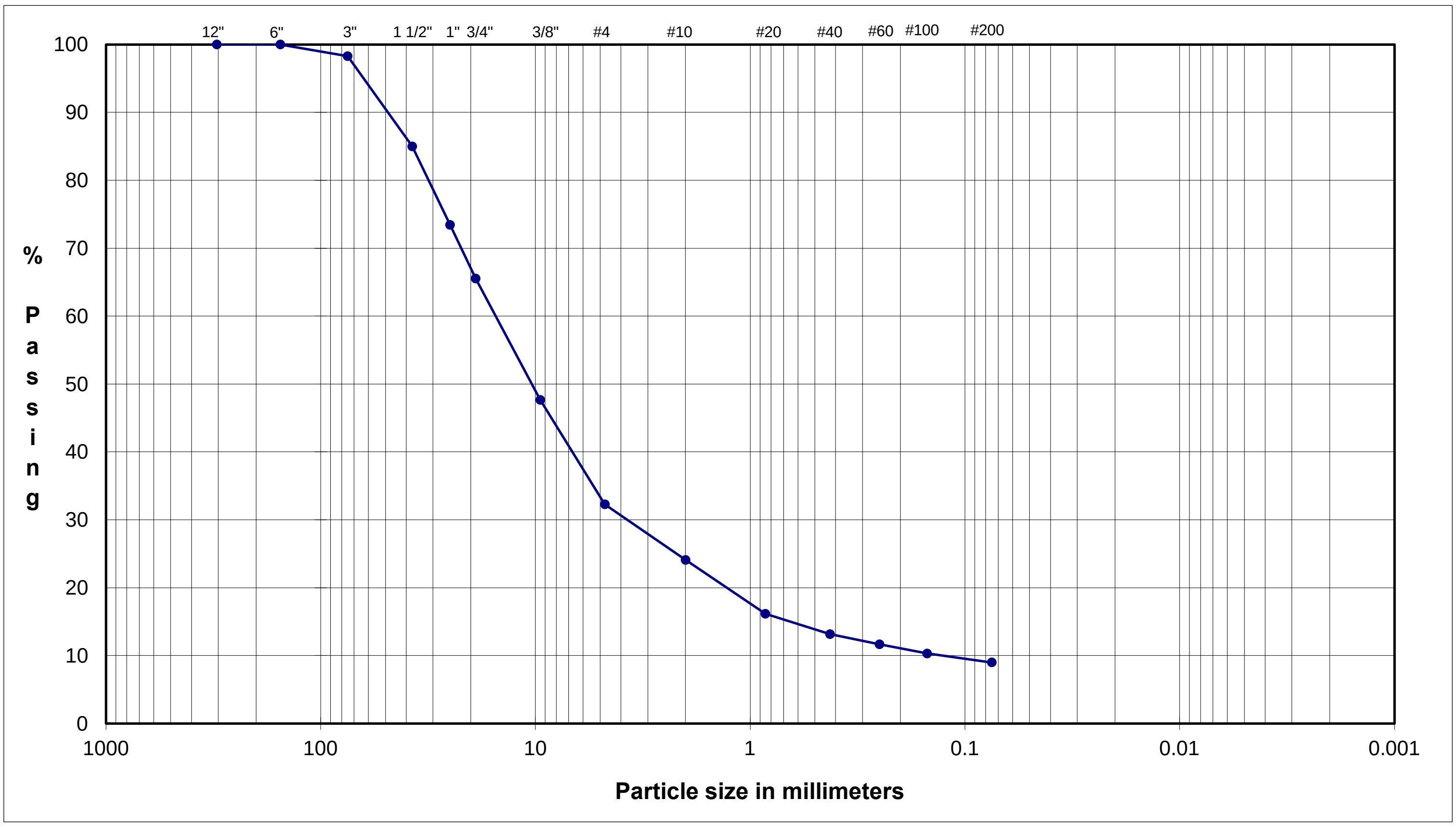
USCS: GC

TECH	BTC
DATE	11/14/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D2487, D2488, D4318

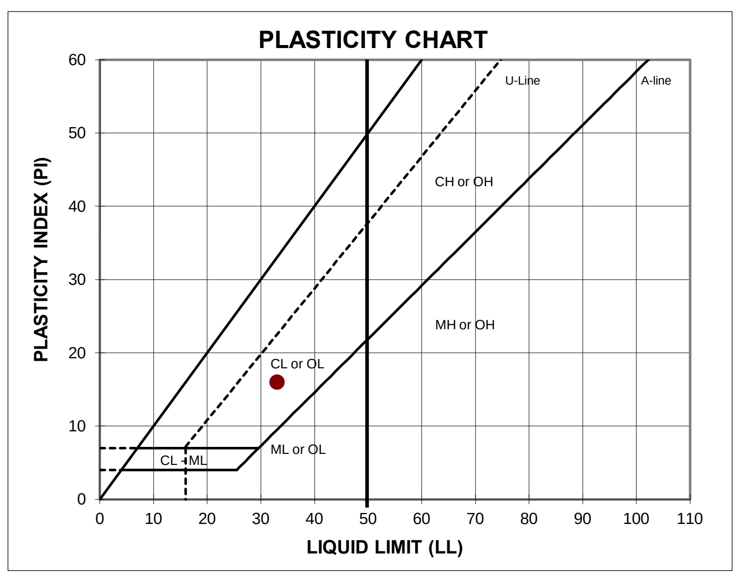
PROJECT NAME: Climax/Geotechnical Investigatio
SAMPLE ID: GA-11P-05, P05-1 Depth (ft) 3-6
TYPE: Pail



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

U.S. Standard Sieves Sizes and Numbers	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	98.3	Cobbles	1.72
1.5"	37.5	85.0		
1.0"	25.0	73.4		
0.75"	19.0	65.5	Coarse Gravel	32.74
0.375"	9.5	47.7		
#4	4.8	32.3	Fine Gravel	33.24
#10	2.0	24.1	Coarse Sand	8.19
#20	0.9	16.2		
#40	0.4	13.2	Medium Sand	10.93
#60	0.3	11.7		
#100	0.2	10.3		
#200	0.1	9.0	Fine Sand	4.15
			Fines	9.01



ATTERBERG LIMITS

M_c	LL	PL	PI
8.9	33	17	16

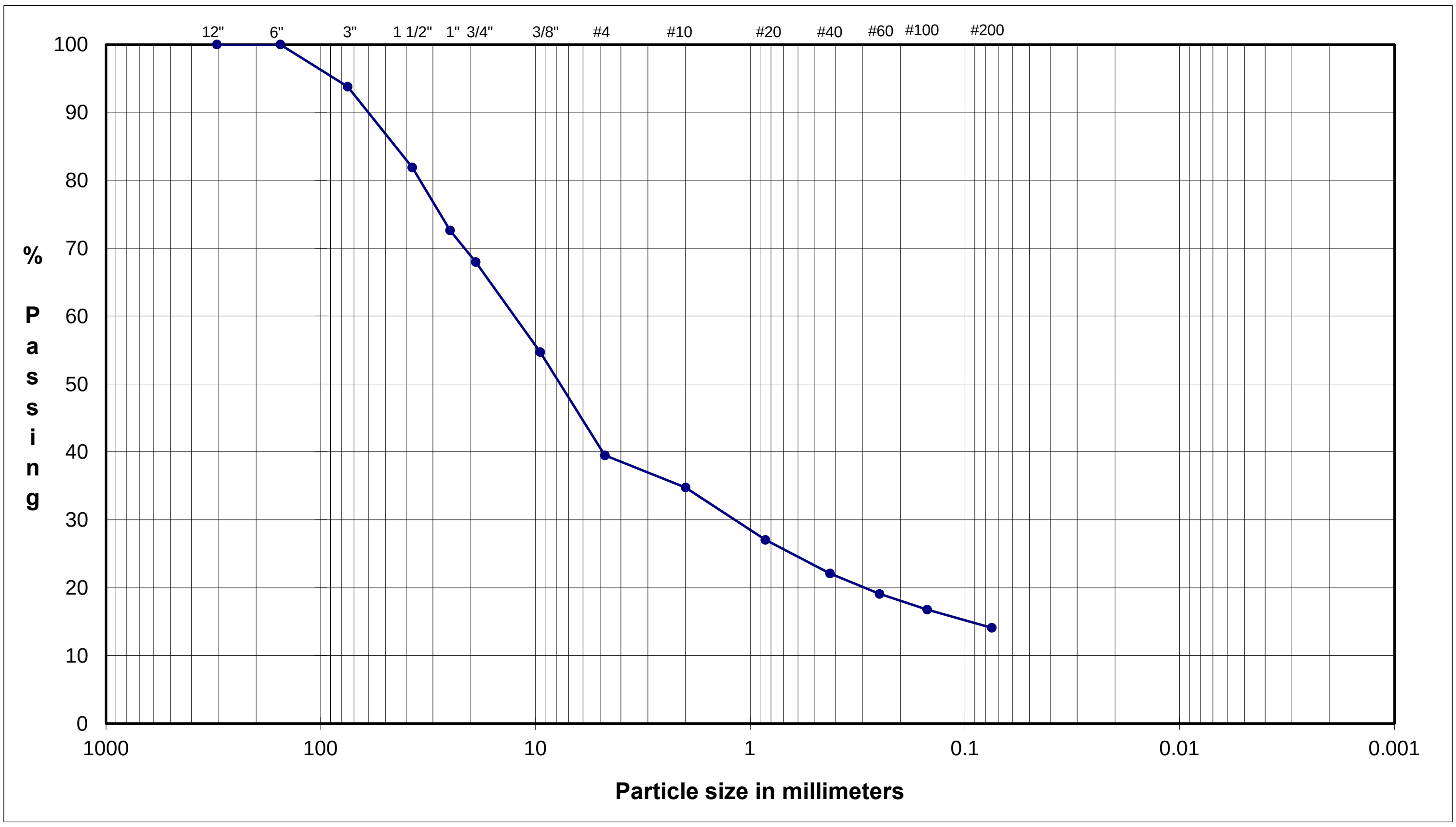
VISUAL DESCRIPTION: Wet, strong brown poorly graded gravel with clay and sand

USCS: GP-GM

TECH	BTC
DATE	12/10/11
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS
ASTM D421, D422, D2487, D2488, D4318

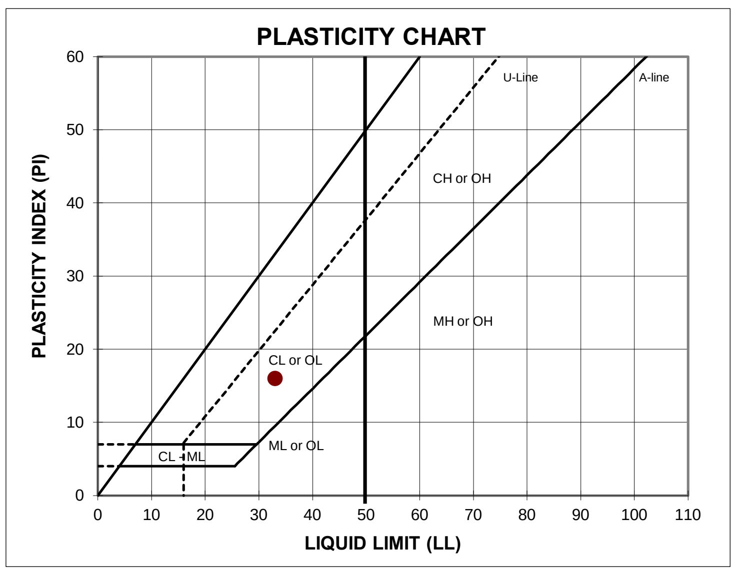
PROJECT NAME: Climax/Geotechnical Investigatio
SAMPLE ID: GA-11B-25, B25-3 Depth (ft) 16'
TYPE: Pail



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	93.8	Cobbles	6.21
1.5"	37.5	81.9		
1.0"	25.0	72.6		
0.75"	19.0	68.0	Coarse Gravel	25.82
0.375"	9.5	54.7		
#4	4.8	39.5	Fine Gravel	28.48
#10	2.0	34.8	Coarse Sand	4.72
#20	0.9	27.0		
#40	0.4	22.1	Medium Sand	12.67
#60	0.3	19.1		
#100	0.2	16.8		
#200	0.1	14.1	Fine Sand	8.00
			Fines	14.11



ATTERBERG LIMITS

M_c	LL	PL	PI
6.0	28	15	13

DESCRIPTION: Moist, orange clayey gravel with sand

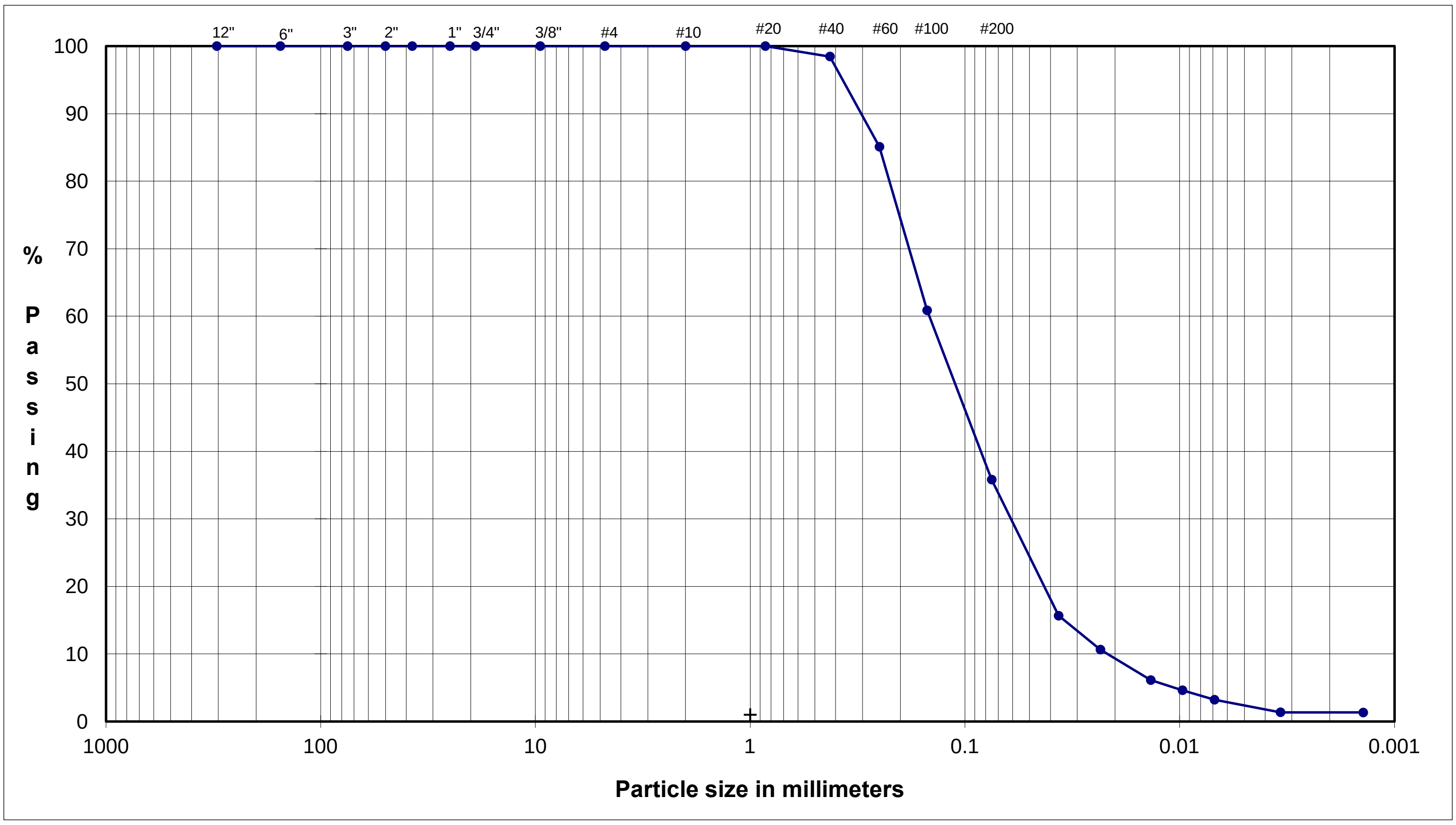
USCS: GC

TECH	BTC
DATE	12/8/11
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-24, B24-15 Depth (ft): 91-92.5
TYPE: Bag



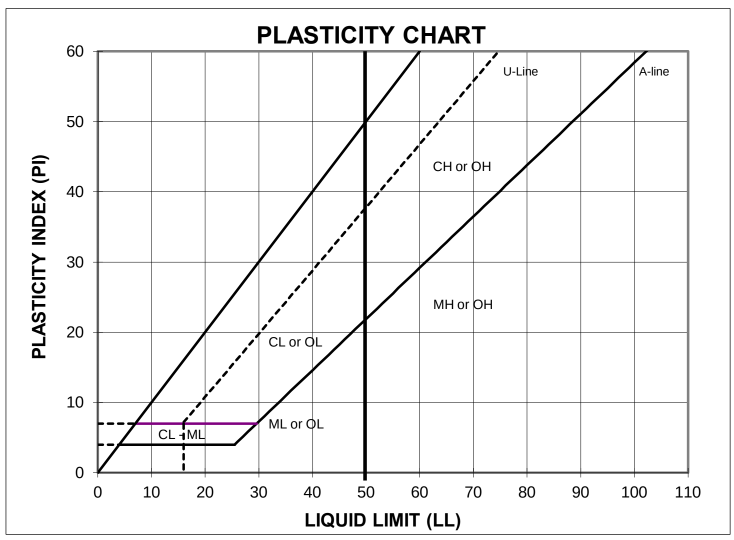
COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	100.0		
1.0"	25.0	100.0		
0.75"	19.0	100.0	Coarse Gravel	0.00
0.375"	9.5	100.0		
#4	4.8	100.0	Fine Gravel	0.00
#10	2.00	100.0	Coarse Sand	0.00
#20	0.85	100.0		
#40	0.43	98.5	Medium Sand	1.54
#60	0.25	85.1		
#100	0.15	60.9		
#200	0.075	35.8	Fine Sand	62.64

Hydrometer Analysis

(mm)	%Finer		
0.037	15.7		
0.023	10.6		
0.014	6.1		
0.010	4.6	Fines Silt or Clay	35.82
0.007	3.2		
0.003	1.4		
0.001	1.3		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	NP	NP	NP	2.70

DESCRIPTION: Light yellowish brown silty sand

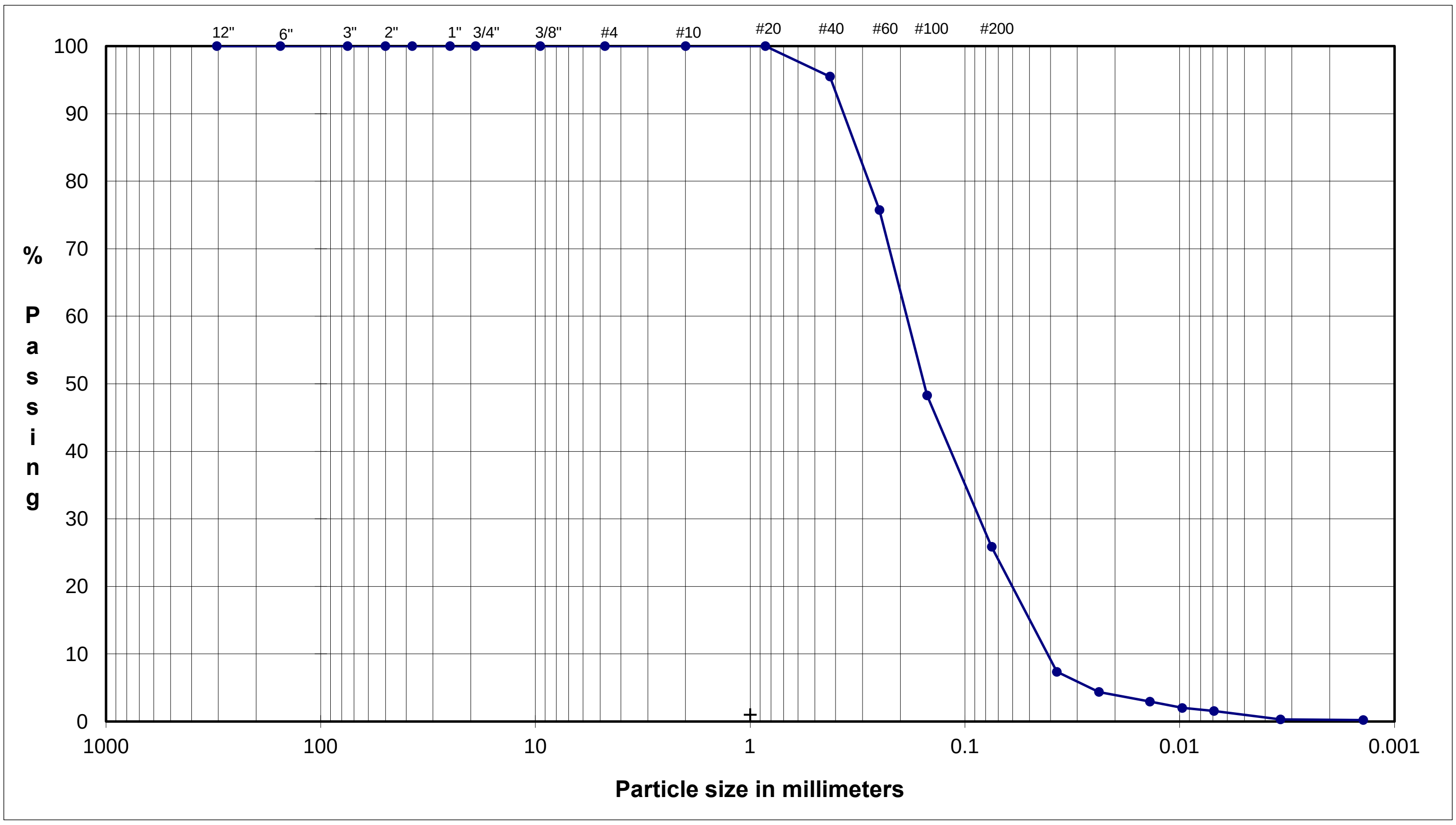
USCS: SM

TECH	BTC
DATE	11/14/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-25, B25-10 Depth (ft): 43.5-45
TYPE: Bag



COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

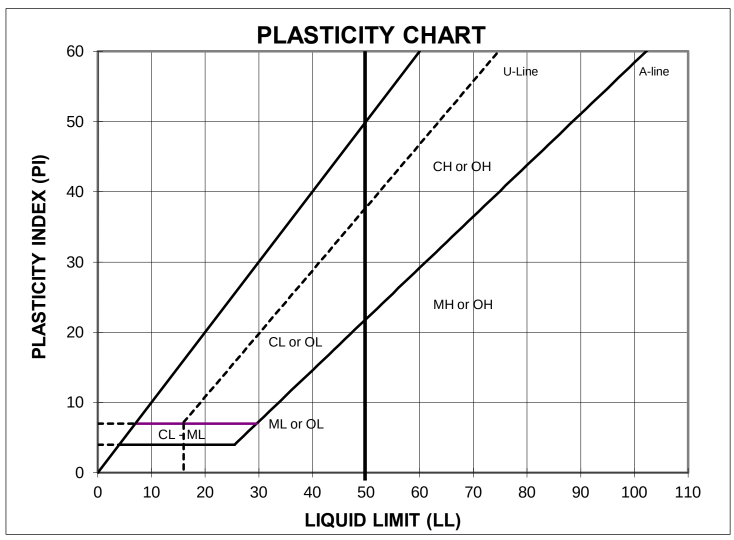
U.S. Standard Sieves Sizes and Numbers

Particle Size	(mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0	Cobbles	0.00
6.0"	154.2	100.0		
3.0"	75.0	100.0		
2.0"	50.0	100.0	Coarse Gravel	0.00
1.5"	37.5	100.0		
1.0"	25.0	100.0		
0.75"	19.0	100.0		
0.375"	9.5	100.0	Fine Gravel	0.00
#4	4.8	100.0		
#10	2.00	100.0	Coarse Sand	0.00
#20	0.85	100.0	Medium Sand	4.51
#40	0.43	95.5		
#60	0.25	75.7	Fine Sand	69.62
#100	0.15	48.3		
#200	0.075	25.9		

Hydrometer Analysis

(mm)	%Finer	Classification	Percentage
0.037	7.4	Fines Silt or Clay	25.88
0.024	4.4		
0.014	2.9		
0.010	2.0		
0.007	1.6		
0.003	0.3		
0.001	0.2		

PLASTICITY CHART



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	NP	NP	NP	2.70

DESCRIPTION: Gray silty sand

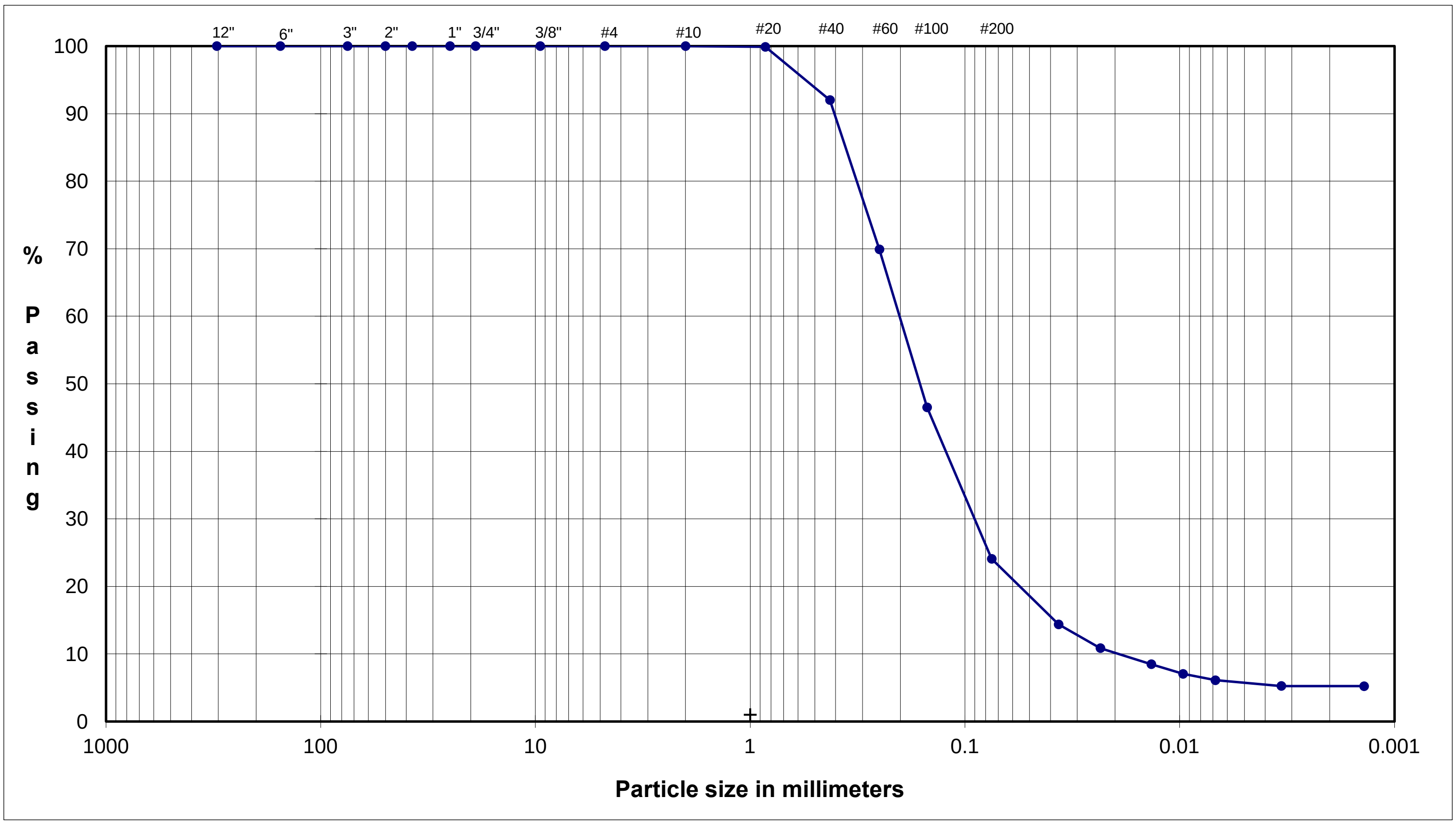
USCS: SM

TECH	BTC
DATE	11/14/2011
REVIEW	MB

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME: Climax/Geotechnical Investigation
SAMPLE ID: GA-11B-25, B25-9 Depth (ft): 55
TYPE: Bag



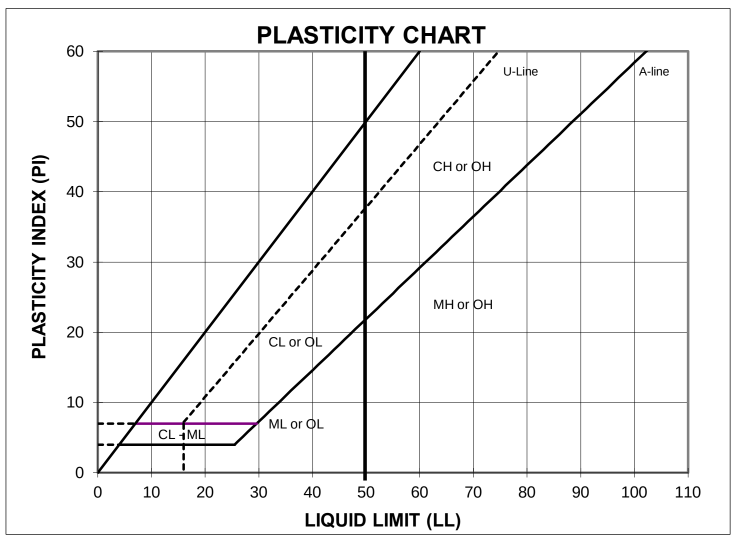
COBBLES	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay
	GRAVEL		SAND			FINES

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	100.0		
1.0"	25.0	100.0		
0.75"	19.0	100.0	Coarse Gravel	0.00
0.375"	9.5	100.0		
#4	4.8	100.0	Fine Gravel	0.00
#10	2.00	100.0	Coarse Sand	0.00
#20	0.85	99.9		
#40	0.43	92.0	Medium Sand	7.99
#60	0.25	69.9		
#100	0.15	46.5		
#200	0.075	24.1	Fine Sand	67.93

Hydrometer Analysis

(mm)	%Finer		
0.037	14.4		
0.023	10.9		
0.014	8.5	Fines	
0.010	7.1	Silt or Clay	24.08
0.007	6.1		
0.003	5.3		
0.001	5.2		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
--	NP	NP	NP	2.70

DESCRIPTION: Pale yellow silty sand

USCS: SM

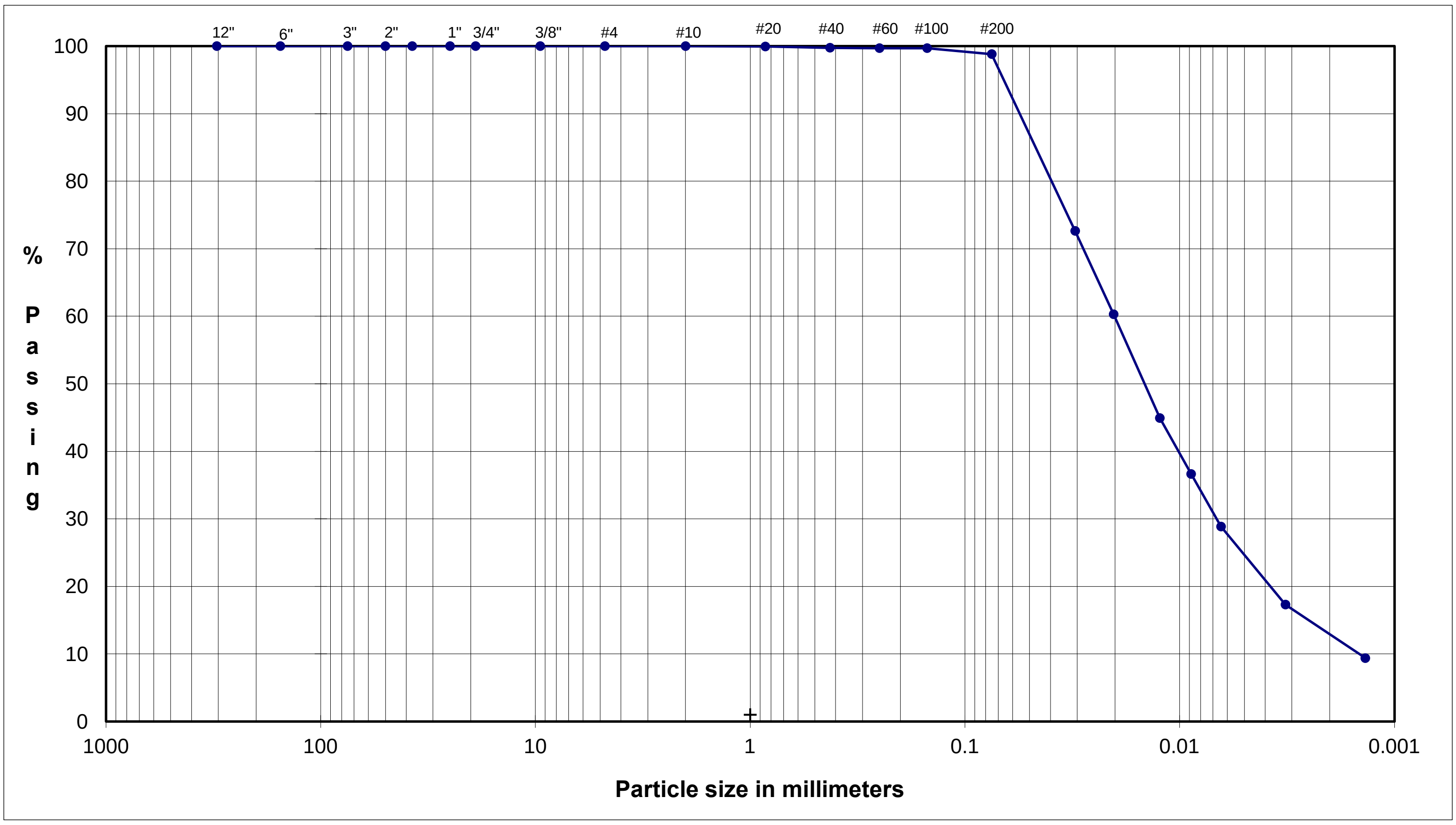
TECH	BTC
DATE	11/14/2011
REVIEW	MB

December-11 113-81608

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME:	Climax/Geotechnical Investigation		
SAMPLE ID:	GA-11B-24, B24-9	Depth (ft):	51.5-53.6
TYPE:	Bag		



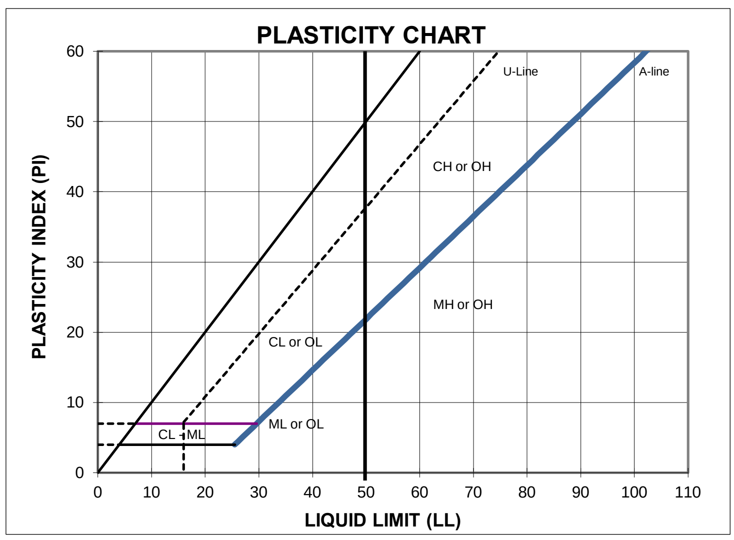
COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

Sieve	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	100.0		
1.0"	25.0	100.0		
0.75"	19.0	100.0	Coarse Gravel	0.00
0.375"	9.5	100.0		
#4	4.8	100.0	Fine Gravel	0.00
#10	2.00	100.0	Coarse Sand	0.00
#20	0.85	99.9		
#40	0.43	99.8	Medium Sand	0.24
#60	0.25	99.7		
#100	0.15	99.7		
#200	0.075	98.8	Fine Sand	0.94

Hydrometer Analysis

(mm)	%Finer		
0.031	72.6		
0.020	60.3		
0.012	44.9		
0.009	36.7	Fines Silt or Clay	98.82
0.006	28.9		
0.003	17.3		
0.001	9.4		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
31.9	NP	NP	NP	2.70

DESCRIPTION: Light gray silt

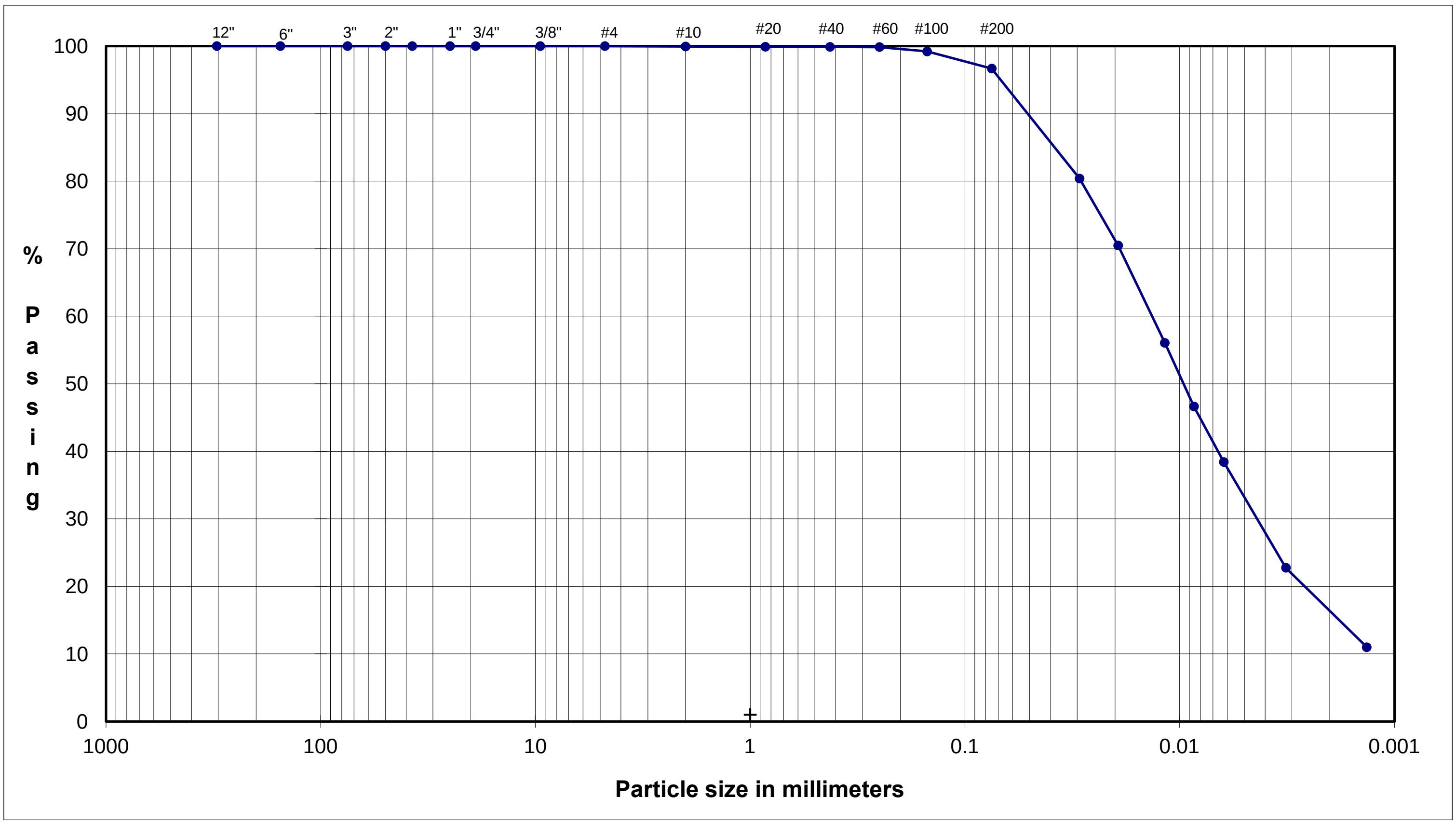
USCS: ML

TECH	JAM
DATE	12/13/2011
REVIEW	PRH

PARTICLE SIZE DISTRIBUTION & ATTERBERG LIMITS

ASTM D421, D422, D4318

PROJECT NAME:	Climax/Geotechnical Investigation		
SAMPLE ID:	GA-11B-24, B24-13	Depth (ft):	71.0-73.2
TYPE:	Post Triax		



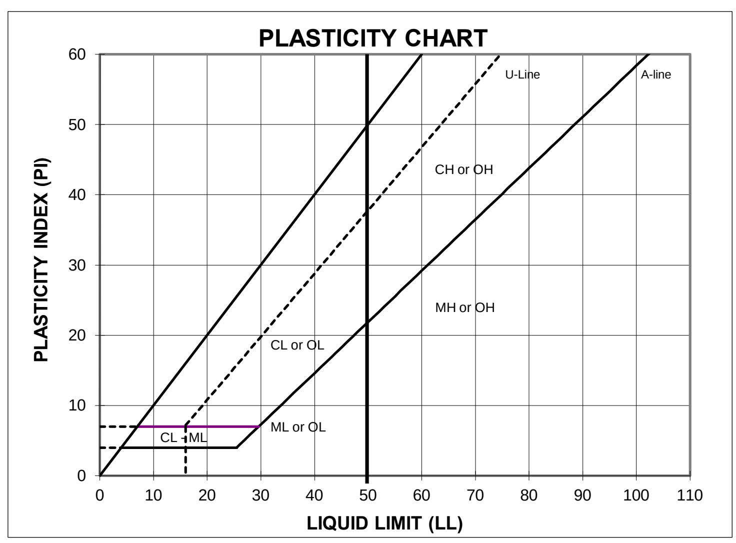
COBBLES	GRAVEL		SAND			FINES
	Coarse	Fine	Coarse	Medium	Fine	Silt or Clay

U.S. Standard Sieves Sizes and Numbers

	Particle Size (mm)	% Passing	Particle Size Classification	Percentage
12.0"	304.8	100.0		
6.0"	154.2	100.0		
3.0"	75.0	100.0	Cobbles	0.00
2.0"	50.0	100.0		
1.5"	37.5	100.0		
1.0"	25.0	100.0		
0.75"	19.0	100.0	Coarse Gravel	0.00
0.375"	9.5	100.0		
#4	4.8	100.0	Fine Gravel	0.00
#10	2.00	99.9	Coarse Sand	0.06
#20	0.85	99.9		
#40	0.43	99.9	Medium Sand	0.06
#60	0.25	99.9		
#100	0.15	99.2		
#200	0.075	96.7	Fine Sand	3.21

Hydrometer Analysis

(mm)	%Finer		
0.029	80.4		
0.019	70.5		
0.012	56.1	Fines	
0.009	46.6	Silt or Clay	96.67
0.006	38.4		
0.003	22.8		
0.001	11.0		



ATTERBERG LIMITS

M_c(%)	LL	PL	PI	SpG (assumed)
30.6	NP	NP	NP	2.70

TECH	JAM
DATE	12/28/2011
REVIEW	PRH

DESCRIPTION: Light gray silt.

USCS: ML

Moisture Content and Density Summary

Soil Type	Boring/Pit Number	Sample Depth (ft)	Moisture Content		In-Situ Density	
			Delivered (%)	Natural (%)	Wet Unit Weight (pcf)	Dry Unit Weight (pcf)
Lincoln Porphyry	GA-11B-25	178	15.0	--	--	--
Glacial Till	GA-11B-25	104-106	7.8	--	--	--
Glacial Till	GA-11B-23	249-251	6.9	--	--	--
Tailing Beach	GA-11B-25	42	--	26.9	124.2	97.8
Tailing Fines	GA-11B-24	71-73.2	--	30.6	166.7	86.0
Tailing Fines	GA-11B-23	190-192	--	25.5	125.9	100.3

MOISTURE / DRY DENSITY CURVE
ASTM D 698 Method B

Manual	Standard	Wet Method

PROJECT NAME: Climax/Geotechnical Investigation
PROJECT NUMBER: 113-81608
SAMPLE ID: GA-11B-25, B25-14 @ 104-106 (ft)
SAMPLE TYPE: Composite of GA-11B-25, B25-14 @ 104-106 & GA-11B-23, B23-16 @ 249-251

COMPACTION POINTS		
Specimen Number	Dry Density (pcf)	Moisture Content (%)
1	119.7	5.6%
2	126.6	6.8%
3	128.8	8.5%
4	128.8	10.0%
5	125.2	11.6%
6	121.0	13.5%

Maximum Dry Density (pcf)	129.5
Optimum Moisture (%)	8.7
Corrected Maximum Dry Density (pcf)	135.2
Corrected Optimum Moisture (%)	7.3
As-Received Moisture Content (%)	6.9
% Passing #4 sieve	70.6
% Passing 3/8" sieve	80.5
% Passing 3/4" sieve	88.6

DESCRIPTION: Moist, yellowish brown clayey gravel with sand and silt

USCS: --

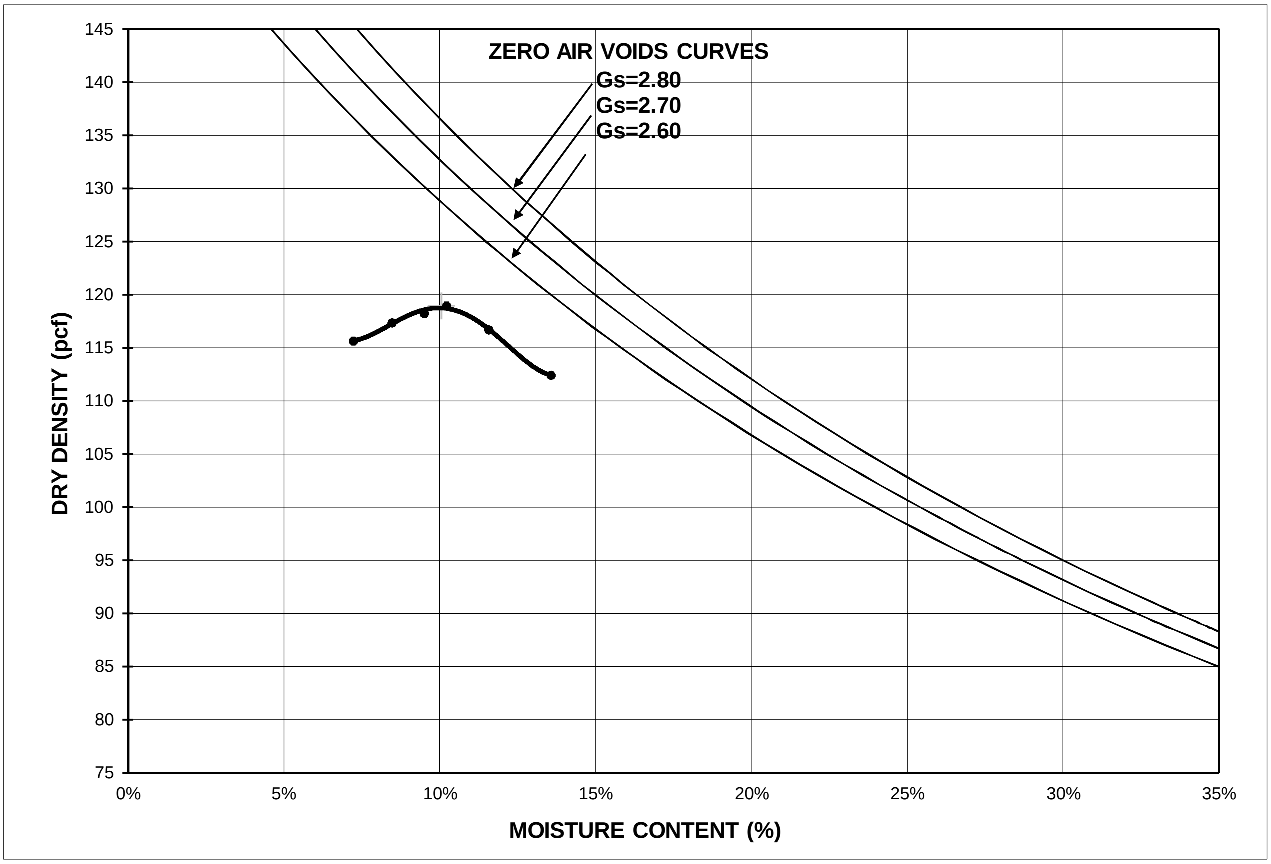
TECH	BTC
DATE	12-3-2011
REVIEW	MB

Golder Associates Inc.

MOISTURE / DRY DENSITY CURVE
AASHTO T180 Method A

Manual	Modified	Wet Method

PROJECT NAME: **Climax/Geotechnical Investigation** SAMPLE TYPE: Pail, Tubes, Bags
PROJECT NUMBER: **113-81608**
SAMPLE ID: **Comp**



COMPACTION POINTS		
Specimen Number	Dry Density (pcf)	Moisture Content (%)
1	115.7	7.2%
2	117.4	8.5%
3	118.2	9.5%
4	119.0	10.2%
5	116.7	11.6%
6	112.4	13.6%

Maximum Dry Density (pcf)	**119.0**
Optimum Moisture (%)	**10.1**
Corrected Maximum Dry Density (pcf)	--
Corrected Optimum Moisture (%)	--
As-Received Moisture Content (%)	**7.4**
% Passing #4 sieve	**100.0**
% Passing 3/8" sieve	**100.0**
% Passing 3/4" sieve	**100.0**

DESCRIPTION Moist, light yellowish brown sandy SILT

USCS --

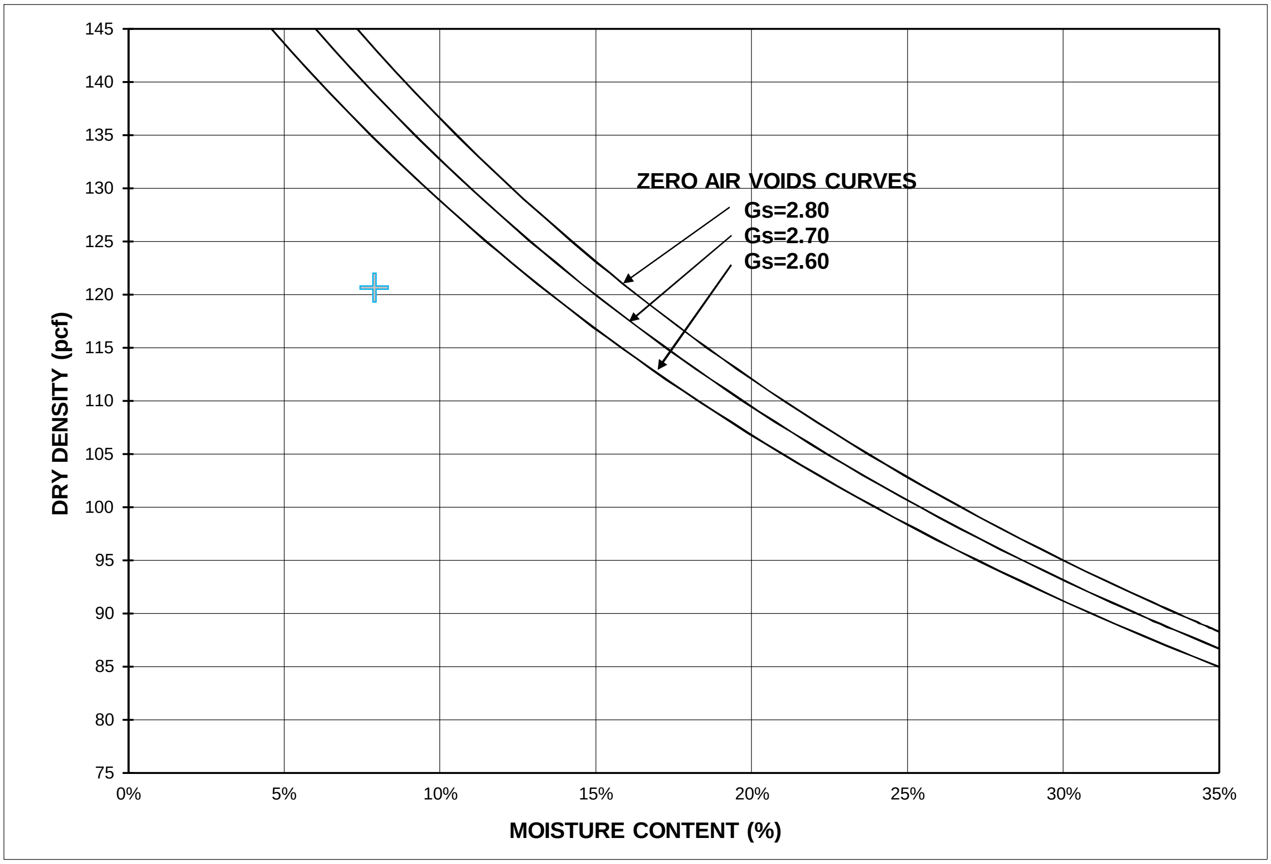
TECH	BTC
DATE	12-10-2011
REVIEW	MB

113-81608

MOISTURE / DRY DENSITY CURVE
Non ASTM Non ASSHTP   Method A

Mechanical	Modified	Wet Method

PROJECT NAME: **Climax/Geotechnical Investigation**
PROJECT NUMBER: **113-81608**
SAMPLE ID: **Comp (Point to Oblivion)**   DEPTH: **--**   SAMPLE TYPE: **Pail**



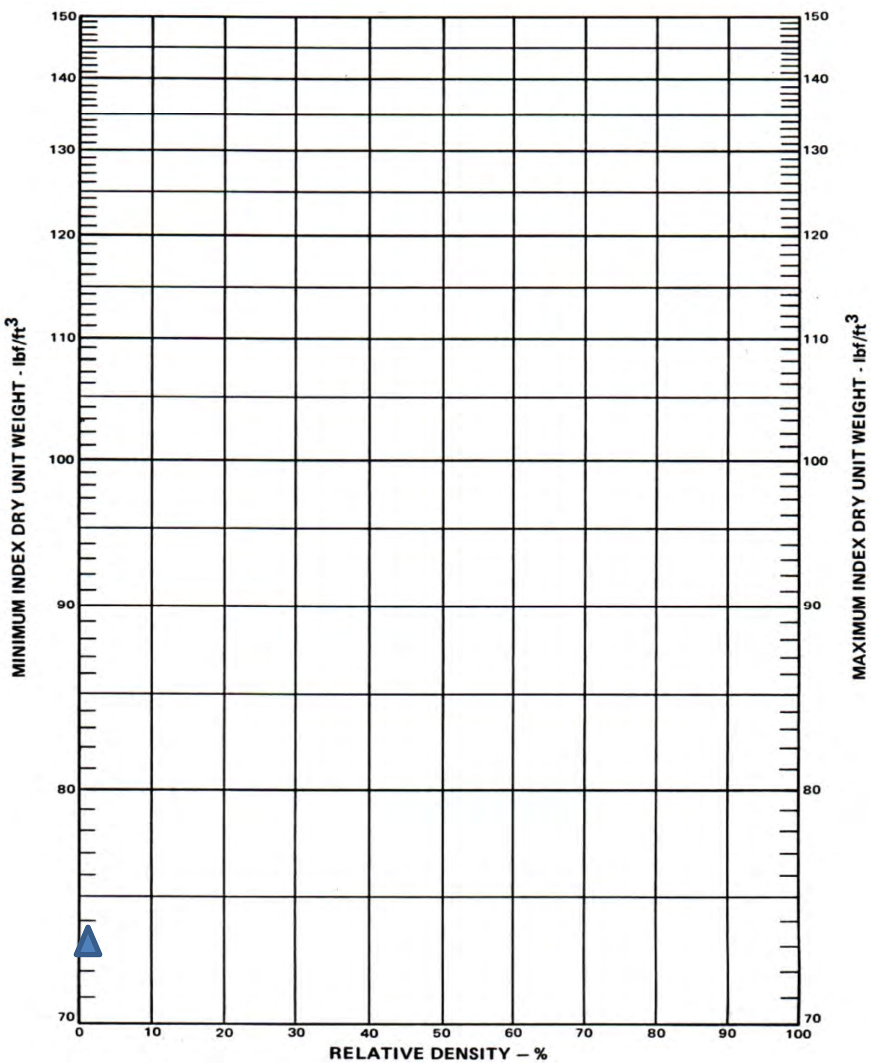
COMPACTION POINTS		
Specimen Number	Dry Density (pcf)	Moisture Content (%)
1	120.7	7.9%

Dry Density (pcf)	**120.7**
Moisture (%)	**7.9**
Corrected Dry Density (pcf)	--
Corrected Moisture (%)	--
As-Received Moisture Content	--
% Passing #4 sieve	**1.0**
% Passing 3/8" sieve	**1.0**
% Passing 3/4" sieve	**1.0**

DESCRIPTION Moist, light yellowish brown sandy SILT

USCS --

TECH	BTC
DATE	2-16-2012
REVIEW	MB



Minimum Index Dry Unit Weight (ASTM D 4254): 73.1 lb/ft^3

Maximum Index Dry Unit Weight (ASTM D 4253): -- lb/ft^3

`

Golder Associates, Inc. Denver, Colorado			**Title:** RELATIVE DENSITY TEST ASTM D4253 (Method B) and D4254		
Job Short Title: Climax/Geotechnical Investigation					
Sample No. Comp	Depth --	Reviewed: MB	**Date:** 12/2/2011	**Job Number:** 113-81051	**Figure:** 1

JAR SLAKE TEST

Kentucky Method 64-514-08
Revised 2/26/08
Supersedes KM 64-514-02
DATED 11/15/02

PROJECT TITLE	**CLIMAX/GEOTECHNICAL INVESTIGATION/CO**
PROJECT NUMBER	**113-81608**
SAMPLE ID	**B25-3/23-3**
SAMPLE DEPTH	-

SPECIMEN ID	**1**	**2**	**3**
APPROX. AMOUNT OF DRIED SPECIMEN (grams)	72.47	89.32	78.21
OBSERVATION SCHEDULE START TIME	9:32 a.m.	9:32 a.m.	9:32 a.m.
TEST FLUID	DISTILLED WATER	DISTILLED WATER	DISTILLED WATER
	JAR SLAKE INDEX*		
1/2 HOUR	**6**	**6**	**6**
1 HOUR	**6**	**6**	**6**
24 HOURS	**6**	**6**	**6**

***JAR SLAKE Category**		**Behavior**
1	-	Degrades to a pile of flakes or mud (Complete Breakdown).
2	-	Break rapidly and/or forms many chips.
3	-	Break slowly and/or forms many chips.
4	-	Break rapidly and/or develops several fractures.
5	-	Break slowly and/or develops few fractures.
6	-	No change.

NOTE: *Three (3) specimen was selected from sample received, selecting material different in appearance.*

TECH:	AK
DATE:	1/18/12
CHECKED BY:	AK
REVIEWED BY:	JA

JAR SLAKE TEST

Kentucky Method 64-514-08
Revised 2/26/08
Supersedes KM 64-514-02
DATED 11/15/02

PROJECT TITLE CLIMAX/GEOTECHNICAL INVESTIGATION/CO
PROJECT NUMBER 113-81608
SAMPLE ID PO5-1 A
SAMPLE DEPTH -

SPECIMEN ID	**1**	**2**	**3**
APPROX. AMOUNT OF DRIED SPECIMEN (grams)	72.79	81.01	76.81
OBSERVATION SCHEDULE START TIME	9:33 a.m.	9:33 a.m.	9:33 a.m.
TEST FLUID	DISTILLED WATER	DISTILLED WATER	DISTILLED WATER
	JAR SLAKE INDEX*		
1/2 HOUR	6	6	6
1 HOUR	6	6	6
24 HOURS	6	6	6

***JAR SLAKE Category**		**Behavior**
1	-	Degrades to a pile of flakes or mud (Complete Breakdown).
2	-	Break rapidly and/or forms many chips.
3	-	Break slowly and/or forms many chips.
4	-	Break rapidly and/or develops several fractures.
5	-	Break slowly and/or develops few fractures.
6	-	No change.

NOTE: *Three (3) specimen was selected from sample received, selecting material different in appearance.*

TECH:	AK
DATE:	1/18/12
CHECKED BY:	AK
REVIEWED BY:	DA

ATTACHMENT 2
ENGINEERING TEST RESULTS

	Point 1			Point 2			Point 3	
Boring # =	GA-11B-21		Boring # =	GA-11B-21		Boring # =	GA-11B-21	
Sample # =	Sonic Core		Sample # =	Sonic Core		Sample # =	Sonic Core	
Depth (ft) =	28		Depth (ft) =	28		Depth (ft) =	28	
Point # =	1		Point # =	2		Point # =	3	
	Initial			**Initial**			**Initial**	
Length =	20.80	cm	Length =	20.80	cm	Length =	20.80	cm
Diameter =	10.21	cm	Diameter =	10.21	cm	Diameter =	10.21	cm
Wet Weight =	3702.90	g	Wet Weight =	3702.90	g	Wet Weight =	3702.90	g
Area =	81.9	cm^2	Area =	81.9	cm^2	Area =	81.9	cm^2
Sample Area =	12.69	in^2	Sample Area =	12.69	in^2	Sample Area =	12.69	in^2
Volume =	1702.9	cm^3	Volume =	1702.9	cm^3	Volume =	1702.9	cm^3
Moisture Content =	13.9%		Moisture Content =	13.9%		Moisture Content =	13.9%	
Specific Gravity =	-		Specific Gravity =	-		Specific Gravity =	-	
Dry Weight of Solids =	3251.01	g	Dry Weight of Solids =	3251.01	g	Dry Weight of Solids =	3251.01	g
Wet Density =	2.17	g/cm^3	Wet Density =	2.17	g/cm^3	Wet Density =	2.17	g/cm^3
Dry Density =	1.91	g/cm^3	Dry Density =	1.91	g/cm^3	Dry Density =	1.91	g/cm^3
Wet Unit Weight =	135.7	pcf	Wet Unit Weight =	135.7	pcf	Wet Unit Weight =	135.7	pcf
Dry Unit Weight =	119.1	pcf	Dry Unit Weight =	119.1	pcf	Dry Unit Weight =	119.1	pcf
Cell Pressure =	75	psi	Cell Pressure =	150	psi	Cell Pressure =	400	psi
Back Pressure =	50	psi	Back Pressure =	50	psi	Back Pressure =	50	psi
Confining Pressure =	25	psi	Confining Pressure =	100	psi	Confining Pressure =	350	psi

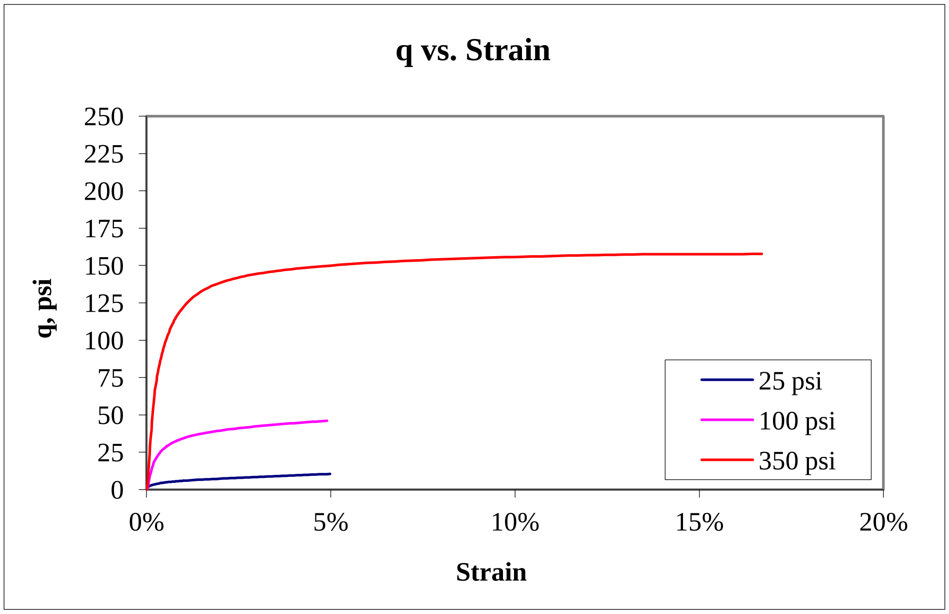
Notes:
Test was a multi-stage test.
Shear rate was 0.001 to 0.002 in./min. t_{50} was 10.2 min.
Sample was intact with ends trimmed flush.
Moisture content was taken from trimmings.
Peak was defined at the maximum principal stress ratio.

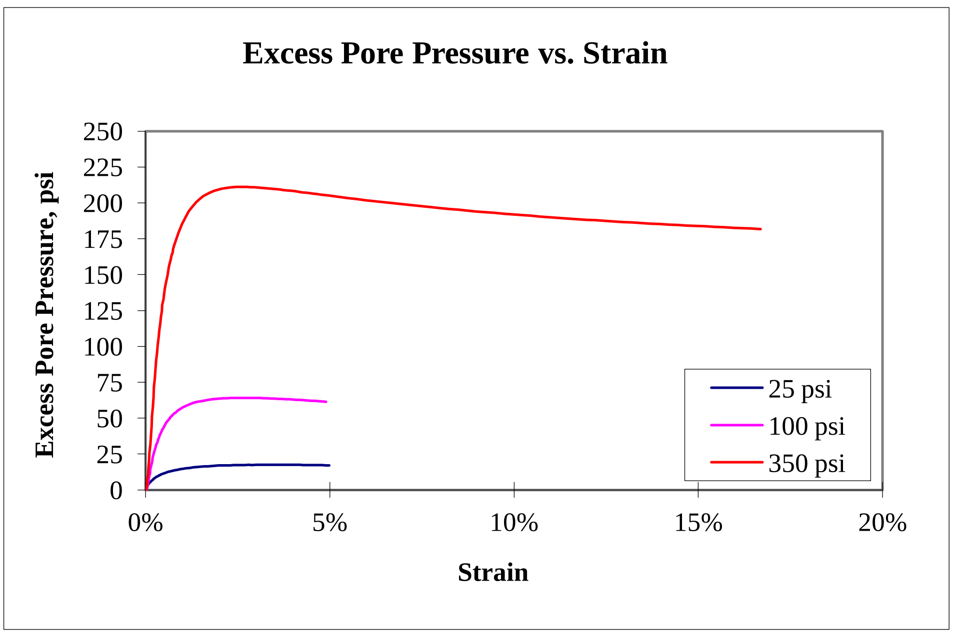
Golder Associates Inc.
Denver, Colorado

Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION

Title:
TRIAXIAL SHEAR TEST REPORT
SAMPLE DATA AND CALCULATIONS

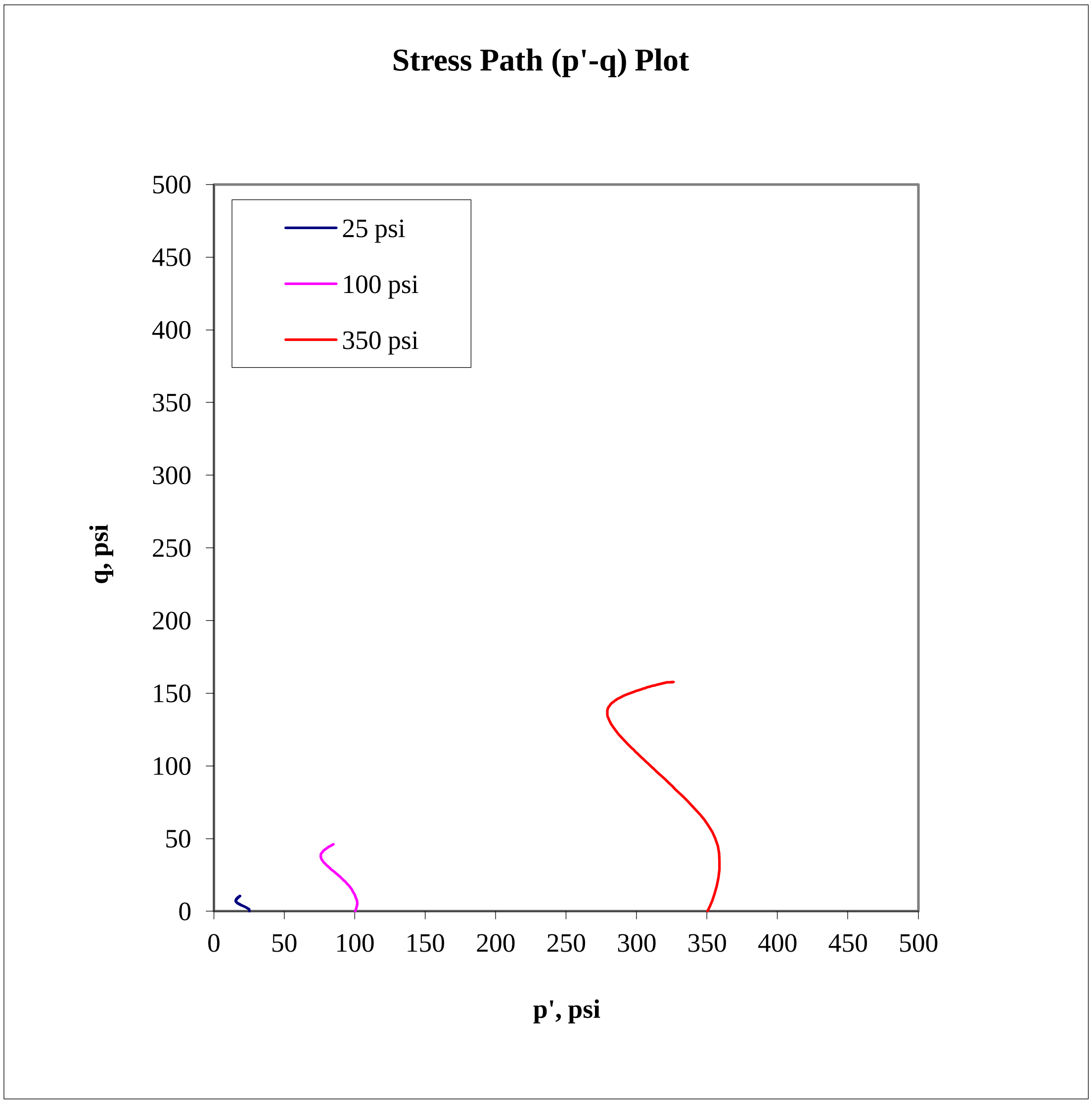
Sample Number:	Reviewed:	Date:	Job Number:	Figure:
GA-11B-21 Sonic Core @ 28'	JEO	12/10/2011	113-81608	1





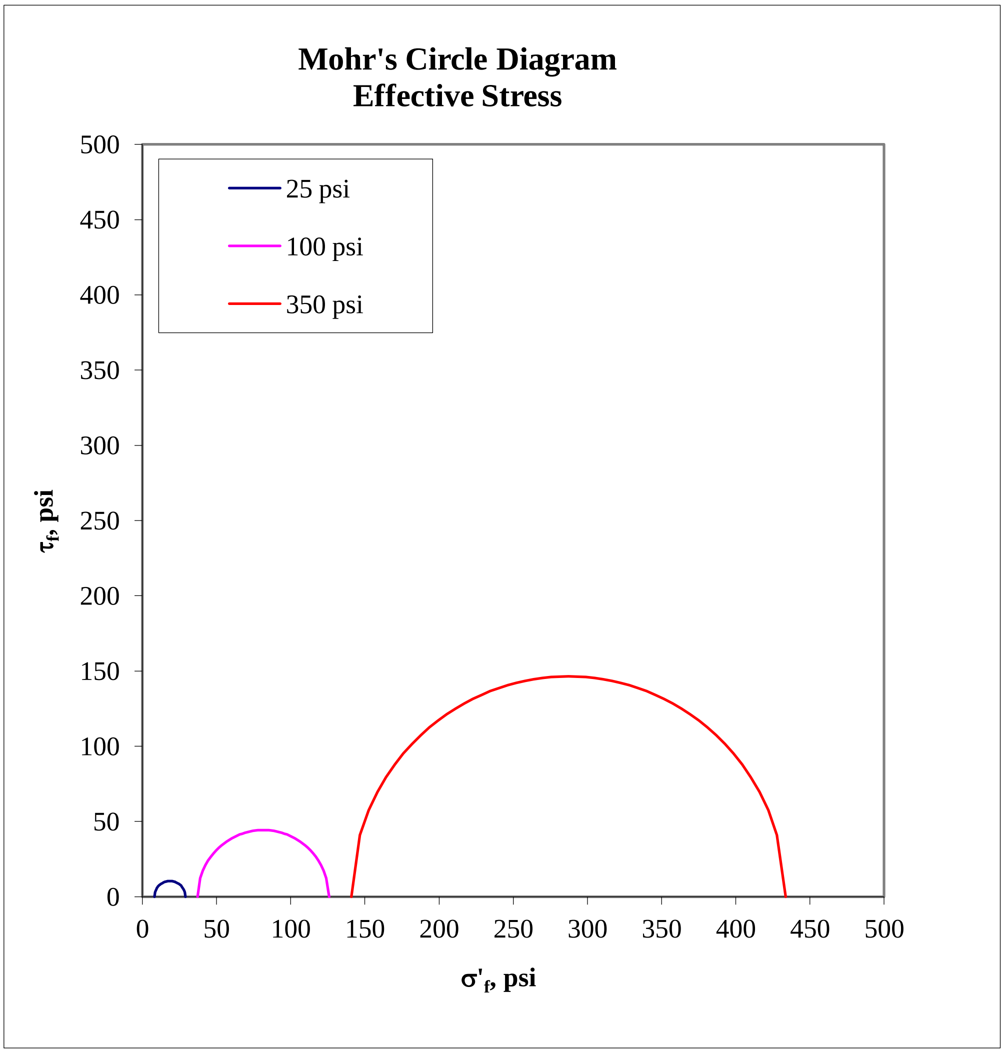
Golder Associates Inc. **Denver, Colorado**	**Title:** **C-U TRIAXIAL SHEAR DATA** **q AND EXCESS PORE PRESSURE PLOTS**
Job Short Title: **CLIMAX/GEOTECHNICAL INVESTIGATION**	

Sample Number:	**Reviewed:**	**Date:**	**Job Number:**	**Figure:**
GA-11B-21 Sonic Core @ 28'	**JEO**	**12/10/2011**	**113-81608**	**2**



Golder Associates Inc. Denver, Colorado	Title:
Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION	C-U TRIAXIAL SHEAR DATA STRESS PATH PLOT

Sample Number:	Reviewed:	Date:	Job Number:	Figure:
GA-11B-21 Sonic Core @ 28'	JEO	12/10/2011	113-81608	3

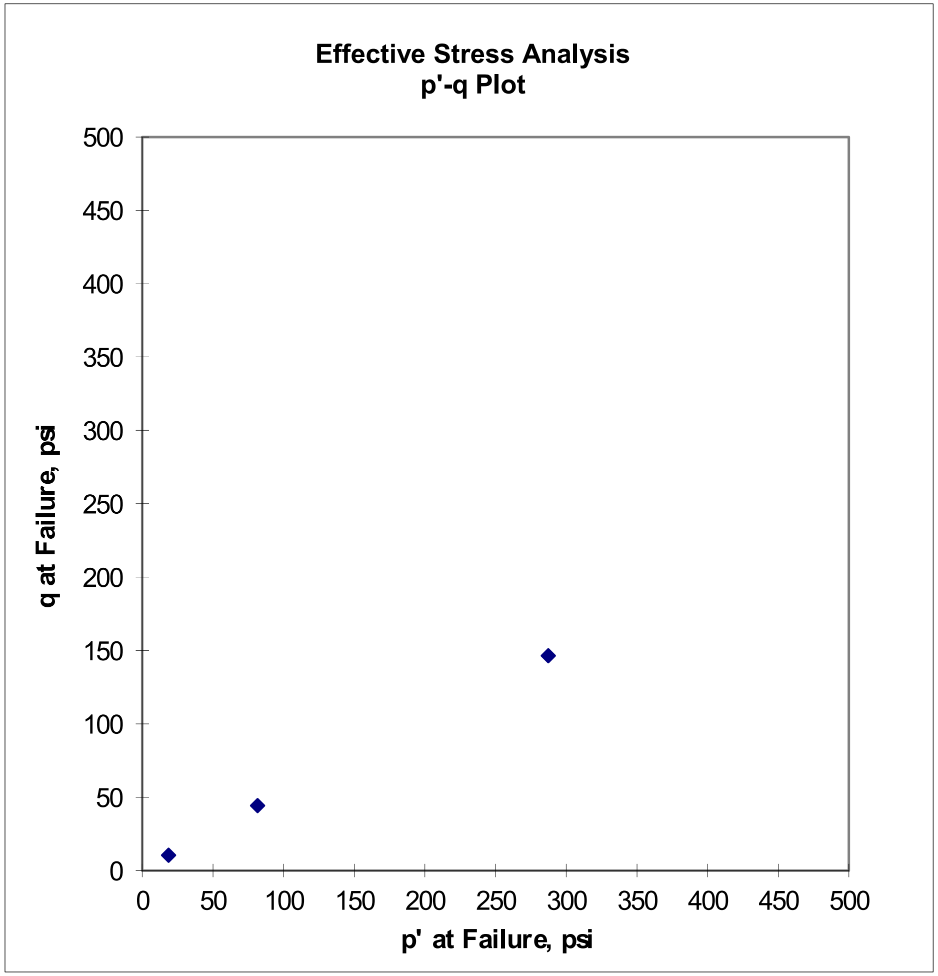


Golder Associates Inc. Denver, Colorado	Title: C-U TRIAXIAL SHEAR DATA MOHR'S CIRCLE DIAGRAM			
Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION				
Sample Number: GA-11B-21 Sonic Core @ 28'	Reviewed: JEO	Date: 12/10/2011	Job Number: 113-81608	Figure: 4

Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **CLIMAX/GEOTECHNICAL INVESTIGATION**
Project Number: **113-81608**

Sample Number	GA-11B-21 Sonic Core @ 28'
Effective Stress Analysis	

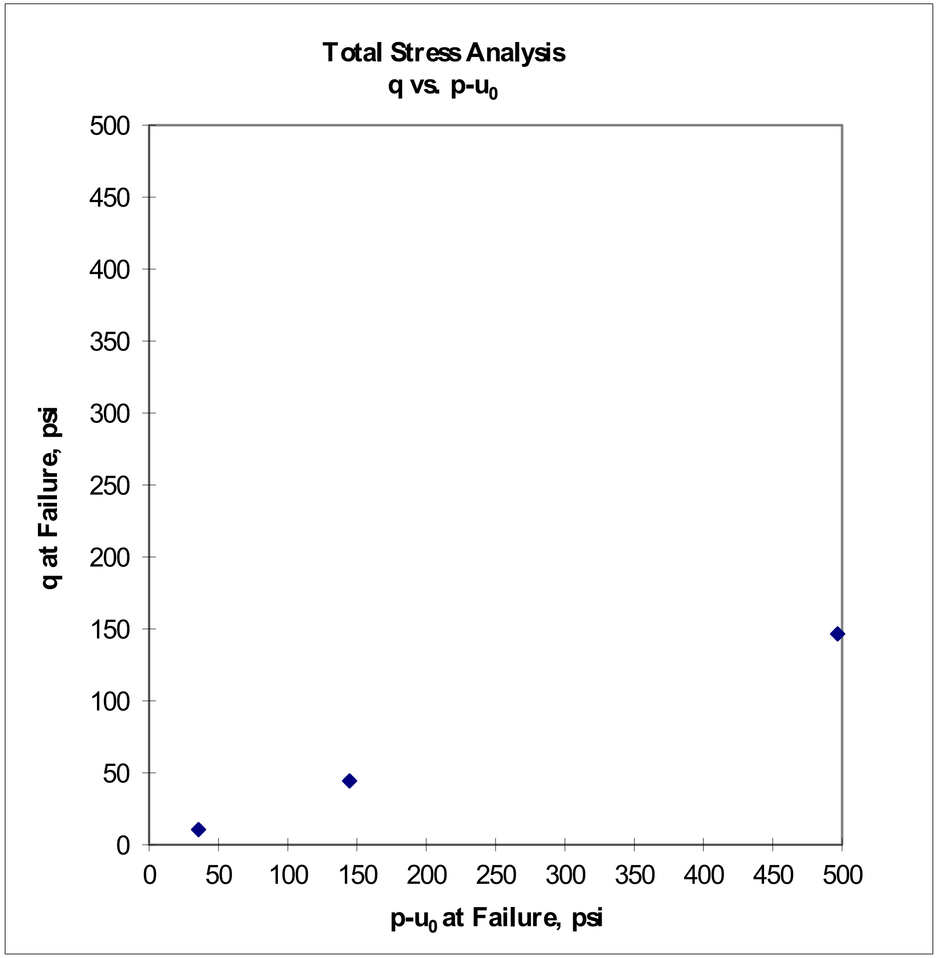
Point Number	p' (psi)	q (psi)
1	18.5	10.5
2	81.5	44.3
3	287.2	146.4



Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **CLIMAX/GEOTECHNICAL INVESTIGATION**
Project Number: **113-81608**

Sample Number	GA-11B-21 Sonic Core @ 28'
Total Stress Analysis	

Point Number	$p-u_o$ (psi)	q (psi)
1	35.7	10.5
2	144.7	44.3
3	496.8	146.4




LINEAR METRIC UNITS AND SYMBOLS
1000 millimeters (mm) 1 meter (m)
To read length in centimeters omit the final zero after the
index line To read length in millimeters, include the final zero
8THS

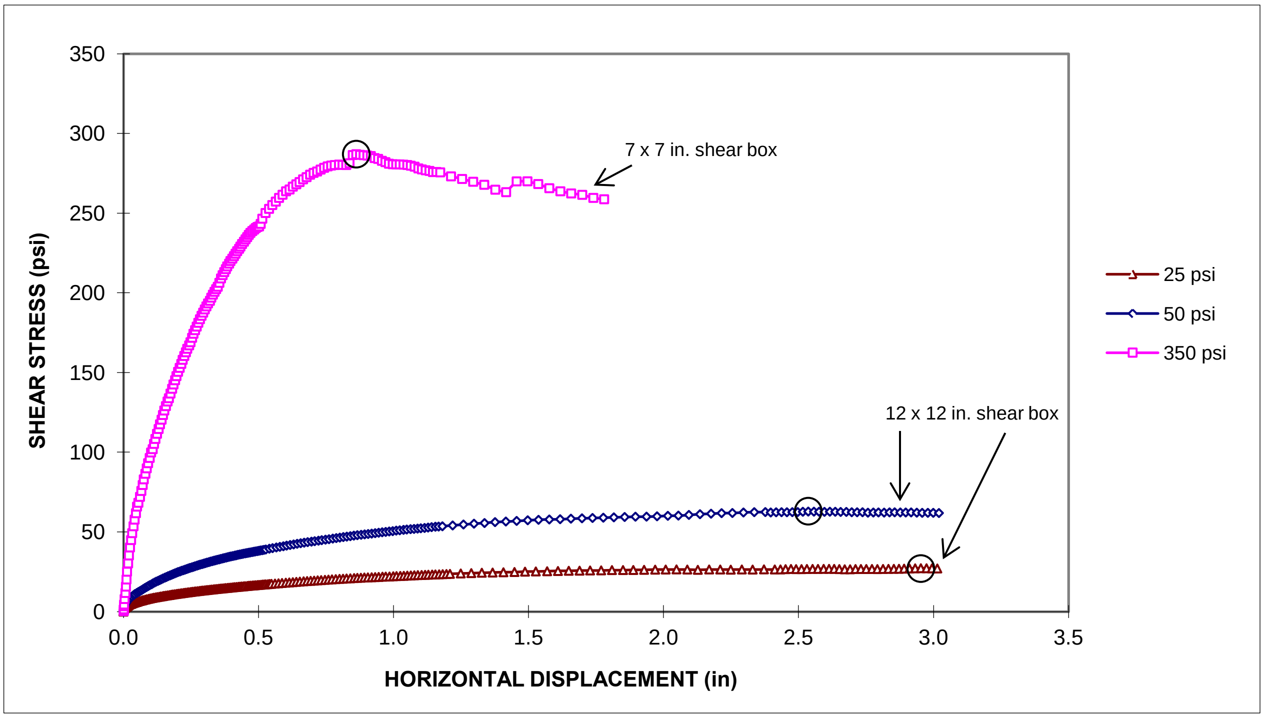
DECEMBER 2011	113-81608

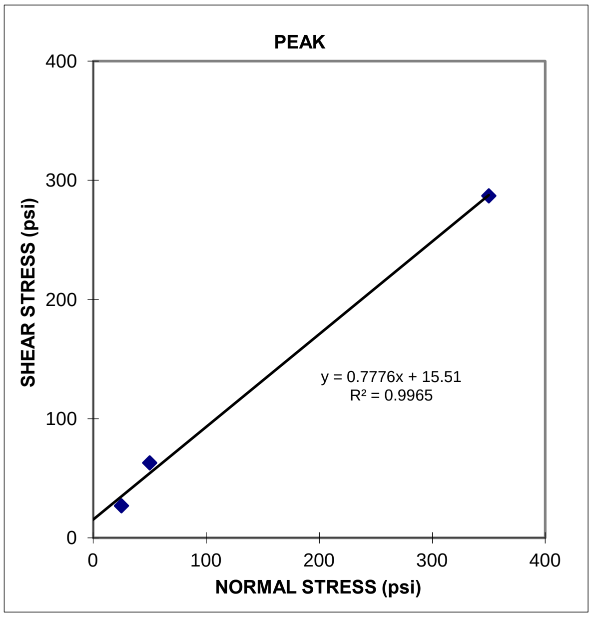
DIRECT SHEAR TEST RESULTS

ASTM D3080 MODIFIED

PROJECT NAME: CLIMAX/GEOTECHNICAL INVESTIGATION/CO
SAMPLE NUMBER: B20-2

SOIL CONDITIONS PLACED A DRY DENSITY OF 117.3 pcf AT A MOISTURE CONTENT OF 6.0%
TEST CONDITIONS: DRY, CONSOLIDATED 15 min AT NORMAL LOAD
SHEAR RATE: 0.04 in/min
SUBSTRATE: RIGID PLATES





Normal Stress (psi)	Shear Stress Peak[1] (psi)	Peak Friction Angle (°)	Peak Cohesion[2] (psi)
25	27		
50	63	37.9	15.5
350	287		

(1) The peak shear stresses for 25, 50, and 350 psi normal stresses were chosen at 2.953, 2.536, and 0.862 inches horizontal displacement, respectively.

(2) The cohesion value is based on the "best-fit" line which may not show true cohesion.

(3) The shear for the 350 psi point was terminated at 1.75 inches in horizontal displacement due to limitations of the test device.

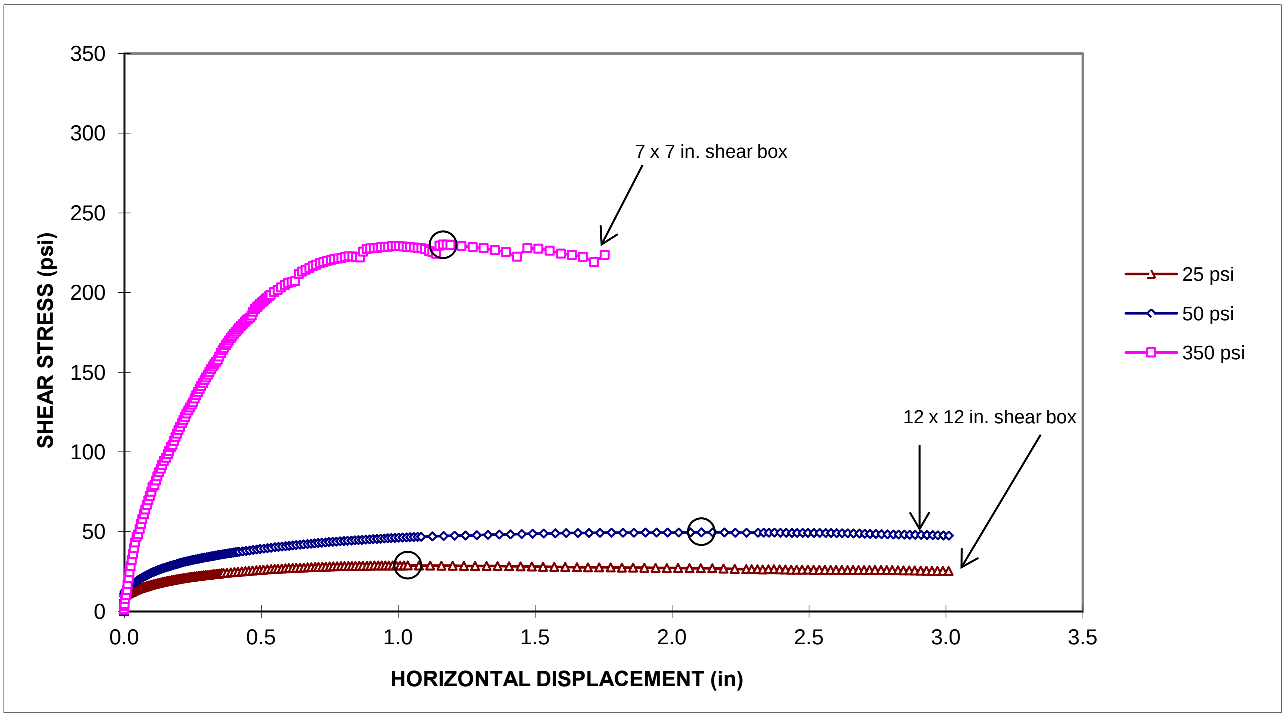
DECEMBER 2011 113-81608

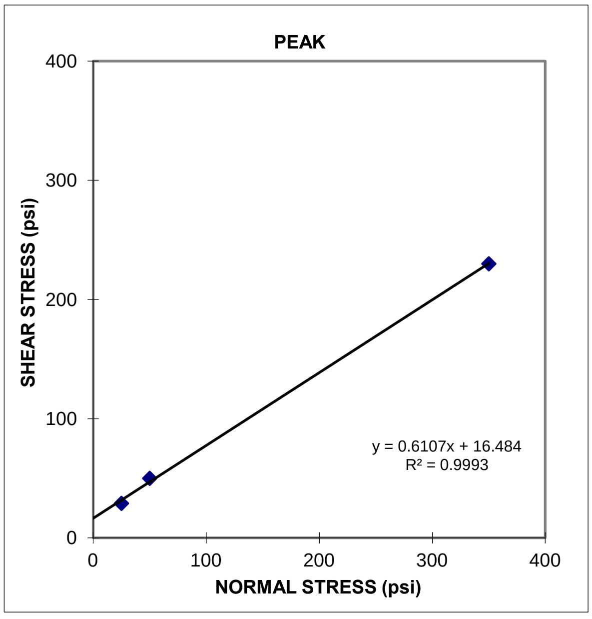
DIRECT SHEAR TEST RESULTS

ASTM D3080 MODIFIED

PROJECT NAME: CLIMAX/GEOTECHNICAL INVESTIGATION/CO
SAMPLE NUMBER: B25-14/B23-16

SOIL CONDITIONS PLACED A DRY DENSITY OF 122.8 pcf AT A MOISTURE CONTENT OF 7.3%
TEST CONDITIONS: DRY, CONSOLIDATED 15 min AT NORMAL LOAD
SHEAR RATE: 0.04 in/min
SUBSTRATE: RIGID PLATES





Normal Stress (psi)	Shear Stress Peak[1] (psi)	Peak Friction Angle (°)	Peak Cohesion[2] (psi)
25	29	31.4	16.5
50	50		
350	230		

(1) The peak shear stresses for 25, 50, and 350 psi normal stresses were chosen at 1.035, 2.107, and 1.164 inches horizontal displacement, respectively.

(2) The cohesion value is based on the "best-fit" line which may not show true cohesion.

(3) The shear for the 350 psi point was terminated at 1.75 inches in horizontal displacement due to limitations of the test device.

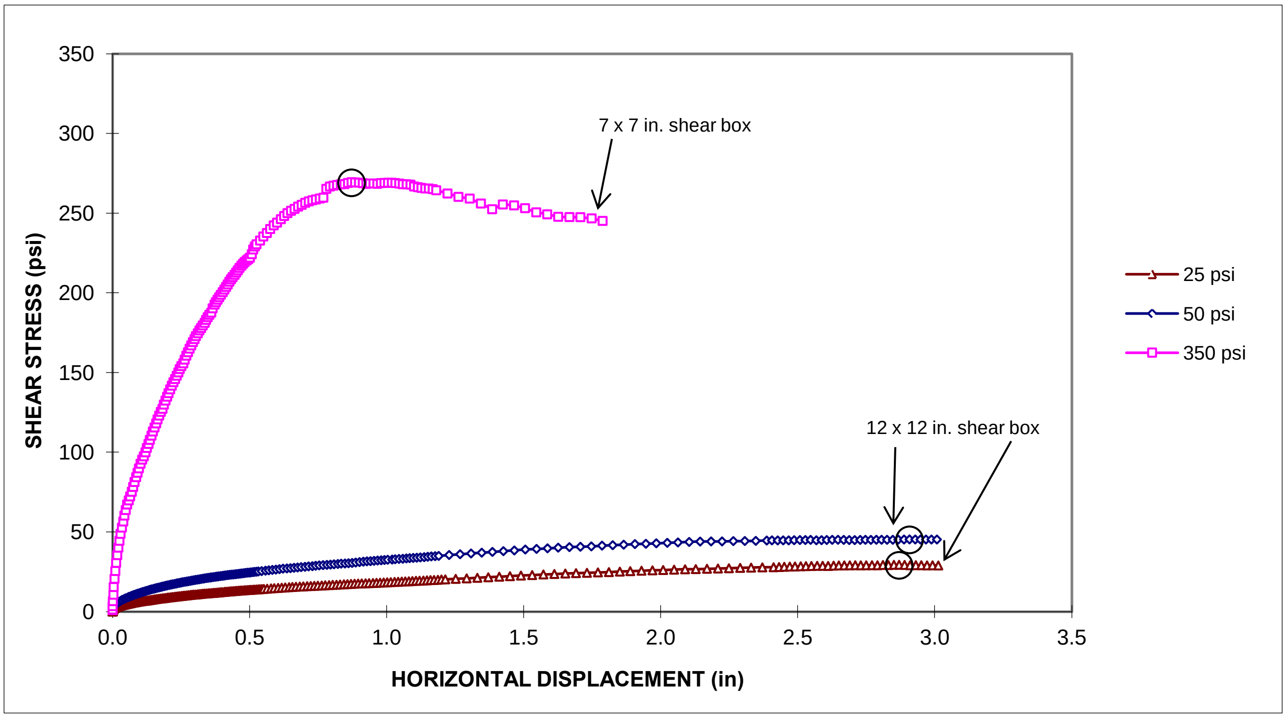
DECEMBER 2011 113-81608

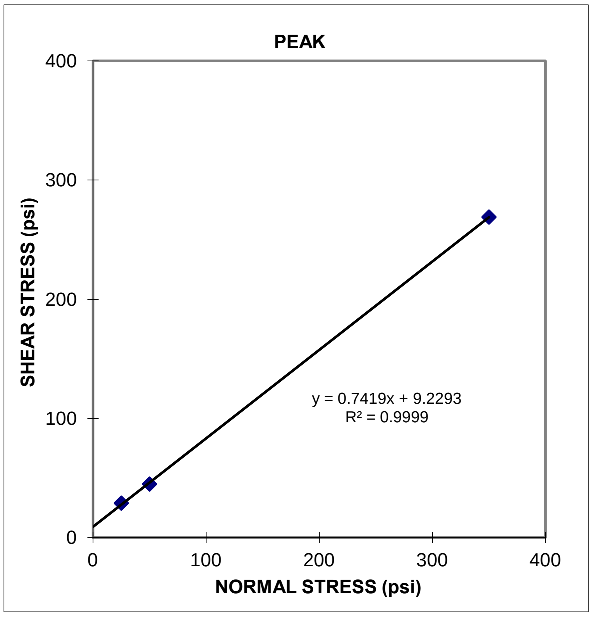
DIRECT SHEAR TEST RESULTS

ASTM D3080 MODIFIED

PROJECT NAME: CLIMAX/GEOTECHNICAL INVESTIGATION/CO
SAMPLE NUMBER: P05-1

SOIL CONDITIONS PLACED A DRY DENSITY OF 104.3 pcf AT A MOISTURE CONTENT OF 8.0%
TEST CONDITIONS: DRY, CONSOLIDATED 15 min AT NORMAL LOAD
SHEAR RATE: 0.04 in/min
SUBSTRATE: RIGID PLATES





Normal Stress (psi)	Shear Stress Peak[1] (psi)	Peak Friction Angle (°)	Peak Cohesion[2] (psi)
25	29		
50	45	36.6	9.2
350	269		

(1) The peak shear stresses for 25, 50, and 350 psi normal stresses were chosen at 2.870, 2.908, and 0.872 inches horizontal displacement, respectively.

(2) The cohesion value is based on the "best-fit" line which may not show true cohesion.

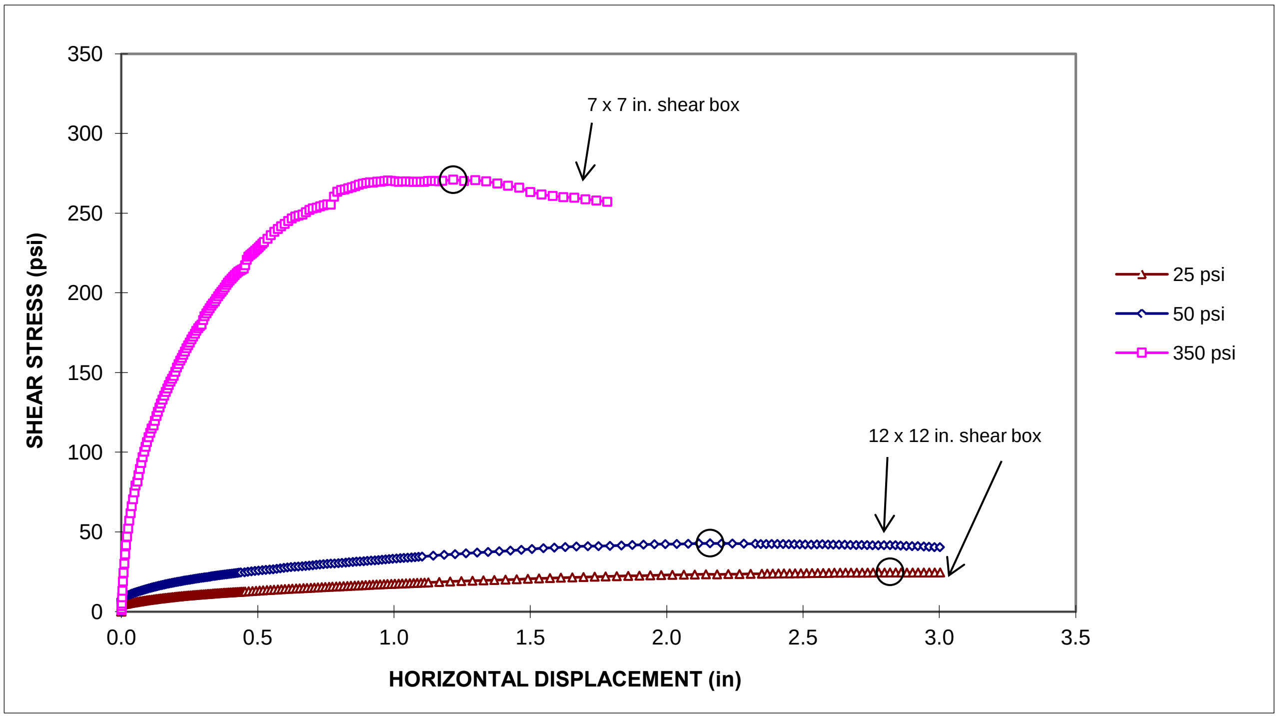
(3) The shear for the 350 psi point was terminated at 1.75 inches in horizontal displacement due to limitations of the test device.

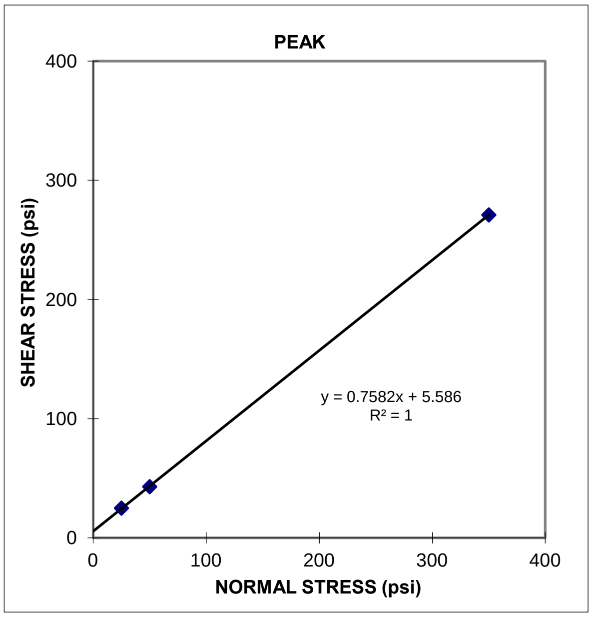
DIRECT SHEAR TEST RESULTS

ASTM D3080 MODIFIED

PROJECT NAME: CLIMAX/GEOTECHNICAL INVESTIGATION/CO
SAMPLE NUMBER: B25-3/B23-3

SOIL CONDITIONS PLACED A DRY DENSITY OF 101.9 pcf AT A MOISTURE CONTENT OF 6.0%
TEST CONDITIONS: DRY, CONSOLIDATED 15 min AT NORMAL LOAD
SHEAR RATE: 0.04 in/min
SUBSTRATE: RIGID PLATES





Normal Stress (psi)	Shear Stress Peak[1] (psi)	Peak Friction Angle (°)	Peak Cohesion[2] (psi)
25	25	37.2	5.6
50	43		
350	271		

(1) The peak shear stresses for 25, 50, and 350 psi normal stresses were chosen at 2.819, 2.159, and 1.217 inches horizontal displacement, respectively.

(2) The cohesion value is based on the "best-fit" line which may not show true cohesion.

(3) The shear for the 350 psi point was terminated at 1.75 inches in horizontal displacement due to limitations of the test device.

	Point 1		Point 2		Point 3	
Boring # =	GA-11B-24		GA-11B-24		GA-11B-24	
Sample # =	B24-9		B24-9		B24-13	
Depth (ft) =	51.5-53.6		51.5-53.6		71.0-73.2	
Point # =	1		2		3	
	Initial		**Initial**		**Initial**	
Length =	15.50	cm	16.27	cm	16.84	cm
Diameter =	7.27	cm	7.20	cm	7.26	cm
Wet Weight =	1233.37	g	1266.30	g	1306.90	g
Area =	41.5	cm^2	40.7	cm^2	41.4	cm^2
Sample Area =	6.43	in^2	6.31	in^2	6.42	in^2
Volume =	643.4	cm^3	662.4	cm^3	697.1	cm^3
Moisture Content =	32.6%		40.0%		25.8%	
Specific Gravity =	2.75		2.75		-	
Dry Weight of Solids =	930.14	g	904.50	g	1038.87	g
Wet Density =	1.92	g/cm^3	1.91	g/cm^3	1.87	g/cm^3
Dry Density =	1.45	g/cm^3	1.37	g/cm^3	1.49	g/cm^3
Wet Unit Weight =	119.6	pcf	119.3	pcf	117.0	pcf
Dry Unit Weight =	90.2	pcf	85.2	pcf	93.0	pcf
Cell Pressure =	65	psi	130	psi	380	psi
Back Pressure =	40	psi	30	psi	30	psi
Confining Pressure =	25	psi	100	psi	350	psi

Notes: Shear rate was 0.005 in/min. t_{50} was 1.8 min. for Point #2.
Specimen consisted of undisturbed core samples with ends trimmed flush.
Moisture content was taken from trimmings.
Peak was defined at the maximum principal stress ratio.

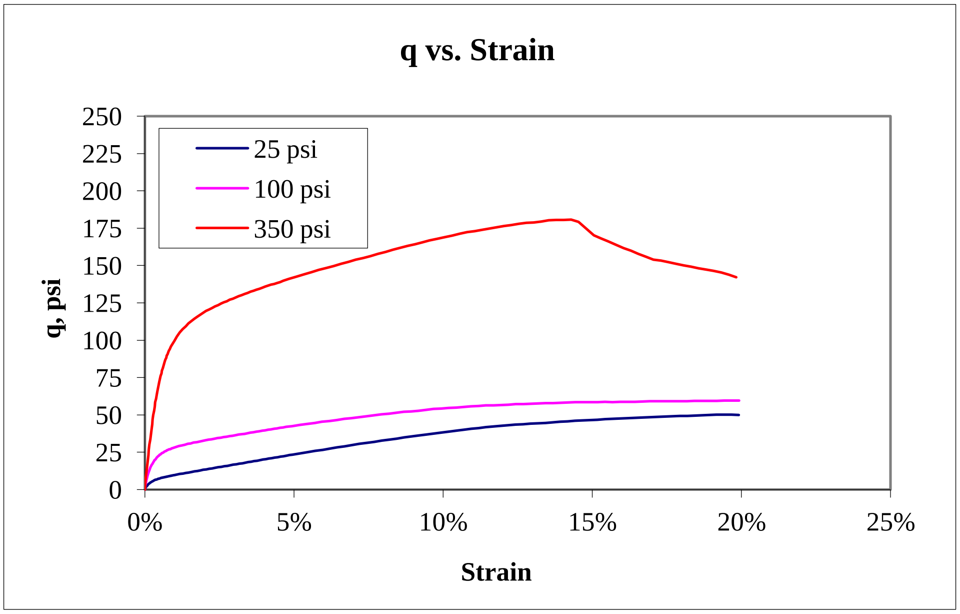
Golder Associates Inc.
Denver, Colorado

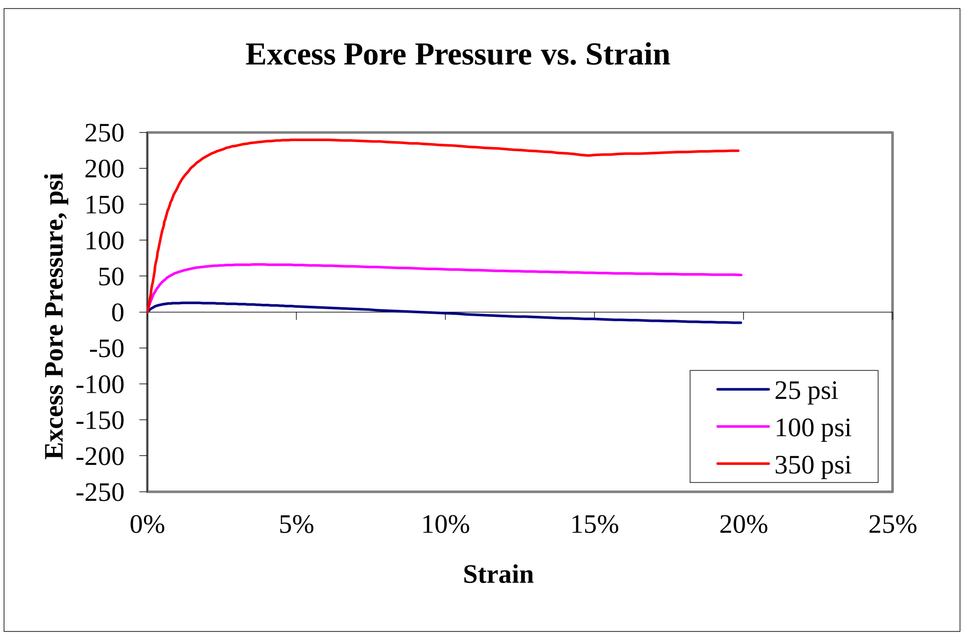
Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION

Title:

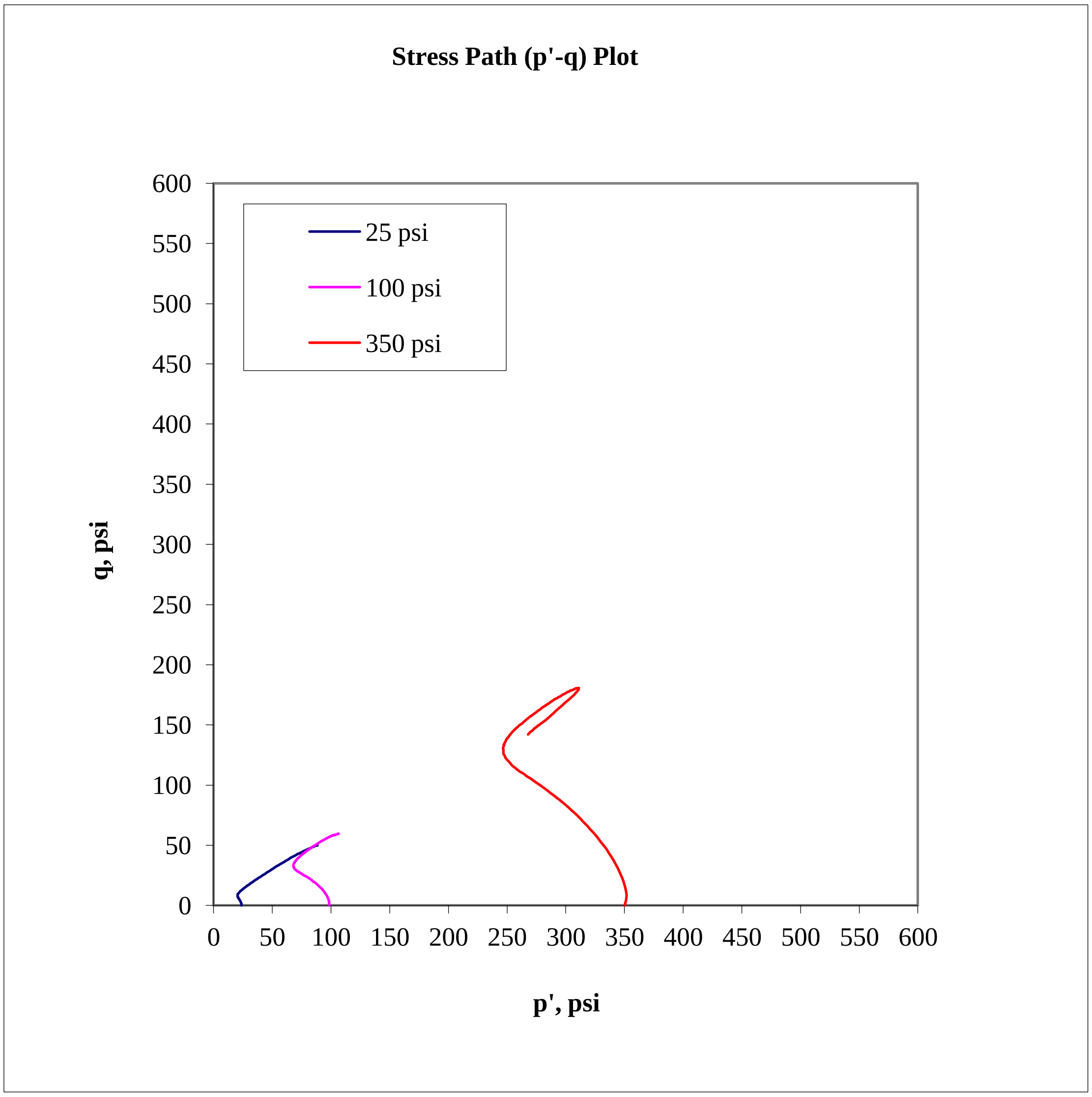
TRIAXIAL SHEAR TEST REPORT
SAMPLE DATA AND CALCULATIONS

Sample Number:	Reviewed:	Date:	Job Number:	Figure:
GA-11B-24 B24-9/B24-13	JEO	12/21/2011	113-81608	1

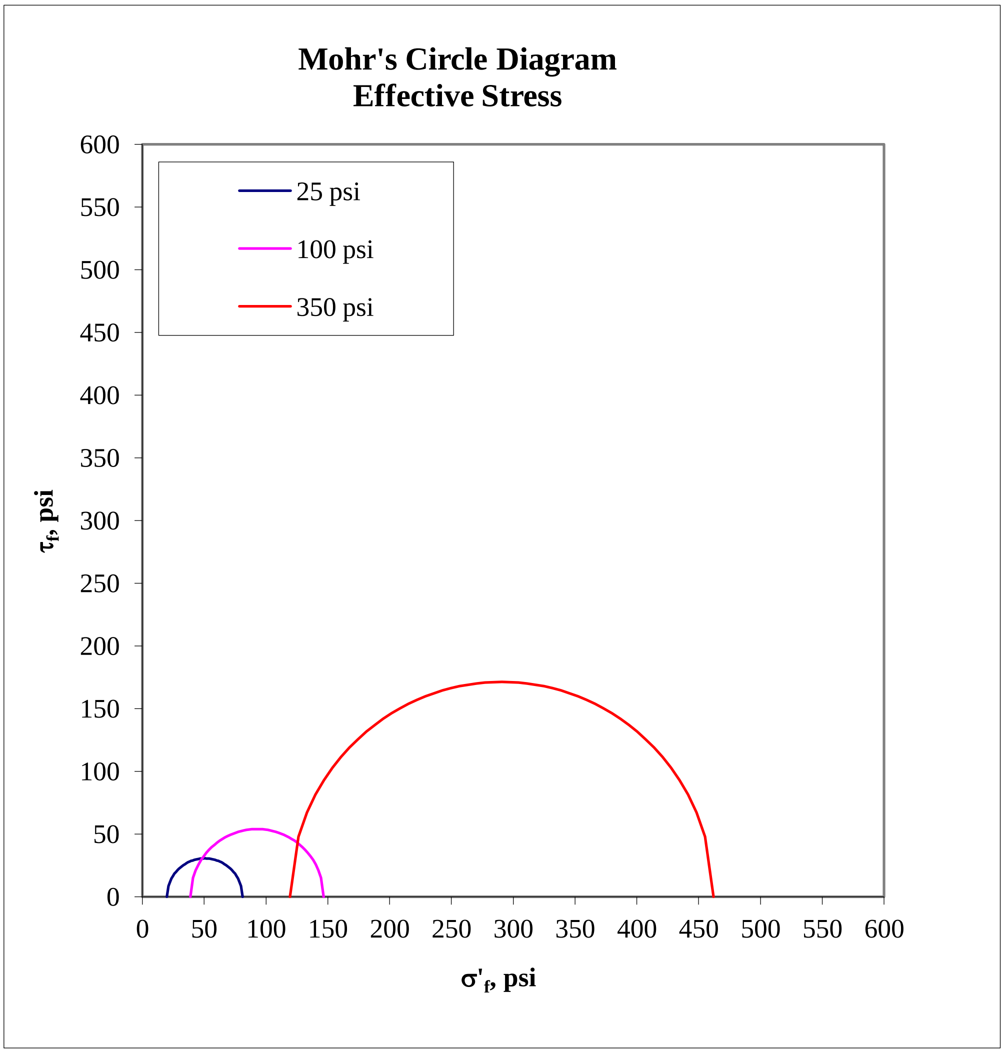




Golder Associates Inc. Denver, Colorado		Title: C-U TRIAXIAL SHEAR DATA q AND EXCESS PORE PRESSURE PLOTS		
Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION				
Sample Number: GA-11B-24 B24-9/B24-13	Reviewed: JEO	Date: 12/21/2011	Job Number: 113-81608	Figure: 2



Golder Associates Inc. **Denver, Colorado**	**Title:** **C-U TRIAXIAL SHEAR DATA** **STRESS PATH PLOT**			
Job Short Title: **CLIMAX/GEOTECHNICAL INVESTIGATION**				
Sample Number: **GA-11B-24 B24-9/B24-13**	**Reviewed:** **JEO**	**Date:** **12/21/2011**	**Job Number:** **113-81608**	**Figure:** **3**

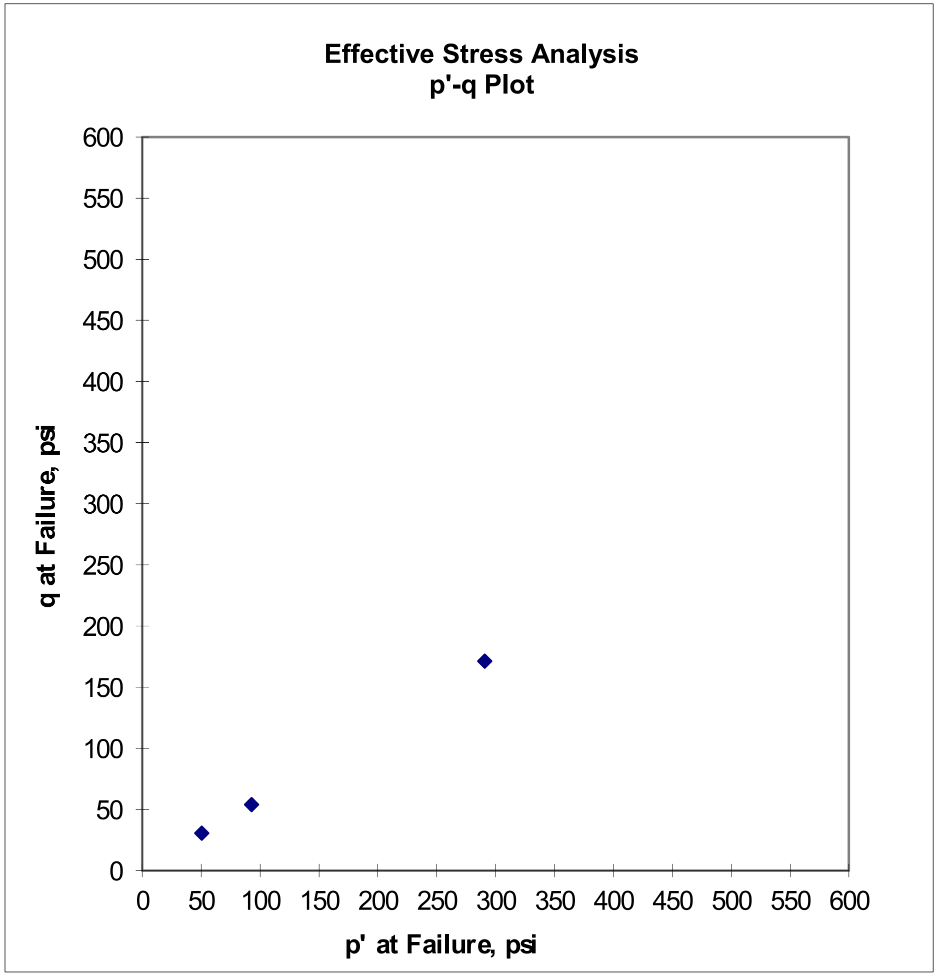


Golder Associates Inc. Denver, Colorado			Title: C-U TRIAXIAL SHEAR DATA MOHR'S CIRCLE DIAGRAM	
Job Short Title: CLIMAX/GEOTECHNICAL INVESTIGATION				
Sample Number: GA-11B-24 B24-9/B24-13	Reviewed: JEO	Date: 12/21/2011	Job Number: 113-81608	Figure: 4

Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: **CLIMAX/GEOTECHNICAL INVESTIGATION**
Project Number: **113-81608**

Sample Number	GA-11B-24 B24-9/B24-13
Effective Stress Analysis	

Point Number	p' (psi)	q (psi)
1	50.4	30.6
2	92.7	53.9
3	290.7	171.3

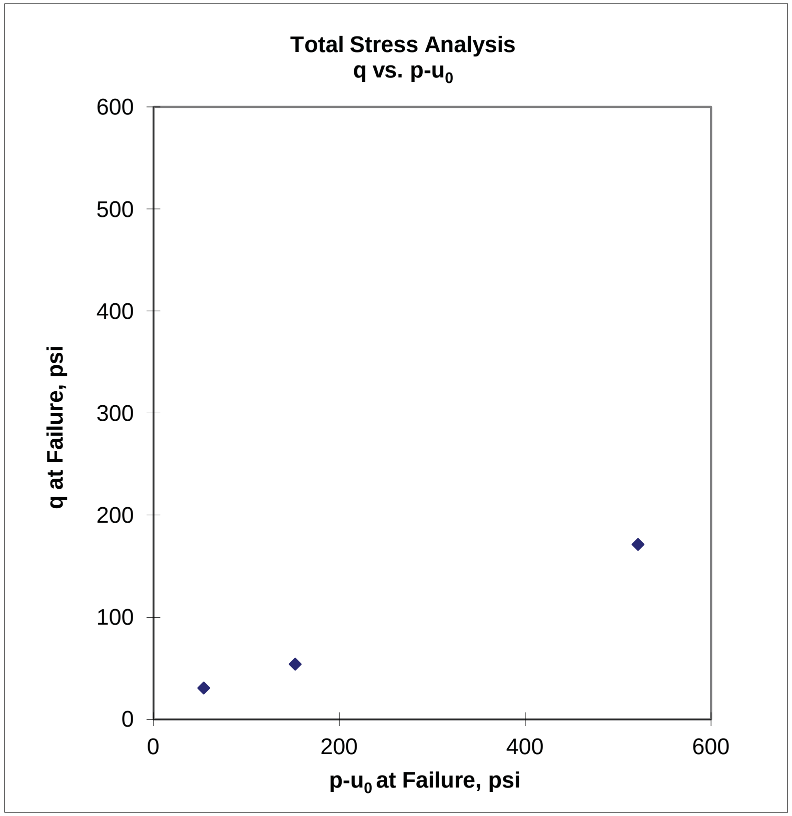


Consolidated-Undrained Triaxial Lab Data
From: GOLDER ASSOCIATES INC.
Project: Climax/Geotechnical Investigation
Project Number: 113-81608

Sample Number Boring GA-11B-24

Total Stress Analysis

Point Number	$p-u_o$ (psi)	q (psi)
1	54.1	30.6
2	152.5	53.9
3	521.4	171.3




25 psi


100 psi


350 psi

ONE-DIMENSIONAL CONSOLIDATION
ASTM D 2435

PROJECT NAME:	**Climax/Geotechnical Investigation**	**SAMPLE:**	**GA-11B-24 B24-9**	DATE	12/14/2011
PROJECT #:	**113-81608**	**DEPTH:**	**51.5-53.6 ft**	TECH	RJM
				REVIEW	JEO

SAMPLE DATA, GENERAL

height (in)	0.9955
diameter (in)	2.492
area (in^2)	4.878
volume (in^3)	4.856
specimen weight,wet (g)	151.95
specimen weight,dry (g)	115.17
water weight (g)	36.78

SAMPLE DATA, INITIAL

total height (in)	0.9955
height of solids (in)	0.5236
height of voids (in)	0.4719
void ratio	0.901
dry density (pcf)	90.1
moist density (pcf)	119.2

SAMPLE DATA, FINAL

total height (in)	0.9308
height of solids (in)	0.5236
height of voids (in)	0.4071
void ratio	0.777
dry density (pcf)	96.6
moist density (pcf)	126.1

VISUAL DESCRIPTION

Light gray, moist silt

LL:	NP
PL:	NP
PI:	NP
Gs:	2.75

MOISTURE CONTENT, INITIAL

tare #	B-7A
wt soil&tare,moist	195.92
wt soil&tare,dry	155.56
wt tare	29.18
wt moisture	40.36
wt dry soil	126.38
% moisture	31.9%

MOISTURE CONTENT, FINAL

tare #	M-5
wt soil&tare,moist	174.82
wt soil&tare,dry	140.40
wt tare	27.35
wt moisture	34.42
wt dry soil	113.05
% moisture	30.4%

PRESSURE (ksf)	h100 Sample Height	D50 Sample Height	t50 TIME (min)	Sample Density (pcf)	VOID RATIO e	DRAINAGE PATH (DOUBLE DRAINAGE) H (in)	H (cm)	DRAINAGE PATH (DOUBLE DRAINAGE) H^2 (in^2)	H^2 (cm^2)	COEFFICIENT OF CONSOLIDATION Cv (cm^2/sec)	(ft^2/day)	Cc
0.10	0.9878	-	-	90.9	0.889	-	-	-	-	-	-	-
0.25	0.9818	-	-	91.4	0.877	-	-	-	-	-	-	-
0.50	0.9780	-	-	91.8	0.870	-	-	-	-	-	-	-
0.50	0.9771	-	-	91.8	0.868	-	-	-	-	-	-	-
1.0	0.9713	0.9739	0.195	92.4	0.857	0.4869	1.2368	0.2371	1.5298	2.58E-02	2.40E+00	0.037
2.0	0.9624	0.9662	0.177	93.3	0.840	0.4831	1.2270	0.2334	1.5056	2.80E-02	2.61E+00	0.057
4.0	0.9497	0.9546	0.164	94.5	0.816	0.4773	1.2124	0.2278	1.4698	2.95E-02	2.75E+00	0.081
8.0	0.9338	0.9403	0.160	96.1	0.785	0.4702	1.1942	0.2211	1.4262	2.92E-02	2.73E+00	0.101
16.0	0.9123	0.9210	0.158	98.4	0.744	0.4605	1.1697	0.2121	1.3682	2.84E-02	2.65E+00	0.136
4.0	0.9158	-	-	98.0	0.751	-	-	-	-	-	-	-
1.0	0.9224	-	-	97.3	0.764	-	-	-	-	-	-	-
0.25	0.9308	-	-	96.4	0.780	-	-	-	-	-	-	-

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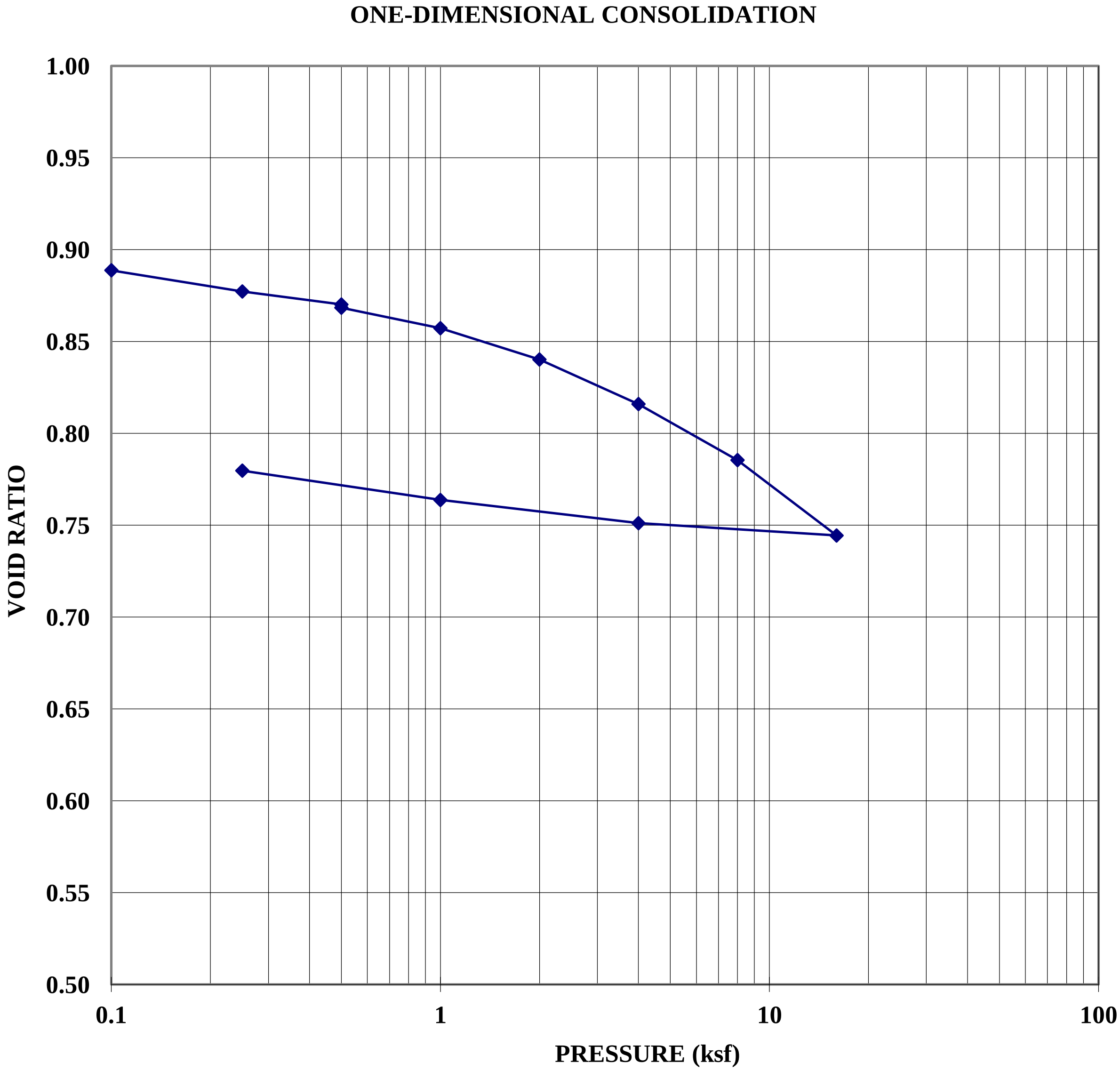


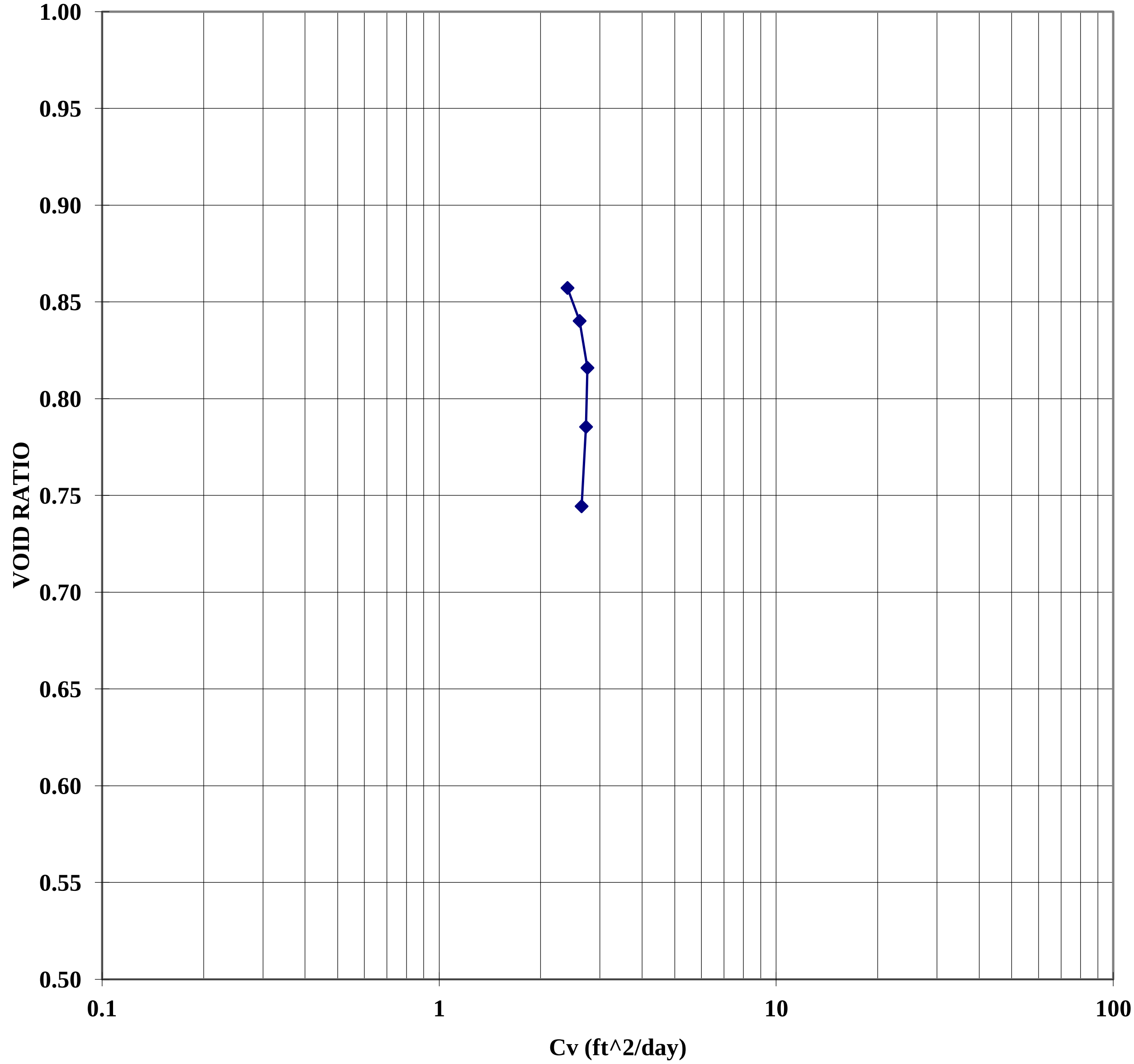
SAMPLE #:	GA-11B-24 B24-9 51.5-53.6 ft	
DESCRIPTION:	Light gray, moist silt	

		DATE	12/14/2011
PROJECT NAME:	Climax/Geotechnical Investigation	TECH	RJM
PROJECT #:	113-81608	REVIEW	JEO

GOLDER ASSOCIATES INC.

LAKEWOOD, COLORADO

ONE-DIMENSIONAL CONSOLIDATION



SAMPLE #:	GA-11B-24 B24-9 51.5-53.6 ft
DESCRIPTION:	Light gray, moist silt

		DATE	12/14/2011
PROJECT NAME:	Climax/Geotechnical Investigation	TECH	RJM
PROJECT #:	113-81608	REVIEW	JEO

GOLDER ASSOCIATES INC.
LAKEWOOD, COLORADO

APPENDIX C

Date:	April 24, 2012	**Prepared by:**	D. Rugg
Project No.:	113-81608.2000	**Checked by:**	D. Geier
Projcect Title:	Climax Molybdenum OSF Design Report	**Reviewed by:**	B. Bronson
RE:	**GEOTECHNICAL STABILITY ANALYSIS**		

1.0 OBJECTIVE

The objective of this appendix is to present the calculations performed to evaluate the global stability of the North 40 and McNulty overburden storage facilities (OSFs) at the Climax molybdenum mine in Climax, Colorado. A geotechnical field investigation consisting of 19 test pits and 7 boreholes was conducted in October and November 2011, and is summarized in the design report (Golder, 2012) and in Appendix A to the report. Representative samples collected during the field program were used to conduct a laboratory-testing program, the results of which are also presented in Appendix B of the design report. The results of the field investigation and laboratory testing program, in conjunction with other available information, including historical reports, existing topography, aerial photographs, and geologic maps, were analyzed to create the geologic cross sections including bedrock contacts and piezometric surfaces. The field and laboratory program results and historical reports were also used to determine the design parameters utilized in this analysis.

2.0 DESIGN CRITERIA

An OSF Loading Plan (OSF Plan) was provided to Golder by Climax on April 23, 2012. The OSF Plan utilizes an ultimate footprint that is consistent with the operational requirements, post-closure requirements, and project design criteria for the facility. Based on Climax's current OSF Plan, the majority (approximately 70%) of the overburden material will consist of igneous and metamorphic rock excavated from the pit which is either unmineralized or contains uneconomical mineralization. The remaining approximate 30% of the overburden removed from the pit will consist of sedimentary rock primarily derived from the Minturn Formation shale, siltstone, and sandstone.

It is common for mines to update OSF Loading Plans throughout the life of the project for a variety of reasons. These routine changes occur within the framework established by the project design criteria. Therefore, the relevant parameters in the design criteria were used to construct the OSF stability cross sections. Sections constructed in this manner are intended to represent the "worst-case" cross-section geometry (i.e., steepest slopes) possible within the constraints of the design criteria. The relevant design criteria are listed below:

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Lakewood, CO 80228 USA
Tel: (303) 980-0540 Fax: (303) 985-2080 www.golder.com

- The operational and closure toe limits of the OSF are shown along with the locations of the design cross sections in Figure 1
- Operational scenario:
  - Inter-bench angle of repose slopes were modeled as 1.4H:1V (or 36 degrees)
  - Operational benches were modeled as 200 feet wide
  - The maximum height between benches was modeled as 200 feet
  - Overall operational slopes were thus approximately 2.4H:1V
  - During operational stages, the low-grade ore stockpile, located in the southwest corner of the OSF footprint, may temporarily have angle of repose slopes in excess of 200 feet. These slopes were considered in the stability analysis.
- Closure scenario:
  - Inter-bench slopes were modeled as 2H:1V
  - Closure benches were modeled as 20 feet wide
  - The maximum height between benches was modeled as 56 feet
  - Overall closure slopes were thus approximately 2.4H:1V

By using the design criteria above, Golder has created the steepest slopes that accommodate the design criteria at each cross-section location. Other pertinent design criteria include the acceptable stability factors of safety and the design earthquake conditions as discussed below:

- Operational scenario:
  - Minimum allowable static factor of safety is 1.4
  - Minimum allowable seismic factor of safety is ≥1.0
  - Operational basis earthquake (OBE) peak ground acceleration (PGA) is 0.06g (representing the 1-in-475-years event).
- Closure scenario:
  - Minimum allowable static factor of safety is 1.5
  - Minimum allowable seismic factor of safety is ≥1.0
  - Maximum design earthquake (MDE) PGA is 0.14g (representing the 1-in-2,475-years event)

3.0 METHODS AND ASSUMPTIONS

- Primary stability analyses were performed with RocScience's 2-D limit equilibrium program, Slide 6.0. Factors of safety were computed based on Spencer's Method of Slices (Spencer 1967).
- Both circular and non-circular (block) failure surfaces were evaluated.
- Since approximately 30% of the OSF is expected to be comprised of overburden derived from the Minturn Formation shale, siltstone, and sandstone, two strength envelopes were utilized for the overburden material:

- Envelope 1: A conservative best approximation of the overburden material strength based on the residual strength obtained from two large scale direct shear tests performed on representative samples of overburden
- Envelope 2: A "lower bound" strength envelope defined by the Leps (1971) low strength curve. Golder considers this envelope to represent a lower bound strength for the OSF suitable for evaluating the stability of the OSF in the event that a significant contiguous portion of the facility is constructed from sedimentary overburden.

- Both deep and shallow failure surfaces were investigated. However, surficial veneer (infinite slope) slip surfaces were excluded from the results. Critical failure surfaces were constrained to a minimum depth of 15 feet.
- Veneer failures on the face of the dump become more common for the operational OSF configuration when the lower bound strength envelope is utilized. Thus, Golder has assumed and recommends that overburden material derived from the higher strength igneous or metamorphic rocks will be placed within 50 ft of the ultimate face of the OSF.
- The geometry, piezometric assumptions, and material parameters were obtained based on the field investigation performed in October and November 2011. In areas without tailing deposits, the existing piezometric surface was measured an average of 14 feet below the native ground surface (i.e., 14 feet below the base of existing fills, or 14 feet below the present ground surface in areas with no fill). No perched water was encountered within any of the existing overburden fills. In areas with historic tailing deposits, the piezometric surface was located within the upper 10 feet of tailing deposits.
- A sensitivity analysis was performed in order to evaluate the effect of varying piezometric levels on OSF stability. For the sensitivity analysis, a conservative, worst-case piezometric surface was assumed to exist at the top of native ground and at the surface of the historic tailing impoundments. Stability was evaluated along the two most critical cross-sections under static conditions.
- Design material parameters were determined based on a series of geotechnical laboratory tests conducted by Golder on samples of native soils (Lincoln Porphyry, Minturn Formation, and Glacial Till) and mine materials (overburden and tailing). These soil parameters are described in Section 4.0.
- Residual strength soil parameters were used for the static stability analysis wherever applicable.
- Seismic stability was evaluated using a pseudo-static analysis procedure generally following the Hynes-Griffin and Franklin method (1984). For this pseudo-static analysis, total stress shear strength parameters were used for the tailing materials, while 80 percent of the effective stress shear strength was used for all other materials (excluding the overburden material since straining and strength degradation due to shaking are expected to be minimal for this material). Seismic load coefficients of 0.03 for the OBE and 0.07 for the MDE (half the PGA for each case) were used for the pseudo-static analyses.

4.0 MATERIAL PARAMETERS

The material properties presented in Table 1 and Figure 2, and Table 2 and Figure 3 were used for the static and pseudo-static analyses, respectively. These parameters were selected based on a review of the available laboratory test data, historical reports, and engineering judgment. A more in-depth discussion of material strength selection is presented in the main text of the design report.

Table 1 presents the material properties used for the static stability analyses. A combination of Mohr-Coulomb and bi-linear Mohr-Coulomb failure envelopes were used for the native materials and tailing. A traditional Mohr-Coulomb failure envelope was used for the Minturn Formation. The envelope was obtained from a staged consolidated-undrained triaxial test performed on a relatively undisturbed sample of clayey residual soil weathered from the Minturn Formation. Bi-linear Mohr-Coulomb failure envelopes were constructed for both the Lincoln Porphyry and Glacial Till. For these materials, bi-linear envelopes were found to provide the best fit to the data provided by large-scale direct shear tests on reconstituted samples of these materials. The strength of the tailing material was determined by a series of consolidated-undrained triaxial tests on tailing fines. The Mohr-Coulomb envelope for tailing assumes no effective cohesion.

Two large scale direct shear tests were performed on samples of mine overburden collected from the site. The shear box was 12 inches by 12 inches, and as a result only the sampled material finer than 2 inches was used in the test. Assuming zero cohesion, the results indicate residual strengths of 35 to 36 degrees (linear Mohr-Coulomb). A power curve best fit the laboratory data lies approximately midway between the Leps (1971) curves for low and average strength rockfill.

For the Climax mine overburden, the curvilinear power curve fit to the large scale direct shear test data was selected for use in stability modeling. This curve is considered representative of expected worst-case conditions within the OSFs for areas where overburden derived from igneous and/or metamorphic rock makes up the majority of the OSF fill. For the majority of the OSF this strength envelope is considered conservative, as the tests were performed only on the finer-grained matrix material, and was not corrected to account for the large amount of oversize material present in the OSFs. Note that approximately 30% of the overburden is expected to consist of sedimentary rock. The power curve described above is also considered representative for areas of the OSF containing average quantities of sedimentary rock derived overburden (i.e., approximately 30%). Golder considers the low strength Leps (1971) curve an appropriate “lower bound” strength envelope for the OSF. The low strength Leps envelope is suitable for evaluating the stability of the OSF in the event that a significant contiguous portion of the facility is constructed primarily from sedimentary overburden.

Table 2 presents the material properties used for the pseudo-static stability analyses. For this case, all native soil shear strengths were reduced by 20 percent, in accordance with Makdisi and Seed (1977) to simulate the elastic reduction in strength (i.e., strain softening) that may be imparted by seismic shaking. This practice was also adopted by Hynes-Griffin and Franklin (1984) for pseudo-static stability analyses. This reduction factor was not applied to the overburden material since seismic shaking is not expected to produce strain softening conditions resulting from the development of excess pore pressure for the overburden materials. The results of a liquefaction screening analysis have shown that the tailing material will not liquefy under the expected seismic loading conditions.

5.0 RESULTS AND CONCLUSIONS

The stability analyses results are summarized in Tables 3 and 4 for the operations and closure scenarios, respectively. Cross-sections showing the critical failure mechanisms for each case considered are presented in Figures 4 through 33. Based on the analyses performed for this study and the summary of results presented in Tables 3 and 4, all computed factors of safety meet or exceed the factors of safety established by the Project Design Criteria, for both the maximum operational and closure slope scenarios.

The sensitivity analysis showed that the factor of safety is relatively insensitive to changes in the piezometric surface. When the worst-case groundwater conditions were modeled, static factors of safety decreased by only 0.02 to 0.08 from the base case. Although they are not anticipated to occur, increased piezometric levels beneath the OSFs would not create unstable conditions. As a result, installation of piezometers and regular monitoring of groundwater levels are not required as a component of the O&M plan.

6.0 REFERENCES

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Leps, T.M. 1971. *Review of Shearing Strength of Rockfill*. Journal of the Soil Mechanics and Foundations Division, Proc. of the American Society of Civil Engineers. Vol. 96, No. SM4, pp. 1159-1170.

Makdisi, F.I., and H.B. Seed. 1977. *A Simplified Procedure for Estimating Earthquake-Induced Deformation in Dams and Embankments,* Report UCB/EERC-77/19, Earthquake Engineering Research Center, University of California, Berkeley, CA.

Spencer, E. 1967. *A Method of Analysis of the Stability of Embankments Assuming Parallel Inter-Slice Forces."* Geotechnique, Vol. XVII, No. 1, pp. 11-26.

List of Attachments

Tables 1 through 4
Figures 1 through 33

TABLES

Table 1
Strength Parameters Utilized for Static Stability Analysis

Soil Type	Total Unit Weight (pcf)	Failure Envelope Type	Failure Envelope Definition (psf)	Notes
Native Materials				
Minturn Formation	119	Mohr-Coulomb	$\tau' = \sigma'\tan(31°)$	This failure envelope was determined from the strength results of a staged undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism.
LincolnPorphyry	117	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(50°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(33°)+4200$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests.
GlacialTill	123	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(44°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(30°)+2688$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests.
Overburden Materials				
Best Approximation Overburden Parameters	120	Power Function	$\tau' = 3.18\sigma'^{0.86}$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests. This envelope lies between the average and low envelopes developed by Leps (1971).
Lower Bound Overburden Strength Parameters	120	Power Function	$\tau' = 2.02\sigma'^{0.90}$	This failure envelope was used to account for a higher proportion of weaker materials with in the OSF. This envelope is analagous to the low strength envelope developed by Leps (1971).
Tailings Materials				
Tailings Materials	100	Mohr-Coulomb	$\tau' = \sigma'\tan(33°)$	This failure envelope was determined from the residual strength results of a staged undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism.

Table 2
Strength Parameters Utilized for Pseudo-Static Stability Analysis

Soil Type	Total Unit Weight (pcf)	Failure Envelope Type	Failure Envelope Definition (pcf)	Notes
Native Materials				
Minturn Formation	119	Mohr-Coulomb	$\tau' = \sigma' \tan(24°)$	This failure envelope was determined from the strength results of a staged consolidated undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism. Values were reduced by 20% for the seismic condition.
Lincoln Porphyry	117	Bi-Linear Mohr Coulomb	$\tau' = \sigma' \tan(44°)$ for $\sigma' < 7200$ $\tau' = \sigma' \tan(28°) + 3360$ for $\sigma' > 7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests with the values reduced by 20%.
Glacial Till	123	Bi-Linear Mohr Coulomb	$\tau' = \sigma' \tan(38°)$ for $\sigma' < 7200$ $\tau' = \sigma' \tan(25°) + 2150$ for $\sigma' > 7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests with the values reduced by 20%.
Overburden Materials				
Best Approximation Overburden Parameters	120	Power Function	$\tau' = 3.18\sigma'^{0.86}$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests. This envelope lies between the average and low envelopes developed by Leps (1971).
Lower Bound Overburden Strength Parameters	120	Power Function	$\tau' = 2.02\sigma'^{0.90}$	This failure envelope was used to account for a higher proportion of weaker materials with in the OSF. This envelope is analagous to the low strength envelope developed by Leps (1971).
Tailings Materials				
Tailings Materials	100	Mohr-Coulomb	$\tau = \sigma \tan(18°)$	This failure envelope was determined from the residual strength results of a consolidated undrained triaxial test with pore pressure measurements. A total stress approach was utilized for the seismic condition.

Table 3
Stability Analysis Results for the Maximum Operational OSF Configuration

Section	Seismicity	Minimum Factor of Safety- Lower Bound Overburden Strength[1]	Minimum Factor of Safety- Best Approximation Overburden
A-A	static	1.42	1.59
A-A	pseudo-static	1.21	1.28
B-B	static	1.41	1.59
B-B	pseudo-static	1.20	1.25
C-C	static	1.48	1.49
C-C	pseudo-static	1.09	1.09
D-D	static	1.45	1.59
D-D	pseudo-static	1.26	1.28
E-E	static	1.44	1.57
E-E	pseudo-static	1.15	1.19

[1]Minimum factors of safety for the lower bound overburden strength are for significant failures that span at least an entire operational lift. Single lift "veneer" failures were excluded. It was assumed and Golder recommends that 50 ft of material derived from igneous or metamorphic sources is placed at the ultimate face of the dump. Minimum factors of safety for single lift veneer failures are greater than 1.4.

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Table 4
Stability Analysis Results for the Post-Closure OSF Configuration

Section	Seismicity	Minimum Factor of Safety- Lower Bound Overburden Strength	Minimum Factor of Safety- Best Approximation Overburden
A-A	static	1.95	2.04
A-A	pseudo-static	1.24	1.30
B-B	static	1.92	2.00
B-B	pseudo-static	1.19	1.27
C-C	static	1.50	1.52
C-C	pseudo-static	1.00	1.00
D-D	static	1.70	1.73
D-D	pseudo-static	1.13	1.15
E-E	static	1.58	1.63
E-E	pseudo-static	1.10	1.10

FIGURES

LEGEND

EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)

LIMITS OF OSF DURING OPERATIONS (2.4H:1V OVERALL SLOPES)

LIMITS OF OSF AT CLOSURE

CLIMAX PROPERTY LIMITS

GAI STABILITY SECTIONS

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. AERIAL PHOTOGRAPH TAKEN IN 2009, FROM USDA AERIAL PHOTOGRAPHY FIELD OFFICE.

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
B	4APR12	ISSUED FOR DESIGN REPORT	DLG	GSL	DLG	BRB
A	24FEB12	ISSUED FOR MEMORANDUM	DLG	GSL	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES, COLORADO

TITLE

OSF FOOTPRINT AND STABILITY SECTIONS

Golder Associates
DENVER, COLORADO

FILE No. 11381608C001_4APR12
PROJECT No. 113-81608

FIGURE 1

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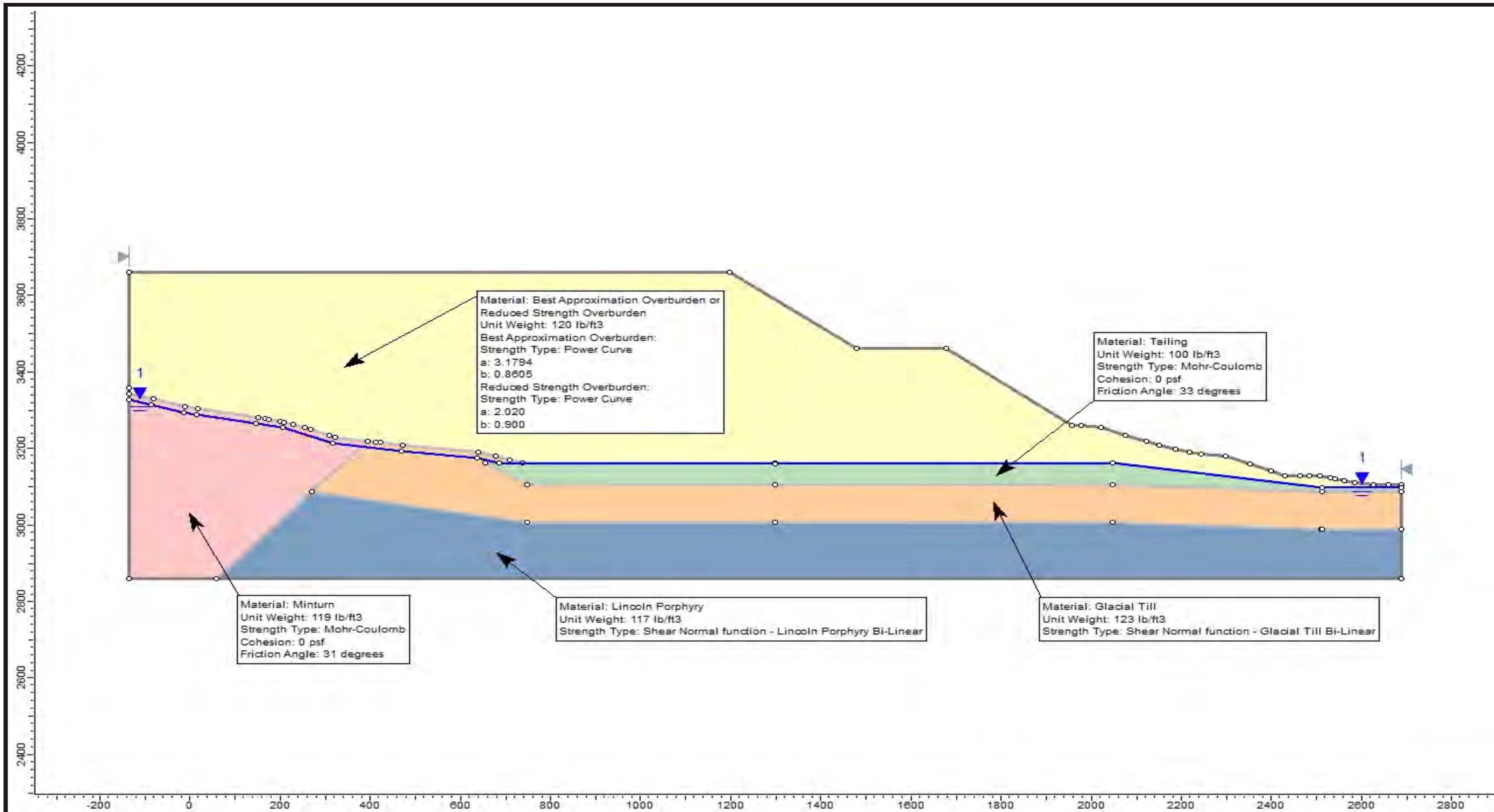

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Denver, Colorado

TITLE

Typical Cross-Section and Material Properties

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

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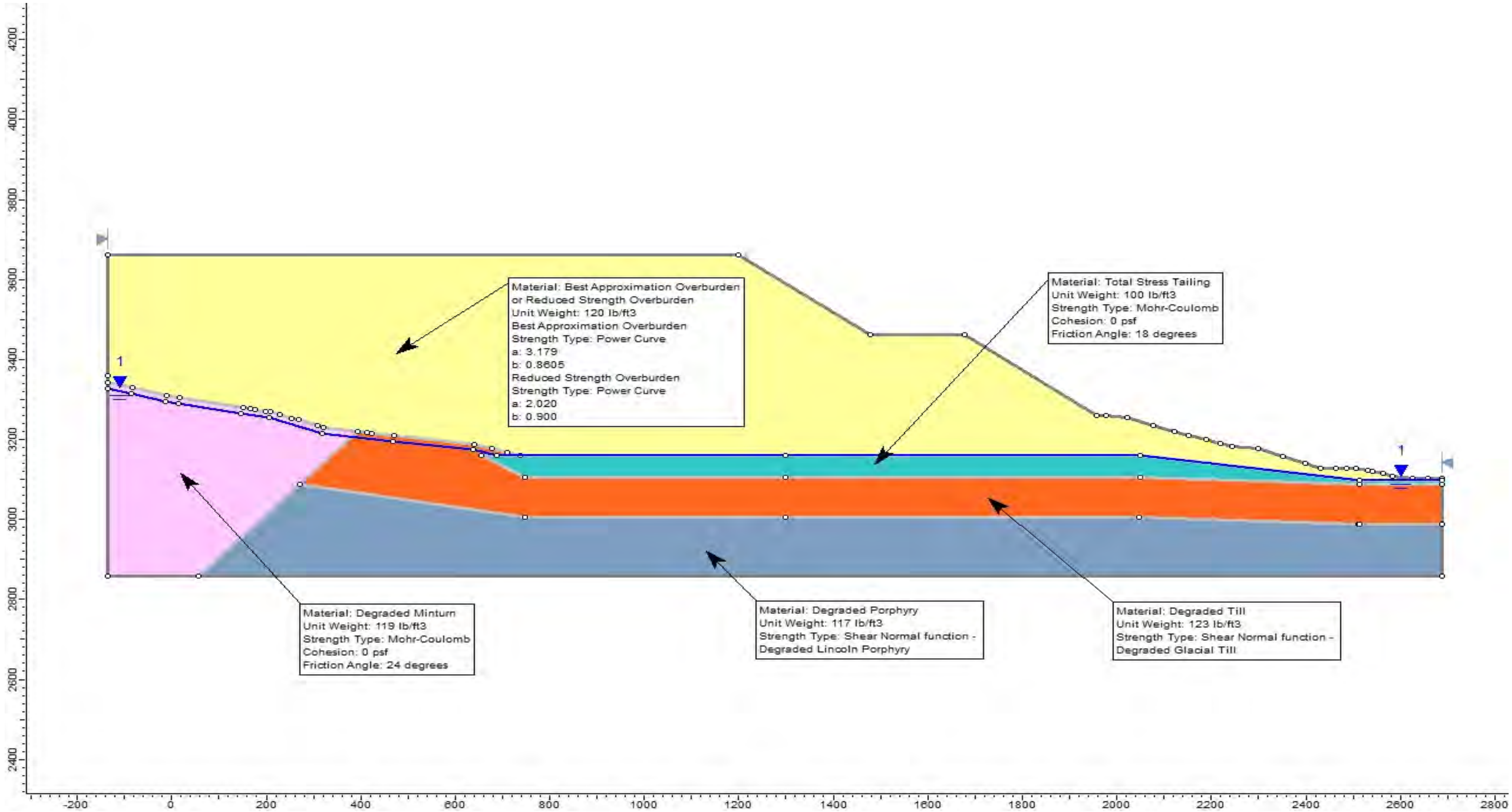

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TITLE

Typical Cross-Section and Seismic Material Properties

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

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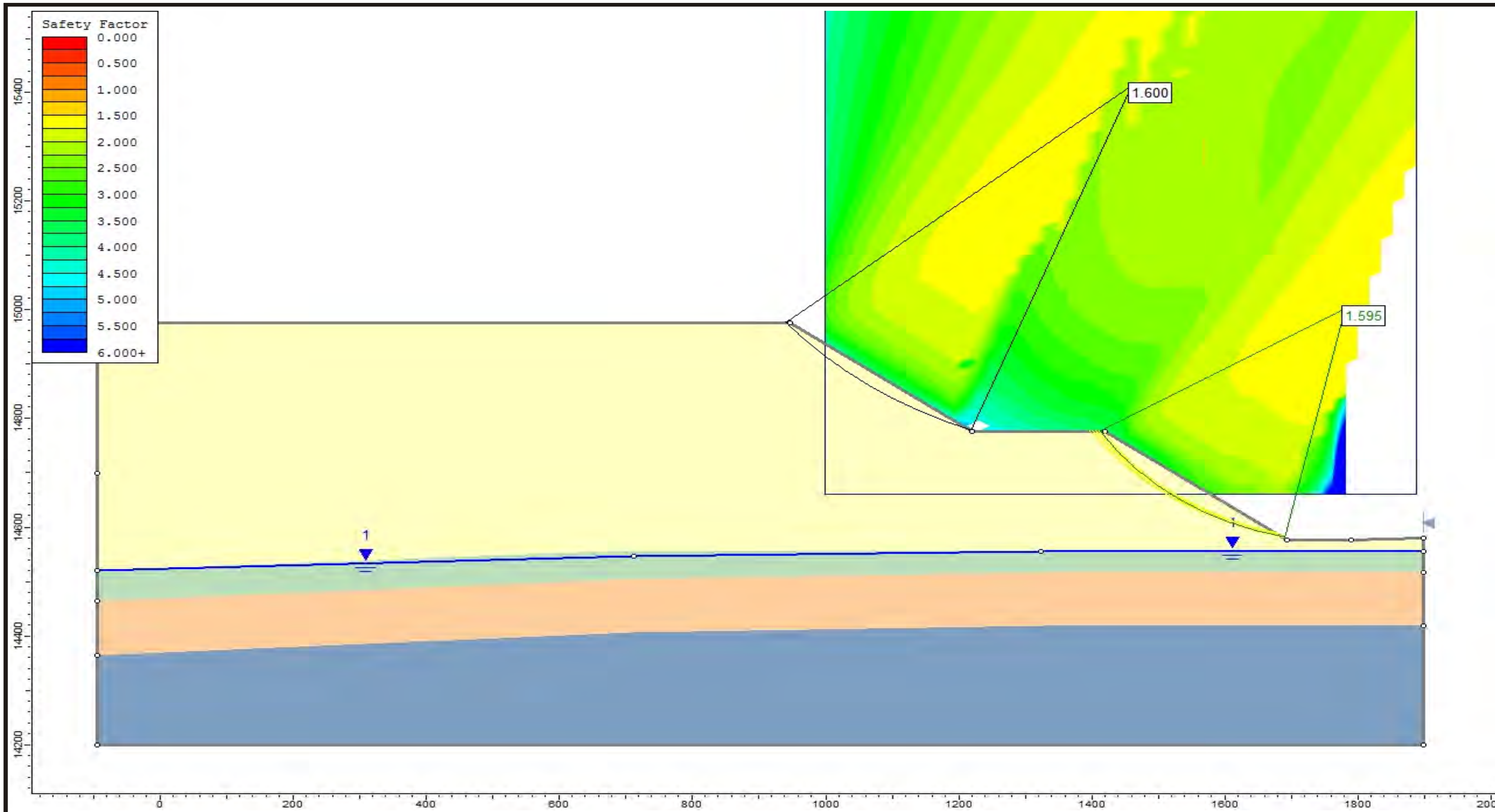

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Denver, Colorado

TITLE

Cross-Section A-A
Circular Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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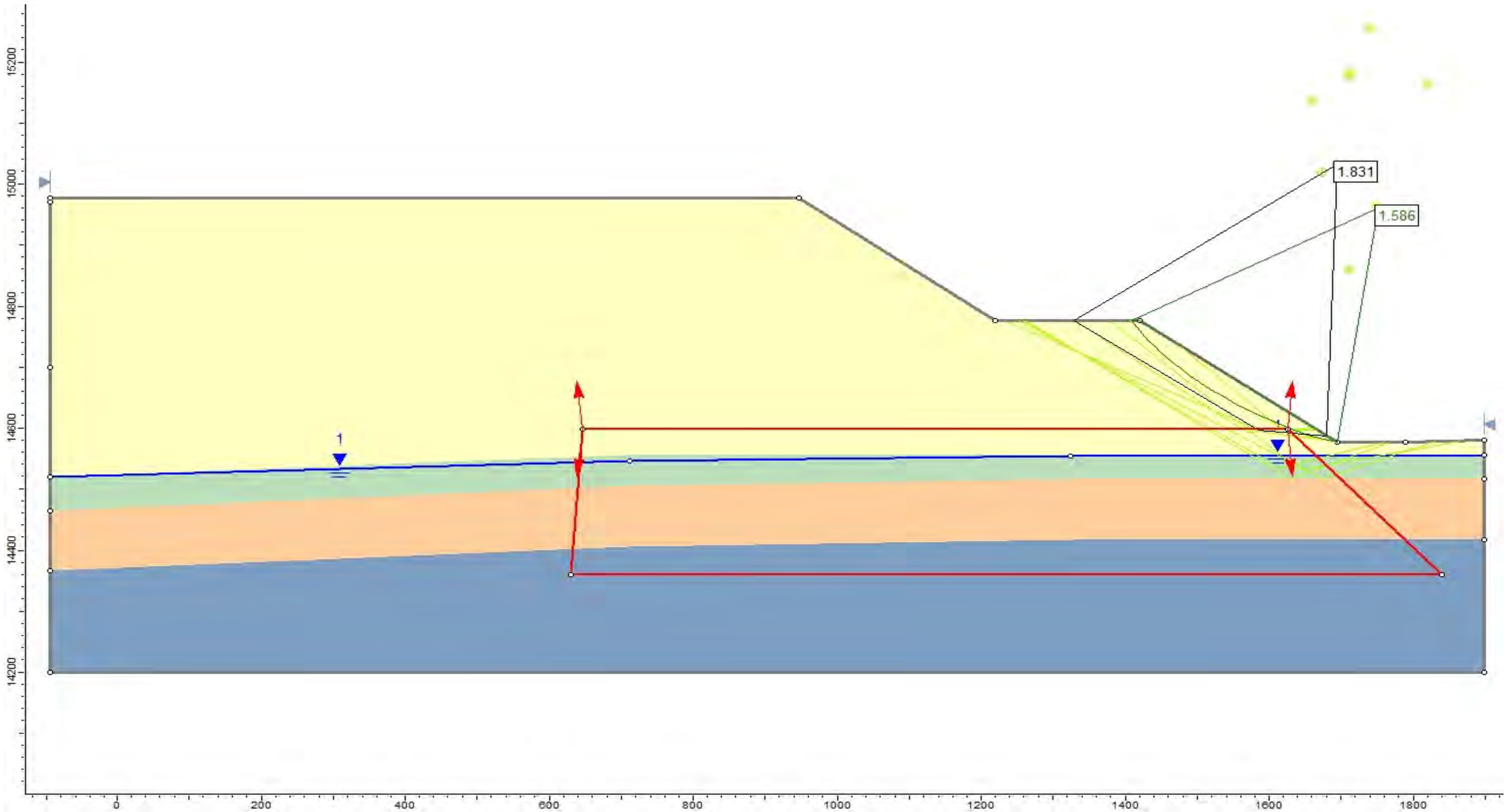

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TITLE

Cross-Section A-A
Block Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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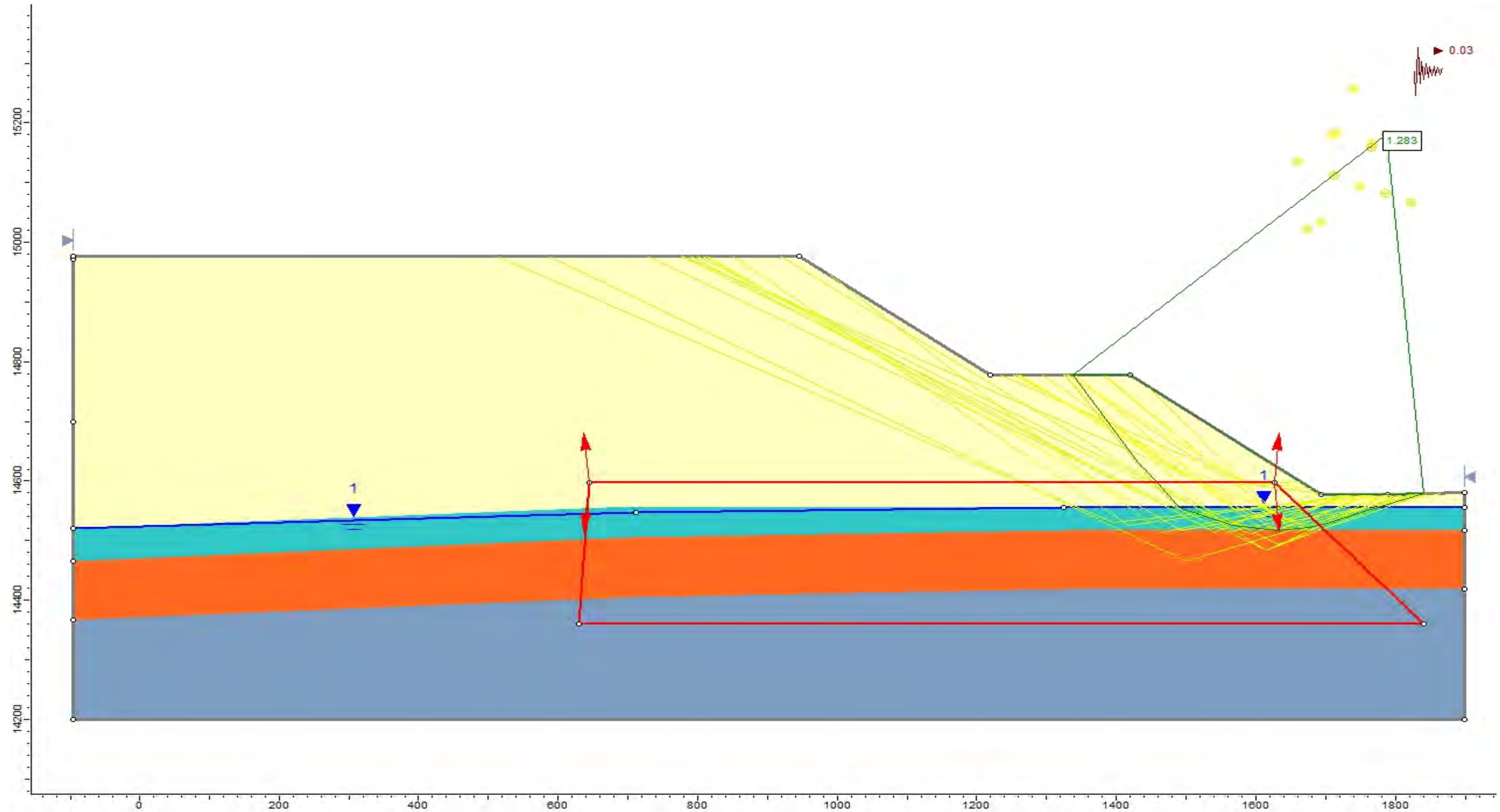

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TITLE

Cross-Section A-A Seismic Analysis (Operational Seismic Coefficient = 0.03g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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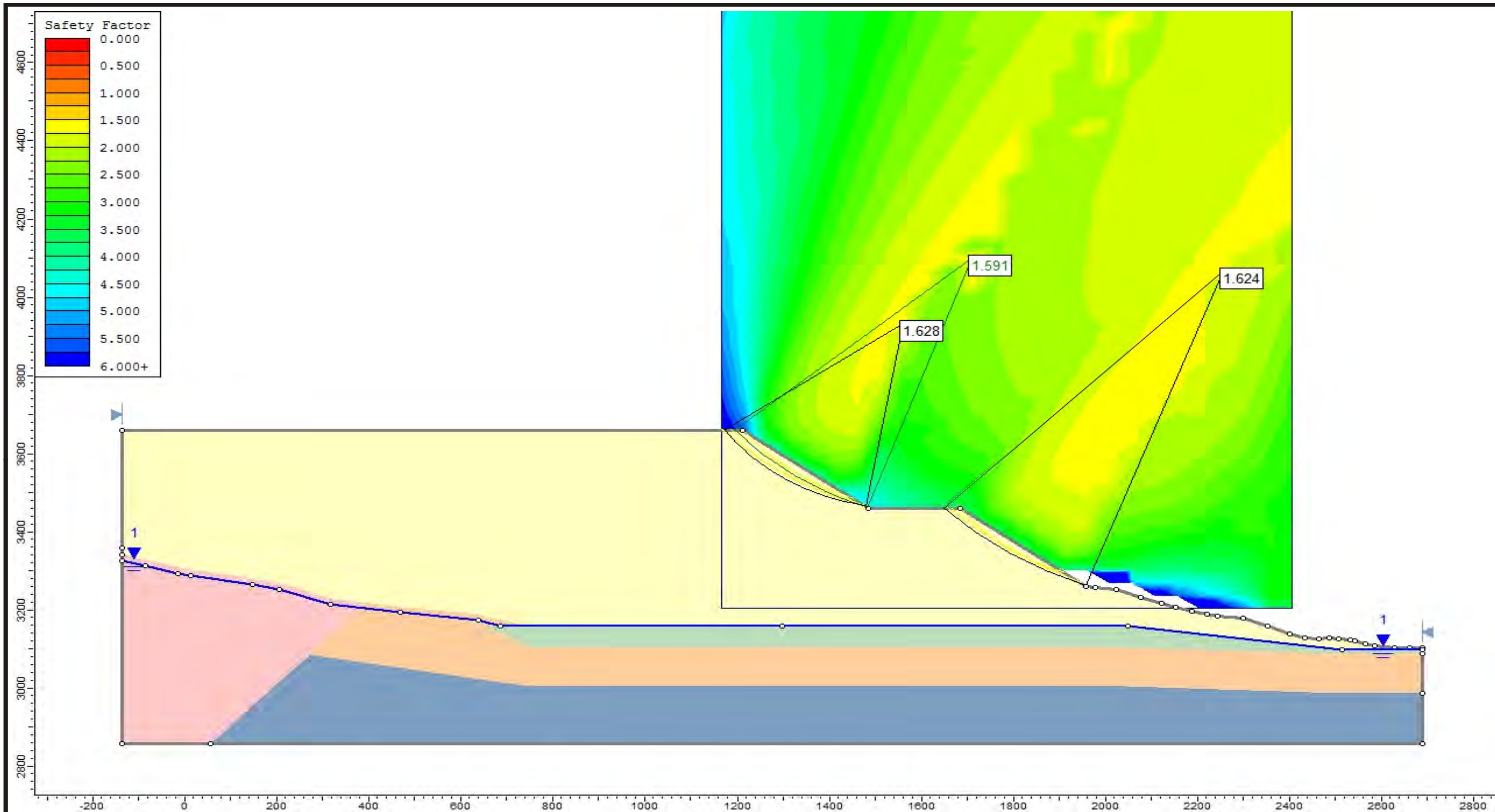

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TITLE

Cross-Section B-B
Circular Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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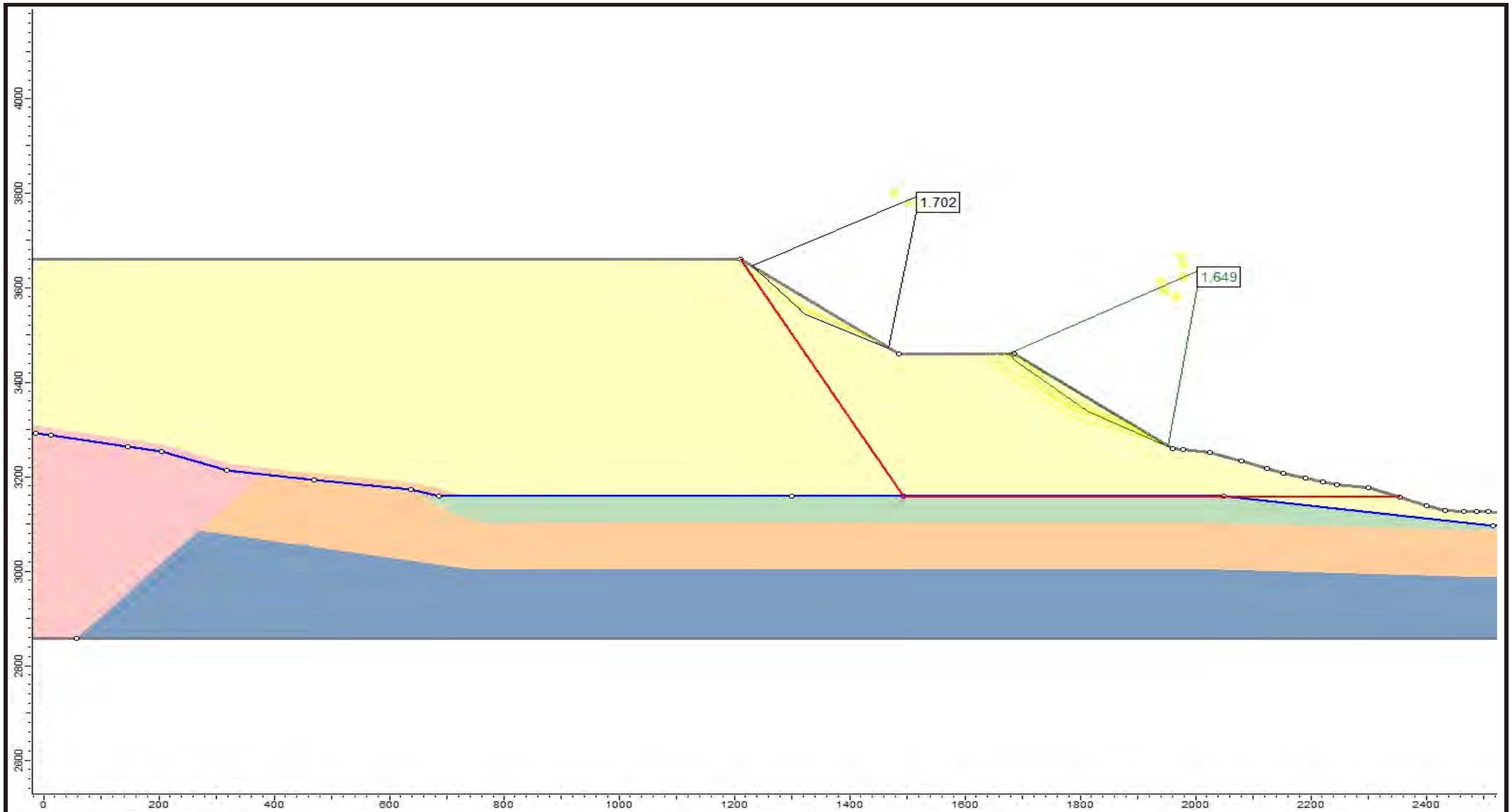

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TITLE

Cross-Section B-B
Block Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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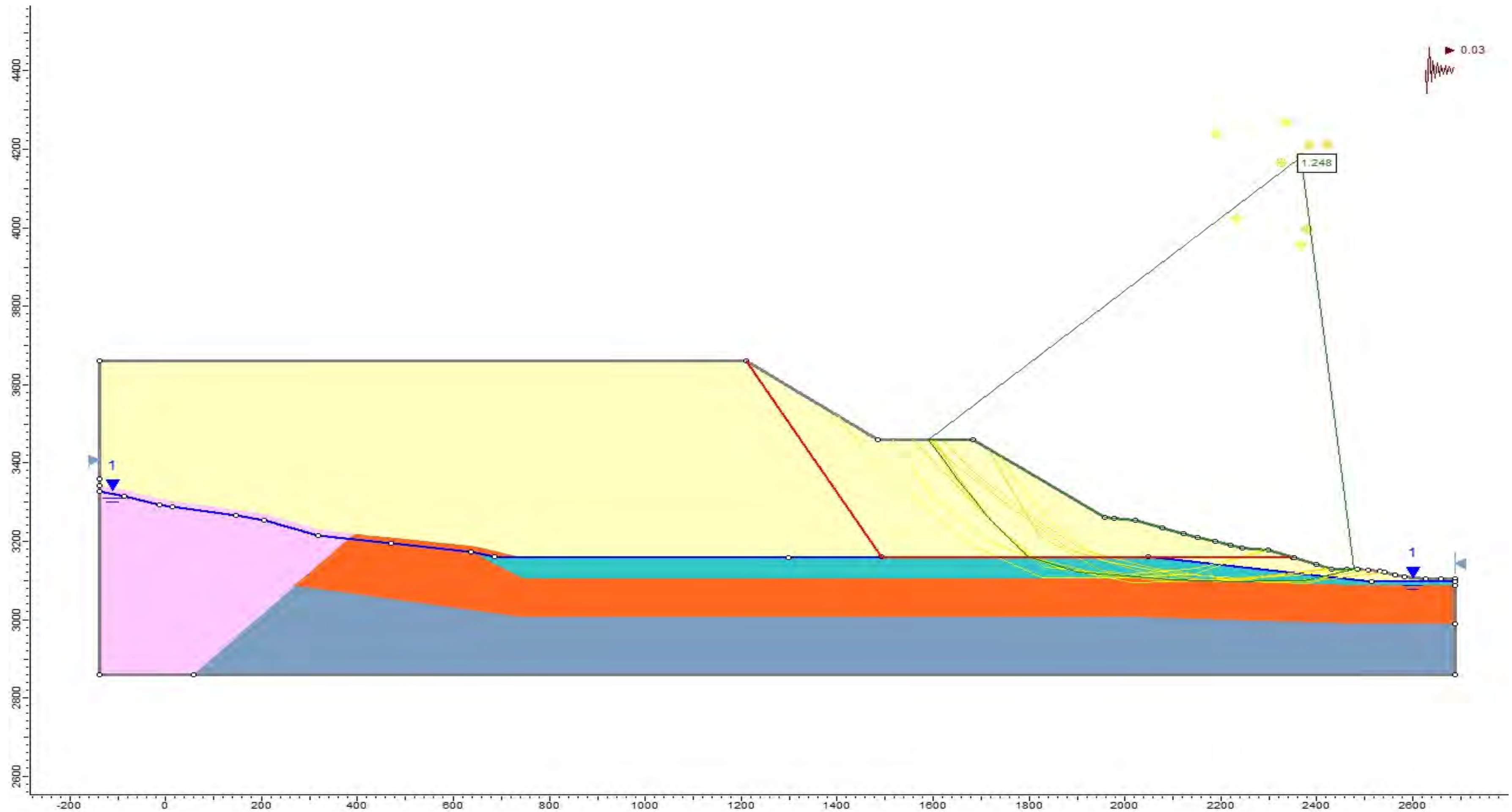

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TITLE

Cross-Section B-B Seismic Analysis
(Operational Seismic Coefficient = 0.03g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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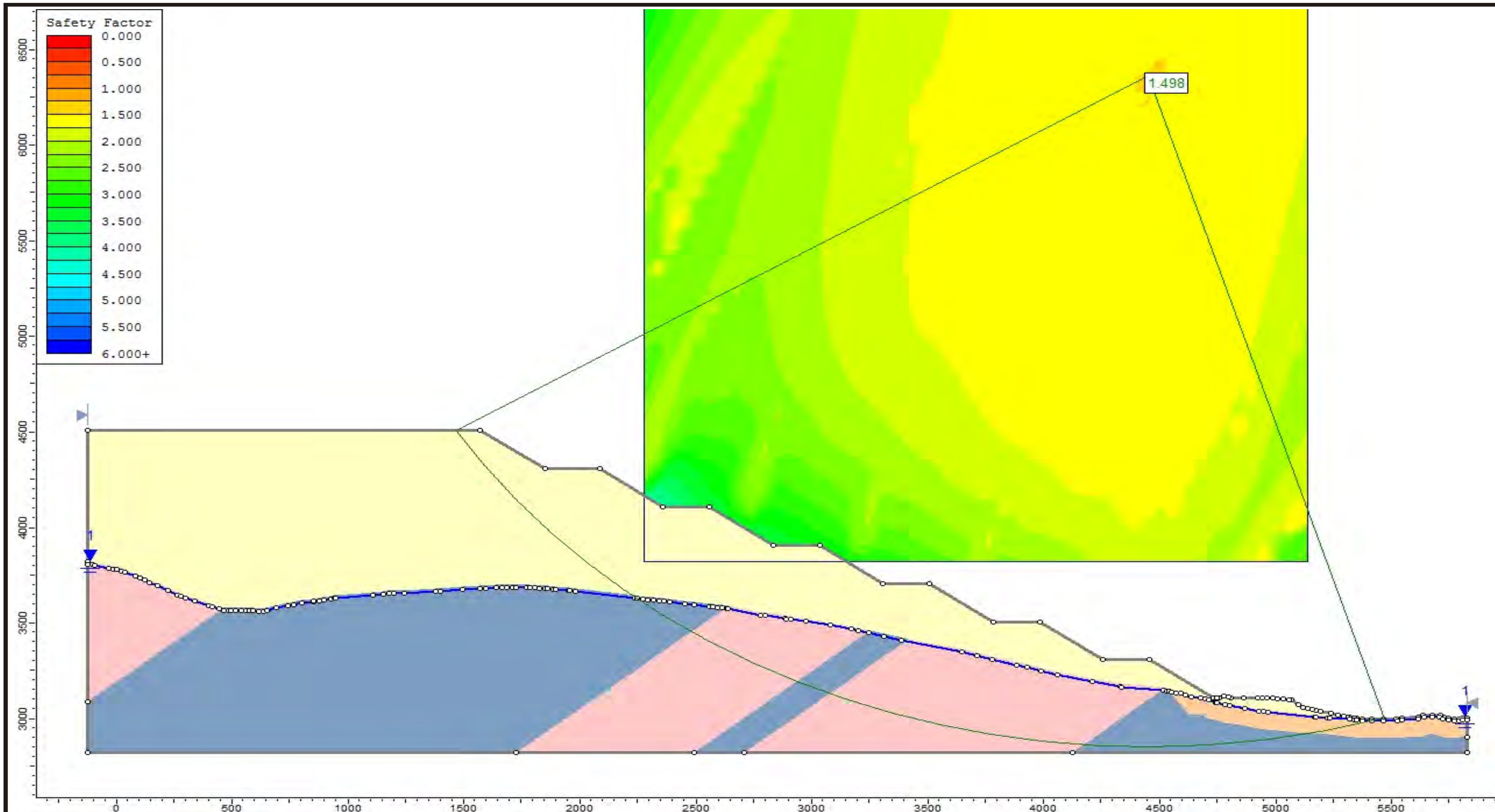

Golder Associates

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TITLE

Cross-Section C-C
Circular Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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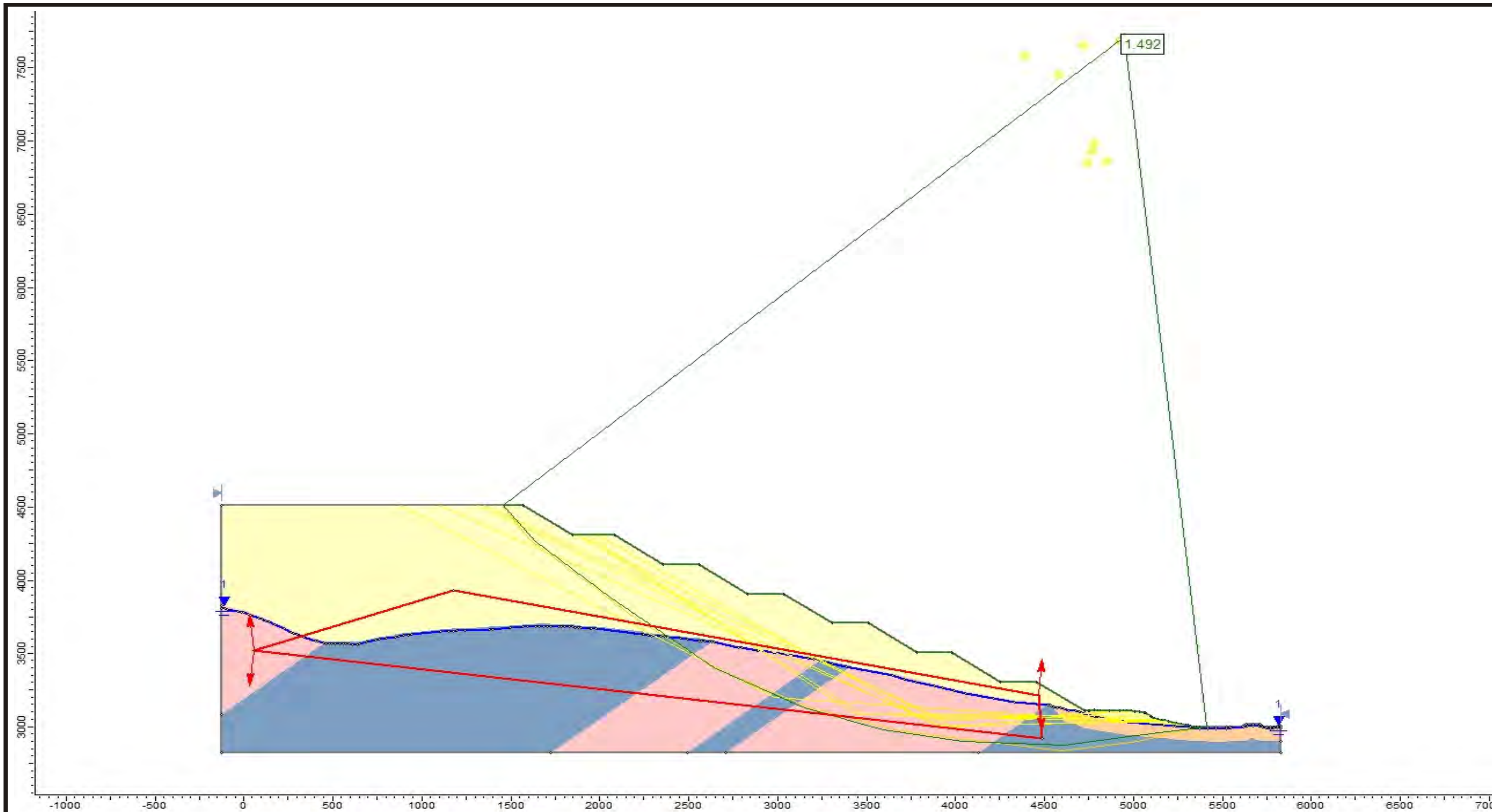

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TITLE
Cross-Section C-C
Block Analysis during Operations

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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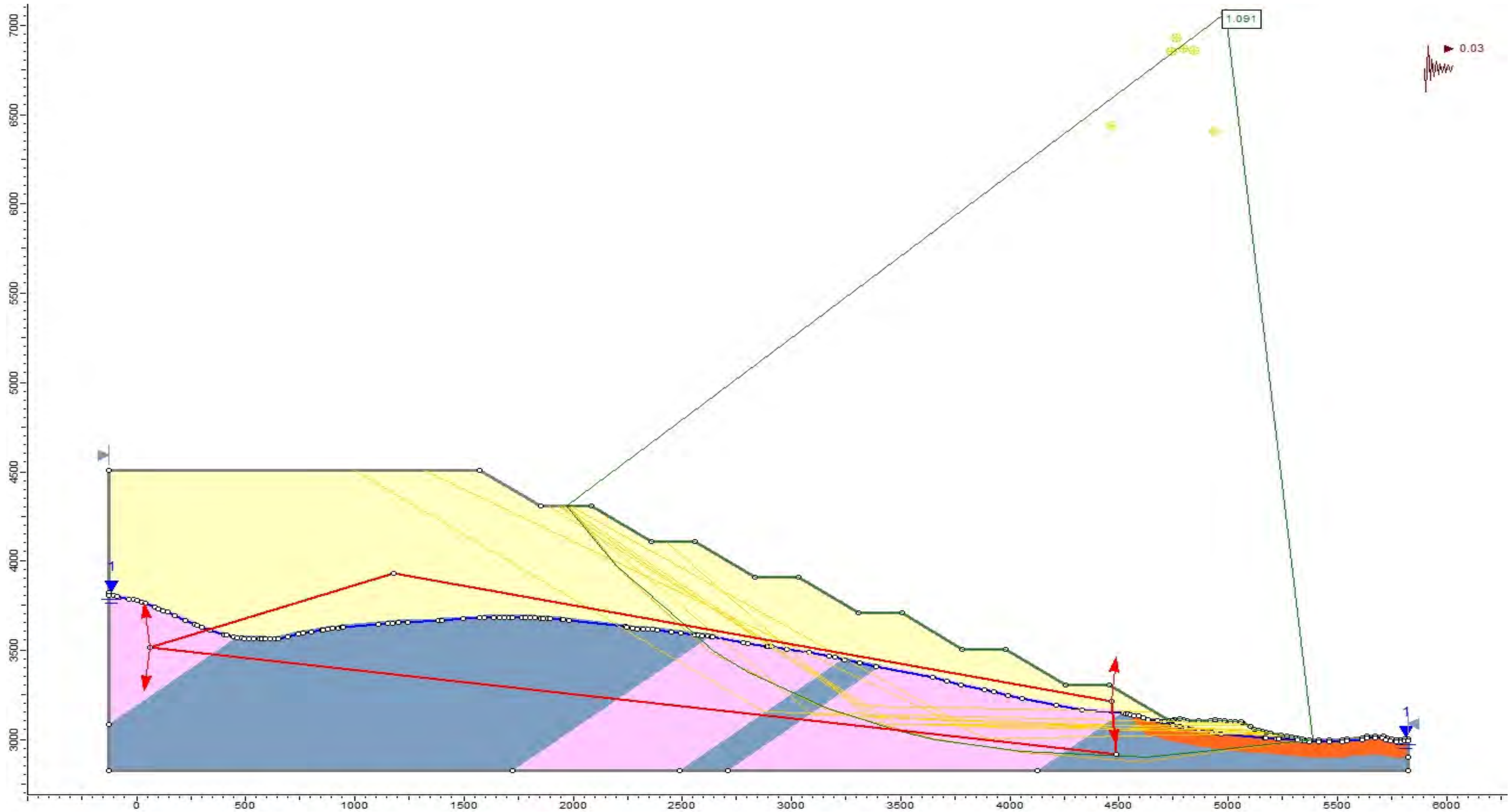

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TITLE	**Cross-Section C-C Seismic Analysis (Operational Seismic Coefficient = 0.03g)**				
CLIENT/PROJECT **Freeport McMoRan/Climax Molybdenum OSF Stability Analysis**					
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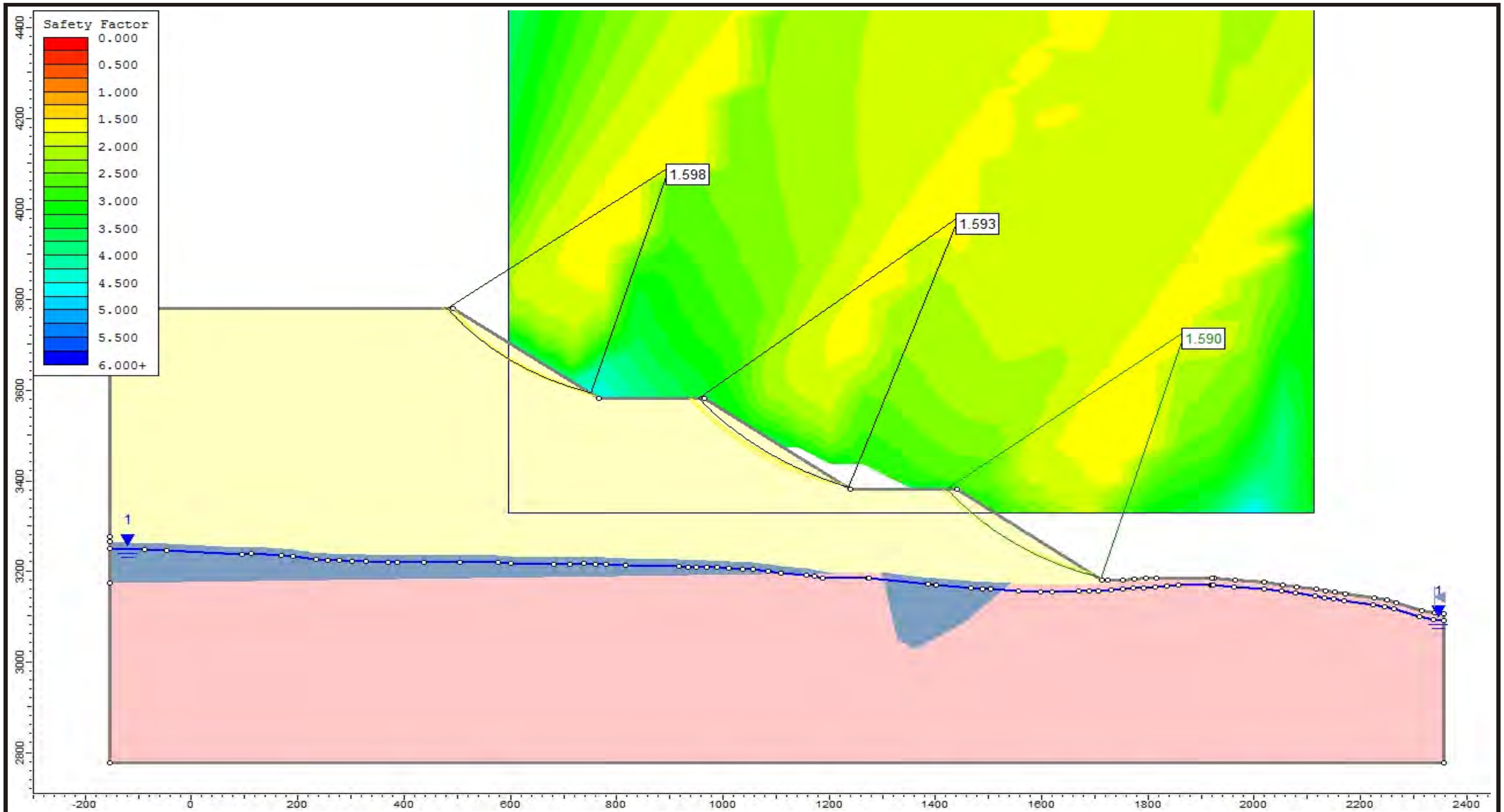

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TITLE
Cross-Section D-D
Circular Analysis during Operations

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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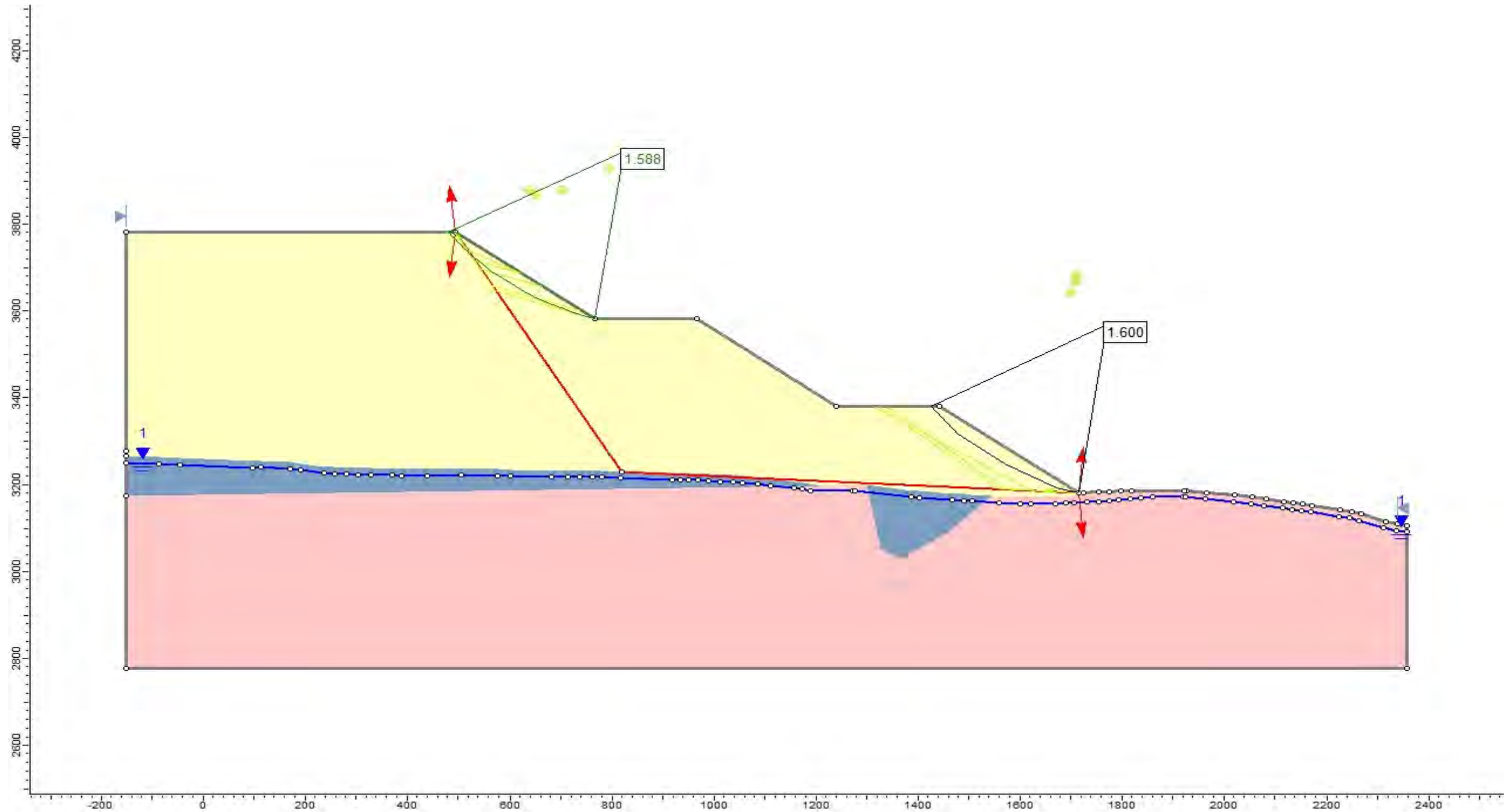

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TITLE

Cross-Section D-D
Block Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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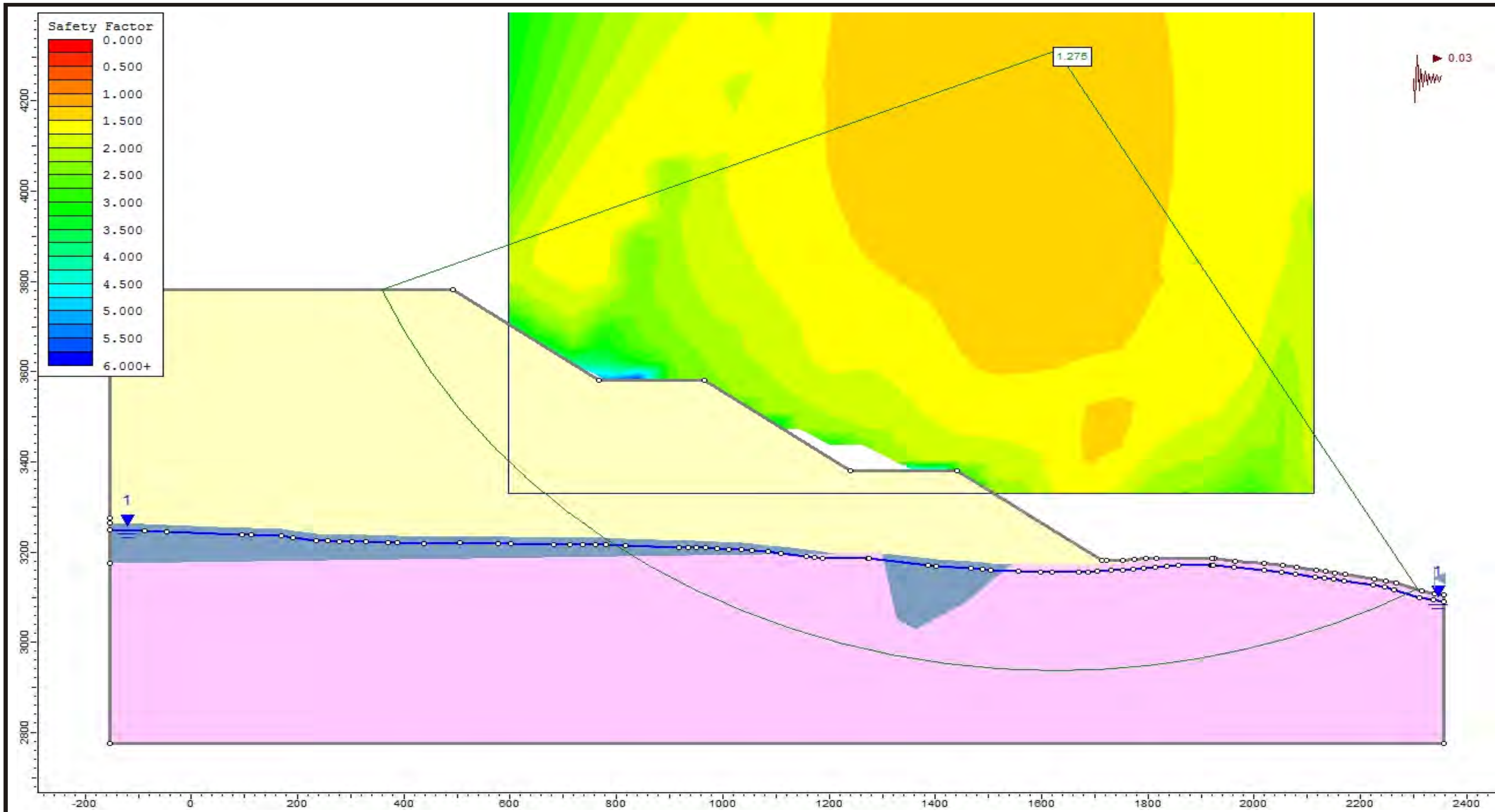

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TITLE

Cross-Section D-D Seismic Analysis (Operational Seismic Coefficient = 0.03g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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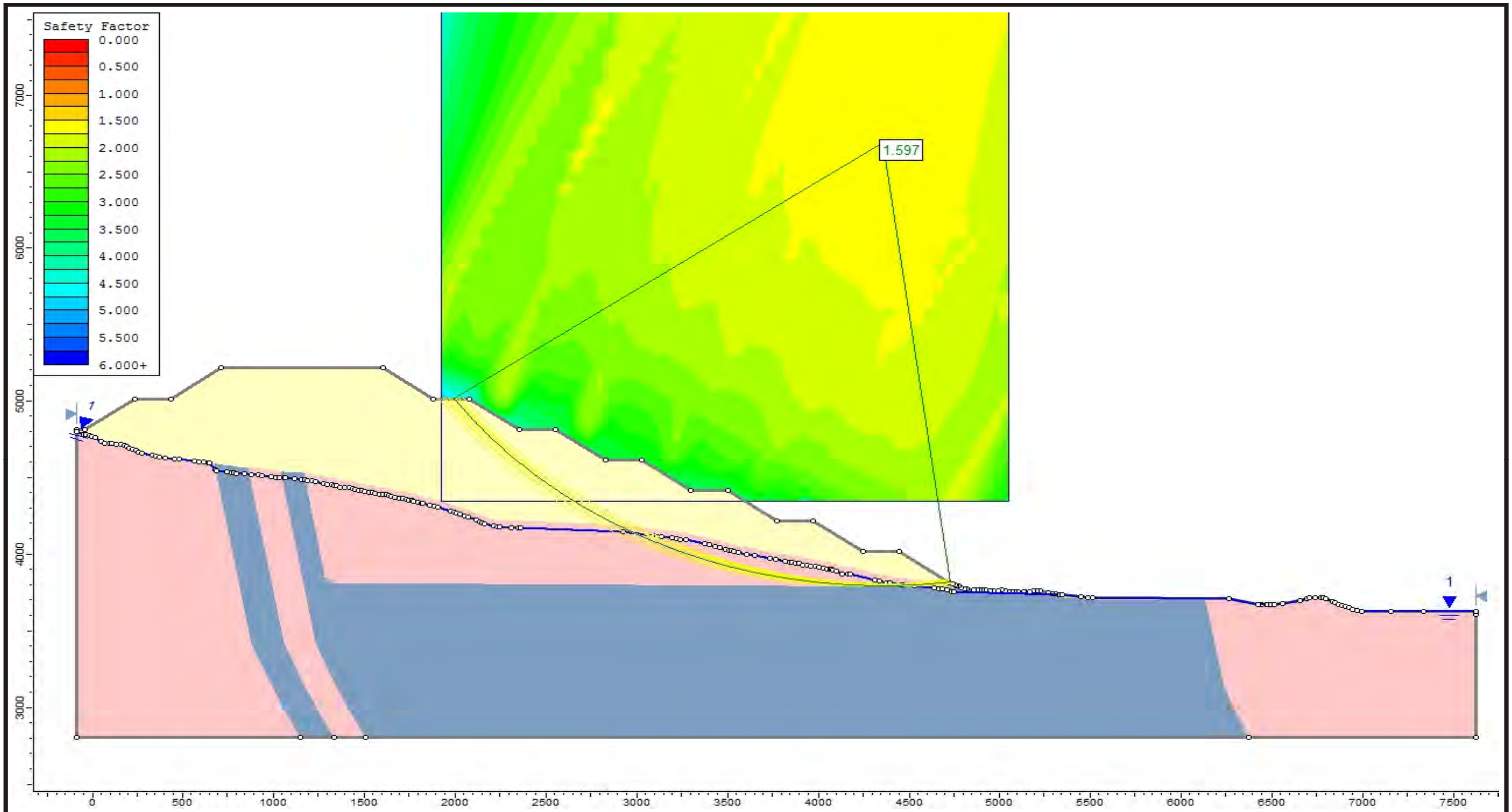

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TITLE

Cross-Section E-E
Circular Analysis during Operations

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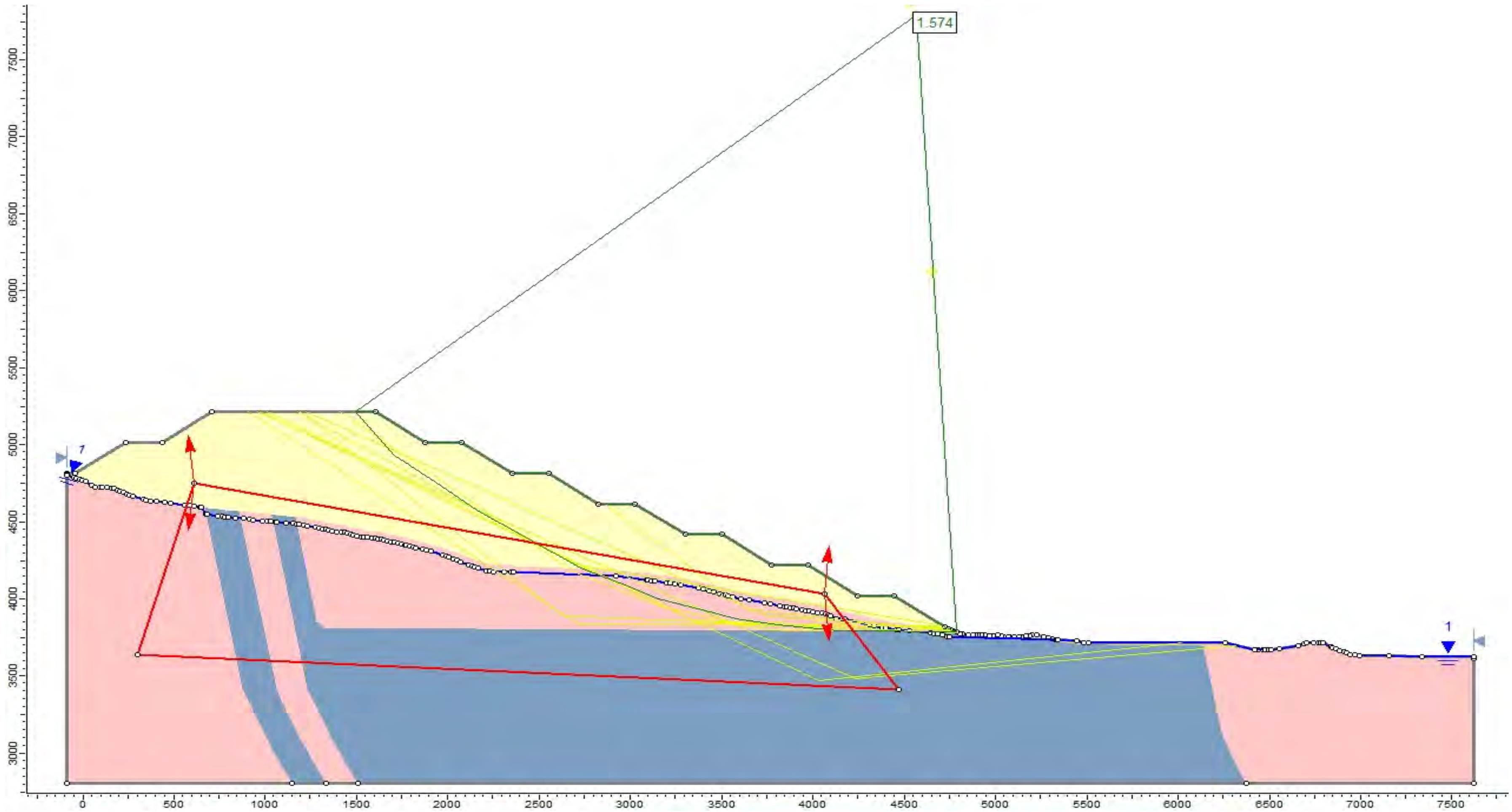

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TITLE

Cross-Section E-E
Block Analysis during Operations

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

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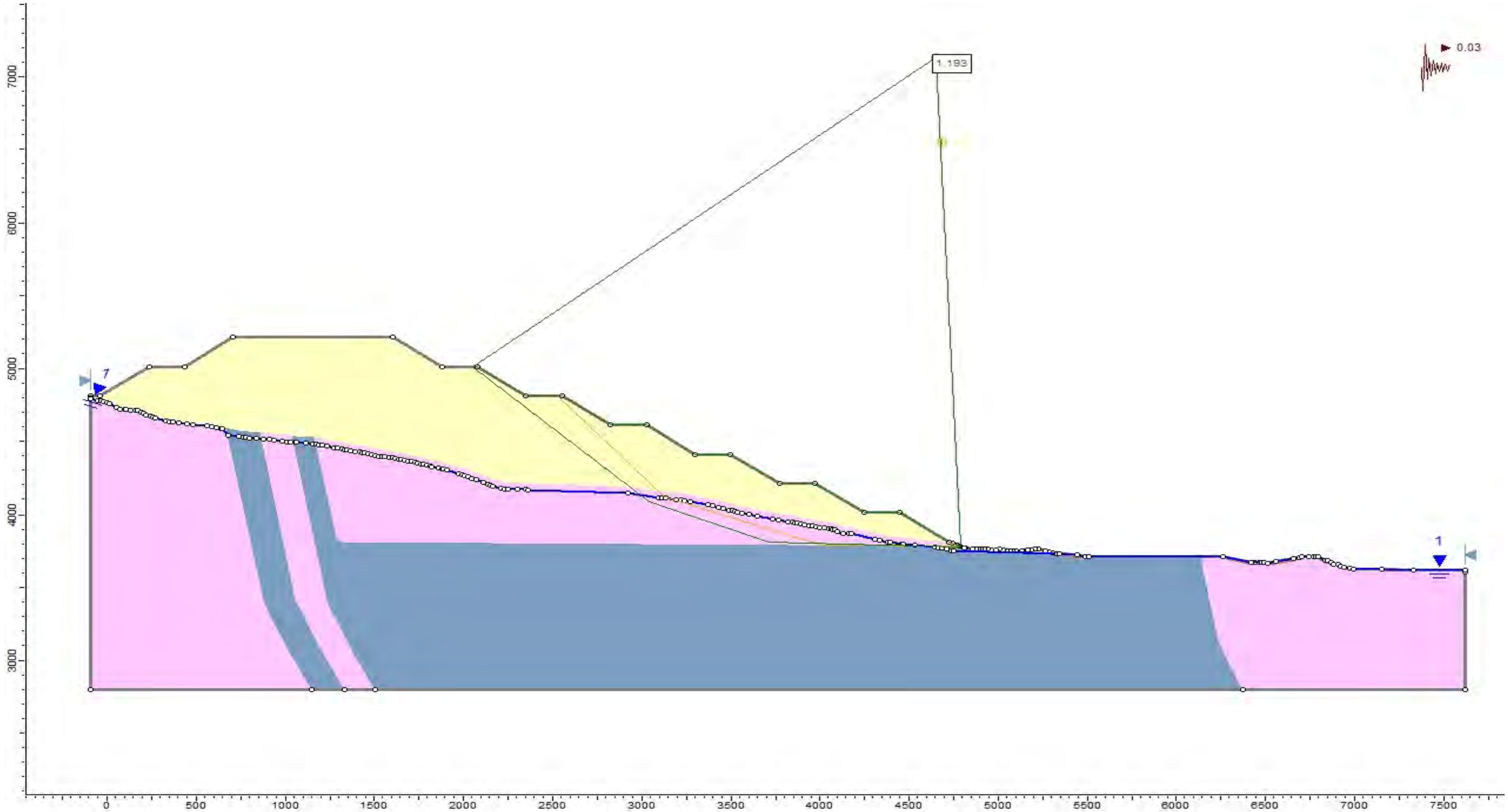

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TITLE

Cross-Section E-E Seismic Analysis (Operational Seismic Coefficient = 0.03g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

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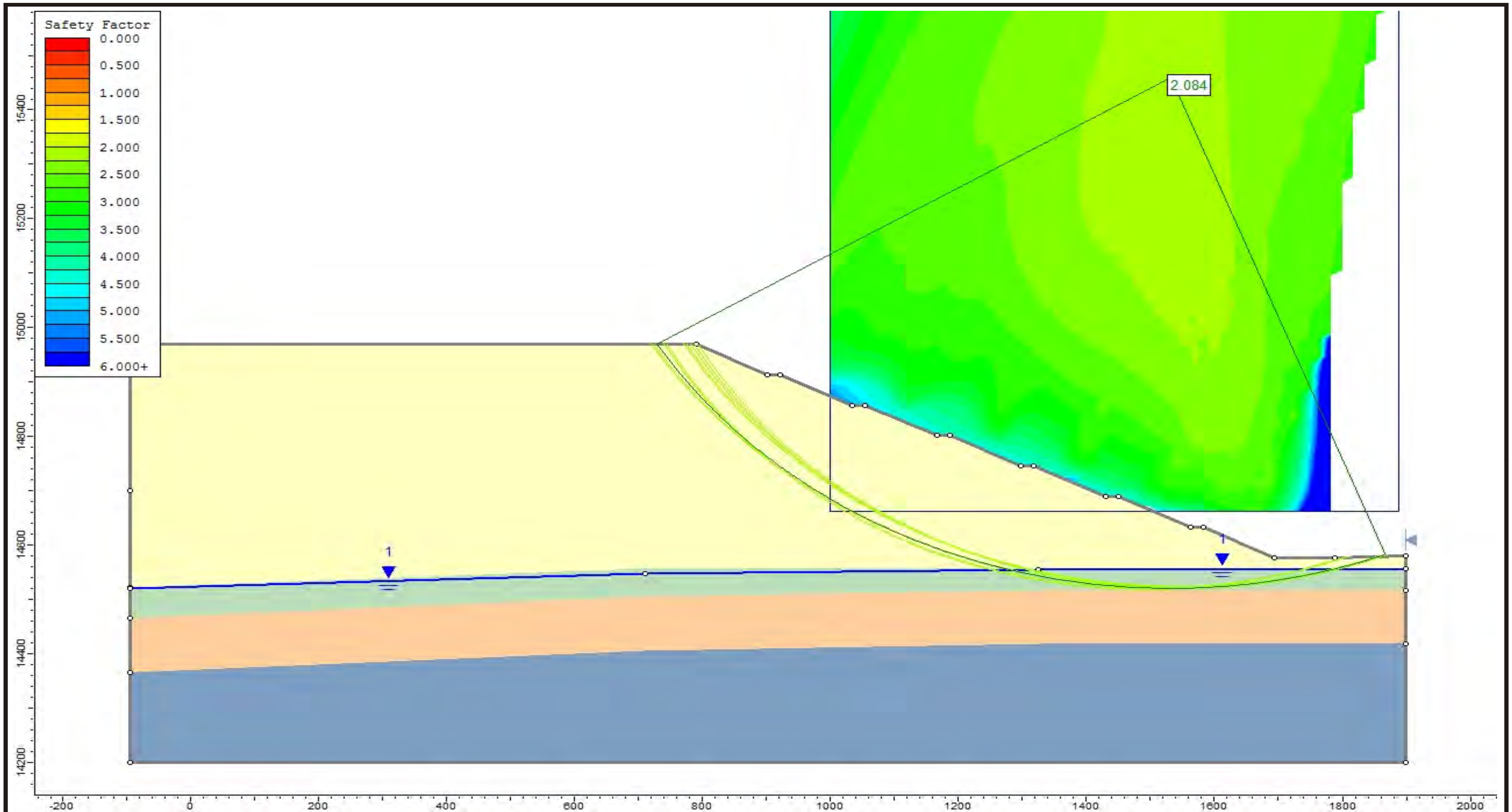

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TITLE

Cross-Section A-A
Circular Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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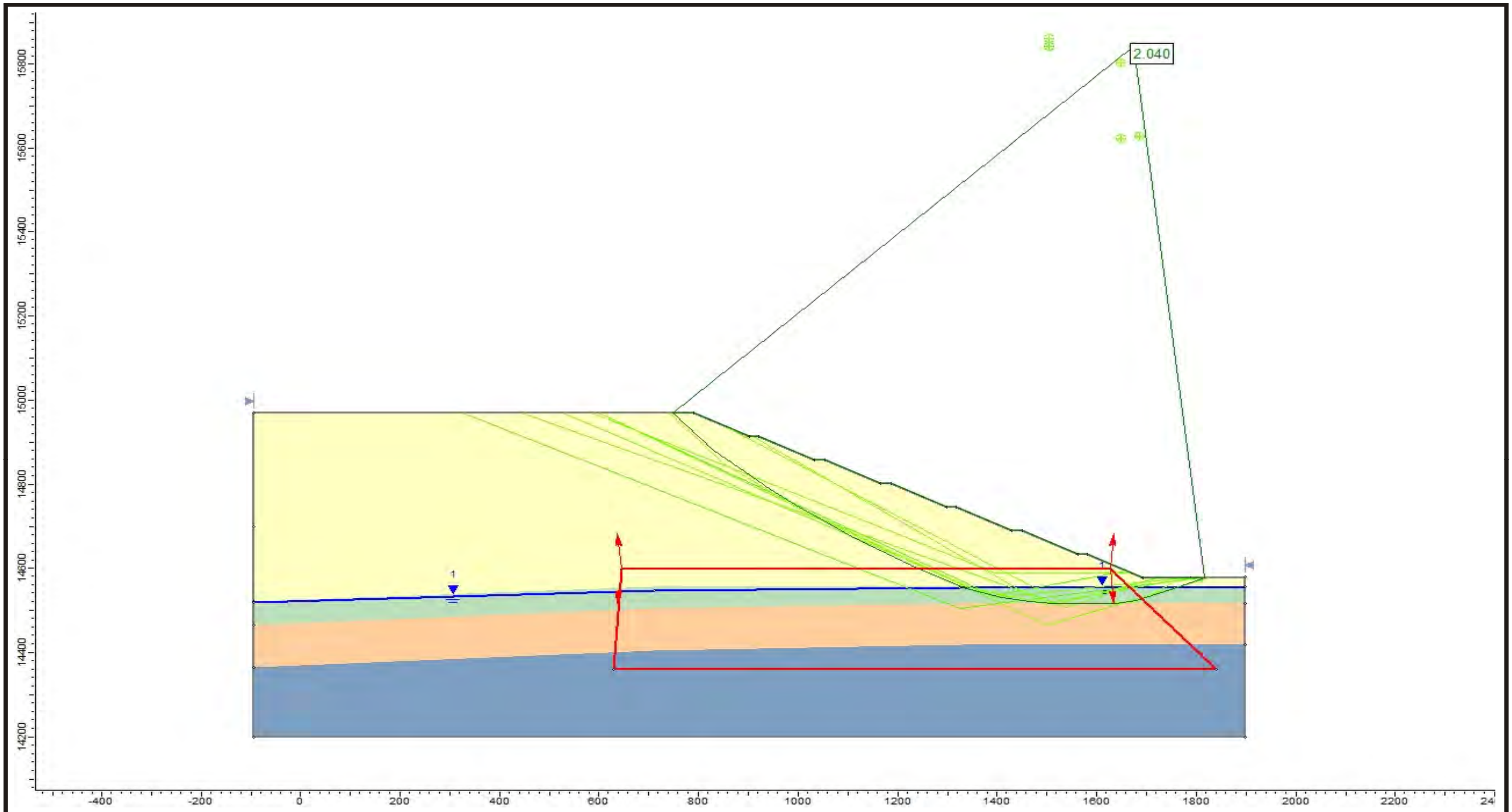

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TITLE

Cross-Section A-A
Block Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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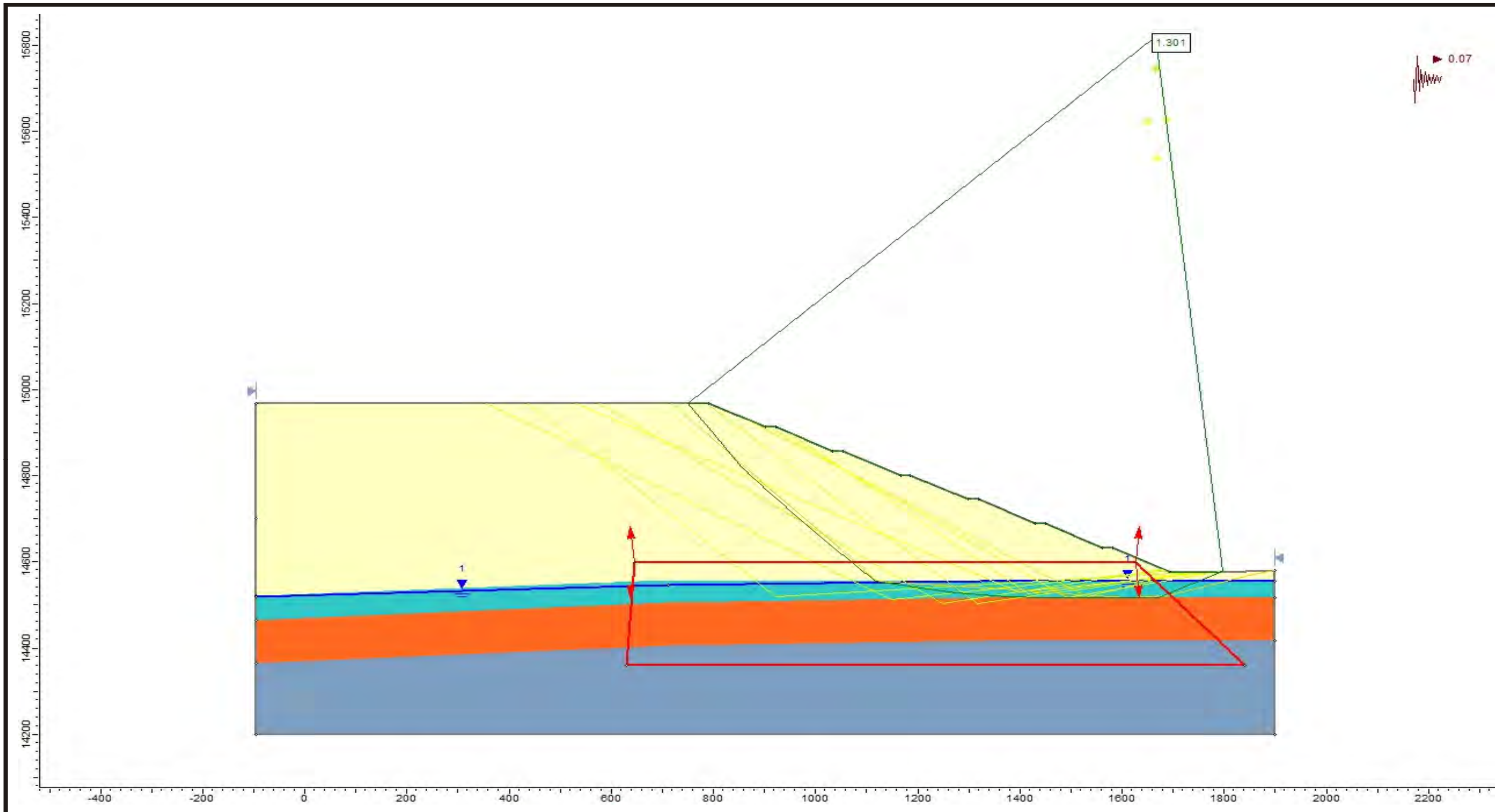

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TITLE

Cross-Section A-A Seismic Analysis (Closure Seismic Coefficient = 0.07g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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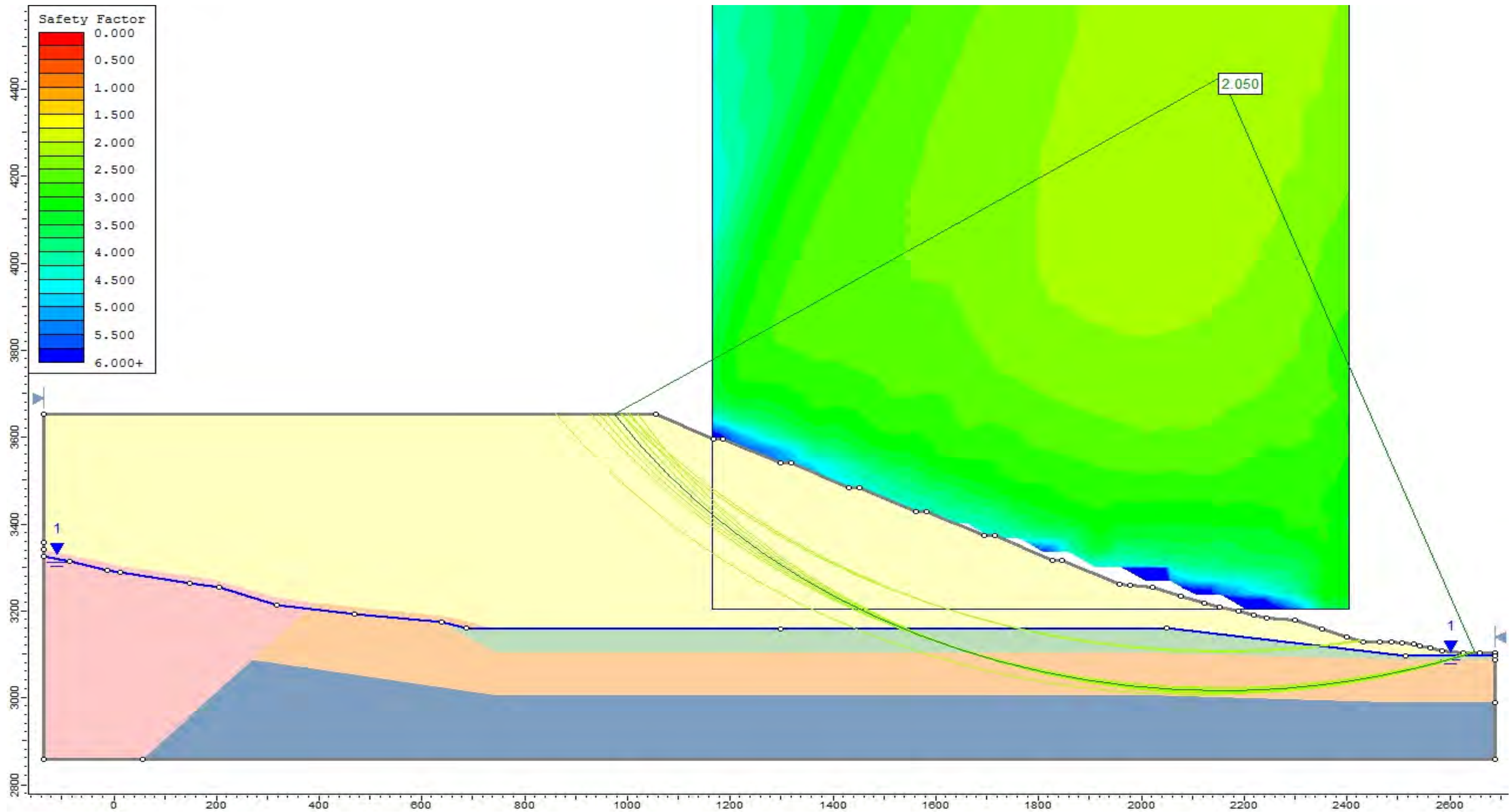

Golder Associates

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TITLE

Cross-Section B-B
Circular Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
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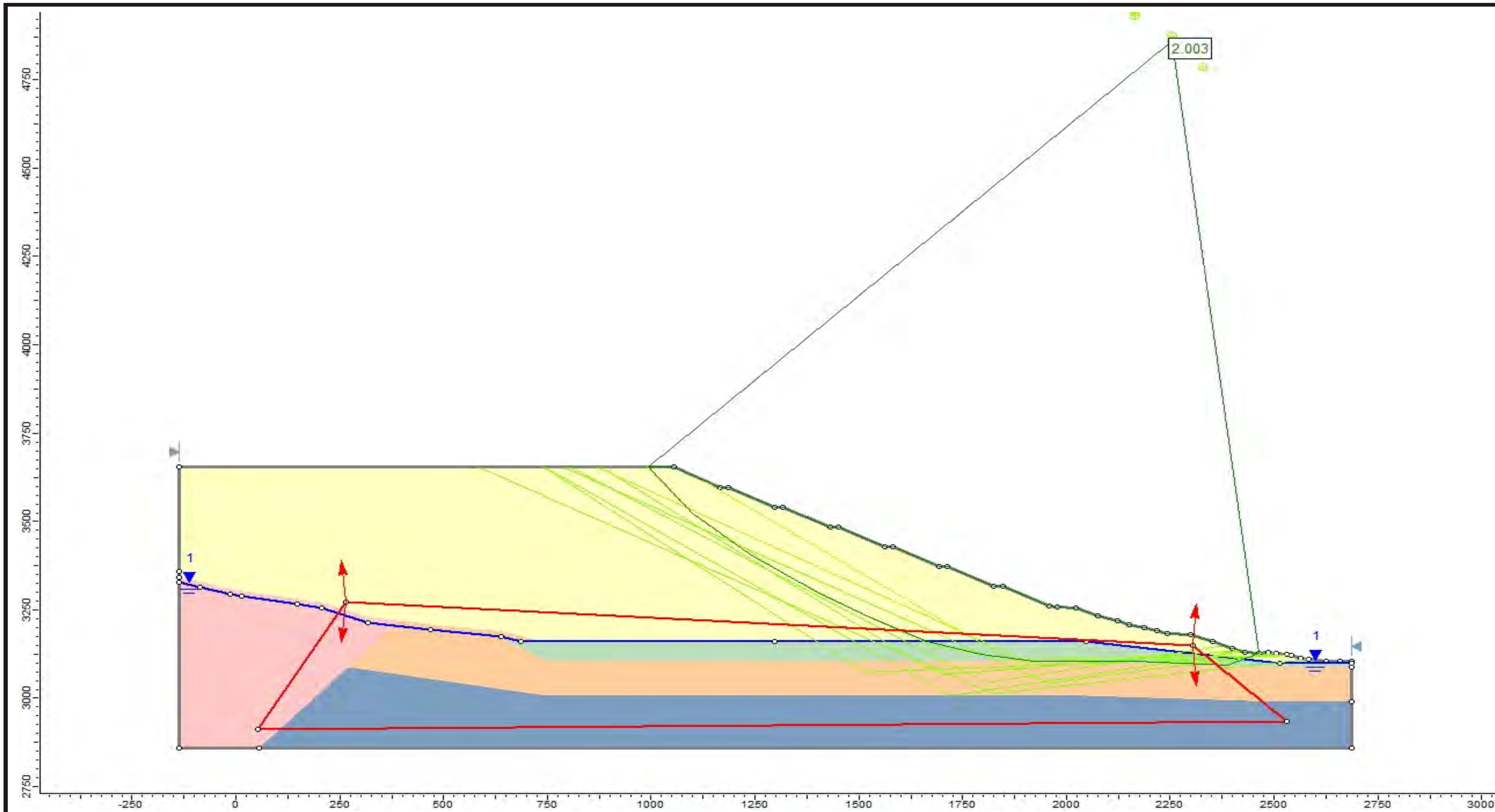
REVIEWED	BRB	FILE NO.		FIGURE NO.	22



Golder Associates

Denver, Colorado

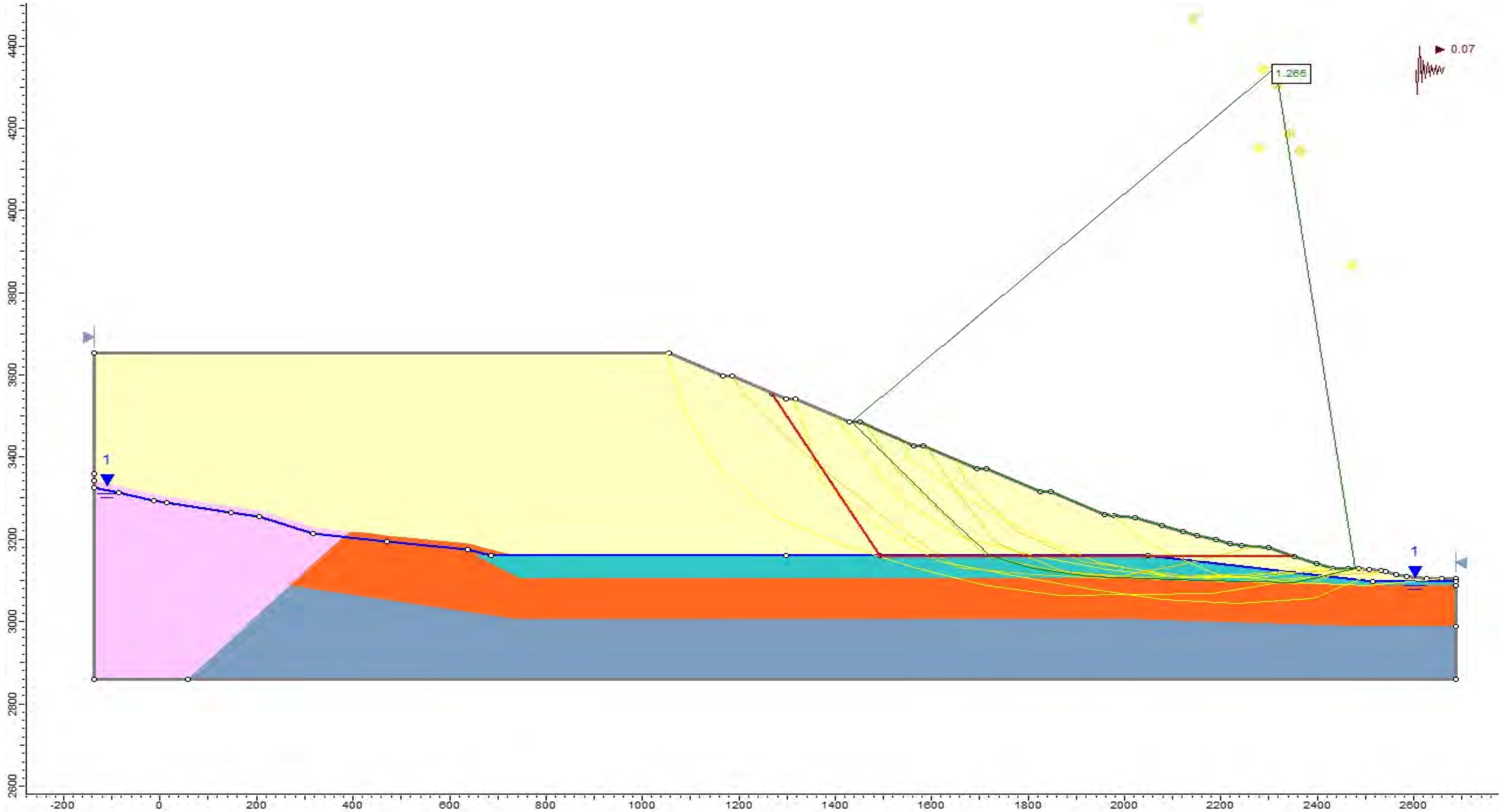
TITLE

Cross-Section B-B
Block Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

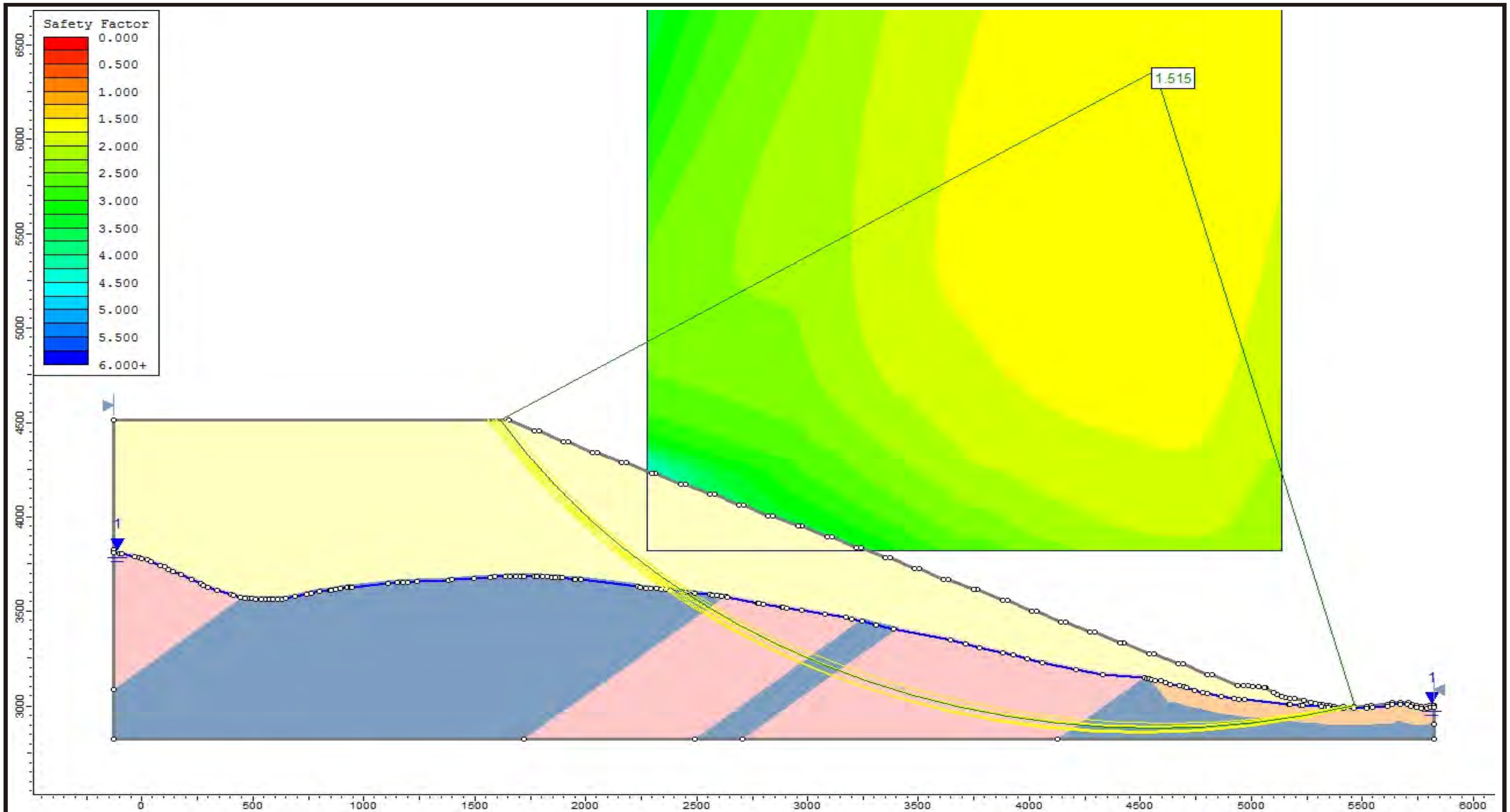
DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	23



Golder Associates

Denver, Colorado

TITLE	Cross-Section B-B Seismic Analysis (Closure Seismic Coefficient = 0.07g)				
CLIENT/PROJECT: Freeport McMoRan/Climax Molybdenum OSF Stability Analysis					
DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	24



Golder Associates

Denver, Colorado

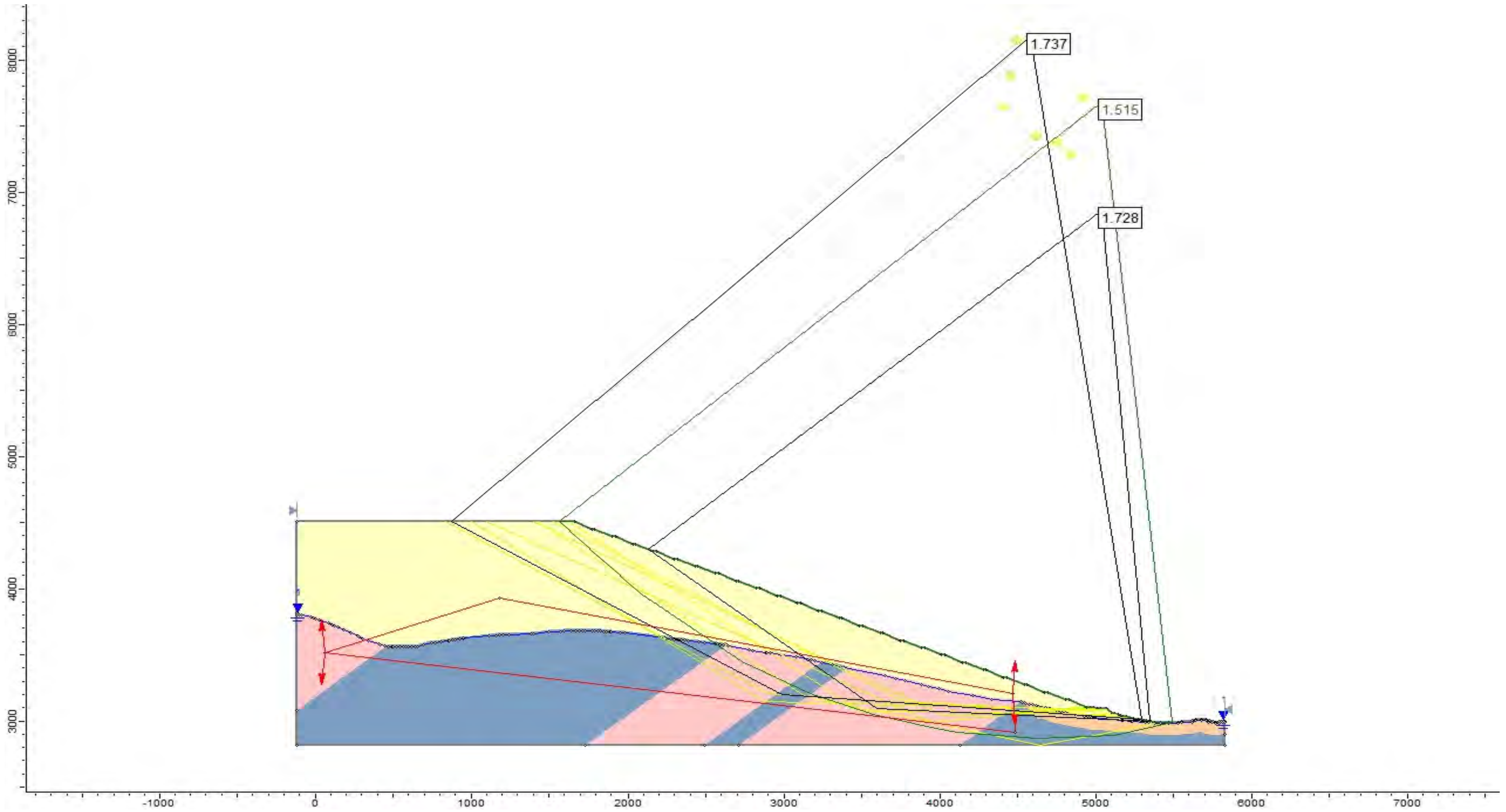
TITLE

Cross-Section C-C
Circular Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	25



Golder Associates

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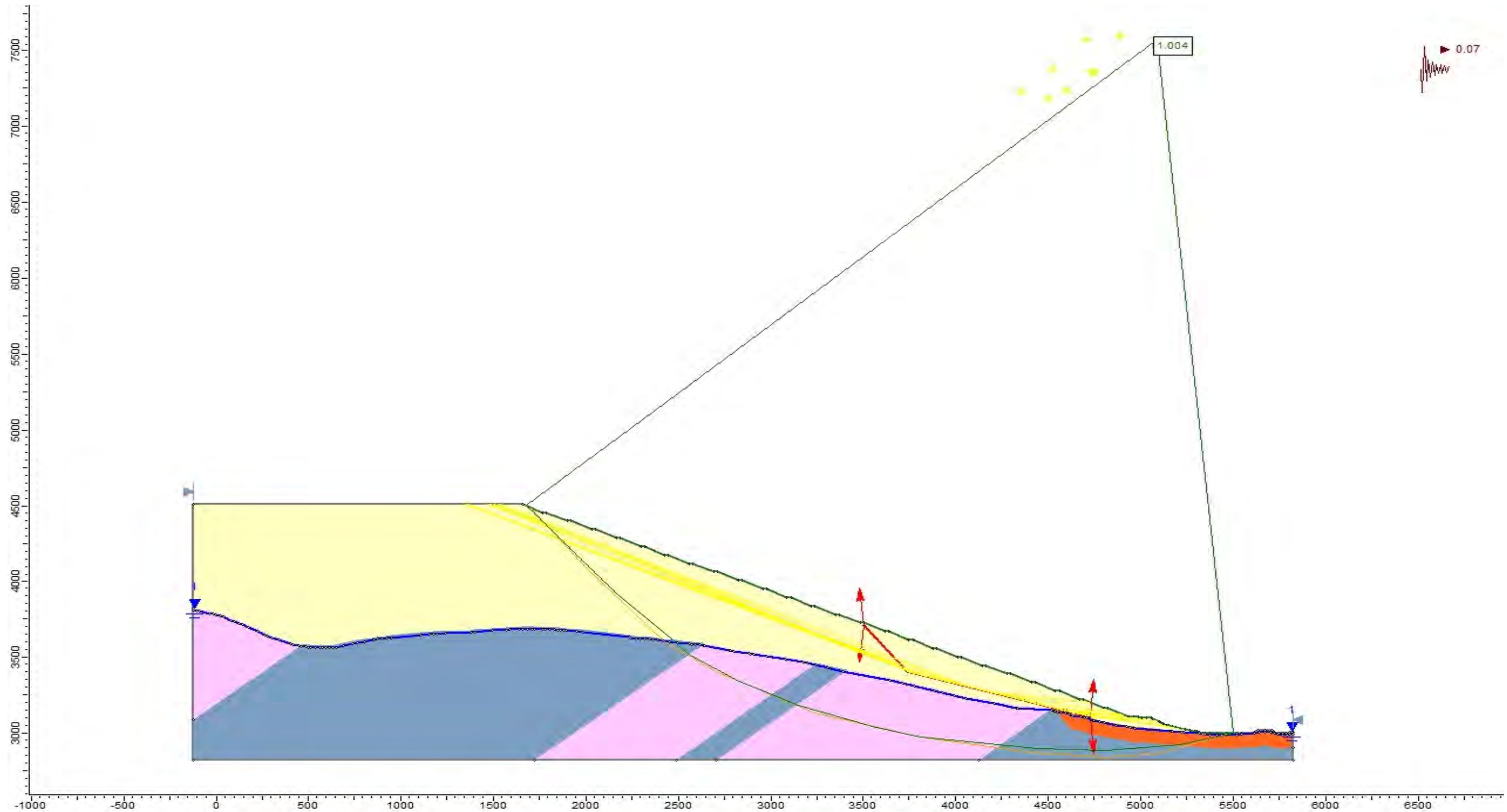
TITLE

Cross-Section C-C
Block Analysis for Closure

CLIENT/PROJECT	DRAWN DAR	DATE Apr-12	JOB NO. 113-81608		
Freeport McMoRan/Climax Molybdenum	CHECKED DLG	SCALE NTS	DWG. NO. N/A		
OSF Stability Analysis	REVIEWED BRB	FILE NO.	FIGURE NO. 26		



Golder Associates

Denver, Colorado

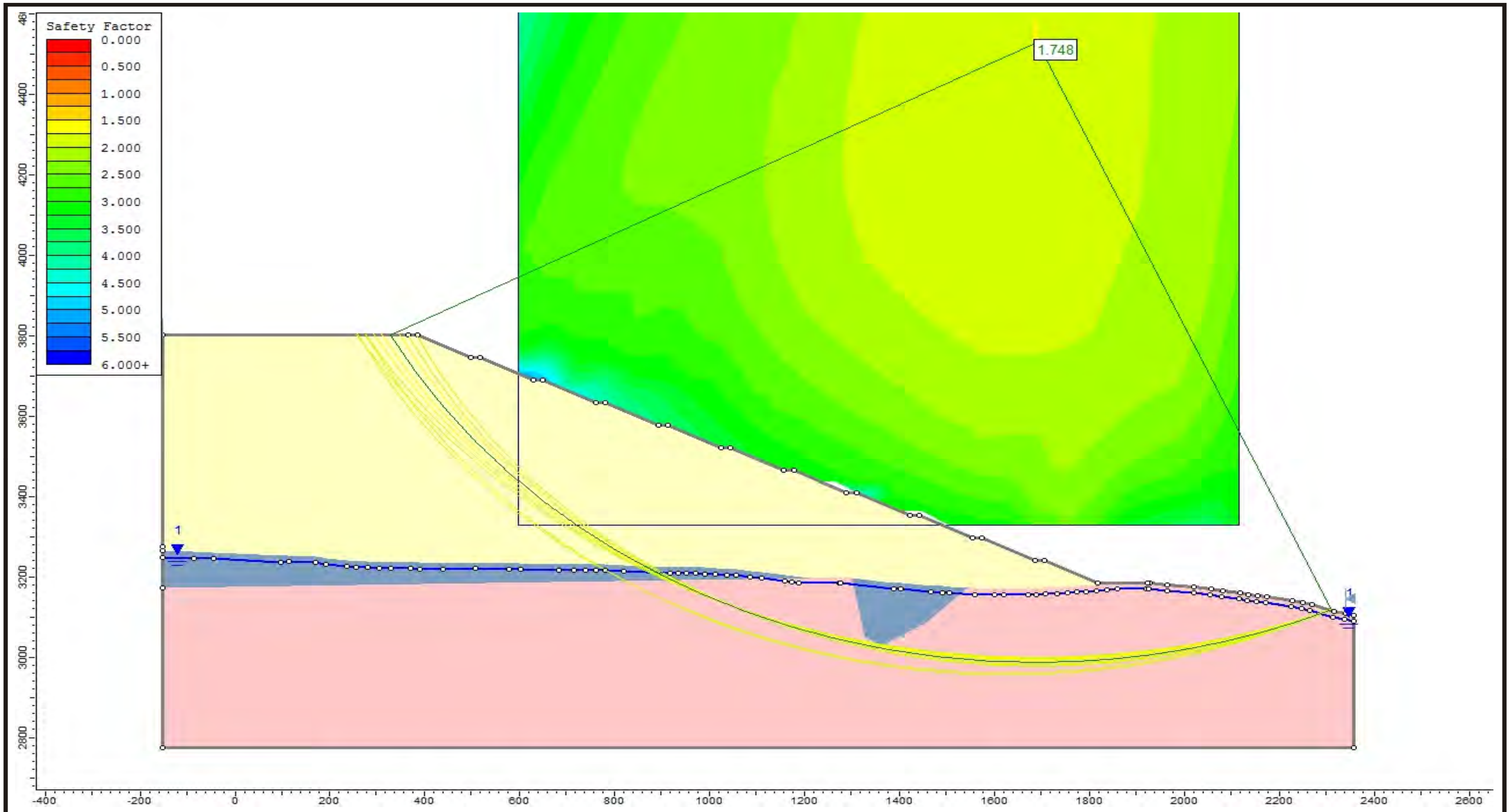
TITLE

Cross-Section C-C Seismic Analysis
(Closure Seismic Coefficient = 0.07g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	27



Golder Associates

Denver, Colorado

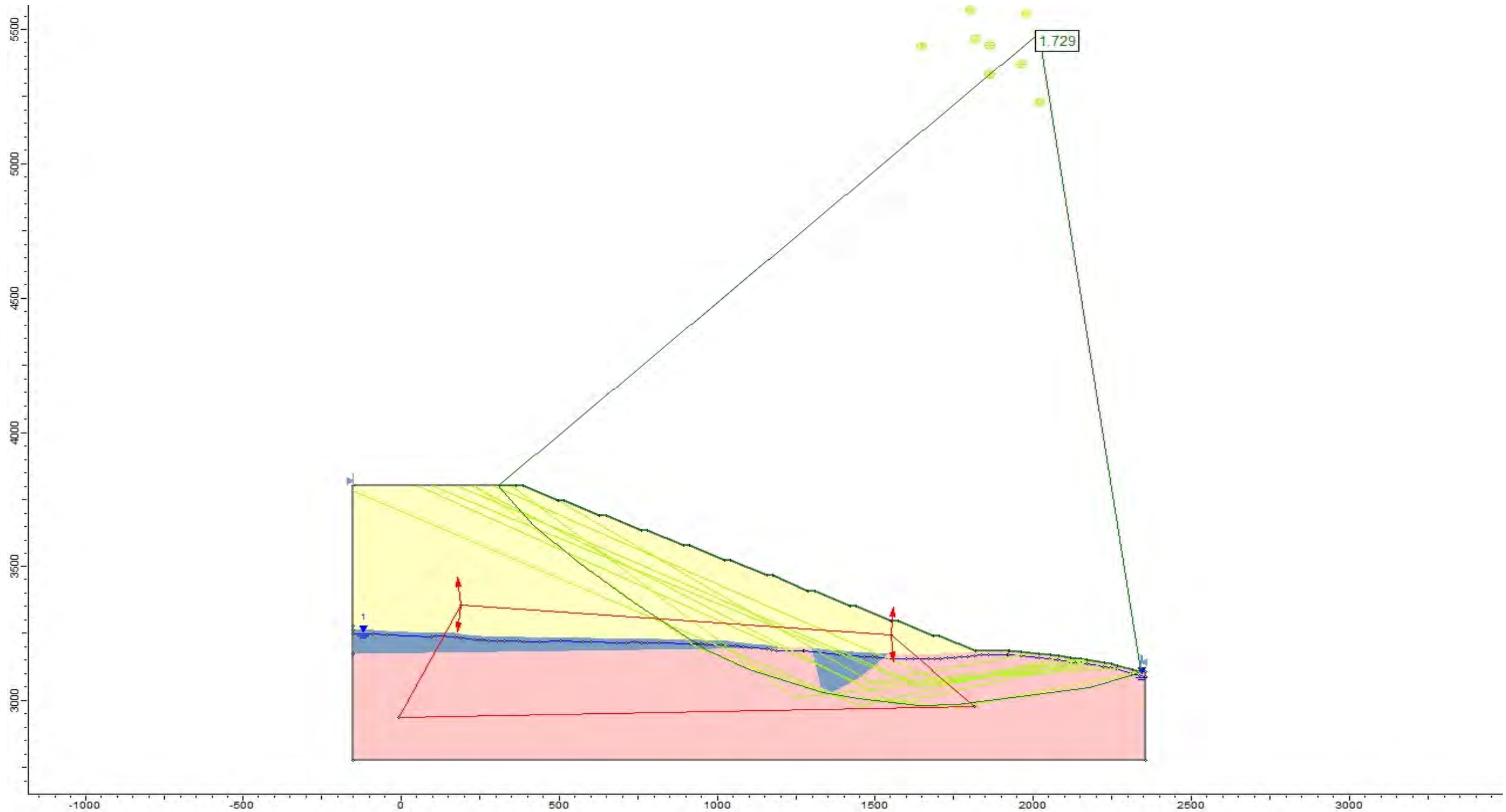
TITLE

Cross-Section D-D
Circular Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	28



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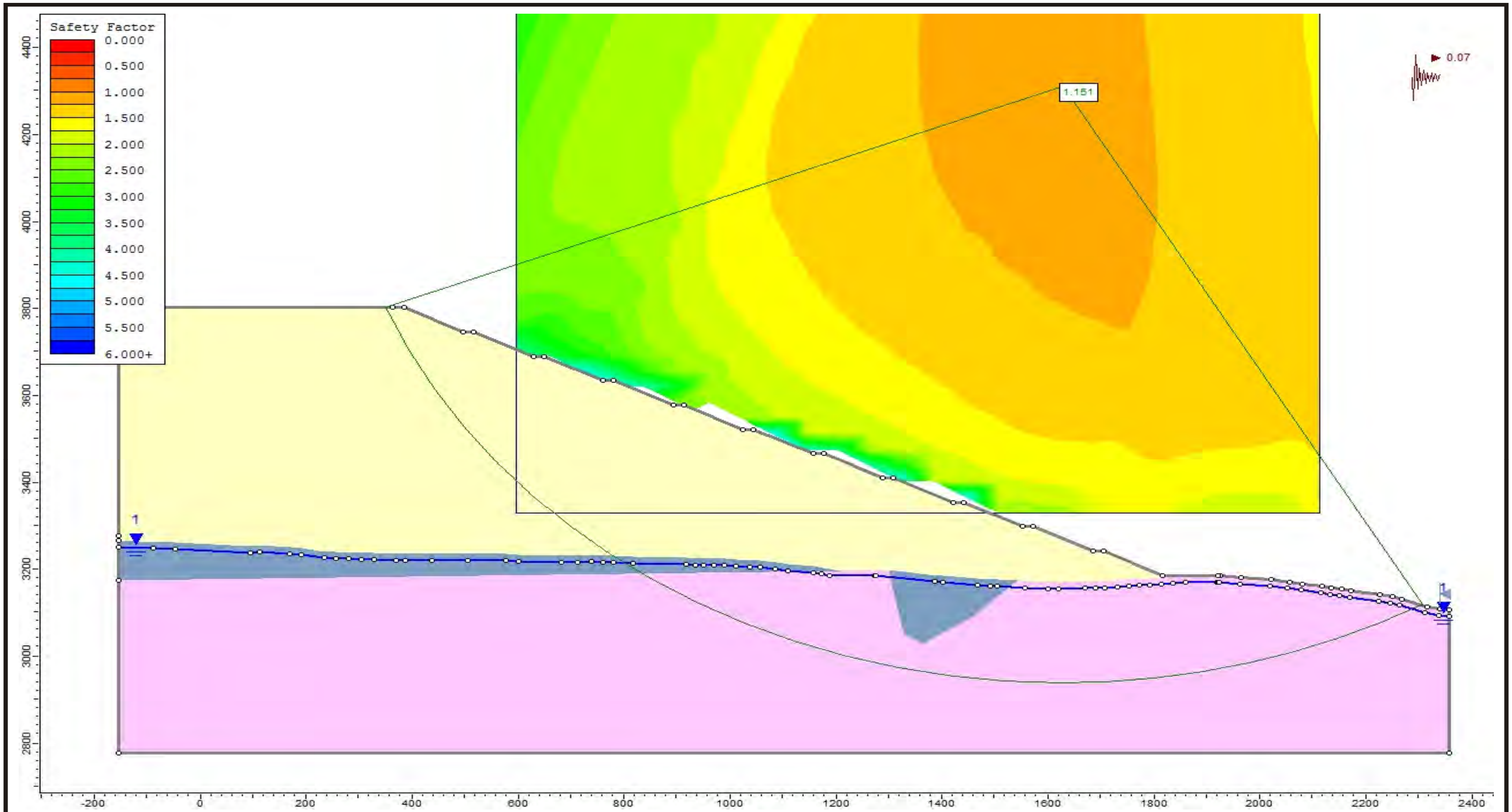
TITLE

Cross-Section D-D
Block Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	29



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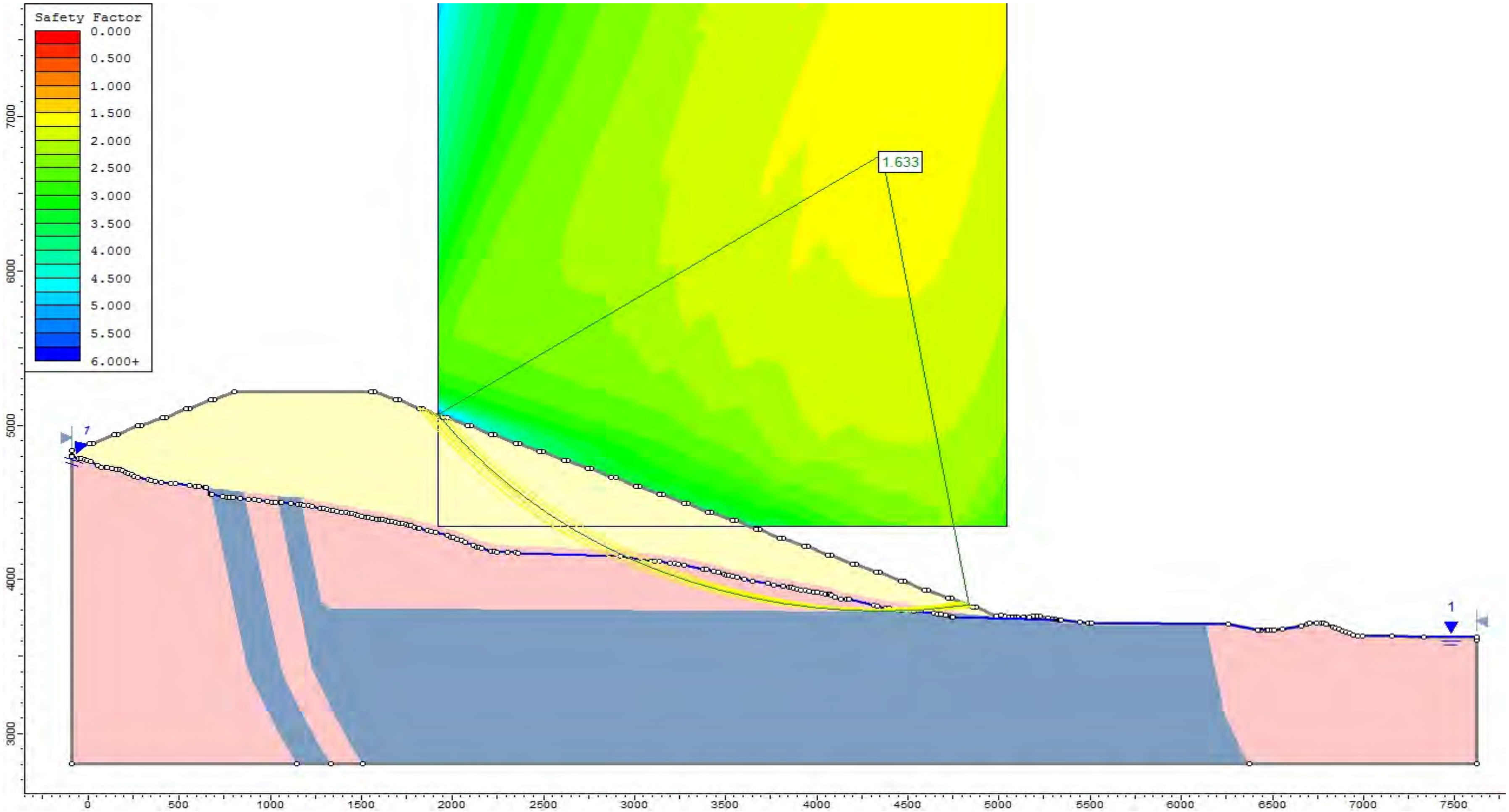
TITLE
Cross-Section D-D Seismic Analysis (Closure Seismic Coefficient = 0.07g)

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	30



Golder Associates

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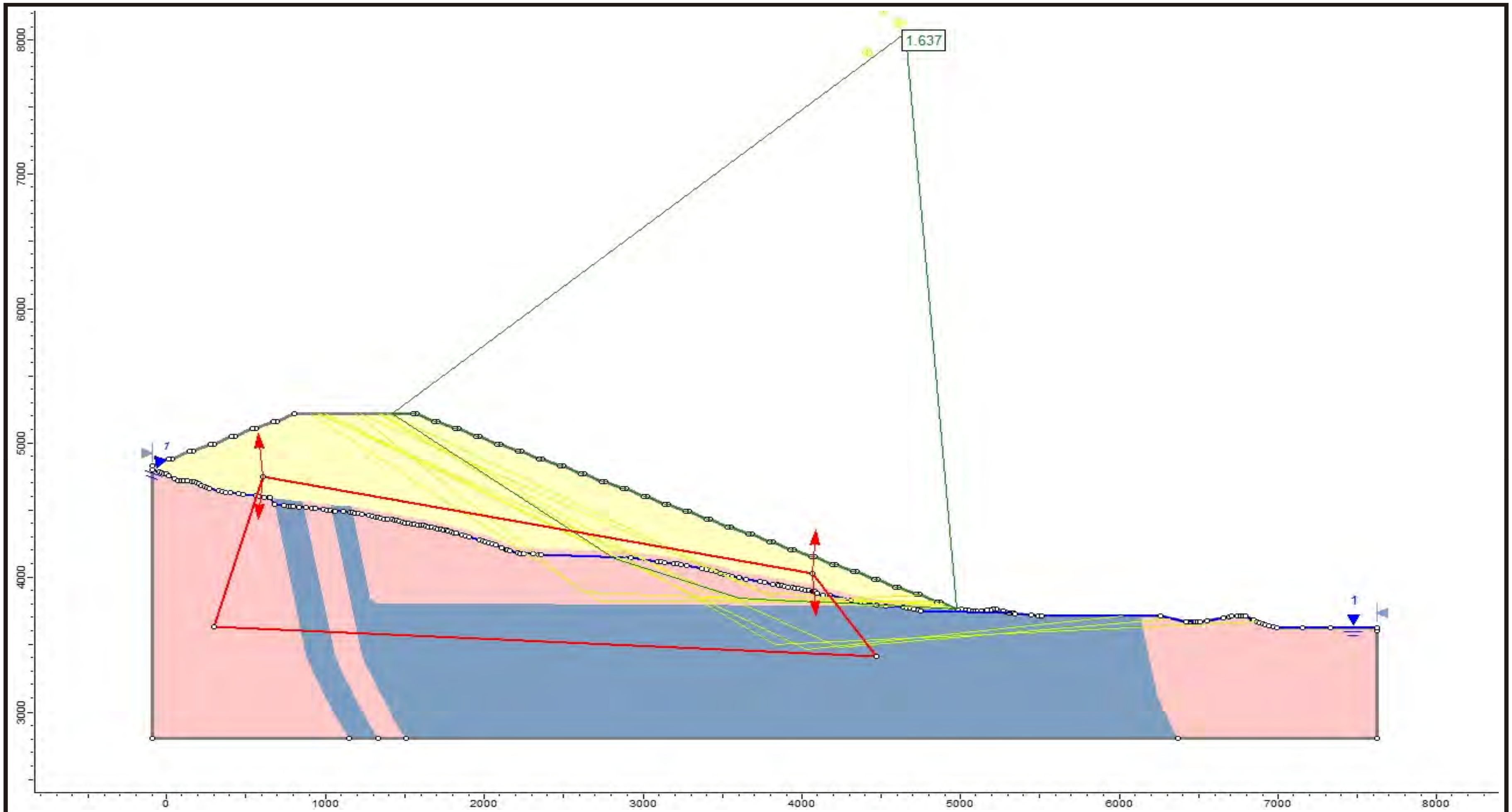
TITLE

Cross-Section E-E
Circular Analysis for Closure

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	31



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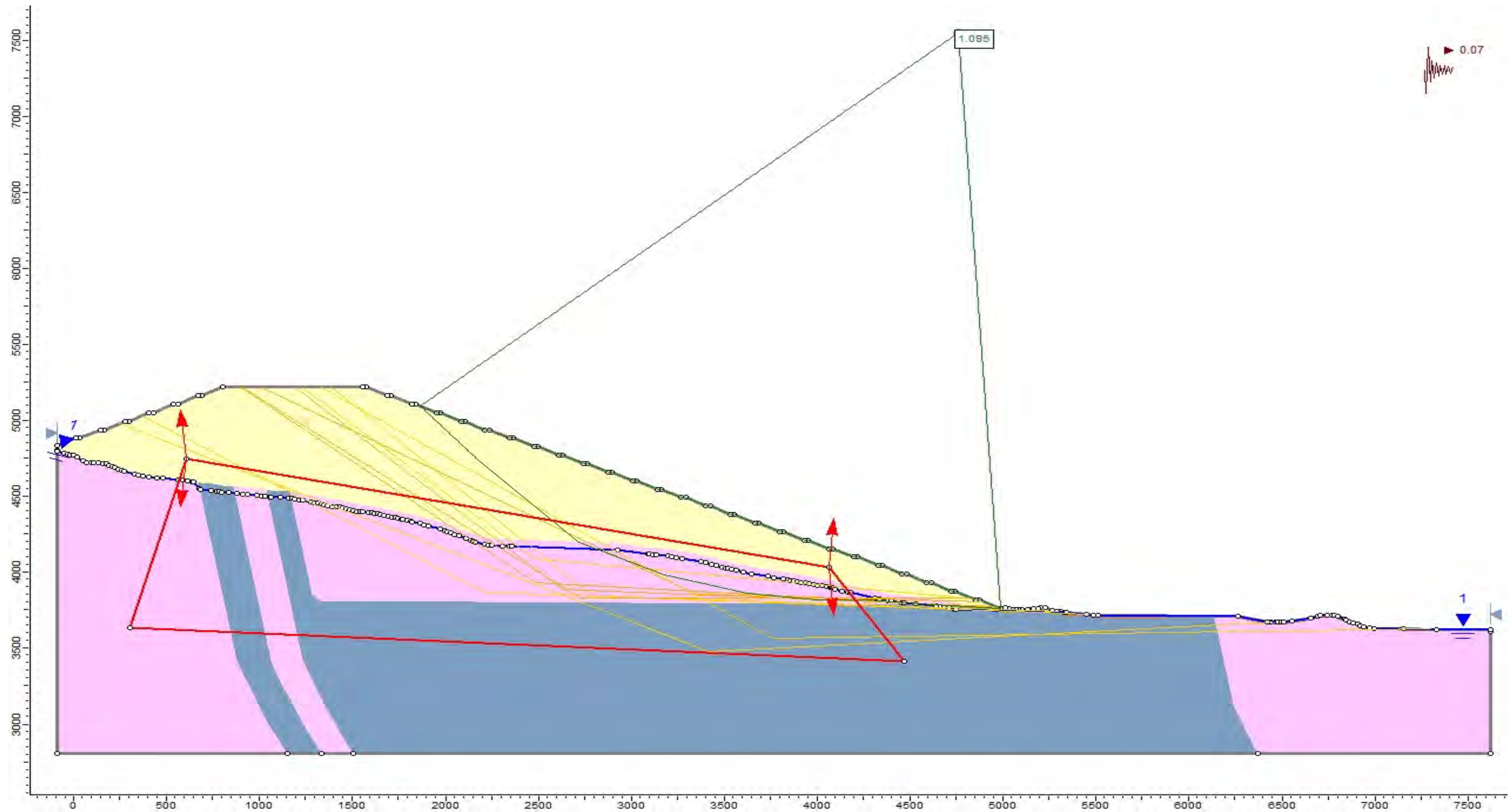
TITLE

Cross-Section E-E
Block Analysis for Closure

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	32

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis



Golder Associates

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TITLE

Cross-Section E-E Seismic Analysis
(Closure Seismic Coefficient = 0.07g)

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum
OSF Stability Analysis

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	N/A
REVIEWED	BRB	FILE NO.		FIGURE NO.	33

Date:	March 7, 2012	**Prepared by:**	D. Rugg
Project No.:	113-81608.2000	**Checked by:**	D. Geier
Projcect Title:	Climax Molybdenum OSF Design Report	**Reviewed by:**	B. Bronson
RE:	**TAILING LIQUEFACTION SUSCEPTIBILITY EVALUATION**		

1.0 OBJECTIVE

The objective of this appendix is to present the analysis performed to determine the potential for the historic tailing deposits located within the footprint of the proposed North 40 overburden storage facility (OSF), located at the Climax Molybdenum Mine (Climax), to liquefy when subjected to the design earthquake loading. Liquefaction is defined as a loss in strength due to a build-up of excess pore water pressure. The generation of this excess pore water pressure is most commonly attributed to cyclic undrained loading, such as that applied by an earthquake.

2.0 ANALYSIS PROCEDURE

In order to determine the liquefaction susceptibility of the historic tailing deposits, a number of assumptions were made, and a number of procedures were employed. The majority of the soil parameters utilized in this study were obtained from the geotechnical field investigation and laboratory testing program, both of which are summarized in the main text of the design report, and which are also presented in detail in Appendices A and B of the design report, respectively.

As a preliminary means of evaluating the nature of the historic tailing deposits a number of correlations between index properties and liquefaction susceptibility were investigated, including the Chinese Criteria (Wang 1979, Youd et al 2001), the Andrews and Martin Criteria (2000), and the Bray and Sancio Criteria (2006). Following this initial investigation, a more detailed look at the liquefaction susceptibility of the historic tailing deposits was performed based on critical state soil mechanics and the state parameter (ψ) as described by Jefferies and Been (2006). Finally, an analysis based on in-situ SPT data and correlations with liquefaction case histories was performed as described by Idriss and Boulanger (2008) and Youd et al (2001).

3.0 SOIL PARAMETERS AND ASSUMPTIONS

As stated previously, the majority of the soil parameters utilized for this study were obtained from the Golder laboratory testing program. This laboratory study included index testing on both tailing fines and tailing beach materials. These index tests included sieve analyses and hydrometers (ASTM C117/C136 and ASTM 422), Atterberg limits (ASTM D4318), specific gravity (ASTM D854), modified Proctor

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compaction (AASHTO T180 Method A), minimum index density determination (ASTM D4254), and natural density and moisture content (ASTM D2937 and D2216). The relevant index properties of these materials are listed in Table 1. The modified Proctor compaction tests were performed on a combination of the tailing beach materials. The maximum density was obtained by utilizing a maximum compactive effort on tailing materials prepared at approximately 2% below the optimum moisture content. All samples tested were determined to be non plastic. The tailing beach materials were non-cohesive with no plastic limit. The tailing fines were cohesive but were also non plastic. Detailed results of all index tests performed can be found in Appendix B to the design report.

In addition to these index properties, a number of relevant soil parameters were determined for the tailing fines by one-dimensional consolidation testing (ASTM D2435) and consolidated undrained (CU) triaxial compression (ASTM D4767). The results of these tests are described in the attached Tables 2 and 3, respectively. The consolidation and triaxial tests were performed on undisturbed samples recovered from Shelby tubes during the field investigation program. Thus, initial void ratios and moisture contents of all tests were assumed to be near the in-situ condition.

During the field investigation program, standard penetration tests (SPTs) were conducted in the tailing deposits to help characterize the in-situ state of the material. Table 4 shows a list of all SPTs conducted in the tailing deposits as well as the associated field blow count values and corrected blow counts.

4.0 LAB DATA ANALYSIS AND INTERPRETATION

4.1 Relative Density

First, to assess the approximate relative density of the tailing deposits, some empirical relationships were investigated based on the SPT results. In addition, the natural density and moisture determined from various Shelby tube samples was compared with the maximum and minimum dry densities achieved in the laboratory. The empirical relationships for relative density with SPT values were originally developed for relatively non cohesive sands and silty sands. Thus, the relationships were not utilized for the tailing fines. Cubrinovsky and Ishihara (1999) defined the relationship shown in Equation 1 for relative density based on the SPT, vertical effective stress, and D_{50}. Table 5 shows the results of applying this relationship to the historic tailing deposits.

$$D_r(\%) = \left[\frac{N_{60}\left(0.23+\frac{0.06}{D_{50}}\right)^{1.7}}{9}\left(\frac{1}{\sigma'_0/P_a}\right)\right]^{0.5}(100) \qquad (1)$$

Kulhawy and Mayne (1990) defined the relationship shown in Equation 2 for relative density based on the SPT, a grain size correlation factor, an aging correlation factor, and an overconsolidation correlation factor. Table 6 shows the results of applying this relationship to the historic tailing deposits. The historic tailing deposits were considered to be normally consolidated (i.e. OCR = 1) and the tailing deposits were considered to be approximately 60 years old.

$$D_r(\%) = \left[\frac{(N_1)_{60}}{C_P C_A C_{OCR}}\right]^{0.5}(100) \qquad (2)$$

Where:

$$C_P = 60 + 25Log(D_{50})$$

$$C_A = 1.2 + 0.05Log\left(\frac{t}{100}\right)$$

$$C_{OCR} = OCR^{0.18}$$

And,

- D_{50} = diameter through which 50% of the soil will pass
- t = age of the soil since deposition (years)
- OCR = overconsolidation ratio of the soil

The data shown in Tables 5 and 6 based on equations 1 and 2 indicate relative densities ranging from approximately 36% to 47%. Natural density and moisture content values obtained from Shelby tube samples were also correlated with maximum and minimum density values achieved in the laboratory. Table 7 shows a comparison between relative compaction (based on the modified proctor test performed in the lab) and relative density (based on minimum and maximum density tests). Natural moisture and density values were utilized to calculate relative densities for 3 field samples and the final void ratio recorded following triaxial testing was also included for comparison. The attached Figure 1 shows a comparison of the modified Proctor results, maximum density test, minimum density test, the approximate in-situ state of the tailing materials, and the final state of the tailing materials following triaxial compression.

4.2 Consolidation

A single one-dimensional consolidation test was performed on a sample of tailing fines from borehole GA-11B-24 at 51.5 ft. Theoretical consolidation modeling and curve fitting were applied to the results of this consolidation test in order to back calculate an approximate permeability of the tailing fines and to develop a continuous relationship between void ratio and effective stress. Equation 3 was utilized to calculate the permeability of the tailing fines based on Terzaghi's theory of consolidation.

$$k = C_v \gamma_w \left(\frac{e_0 - e_1}{\sigma_1 - \sigma_0}\right)\left(\frac{1}{(1+e_0)}\right) \qquad (3)$$

The average permeability of the tailing fines, based on this single one-dimensional consolidation test was thus 2×10^{-7} cm/s. Equation 4 was utilized to develop a continuous relationship between void ratio and effective stress. The best fit parameters utilized in Equation 4 are shown in Table 8. The one-dimensional consolidation laboratory data and best fit curve are shown in Figure 2.

$$e = A(\sigma' + Z)^B \qquad (4)$$

4.3 Critical State

A critical state line (CSL) was developed based on the results of the CU triaxial tests with pore pressure measurements run on the tailing fines. The parameters pertaining to the critical state of the tailing fines are presented in Table 9. Figure 3 displays the CSL for the tailing fines. The CSL displayed in Figure 3 is defined by equation 5.

$$e_{ss} = \Gamma - \lambda \cdot \log{(p')} \qquad (5)$$

Where for this sample of tailing fines, Γ = 1.0967 is the critical state void ratio at a reference pressure of 1 kPa and λ = 0.064 is the slope of the CSL in e- log(p') space. In Figure 3, the solid squares represent the state of the soil at large strains, while the empty diamonds represent the initial state of the soil prior to shearing. Since the tests were performed undrained, the void ratio was constant during shear and pore pressures were generated. The first two triaxial tests, starting at initial effective confining pressures of approximately 25 and 100 psi, displayed dilation during shearing and thus produced positive pore pressures, while the third test, performed at an initial effective stress of 350 psi was slightly contractive.

5.0 INDEX PROPERTY CRITERIA FOR LIQUEFACTION SUSCEPTIBILITY

Three liquefaction susceptibility criteria were considered based on index properties and were utilized to determine if the buried tailing material might be susceptible to liquefaction. The first method utilized was the Chinese Criteria. In order for a material to be potentially liquefiable, based on the Chinese Criteria, all three of the following statements should be true:

- 15% or less of the soil is finer than 0.005 mm.
- Liquid limit less than or equal to 35%
- Natural water content greater than or equal to 0.9*LL

Table 10 gives a summary of each of these criteria for all tailing samples tested. Although the criteria cannot be applied to every sample, the Chinese screening criteria seem to suggest that the tailing beach materials may be potentially susceptible to liquefaction, while the tailing fines are not susceptible to liquefaction due to the high fines content.

The next index based screening criteria that was investigated was the Andrews and Martin (2000) Criteria. These criteria state that in order to be liquefiable the liquid limit should be less than 32% and the percent finer than 2 μm should be less than 10%. This criteria was plotted in Figure 4. Based on these screening criteria, the tailing beach materials are classified as potentially susceptible to liquefaction while the tailing fines fall in a region that suggests more rigorous analyses are required to determine if liquefaction is a possibility.

The final index based criteria that was investigated was the Bray and Sancio (2006) Criteria. These criteria state that the moisture content to liquid limit ratio should be greater than 0.85 and the plasticity index should be less than 12 in order for the soil to be potentially liquefiable. These criteria were plotted on Figure 5. By using the in-situ water content obtained by natural moisture tests performed on Shelby tube samples and by assuming that the liquid limit of all materials is no greater than the plastic limit of the tailing fines (this gives the highest LL possible while still maintaining non-plastic conditions, i.e. PI = 0), then it appears as though both the tailing beach and tailing fines may be potentially susceptible to liquefaction.

5.1 Determination of Earthquake Induced Stresses

For this screening study, a simplified procedure was used to estimate the earthquake induced stresses on the historic tailing deposits. The Maximum Design Earthquake event (MDE) was utilized for this analysis. This method of analysis is applicable only to a depth of approximately 30 meters or 100 feet. Given the current depth of the historic tailing deposits, this analysis is applicable. Following the continued placement of overburden above the tailing deposits, the increase in stress on the tailing deposits will increase the resistance to liquefaction. The approach utilized for this analysis was proposed by Seed and Idriss (1971) and further recommended by Youd et al (2001) and assumes that a soil column acts as a rigid body. Then, given the Peak Ground Acceleration (PGA), the shear stress at the base of this soil column can be calculated as shown in Equation 6.

$$\tau_{max} = 0.65 \cdot PGA \cdot \sigma_v \cdot r_d \qquad (6)$$

Where 0.65 is a scaling factor to represent the reference earthquake stress level, PGA is the peak ground acceleration, σ_v is the total overburden stress, and r_d is a reduction factor to account for the fact that the soil column will behave as a deformable body and not as a rigid body. This reduction factor is a function of depth and can be calculated by Equation 7.

$$r_d = \frac{(1.000-0.4113\cdot z^{0.5}+0.04052\cdot z+0.001753\cdot z^{1.5})}{(1.000-0.4177\cdot z^{0.5}+0.05729\cdot z+0.006205\cdot z^{1.5}+0.001210\cdot z^2)} \qquad (7)$$

The expression for r_d expressed by Equation 7 is a numerical approximation for the curve presented by Youd et al (2001). The uncertainty in r_d increases with depth, and thus below 100 ft, r_d should be approximated as 0.5 (Youd and Idriss 1997). Finally, the Critical Stress Ratio (CSR) can then be calculated by Equation 8.

$$CSR_{eq} = \frac{\tau_{max}}{\sigma'_v} \qquad (8)$$

Where τ_{max} is the maximum shear stress calculated by Equation 6, and σ'_v is the effective overburden stress. The parameters utilized for the MDE are listed in Table 11.

For this analysis, the minimum depth to the top of the historic tailing deposits was assumed to be 25 ft. The water table was assumed to be at the top of the tailing deposit and the thickness of the tailing deposit was assumed to be a maximum of 75 ft, based on the results of the field investigation. Given the MDE and the design equations described above, the CSR can then be calculated with depth. The strength reduction factor, r_d is plotted with depth in Figure 6, and the CSR_{eq} is plotted with depth in Figure 7. A table of these values is presented in Table 12.

Once the expected earthquake stresses have been calculated, then the Cyclic Resistance Ratio (CRR) of the soil must be determined in order to compute a factor of safety against liquefaction (FS_{liq}) as shown in Equation 9.

$$FS_{liq} = \frac{CRR_{scaled}}{CSR_{eq}} \qquad (9)$$

5.2 State Parameter Approach

Jefferies and Bean (2006) presented a state parameter approach for determining the Cyclic Resistance Ratio of a soil. Most correlations between CRR and liquefaction are based on a M_w = 7.5 earthquake, a reference overburden stress of 100 kPa, and a gently sloping ground surface. Thus, these correlations are generally corrected with a Magnitude Scaling Factor (MSF), an overburden scaling factor K_σ, and a static shear stress factor K_α. Due to the nature of the State Parameter (ψ), however, no overburden scaling factor needs to be applied. This is due to the fact that ψ is determined at a range in effective stresses and thus should already account for changes in effective overburden stress. Equation 10 shows the relationship proposed by Jefferies and Been (2006) for $CRR_{7.5}$.

$$CRR_{7.5} = 0.03 \cdot exp^{(-11 \cdot \psi)} \qquad (10)$$

Where ψ is the state parameter, defined as the vertical distance between the in-situ void ratio and the critical state line (CSL).

$$\psi = e_{in-situ} - e_{CSL} \qquad (11)$$

A positive state parameter indicates that the soil is contractive (looser than critical state) and thus will generate positive pore pressures and have the potential to liquefy. A negative ψ indicates that the soil structure is dilative and will not liquefy. Figure 3 displays the CSL as determined from CU triaxial tests with pore pressure measurements (the parameters used to define this line can be seen in Table 9).

Once the CSL has been adequately defined, the expected in-situ void ratio of the historic tailing deposits must be defined. In-situ void ratios were defined in two ways. The first method was based on the results of the one-dimensional consolidation test (Figure 2). This sample was taken from a Shelby tube collected during the field investigation. Thus, by calculating the vertical effective stress throughout the tailing layer, the in-situ void ratio can be estimated by Equation 4 and the parameters listed in Table 8. The second method for estimating the in-situ void ratios was by performing natural moisture and density tests on Shelby tube samples recovered from the tailing deposits. Both methods for estimating the in-situ void ratio and effective stresses are shown in Figure 8 in relation to the CSL. All void ratios were plotted in Figure 8 with the corresponding mean effective stress as calculated by Equation 12.

$$\sigma'_m = \frac{(\sigma'_{v0} + 2 \cdot k_0 \cdot \sigma'_{v0})}{3} \qquad (12)$$

Where σ'_{v0} is the vertical effective stress and K_0 is the at rest earth pressure coefficient determined by Jaky's relationship (1948).

$$K_0 = 1 - \sin(\emptyset') \qquad (13)$$

The calculated values of state parameter, at each depth can be seen in Table 13. The void ratio for no liquefaction was determined by the relationship described in Equation 5 and the parameters shown in Table 9.

Once the state parameter has been determined, then the Jefferies and Been (2006) relationship was utilized to calculate $CRR_{7.5}$ as shown in Equation 10. The Cyclic Resistance Ratio was then corrected for a Magnitude Scaling Factor as recommended by Youd et al (2001). The MSF was calculated to be 1.770 by utilizing Equation 14.

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \qquad (14)$$

Where M_w is the moment magnitude determined for the MDE (Table 11).

As stated previously, $CSR_{7.5}$ was not corrected for K_σ since ψ already takes the overburden stress into account. Also, Jefferies and Been (2006) recommend numerical modeling to determine K_α, however, for preliminary purposes, K_α was conservatively neglected. Therefore CRR_{scaled} can be determined by Equation 15.

$$CRR_{scaled} = CRR_{7.5} \cdot MSF \qquad (15)$$

Table 14 shows the factor of safety against liquefaction calculated by Equation 9. If FS_{liq} is less than 1.0, potential liquefaction exists. If FS_{liq} is greater than 1.0, liquefaction is unlikely. Experience has shown that the Simplified Method is somewhat conservative; so many designers consider FS values close to unity as an indication of no liquefaction. As can be seen in Table 14, this method of analysis indicates that the factor of safety against liquefaction is between 1.6 and 3.6 and thus the historic tailing deposits are not expected to liquefy under the design earthquake loading. Figure 9 shows a plot of the factor of safety against liquefaction versus depth.

5.3 Standard Penetration Test Approach

As a check to the State Parameter approach, which utilized laboratory testing to estimate the in-situ state of the historic tailing deposits, a second analysis was performed based on the Standard Penetration Tests (SPTs) performed during the October-November 2011 field investigation.

The SPT approach to liquefaction evaluation is one of the most reliable methods of liquefaction analysis due to the abundance of SPT data at sites that have experienced earthquake shaking.

A record of all SPTs performed during the field investigation are summarized in Appendix A of the design report. Of the 20 SPTs performed during the drilling program, 5 were performed in the historic tailing deposits. The depth, location, calculated effective stresses, calculated CSR, and recorded and corrected SPT values are shown in Table 15. All values presented in Table 15 were calculated as described previously. Field recorded N values (blow counts) were corrected for rod length, hammer efficiency, and overburden stress to determine $(N_1)_{60}$.

Most relationships in the literature correlate $(N_1)_{60}$ blow count values with case histories of liquefaction in clean sands (i.e. sands with < 5% passing the #200 sieve). Thus, many researchers have formulated methods to account for sands with higher fines contents. Generally the method has been to correct blow counts to some “clean sand” blow count value. Youd et al (2001) presented a procedure for this adjustment. Based on the Youd et al. (2001) procedure, $(N_1)_{60cs}$ can be calculated by Equation (16).

$$(N_1)_{60cs} = \alpha + \beta \cdot (N_1)_{60} \qquad (16)$$

where,

$\alpha = 0$ For fines content (FC) <= 5%

$\alpha = e^{\left(1.76-\left(\frac{190}{FC^2}\right)\right)}$ For 5% < FC < 35%

$\alpha = 5$ For FC >= 35%

and,

$\beta = 1.0$ For fines content (FC) <= 5%

$\beta = 0.99 + \left(\frac{FC^{1.5}}{1000}\right)$ For 5% < FC < 35%

$\beta = 1.2$ For FC >= 35%

Table 16 shows the calculation of $(N_1)_{60cs}$ based on Equation 16 and the fines content determined for each sample in the laboratory.

Next, the unscaled Cyclic Resistance Ratio can be determined for the corrected clean sand SPT blow counts by utilizing Equation 17 developed by Idriss and Boulanger (2008). Table 17 shows the calculated CRR values.

$$CRR_{unscaled} = e^{\left[\left(\frac{(N_1)_{60cs}}{14.1}\right)+\left(\frac{(N_1)_{60cs}}{126}\right)^2-\left(\frac{(N_1)_{60cs}}{23.6}\right)^3+\left(\frac{(N_1)_{60cs}}{25.4}\right)^4-2.8\right]} \quad (17)$$

Next CRR must be scaled for the moment magnitude of the design earthquake and the vertical effective stress. The magnitude scaling factor (MSF) is the same as calculated by Equation 14. The CRR must also be corrected for overburden stress (K_σ) and static shear stress (K_α). K_σ was determined based on the recommendations from Youd et al (2001) where f was assumed to be 0.7 for medium dense silty sand. Thus, Equation 18 was applied for K_σ.

$$K_\sigma = (\sigma'_{v0})^{f-1} \quad (18)$$

Where σ'_{v0} is the effective overburden stress expressed in atmospheres. Finally, since Youd et al (2001) did not give a recommendation for K_α, this factor was again conservatively ignored. Table 17 shows the scaled values of CRR based on the corrected SPT blow counts.

Finally, the factor of safety against liquefaction was calculated according to Equation 9. These values are shown for each SPT in Table 18. The FS_{liq} calculated by the SPT method are shown in Figure 10. The calculated FS_{liq} based on the SPT method gives factors of safety ranging from 2.1 to 2.9, reinforcing the results from the state parameter based approach.

6.0 CONCLUSIONS

One of the most challenging components to determine during a liquefaction analysis is the in-situ state of the material. For this study, a number of different methods were employed to investigate the in-situ state of the historic tailing deposits and a number of analysis methods were utilized to investigate liquefaction susceptibility. Laboratory tests and SPT correlations showed that the approximate in-situ relative density of the historic tailing deposits ranges from 36 to 67%. This was used as an initial screening tool and indicates that the tailing deposits are potentially in a liquefiable state. Three different screening level liquefaction susceptibility criteria were then investigated. These criteria are based on index properties including, in-situ moisture content, grain size (notably fines content) of the materials, and Atterberg limits. The Chinese criteria indicated that the tailing beach materials are potentially liquefiable and the tailing fines are likely not liquefiable. The Andrews and Martin criteria also indicated that the tailing beach materials are potentially liquefiable and that the tailing fines require more rigorous testing and analysis to determine their liquefaction potential. Finally, the Bray and Sancio criteria indicates that both the tailing beach and tailing fines are potentially liquefiable. Thus, further analysis was justified.

The critical stress ratio (CSR) predicted based on the MDE was determined by the method proposed by Youd et al. (2001). After the determination of the CSR, two methods were used to estimate the cyclic resistance ratio (CRR) of the historic tailing deposits. Both methods are based on the in-situ state of the deposits, however, this in-situ state is measured by completely different methods. First the State Parameter was evaluated as recommended by Jefferies and Been (2006). The state parameter corresponding to no-liquefaction was determined by a series of three consolidated undrained triaxial tests. The in-situ state of the soil was then estimated by a continuous void ratio-effective stress relationship based on one-dimensional laboratory consolidation and by natural density results from Shelby tube samples. Based on this state parameter analysis, the in-situ state of the soil indicates that shearing will cause dilation and thus strain hardening. Factors of safety calculated from this analysis range from 1.6 to 3.6 (Table 14).

Finally, the in-situ state of the tailing deposits was investigated by the use of the Standard Penetration Test (SPT). SPT values were recorded during the October-November 2011 field investigation and these values were correlated with CRR. The relationship between SPT blow counts and CRR utilized in this study was recommended by Idriss and Boulanger (2008) and is based on a database of SPT values recorded in locations subjected to earthquake loading where liquefaction has either occurred or not occurred. The blow counts recorded in the field were corrected for overburden stress, rod length, and hammer efficiency to obtain the $(N_1)_{60}$ blow count value. Next, these blow counts were corrected for fines content to determine the equivalent clean sand blow count values. Finally, CRR was determined by the Idriss and Boulanger (2008) relationship and this CRR was corrected for the earthquake magnitude and overburden stress. The factor of safety against liquefaction determined by this method ranges from 2.1 to 2.9.

7.0 REFERENCES

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List of Attachments

Tables 1 through 18
Figures 1 through 10

TABLES

Table 1
Summary of Index Testing Results on Tailing Materials

Soil Type	Boring/Pit Number	Sample Depth (ft)	USCS Soil Classification	Moisture Content (%)	In-Situ Dry Density (pcf)	Specific Gravity	Atterberg Limits			Grain Size Distribution				Moisture/Density Relationship		Minimum Density (pcf)	Maximum Density (pcf)
							LL	PL	PI	% Finer 3/4"	% Finer #4	% Finer #200	% Finer 0.005 mm	Dry Density (pcf)	Moisture (%)		
Tailing Beach	GA-11B-24	91-92	SM	--	--	--	NP	NP	NP	100	100	36	2.5	119.0[1]	10.1	73.1	120.7[2]
	GA-11B-25	43.5	SM	--	--	--	NP	NP	NP	100	100	26	1				
	GA-11B-25	55	SM	--	--	--	NP	NP	NP	100	100	25	6.5				
	GA-11B-25	42	--	26.9	97.8	--	--	--	--	--	--	--	--				
Tailing Fines	GA-11B-24	51.5-53.6	ML	31.9	--	2.75	<25.3	25.3	NP	100	100	99	25	--	--	--	--
	GA-11B-24	71-73.2	ML	30.6	86	--	NP	NP	NP	100	100	97	33.5	--	--	--	--
	GA-11B-23	190-192	--	25.5	100.3	--	--	--	--	--	--	--	--	--	--	--	--

[1] Maximum dry density and corresponding moisture content determined by the modified Proctor method.

[2] Maximum dry density determined by compaction in a Proctor mold utilizing maximum compactive effort performed on soil at 2% below the optimum moisture content.

Table 2
Summary of Consolidation Testing Results for Tailing Fines

Soil Type	Boring/Pit Number	Sample Depth (ft)	Test Type	Initial Void Ratio	Initial Dry Density (pcf)	Final Void Ratio	Final Dry Density (pcf)	Average Cv (ft^2/day)	Recompression Index C_r	Compression Index C_c	Swell Index C_s
Tailing Fines	GA-11B-24	51.5-53.6	One-Dimensional Consolidation	0.9	90.1	0.78	96.6	2.63	0.03	0.11	0.02

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Table 3
Summary of Strength Testing Results for Tailing Fines

Soil Type	Boring/Pit Number	Sample Depth (ft)	Test Type	Ave. Initial Void Ratio	Total Stress		Effective Stress	
					Cohesion (psi)	Friction Angle (degrees)	Cohesion (psi)	Friction Angle (degrees)
Tailing Fines	GA-11B-24	51.5-53.6	CU Triaxial	0.921	11.6	17.8	0	32.5

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Table 4
Standard Penetration Test Results and Corrections

Depth (ft)	Location	Material	N first 6"	N second 6"	N third 6"	N	Rod Correction (C_r)	N_{60}	Overburden Stress (psf) σ'_{vo}	Overburden Correction C_n	$(N_1)_{60}$
61	GA-11B-24	Tailings	5	7	8	15	1	16	6634	0.57	9
73.2	GA-11B-24	Tailings	5	6	6	12	1	13	7276	0.54	7
91	GA-11B-24	Tailings	5	6	7	13	1	14	8212	0.51	7
44.5	GA-11B-25	Tailings	4	4	5	9	1	9	4456	0.69	7
55	GA-11B-25	Tailings	3	4	5	9	1	9	5008	0.65	6

Table 5
SPT and Relative Density Relationship developed by Cubrinovski and Ishihara (1999)

Depth (ft)	Location	Material	N_{60}	D_{50}	Overburden Stress (psf) σ'_{vo}	D_r (%)
91	GA-11B-24	Tailing Beach	14	0.12	8212	47
44.5	GA-11B-25	Tailing Beach	9	0.16	4456	45
55	GA-11B-25	Tailing Beach	9	0.17	5008	41

Table 6
SPT and Relative Density Relationship developed by Kulhawy and Mayne (1990)

Tailings age: 60 years OCR: 1

Depth (ft)	Location	Material	$(N_1)_{60}$	D_{50}	C_p	C_a	C_{OCR}	D_r (%)
91	GA-11B-24	Tailings	7	0.12	37.0	1.19	1.00	40
44.5	GA-11B-25	Tailings	7	0.16	40.1	1.19	1.00	37
55	GA-11B-25	Tailings	6	0.17	40.8	1.19	1.00	36

Table 7
Comparison of Relative Compaction and Relative Density for Tailing Materials

	Sample and Depth	Relative Compaction (%)	Dry Density (pcf)	Total Density (pcf)	Relative Density (%)
Modified Proctor Results	GA-11B-24 @91'	95%	113.05	124.58	90%
	GA-11B-25 @43.5'	90%	107.10	118.02	80%
	GA-11B-25 @55'	85%	101.15	111.47	70%
Natural Density Results	GA-11B-25 @42'	82%	97.80	124.11	64%
	GA-11B-24 @71'	72%	86.00	112.32	38%
	GA-11B-23 @190'	84%	100.30	125.88	69%
Triaxial Test Conditions	GA-11B-24 @51.5'	80%	95.76	123.34	60%
	GA-11B-24 @51.5'	79%	94.17	122.33	57%
	GA-11B-24 @71'	83%	99.26	125.57	67%

Table 8
Best-fit Consolidation Parameters

Material	A	B	Z (psi)
Tailing Fines	1.16	-0.09	19.95

Table 9
Critical State Parameters Determined from CU Triaxial Testing

Sample	$\gamma_{d,\ shear}$ (psi)	e_{shear}	$\sigma_{1,\ initial}$ (psi)	$\sigma_{3,\ initial}$ (psi)	$\sigma'_{1,\ initial}$ (psi)	$\sigma'_{3,\ initial}$ (psi)	$p'_{initial}$ (psi)	$\sigma_{1,\ final}$ (psi)	$\sigma_{3,\ final}$ (psi)	$\sigma'_{1,\ final}$ (psi)	$\sigma'_{3,\ final}$ (psi)	p'_{final} (psi)
GA-11B-24 @51.5'	95.8	0.792	65	65	23.6	23.6	23.6	165	65	138.5	38.5	88.5
GA-11B-24 @51.5'	94.2	0.822	130	130	98.6	98.6	98.6	249	130	166.1	47	106.55
GA-11B-24 @71'	99.3	0.729	380	380	350.1	350.1	350.1	729.6	380	481.9	132.3	307.1

Table 10
Chinese Criteria for Liquefaction Susceptibility Screening (Wang 1979)

	Sample	Wc (%)	LL (%)	PL (%)	PI	% Passing 5 μm	Chinese Criteria: % smaller than 5 μm < 15%	Chinese Criteria: LL < 35	Chinese Criteria: Wc/LL > 0.9
Tailing Beach	GA-11B-24@91	n/a	0	0	0	2.5	True	True	n/a
	GA-11B-25@43	n/a	0	0	0	1	True	True	n/a
	GA-11B-25@55	n/a	0	0	0	6.5	True	True	n/a
	GA-11B-25@42	26.9	0	0	0	n/a	n/a	True	True
Tailing Fines	GA-11B-24@51.5	31.9	<25.3	25.3	0	25	False	True	True
	GA-11B-24@71	30.6	<25.3	25.3	0	33.5	False	True	True
	GA-11B-23@190	25.5	<25.3	25.3	0	n/a	n/a	True	True

Table 11
Maximum Design Earthquake Parameters

Return Period	0.00040404	years
Moment Magnitude	6.0	
Peak Ground Acceleration	0.1446	g

Table 12
Values for the Calculation of CSR_{eq}

Depth (ft)	σ_{v0} (psi)	σ'_{v0} (psi)	r_d	τ_{max}	CSR_{eq}
25	20.8	20.8	0.942	1.84	0.089
30	25.1	22.9	0.921	2.17	0.095
35	29.4	25.1	0.891	2.46	0.098
40	33.7	27.2	0.851	2.69	0.099
45	37.9	29.3	0.804	2.87	0.098
50	42.2	31.4	0.753	2.99	0.095
55	46.5	33.5	0.703	3.07	0.092
60	50.8	35.6	0.659	3.15	0.088
65	55.1	37.7	0.622	3.22	0.085
70	59.3	39.8	0.592	3.30	0.083
75	63.6	41.9	0.568	3.39	0.081
80	67.9	44.1	0.548	3.50	0.079
85	72.2	46.2	0.532	3.61	0.078
90	76.4	48.3	0.519	3.73	0.077
95	80.7	50.4	0.508	3.86	0.077
100	85.0	52.5	0.499	3.98	0.076

Table 13
Estimated In-Situ Void Ratios, Void Ratios for No Liquefaction, and State Parameter

Depth (ft)	σ_{v0} (psi)	σ'_{v0} (psi)	σ_m' (psi)	Estimated $e_{in\text{-}situ}$	$e_{no\ liquefaction}$	State Parameter ψ
Void Ratios Based on One-Dimensional Consolidation						
25	20.8	20.8	13.4	0.82779	0.931	-0.103
30	25.1	22.9	14.7	0.82398	0.925	-0.101
35	29.4	25.1	16.1	0.82037	0.919	-0.099
40	33.7	27.2	17.4	0.81694	0.914	-0.097
45	37.9	29.3	18.8	0.81367	0.909	-0.095
50	42.2	31.4	20.1	0.81056	0.905	-0.094
55	46.5	33.5	21.5	0.80758	0.900	-0.093
60	50.8	35.6	22.9	0.80472	0.896	-0.092
65	55.1	37.7	24.2	0.80198	0.893	-0.091
70	59.3	39.8	25.6	0.79935	0.889	-0.090
75	63.6	41.9	26.9	0.79682	0.886	-0.089
80	67.9	44.1	28.3	0.79438	0.883	-0.088
85	72.2	46.2	29.6	0.79203	0.880	-0.088
90	76.4	48.3	31.0	0.78975	0.877	-0.087
95	80.7	50.4	32.3	0.78756	0.874	-0.087
100	85.0	52.5	33.7	0.78543	0.872	-0.086
Void Ratios Based on Natural Density and Moisture Testing on Shelby Tubes						
42	35.4	28.0	18.0	0.73975	0.912	-0.172
72	61.0	40.7	26.1	0.84150	0.888	-0.046
191	162.9	90.9	58.4	0.70125	0.836	-0.135

Table 14
Factor of Safety Against Liquefaction Determined by the State Parameter Approach

Depth (ft)	CSR_{eq}	CRR_{scaled}	$FS_{liquefaction}$
Values Determined by One-Dimensional Consolidation			
25	0.089	0.165	1.86
30	0.095	0.161	1.70
35	0.098	0.157	1.60
40	0.099	0.154	1.55
45	0.098	0.151	1.55
50	0.095	0.149	1.57
55	0.092	0.147	1.61
60	0.088	0.146	1.65
65	0.085	0.144	1.69
70	0.083	0.143	1.72
75	0.081	0.142	1.75
80	0.079	0.140	1.77
85	0.078	0.139	1.78
90	0.077	0.139	1.79
95	0.077	0.138	1.80
100	0.076	0.137	1.81
Values Determined by Natural Density and Moisture of Shelby Tube Samples			
42	0.099	0.352	3.56
72	0.082	0.088	1.08
191	0.067	0.235	3.48

Table 15
Parameters Pertaining to SPT Based Liquefaction Analysis

Depth (ft)	Location	σ_{v0} (psi)	σ'_{v0} (psi)	r_d	τ_{max}	CSR_{eq}	$(N_1)_{60}$
44.5	GA-11B-25	37.3	32.6	0.808	2.84	0.087	7
55	GA-11B-25	46.3	37.0	0.703	3.06	0.083	6
51	GA-11B-24	42.9	40.4	0.742	2.99	0.074	9
73.2	GA-11B-24	61.9	49.8	0.576	3.35	0.067	7
91	GA-11B-24	77.1	57.3	0.517	3.75	0.065	7

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Table 16
Calculation of Clean Sand Blow Counts based on Fines Content

Depth (ft)	Location	$(N_1)_{60}$	% Passing #200	α	β	$(N_1)_{60cs}$
44.5	GA-11B-25	7	26	4.388	1.123	12
55	GA-11B-25	6	25	4.289	1.115	11
51	GA-11B-24	9	99	5.000	1.200	16
73.2	GA-11B-24	7	97	5.000	1.200	13
91	GA-11B-24	7	36	5.000	1.200	13

Table 17
Unscaled and Scaled Cyclic Resistance Ratios Based on Clean Sand SPT Blow Counts

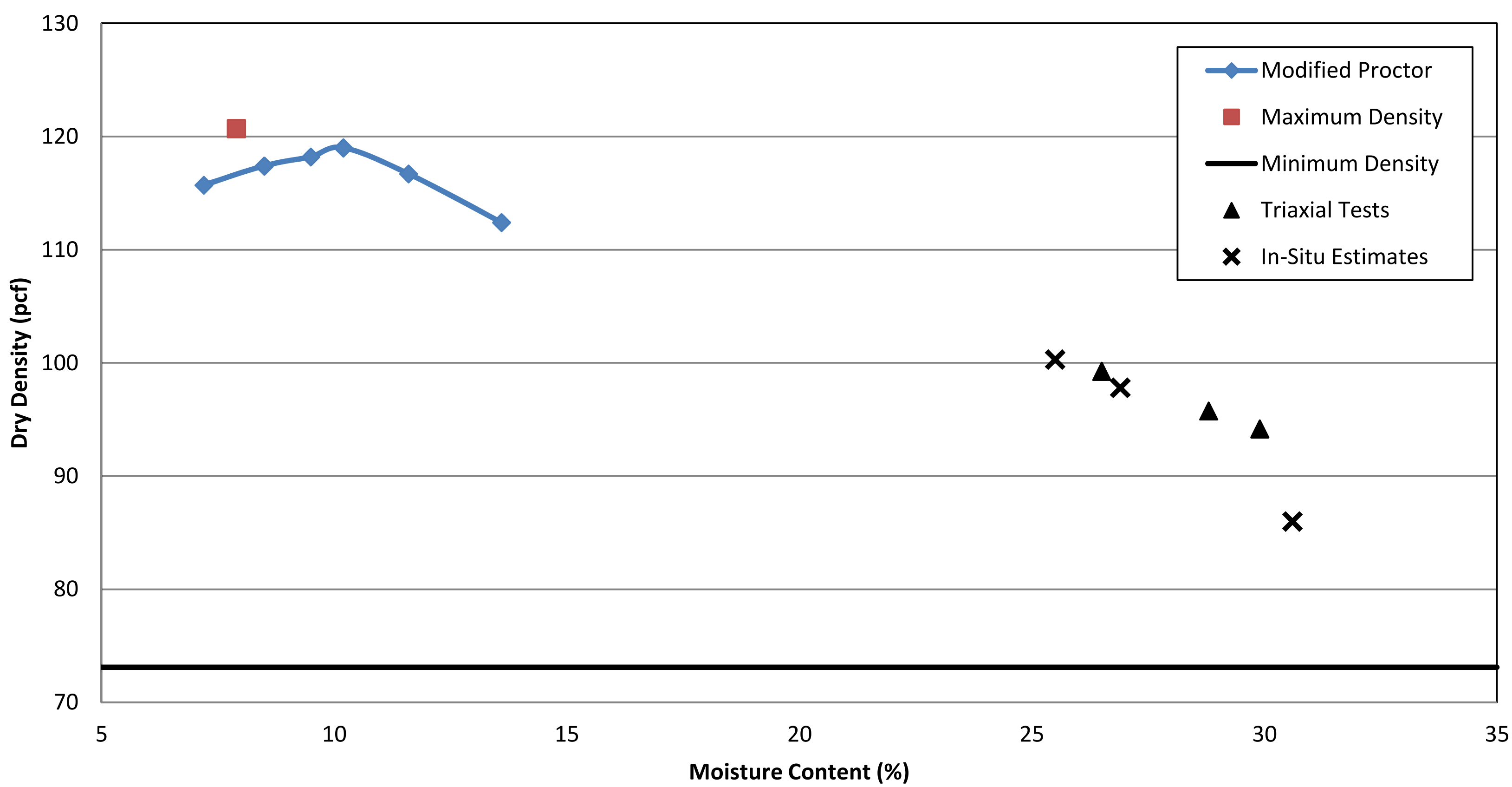
Depth (ft)	Location	$(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{unscaled}$	K_{MSF}	σ_{v0}' (atm)	K_σ for f= 0.7	CRR_{scaled}
44.5	GA-11B-25	7	12	0.130	1.770	2.214	0.788	0.182
55	GA-11B-25	6	11	0.126	1.770	2.516	0.758	0.169
51	GA-11B-24	9	16	0.162	1.770	2.750	0.738	0.212
73.2	GA-11B-24	7	13	0.141	1.770	3.388	0.693	0.173
91	GA-11B-24	7	13	0.143	1.770	3.899	0.665	0.168

Table 18

Factor of Safety Against Liquefaction for the SPT Based Approach

Depth (ft)	Location	CSR_{eq}	CRR_{scaled}	$FS_{liquefaction}$
44.5	GA-11B-25	0.087	0.182	2.08
55	GA-11B-25	0.083	0.169	2.05
51	GA-11B-24	0.074	0.212	2.86
73.2	GA-11B-24	0.067	0.173	2.58
91	GA-11B-24	0.065	0.168	2.57

FIGURES



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Denver, Colorado

TITLE

Comparison of Min Density, Max Density, Modified Proctor and In-situ Density Estimates

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility

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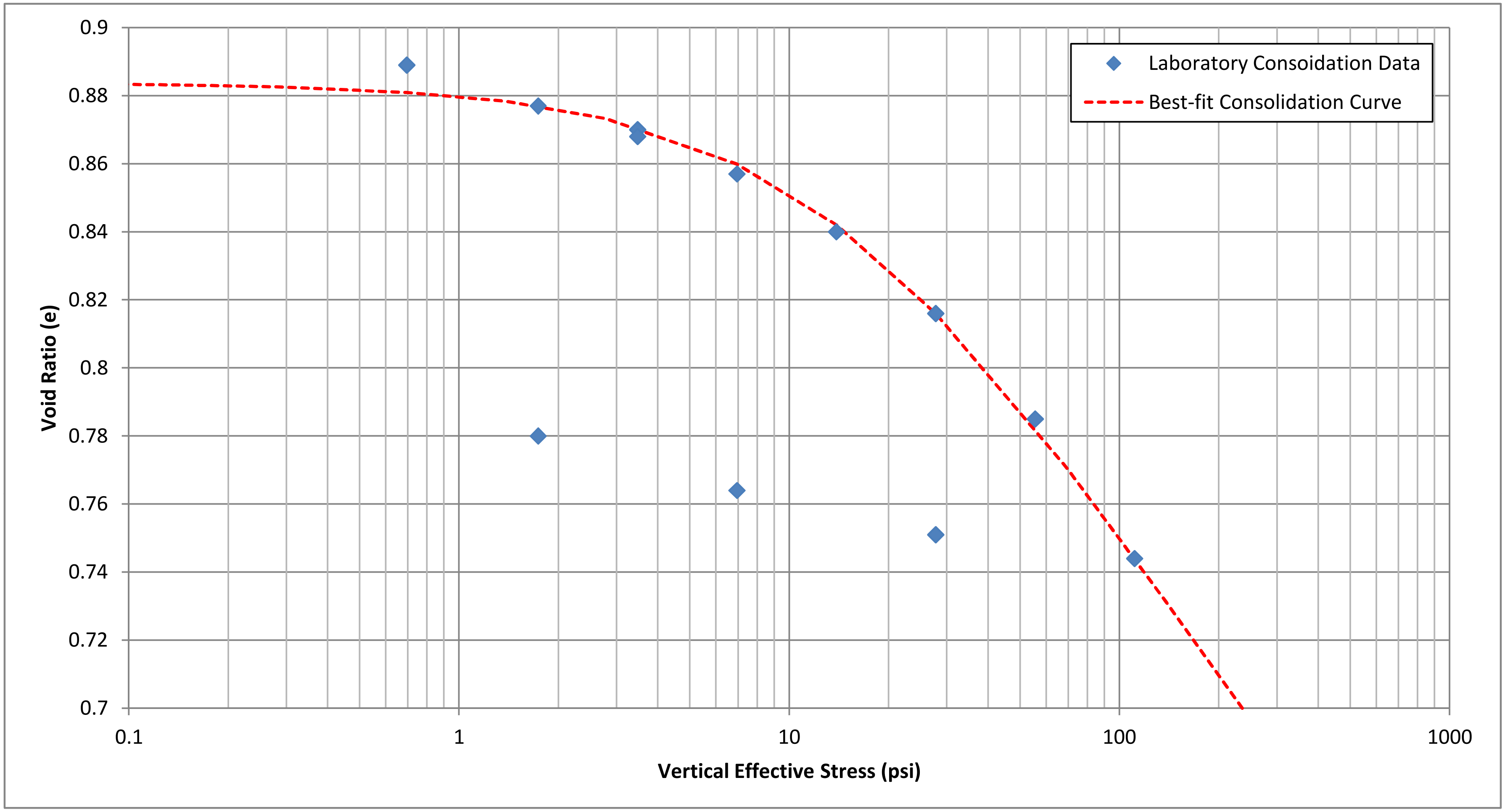

Golder Associates

Denver, Colorado

TITLE

Comparison of One-Dimensional Laboratory Consoidation Data and Best-Fit Consolidation Curve

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility

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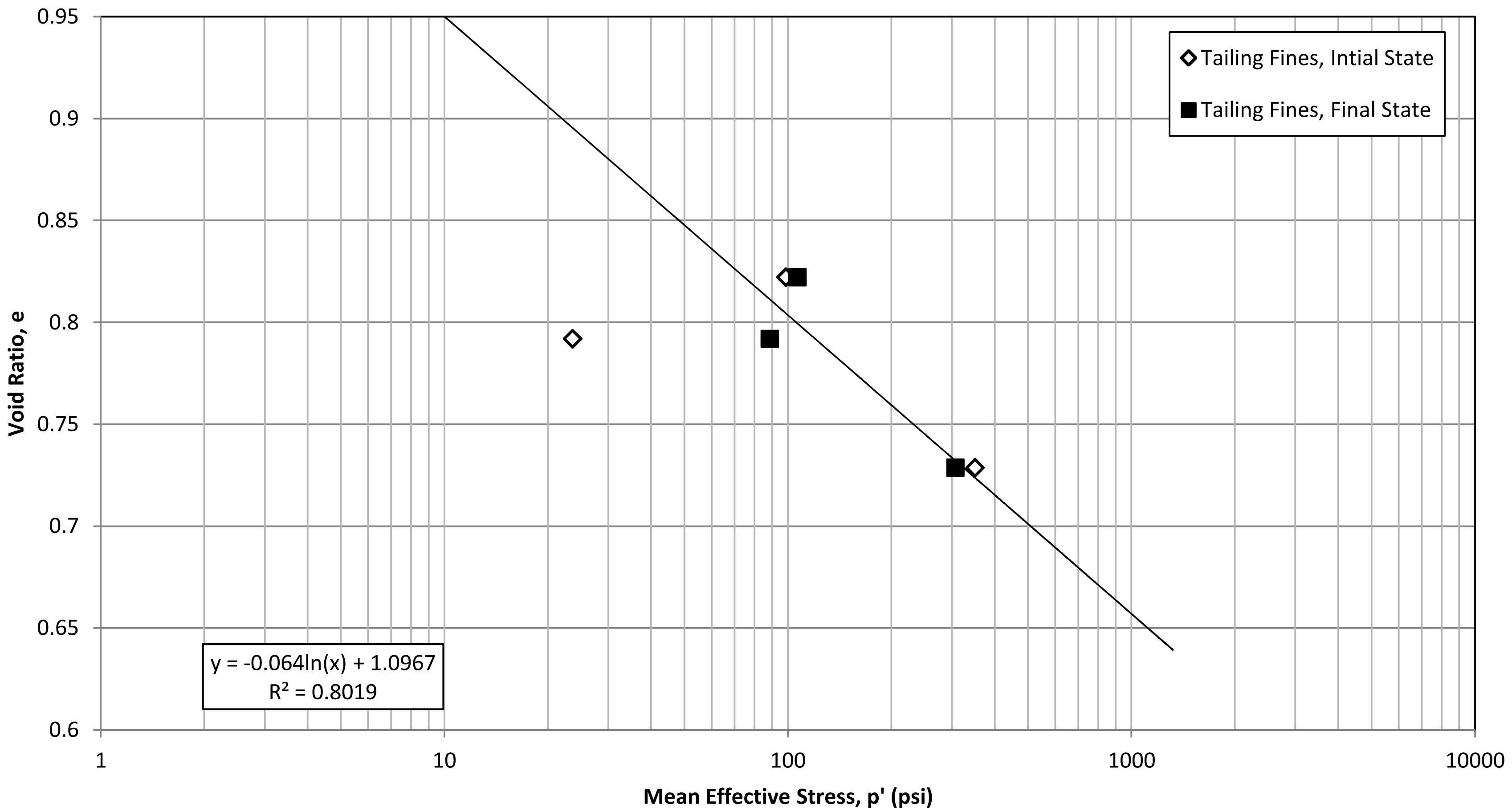

Golder Associates

Denver, Colorado

TITLE

Critical State Line as Determined by CU Triaxial Tests with Pore Pressure Measurements

CLIENT/PROJECT

Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility

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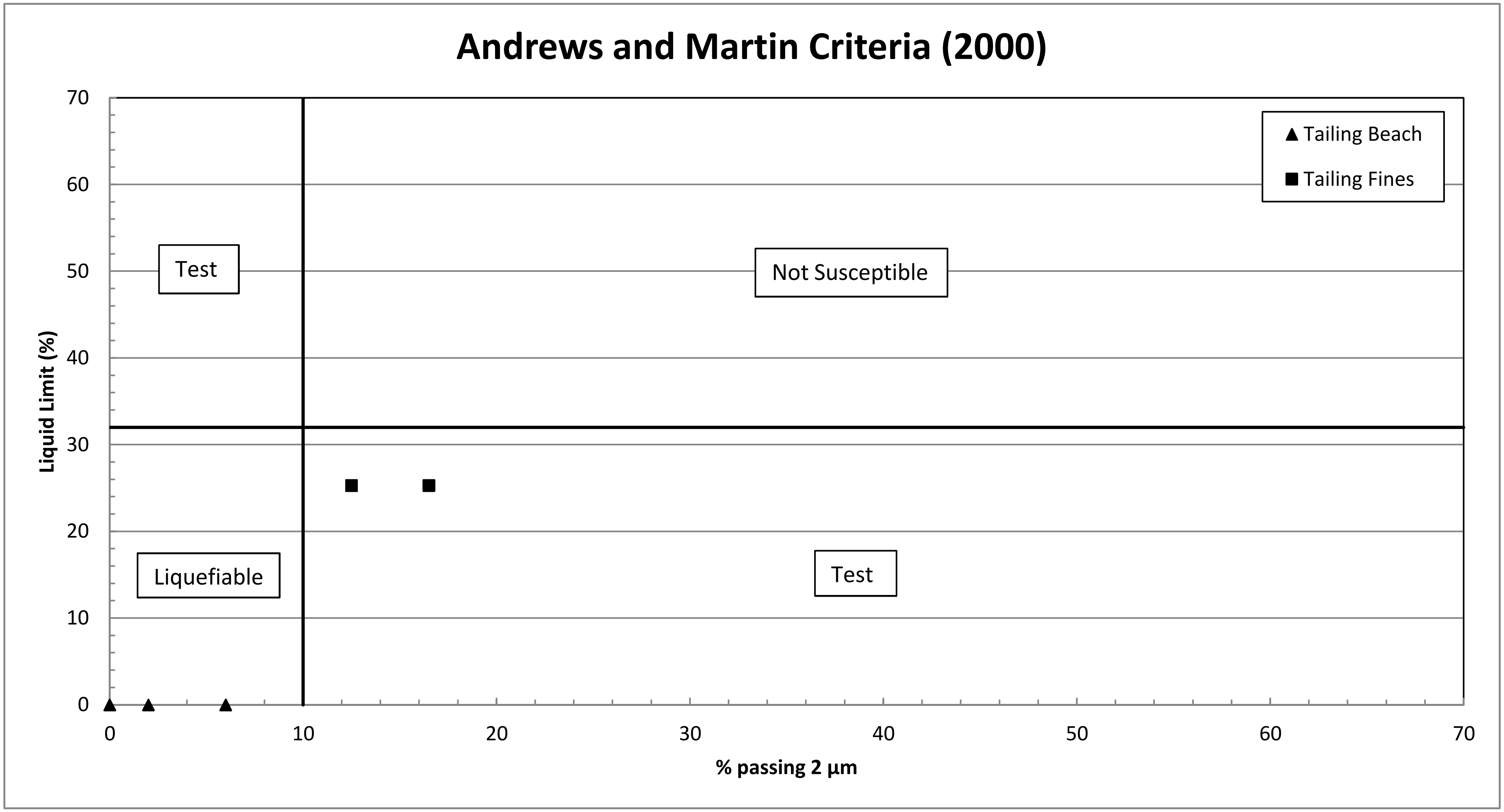

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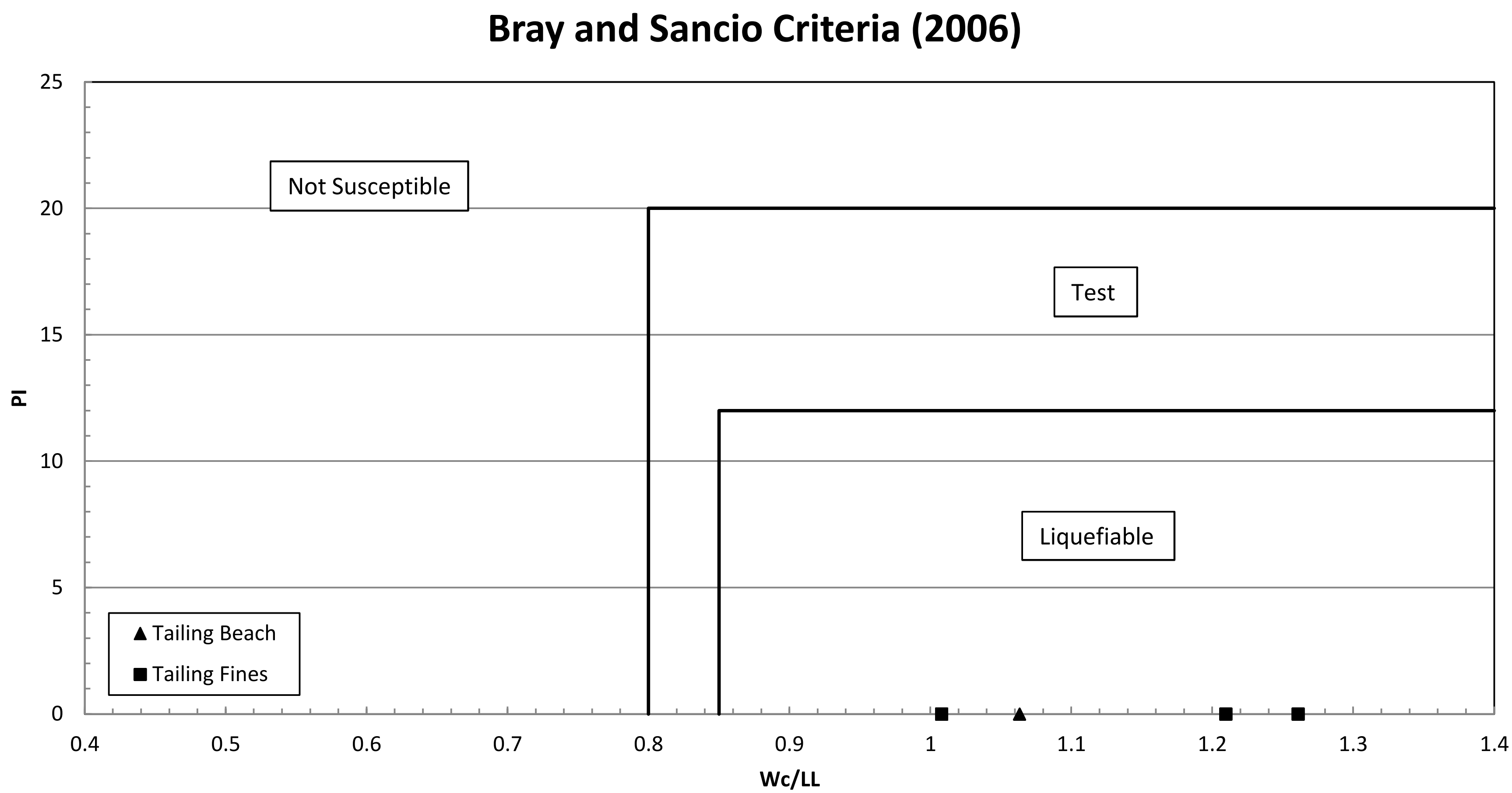

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TITLE **Bray and Sancio Criteria for Liquefaction Susceptibility (2006)**

CLIENT/PROJECT **Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility**

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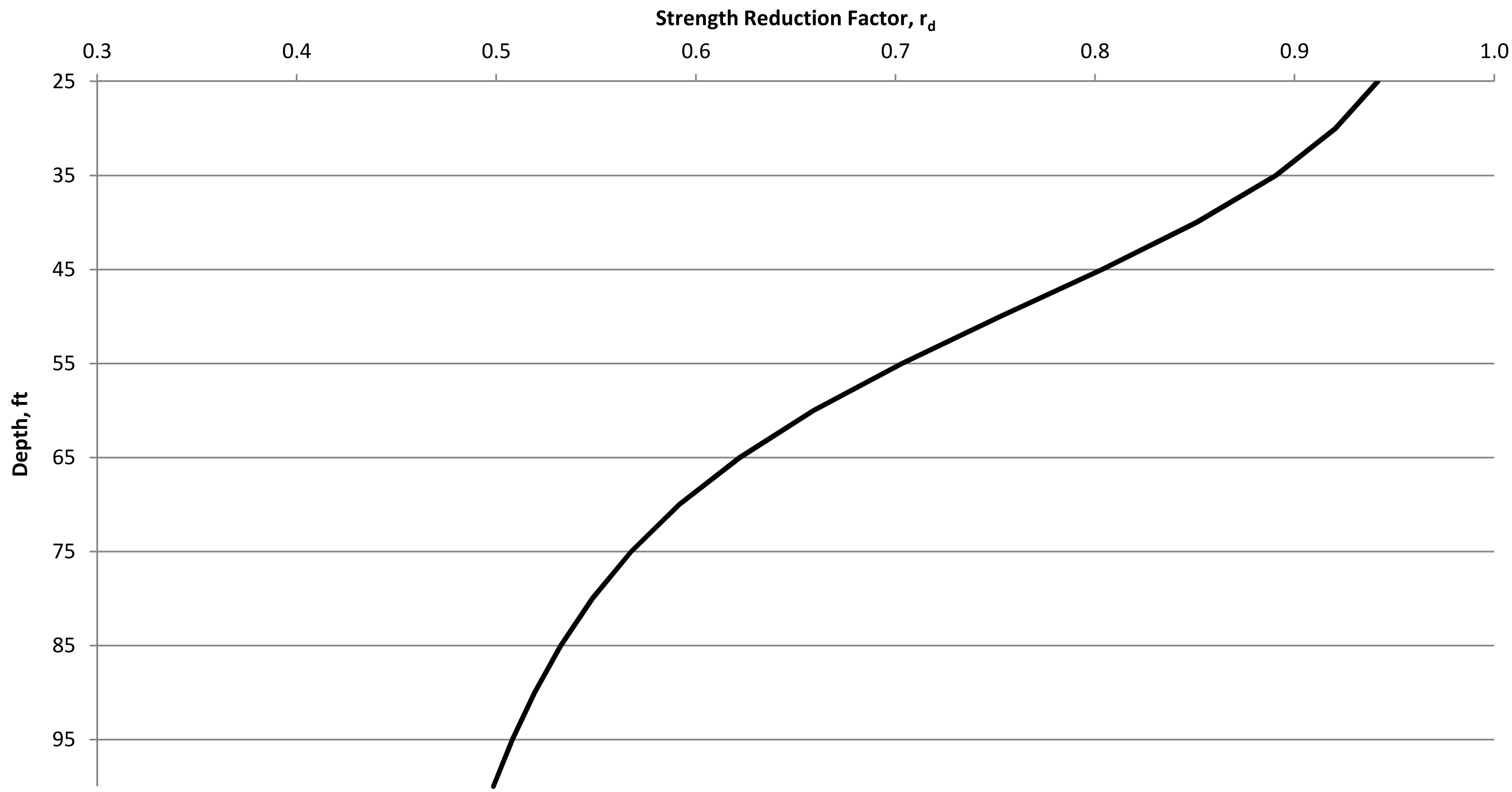

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Denver, Colorado

TITLE **Strength Reduction Factor, r_d with Depth for the Historic Tailing Deposits**

CLIENT/PROJECT **Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility**

DRAWN	DAR	DATE	Mar-12	JOB NO.	113-81608
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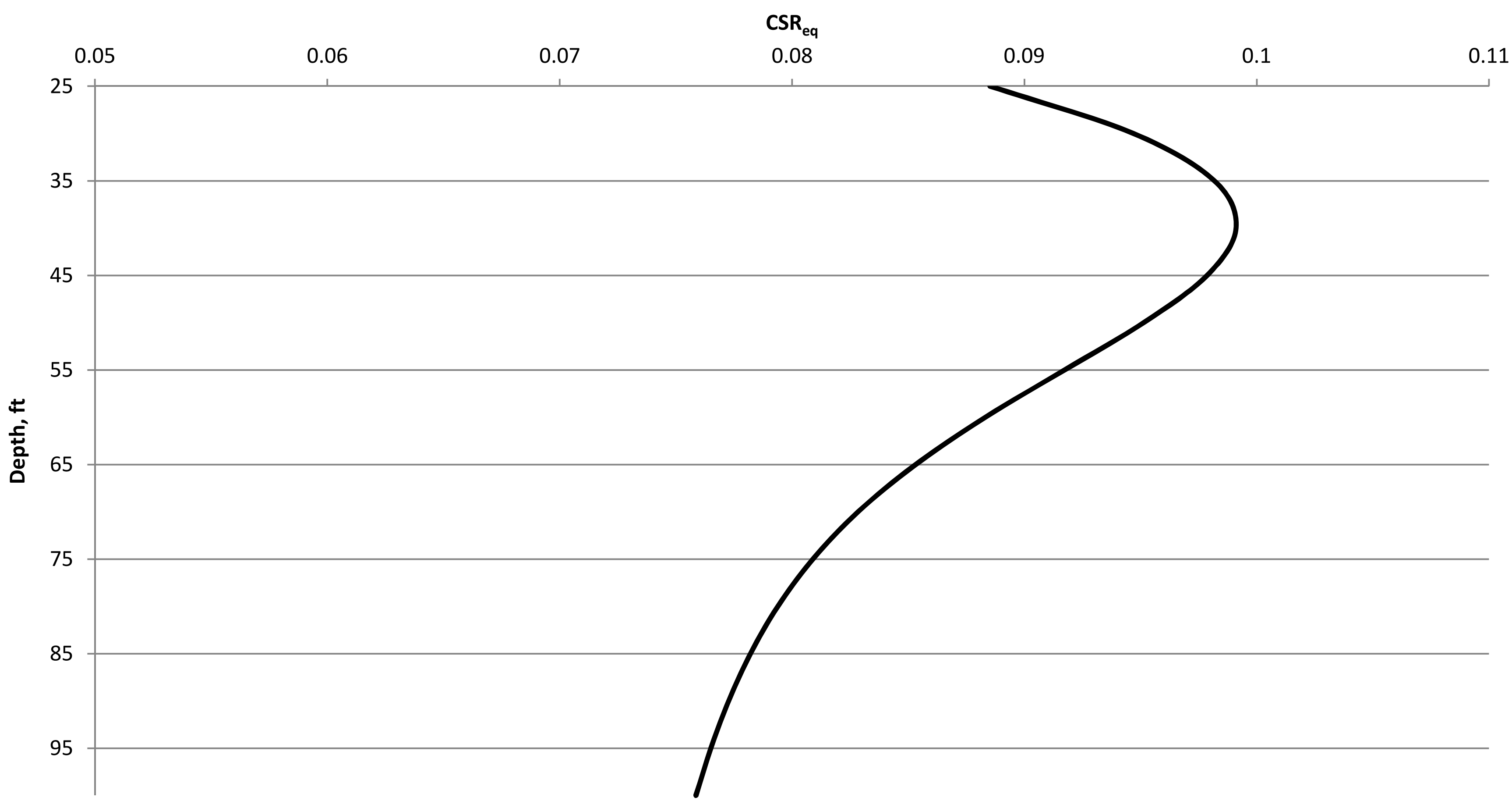

Golder Associates

Denver, Colorado

TITLE **Critical Stress Ratio with Depth for the Historic Tailing Deposits based on the MDE**

CLIENT/PROJECT **Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility**					
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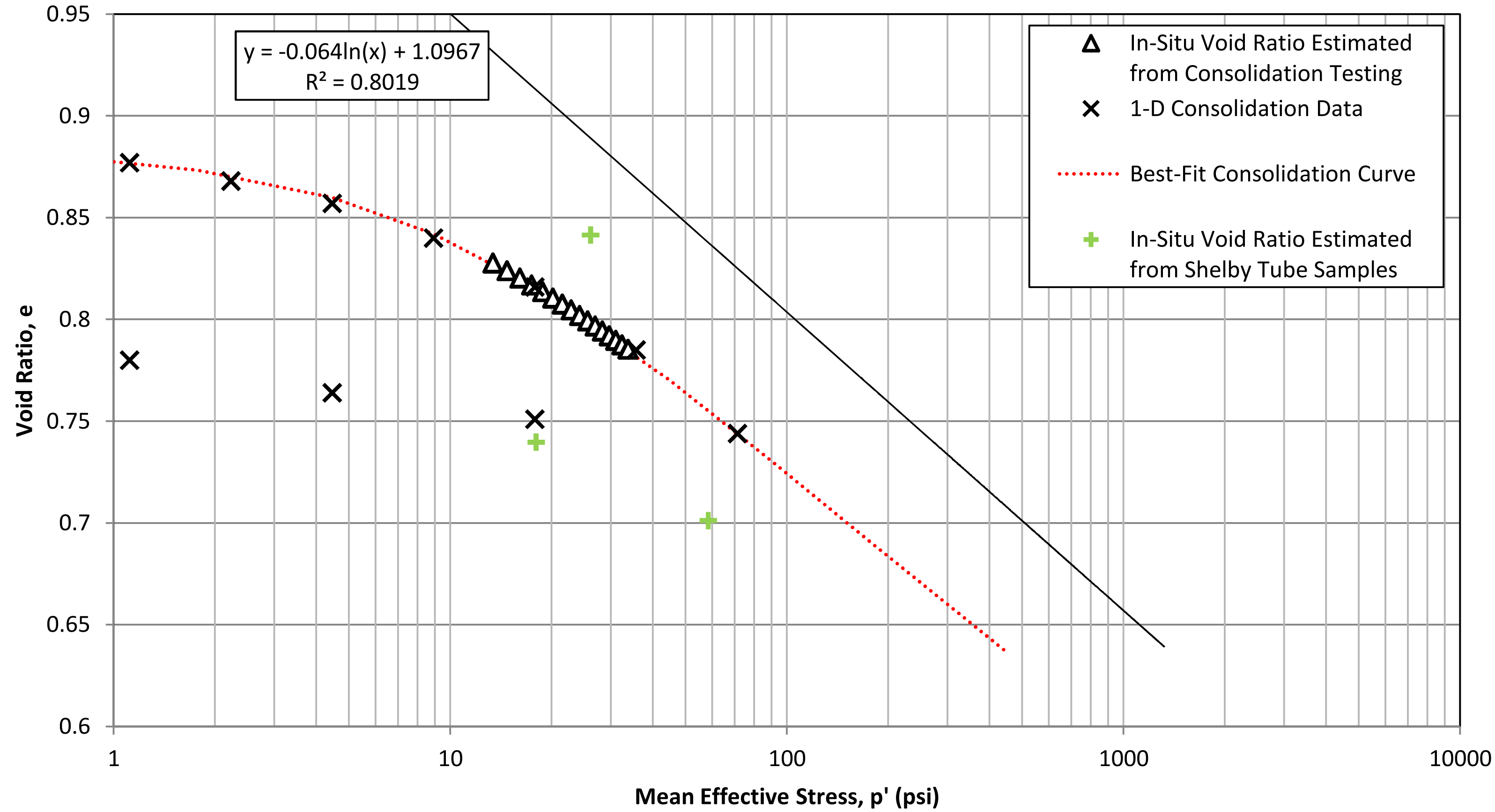

Golder Associates
Denver, Colorado

TITLE
Estimated In-Situ Void Ratios and Effective Stresses in Relation to the CSL

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum
Liquefaction Susceptibility

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Golder Associates
Denver, Colorado

TITLE
Factor of Safety Against Liquefaction Versus Depth for the State Parameter Approach

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility

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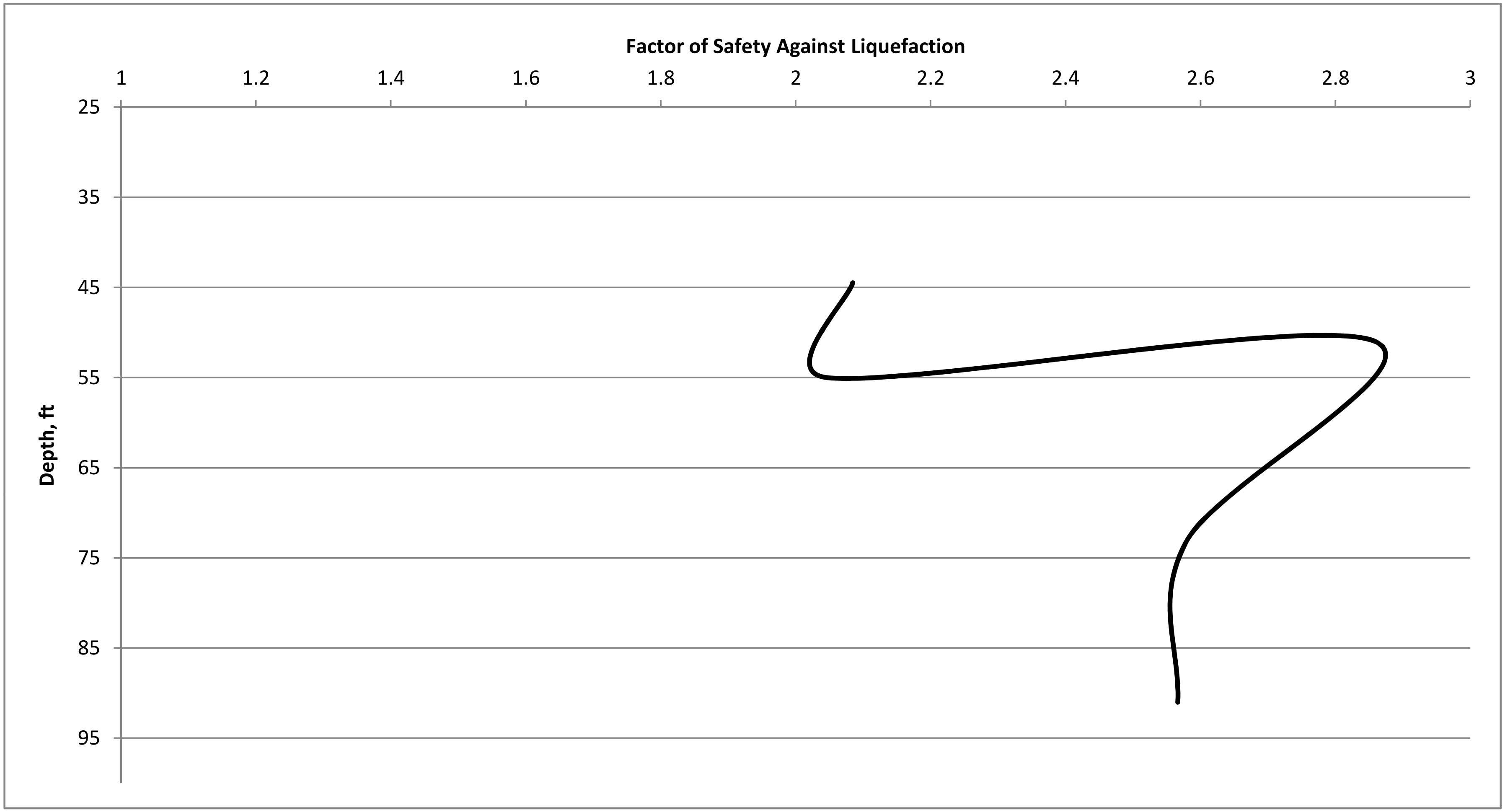

Denver, Colorado

TITLE
Factor of Safety Against Liquefaction Versus Depth for the SPT Based Approach

CLIENT/PROJECT
Freeport McMoRan/Climax Molybdenum Liquefaction Susceptibility

DRAWN	DAR	DATE	Mar-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	n/a
REVIEWED	BRB	FILE NO.		FIGURE NO.	10

APPENDIX E

Date:	May 17, 2012	**Prepared by:**	P. Kaini
Project No.:	113-81608	**Checked by:**	M. Richey
Project Title:	Climax Molybdenum OSF Design Report	**Reviewed by:**	T. Atayee B. Bronson
RE:	**SURFACE WATER CALCULATIONS**		

1.0 OBJECTIVE

This appendix presents the results of the hydrologic and hydraulic calculations performed to evaluate the surface water management system designed for the Climax Molybdenum Mine (Climax) North 40 and McNulty Gulch overburden storage facilities (OSFs). The analyses performed were performed in order to:

- Determine the runoff volumes and discharges generated during operations of the OSFs.
- Determine sizes for proposed bench channels, downdrains, OSF perimeter channels, and temporary operational channels.
- Estimate the closure and post-closure runoff to support development of the closure and post-closure water management strategy.

Throughout this appendix, the term "contact water" refers to precipitation-related water and groundwater that has flowed over, through, or otherwise been in contact with mined materials, including overburden. The term "non-contact" water, as used in this report, refers to precipitation-related water and groundwater that has not been in contact with mined materials. Water may retain its "non-contact" designation after being allowed to flow over or through engineered cover materials or engineered drainage layers derived from non-acid generating sources, as well as undisturbed native material.

2.0 METHOD

This appendix presents the surface water management strategies for two cases: Case 1 (operations) and Case 2 (after closure). Figure 1 shows the drainage basin delineations for the OSFs during operations. Figures 2 and 3 illustrate the conceptual operational and post-closure surface water management layouts, respectively. The hydrologic modeling system (HMS) developed by the U.S. Army Corps of Engineers Hydrologic Engineering Center (HEC) was used to apply the SCS Unit Hydrograph method and simulate runoff hydrographs from which runoff volume and peak flows were determined. (USACE, 2010). To setup the HEC-HMS model, the steps required are:

- Divide areas contributing runoff to the OSFs into smaller sub-basins based on the topography and land cover type.

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- Assign curve numbers (CN) and lag times to each sub-basin based on the land type and the water travel time, respectively, using the method developed by the USDA Soil Conservation Service (SCS)
- Apply a SCS storm distribution. A Type II SCS distribution was used for modeling rainfall at the Climax OSFs.

In all cases, channels were designed to meet or exceed the requirements established by the project design criteria, provided as Appendix F of the design report.

3.0 ASSUMPTIONS & DATA

- Design storm depths (Climax, 2010)

Storm Event	Storm Depth (in)	Used For
10-Year, 24-hour	1.40	Temporary operations channels
100-Year, 24-hour	1.99	Closure channels

For operations (Case 1), the design storm used to design temporary run-on/run-off channels was defined as the 10-year, 24-hour storm event (1.40 inches). For perimeter channels which will be utilized at closure in addition to during operations, the post-closure criteria were used. The post-closure channel design criteria consider both potential rainfall and potential snowmelt. The post-closure design storm was defined to be the less favorable (more severe) of either the 100-year, 24-hour storm event or the 10-year, 24-hour storm event superimposed with the estimated flow produced by the 100-year, 24-hour snowmelt event. Based on the available climate data, the 100-year, 24-hour storm event will produce a larger peak runoff, and therefore was used as the basis for channel design. These storm depths were developed using Log Pearson Type III statistical analysis and rainfall data from the Climax Weather station (Exhibit K, Climax, 2010).

- HEC-HMS model time step is 5 minutes
- NRCS Type II Storm distribution
- Time of concentration is computed using the segment method. Lag time is estimated to be 0.6 times the time of concentration.
- Bench channels, downdrains, and perimeter channels are sized to convey the 100-year, 24-hour storm runoff with a minimum 1 foot of freeboard.
- Minimum bottom width of all channels = 5.0 feet, based on constructability and maintenance considerations.
- Curve numbers (CN):

Land type	Curve Number (CN)	Soil Group
Undisturbed native area	80	C
Waste rock overburden	65	A or B
Disturbed area other than overburden	85	B or C
Covered and revegetated overburden	75	C

- Minimum 1% grade for top surface channels, 2% slope gradient for bench channels, and the slopes for downdrain and perimeter channels are determined based on the OSF design and topography, respectively.
- Revetment for downdrains and energy dissipation structures will be Articulated Concrete Block (ACB).
- Revetment for perimeter channels and water management berms will be riprap where velocities exceed 5 feet per second for the design flow.
- Top surface channels and bench channels are designed with grass lining where flow velocities do not exceed 5 feet per second for the design flow.
- Temporary operational channels will be earthen-lined and will be maintained as necessary following major storm events.

4.0 CALCULATIONS

4.1 Hydrology – Case 1 (Operations)

During operations (Case 1), the entire contributing watershed is divided into 11 sub-basins as shown on Figure 2. Sub-basins PRESUB-1 through PRESUB-4 drain towards the west and presently report to the East Side Channel or ETDL. Runoff from these areas is considered contact water. Runoff from sub-basins PRESUB-5 through PRESUB-8 is presently considered non-contact water. Runoff from PRESUB-6, 7, 8, and 9 presently reports to the base of McNulty Gulch. Non-contact water from PRESUB-7 and 8 and portions of PRESUB-6 is directed to the East Interceptor. The sub-basin PRESUB-7 and PRESUB-8 areas will be reduced as McNulty Gulch is filled in with mine overburden during operations. PRESUB-9 is located within the southern sector of the McNulty Gulch OSF, and incorporates areas that presently contain mine overburden fill. Runoff from PRESUB-9 is considered contact water, and is presently directed to the East Side Channel. Runoff from sub-basins PRESUB-10 and PRESUB-11 will be diverted and contained in the pit. Table 1 presents flows calculated for case 1 (operations).

Table 1: Summary of HEC-HMS Watershed Analysis

Watershed	Watershed Area (ac)	Curve Number (CN)	100-Year Peak Discharge (ft^3/s)	100-Year Runoff Volume (acre-feet)	Runoff Reporting Location
PRESUB-1	109.3	85	98.1	7.2	East Side Channel
PRESUB-2	182.3	83	111.6	10.5	East Side Channel
PRESUB-3	78.7	81	43.6	3.7	East Side Channel
PRESUB-4&11*	192.3	83	112.2	8.9	East Side Channel
PRESUB-5	104.4	80	35.7	4.8	East Side Channel
PRESUB-6	120.4	80	70.6	5.6	East Interceptor
PRESUB-7	113.1	77	42.4	4.2	East Interceptor
PRESUB-8	126.6	77	46.4	4.7	East Interceptor
PRESUB-9&10*	366.8	80	184.5	14.6	McNulty OSFs toe

*Sub-basins are combined

The detailed HEC-HMS output for Case 1 is presented in the Attachment 1.

4.2 Hydrology – Case 2 (Closure)

Water management post-closure (Case 2) will be maintained through a network of surface water channels and other systems. The conceptual closure grading and water management plan (illustrated on Drawing 4 in the Design Report) provides for a “barber-pole” channel layout, e.g., top-surface and outslope channels that will convey runoff to steeper downdrains and then to perimeter channels and/or water management berms. While storm water falling on the OSF during operations is considered contact water, storm water falling on the OSF post-closure will remain non-contact water due to the cover system. As shown on Drawing 4 the post-closure surface water network will include:

- Top surface channels will be constructed to collect and convey storm water runoff from the top surfaces of the OSF and direct it to downdrains.
- Channels will be constructed in the 20 ft wide benches on the regraded OSF outslopes to convey runoff from the reclaimed interbench slopes to the downdrains.
- Downdrain channels will collect water from the top surface and outslope bench channels and transmit the water down the OSF outslopes to energy dissipaters, located at the toe of the OSF, and then on to perimeter channels and/or water management berms.
- Perimeter channels and/or water management berms will collect water from downdrains and convey the water along the toe of the OSF to the East Interceptor channel or other conveyance, where the non-contact water will be discharged, eventually flowing north to the Clinton Reservoir.
- A toe drain network is expected to be constructed at closure along the toe of the OSFs in areas where water internal to the OSFs has potential to exit the OSFs as seepage near the toe of the slope. The toe drain network will collect this contact water and transmit it to the ETDL.
- The underdrain system will remain in service at closure to convey flows from springs within the McNulty Gulch OSF footprint, and transmit them to the toe of the slope while

maintaining separation between the collected water and the overlying mine overburden fill.

The HEC-HMS model was used to calculate peak flows and flow volumes at various outlets for Case 2. Peak flows at the bottom of the downdrains were calculated by routing flows through the benches, top surfaces, and downdrains. Routing was performed using a kinematic wave routing method assuming trapezoidal channel sections. Flows through various channels and along various water management berms are summarized in Tables 2 and 3.

Table 2: Downdrain Flows

Downdrains	Drainage area (ac)	Channel length (ft)	Peak flow (ft^3/s)	Volume (ac-ft)
DS-1	41.7	881.3	15	1.3
DS-2	35.5	1521.0	13	1.1
DS-3	125.1	1521.0	44	3.9
DS-4	44.4	592.8	16	1.4
DS-5	56.9	1110.9	20	1.8
DS-6	46.2	1518.7	17	1.4
DS-7	44.5	1084.9	15	1.4
DS-8	20.0	348.5	7	0.6
DS-9	40.3	315.4	14	1.3
DS-10	32.8	206.4	11	1.0

Table 3: Flow through Perimeter Channels or Along Water Management Berms

Perimeter Channel	Length (ft)	Slope	Peak flow (ft^3/s)	Flow type
CC-1	4844	15%	24	Non-contact
CC-2[1]	2626	17%	N/A	Non-contact
CC-3	2449	16%	16	Non-contact
CC-4	5495	2%	56	Non-contact
CC-5	8130	2%	50	Non-contact
CC-7	4901	14%	31	Non-contact
CC-8	1492	17%	63	Non-contact

[1] Downdrain and/or bench channels are not contributing to this channel

4.3 Channel Design

HEC-HMS was also used to calculate peak discharges for the proposed water management features for Case 1 and Case 2 (i.e., perimeter channels, water management berms, bench channels and downdrains), with the designs completed in accordance with the design criteria. Input parameters for design (flow depth, velocity, etc.) were calculated using Manning's normal depth equations. Perimeter channels and water management berms are designed with riprap revetment due to high flow velocities (> 5 ft/s). Bench channels and top surface channels are designed with grass lining and bed slopes of 2% and 1% respectively; flow velocity is maintained below 5 feet per second (ft/s) for both cases. Articulated Concrete Block (ACB) was used to armor all downdrains and energy dissipaters. Tables 4 and 5 summarize the proposed designs for operations and at closure, respectively.

Table 4: Channels and Water Management Berms for OSF during Operations

Channel / Berm Description	Slope (%)	Revetment	Manning's n[1]	Design Flow (ft^3/s)	Flow Depth (ft)	Flow Velocity (ft/s)	Flow Type
OC-1	20.0%	Riprap	0.04	88	0.91	13.8	Non-contact
OC-3**	17.0%	Riprap	0.04	25	0.48	8.9	Contact
OC-4	16.0%	Riprap	0.04	34	0.58	9.6	Contact
OC-5	2.0%	Riprap	0.04	203	2.43	7.5	Contact

**20% of PRESUB-4 is assumed to contribute; [1] Manning's n for capacity (depth)

OC-6 is a temporary operational channel designed to convey contact flow from PRESUB-9 and PRESUB-10. This temporary channel will be continuously shifted towards the west as the OSF work progresses, and therefore it is expected to be constructed with earthen lining. Temporary rock dams may be placed to reduce channel grade and reduce flow velocity, which will reduce channel erosion. This channel will be maintained and repaired by Climax as required following major storms.

Table 5: Channels and Water Management Berms for OSF at Closure

Channel Description	Channel Slope (%)	Channel Lining	Manning's n[1]	Design Flow (ft^3/s)	Flow Depth (ft)	Flow Velocity (ft/s)	Flow Type
DS-1	50.0%	ACB	0.026	15	0.20	12.9	Non-contact
DS-2	50.0%	ACB	0.026	13	0.19	12.3	Non-contact
DS-3	50.0%	ACB	0.026	44	0.38	18.7	Non-contact
DS-5	50.0%	ACB	0.026	16	0.21	13.3	Non-contact
DS-6	50.0%	ACB	0.026	20	0.24	14.5	Non-contact
DS-7	50.0%	ACB	0.026	17	0.22	13.5	Non-contact
DS-4	50.0%	ACB	0.026	15	0.21	13.2	Non-contact
DS-8	50.0%	ACB	0.026	7	0.13	9.9	Non-contact
DS-9	50.0%	ACB	0.026	14	0.20	12.7	Non-contact
DS-10	50.0%	ACB	0.026	11	0.17	11.8	Non-contact
Typical bench channel	2.0%	Grass lined	0.035	3	0.15	1.8	Non-contact

Channel Description	Channel Slope (%)	Channel Lining	Manning's n[1]	Design Flow (ft^3/s)	Flow Depth (ft)	Flow Velocity (ft/s)	Flow Type
Typical top surface channel	2.0%	Grass lined	0.035	12	0.65	3.0	Non-contact
CC-1	15.0%	Riprap	0.04	24	0.48	8.4	Non-contact
CC-2	17.0%	Riprap	0.04	36	0.59	10.0	Non-contact
CC-3	16.0%	Riprap	0.04	16	0.38	7.5	Non-contact
CC-4	2.0%	Riprap	0.04	56	1.30	5.3	Non-contact
CC-5	2.0%	Riprap	0.04	50	1.23	5.2	Non-contact
CC-7	14.0%	Riprap	0.04	31	0.57	8.9	Non-contact
CC-8	17.0%	Riprap	0.04	63	0.79	11.8	Non-contact

[1] Manning's n for capacity (depth)

Note: All channels are designed as trapezoidal with 3:1 (H:V) side slopes and 5 ft of bottom width.

4.4 Closure Top Surface and Bench Channels

Top surface closure channels were evaluated in support of the closure strategy. The conceptual layout used for the evaluation is shown on Figure 3. The actual closure channels will need to be designed to be consistent with the final OSF build out configuration. Peak flow on these conceptual channels varied from 1.0 ft^3/s to 11.0 ft^3/s for the given design storm (i.e., 100-year 24-hr flow) and therefore. A typical section is designed as shown on Drawing 11 of the main report for the maximum flow of 2.5 ft^3/s and bed slope of 2%. A spreadsheet calculation is presented in Attachment 1.

4.5 Energy Dissipation

Energy dissipators have been designed for use at the toe of downdrains for energy dissipation. Calculations were performed using hydraulic jump equations (i.e., relations among conjugate depth, Froude's number, and stilling basin length, see Attachment 2). Table 6 provides dimensions computed at these locations. These dimensions were used to verify that the final operations configuration of the OSFs is compatible for closure. These dimensions were developed to contain the anticipated hydraulic jump dimensions (jump length and conjugate depth) that may be expected at the toe of the channels. The energy dissipator at the DS1-OUTLET will also dissipate energy of the flow from channels CC-1 and CC-2. Some perimeter channels have higher slopes and therefore will also require energy dissipators. Locations of those structures are shown on Figure 3.

Table 6: Stilling Basin Design Summary

Downdrains	Stilling Basin Dimensions	
	Minimum Depth (ft)	Minimum Length (ft)
DS1-OUTLET	1.3	8
DS2-OUTLET	1.2	8
DS3-OUTLET	2.7	17
DS4-OUTLET	1.7	10
DS5-OUTLET	1.5	9
DS6-OUTLET	1.4	9
DS7-OUTLET	1.4	9
DS8-OUTLET	0.8	5
DS9-OUTLET	1.3	8
DS10-OUTLET	1.2	7

The detailed calculations for energy dissipator and stilling basin design are in Attachment 2.

5.0 CONCLUSIONS

A hydrologic analysis was performed by using HEC-HMS model for two cases, Case 1 during operations and Case 2 after closure of the North 40 and McNulty Gulch OSFs. During operations and post-closure, non-contact water will be separated from contact water. Bench channels, top surface channels, downdrains, perimeter channels, water management berms, and temporary operations channels have been designed to support the closure strategy. At the toe of all the downdrains, energy dissipaters are designed to dissipate the high kinetic energy. All downdrains and energy dissipaters will be ACB-armored, perimeter channels and water management berms will be riprap-armored, and bench channels and top-surface channels will be grassed-lined. Temporary operational channels will be earthen-lined and will be maintained as necessary following major storm events. All channels designed meet the requirements established by the project design criteria (Appendix F of the design report).

6.0 REFERENCES

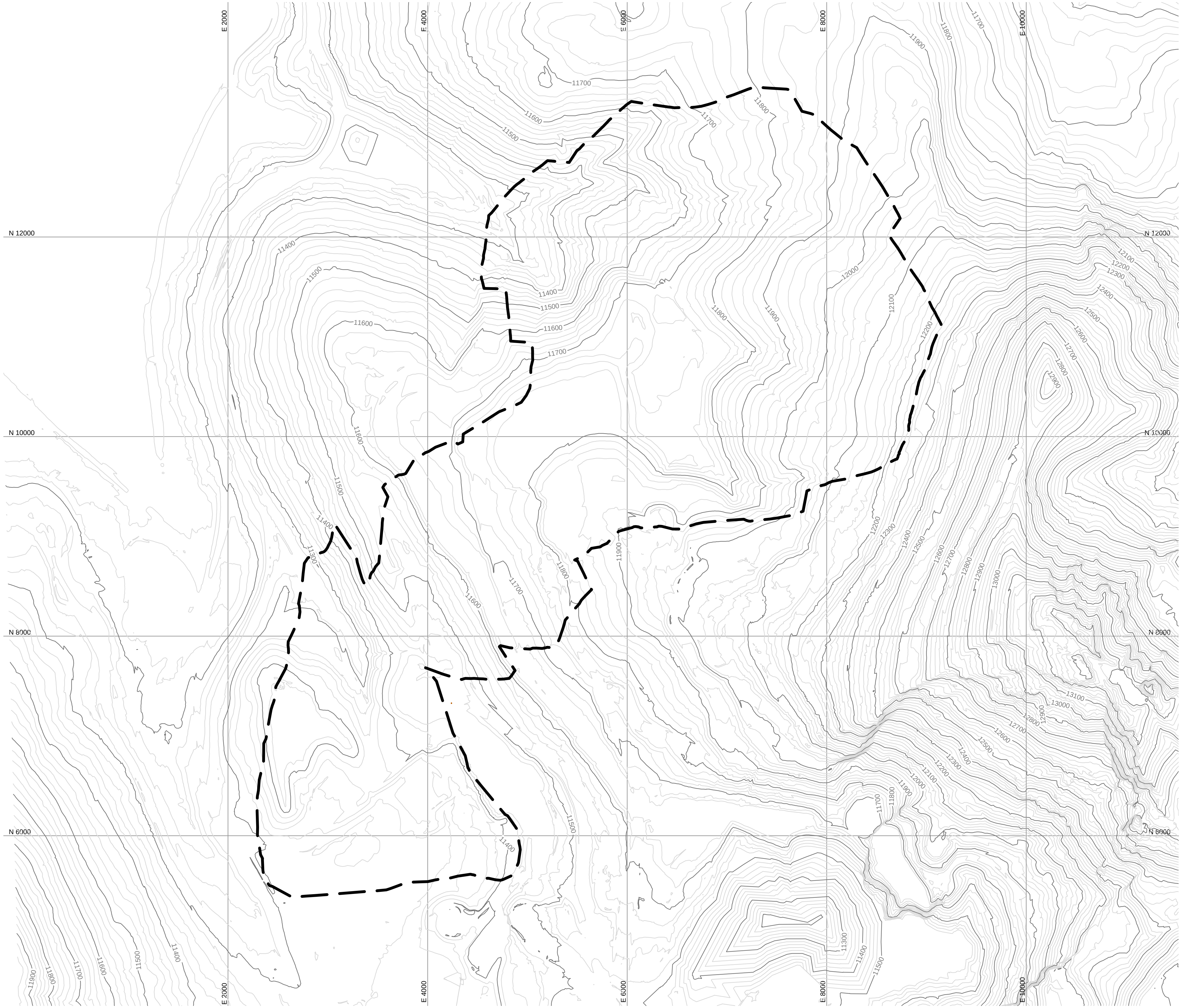
United States Army Corps of Engineers (USACE). 2010. Hydraulic Engineering Center Hydrologic Modeling System (HEC-HMS). Version 3.5. August 2010.

U.S. Soil Conservation Service (USSCS). 1986. *Urban hydrology for small watersheds, 2nd edition* (USSCS Technical Release Number 55). Washington D.C.: United States Department of Agriculture.

Climax. 2010. Climax Molybdenum Company Permit M-1977-493, Exhibit K—Climate Information.

FIGURES

Drawing File: N:\11\113-81608_ Surface Water Support Figures_REV3\ 11381608E001.dwg | Layout: FIGURE 1 | Modified: May 30, 2012 11:22 DGEER | Plotted: 05/30/12 11:22am DGEER



LEGEND

11500 — EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)

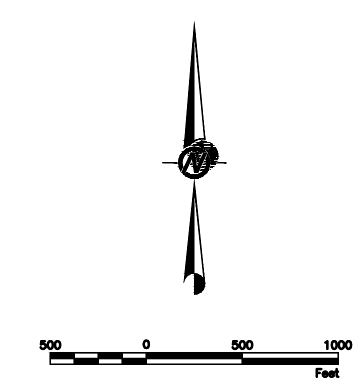
FACILITY BOUNDARY

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.



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PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

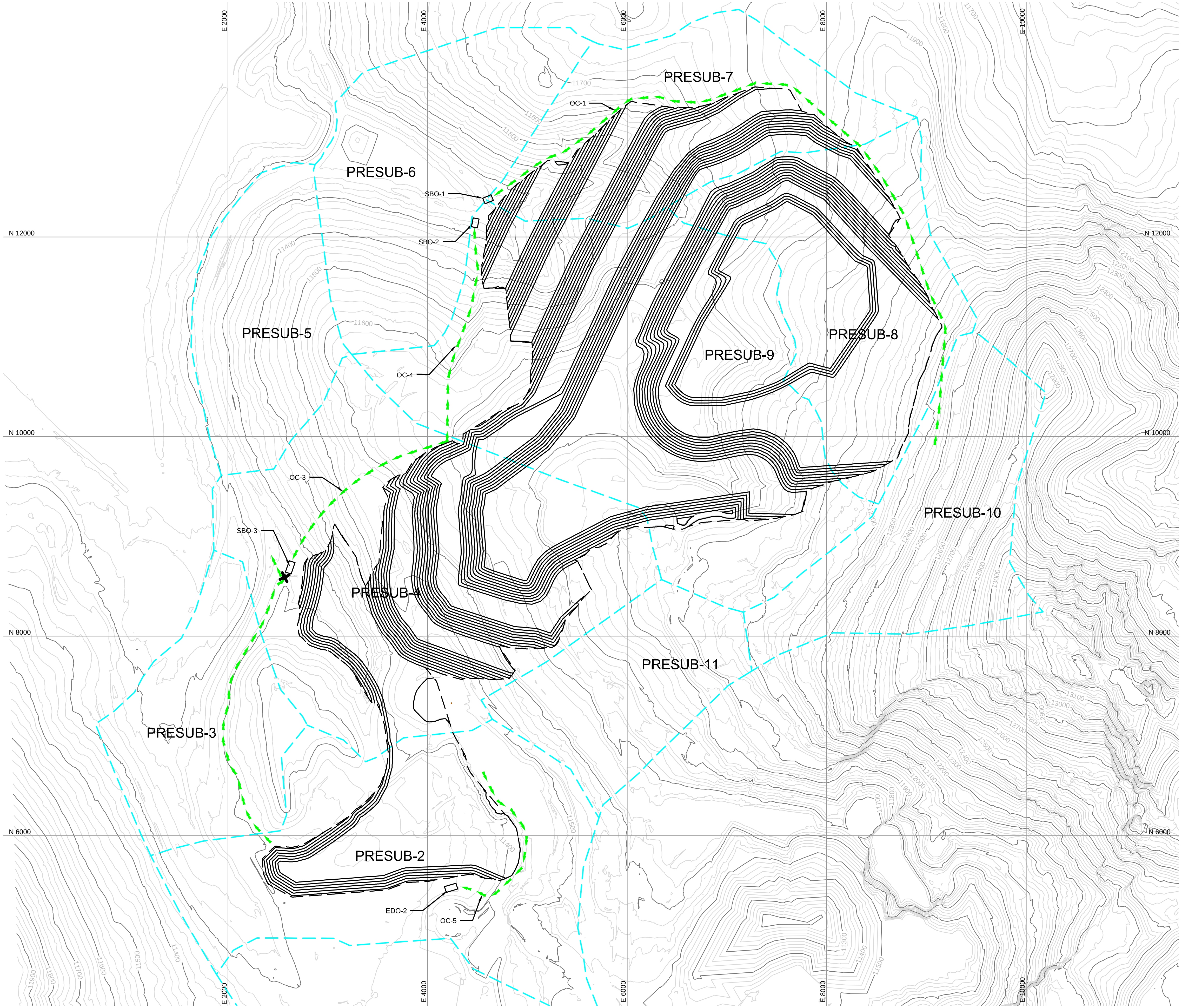
EXISTING CONDITIONS

Golder Associates

FILE No. 11381608E001

PROJECT No. 113-81608

FIGURE 1



LEGEND

11500 — EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)

OPERATIONS GRADING

PERIMETER CHANNEL OR WATER MANAGEMENT BERM

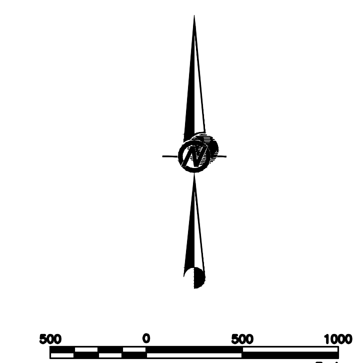
SUB-BASIN DELINEATION

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.



REV.	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	04/03/12	ISSUED WITH DESIGN REPORT	HNL	HNL	PK	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

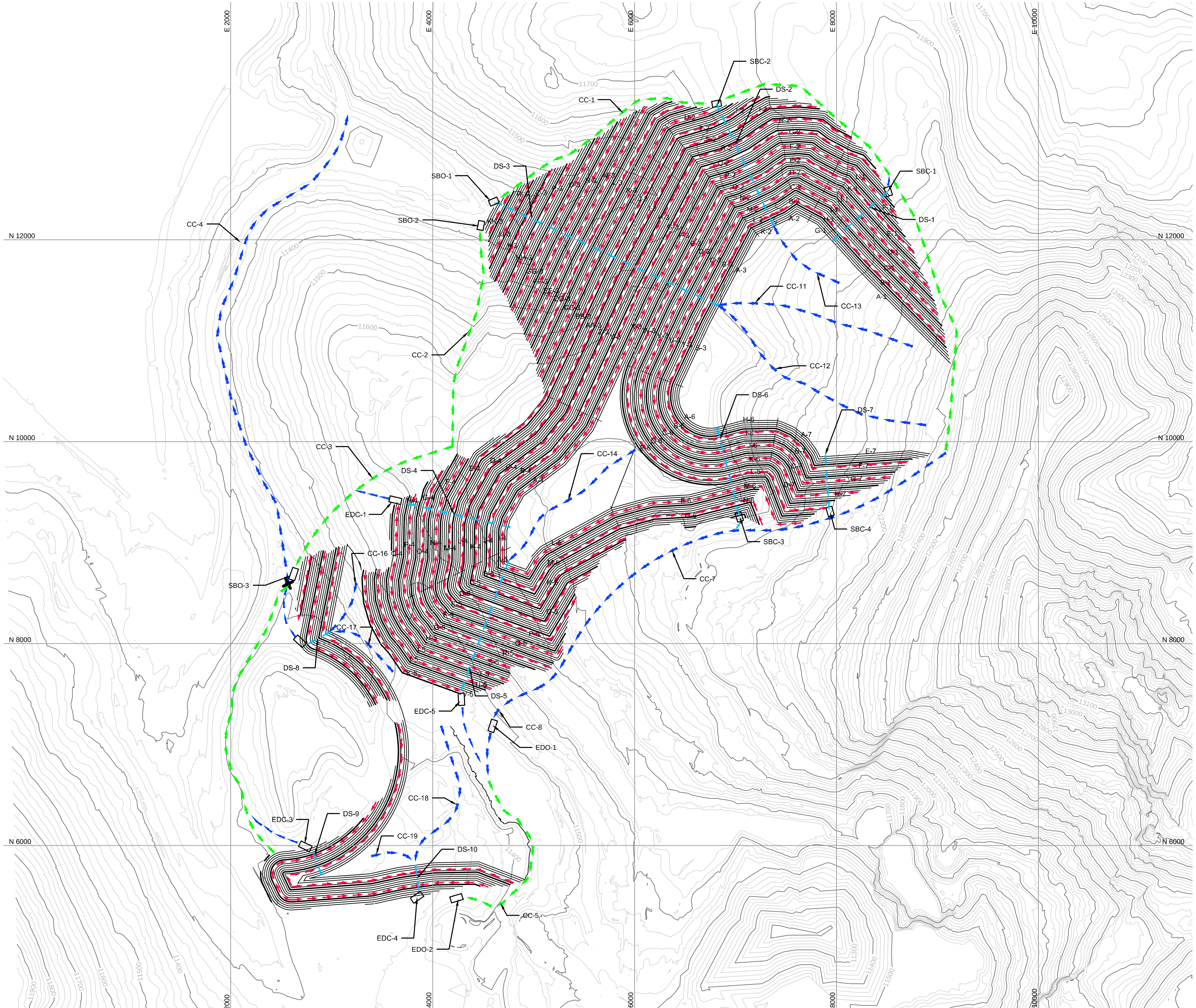
TITLE

OPERATIONS BASIN DELINEATION MAP

Golder Associates

FILE No.	11381608E002
PROJECT No.	113–81608

FIGURE 2



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- 11500 — CLOSURE GRADING
- PROPOSED DOWNDRAIN CHANNEL TO BE CONSTRUCTED AT CLOSURE
- FLOW DIRECTION TRANSITION
- PROPOSED PERIMETER/TOP SURFACE CHANNEL OR WATER MANAGEMENT BERM TO BE CONSTRUCTED AT CLOSURE
- OPERATIONS CHANNEL OR WATER MANAGEMENT BERMS TO BE CONVERTED TO A CLOSURE CHANNEL OR BERM
- PROPOSED OUTSLOPE CHANNELS TO BE CONSTRUCTED AT CLOSURE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.

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PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

CLOSURE
BASIN DELINEATION MAP

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FILE No.	11381608E003
PROJECT No.	113-81608

FIGURE 3

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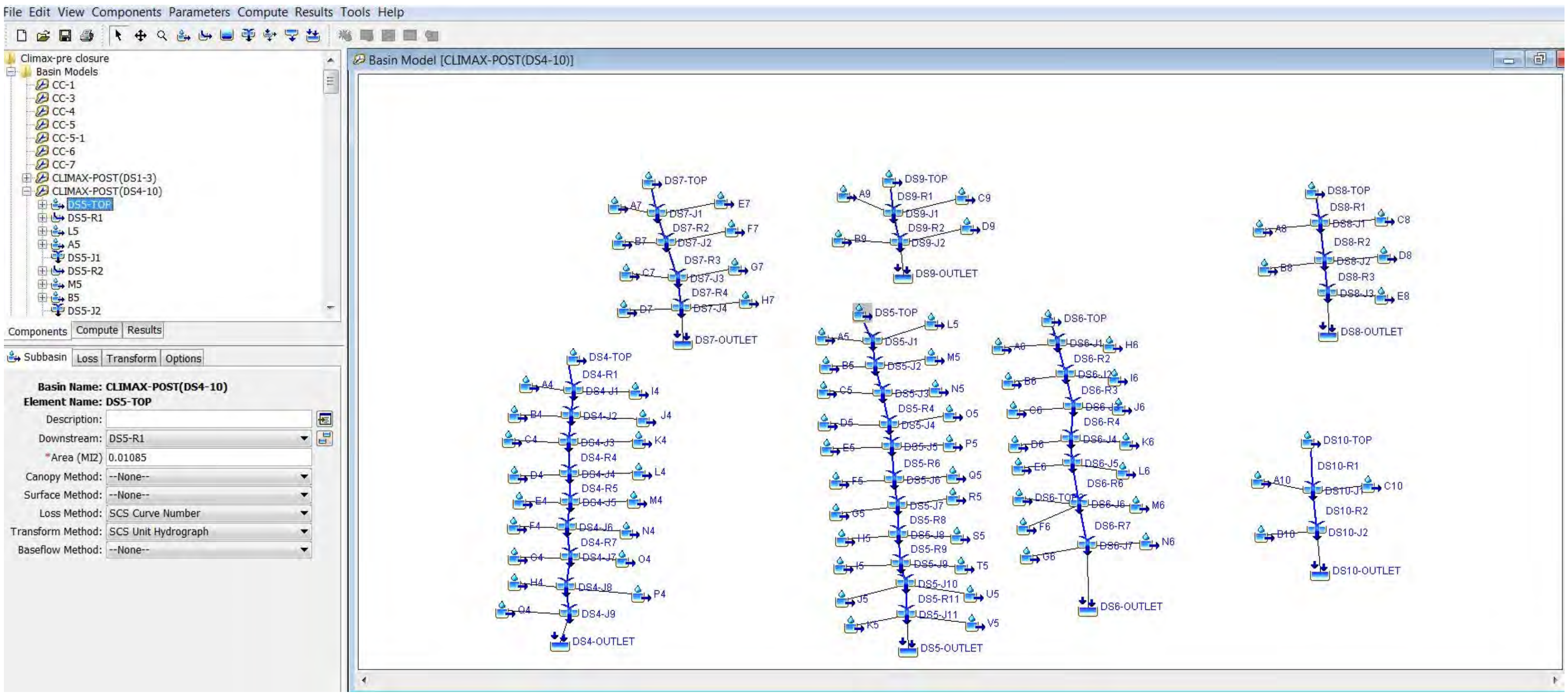
ATTACHMENT 1
HYDROLOGY AND HYDRAULIC CALCULATIONS
PRE-CLOSURE (OPERATIONS) AND AT CLOSURE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CLIMAX-POST(DS1-3)
CLIMAX-POST(DS4-10)
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DS5-R1
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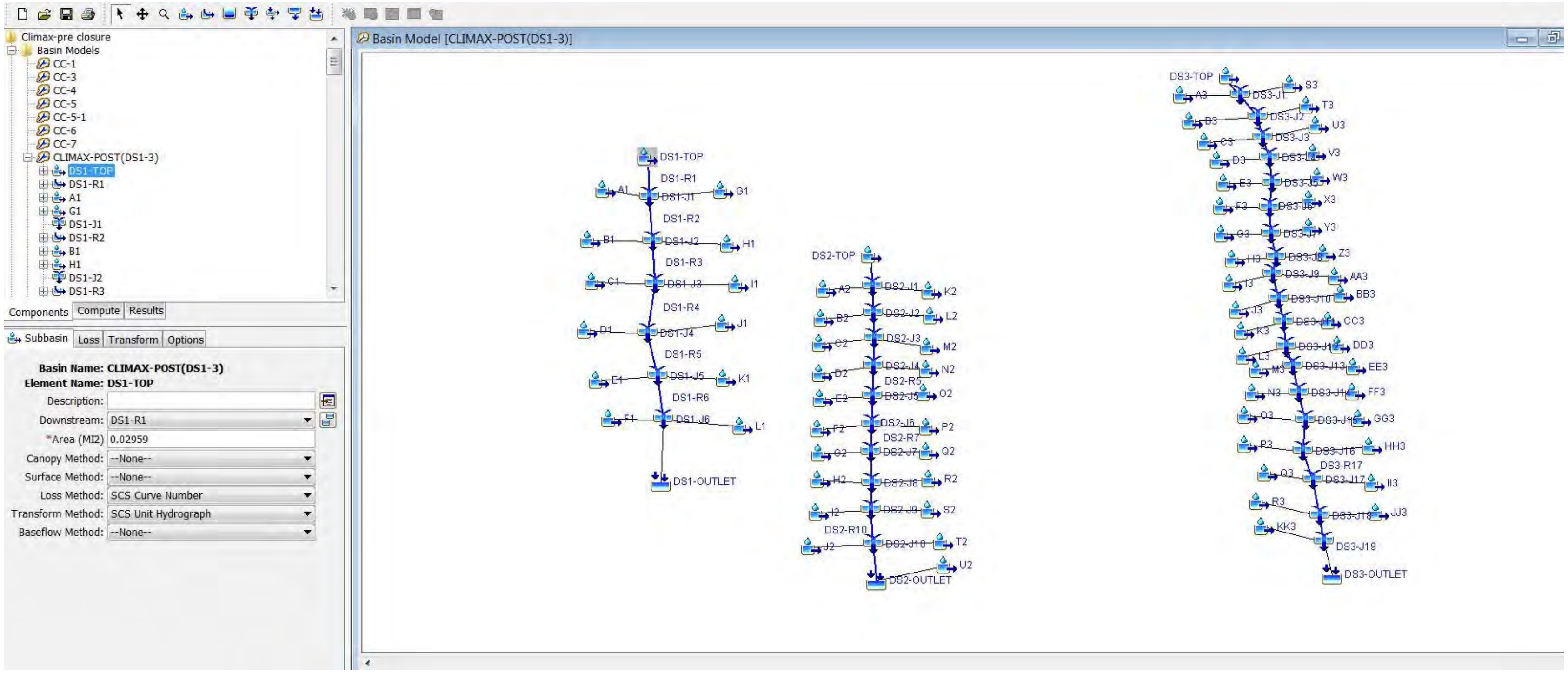
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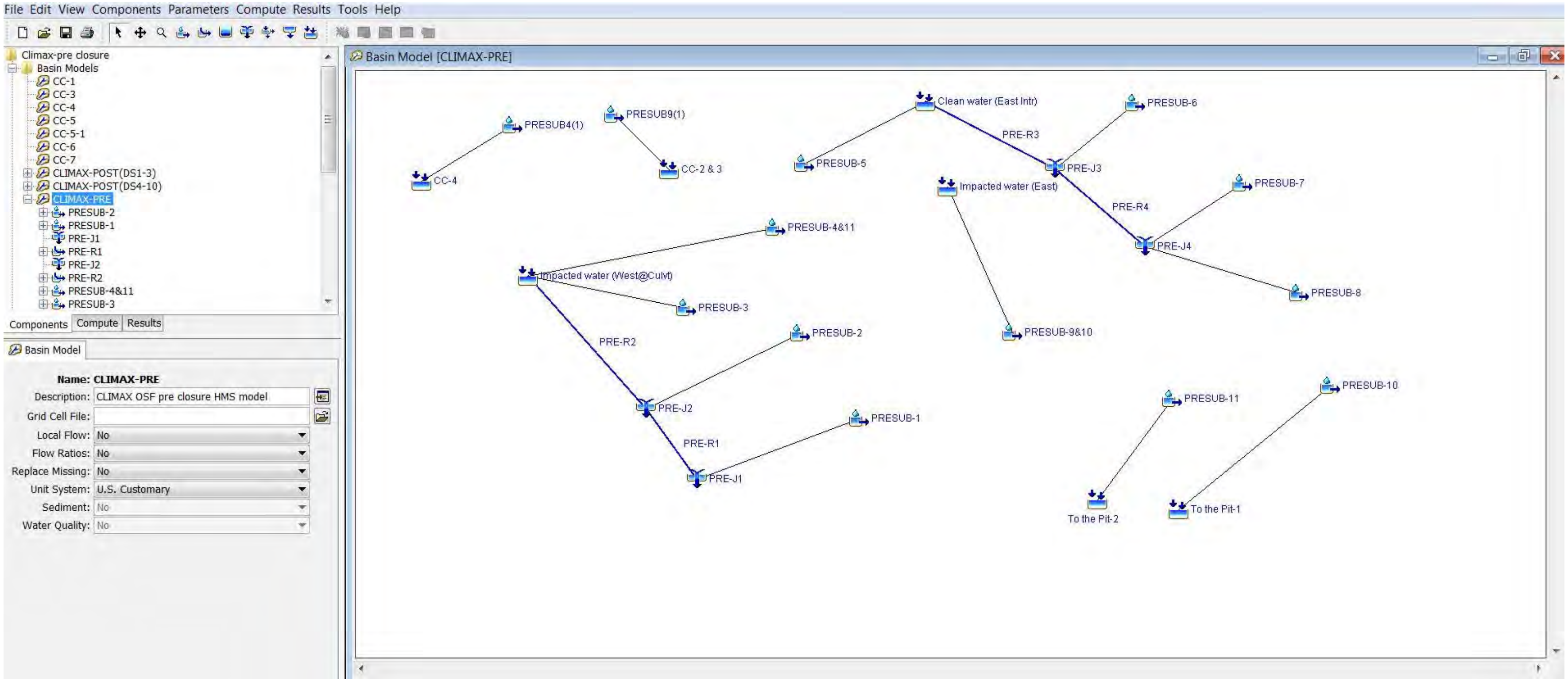

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PRE-R2
PRE-J2
PRESUB-1
PRE-R1
PRE-J1
PRESUB-11
PRESUB-10
To the Pit-2
To the Pit-1

TABLE 1
SUBBASIN CURVE NUMBER TABLE

CLIMAX MINE, COLORADO
Surface Water:Pre-closure Condition
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Apprvd:	TA

DESIGN STORM: 100 -YEAR RECURRENCE INTERVAL

STORM DURATION (hours)	2-YEAR DEPTH (inches)	100 -YEAR DEPTH (inches)	Storm Distribution
24	1.40	1.99	II

SUBBASIN ID	AREA (acres)	CN = 65 Waste Rock	CN = 85 Disturbed area including parking areas	CN = 80 Undisturbed (west and south)	Composite SCS Curve No.	Preliminary runoff estiamtes S = 1000/CN-10	Unit Runoff Q (in)	Runoff Volume (ac-ft)	Runoff Volume (ft^3)
1	109	0.0	109.3	0.0	CN = 85	1.76	0.79	7.17	312,455
2	182	18.2	164.1	0.0	CN = 83	2.05	0.69	10.46	455,526
3	83	5.0	39.1	39.1	CN = 81	2.28	0.62	4.28	186,314
4&11	305	25.6	279.2	0.0	CN = 83	2.00	0.70	17.87	778,339
5	79	0.0	0.0	78.7	CN = 80	2.50	0.56	3.65	159,044
6	120	0.0	0.0	120.4	CN = 80	2.50	0.56	5.58	243,175
7	113	37.7	37.7	37.8	CN = 77	3.04	0.43	4.07	177,086
8	127	42.2	42.2	42.3	CN = 77	3.04	0.43	4.55	198,243
9	262	94.5	167.9	0.0	CN = 78	2.85	0.47	10.31	449,036
9&10	367	94.5	272.4	0.0	CN = 80	2.52	0.55	16.82	732,805
11	71.9	0.0	71.9	0.0	CN = 85	1.76	0.79	4.72	205,724
Total:	2255.78							114.21	4,974,804

TABLE 2
SUBBASIN TIME OF CONCENTRATION CALCULATIONS

CLIMAX MINE, COLORADO
Surface Water:Pre-closure Condition
PROJECT NO.: 113-81608

Date:	**Date:**	5/30/12
By:	**By:**	PK
Chkd:	**Chkd:**	MR
Appvd:	**Appvd:**	TA

					Flow Segment 1							Flow Segment 2						
SUBBASIN ID	Subbasin Area (sq mile)	Composite Curve Number	Total Lag (0.6*Tc) (min)	Total Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[1]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
1	0.1707	85	8.4	14.03	Sheet	100	0.50	E	Short Grass		4.09	Shallow	4286	0.50	U	Unpaved		6.26
2	0.2849	83	14.9	24.79	Sheet	300	0.07	B	Fallow		8.87	Shallow	4138	0.07	U	Unpaved		15.92
3	0.1230	81	11.1	18.46	Sheet	300	0.16	B	Fallow		6.43	Shallow	1189	0.16	U	Unpaved		3.06
4&11	0.1767	83	9.9	16.42	Sheet	300	0.21	B	Fallow		5.79	Shallow	2876	0.21	U	Unpaved		6.50
5	0.1632	80	25.3	42.19	Sheet	300	0.26	E	Short Grass		12.73	Shallow	1297	0.26	U	Unpaved		2.61
6	0.1881	80	9.7	16.15	Sheet	300	0.26	E	Short Grass		12.77	Shallow	1675	0.26	U	Unpaved		3.39
7	0.1767	77	12.4	20.67	Sheet	300	0.17	E	Short Grass		15.10	Shallow	2235	0.17	U	Unpaved		5.57
8	0.1978	77	13.1	21.80	Sheet	300	0.24	E	Short Grass		13.21	Shallow	1099	0.24	U	Unpaved		2.32
9&10	0.5732	80	10.5	17.51	Sheet	300	0.65	E	Short Grass		8.89	Shallow	1625	0.45	U	Unpaved		2.50

Notes:
(1) Refer to Attachment A for Roughness Condition descriptions and Tc Coefficients

TABLE 2
SUBBASIN TIME OF CONCENTRATION CALCULATIONS

CLIMAX MINE, COLORADO
Surface Water:Pre-closure Condition
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

			Flow Segment 3							Flow Segment 4						
SUBBASIN ID	Subbasin Area (sq mile)	Composite Curve Number	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[(1)]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition[(1)]		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
1	0.1707	85	Channel	740	0.02	R	Riprap	0.51	3.68	Channel	0	0.01	R	Riprap	1.26	0.00
2	0.2849	83	Channel	0	0.28	R	Riprap	0.45	0.00	Channel	0	0.01	R	Riprap	1.26	0.00
3	0.1230	81	Channel	1689	0.01	R	Riprap	0.87	8.97	Channel	0	0.01	R	Riprap	1.26	0.00
4&11	0.1767	83	Channel	1445	0.0590	R	Riprap	0.52	4.13	Channel	0	0.01	R	Riprap	1.26	0.00
5	0.1632	80	Channel	4003	0.0050	R	Riprap	0.92	26.85	Channel	0	0.01	R	Riprap	1.26	0.00
6	0.1881	80	Channel	0	0.0086	E	Earth-lined	0.57	0.00	Channel	0	0.01	R	Riprap	1.26	0.00
7	0.1767	77	Channel	0	0.1457	R	Riprap	0.90	0.00	Channel	0	0.01	R	Riprap	1.26	0.00
8	0.1978	77	Channel	2947	0.1289	R	Riprap	0.45	6.27	Channel	0	0.01	R	Riprap	1.26	0.00
9&10	0.5732	80	Channel	4363	0.1160	R	Riprap	0.91	6.12	Channel	0	0.01	R	Riprap	1.26	0.00

Notes:
(1) Refer to Attachment A for Roughness Condition descriptions and Tc Coefficients

TABLE 2
SUBBASIN TIME OF CONCENTRATION CALCULATIONS

CLIMAX MINE, COLORADO
Surface Water:Pre-closure Condition
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

			Flow Segment 5						
SUBBASIN ID	Subbasin Area (sq mile)	Composite Curve Number	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
1	0.1707	85	Shallow	0	0.01	U	Unpaved		0.00
2	0.2849	83	Shallow	0	0.01	U	Unpaved		0.00
3	0.1230	81	Shallow	0	0.01	U	Unpaved		0.00
4&11	0.1767	83	Shallow	0	0.01	U	Unpaved		0.00
5	0.1632	80	Shallow	0	0.01	U	Unpaved		0.00
6	0.1881	80	Shallow	0	0.01	U	Unpaved		0.00
7	0.1767	77	Shallow	0	0.01	U	Unpaved		0.00
8	0.1978	77	Shallow	0	0.01	U	Unpaved		0.00
9&10	0.5732	80	Shallow	0	0.01	U	Unpaved		0.00

Notes:
(1) Refer to Attachment A for Roughness Condition descriptions and Tc Coefficients

Table 3
Case-1 (Operations)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water:Pre-closure Condition
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
PRESUB-2	0.285	111.6	01Jul2020, 13:10	0.690
PRESUB-1	0.171	98.1	01Jul2020, 13:00	0.790
PRE-J1	0.171	98.1	01Jul2020, 13:00	0.790
PRE-R1	0.171	97.4	01Jul2020, 13:05	0.790
PRE-J2	0.456	202.9	01Jul2020, 13:05	0.730
PRE-R2	0.456	194.7	01Jul2020, 13:15	0.720
PRESUB-4&11	0.301	112.2	01Jul2020, 13:05	0.560
PRESUB-3	0.123	43.6	01Jul2020, 13:05	0.560
Contact water (West@Culvt)	0.879	324	01Jul2020, 13:10	0.640
PRESUB-8	0.198	46.4	01Jul2020, 13:10	0.440
PRESUB-7	0.177	42.4	01Jul2020, 13:05	0.440
PRE-J4	0.375	88.4	01Jul2020, 13:10	0.440
PRE-R4	0.375	88	01Jul2020, 13:10	0.440
PRESUB-6	0.188	70.6	01Jul2020, 13:05	0.560
PRE-J3	0.563	147.4	01Jul2020, 13:05	0.480
PRE-R3	0.563	144.9	01Jul2020, 13:10	0.480
PRESUB-5	0.163	35.7	01Jul2020, 13:20	0.560
Non Contact (East Intr)	0.726	174.1	01Jul2020, 13:15	0.500
PRESUB-9&10	0.573	184.5	01Jul2020, 13:05	0.480
Contact water (East)	0.573	184.5	01Jul2020, 13:05	0.480
PRESUB-10	0.163	83.7	01Jul2020, 13:05	0.790
To the Pit-1	0.163	83.7	01Jul2020, 13:05	0.790
PRESUB-11	0.112	62.8	01Jul2020, 13:05	0.790
To the Pit-2	0.112	62.8	01Jul2020, 13:05	0.790
PRESUB4(1)	0.073	34.2	01Jul2020, 13:05	0.690
CC-4	0.073	34.2	01Jul2020, 13:05	0.690
PRESUB9(1)	0.082	25	01Jul2020, 13:05	0.480
CC-2 & 3	0.082	25	01Jul2020, 13:05	0.480

TABLE 4
AT CLOSURE SUBBASIN TIME OF CONCENTRATION CALCULATIONS

CLIMAX MINE, COLORADO
Bench and Top Surface Channels
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

					Flow Segment 1							Flow Segment 2						
SUBBASIN ID	Subbasin Area (sq mile)	Composite Curve Number	Total Lag (0.6*Tc) (min)	Total Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
Typical bench channel	0.0129	75	7.9	13.14	Sheet	50	0.02	E	Short Grass		8.51	Shallow	0	0.02	U	Unpaved		0.00
Typical top channel	0.0094	75	10.3	17.24	Sheet	50	0.01	E	Short Grass		11.23	Shallow	0	0.07	U	Unpaved		0.00

TABLE 4
AT CLOSURE SUBBASIN TIME OF CONCENTRATION CALCULATIONS

CLIMAX MINE, COLORADO
Bench and Top Surface Channels
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

			Flow Segment 3								Flow Segment 4							
SUBBASIN ID	Subbasin Area (sq mile)	Composite Curve Number	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)	Type of Flow	Length (ft)	Slope (ft/ft)	Roughness Condition(1)		Typical Hydraulic Radius (Channel Only) (ft)	Travel Time (min)
Typical bench channel	0.0129	75	Channel	1065	0.02	G	Grass-lined	0.51	4.63	Channel	0	0.01	R	Riprap	1.26	0.00
Typical top channel	0.0094	75	Channel	900	0.01	G	Grass-lined	0.45	6.02	Channel	0	0.01	R	Riprap	1.26	0.00

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
DS1-TOP	0.02959	6.5	01Jul2020, 13:05	0.6
DS1-R1	0.02959	6.5	01Jul2020, 13:05	0.6
A1	0.00672	1.6	01Jul2020, 13:05	0.1
G1	0.00125	0.3	01Jul2020, 13:05	0
DS1-J1	0.03756	8.4	01Jul2020, 13:05	0.8
DS1-R2	0.03756	8.3	01Jul2020, 13:05	0.8
B1	0.006	1.4	01Jul2020, 13:05	0.1
H1	0.00144	0.3	01Jul2020, 13:05	0
DS1-J2	0.045	10.1	01Jul2020, 13:05	0.9
DS1-R3	0.045	10	01Jul2020, 13:05	0.9
C1	0.00503	1.2	01Jul2020, 13:05	0.1
I1	0.00163	0.4	01Jul2020, 13:05	0
DS1-J3	0.05166	11.6	01Jul2020, 13:05	1
DS1-R4	0.05166	11.5	01Jul2020, 13:05	1
D1	0.00401	1	01Jul2020, 13:05	0.1
J1	0.00182	0.4	01Jul2020, 13:05	0
DS1-J4	0.05749	12.9	01Jul2020, 13:05	1.2
DS1-R5	0.05749	12.8	01Jul2020, 13:05	1.2
E1	0.00253	0.6	01Jul2020, 13:05	0.1
K1	0.00201	0.5	01Jul2020, 13:05	0
DS1-J5	0.06203	13.9	01Jul2020, 13:05	1.2
DS1-R6	0.06203	13.9	01Jul2020, 13:05	1.2
L1	0.00221	0.5	01Jul2020, 13:05	0
F1	0.00094	0.2	01Jul2020, 13:05	0
DS1-J6	0.06518	14.6	01Jul2020, 13:05	1.3
DS1-OUTLET	0.06518	14.6	01Jul2020, 13:05	1.3
DS2-TOP	0.00437	1	01Jul2020, 13:05	0.1
DS2-R1	0.00437	0.9	01Jul2020, 13:05	0.1
A2	0.00144	0.3	01Jul2020, 13:05	0
K2	0.00071	0.2	01Jul2020, 13:05	0
DS2-J1	0.00652	1.5	01Jul2020, 13:05	0.1
DS2-R2	0.00652	1.4	01Jul2020, 13:05	0.1
B2	0.00199	0.5	01Jul2020, 13:05	0
L2	0.00086	0.2	01Jul2020, 13:05	0
DS2-J2	0.00937	2.1	01Jul2020, 13:05	0.2
DS2-R3	0.00937	2.1	01Jul2020, 13:05	0.2

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
C2	0.00253	0.6	01Jul2020, 13:05	0.1
M2	0.00102	0.2	01Jul2020, 13:05	0
DS2-J3	0.01292	2.9	01Jul2020, 13:05	0.3
DS2-R4	0.01292	2.9	01Jul2020, 13:05	0.3
D2	0.00304	0.7	01Jul2020, 13:05	0.1
N2	0.00118	0.3	01Jul2020, 13:05	0
DS2-J4	0.01714	3.9	01Jul2020, 13:05	0.3
DS2-R5	0.01714	3.9	01Jul2020, 13:05	0.3
E2	0.00354	0.8	01Jul2020, 13:05	0.1
O2	0.00133	0.3	01Jul2020, 13:05	0
DS2-J5	0.02201	5.1	01Jul2020, 13:05	0.4
DS2-R6	0.02201	5	01Jul2020, 13:05	0.4
F2	0.00404	1	01Jul2020, 13:05	0.1
P2	0.00149	0.4	01Jul2020, 13:05	0
DS2-J6	0.02754	6.3	01Jul2020, 13:05	0.6
DS2-R7	0.02754	6.3	01Jul2020, 13:05	0.6
G2	0.0064	1.5	01Jul2020, 13:05	0.1
Q2	0.00165	0.4	01Jul2020, 13:05	0
DS2-J7	0.03559	8.2	01Jul2020, 13:05	0.7
DS2-R8	0.03559	8.2	01Jul2020, 13:05	0.7
H2	0.00513	1.2	01Jul2020, 13:05	0.1
R2	0.00181	0.4	01Jul2020, 13:05	0
DS2-J8	0.04253	9.8	01Jul2020, 13:05	0.9
DS2-R9	0.04253	9.8	01Jul2020, 13:05	0.9
I2	0.00376	0.9	01Jul2020, 13:05	0.1
S2	0.00197	0.5	01Jul2020, 13:05	0
DS2-J9	0.04826	11.1	01Jul2020, 13:05	1
DS2-R10	0.04826	11.1	01Jul2020, 13:05	1
T2	0.00212	0.5	01Jul2020, 13:05	0
J2	0.00173	0.4	01Jul2020, 13:05	0
DS2-J10	0.05211	12	01Jul2020, 13:05	1
DS2-R11	0.05211	11.9	01Jul2020, 13:05	1
U2	0.003349	0.8	01Jul2020, 13:05	0.1
DS2-OUTLET	0.055459	12.7	01Jul2020, 13:05	1.1
DS3-TOP	0.03751	8.3	01Jul2020, 13:05	0.8

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
DS3-R1	0.03751	8.2	01Jul2020, 13:05	0.8
S3	0.00398	0.9	01Jul2020, 13:05	0.1
A3	0.00316	0.7	01Jul2020, 13:05	0.1
DS3-J1	0.04465	9.9	01Jul2020, 13:05	0.9
DS3-R2	0.04465	9.8	01Jul2020, 13:05	0.9
T3	0.00372	0.9	01Jul2020, 13:05	0.1
B3	0.00335	0.8	01Jul2020, 13:05	0.1
DS3-J2	0.05172	11.5	01Jul2020, 13:05	1
DS3-R3	0.05172	11.4	01Jul2020, 13:05	1
U3	0.00375	0.9	01Jul2020, 13:05	0.1
C3	0.00358	0.8	01Jul2020, 13:05	0.1
DS3-J3	0.05905	13.2	01Jul2020, 13:05	1.2
DS3-R4	0.05905	13.1	01Jul2020, 13:05	1.2
D3	0.00379	0.9	01Jul2020, 13:05	0.1
V3	0.00377	0.9	01Jul2020, 13:05	0.1
DS3-J4	0.06661	14.9	01Jul2020, 13:05	1.3
DS3-R5	0.06661	14.8	01Jul2020, 13:05	1.3
E3	0.004	0.9	01Jul2020, 13:05	0.1
W3	0.0038	0.9	01Jul2020, 13:05	0.1
DS3-J5	0.07441	16.7	01Jul2020, 13:05	1.5
DS3-R6	0.07441	16.6	01Jul2020, 13:05	1.5
X3	0.00672	1.6	01Jul2020, 13:05	0.1
F3	0.00421	1	01Jul2020, 13:05	0.1
DS3-J6	0.08534	19.2	01Jul2020, 13:05	1.7
DS3-R7	0.08534	19.1	01Jul2020, 13:05	1.7
Y3	0.00668	1.6	01Jul2020, 13:05	0.1
G3	0.00441	1	01Jul2020, 13:05	0.1
DS3-J7	0.09643	21.7	01Jul2020, 13:05	1.9
DS3-R8	0.09643	21.6	01Jul2020, 13:05	1.9
Z3	0.00664	1.6	01Jul2020, 13:05	0.1
H3	0.00462	1.1	01Jul2020, 13:05	0.1
DS3-J8	0.10769	24.3	01Jul2020, 13:05	2.2
DS3-R9	0.10769	24.2	01Jul2020, 13:05	2.2
AA3	0.0066	1.6	01Jul2020, 13:05	0.1
I3	0.00483	1.1	01Jul2020, 13:05	0.1

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: **113-81608**

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
DS3-J9	0.11912	26.9	01Jul2020, 13:05	2.4
DS3-R10	0.11912	26.8	01Jul2020, 13:05	2.4
BB3	0.00656	1.6	01Jul2020, 13:05	0.1
J3	0.00503	1.2	01Jul2020, 13:05	0.1
DS3-J10	0.13071	29.5	01Jul2020, 13:05	2.6
DS3-R11	0.13071	29.4	01Jul2020, 13:05	2.6
K3	0.00524	1.2	01Jul2020, 13:05	0.1
CC3	0.00395	0.9	01Jul2020, 13:05	0.1
DS3-J11	0.1399	31.6	01Jul2020, 13:05	2.8
DS3-R12	0.1399	31.4	01Jul2020, 13:05	2.8
L3	0.00666	1.6	01Jul2020, 13:05	0.1
DD3	0.0052	1.2	01Jul2020, 13:05	0.1
DS3-J12	0.15176	34.2	01Jul2020, 13:05	3.1
DS3-R13	0.15176	34.1	01Jul2020, 13:05	3.1
M3	0.00579	1.4	01Jul2020, 13:05	0.1
EE3	0.00476	1.1	01Jul2020, 13:05	0.1
DS3-J13	0.16231	36.6	01Jul2020, 13:05	3.3
DS3-R14	0.16231	36.4	01Jul2020, 13:05	3.3
N3	0.004898	1.2	01Jul2020, 13:05	0.1
FF3	0.00433	1	01Jul2020, 13:05	0.1
DS3-J14	0.171538	38.6	01Jul2020, 13:05	3.5
DS3-R15	0.171538	38.5	01Jul2020, 13:05	3.5
GG3	0.00399	0.9	01Jul2020, 13:05	0.1
O3	0.00378	0.9	01Jul2020, 13:05	0.1
DS3-J15	0.179308	40.3	01Jul2020, 13:05	3.6
DS3-R16	0.179308	40.2	01Jul2020, 13:05	3.6
HH3	0.00353	0.8	01Jul2020, 13:05	0.1
P3	0.00232	0.6	01Jul2020, 13:05	0
DS3-J16	0.185158	41.6	01Jul2020, 13:05	3.7
DS3-R17	0.185158	41.4	01Jul2020, 13:05	3.7
II3	0.00322	0.8	01Jul2020, 13:05	0.1
Q3	0.00166	0.4	01Jul2020, 13:05	0
DS3-J17	0.190038	42.5	01Jul2020, 13:05	3.8
DS3-R18	0.190038	42.3	01Jul2020, 13:05	3.8
JJ3	0.00275	0.7	01Jul2020, 13:05	0.1

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
R3	0.00091	0.2	01Jul2020, 13:05	0
DS3-J18	0.193698	43.2	01Jul2020, 13:05	3.9
DS3-R19	0.193698	43	01Jul2020, 13:05	3.9
KK3	0.00183	0.4	01Jul2020, 13:05	0
DS3-J19	0.195528	43.5	01Jul2020, 13:05	3.9
DS3-OUTLET	0.195528	43.5	01Jul2020, 13:05	3.9
DS5-TOP	0.01085	2.4	01Jul2020, 13:05	0.2
DS5-R1	0.01085	2.4	01Jul2020, 13:05	0.2
L5	0.00951	2.3	01Jul2020, 13:05	0.2
A5	0.00048	0.1	01Jul2020, 13:05	0
DS5-J1	0.02084	4.7	01Jul2020, 13:05	0.4
DS5-R2	0.02084	4.7	01Jul2020, 13:05	0.4
M5	0.00638	1.5	01Jul2020, 13:05	0.1
B5	0.00107	0.3	01Jul2020, 13:05	0
DS5-J2	0.02829	6.5	01Jul2020, 13:05	0.6
DS5-R3	0.02829	6.4	01Jul2020, 13:05	0.6
N5	0.00647	1.5	01Jul2020, 13:05	0.1
C5	0.00167	0.4	01Jul2020, 13:05	0
DS5-J3	0.03643	8.4	01Jul2020, 13:05	0.7
DS5-R4	0.03643	8.3	01Jul2020, 13:05	0.7
O5	0.00504	1.2	01Jul2020, 13:05	0.1
D5	0.00214	0.5	01Jul2020, 13:05	0
DS5-J4	0.04361	10	01Jul2020, 13:05	0.9
DS5-R5	0.04361	10	01Jul2020, 13:05	0.9
P5	0.00448	1.1	01Jul2020, 13:05	0.1
E5	0.00257	0.6	01Jul2020, 13:05	0.1
DS5-J5	0.05066	11.7	01Jul2020, 13:05	1
DS5-R6	0.05066	11.6	01Jul2020, 13:05	1
Q5	0.00353	0.8	01Jul2020, 13:05	0.1
F5	0.00301	0.7	01Jul2020, 13:05	0.1
DS5-J6	0.0572	13.2	01Jul2020, 13:05	1.1
DS5-R7	0.0572	13.1	01Jul2020, 13:05	1.2
G5	0.00347	0.8	01Jul2020, 13:05	0.1
R5	0.00296	0.7	01Jul2020, 13:05	0.1
DS5-J7	0.06363	14.6	01Jul2020, 13:05	1.3

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
DS5-R8	0.06363	14.6	01Jul2020, 13:05	1.3
H5	0.00393	0.9	01Jul2020, 13:05	0.1
S5	0.00141	0.3	01Jul2020, 13:05	0
DS5-J8	0.06897	15.8	01Jul2020, 13:05	1.4
DS5-R9	0.06897	15.8	01Jul2020, 13:05	1.4
I5	0.00439	1	01Jul2020, 13:05	0.1
T5	0.00155	0.4	01Jul2020, 13:05	0
DS5-J9	0.07491	17.2	01Jul2020, 13:05	1.5
DS5-R10	0.07491	17.1	01Jul2020, 13:05	1.5
J5	0.00602	1.4	01Jul2020, 13:05	0.1
U5	0.00102	0.2	01Jul2020, 13:05	0
DS5-J10	0.08195	18.8	01Jul2020, 13:05	1.6
DS5-R11	0.08195	18.7	01Jul2020, 13:05	1.6
K5	0.00662	1.6	01Jul2020, 13:05	0.1
V5	0.00037	0.1	01Jul2020, 13:05	0
DS5-J11	0.08894	20.4	01Jul2020, 13:05	1.8
DS5-OUTLET	0.08894	20.4	01Jul2020, 13:05	1.8
DS6-TOP	0.00908	2	01Jul2020, 13:05	0.2
DS6-R1	0.00908	2	01Jul2020, 13:05	0.2
H6	0.00246	0.6	01Jul2020, 13:05	0
A6	0.00224	0.5	01Jul2020, 13:05	0
DS6-J1	0.01378	3.1	01Jul2020, 13:05	0.3
DS6-R2	0.01378	3.1	01Jul2020, 13:05	0.3
B6	0.00343	0.8	01Jul2020, 13:05	0.1
I6	0.00223	0.5	01Jul2020, 13:05	0
DS6-J2	0.01944	4.4	01Jul2020, 13:05	0.4
DS6-R3	0.01944	4.4	01Jul2020, 13:05	0.4
C6	0.00463	1.1	01Jul2020, 13:05	0.1
J6	0.00201	0.5	01Jul2020, 13:05	0
DS6-J3	0.02608	6	01Jul2020, 13:05	0.5
DS6-R4	0.02608	5.9	01Jul2020, 13:05	0.5
D6	0.00582	1.4	01Jul2020, 13:05	0.1
K6	0.00179	0.4	01Jul2020, 13:05	0
DS6-J4	0.03369	7.7	01Jul2020, 13:05	0.7
DS6-R5	0.03369	7.7	01Jul2020, 13:05	0.7

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
E6	0.00702	1.7	01Jul2020, 13:05	0.1
L6	0.00472	1.1	01Jul2020, 13:05	0.1
DS6-J5	0.04543	10.5	01Jul2020, 13:05	0.9
DS6-R6	0.04543	10.4	01Jul2020, 13:05	0.9
DS6-TOP2	0.01302	2.9	01Jul2020, 13:05	0.3
F6	0.00407	1	01Jul2020, 13:05	0.1
M6	0.0038	0.9	01Jul2020, 13:05	0.1
DS6-J6	0.06632	15.2	01Jul2020, 13:05	1.3
DS6-R7	0.06632	15.1	01Jul2020, 13:05	1.3
G6	0.00406	1	01Jul2020, 13:05	0.1
N6	0.00184	0.4	01Jul2020, 13:05	0
DS6-J7	0.07222	16.5	01Jul2020, 13:05	1.5
DS6-OUTLET	0.07222	16.5	01Jul2020, 13:05	1.5
DS7-TOP	0.04677	10.3	01Jul2020, 13:05	0.9
DS7-R1	0.04677	10.2	01Jul2020, 13:05	0.9
E7	0.00426	1	01Jul2020, 13:05	0.1
A7	0.0025	0.6	01Jul2020, 13:05	0.1
DS7-J1	0.05353	11.8	01Jul2020, 13:05	1.1
DS7-R2	0.05353	11.7	01Jul2020, 13:05	1.1
F7	0.00325	0.8	01Jul2020, 13:05	0.1
B7	0.00273	0.6	01Jul2020, 13:05	0.1
DS7-J2	0.05951	13.2	01Jul2020, 13:05	1.2
DS7-R3	0.05951	13.1	01Jul2020, 13:05	1.2
C7	0.00296	0.7	01Jul2020, 13:05	0.1
G7	0.00255	0.6	01Jul2020, 13:05	0.1
DS7-J3	0.06502	14.4	01Jul2020, 13:05	1.3
DS7-R4	0.06502	14.3	01Jul2020, 13:05	1.3
D7	0.00319	0.8	01Jul2020, 13:05	0.1
H7	0.00125	0.3	01Jul2020, 13:05	0
DS7-J4	0.06946	15.4	01Jul2020, 13:05	1.4
DS7-OUTLET	0.06946	15.4	01Jul2020, 13:05	1.4
DS4-TOP	0.0151	3.3	01Jul2020, 13:05	0.3
DS4-R1	0.0151	3.3	01Jul2020, 13:05	0.3
A4	0.00524	1.2	01Jul2020, 13:05	0.1
I4	0.00119	0.3	01Jul2020, 13:05	0

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: 113-81608

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
DS4-J1	0.02153	4.8	01Jul2020, 13:05	0.4
DS4-R2	0.02153	4.8	01Jul2020, 13:05	0.4
B4	0.00542	1.3	01Jul2020, 13:05	0.1
J4	0.00151	0.4	01Jul2020, 13:05	0
DS4-J2	0.02846	6.4	01Jul2020, 13:05	0.6
DS4-R3	0.02846	6.4	01Jul2020, 13:05	0.6
C4	0.0056	1.3	01Jul2020, 13:05	0.1
K4	0.00183	0.4	01Jul2020, 13:05	0
DS4-J3	0.03589	8.1	01Jul2020, 13:05	0.7
DS4-R4	0.03589	8.1	01Jul2020, 13:05	0.7
D4	0.00578	1.4	01Jul2020, 13:05	0.1
L4	0.00218	0.5	01Jul2020, 13:05	0
DS4-J4	0.04385	10	01Jul2020, 13:05	0.9
DS4-R5	0.04385	9.9	01Jul2020, 13:05	0.9
E4	0.00596	1.4	01Jul2020, 13:05	0.1
M4	0.00252	0.6	01Jul2020, 13:05	0.1
DS4-J5	0.05233	12	01Jul2020, 13:05	1.1
DS4-R6	0.05233	11.9	01Jul2020, 13:05	1.1
N4	0.00287	0.7	01Jul2020, 13:05	0.1
F4	0.00232	0.6	01Jul2020, 13:05	0
DS4-J6	0.05752	13.1	01Jul2020, 13:05	1.2
DS4-R7	0.05752	13.1	01Jul2020, 13:05	1.2
O4	0.00318	0.8	01Jul2020, 13:05	0.1
G4	0.00164	0.4	01Jul2020, 13:05	0
DS4-J7	0.06234	14.2	01Jul2020, 13:05	1.3
DS4-R8	0.06234	14.1	01Jul2020, 13:05	1.3
P4	0.0035	0.8	01Jul2020, 13:05	0.1
H4	0.00022	0.1	01Jul2020, 13:05	0
DS4-J8	0.06606	15	01Jul2020, 13:05	1.3
DS4-R9	0.06606	14.9	01Jul2020, 13:05	1.3
Q4	0.00381	0.9	01Jul2020, 13:05	0.1
DS4-J9	0.06987	15.8	01Jul2020, 13:05	1.4
DS4-OUTLET	0.06987	15.8	01Jul2020, 13:05	1.4
DS8-TOP	0.016	3.5	01Jul2020, 13:05	0.3
DS8-R1	0.016	3.5	01Jul2020, 13:05	0.3

TABLE 5
Case-2 (Closure)
FLOW RESULTS FROM HEC-HMS

CLIMAX MINE, COLORADO
Surface Water: At Closure
PROJECT NO.: **113-81608**

Date:	5/30/12
By:	PK
Chkd:	MR
Appvd:	TA

HEC-HMS Basin Model:	rev 0
HEC-HMS Met. Model:	100 yr 24 hr
HEC-HMS Control Specs:	36 hr 5 min

Hydrologic Element	Drainage Area (Sq mi)	Discharge Peak (cfs)	Time of Peak	Total Volume (ac ft)
C8	0.00426	1	01Jul2020, 13:05	0.1
A8	0.0025	0.6	01Jul2020, 13:05	0.1
DS8-J1	0.02276	5.1	01Jul2020, 13:05	0.5
DS8-R2	0.02276	5.1	01Jul2020, 13:05	0.5
D8	0.00325	0.8	01Jul2020, 13:05	0.1
B8	0.00273	0.6	01Jul2020, 13:05	0.1
DS8-J2	0.02874	6.5	01Jul2020, 13:05	0.6
DS8-R3	0.02874	6.4	01Jul2020, 13:05	0.6
E8	0.00255	0.6	01Jul2020, 13:05	0.1
DS8-J3	0.03129	7	01Jul2020, 13:05	0.6
DS8-OUTLET	0.03129	7	01Jul2020, 13:05	0.6
DS9-TOP	0.0502	11	01Jul2020, 13:05	1
DS9-R1	0.0502	11	01Jul2020, 13:05	1
C9	0.00426	1	01Jul2020, 13:05	0.1
A9	0.0025	0.6	01Jul2020, 13:05	0.1
DS9-J1	0.05696	12.6	01Jul2020, 13:05	1.1
DS9-R2	0.05696	12.5	01Jul2020, 13:05	1.1
D9	0.00325	0.8	01Jul2020, 13:05	0.1
B9	0.00273	0.6	01Jul2020, 13:05	0.1
DS9-J2	0.06294	13.9	01Jul2020, 13:05	1.3
DS9-OUTLET	0.06294	13.9	01Jul2020, 13:05	1.3
DS10-TOP	0.03589	7.9	01Jul2020, 13:05	0.7
DS10-R1	0.03589	7.8	01Jul2020, 13:05	0.7
A10	0.00648	1.5	01Jul2020, 13:05	0.1
C10	0.00408	1	01Jul2020, 13:05	0.1
DS10-J1	0.04645	10.3	01Jul2020, 13:05	0.9
DS10-R2	0.04645	10.3	01Jul2020, 13:05	0.9
B10	0.00479	1.1	01Jul2020, 13:05	0.1
DS10-J2	0.05124	11.4	01Jul2020, 13:05	1
DS10-OUTLET	0.05124	11.4	01Jul2020, 13:05	1

Table 6
Hydraulic Calculations of Channels

CLIMAX MINE, COLORADO
Channel Calculaitons
PROJECT NO.: 113-81608

Date:	5/31/12
By:	PK
Chkd:	MR
Appvd:	TA

			Channel Design Geometry						Channel Roughness Parameters		
Channel Designation	**Q100 from HEC-HMS (cfs)**	**HEC HMS Element ID for Q**	**Channel Length (ft)**	**Bed Slope (ft/ft)**	**Left Side Slope (H:1V)**	**Right Side Slope (H:1V)**	**Bottom Width (ft)**	**Maximum Channel Depth (ft)**	**Design Channel Lining**	**Mannings 'n' for Capacity (Depth Calculation)**	**Mannings 'n' for Stability (Velocity Calculation)**
DS-1	15	DS-1	881.3	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-2	13	DS-2	1521.0	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-3	44	DS-3	1521.0	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-5	16	DS-5	1110.9	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-6	20	DS-6	1518.7	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-7	17	DS-7	1084.9	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-4	15	DS-4	592.8	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-8	7	DS-8	348.5	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-9	14	DS-9	315.4	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
DS-10	11	DS-10	206.4	50.0%	3.0	3.0	5	3.5	A ACB	0.026	0.026
Typical bench channel	3	Typical bench channel	1800	2.0%	3.0	3.0	10	2.0	G Grass-lined	0.035	0.030
Typical top surface channel	12	Typical top surface channel	1800	1.0%	3.0	3.0	5	2.0	G Grass-lined	0.035	0.030
CC-1	24	CC-1	4844	15.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-2	36	CC-2	2626	17.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-3	16	CC-3	2449	16.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-4	56	CC-4	5495	2.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-5	50	CC-5	8130	2.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-7	31	CC-7	4901	14.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
CC-8	63	CC-8	1492	17.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
OC-1	88	OC-1	1912	20.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
OC-3	25	OC-3	2626	17.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
OC-4	34	OC-4	2449	16.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035
OC-5	203	OC-5	8130	2.0%	3.0	3.0	5	3.5	R Riprap	0.040	0.035

Table 6
Hydraulic Calculations of Channels

CLIMAX MINE, COLORADO
Channel Calculaitons
PROJECT NO.: 113-81608

Date:	5/31/12
By:	PK
Chkd:	MR
Appvd:	TA

		Hydraulic Calculations				Channel Evaluations					
Channel Designation	**Q100 from HEC-HMS (cfs)**	**Maximum Velocity (ft/sec)**	**Maximum Normal Flow Depth (ft)**	**Froude Number**	**Normal Depth Shear Stress (lb/ft^2)**	**Available Freeboard (ft)**	**Top Width of Flow (ft)**	**Channel Lining Evaluation Method**	**Calculated Riprap d_{50} (in)**	**Suggested Riprap Size (1)**	**Channel is Stable**
DS-1	15	12.9	0.20	5.34	6.29	3.3	6.2	Max. Velocity	--	--	Stable
DS-2	13	12.3	0.19	5.27	5.80	3.3	6.1	Max. Velocity	--	--	Stable
DS-3	44	18.7	0.38	5.84	11.81	3.1	7.3	Max. Velocity	--	--	Stable
DS-5	16	13.3	0.21	5.37	6.59	3.3	6.3	Max. Velocity	--	--	Stable
DS-6	20	14.5	0.24	5.49	7.64	3.3	6.5	Max. Velocity	--	--	Stable
DS-7	17	13.5	0.22	5.39	6.76	3.3	6.3	Max. Velocity	--	--	Stable
DS-4	15	13.2	0.21	5.36	6.49	3.3	6.2	Max. Velocity	--	--	Stable
DS-8	7	9.9	0.13	5.00	4.08	3.4	5.8	Max. Velocity	--	--	Stable
DS-9	14	12.7	0.20	5.32	6.11	3.3	6.2	Max. Velocity	--	--	Stable
DS-10	11	11.8	0.17	5.23	5.44	3.3	6.0	Max. Velocity	--	--	Stable
Typical bench channel	3	1.8	0.15	0.88	0.18	1.9	10.9	Max. Velocity	--	--	Stable
Typical top surface channel	12	3.0	0.65	0.76	0.40	1.4	8.9	Max. Velocity	--	--	Stable
CC-1	24	8.4	0.48	2.43	4.54	3.0	7.9	Steep Riprap	7.5	Type L	Riprap
CC-2	36	10.0	0.59	2.66	6.22	2.9	8.5	Steep Riprap	9.5	Type M	Riprap
CC-3	16	7.5	0.38	2.43	3.78	3.1	7.3	Steep Riprap	6.3	Type L	Riprap
CC-4	56	5.3	1.30	1.01	1.63	2.2	12.8	Mild Riprap	0.7	Type VL	Riprap
CC-5	50	5.2	1.23	1.00	1.54	2.3	12.4	Mild Riprap	0.7	Type VL	Riprap
CC-7	31	8.9	0.57	2.40	4.98	2.9	8.4	Steep Riprap	8.3	Type L	Riprap
CC-8	63	11.8	0.79	2.77	8.43	2.7	9.8	Steep Riprap	12.2	Type M	Riprap
OC-1	88	13.8	0.91	3.05	11.36	2.6	10.5	Steep Riprap	14.9	Type M	Riprap
OC-3	25	8.9	0.48	2.59	5.08	3.0	7.9	Steep Riprap	8.0	Type L	Riprap
OC-4	34	9.6	0.58	2.57	5.78	2.9	8.5	Steep Riprap	9.0	Type M	Riprap
OC-5	203	7.5	2.43	1.10	3.03	1.1	19.6	Mild Riprap	1.5	Type VL	Riprap

Table 6
Hydraulic Calculations of Channels

CLIMAX MINE, COLORADO
Channel Calculaitons
PROJECT NO.: 113-81608

Date:	5/31/12
By:	PK
Chkd:	MR
Appvd:	TA

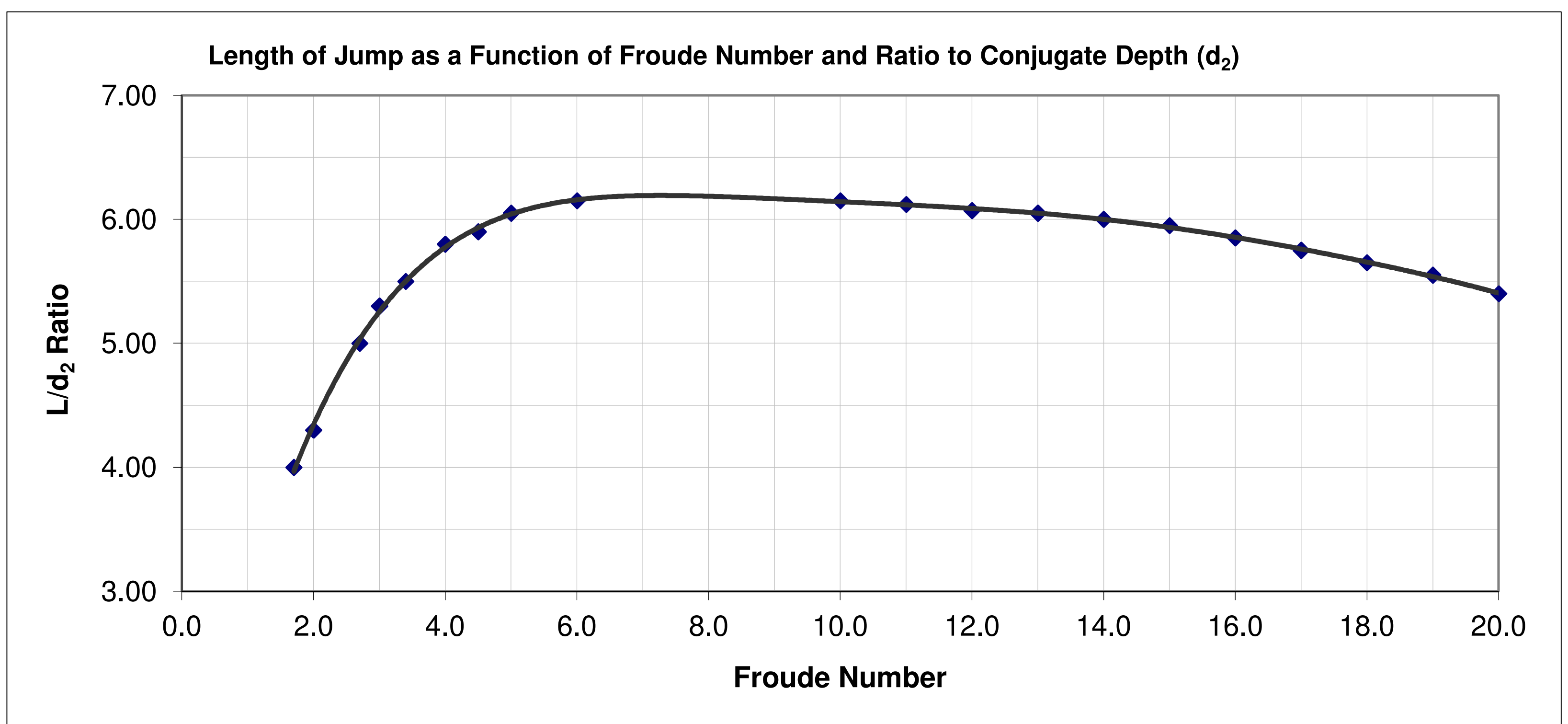
			Riprap Calculations for Steep Riprap (Bed Slopes >2% but <40%)						Riprap Calculations for Mild Riprap (Bed Slopes <2%)					
Channel Designation	Q100 from HEC-HMS (cfs)	Riprap Size Calculation Method	Unit Flow q (1) (cfs/ft)	Design Flow Q (cms)	Unit Width Flow q (cms/m)	Calculated Particle Size D_{50} (2) (mm)	Factor of Safety	Riprap Size D_{50} (inches)	Design Flow Q (cfs)	Normal Flow Depth d (ft)	Maximum Velocity V (ft/s)	Channel Side Slopes Correction K_1 (3)	Calculated Particle Size D_{30} (4) (ft)	Riprap Size D_{50} (5) (inches)
DS-1	15	NA												
DS-2	13	NA												
DS-3	44	NA												
DS-5	16	NA												
DS-6	20	NA												
DS-7	17	NA												
DS-4	15	NA												
DS-8	7	NA												
DS-9	14	NA												
DS-10	11	NA												
Typical bench channel	3	NA												
Typical top surface channel	12	NA												
CC-1	24	Steep Riprap	3.78	0.680	0.351	159	1.2	7.5						
CC-2	36	Steep Riprap	5.43	1.019	0.504	200	1.2	9.5						
CC-3	16	Steep Riprap	2.64	0.453	0.246	134	1.2	6.3						
CC-4	56	Mild Riprap							56.0	1.30	5.32	3	0.04	0.7
CC-5	50	Mild Riprap							50.0	1.23	5.15	3	0.03	0.7
CC-7	31	Steep Riprap	4.71	0.878	0.437	175	1.2	8.3						
CC-8	63	Steep Riprap	8.73	1.784	0.811	257	1.2	12.2						
OC-1	88	Steep Riprap	11.72	2.503	1.088	316	1.2	14.9						
OC-3	25	Steep Riprap	3.95	0.708	0.367	169	1.2	8.0						
OC-4	34	Steep Riprap	5.16	0.966	0.479	191	1.2	9.0						
OC-5	203	Mild Riprap							203.0	2.43	7.50	3	0.07	1.5

ATTACHMENT 2
ENERGY DISSIPATER
HYDRAULIC JUMP CALCULATIONS

CLIMAX PROJECT
Energy Dissipator Hydraulic Jump Calculation
PROJECT NO.: 113-81608

Date:	3/29/12
By:	PK
Chkd:	MR
Apprvd:	TA

Reach Designation	Design Flow (cfs)	Channel Configuration: Bed Slope	Left Side Slope (H:1V)	Right Side Slope (H:1V)	Bottom Width (ft)	Maximum Channel Depth (ft)	Mannings 'n' for Capacity (Depth Calculation)	Mannings 'n' for Stability (Velocity Calculation)	Hydraulic Calculations: Maximum Velocity (ft/sec)	Maximum Normal Flow Depth (ft)	Normal Depth with Velocity 'n' (ft)	Froude Number	Conjugate Depth (ft)	L/d2 Ratio	Minimum Length of Jump (ft)
DS1-OUTLET	14.6	50.00%	3.0	3.0	5	3.0	0.026	0.026	12.9	0.20	0.20	5.34	1.3	6.09	8
DS2-OUTLET	12.8	50.00%	3.0	3.0	5	3.0	0.026	0.026	12.3	0.19	0.19	5.28	1.2	6.08	8
DS3-OUTLET	43.9	50.00%	3.0	3.0	5	3.0	0.026	0.026	18.8	0.38	0.38	5.84	2.7	6.15	17
DS5-OUTLET	20.4	50.00%	3.0	3.0	5	3.0	0.026	0.026	14.5	0.24	0.24	5.49	1.7	6.11	10
DS6-OUTLET	16.9	50.00%	3.0	3.0	5	3.0	0.026	0.026	13.6	0.22	0.22	5.41	1.5	6.10	9
DS7-OUTLET	15.4	50.00%	3.0	3.0	5	3.0	0.026	0.026	13.2	0.21	0.21	5.36	1.4	6.10	9
DS4-OUTLET	15.9	50.00%	3.0	3.0	5	3.0	0.026	0.026	13.3	0.21	0.21	5.38	1.4	6.10	9
DS8-OUTLET	7.0	50.00%	3.0	3.0	5	3.0	0.026	0.026	9.9	0.13	0.13	5.00	0.8	6.04	5
DS9-OUTLET	13.9	50.00%	3.0	3.0	5	3.0	0.026	0.026	12.7	0.20	0.20	5.32	1.3	6.09	8
DS10-OUTLET	12.5	50.00%	3.0	3.0	5	3.0	0.026	0.026	12.2	0.18	0.18	5.27	1.2	6.08	7



$$d_2 = -\frac{d_1}{2} + \sqrt{\frac{2V_1^2 d_1}{g} + \frac{d_1^2}{4}}$$

Where:
d1 = Depth before the jump
V1 = Velocity before the jump
d2 = Depth after the jump,
g = Acceleration of gravity

PROJECT DESIGN CRITERIA

Client: Climax Molybdenum Company (Climax)	
Project Title: North 40 / McNulty Overburden Storage Facility	
Design Criteria: Revision D – Issued for Report	

Revision				Pages	
#	By	App.	Date	Revised	Remarks
A	Golder	BB	29-Nov-11	All	Issued for Comment
B	Golder	BB	15-Dec-11	All	Revised with Climax Comments, Issued for Comment
C	Golder	BB	8-Mar-12	All	Issued for Stability Memorandum
D	Golder	BB	30-Mar-12	All	Issued for Report

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Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

GENERAL PROJECT SUMMARY	Date	Rev. #
Develop design criteria to support the permitting and final engineering design for the proposed expansion of the North 40 and McNulty overburden storage facilities (OSFs). ▪ Initial design criteria (Rev A) established to solicit input amongst project team	29-Nov-11	A
Design criteria revised based on input from Climax.	15-Dec-11	B
Design criteria revised based on project team discussions.	8-Mar-12	C
Design criteria revised based on project team discussions	30-Mar-12	D

DESIGN AND VERIFICATION CODES

Design Input
A=Assumed
B=Calculated
C=Client Information/Request
GA=Golder Associates
I=Industry Standard Practice
O=Information Provided by Others
P=Published Information/Criteria
T=Testwork Data
TBD=To Be Determined
V=Vendor Data

Discipline
Ci=Civil
G=Geotechnical
St=Structural
Pi=Piping
Pr=Process
El=Electrical
In=Instrumentation
N/A=Not Applicable

Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

LIST OF ABBREVIATIONS				
Ft = Feet			29-Nov-11	A
fps = Feet per Second			29-Nov-11	A
OSF = Overburden Storage Facility			29-Nov-11	A
CWCS = Contact Water Collection System			3-Apr-12	D
TBD = To Be Determined			29-Nov-11	A
Climax = Climax Molybdenum Company			29-Nov-11	A
Regional Design Factors				
Location	Climax Mine, Climax, Colorado		29-Nov-11	A
Regulatory Considerations	Comply with applicable DMRS regulations and corporate policies.		29-Nov-11	A
Climate	Infiltration and runoff analysis to rely primarily on climate data obtained from the Climax weather station, as presented in Exhibit K of the AM-06 Amendment to the Climax Mine Reclamation Permit M-1977-493. Other nearby climate data may be used, if the necessary information cannot be obtained from the Climax weather station. Such climate data will be adjusted for elevation, precipitation, and other factors, if needed.		8-Mar-11	C
Project Disturbance Areas	All project disturbance, including overburden storage areas and constructed channels, shall remain within areas identified by Climax. No project components shall extend outside areas with Climax patented rights, surface right areas, or ownership.		29-Nov-11	A
Operational Base Earthquake (OBE)	1 in 475 year event (0.06 g) to be used for design of operational slopes and structures.	G	15-Dec-11	B
Maximum Design Earthquake (MDE)	1 in 2,475 year event (0.14 g) to be used for design of reclaimed slopes for the closure and post-closure periods.	G	30-Mar-12	D

Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

Hydrology and Surface Water Management Strategy				
Operations Strategy	Maintain separation of contact versus non-contact water to the extent possible via perimeter operational channels, a contact water pipe system, and an underdrain system. Non-contact water will be discharged to the East Interceptor. Contact water will be directed to the Climax water treatment facility. The term "contact water" refers to waters that have flowed over, through, or otherwise been in contact with mined overburden materials, tailing, or other mine waste, which may or may not have the potential to induce adverse changes in water quality. The term "non-contact" water refers to water that has not been in contact with mined overburden or mine waste materials.	Ci	30-Mar-12	D
Operations Run-on / Run-off control channels	Design Storm: 10-year, 24-hour (1.40 inches)	Ci	29-Nov-11	A
Management of water internal to the OSF	A contact water collection system (CWCS), consisting of drain gravel and pipe, will be constructed along the McNulty OSF foundation to drain contact water internal to the OSF. At closure, toe drains will be constructed where required along the toes of both OSFs. The toe drains are intended to capture contact water internal to the OSFs at those locations where there is potential for contact water to exit the toes of the OSFs as seepage.	Ci	30-Mar-12	D
Reclamation and Closure Strategy	Barber pole channel layout, i.e., top-surface and outslope channels will convey runoff to steeper downdrains, and then to an energy dissipater & perimeter channel.	Ci	29-Nov-11	A

Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

Design Storm for Top-Surface, Outslope, and Perimeter Reclamation Channels	The reclamation channel design criteria consider both potential rainfall and potential snowmelt. The design storm was defined to be the less favorable (more severe) of either the 100-year, 24-hour storm event or the 10-year, 24-hour storm event superimposed with the estimated flow produced by the 100-year, 24-hour snowmelt event. Based on the available climate data, the 100-year, 24-hour storm event will produce a larger peak runoff, and will therefore be used as the basis for channel design. Based on data from the Climax weather station, the 100-year, 24-hour storm event is 1.99 inches.	Ci	8-Mar-12	C
Design Storm for Downdrains	Use same criteria as for top surface, outslope, and toe reclamation channels.	Ci	8-Mar-12	C
Channel Freeboard	1-Ft Minimum	Ci	29-Nov-11	A
Channel Design Methodology	HEC HMS method (TR-55). Time of concentration based on USBR for basins greater than 1 square mile and TR-55 for smaller basins; Two models developed using the low end of the friction coefficient range to evaluate peak velocities, and the second using the higher end of the range to evaluate peak flow depths.	Ci	29-Nov-11	A
Curve Numbers (CN)	Undisturbed native areas: CN = 80 Un-reclaimed overburden: CN = 65 Disturbed areas other than overburden (e.g., dirt roads, parking and laydown areas, etc): CN = 85 Covered and revegetated overburden: CN = 75	Ci	8-Mar-12	C
Minimum Velocity	Target minimum 1% grade for top surface channels, wherever possible. Outslope reclamation benches, target 2% slope.	Ci	29-Nov-11	A
Maximum Velocity	Less than 5.0 feet per second (fps), unlined channel; Greater than 5.0 fps and less than 15 fps, riprap channel; Greater than 15 fps and less than 25 fps, concrete lined or articulated concrete block channels or competent bedrock; Velocities greater than 25 fps to be avoided.	Ci	29-Nov-11	A

Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

Rip Rap Sizing Methodologies	USACE Method and Robinson et al.	Ci	29-Nov-11	A
Erosion Protection Revetment	To be selected based on calculated channel flows.	Ci	15-Dec-11	B
Channel Friction Coefficient	TBD based on revetment selected.	Ci	15-Dec-11	B
OSF Development Criteria				
Operations Criteria				
McNulty OSF Design Slope Constraints	Toe constraints exist, controlling allowable operations and final reclamation toe locations. Operations outslope design to be driven by stability and to accommodate the reclamation toe limits. Maximum toe limit of reclaimed slopes to be compatible with providing separation of contact and non-contact flows.	G	15-Dec-11	B
North 40 Design Slope Constraints	The toe is generally constrained between operations and final reclamation due to location of state highway. Operations composite OSF outslopes to be compatible with the composite reclamation slope and stability constraints.	G	15-Dec-11	B
Operation Slope Criteria	Bench height: 200-ft (typical) Operation bench width: 200-ft (min) Interbench slope gradient: ~1.4H:1V (angle-of-repose) Composite slope: 2.4H:1V	C, G	8-Mar-12	C
OSF Staging	Intermediate OSF configurations to be compatible with stability requirements and water management strategy (i.e., separation of contact and non-contact flows).	C	8-Mar-12	C
Haul Roads	Minimum Total Width (w/berms) = 110-ft; 85-ft driving width; Maximum slope = 10%; Berms – MSHA compatible	G	8-Mar-12	C
Closure and Reclamation Criteria				
Concurrent Reclamation	To be completed to the extent practical and feasible	A	29-Nov-11	A
Reclamation Cover System Objectives	Minimize erosion, reduce infiltration, promote revegetation	G	29-Nov-11	A
Vegetation Species	< 11,800 ft – upland standard seed mix > 11,800 ft – alpine seed mix	C	29-Nov-11	A
Maximum OSF Outslope Interbench Gradients	2.0(H):1(V)	G	8-Mar-12	C

Client: Climax Molybdenum Company (Climax)	Discipline	Date	Rev. No.
Project Title: North 40 / McNulty Overburden Storage Facility			
Design Criteria: Revision D – Issued for Report			

Reclamation Bench (i.e., Outslope Channel) Width	Nominally 20 ft wide	G	8-Mar-12	C
Slope Length Between Reclamation Benches	Nominal 125 ft (+/- 10 ft) as measured parallel to slope	G	8-Mar-12	C
Reclamation Bench Gradients	Gradient 1% to 2% (2% typ.)	G	8-Mar-12	C
Maximum Bench Channel Length Between Downdrains	Approximately 1,800-ft unless justified	G	8-Mar-12	C
OSF Final Top Surface Grades	Between one and five percent backsloped, wherever feasible	G	8-Mar-12	C
Underdrain, Toe Drain, and CWCS System Criteria				
Underdrain (non-contact flows) FOS	Flow capacity to be designed with FOS = 10. Underdrain flows to be initially assumed, and modified based on monitoring during spring runoff completed by Climax. Provide liner system to keep non-contact status.	G	29-Nov-11	A
CWCS	Flow capacity to be designed with FOS = 2. Design flows to be initially calculated based on the assumption that the entire 100-yr, 24-hr storm infiltrates the OSF passes through the CWCS within 48 hours. Design inflows may be modified based on future monitoring and infiltration model predictions.	G	29-Nov-11	A
OSF Toe Drains	At closure, toe drains will be constructed where required along the toes of both OSFs. The toe drains are intended to capture contact water internal to the OSFs at those locations where there is potential for contact water to exit the toes of the OSFs as seepage.	C	3-Apr-12	D
Stability Criteria				
Stability FOS	Operations – Static Conditions: > 1.4; Operations – Seismic Conditions: > 1.0 (using Hynes and Franklin pseudo-static method and OBE); Reclamation – Static Conditions: > 1.5; Reclamation – Seismic Conditions: > 1.0 (using Hynes and Franklin method and MDE);	G	29-Nov-11	A

APPENDIX G

Date:	April 3, 2012	**Prepared by:**	D. Rugg
Project No.:	113-81608.2000	**Checked by:**	D. Geier
Project Title:	Climax Molybdenum OSF Design Report	**Reviewed by:**	B. Bronson
RE:	**CONTACT WATER PIPE SYSTEM DESIGN CALCULATIONS**		

This appendix presents the calculations performed to determine the appropriate pipe sizes for the Contact Water Collection System (CWCS) drainage pipes, to be installed at the Climax Mine (Climax Molybdenum/Freeport McMoRan) McNulty Gulch overburden storage facility (OSF). The purpose of the CWCS is to collect all water that has contacted and percolated into the OSF. This water will then be transferred to the toe of the OSF, where it will be discharged to pipes carrying the flows to the Climax water treatment facility.

1.0 ASSUMPTIONS AND GIVEN INFORMATION

Golder has designed the CWCS to collect and convey 100% of the 100-year, 24-hour design storm. The CWCS conservatively assumes 100% of the design storm will be conveyed in pipes, i.e. the calculations conservatively neglect flow through mine waste rock at the base of the OSF. Considerable segregation of mine waste occurs at the base of each mine waste lift as the coarsest rocks accumulate at the base of each lift with the “fines” deposited at the top of the lifts. The coarse rock rubble at the base of the OSF exhibits a high permeability which is neglected in the flow calculations. By definition, the entire 100-yr, 24-hour storm falls within a period of 24 hours. However, storm water contacting the OSF must percolate through the entire thickness of overburden material within the OSF before reaching the foundation of the facility, where the CWCS will be constructed. This process can take days, weeks, or months. As a result, the peak flow produced by the design storm will be significantly attenuated based on travel time through the OSF. For sizing the CWCS pipes, Golder has conservatively assumed the entire design storm will report to the CWCS pipes within a period of 48 hours.

The CWCS piping system will contain primary and secondary pipes. There will be two main primary pipes, one within the main McNulty Gulch north fork, and one within the main McNulty Gulch south fork. Secondary pipes will be smaller, and will be placed in all side drainages reporting to the two main forks of the McNulty Gulch. Pipe sizes have been calculated for both the primary CWCS pipes and the secondary CWCS pipes. A list of other assumptions and design criteria used to size the CWCS piping is provided below:

- 100 year, 24 hour design storm: 1.99 in
- Assume 100% of this storm infiltrates the OSF

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- Assume this flow must be conveyed through pipes within 48 hours
- Assume pipes lie on worst case (lowest gradient) slope of 2.5%
- Assume zero flow contribution from drain rock or mine overburden
- Maximum total collection area for primary CWCS pipes: 207.2 acres
- Maximum total collection are for secondary CWCS pipes: 51.4 acres
- Total infiltration volume for primary CWCS pipes: 1,496,540 cubic feet
- Total infiltration volume for secondary CWCS pipes: 370,987 cubic feet

Therefore,

- Primary CWCS pipe design flow: 8.66 cfs
- Secondary CWCS pipe design flow: 2.15 cfs
- Assume all pipes are perforated corrugated polyethylene (PCPE)
- Design all pipes with a minimum factor of safety (FOS) of 2.0

2.0 CALCULATIONS

- Maximum pipe flow capacity was calculated using Manning's equation:

$$Q = \frac{1.49}{n} S^{1/2} A R^{2/3}$$

where: Q = Flow rate (cfs);

n = Manning's roughness coefficient = 0.012 for polyethylene pipes;

S = Slope = 2.5%;

A = Area of pipe (ft^2);

For 18 inch PCPE = 1.77 square feet;

For 10 inch PCPE = 0.55 square feet;

R = Hydraulic radius (ft);

For 18 inch PCPE = 0.38 ft;

For 10 inch PCPE = 0.21 ft.

- Therefore the maximum conveyance is:
  - 18 inch PCPE: 18.04 cfs
  - 10 inch PCPE: 5.32 cfs

- Calculate the FOS:
  - 18 inch PCPE: 18.04 cfs / 8.66 cfs = 2.1
  - 10 inch PCPE: 5.32 cfs / 2.15 cfs = 2.5

3.0 CONCLUSIONS

The calculations show that a single 18 inch diameter PCPE pipe will be sufficient for use as the primary collection pipe for the McNulty Gulch OSF CWCS. 10 inch diameter PCPE pipe can be used in all locations requiring secondary pipes. All pipes will be bedded with drain gravel.

OPERATION AND MONITORING PLAN

NORTH 40 AND MCNULTY GULCH OSF OPERATION AND MONITORING PLAN

REVISION 0

Climax Molybdenum Company – Climax Mine

Revision				Pages Revised	Remarks
No.	By	App.	Date		
0	DLG	BB	17-May-12	ALL	

May 2012 **113-81608**

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1.0 INTRODUCTION

1.1 Purpose

This report presents the operation and monitoring (O&M) plan for the proposed North 40 and McNulty Gulch overburden storage facility (OSF) expansions. This O&M plan was developed in support of the design and permitting process by the OSF Engineer of Record (Golder Associates Inc.). Both OSFs are located at the Climax Molybdenum Mine (Climax).

The O&M plan is intended to provide O&M protocol to ensure that the Climax OSFs are stable and properly managed during the life of mine operations. This O&M plan also is intended to be a living document that is periodically reviewed and updated to ensure it includes appropriate consideration of changing conditions and practices.

2.0 PERFORMANCE AND OPERATIONAL CONSIDERATIONS

2.1 OSF Development and Design Criteria

OSF development and project design criteria provide the design basis for the Climax OSFs and are included in the Design Report (Golder 2012) as Appendix F. The design criteria include the OSF development criteria, closure and reclamation criteria, stability criteria, and water management strategies.

2.2 Construction on Steep Foundations

The investigations and analyses that have been carried out for the Climax OSFs support that after these facilities are completed, they will have adequate long-term stability. Experience at other mines with OSFs also supports the conclusion that facilities of this size can be safely constructed and will be stable in the final configuration. However, although detailed stability analyses of the final OSF configurations have confirmed the long-term stability of the facilities, during the early and intermediate stages of development there may be locations where the OSF operating slopes will be at an angle of repose and locally supported on steep foundation slopes. Since OSF slope and height relative to foundation slope influence the likelihood of failure, attention will need to be paid when increasing the height of the OSF for a given foundation slope during material placement to minimize the potential for failures. This O&M plan is intended to ensure that safe, long-term development occurs through implementation of a detailed OSF monitoring and operation plan, and to provide a protocol that allows for advance warning of potential failures.

2.3 Water Management

The Climax Mine site receives precipitation throughout the year, with winter precipitation occurring as snow, and summer precipitation occurring as rain. The strategy for water management is to maintain the separation of contact and non-contact water, and to minimize the creation of contact water after closure, to the extent practicable. Groundwater seeps beneath the McNulty OSF will be captured by the underdrain system. While the underdrain system is primarily designed to maintain separation between groundwater and OSF materials, an important secondary function is to minimize the potential for excess pore pressures at the OSF foundation. The extent of the watershed upgradient of the OSF during operations is limited, and at closure perimeter channels will prevent run-on to the OSFs. Perimeter channels will also control runoff of contact water during operations. In addition to the underdrain system and control of upgradient run-on, a Contact Water Collection System (CWCS) has been designed to capture infiltration that may occur and migrate through the OSFs and report at the OSF toe area. The CWCS will ensure that saturated conditions do not develop above the underdrains and will also be used at closure to prevent contact water from exiting the toe of the OSF as seepage and entering the non-contact water management system.

2.4 Snow

In most years, the Climax mine receives significant accumulations of snow. Experience in other high snowfall areas shows that accumulations of up to about 2 to 3 feet of snow on OSF faces do not significantly impact the operation of large OSFs. Thin layers of snow are typically disrupted and pushed from the face as material placement occurs. Larger accumulations (e.g. in excess of 3 feet) may be trapped as a continuous layer within the OSFs and form a weak layer or saturate finer grained layers as melting occurs. The North 40 and McNulty OSFs face in different directions, and snow drifting and accumulation is likely to occur more on one OSF than the other, depending on wind directions. This should afford the mine the option of operating the OSF faces with the least accumulations of snow.

Similar to stockpile faces, OSF construction experience at other high snowfall mines is that thin layers of snow (e.g. less than 3 feet) that are present in the toe areas are disrupted and integrated into the waste when constructed using haulage truck placement techniques. However, placement of waste over thicker accumulations of waste can result in development of a weak ice layer at the toe of the OSF, that can result in localized failures and displacements. Therefore, thick accumulations of snow in excess of 3 feet thickness are to be removed via bulldozers or other methods where present on the foundation toe areas, prior to advancement of the OSF onto these areas.

In addition, snow accumulation in surface water channels impacts their performance once snow melt begins in the spring. Therefore, Climax will clear snow from the OSF storm water channels as needed in order to ensure that the OSF storm water channels operate as designed.

3.0 OPERATIONS AND MONITORING STRATEGY

OSFs are large landforms that are constructed over a period of many years. Given the size of these structures, it is not considered practical at the design phase to consider the stability and geotechnical conditions for all of the various intermediate configurations and conditions that will occur during OSF construction. As a result, the focus of OSF stability evaluations is typically centered on ensuring long term stability of the final configuration. Based on the available data and experience, OSF design criteria, and a "living" O&M plan approach, the observational method proposed by Karl Terzaghi is appropriate and relevant for use to modify the OSF design during the life of the mine. Terzaghi (Peck, 1969) described the observational method as:

> *The procedure is as follows: Base the design on whatever information can be secured. Make a detailed inventory of all the possible differences between reality and the assumptions. Then compute, on the basis of the original assumptions, various quantities that can be measured in the field. For instance, if assumptions have been made regarding pressure in the water beneath the structure, compute the pressure at various easily accessible points, measure it, and compare the results with the forecast. Or, if assumptions have been made regarding stress-deformation properties, compute displacements, measure them, and make a similar comparison. On the basis of the results of such measurements, gradually close the gaps in knowledge and, if necessary, modify the design during construction.*

The method is dependent upon the ability to modify the design and construction method or rate of construction during the construction period. OSFs are well suited to this approach.

It is anticipated that the Climax mine planning department will revise the mine plan throughout the life of the mine as more information is developed on the ore body and as the economics of the deposit change. The OSF plans will also be revised as an understanding of the performance of the OSF is developed. The guidelines that are presented in this document set out the current understanding of the critical constraints for stability of the OSF that must be incorporated in each stage of the OSF development plan.

It is also anticipated that the OSF Engineer of Record will periodically review the performance of the OSFs with the Climax operations personnel. Based on these reviews, the O&M plan will be updated with tracked reviews. The focus of many of these guidelines is to protect the personnel and equipment working on and below the OSFs and to minimize the risk to infrastructure located below the OSFs during the life of the mine.

3.1 Construction Quality Control / Quality Assurance

Climax will implement a construction quality assurance and construction quality control program (QA/QC program) in order to verify that critical components of the OSFs are constructed in accordance with the design drawings and project specifications, and that the intent of the design is met. The QA/QC plan for water management controls will be finalized prior to construction. Components to be included in the operations QA/QC program for water management controls include:

- Underdrain system, including pipes, drain fill, overliner fill, low permeability bedding fill, geomembrane, and geotextile components
- CWCS system, including pipes, drain fill, and geotextile components
- Energy dissipators, including bedding fill, geosynthetic components, and articulated block
- Concrete manholes
- Water management berms, including structural fill and rock armor
- Perimeter channels, including bedding fill and riprap

The QA/QC plan will include several components:

- Develop technical specifications for construction materials and methods
- Verify contractor qualifications
- Review manufacturers data and quality control certificates
- Maintain inventory of construction materials
- Perform field and laboratory testing to confirm material properties and construction methods meet project specifications
- QA during construction for compatibility with design objectives and method specifications
- Create photographic log of construction activity
- Verify that lines and grades are as shown on the drawings
- Document design clarifications, modifications, and changes
- Maintain open lines of communication between Climax, the Engineer of Record, the contractor, and other parties involved in construction
- Identify and verify correction of deficiencies
- Provide periodic progress reports
- Provide as-built drawings and report

Climax will develop a separate QA/QC plan prior to closure, which will include additional components not present in the operational design. The O&M plan will be updated to include additional information regarding the closure QA/QC program once the program is developed.

The QA/QC components for OSF construction are incorporated into the material placement designs and in the monitoring program. These are described in the subsequent sections of this O&M plan.

3.2 Direction of Crest Advance

Both the foundation slope and degree of confinement afforded by the shape of the foundation will affect the stability of an advancing OSF. The stability of the advancing face will be maximized if the toe of the

OSF is supported on rising topography. If this is not practical, the next most stable configuration will be a toe advancing along the topographic contours. Where overburden placement must occur down a slope, such as at the start of a new OSF or wrap-around lift, stability can be increased by placement of overburden material down the axis of a drainage to take advantage of the three-dimensional wedging effect. Once the toe is supported on the flatter slopes at the bottom of the valley, the direction of advance can be turned so that the toe advances along the topographic contours on the valley walls and the toe advances down the axis of the valley.

3.3 Overburden Placement Based on Material Type

The majority of the waste rock to be placed within the North 40 and McNulty Gulch OSFs is expected to be durable igneous and metamorphic rock excavated from the ore zone and areas east of the Mosquito Fault. However, approximately 30% of the waste material will consist of sedimentary rock. To the extent that it is feasible, sedimentary and weaker igneous overburden materials should be placed such that:

- Only minor amounts of the minority constituent material should be placed within 200 feet of the ultimate toe of the OSFs
- Only minor amounts of the minority constituent material should be placed in the lowermost operating lift within 100 feet of the centerline of any valleys, drainages, or gulches

3.4 OSF Monitoring

Slope failures of OSFs are typically preceded by warning indications such as an increased rate of deformation, increased rate of cracking of the OSF platform, bulging of the OSF face, cracking and bulging at the OSF toe or increased rate of pore water pressure buildup in the OSF foundation.

The purpose of a OSF monitoring program is:

- To provide early identification of conditions that could lead to failure so that preventative measures can be taken;
- To provide early warning of impending failure so that personnel and equipment can be removed from the area at risk; and
- To collect and assess data that will confirm or negate the assumptions made during the design studies and to provide data that will allow the design of the OSF to be modified during the life of the mine to improve the performance of the OSF.

A comprehensive OSF O&M program consists of regular visual inspection and measurement of the crest displacement rate by the operating personnel, and periodic inspection and ongoing assessment of the accumulated data by the mine geotechnical engineer. Extensive instrumentation of the OSF foundation is typically not practiced for large OSFs, but due to the proximity of Colorado Highway 91, Climax mine is instituting a program for monitoring movements of the foundation along the downgradient margins of the OSFs.

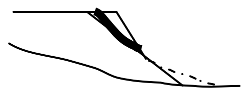
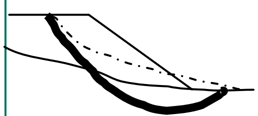
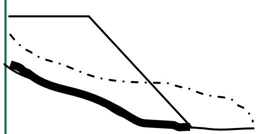
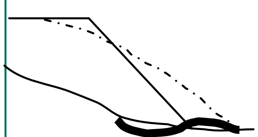
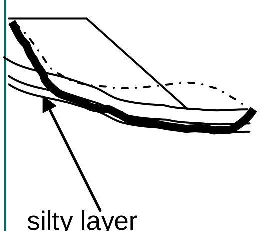
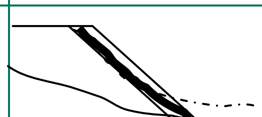
3.4.1 Visual Inspections

Visual observations are critical to maintaining a safe overburden placement operation. Visual indicators of problem areas in an OSF are:

- Excessive surface cracking;
- Safety berm movement;
- Surface buildup required;
- Bulging of the OSF face;
- Toe or foundation creep and bulging (movement); and
- Changes in the rate or quality of seepage from the OSF toe.

Visual inspection is the most common and the most practical method of OSF monitoring. Climax engineering, survey, and operations staff as well as equipment operators visit the OSF frequently and should be trained to look for and recognize signs of instability. A description of failure types, causes, and warning signs is provided in Table 1.

Table 1: Failure Types Causes and Warning Signs

Failure Type	Diagram	Usual Cause	Likely Effects	Warning Signs
Sliver Failure		Oversteepened crest due to high fines content, rapid placement rate, wet material.	Small scale crest failure, subsidence at crest of stockpile.	Crest cracking, subsidence near crest, steep slope below crest, increasing crest displacement rates.
Foundation Failure		Weak material in foundation. Rapid loading rate, high pore pressure in foundation.	Can cause large failure involving significant part of stockpile outslope.	Seepage at toe, bulging or spreading of stockpile toe, cracks on stockpile top surface well behind crest.
Overall Failure		Weak material along base of stockpile, poor drainage along base of stockpile, steep foundation, rapid loading rate.	Entire stockpile fails along base.	Cracking of stockpile surface back as far as contact with natural ground. Settlement of entire platform. Bulging of foundation soils and stockpile material at toe.
Toe Failure		Weak foundation material at toe. High pore pressures at toe. Steep slope at toe. Rapid loading rate.	Loss of support of toe. May lead to propagation of failure up slope.	Spreading of toe. Yielding and bulging of foundation soils and stockpile material at toe. Cracking of stockpile platform near crest. Increased rate of crest displacement.
Foundation Liquefaction		Silty to sandy material in foundation, possibly confined by aquitards. Pore pressure build-up due to rapid loading or earthquake loading.	Possible major failure of significant portion of the stockpile with large runout distance. May occur on flat foundation.	High piezometric pressures in foundation. In some case, sand boils may be present prior to complete failure.
Planar Failure		Weak plane in stockpile material approximately parallel to stockpile face due to poor material.	May involve large amount of material with large runout distance.	Slumping of stockpile crest, bulging of toe or face, cracks on platform well behind crest.

An OSF log book should be maintained for each active lift. As an alternative to an OSF log book, an OSF shift inspection form is acceptable. An example inspection form is provided as Table 2A. Documentation of the visual inspections should be reviewed and maintained by the mine department. The engineer responsible for the OSF also should perform periodic visual inspections and review the log books on a regular basis. An example of a mine engineering inspection form is provided as Table 2B. Examples of information that could be included, as appropriate, in the OSF log or inspection forms are as follows:

- Lift designation;
- Date;
- Plan map based on most recent survey information available with sketches showing current and approximate crest and toe locations;
- Active placement crest length;
- Monitoring instrumentation in use;
- Irregularities noted, for instance, new crack formation;
- Approximate rate of crest advance;
- A description of the material being stockpiled;
- Weather conditions;
- Any maintenance or repairs needed for water controls;
- Current restrictions or instructions regarding lift development; and
- Conclusions regarding investigations or discussions of any special conditions brought to the attention of the foreman by anyone who had visited the OSF during the shift.

Detailed inspections of the various lifts of the OSF should be carried out to ensure that any changes or irregularities regarding performance, appearance or construction methods are noted and that any maintenance items identified by the operations foreman are repaired and/or maintained. The detailed OSF inspection is to be completed by the Climax mine engineer responsible for OSF monitoring and documented. It is recommended that the Engineer of Record attend the detailed OSF inspection at a minimum of once per year. Inspection reports should be maintained by the mine engineering department. Photographic records should be taken to document the development of the OSFs.

During the initial stages of OSF construction, when material placement is occurring on relatively steep slopes, the detailed OSF inspections should be carried out at least monthly. After the OSF is well established and the critical constraints on OSF performance are better understood and material placement is occurring mostly onto lower lifts, the frequency of detailed OSF inspection can be reduced to a quarterly inspection.

3.4.2 Crest Displacement Monitoring

OSFs rarely fail without advanced warning as they are preceded by accelerating displacements, surface cracking, and/or toe bulging. Rather, early detection of potential failure is determined by the visual inspections conducted on each shift. Should visual inspection indicators be observed that indicate a potential OSF failure, the following actions will be taken:

- The shift foreman will notify personnel that all mining operations will cease at this location and be moved to an alternate and stable OSF platform;
- The mine manager will be notified; and,
- Material placement on the OSF will not resume at the platform where visual instability indicators were observed until crest displacement monitoring methods (e.g. wireline extensometers, lidar, prism surveys etc.) can be performed.

OSF crest displacement monitoring using wireline extensometers or other alternative systems (e.g. prism monitoring, lidar, etc.) has been proven to be a simple and effective method of providing advance warning of impending failure. A typical set up of a two-stand wireline extensometer is shown in Figure 1.

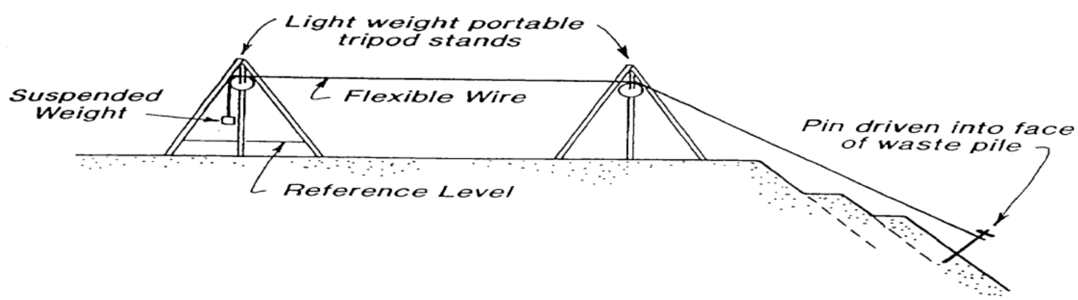


Figure 1: Typical Wireline Extensometer

In the event that Climax mine operations elect to use wireline extensometers to monitor areas identified by visual inspections as potentially exhibiting excessive and/or accelerating displacement, the extensometers should be spaced at about 300 feet intervals along the active OSF crest. The rear tripod, with the weight and measurement scale, must be located behind any active cracks near the OSF crest. In areas of active material placement, the wirelines should be read at about 4-hour intervals and the rate of displacement calculated immediately. The wirelines on inactive OSFs should be read once per shift. The results are to be plotted so that any trend of accelerating crest displacement can be noted. An initial threshold displacement rate of 2 in/hr, measured with wireline extensometers near the crest of the OSFs, should be considered, with the expectation that the threshold rate will be increased to 4 to 8 in/hr as performance data is collected and the OSF toe reaches the flatter slopes at the valley bottom. The rate can be adjusted as OSF performance data are accumulated and assessed and as the active face heights of the OSFs increase. A section of OSF that has been closed due to high crest displacement rate should not be reactivated until the crest displacement rate is less than the critical rate for a period of 24 hours. The plots of the monitoring data should be reviewed by the engineer responsible for the OSFs on a daily basis.

Should Climax elect to use alternatives to wireline extensometers for crest displacement monitoring, similar protocols will be developed for the specific methods used.

3.4.3 Foundation Displacement Monitoring

Climax will install semi-permanent GPS monuments at several critical locations along the toe of the OSFs. These monuments will periodically record their position. Because the monuments will be founded adjacent to the OSF toe, displacements measured at the monuments will be indicative of movement of the OSF foundation. The monuments are intended to provide warning or indications of major, deep-seated failures of the OSF occurring through the foundation. The GPS monuments are not intended to and will not provide warning or indications of minor failures or creep occurring within the OSF mass, because these types of movement will not cause displacements of the OSF foundation.

The GPS monuments are to be placed approximately 40 feet from the proposed toe of the OSFs and be protected with bollards. This offset distance is intended to provide protection from loose rocks rolling off of the OSF facilities, while still remaining close enough to the OSF to provide indications of foundation movement. The North 40 OSF will be developed prior to that of the McNulty OSF. Therefore, four (4) monuments will initially be installed along the toe of the North 40 OSF to assess the foundation stability of the facility. Preliminary locations for the monuments are shown on Figure 2. If the proposed footprint of the OSF changes, the monuments may need to be re-located. Additional GPS monuments may be installed adjacent to the McNulty OSF as overburden loading activity shifts to that facility. Alternatively, some of the North 40 GPS monuments may be relocated to the McNulty OSF provided that a history of measurements is established for the North 40 OSF showing no indications of slope failure. It should be noted that the relative importance of each monument will change throughout the life of the OSF, depending on where and when active overburden placement within the North 40 area occurs.

During active material placement, the monument readings and locations are to be recorded at a minimum of once per day. If the displacement rate exceeds 1 inch/month (0.03 inch/day), or if the measurements show a trend of accelerating displacement, material placement activity should be moved to a different section of the OSF until displacement rates stabilize. If a section of foundation repeatedly shows accelerating displacements attributable to active material placement, the Engineer of Record is to be notified and the OSF design for that section should be re-evaluated. The results are to be plotted so that any trend of accelerating crest displacement can be noted.

3.4.4 Underdrain and CWCS Monitoring

After construction of the underdrain and CWCS systems, Climax will periodically measure flows from both systems. Flow rates can be used to assess the performance of the systems. Flow measurements will initially occur on a monthly basis, with monitoring frequency subject to change in the future. Monitoring should also include periodic water quality monitoring to verify suitability of underdrain flows for inclusion in the non-contact water circuit.

3.5 Restricted Access Areas

Access will be restricted in the area below the active OSF lift faces. No persons or equipment should enter the boulder roll out area as defined below in Figure 3. The restricted area should be taken as that area within a line extending downward from the lowest OSF lift crest at 23 degrees from the horizontal. Access should also be restricted in the area below OSF areas that have been closed due to high crest displacement rates or visual signs of impending failure.

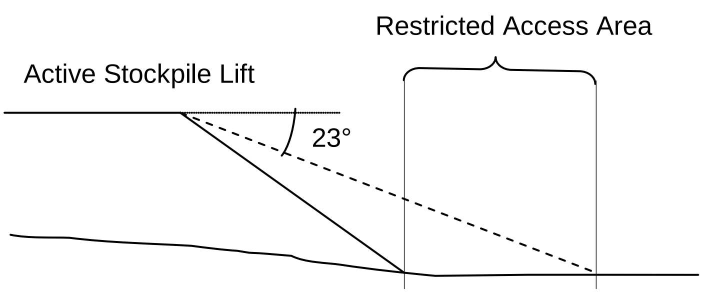


Figure 3: Illustration of Restricted Area Limits

3.6 Back Analysis Report

Small-scale, localized failures can occur during OSF construction, particularly during the early stages of material placement and if material is placed on steep foundation slopes. These types of failures (e.g., surface cracking and minor displacement along OSF crests) pose relatively low risk to human health and the environment. Should a more significant failure occur during development of the OSFs, there will be a need for detailed reporting, evaluation, and relevant back analysis of the event. This analysis will provide for a more thorough understanding of the critical constraints and displacement thresholds to developing stable OSFs at Climax, and will lead to better practice to avoid future occurrences. Significant failures, as defined herein, include any failures that involve a failure plane with a vertical depth of ten (10) feet or greater and/or with a runout that exceeds fifty (50) feet.

Back analysis of significant failures is a useful means of calibrating models of OSF performance. At the time of failure, the geometry of the failure surface and the factor of safety are known with certainty. Parametric studies should be carried out to assess the relative importance and possible values of the piezometric conditions and shear strengths and to refine the assumptions that were made during initial design.

The back analysis report must be a clear and complete factual record of the failure that will provide the information required to understand the cause and mechanism of the failure and to allow back analysis of the failure. A discussion of the information that should be provided in the failure report follows.

3.6.1.1 Failure Location and Date

The location and elevation of the failure should be provided in terms of the mine coordinate grid, and both the date and time of the failure should be recorded.

3.6.1.2 Geometry of Failure

Whenever possible, detailed topography of the natural ground as well as both pre- and post-failure OSF surfaces and debris should be provided. If detailed topography is not available, crests and toes and as many intermediate points as practical should be provided. A profile through the center of the failure and along the centerline of the runout, including the deposition area, should be prepared. Where different types of material are involved in the failure, an estimate of the relative percentages of each material, and the location in the debris should be provided. Foundation conditions should be described in as much detail as possible. This should include soil types and piezometric conditions.

3.6.1.3 Volume of Failure

The volume of failed material should be estimated from the topographic and profile information.

3.6.1.4 Runout

Runout should be described by the distance from the original OSF toe to the farthest limit of the debris and by the runout angle. The runout angle is measured as the angle below the horizontal from the pre-failure OSF crest to the farthest limit of the debris. Runout angle should be measured on a vertical section along the centerline of the flow path.

3.6.1.5 Material Description

A full description of the failed material should be provided. This should include the material type and grain size of the debris and material on the failure surface. Variations of material type along the debris run path should be recorded. Photographs should be taken from various locations and at various levels of detail. The moisture content of the debris and material left behind on the failure surface should be estimated.

3.6.1.6 Monitoring Data

Complete monitoring records for the period preceding failure should be included. These would include the records of visual observations as well as quantitative data from the crest displacement monitors.

3.6.1.7 Weather Data

Rain and temperature data should be provided for the period leading up to the failure.

3.6.1.8 Material Placement Rates

Material placement rates prior to failure should be provided. If detailed records are not available, rates of crest advance should be estimated by comparison of periodic surveys of crest and toe locations.

3.6.1.9 Seismic Information

Documentation should be provided of any earthquake activity which may have had an effect on stability.

4.0 REFERENCES

Golder, 2012: North 40 and McNulty Gulch Overburden Storage Facility Design Report, Dated April 2012.

TABLES

Table 2A
Example Inspection Form - Mine Operations Daily or Shift Inspection

Operations Information	
Inspection Date / Time	
Inspector Name	
Stockpile Name	
Active Lift Designation (i.e., 11,640 ft)	
Inspection Information	
Approximate Material Placement Rate (Tons or Trucks Per Shift or Day)	
Approximate Crest Length Being Used For Material Placement)	
Approximate Lift Height	
Description of Material Being Dumped	
Weather Conditions	
Visual Indicators of Potential Problems (Excessive Surface Cracking, Safety Berm Movement, OSF or Foundation Bulging, Excess Erosion, Excess Settlement, etc)	
Description of Any Seepage From the OSF or Foundation Areas	
Description of Maintenance or Repair Needs Identified	
Desription of Any Maintenance or Repairs Performed	
Current Restrictions on Material Placement or Instructions Regarding Lift Development	
Summary of Discussions With or Concerns Identified by Climax Personnel and/or Visitors	
Were Any Photographs, Sketches, or Additional Notes Made? If Yes, Attach	
Documentation of Review	
Signature of Shift Foreman / Mine Engineer	

Golder Associates

Table 2B
Example Inspection Form - Mine Engineering Monthly Inspection

Operations Information	
Inspection Date / Time	
Inspector Name	
Stockpile Name	
Active Lift Designation (i.e., 11,640 ft)	
Inspection Information	
Wireline Extensometer Displacement Rates	
GPS Monument Displacement Rates	
Visual Indicators of Potential Problems (Excessive Surface Cracking, Safety Berm Movement, OSF or Foundation Bulging, Excess Erosion, Excess Settlement, etc)	
Description of Any Seepage From the OSF or Foundation Areas	
Description of Maintenance or Repair Needs Identified	
Desription of Any Maintenance or Repairs Performed	
Current Restrictions on Material Placement or Instructions Regarding Lift Development	
Summary of Discussions With or Concerns Identified by Climax Personnel and/or Visitors	
Were Any Photographs, Sketches, or Additional Notes Made? If Yes, Attach	
Documentation of Review	
Signature of Mine Engineer	

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FIGURES

LEGEND

- 11975 / 11950 — EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
- PROPOSED WASTE ROCK STOCKPILE TOPOGRAPHY (SEE REFERENCE 3)
- OSF BOUNDARY
- PROPOSED GPS MONITORING STATION

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. AERIAL PHOTOGRAPH TAKEN IN 2009, FROM USDA AERIAL PHOTOGRAPHY FIELD OFFICE.
3. PRELIMINARY WASTE ROCK STOCKPILE TOPOGRAPHY PROVIDED BY CLIMAX MINE, 24 APRIL 2012.

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
B	19APR12	ISSUED FOR REPORT	DLG	DLG	BRB	BRB
A	04APR12	ISSUED FOR REPORT	DLG	DLG	BRB	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES, COLORADO

TITLE

PRELIMINARY LOCATIONS FOR GPS MONUMENTS

Golder Associates
DENVER, COLORADO

FIGURE 2 - GPS LOCATIONS - REV2

PROJECT No. 113-81608

FIGURE 2

Drawing File: J:\1JOBS\113-81608 Climax\Design Report\Rev3\AppH - GM\supporting docs\ Figure 2 - GPS locations - Rev2.dwg | Layout: 11x17 Landscape | Modified: May 30, 2012 10:45 DGEIER | Plotted: 05/30/12 10:46am DGEIER

OVERBURDEN STORAGE FACILITY DESIGN DRAWINGS

CLIMAX MINE
LAKE AND SUMMIT COUNTIES
COLORADO

PREPARED FOR:
CLIMAX MOLYBDENUM

LIST OF DRAWINGS

GENERAL LEGEND

℄	CENTERLINE
CWCS	CONTACT WATER COLLECTION SYSTEM
EL.	ELEVATION
in	INCHES
MAX.	MAXIMUM
MIN.	MINIMUM
NOM.	NOMINAL
N.T.S.	NOT TO SCALE
OSF	OVERBURDEN STORAGE FACILITY
P.E.	POLYETHYLENE
TYP.	TYPICAL
ft	FEET
TBD	TO BE DETERMINED
2.5:1	2.5 HORIZONTAL TO 1 VERTICAL SLOPE
°	DEGREE
~	APPROXIMATELY
DETAIL NUMBER; DRAWING NUMBER WHERE DETAIL IS SHOWN	DETAIL CALL–OUT
A / 5: SECTION LETTER; DRAWING NUMBER WHERE SECTION IS SHOWN; LINE WHERE SECTION IS CUT	CROSS–SECTION CALL–OUT

ENGINEER'S STAMP

COLORADO REGISTERED BRENT R. BRONSON 34969 PROFESSIONAL ENGINEER

5-17-12

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES, COLORADO

TITLE

COVER SHEET

Golder Associates

FILE No. 11381608H001
PROJECT No. 113–81608

1

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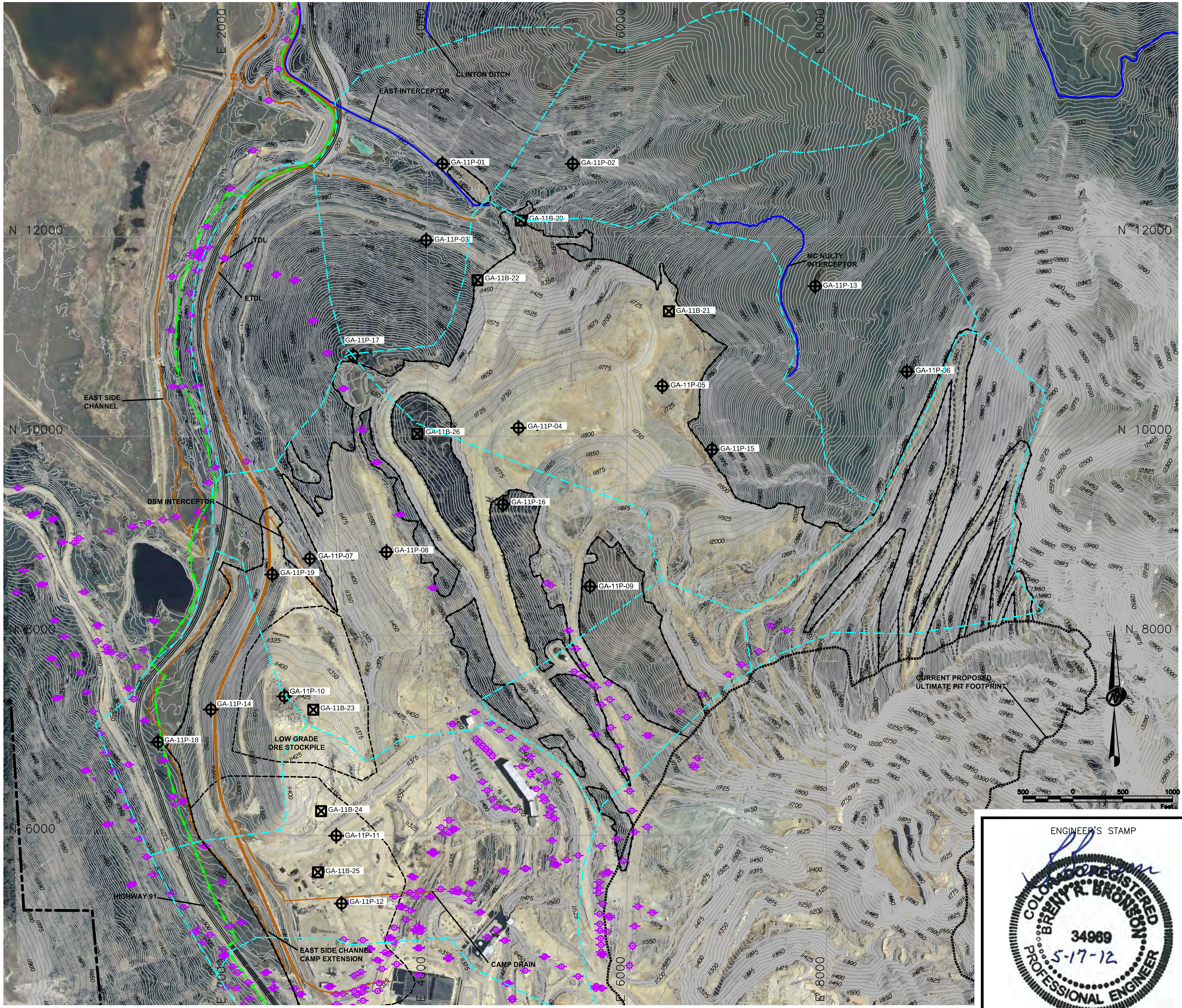

LEGEND

- EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
- HYDROLOGIC SUB-BASINS
- EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 4)
- EXISTING CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 4)
- EXISTING HIGH PRESSURE GAS LINE (NOTE 3)
- APPROXIMATE LIMITS OF BURIED HISTORIC TAILING IMPOUNDMENTS
- APPROXIMATE LIMITS OF RECENT OVERBURDEN AND/OR FILL
- CLIMAX PROPERTY LIMITS
- EXISTING ABOVE GROUND ELECTRIC POWER POLE (SEE REFERENCE 4)
- GA-11B-21 GOLDER 2011 TEST PIT LOCATION
- GA-11P-06 GOLDER 2011 BOREHOLE LOCATION

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. AERIAL PHOTOGRAPHY TAKEN IN 2009, FROM USDA AERIAL PHOTOGRAPHY FIELD OFFICE.
3. CLIMAX MINE PROVIDED HIGH PRESSURE GAS LINE LOCATION 18 JANUARY 2010.
4. EXISTING CHANNELS, EXISTING ABOVE GROUND ELECTRICAL, AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

REV	DATE	DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

EXISTING CONDITIONS

Golder Associates

FILE No.	11381608H002
PROJECT No.	113-81608

2

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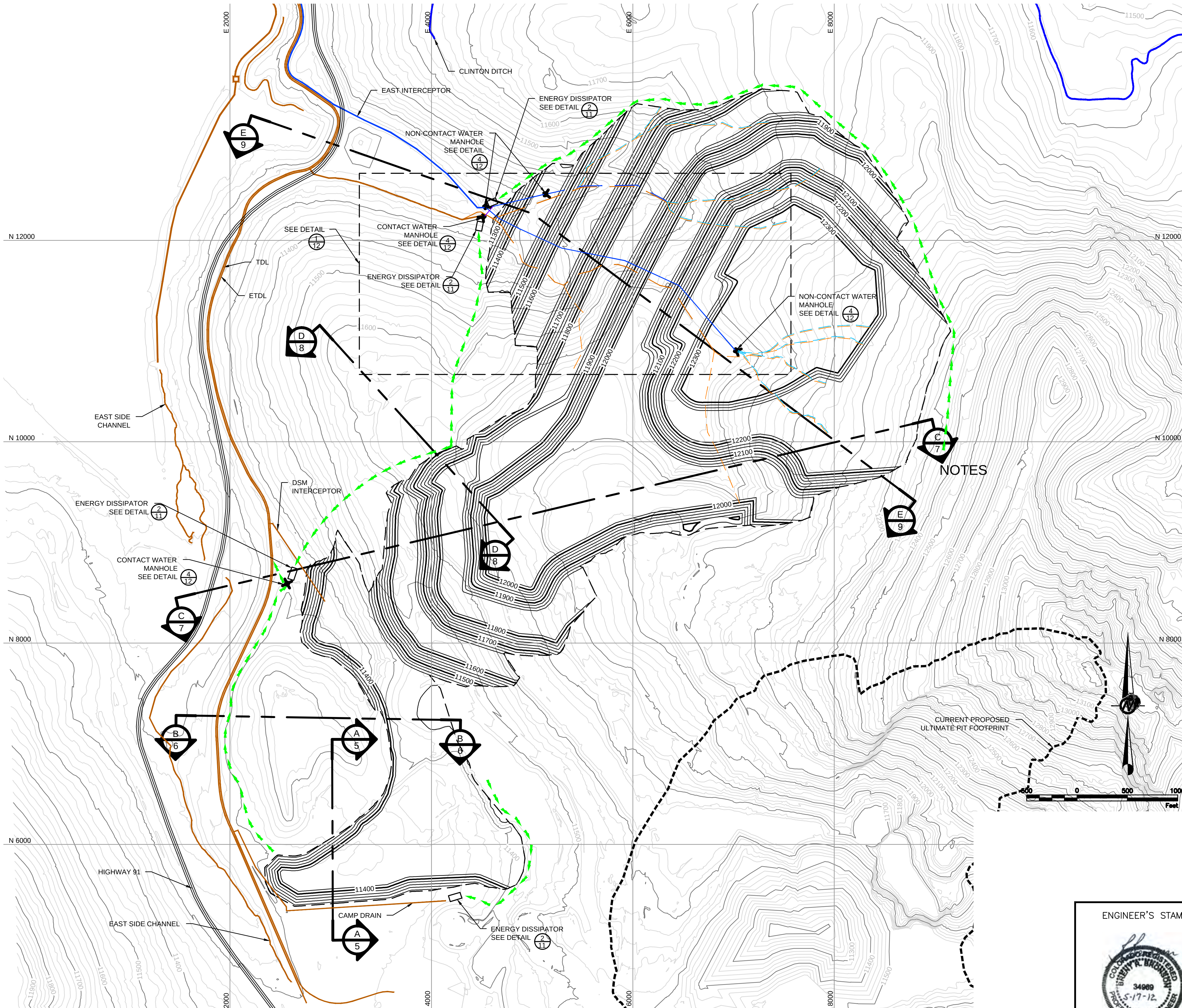

LEGEND

- 11500 — EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
- 11500 — PROPOSED OPERATIONS GRADING (SEE REFERENCE 3)
- PROPOSED OSF BOUNDARY
- OPERATIONS PERIMETER CHANNEL OR WATER MANAGEMENT BERM (SEE DETAILS) 2/12 AND 3/12
- EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 2)
- EXISTING CONTACT WATER CHANNELS / PIPES
- PROPOSED PRIMARY NON-CONTACT HDPE PIPE LOCATIONS (NOTE 2)
- PROPOSED PRIMARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
- PROPOSED SECONDARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
- PROPOSED PRIMARY CONTACT WATER PIPE LOCATIONS (NOTE 2)
- PROPOSED PRIMARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
- PROPOSED SECONDARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
- PROPOSED UNDERDRAIN BYPASS TO CONTACT MANHOLE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.
2. THE PROPOSED UNDERDRAIN AND CONTACT WATER LAYOUT AND CONCEPTUAL DESIGN DETAILS ARE PROVIDED TO ILLUSTRATE THE GENERAL WATER MANAGEMENT STRATEGY. THE DESIGNS WILL BE DETAILED AND FINALIZED UPON CONFIRMATION OF THE DESIGN APPROACH.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. EXISTING CHANNELS AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.
3. PROPOSED OPERATIONS GRADING PROVIDED BY CLIMAX MINE 23 APRIL 2012.

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER BRENT R. BRONSON 34989 5-17-12

TITLE

OSF PLAN - END OF OPERATIONS

Golder Associates

FILE No. 11381608H003
PROJECT No. 113-81608

3

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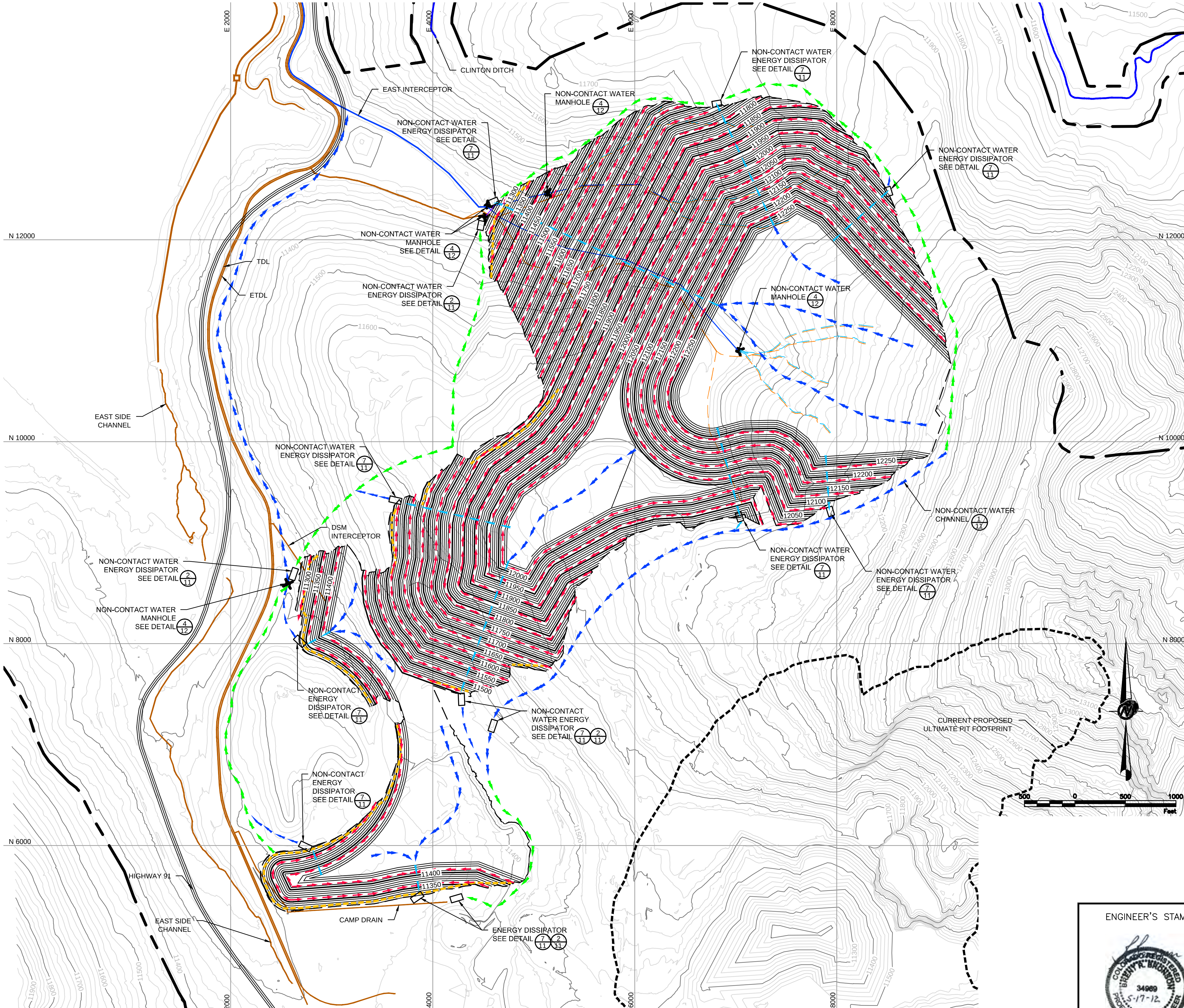

LEGEND

11500	EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
11500	PROPOSED CLOSURE GRADING (SEE NOTE 3)
	PROPOSED OSF BOUNDARY
	CLIMAX AFFECTED LANDS BOUNDARY
	OPERATIONS BERM OR CHANNEL TO BE CONVERTED TO A CLOSURE BERM OR CHANNEL (SEE DETAILS) 1/11 AND 6/11
	PROPOSED PERIMETER/TOPSURFACE BERM OR CHANNEL TO BE CONSTRUCTED AT CLOSURE (SEE DETAILS) 1/11 AND 6/11
	PROPOSED DOWNDRAIN CHANNELS TO BE CONSTRUCTED AT CLOSURE (SEE DETAIL) 3/11
	PROPOSED OUTSLOPE CHANNELS TO BE CONSTRUCTED AT CLOSURE (SEE DETAIL) 4/11
	EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 2)
	EXISTING CONTACT WATER CHANNELS / PIPES
	PROPOSED TOE-DRAIN
	PROPOSED PRIMARY NON-CONTACT HDPE PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED PRIMARY CONTACT WATER PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED UNDERDRAIN BYPASS TO CONTACT MANHOLE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.
2. THE PROPOSED UNDERDRAIN AND CONTACT WATER LAYOUT AND CONCEPTUAL DESIGN DETAILS ARE PROVIDED TO ILLUSTRATE THE GENERAL WATER MANAGEMENT STRATEGY. THE DESIGNS WILL BE DETAILED AND FINALIZED UPON CONFIRMATION OF THE DESIGN APPROACH.
3. THIS CONCEPTUAL OSF CLOSURE PLAN IS PROVIDED TO ILLUSTRATE THE ANTICIPATED FINAL CLOSURE TOE LIMITS AND CLOSURE STRATEGY FOR THE CLIMAX OSF'S. THE BARBER-POLE CLOSURE STRATEGY PROVIDES FOR RUNOFF CONVEYANCE FROM A SERIES OF TOP SURFACE AND OUT SLOPE CHANNELS TO DOWNDRAINS, THEN TO PERIMETER CHANNELS.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. EXISTING CHANNELS AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER 34909 5-17-12

TITLE

OSF PLAN - CONCEPTUAL CLOSURE PLAN

Golder Associates

FILE No.	11381608H004
PROJECT No.	113-81608

4

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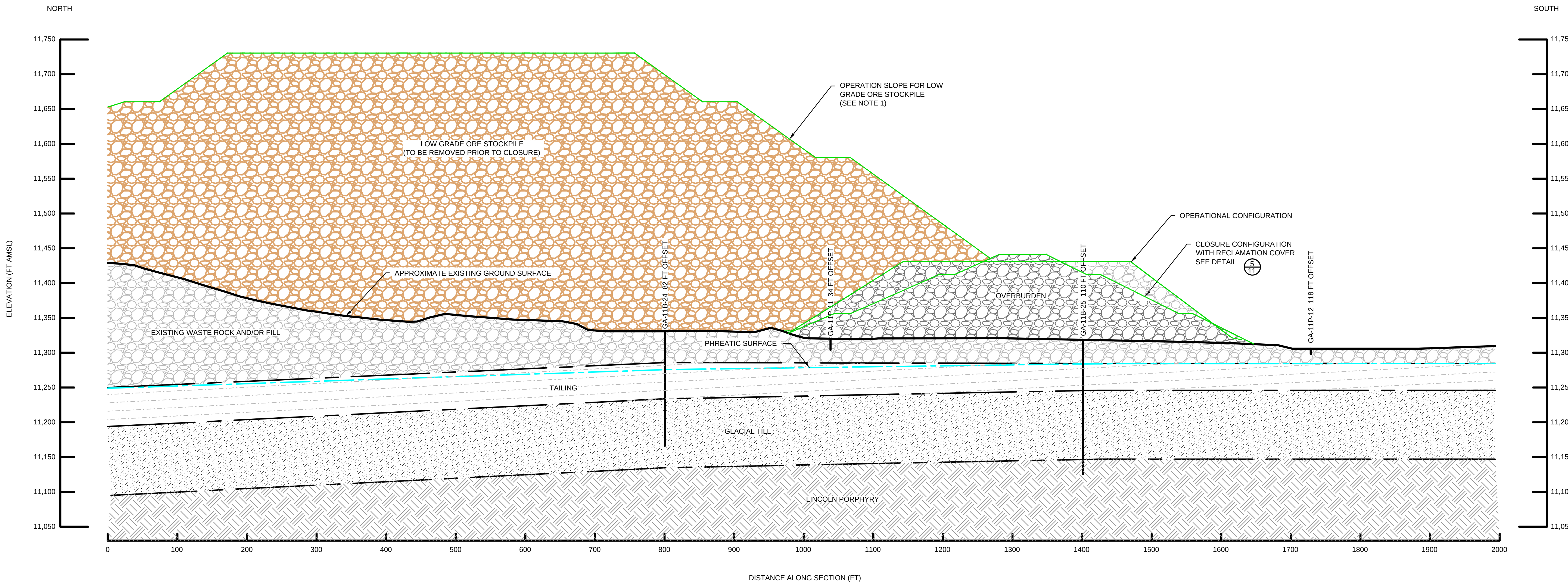


NOTES

1. THE LOW GRADE STOCKPILE SHOWN REPRESENTS THE MAXIMUM ANTICIPATED BUILD-OUT OF THE STOCKPILE. THE STOCKPILE WILL BE REMOVED PRIOR TO CLOSURE.

150 0 150
Feet

ENGINEER'S STAMP

COLORADO REGISTERED
BRENT R. BRONSON
34969
5-17-12
PROFESSIONAL ENGINEER

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

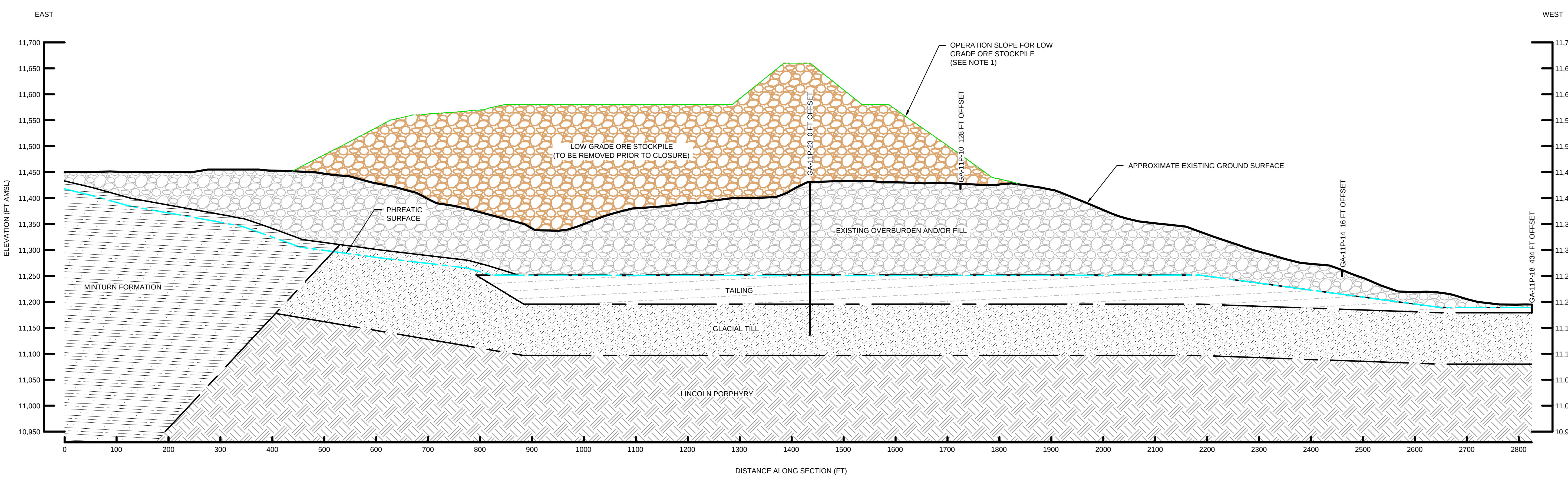
TITLE

OSF CROSS-SECTION A

Golder Associates

FILE No. 11381608H005
PROJECT No. 113-81608

5



NOTES

1. THE LOW GRADE STOCKPILE SHOWN REPRESENTS THE MAXIMUM ANTICIPATED BUILD-OUT OF THE STOCKPILE. THE STOCKPILE WILL BE REMOVED PRIOR TO CLOSURE.

200 0 200
Feet

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

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A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OSF CROSS-SECTION B

Golder Associates

FILE No. 11381608H006
PROJECT No. 113-81608

6

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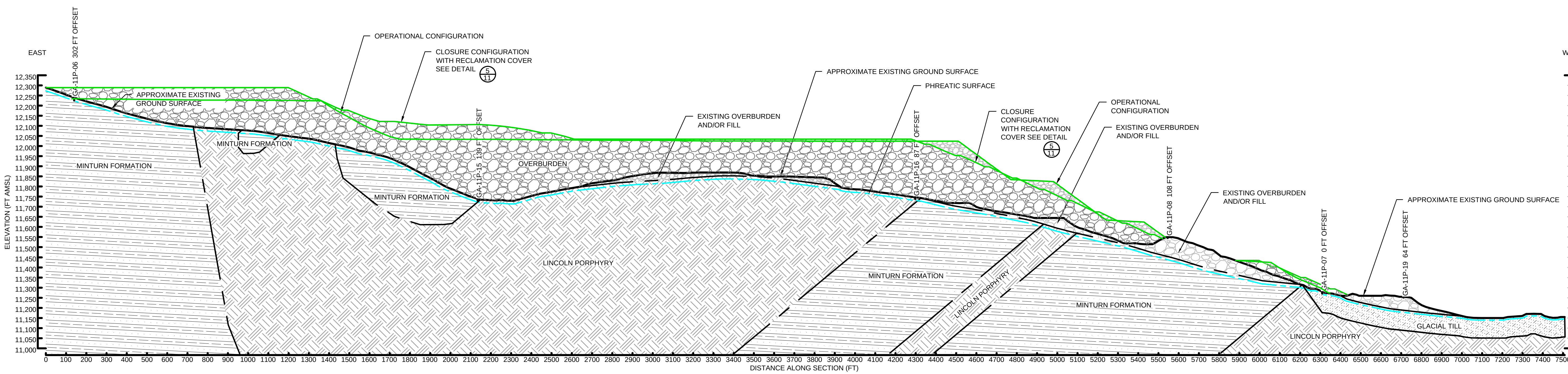

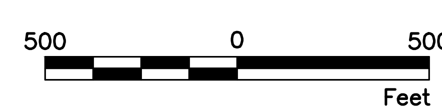

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COLORADO REGISTERED
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34969
5-17-12
PROFESSIONAL ENGINEER

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OSF CROSS-SECTION C

Golder Associates

FILE No. 11381608H007
PROJECT No. 113-81608

7

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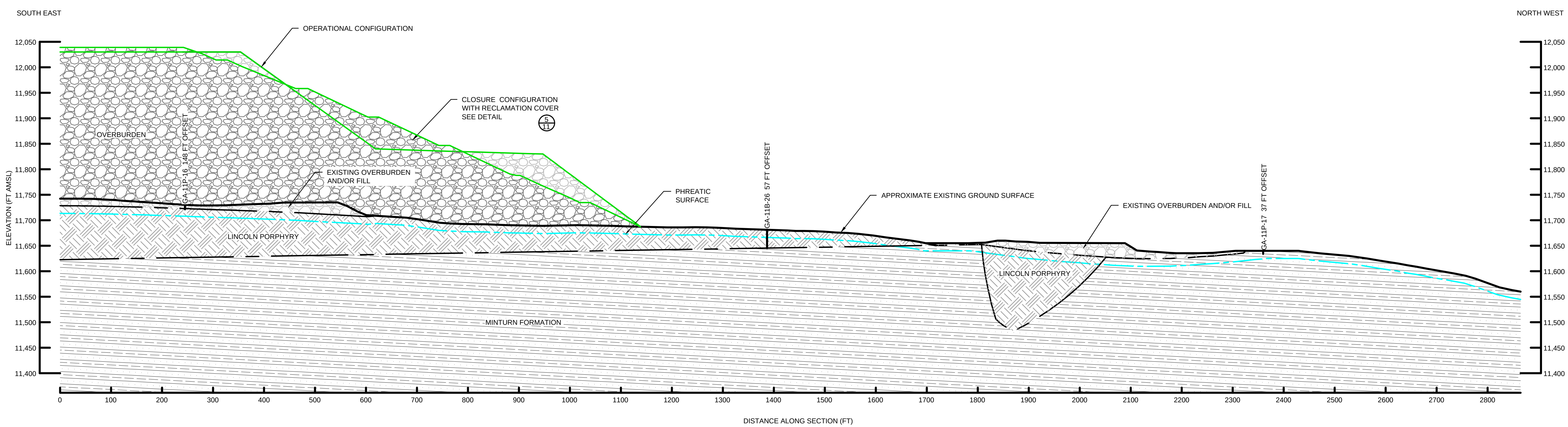

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Feet

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

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A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OSF CROSS-SECTION D

Golder Associates

FILE No. 11381608H008
PROJECT No. 113-81608

8

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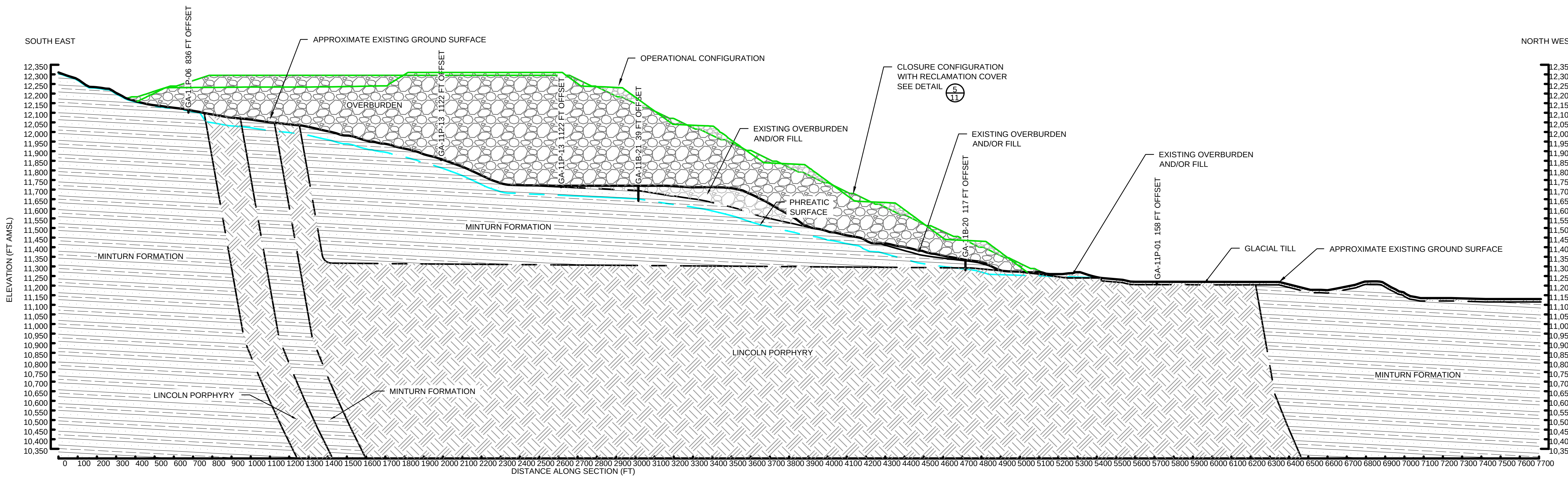

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Feet

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
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5-17-12

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OSF CROSS-SECTION E

Golder Associates

FILE No. 11381608H009
PROJECT No. 113-81608

9

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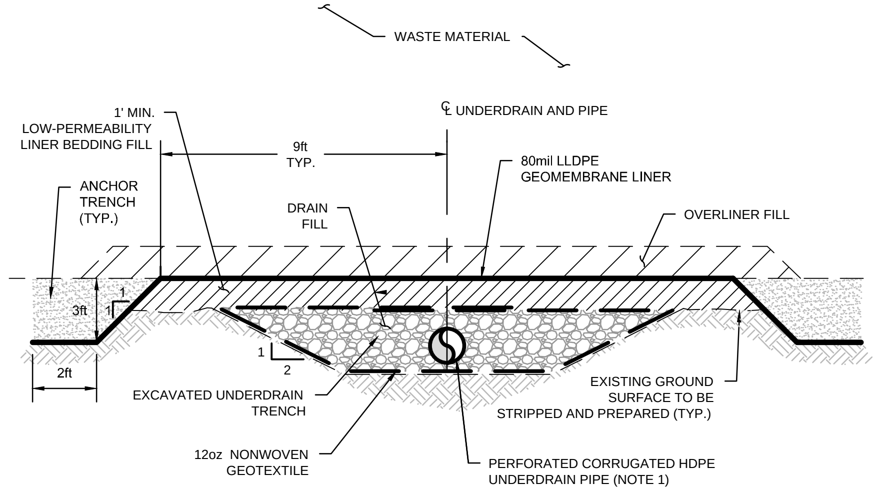
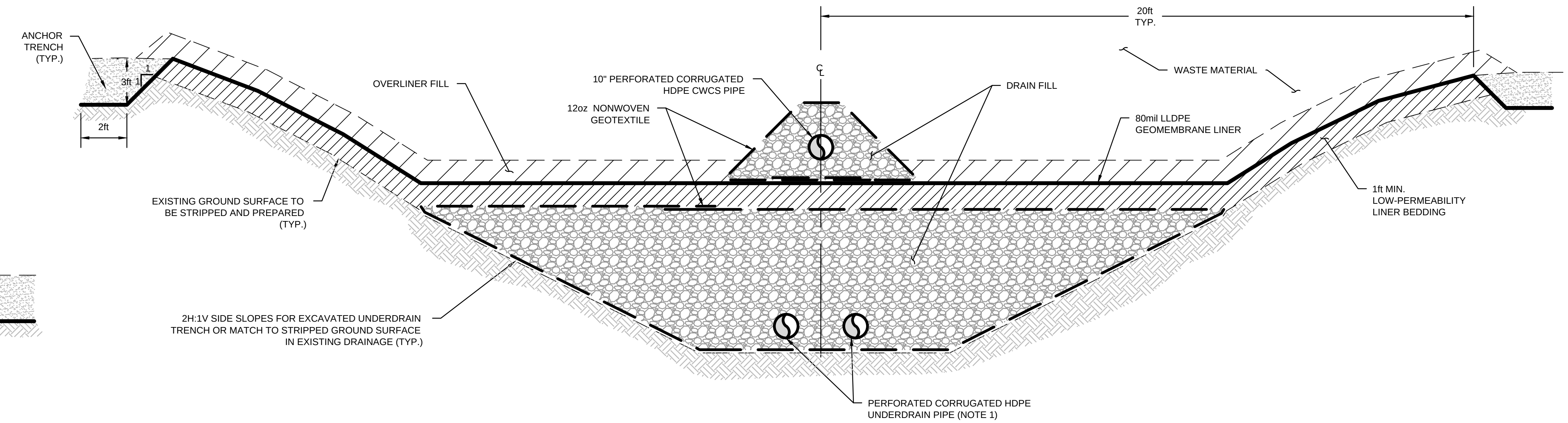


1/10 **TERTIARY UNDERDRAIN SYSTEM**
N.T.S.



2/10 **SECONDARY CONTACT WATER COLLECTION SYSTEM (CWCS) AND UNDERDRAIN SYSTEM**
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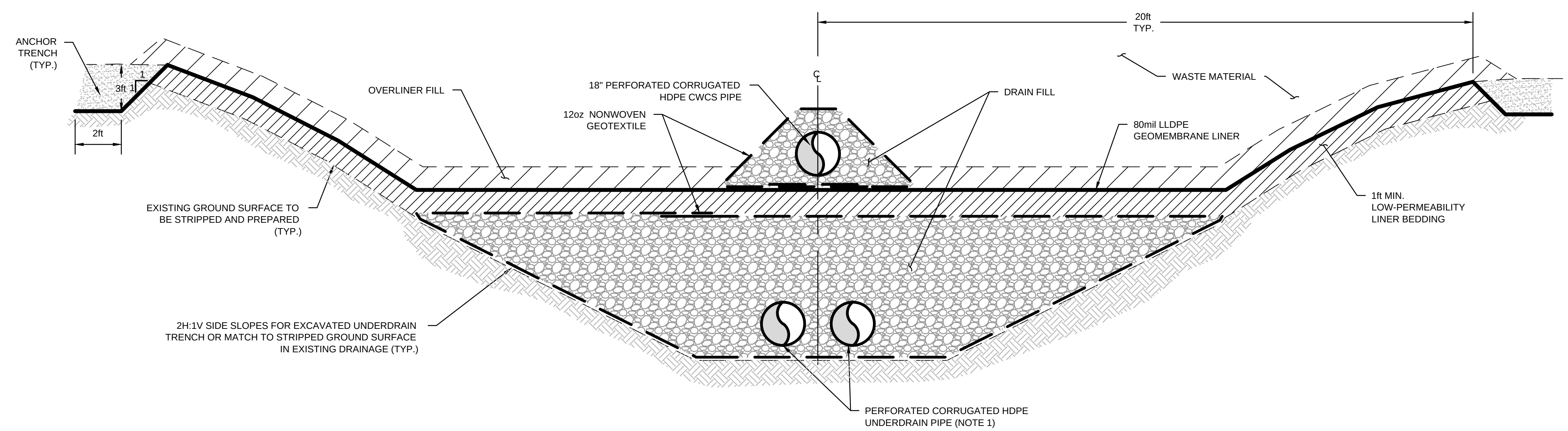

3/10 **PRIMARY CWCS AND UNDERDRAIN SYSTEM**
N.T.S.

NOTES

1. UNDERDRAIN PIPE AND GRANULAR DRAIN DIMENSIONS TO BE SIZED BASED ON PEAK BASELINE FLOWS TO PROVIDE A F.S. = 10 FOR CONVEYANCE THROUGH BOTH THE PIPE (F.S. = 5) AND DRAIN SYSTEMS (F.S. = 5).
2. TERTIARY UNDERDRAIN IS FOR CONVEYANCE OF MINOR SEEPS AND SPRINGS ENCOUNTERED DURING CONSTRUCTION.
3. UNDERDRAINS AND CWCS TO BE EXTENDED IN STAGES AS THE OSF IS ADVANCED UPGRADIENT

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A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT
CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE
OSF TYPICAL WATER MANAGEMENT DETAILS

Golder Associates

FILE No. 11381608H010
PROJECT No. 113–81608
10

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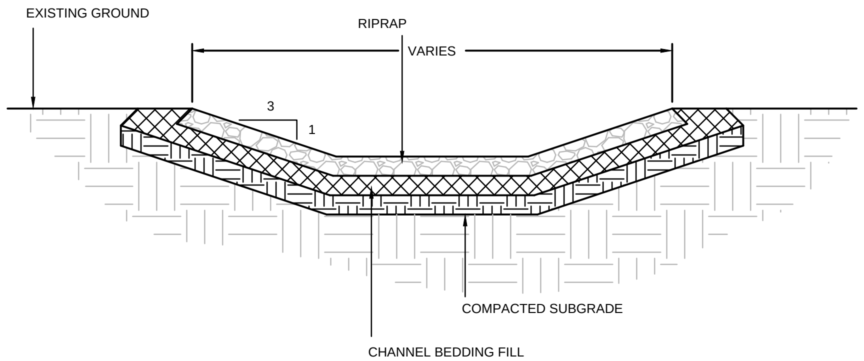

1/11 **PERIMETER/TOPSURFACE CHANNEL**
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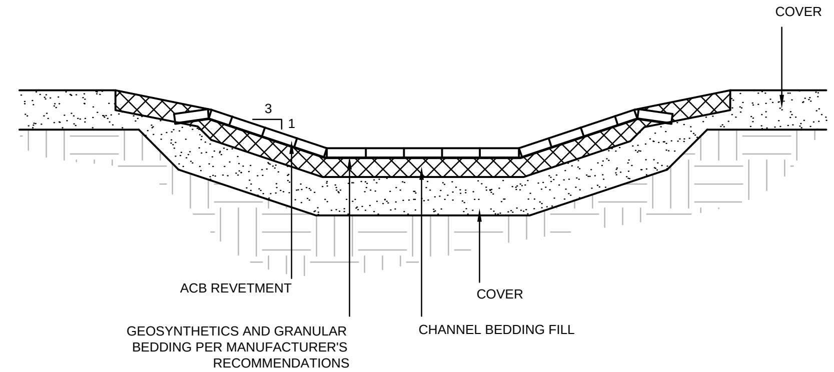
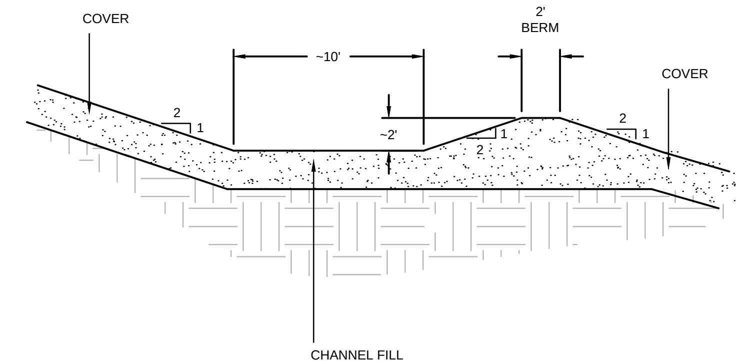
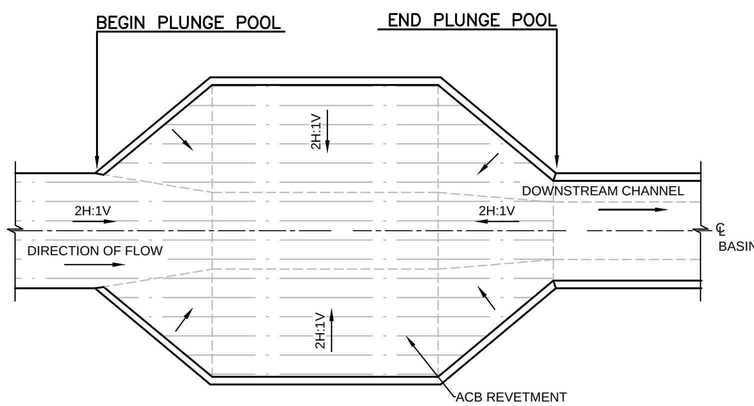


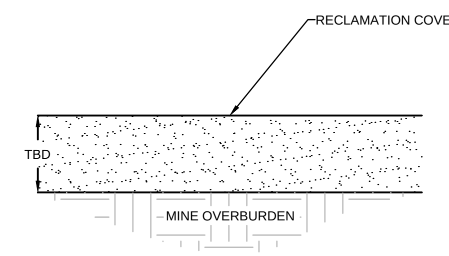
3/11 **DOWNDRAIN CHANNEL SECTION**
N.T.S.



4/11 **OUTSLOPE CHANNEL**
N.T.S.



2/11 **ENERGY DISSIPATOR DETAIL**
N.T.S.



5/11 **RECLAMATION COVER**
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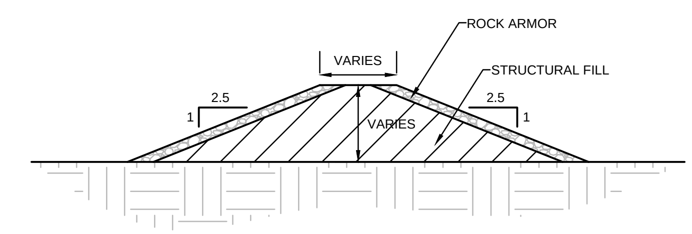

6/11 **DIVERSION BERM**
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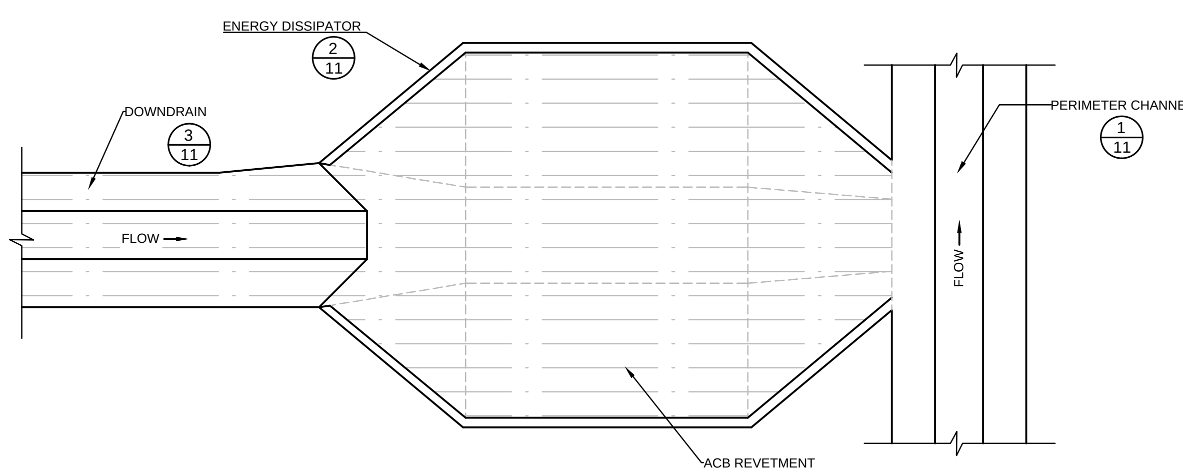

7/11 **DOWNDRAIN ENERGY DISSIPATOR PLAN**
N.T.S.

ENGINEER'S STAMP

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A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

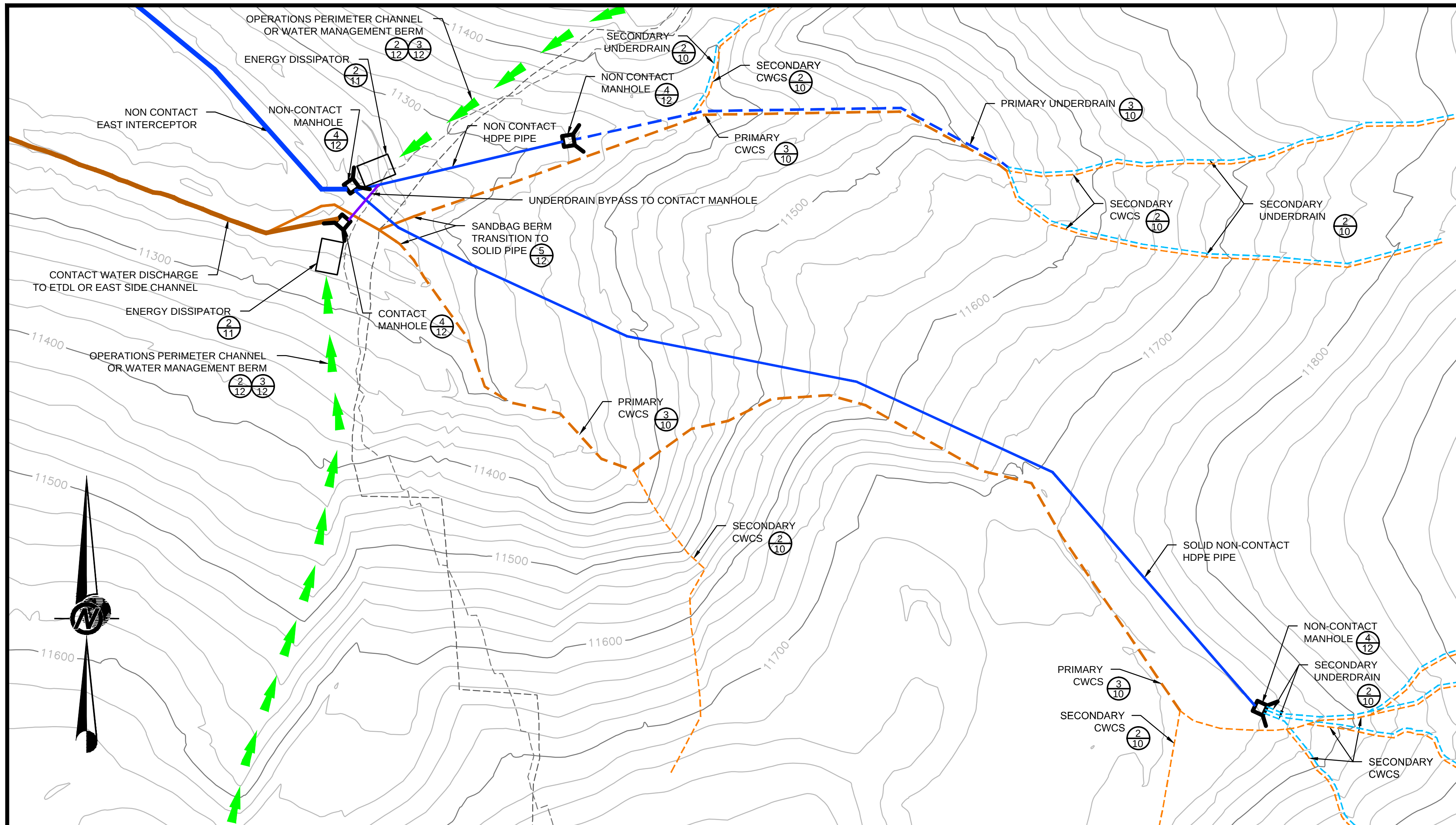
PROJECT
CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE
OSF CONCEPTUAL CLOSURE DETAILS

Golder Associates

FILE No. 11381608H011
PROJECT No. 113-81608

11



1/12 MCNULTY GULCH CONCEPTUAL WATER MANAGEMENT PLAN

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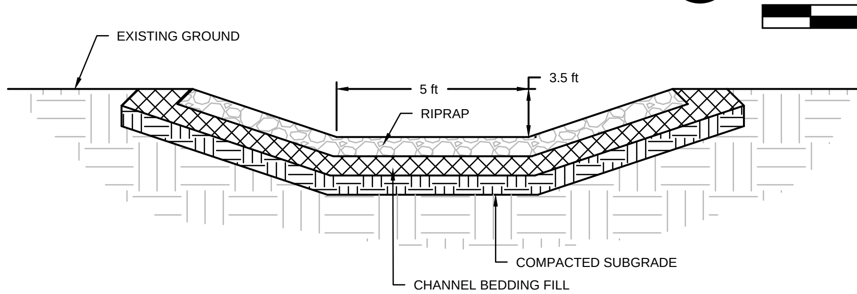
Feet



2/12 OPERATIONS PERIMETER CHANNEL

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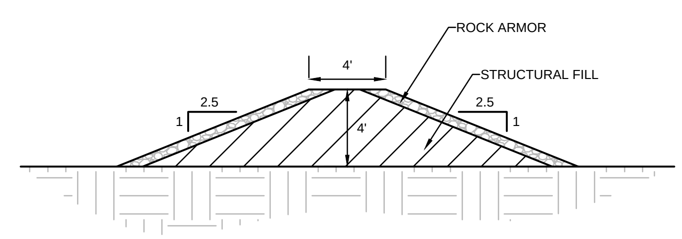

3/12 OPERATIONS WATER MANAGEMENT BERM

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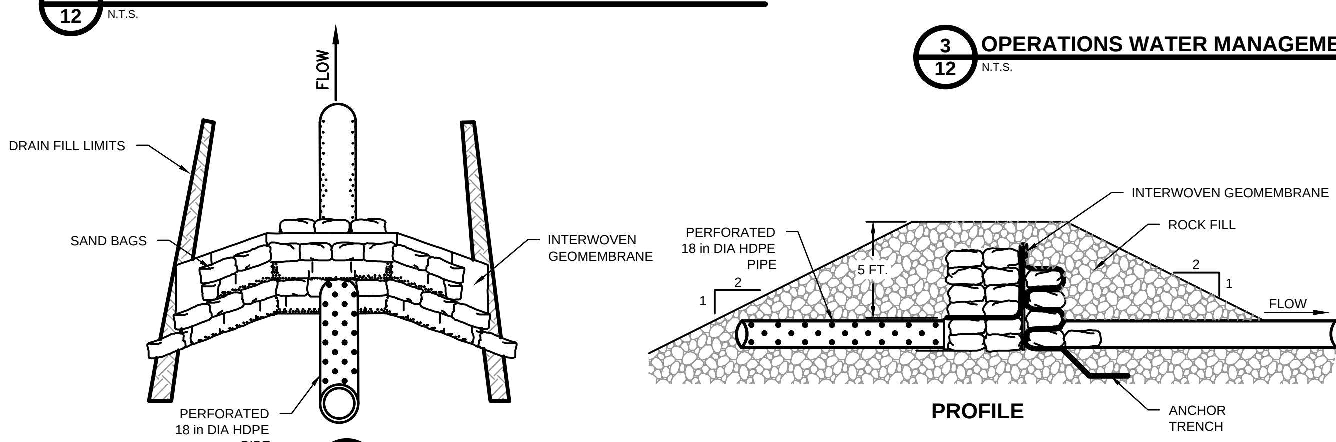
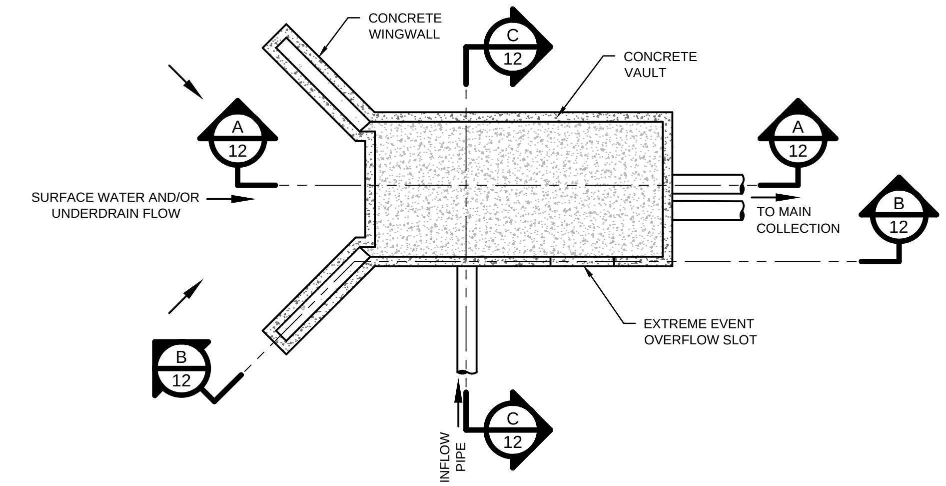


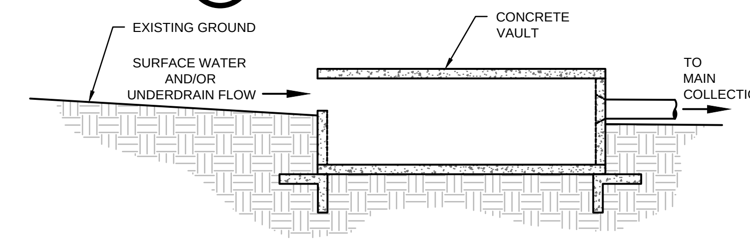
5/12 SANDBAG BERM

N.T.S.



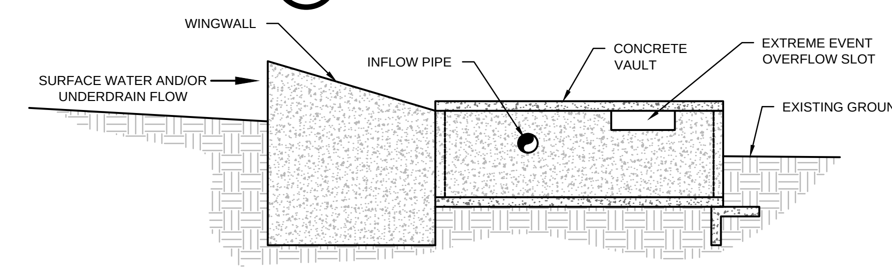
4/12 TYPICAL CONCRETE MANHOLE PLAN

N.T.S.



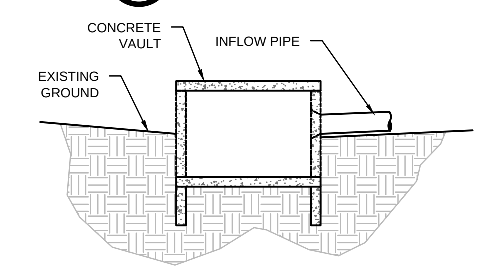
A/12 MANHOLE SECTION THROUGH DISCHARGE PIPE

N.T.S.



B/12 MANHOLE SECTION ALONG WING WALL

N.T.S.



C/12 MANHOLE SECTION THROUGH INLET PIPE

N.T.S.

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

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A	5/16/12	ISSUED WITH DESIGN REPORT	HNL	HNL	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OPERATIONS WATER MANAGEMENT STRAGEGY & CONCEPTUAL DESIGN DETAILS

Golder Associates

FILE No. 11381608H012
PROJECT No. 113-81608

12

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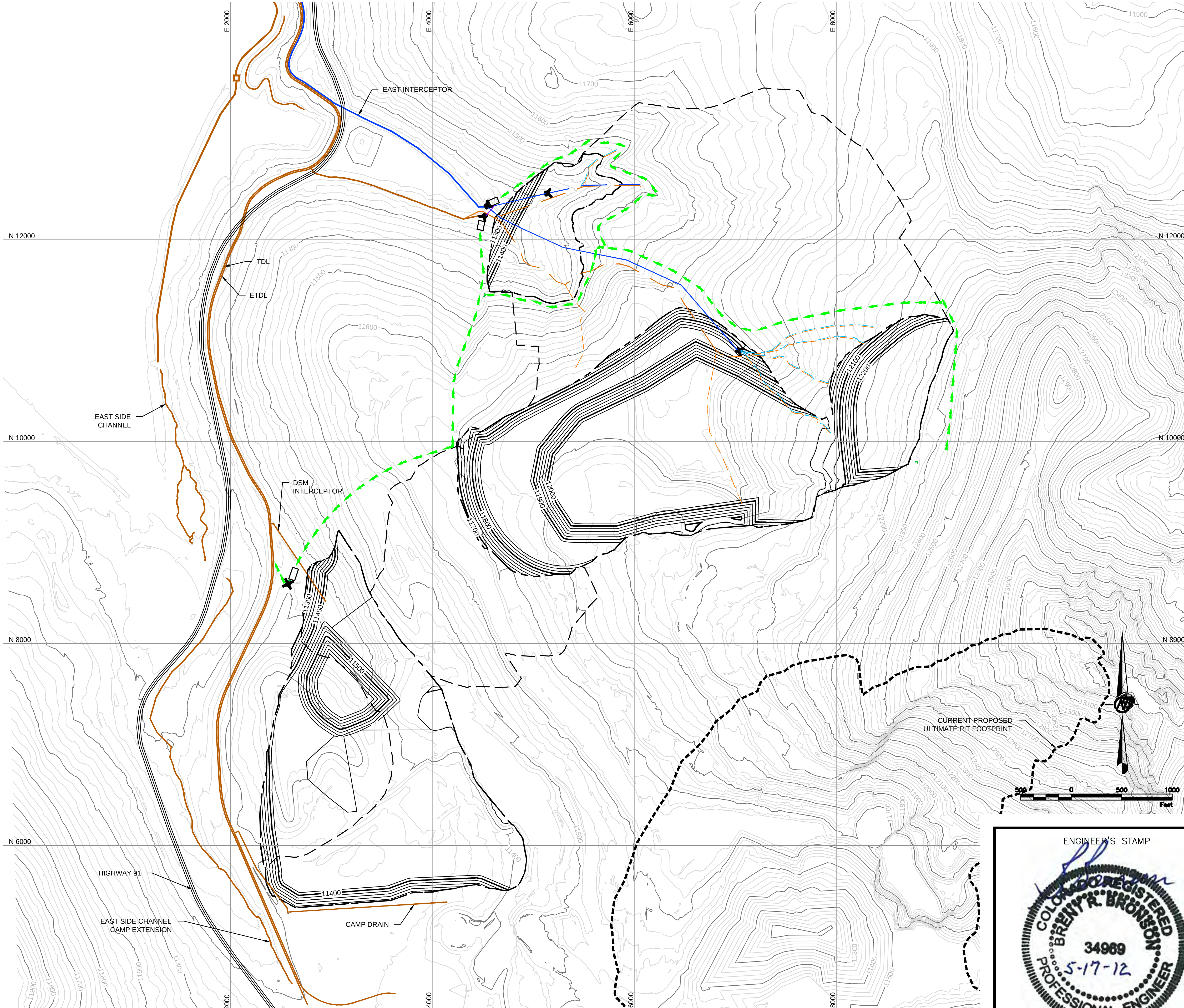

LEGEND

11500	EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
11500	PROPOSED OPERATIONS GRADING (SEE REFERENCE 3)
	PROPOSED OSF BOUNDARY
	OPERATIONS PERIMETER CHANNEL OR WATER MANAGEMENT BERM (SEE DETAILS) 2/12 AND 3/12
	EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 2)
	EXISTING CONTACT WATER CHANNELS / PIPES
	PROPOSED PRIMARY NON-CONTACT HDPE PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED PRIMARY CONTACT WATER PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED UNDERDRAIN BYPASS TO CONTACT MANHOLE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.
2. THE PROPOSED UNDERDRAIN AND CONTACT WATER LAYOUT AND CONCEPTUAL DESIGN DETAILS ARE PROVIDED TO ILLUSTRATE THE GENERAL WATER MANAGEMENT STRATEGY. THE DESIGNS WILL BE DETAILED AND FINALIZED UPON CONFIRMATION OF THE DESIGN APPROACH.
3. SEE DRAWING 3 TO ILLUSTRATE THE ANTICIPATED FINAL TOE LIMITS AND WATER MANAGEMENT STRATEGY FOR THE CLIMAX OSF'S.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. EXISTING CHANNELS AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.
3. PROPOSED OSF GRADING PROVIDED BY CLIMAX ON 4 APRIL 2012.

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

TITLE

OSF PLAN - END OF YEAR 5

Golder Associates

FILE No. 11381608H013
PROJECT No. 113-81608

13

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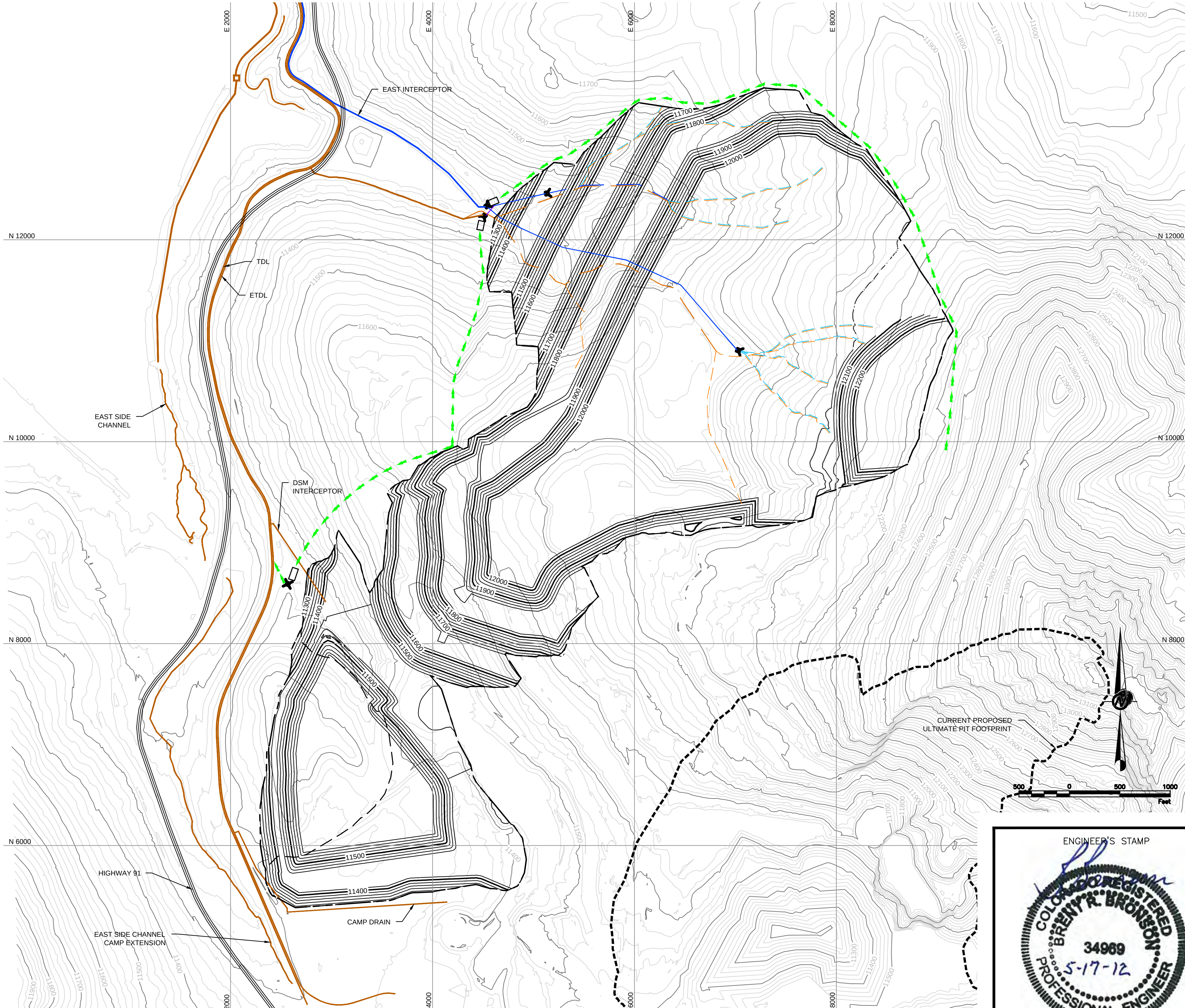

LEGEND

- 11500 — EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
- 11500 — PROPOSED OPERATIONS GRADING (SEE REFERENCE 3)
- PROPOSED OSF BOUNDARY
- OPERATIONS PERIMETER CHANNEL OR WATER MANAGEMENT BERM (SEE DETAILS) 2/12 AND 3/12
- EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 2)
- EXISTING CONTACT WATER CHANNELS / PIPES
- PROPOSED PRIMARY NON-CONTACT HDPE PIPE LOCATIONS (NOTE 2)
- PROPOSED PRIMARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
- PROPOSED SECONDARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
- PROPOSED PRIMARY CONTACT WATER PIPE LOCATIONS (NOTE 2)
- PROPOSED PRIMARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
- PROPOSED SECONDARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
- PROPOSED UNDERDRAIN BYPASS TO CONTACT MANHOLE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.
2. THE PROPOSED UNDERDRAIN AND CONTACT WATER LAYOUT AND CONCEPTUAL DESIGN DETAILS ARE PROVIDED TO ILLUSTRATE THE GENERAL WATER MANAGEMENT STRATEGY. THE DESIGNS WILL BE DETAILED AND FINALIZED UPON CONFIRMATION OF THE DESIGN APPROACH.
3. SEE DRAWING 3 TO ILLUSTRATE THE ANTICIPATED FINAL TOE LIMITS AND WATER MANAGEMENT STRATEGY FOR THE CLIMAX OSF'S.

REFERENCES

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2. EXISTING CHANNELS AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.
3. PROPOSED OSF GRADING PROVIDED BY CLIMAX ON 4 APRIL 2012.

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

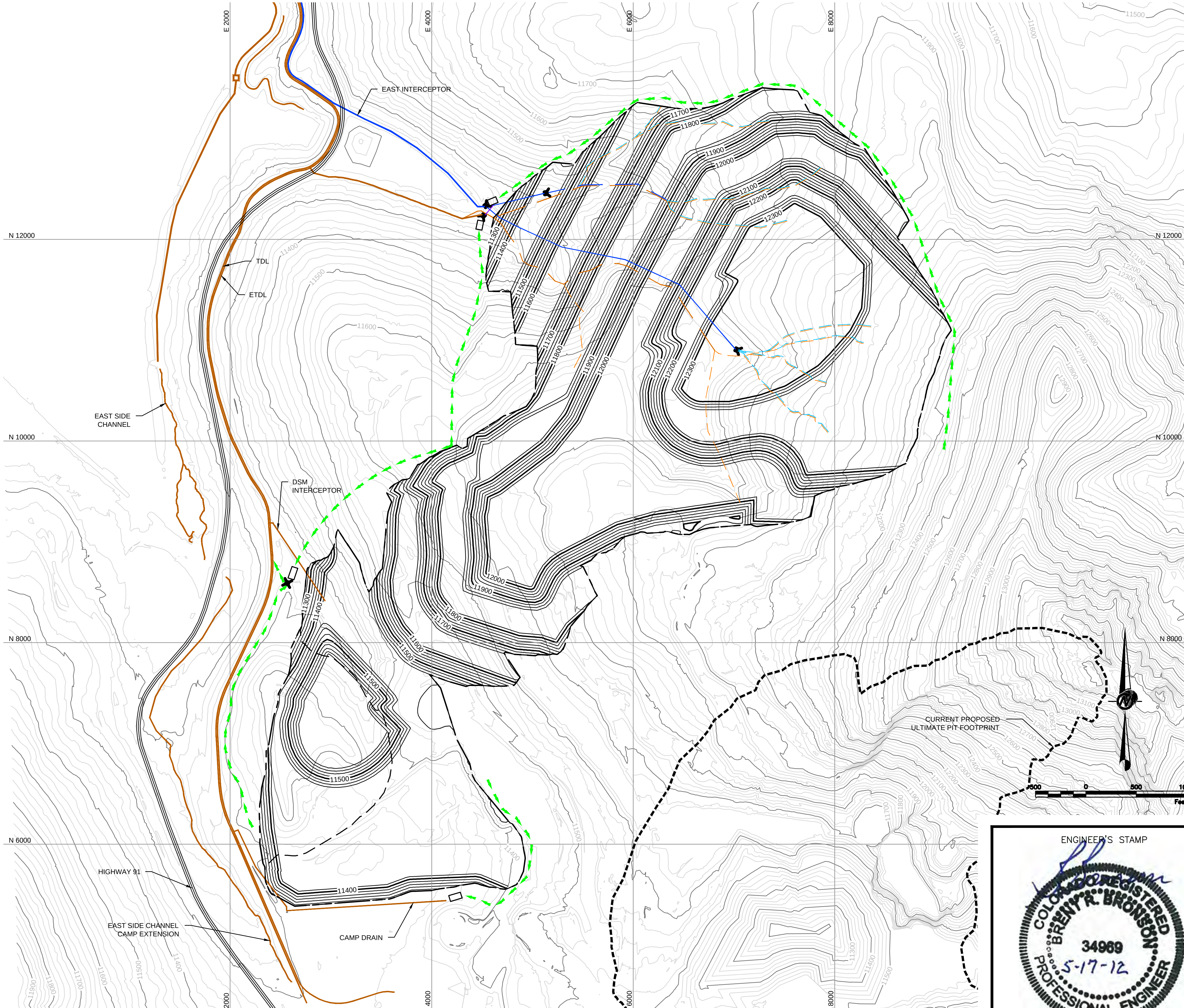
TITLE

OSF PLAN - END OF YEAR 10

Golder Associates

FILE No. 11381608H014
PROJECT No. 113-81608

14



LEGEND

11500	EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)
11500	PROPOSED OPERATIONS GRADING (SEE REFERENCE 3)
	PROPOSED OSF BOUNDARY
	OPERATIONS PERIMETER CHANNEL OR WATER MANAGEMENT BERM (SEE DETAILS) 2/12 AND 3/12
	EXISTING NON-CONTACT WATER CHANNELS / PIPES (SEE REFERENCE 2)
	EXISTING CONTACT WATER CHANNELS / PIPES
	PROPOSED PRIMARY NON-CONTACT HDPE PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY UNDERDRAIN COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED PRIMARY CONTACT WATER PIPE LOCATIONS (NOTE 2)
	PROPOSED PRIMARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 3/10
	PROPOSED SECONDARY CONTACT WATER COLLECTION PIPE LOCATIONS (NOTE 2) SEE DETAIL 2/10
	PROPOSED UNDERDRAIN BYPASS TO CONTACT MANHOLE

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.
2. THE PROPOSED UNDERDRAIN AND CONTACT WATER LAYOUT AND CONCEPTUAL DESIGN DETAILS ARE PROVIDED TO ILLUSTRATE THE GENERAL WATER MANAGEMENT STRATEGY. THE DESIGNS WILL BE DETAILED AND FINALIZED UPON CONFIRMATION OF THE DESIGN APPROACH.
3. SEE DRAWING 3 TO ILLUSTRATE THE ANTICIPATED FINAL TOE LIMITS AND WATER MANAGEMENT STRATEGY FOR THE CLIMAX OSF'S.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. EXISTING CHANNELS AND EXISTING WATER PIPELINE LOCATIONS PROVIDED BY W. W. WHEELER AND ASSOCIATES, INC. 12 JANUARY 2012.
3. PROPOSED OSF GRADING PROVIDED BY CLIMAX ON 4 APRIL 2012.

ENGINEER'S STAMP

COLORADO REGISTERED PROFESSIONAL ENGINEER
BRENT R. BRONSON
34969
5-17-12

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	5/16/12	ISSUED FOR DESIGN REPORT	DAR	DAR	DLG	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES COLORADO

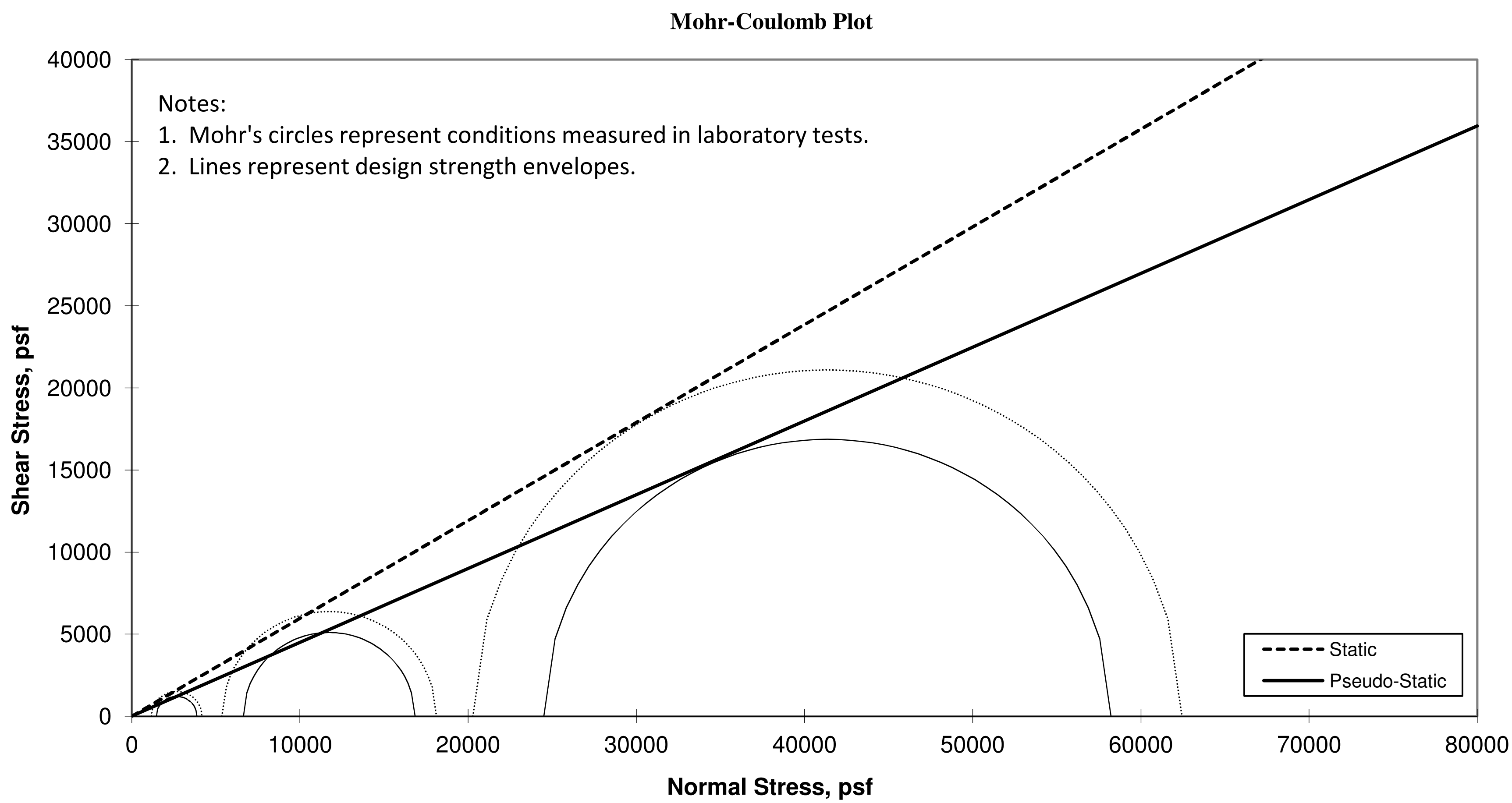
TITLE

OSF PLAN - END OF YEAR 15

Golder Associates

FILE No.	11381608H015
PROJECT No.	113-81608

15



Golder Associates

Denver, Colorado

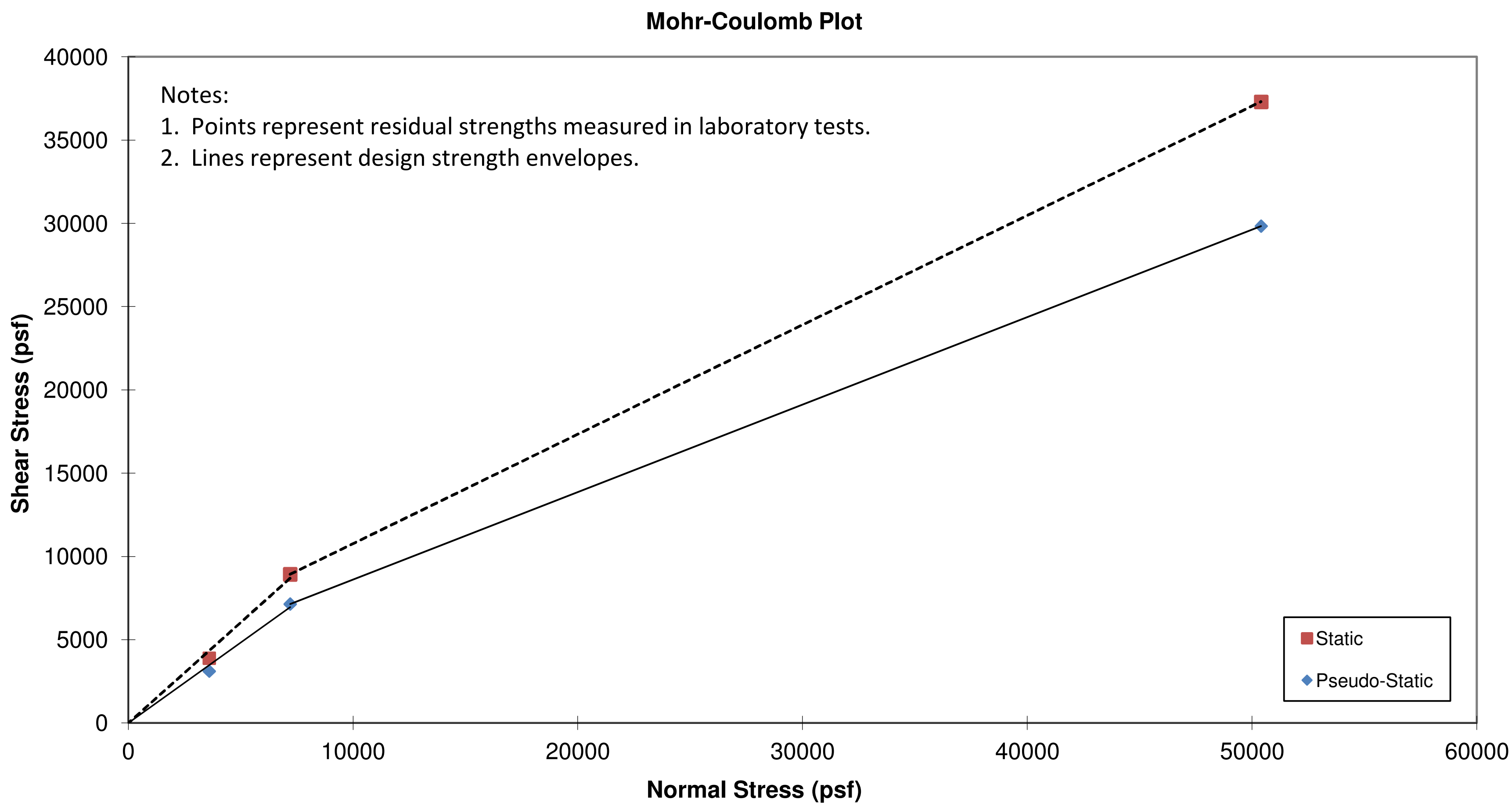
TITLE

Static and Pseudo-Static Design Strengths for the Minturn Formation

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	1



Golder Associates

Denver, Colorado

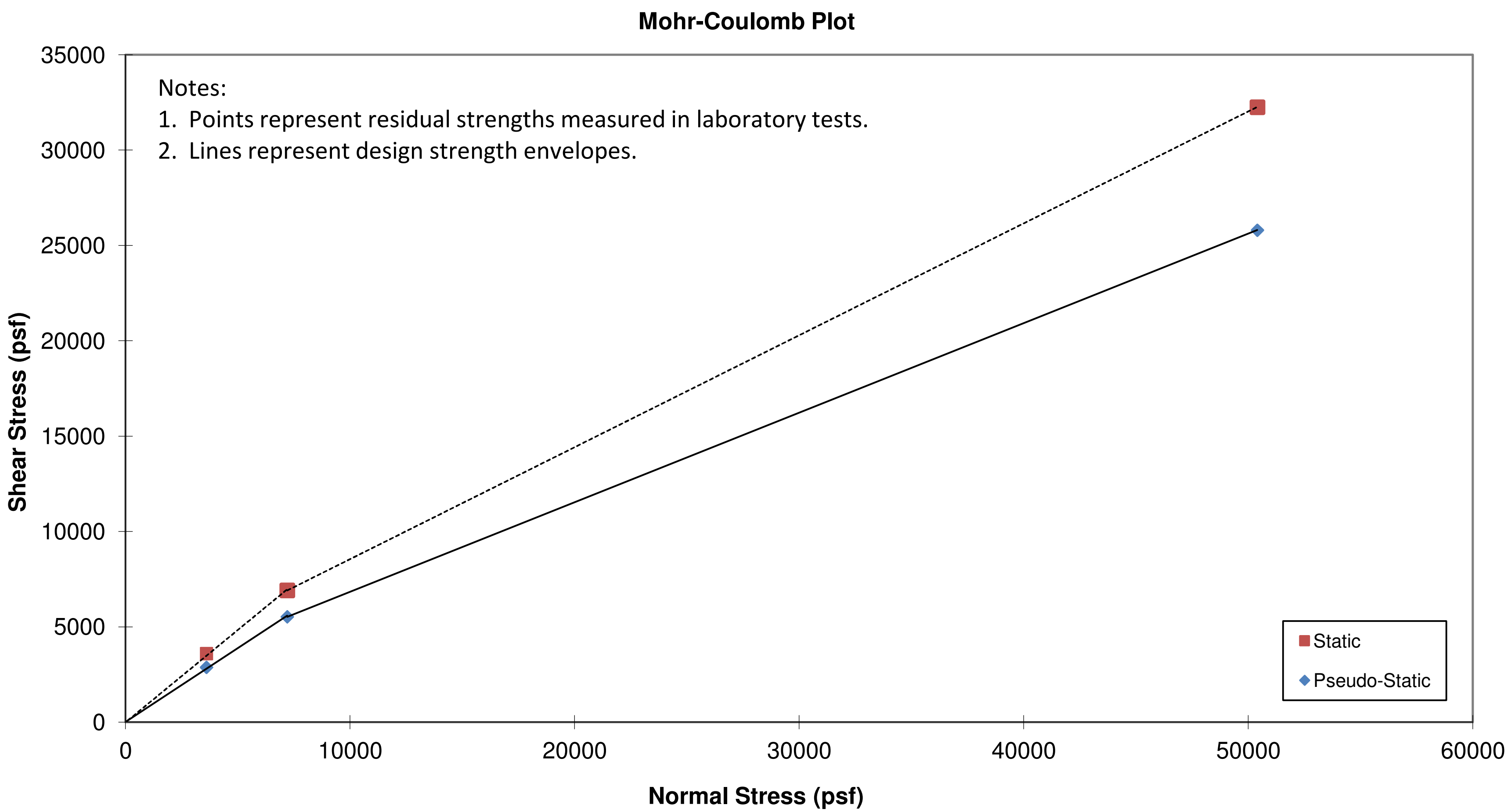
TITLE

Static and Pseudo-Static Design Strengths for the Lincoln Porphyry

CLIENT/PROJECT

Climax Molybdenum, OSF Design

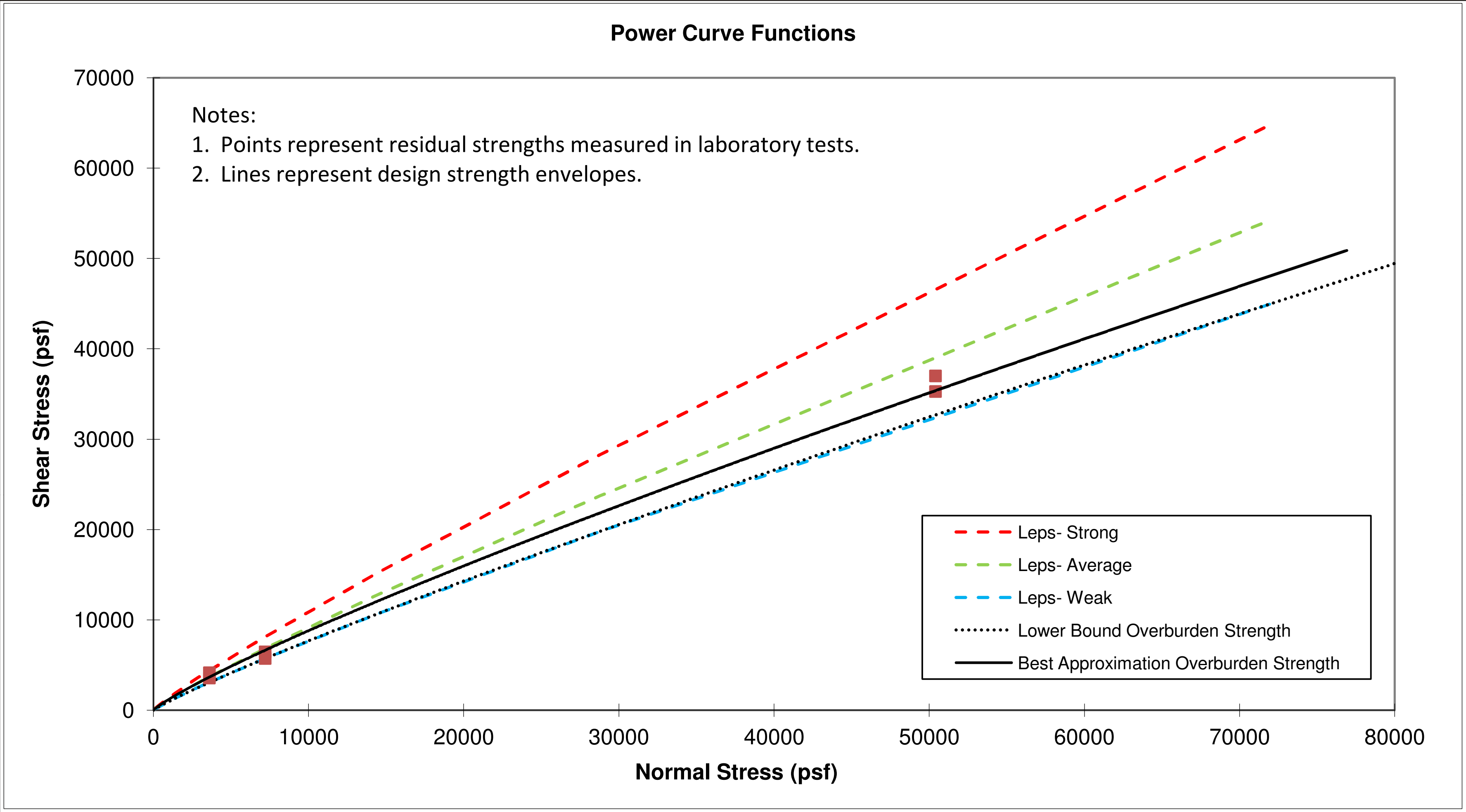
DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	2



Golder Associates

Denver, Colorado

TITLE	**Static and Pseudo-Static Design Strengths for Glacial Till**				
CLIENT/PROJECT	**Climax Molybdenum, OSF Design**				
DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	3



Golder Associates Denver, Colorado

TITLE

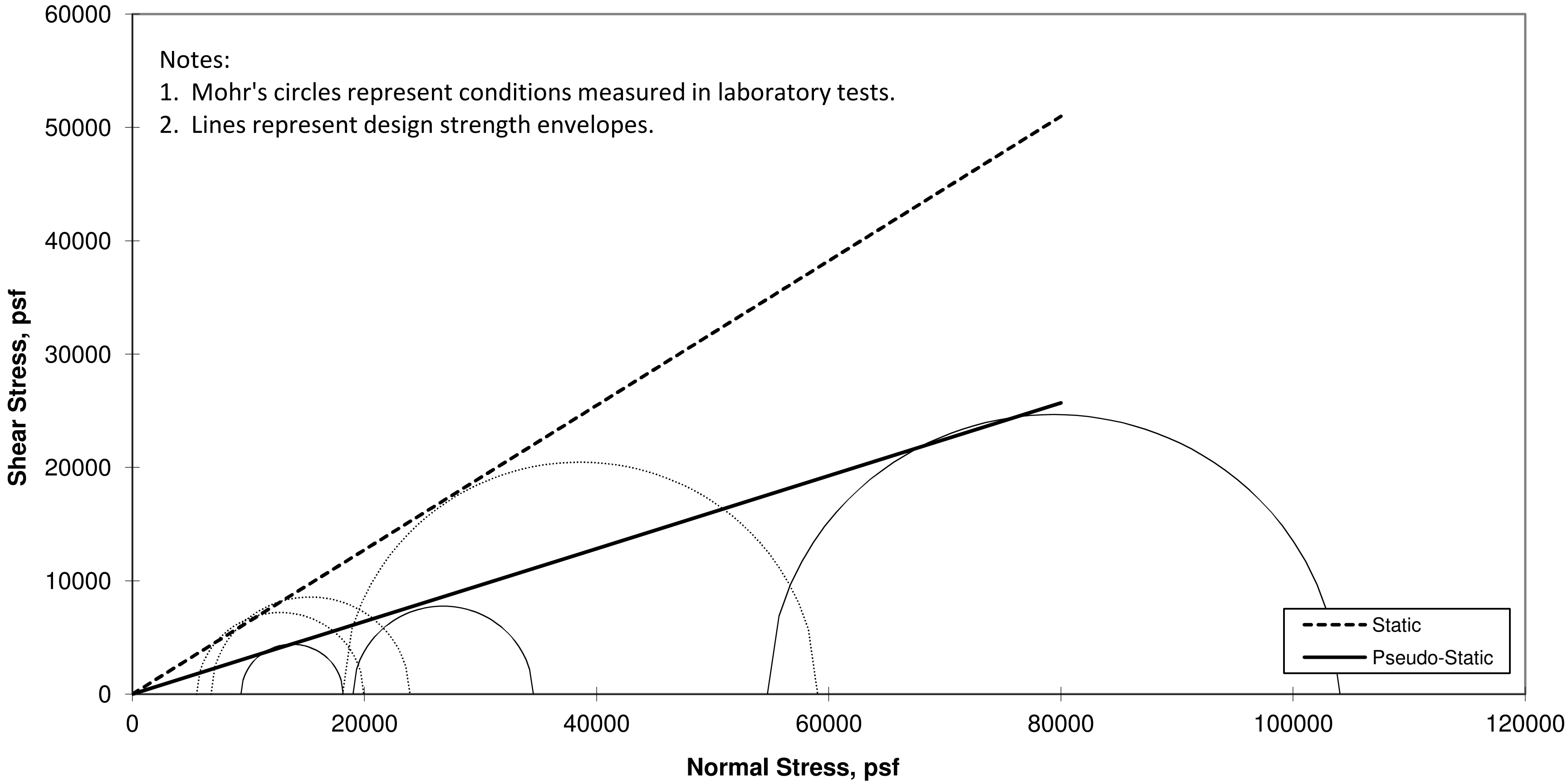
Static and Pseudo-Static Design Strengths for Waste Rock

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	4

Mohr-Coulomb Plot



Golder Associates

Denver, Colorado

TITLE

Static and Pseudo-Static Design Strengths for Tailing

CLIENT/PROJECT

Climax Molybdenum, OSF Design

DRAWN	DAR	DATE	Apr-12	JOB NO.	113-81608
CHECKED	DLG	SCALE	NTS	DWG. NO.	--
REVIEWED	BRB	FILE NO.	--	FIGURE NO.	5

LEGEND

EXISTING GROUND TOPOGRAPHY (SEE REFERENCE 1)

PROPOSED WASTE ROCK STOCKPILE TOPOGRAPHY (SEE REFERENCE 3)

OSF BOUNDARY

PROPOSED WATER MONITORING POINT

NOTES

1. GRID SHOWS MINE SURVEY COORDINATES.

REFERENCES

1. 2010 TOPOGRAPHY PROVIDED BY CLIMAX MINE, 22 MARCH 2011.
2. AERIAL PHOTOGRAPH TAKEN IN 2009, FROM USDA AERIAL PHOTOGRAPHY FIELD OFFICE.
3. PRELIMINARY WASTE ROCK STOCKPILE TOPOGRAPHY PROVIDED BY CLIMAX MINE, 24 APRIL 2012.

REV	DATE	REVISION DESCRIPTION	DESIGN	CADD	CHECK	REVIEW
A	24MAY12	ISSUED FOR REPORT	DLG	DLG	BRB	BRB

PROJECT

CLIMAX MINE
LAKE AND SUMMIT COUNTIES, COLORADO

TITLE

PRELIMINARY LOCATIONS FOR WATER MONITORING POINTS

Golder Associates
DENVER, COLORADO

FILE No. FIGURE 6
PROJECT No. 113-81608

FIGURE 6

Drawing File: J:\11JOBS\113-81608 Climax\Design Report\Rev3\Figures\ Figure 6.dwg | Layout: 11x17 Landscape | Modified: May 30, 2012 10:21 DGEIER | Plotted: 05/30/12 10:21am DGEIER

Table 1
Borehole Locations and Summary of Findings

Borehole ID	Date	Approximate Location	Summary of Materials Found	Depth to Native (ft)	Depth to Bedrock (ft)	Depth to Water Table (ft)	Total Depth (ft)	Northing[1]	Easting[1]	Elevation (ft asl)
GA-11B-20	11/3/2011	McNulty Gulch	0 to 38.5 ft: Colluvial material derived from the Minturn 38.5 to 54 ft: Weathered Lincoln Porphyry	0	38.5	24.8	54	12162	4936	11317
GA-11B-21	10/20/2011	Waste Rock in McNulty Gulch	0 to 25 ft: Waste Rock 25 to 43 ft: Weathered Minturn Sandstone 43 to 78.5: Minturn Sandstone with thin beds of shale and clay	25	43	42.9	78.5	11251	6420	11400
GA-11B-22	11/1/2011	McNulty Gulch	0 to 10 ft: Road fill 10 to 33.5 ft: Weathered Lincoln Porphyry	10	10	12	33.5	11562	4503	11400
GA-11B-23	10/24/2011	Waste Rock Pile on top of Tailings	0 to 179 ft: Waste Rock 179 to 235: Tailings 235 to 294 ft: Glacial Till	235	NA	179.8	294	7257	2855	11430
GA-11B-24	10/18/2011	Gravel Pit	0 to 45 ft: Waste Rock 45 to 98 ft: Tailings 98 to 164.5 ft: Glacial Till	98	NA	54.9	164.5	6241	2935	11330
GA-11B-25	10/21/2011	Laydown Yard	0 to 7 ft: Fill 7 to 33.5 ft: Waste Rock 33.5 to 72 ft: Tailings 72 to 171 ft: Glacial Till 171 to 193 ft: Weathered Lincoln Porphyry	72	171	33.9	193	5628	2903	11320
GA-11B-26	10/20/2011	Above McNulty Gulch	0 to 4 ft: Topsoil 4 to 36 ft: Weathered to Slightly Weathered Lincoln Porphyry	0	4	13.1	36	10023	3900	11682

[1]Northings and Eastings are listed based on the local mine coordinate system

Table 2
Summary of Previously Reported Minturn and Lincoln Formation Strengths[1]

Formation	Back Calculated or From Field/Laboratory Data	Where Values Were Originally Applied	General Notes	Rock Mass Information		Rock Mass Strength		Intact Rock Strength	Shear Strength Along Joints/Bedding Planes	
				RQD	GSI	phi (degrees)	c (psi)	UCS (psi)	phi (degrees)	c (psi)
Minturn Formation	Back Calculated	Pit West Wall		--	--	29.0	4.9	--	--	--
	Field / Lab Data	Pit West Wall		--	--	19.0	32.9	--	--	--
	Field / Lab Data	Pit West Wall		12	--	30.8	895.0	3607	15.8	4.1
Lincoln Porphyry	Back Calculated	Pit West Wall		--	--	27.0	1.4	--	--	--
	Field / Lab Data	Pit West Wall		--	--	--	--	5179	24.4	4.9
	Field / Lab Data	Pit West Wall	Very Altered Rock	--	--	23.1	21.9	--	--	--
	Field / Lab Data	Pit West Wall	Altered Rock	--	--	29.4	57.2	--	--	--
	Field / Lab Data	Pit West Wall	Upper Zone	--	--	33.1	515.0	2193	20.6	2.4
	Field / Lab Data	Pit West Wall	Transition Zone	--	--	41.7	946.0	5479	24.9	4.9
	Field / Lab Data	Pit West Wall	Lower Zone	--	--	53.5	1717.0	12967	27.3	4.9

[1] Values summarized from: Call and Nicholas, Inc. 2007. "Slope Stability Evaluation For Feasibility Level Pit Design of Long Range Reserves at Climax Mine."

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Table 3
Strength Parameters Utilized for Static Stability Analysis

Soil Type	Total Unit Weight (pcf)	Failure Envelope Type	Failure Envelope Definition (psf)	Notes
Native Materials				
Minturn Formation	119	Mohr-Coulomb	$\tau' = \sigma'\tan(31°)$	This failure envelope was determined from the strength results of a staged undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism.
LincolnPorphyry	117	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(50°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(33°)+4200$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests.
GlacialTill	123	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(44°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(30°)+2688$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests.
Overburden Materials				
Best Approximation Overburden Parameters	120	Power Function	$\tau' = 3.18\sigma'^{0.86}$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests. This envelope lies between the average and low envelopes developed by Leps (1971).
Lower Bound Overburden Parameters	120	Power Function	$\tau' = 2.02\sigma'^{0.90}$	This failure envelope was used to account for a higher proportion of weaker materials with in the OSF. This envelope is analagous to the low strength envelope developed by Leps (1971).
Tailings Materials				
Tailings Materials	100	Mohr-Coulomb	$\tau' = \sigma'\tan(33°)$	This failure envelope was determined from the residual strength results of a staged undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism.

Table 4
Strength Parameters Utilized for Pseudo-Static Stability Analysis

Soil Type	Total Unit Weight (pcf)	Failure Envelope Type	Failure Envelope Definition (pcf)	Notes
Native Materials				
Minturn Formation	119	Mohr-Coulomb	$\tau' = \sigma'\tan(24°)$	This failure envelope was determined from the strength results of a staged consolidated undrained triaxial test with pore pressure measurements. No cohesion was included for conservatism. Values were reduced by 20% for the seismic condition.
Lincoln Porphyry	117	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(44°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(28°)+3360$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests with the values reduced by 20%.
Glacial Till	123	Bi-Linear Mohr Coulomb	$\tau' = \sigma'\tan(38°)$ for $\sigma'<7200$ $\tau' = \sigma'\tan(25°)+2150$ for $\sigma'>7200$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests with the values reduced by 20%.
Overburden Materials				
Best Approximation Overburden Parameters	120	Power Function	$\tau' = 3.18\sigma'^{0.86}$	This failure envelope was determined from the residual strength results of a series of large scale direct shear tests. This envelope lies between the average and low envelopes developed by Leps (1971).
Lower Bound Overburden Parameters	120	Power Function	$\tau' = 2.02\sigma'^{0.90}$	This failure envelope was used to account for a higher proportion of weaker materials with in the OSF. This envelope is analagous to the low strength envelope developed by Leps (1971).
Tailings Materials				
Tailings Materials	100	Morh-Coulomb	$\tau = \sigma\tan(18°)$	This failure envelope was determined from the residual strength results of a consolidated undrained triaxial test with pore pressure measurements. A total stress approach was utilized for the seismic condition.

Table 5
Stability Analysis Results for the Maximum Operational OSF Configuration

Section	Seismicity	Minimum Factor of Safety- Reduced Strength Overburden	Minimum Factor of Safety- Best Approximation Overburden
A-A	static	1.40	1.59
A-A	pseudo-static	1.21	1.28
B-B	static	1.41	1.59
B-B	pseudo-static	1.25	1.34
C-C	static	1.40	1.49
C-C	pseudo-static	1.09	1.09
D-D	static	1.40	1.59
D-D	pseudo-static	1.26	1.28
E-E	static	1.55	1.57
E-E	pseudo-static	1.15	1.19

Table 6
Stability Analysis Results for the Post-Closure OSF Configuration

Section	Seismicity	Minimum Factor of Safety- Reduced Strength Overburden	Minimum Factor of Safety- Best Approximation Overburden
A-A	static	1.95	2.04
A-A	pseudo-static	1.24	1.30
B-B	static	1.92	2.00
B-B	pseudo-static	1.35	1.42
C-C	static	1.50	1.52
C-C	pseudo-static	1.00	1.00
D-D	static	1.70	1.73
D-D	pseudo-static	1.13	1.15
E-E	static	1.58	1.63
E-E	pseudo-static	1.10	1.10

DESIGN REPORT

NORTH 40 AND MCNULTY OVERBURDEN STORAGE FACILITY DESIGN REPORT

Climax Molybdenum Company – Climax Mine

Submitted To: Climax Molybdenum Company – Climax Mine
Highway 91 – Fremont Pass
Climax, Colorado 80429

Submitted By: Golder Associates Inc.
44 Union Boulevard, Suite 300
Lakewood, Colorado 80228 USA

Distribution: 3 Copies – Climax Mine
2 Copies – Colorado Division of Reclamation, Mining and Safety
2 Copies – Golder Associates Inc.

May 2012 **113-81608**

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1.0 INTRODUCTION

1.1 Purpose

This report presents the Golder Associates Inc. (Golder) engineering evaluations and analyses for the proposed North 40 and McNulty overburden storage facility (OSF) designs. The report includes results of the field investigation, laboratory investigation, operations and monitoring (O&M) recommendations, and design calculations performed in support of the ultimate OSF designs. Both OSFs are located at the Climax Molybdenum Company – Climax Mine (Climax). The work presented in this report was performed to fulfill the scope of work titled "*Detailed Work Plan for Overburden Storage Facility Geotechnical Investigation and Design*", which was presented to Climax on 3 June 2011.

The primary focus of the work presented in this report is intended to demonstrate the geotechnical stability of the proposed expansions of the North 40 and McNulty OSFs, which are collectively designed to store 280 million tons (Mt) of mined overburden material. A conceptual OSF closure and water management plan is also provided.

1.2 OSF Design Overview

Climax is an open pit Molybdenum mine utilizing a truck/shovel operation. Ore extracted from the pit is sent to the mill for mineral extraction. Material with marginal mineralization at or slightly above the cutoff grade will be temporarily stockpiled in the low grade stockpile, located near the mill site, adjacent to Highway 91, on top of the south portion of the North 40 OSF, as shown on Drawing 14. The low grade stockpile is expected to be removed and processed near the end of mine operations, after extraction and processing of higher grade materials from the pit. The overburden materials excavated from the pit are trucked to the OSFs. Overburden material at the site consists of three different material types. The majority of the overburden will consist of igneous and metamorphic rock excavated from the mine pit, which is either unmineralized or contains uneconomical mineralization. The ore body is primarily quartz monzonite porphyry, while some migmatite is present surrounding the ore body. Approximately 30% of the overburden removed from the pit will consist of sedimentary rock, primarily derived from the Minturn Formation shale, siltstone, and sandstone.

This design report is focused on supporting the proposed North 40 and McNulty OSF designs. The designs for both facilities focus on meeting three goals:

- Design for long-term geotechnically stable slopes.
- Provide a generalized operations management plan for surface water and groundwater. More specifically, include the grades, channels, berms, drains, and hydraulic barriers necessary to maintain separation of contact water and non-contact water, to the extent practicable.

- Integrate design for closure concepts to maintain compatibility of the operational OSF configuration with eventual closure, and provide closure design concepts for guiding operations.

The term "contact" water, as used in this report, refers to precipitation-related water and groundwater that has flowed over, through, or otherwise been in contact with mined materials, including overburden. The term "non-contact" water, as used in this report, refers to precipitation-related water and groundwater that has not been in contact with mined materials. Water may retain its "non-contact" designation after being allowed to flow over or through engineered cover materials or engineered drainage layers derived from non-acid generating sources, as well as undisturbed native material.

The OSF designs presented in this report were developed jointly through collaboration between Climax and Golder. In order to guide the design process and form a framework for evaluating the suitability of the designs, Golder and Climax developed a set of project design criteria. These design criteria (presented in Appendix F) provide:

- Relevant design inputs (e.g., design storms, design seismic events, etc.)
- Guidance on which engineering analytical methods will be used
- Criteria for the design components (e.g., include underdrain system)
- Geometrical constraints (e.g., allowable slopes, bench widths, channel criteria, freeboard requirements, etc.)
- Acceptable factors of safety (FOS)

The designs developed for the McNulty and North 40 OSFs are presented in the project drawings (Drawings 1 through 15, attached). The grading plans shown on Drawings 13, 14, and 15 were developed from the Climax mine plan to illustrate intermediate OSF development for years 5, 10, and 15, respectively. The components of the design and the design process are described in the subsequent sections of this report. A summary of the major design components is provided below:

- During placement of overburden materials, intermediate OSF configurations will be compatible with the OSF stability requirements and water management strategy. An O&M plan will be implemented to facilitate conformance with these design objectives and criteria.
- The McNulty OSF will contain an underdrain system to capture existing water springs within the OSF footprint and transmit the water from springs to the toe of the OSF, to prevent contact between the captured water and overlying OSF fill material (see Drawings 3 and 12).
- The McNulty OSF will contain a contact stormwater collection system (CWCS), as shown in plan on Drawings 3 and 12. The purpose of this system is to collect contact stormwater internal to the OSF and transmit it to the east tailing delivery line (ETDL) or East Side Channel, and then to the Climax water treatment system prior to discharge.
- The overall final slope of the OSFs, during operations and after closure, will not exceed 2.4H:1V.

- During operations, this slope will be achieved by constructing nominal 200 ft high angle-of-repose lifts separated by 200 ft wide horizontal benches. The final operating OSF configuration is shown in plan on Drawing 3.
- Prior to closure, the OSF slopes will be regraded. The regraded interbench slopes will be 2H:1V and nominally 125 ft long. The regraded slopes will be separated by horizontal benches nominally 20 ft wide with a nominal 2% slope gradient.

- At closure, the toe limits of the OSFs will expand because of slope regrading. This is due to excavation of material from the operational lift crests and fill placement of material at the toes of the operational lifts. The operational limits of the OSFs have been located to be compatible with toe expansion at closure to provide a 2H:1V interbench outslope. The operational toe limits provide sufficient space to allow extension of the slope for the final reclamation limits plus a 50 foot (minimum) buffer between the closure toe limits and any critical utilities, property boundaries, watershed boundaries, or other areas where overburden materials cannot be placed. The final OSF closure configuration is shown in plan on Drawing 4.
- At closure, a reclamation cover will be constructed on the OSF. The objective of the reclamation cover will be to minimize erosion, reduce infiltration, minimize contact between stormwater and OSF fill material, and promote revegetation.
- At closure, channels constructed on the regraded benches will transport non-contact surface water to downdrains, which will run down the OSF outslopes to perimeter channels and/or water management berms, which will direct flows to either the East Interceptor System or other conveyances such as the East Side Channel which will be converted to a clean water diversion system when the Tenmile and Mayflower TSFs are reclaimed.
- The OSFs will be constructed with perimeter surface water channels and/or water management berms.
  - During operations, the perimeter berms and channels will be used to prevent non-contact stormwater from flowing onto the OSF surface, and to prevent runoff of contact stormwater from the OSFs to surrounding unimpacted areas.
  - Following completion and regulatory approval of reclamation, storm water run-off from the reclaimed OSFs will be classified as non-contact stormwater. The perimeter channels and berms will collect and convey non-contact stormwater flow to the East Interceptor System or other conveyances for discharge from the site (See Drawing 4). Where necessary, channels and/or berms will be utilized to prevent run-on from watersheds upgradient of the OSFs.
- At closure, additional toe drains will be constructed where required along the toes of both OSFs. The toe drains are intended to capture contact water internal to the OSFs at those locations where there is potential for contact water to exit the toes of the OSFs as seepage. The toe drains will transmit collected water to the toe of the facility and transfer it to the ETDL or other conveyance for treatment.

2.0 SITE CHARACTERIZATION

2.1 Review of Existing Information

2.1.1 Historical Reports and Documents

Golder reviewed available reports and other relevant information in order to develop an understanding of the North 40 and McNulty Gulch areas, identify available geotechnical information (e.g., existing boring logs, laboratory testing, piezometric data, geologic maps, etc.), and identify data gaps where additional data was required to support the geotechnical stability evaluation. Portions of the existing information were directly applicable for use as inputs for developing the geologic model and material parameters used in the stability evaluation. The available information was also used by Golder to focus the subsequent field and laboratory programs to collect additional information in those areas where insufficient information existed or where the greatest potential for geotechnical concerns were identified.

Golder completed a historical data review that included a review of aerial photographs, historical topography, and surface geologic maps of the Climax site. Photographs were available from 1938, 1944, 1955, 1956, 1957, 1958, 1967, 1969, 1971, 1977, 1978, 1979, 1980, 1985, 1990, and 2006. Historical topography was available from 1882, 1939, 1955, 1970, 1981, 1985, 1990, and 2010. Some photographs and topographic maps only showed portions of the site, and some of the photos were previously reproduced or scanned at relatively low resolution and were not useful. The primary focus of the historical data review was to estimate the depositional history and limits of the various materials within the footprint limits of the OSFs (e.g., buried tailing impoundments, mine waste, random fill, etc.) as well as the location of major geologic features with respect to the proposed OSFs (e.g., the Mosquito Fault). Other reports or information reviewed by Golder include the following:

- The proposed ultimate grading plan for the OSFs, provided by Climax on 17 February 2012
- Seismic hazard maps and data available on USGS seismic hazards program website
- *Slope Stability Evaluation For Feasibility Level Pit Design of Long Range Reserves at Climax Mine.* Call and Nicholas, Inc. August 2007
- *Geology of the Northern Part of the Tenmile Range, Summit County, Colorado*. USGS. 1963
- *Ore Deposits of the Kokomo-Tenmile District, Colorado*. USGS. 1971
- *The Climax Molybdenum Deposit, Colorado*. USGS. 1933
- Portions of the AM-06 permit amendment, including the following:
  - Exhibit C – Mining Plan Maps
  - Exhibit D – Mining Plan
  - Exhibit E – Reclamation Plan
  - Exhibit F – Reclamation Plan Maps

- Exhibit G – Water Information
- Exhibit I – Soils Information
- Exhibit K – Climate Information

- *McNulty OSF Cut-Off Design*. AMEC. November 2008

2.1.2 Overview of Existing Conditions

Drawing 2 depicts the existing conditions which are further discussed in this section. Based on review of the above referenced sources, the proposed McNulty and North 40 OSF areas are located between approximately 11,200 and 12,500 ft above sea level. Elevations are lowest along the western sides of the proposed OSFs, which are roughly parallel to the ancestral Tenmile Creek and Colorado Highway 91. The lower elevations are covered with glacial deposits, although much of the area has been modified by various mine and transportation construction projects. A historical tailing impoundment lies in the southwest corner of the proposed North 40 OSF, which was operated in the first half of the 20th century. After tailing deposition ceased, several buildings were constructed on the southern portion of the tailing, and a stockpile of fill or mine overburden has been placed on the northern portion.

The bedrock beneath the majority of the OSF footprints consists of various strata of the Minturn formation, which has been intruded locally by the Lincoln porphyry. Locally, the Minturn generally dips at 30 to 45 degrees in an east to north-northeast direction. Soils at the site are generally shallow, and are predominantly composed of residual and colluvial soils weathered from the underlying bedrock. The Mosquito Fault passes along the far northeast boundary of the McNulty OSF footprint, and several smaller splay faults lie within the northeast corner of the facility.

A general timeline of mine activity in the OSF footprint areas, as determined based on Golder's historical review of the topography and aerial photographs is provided below:

- Between 1882 and 1938 tailing placement begins.
- Between 1944 and 1955 tailing placement is complete and construction of mine buildings on the south tailings deposit occurs.
- Between 1971 and 1977 fill or mine overburden placement begins on the northern portion of the tailing impoundments which are covered with fill materials. Road construction and minor fill or mine overburden rock placement begins north of the tailing, adjacent to McNulty gulch.

In more recent years, starting in about 1971, fill and mine overburden has been placed north of the buried tailing impoundments, adjacent to the south side of McNulty Gulch, and within the south fork of the gulch. Based on the 2010 topography, it appears that a maximum thickness of approximately 300 feet of material has been placed in the south fork of the gulch. The average thickness outside the gulch is approximately 50 feet or less. The north fork of the McNulty Gulch does not currently contain any overburden materials. The north fork watershed contains several perennial springs within the proposed

OSF footprint. Non-contact stormwater and the perennial stream flows are directed into the East Interceptor and conveyed to Clinton Reservoir to the north, and ultimately discharges to the Tenmile Creek drainage basin.

The site receives an approximate average of 23 inches of precipitation per year. Average daily high and low temperatures range from 65° F and 39° F in July, respectively, to 25° F and 2° F in January. A detailed description of the climatic conditions at the site can be found in Exhibit K of the AM-06 permit amendment (Climax, 2010).

In disturbed areas and areas where fill or mine overburden material has been placed, vegetation is sparse. Undisturbed areas consist of a mixture of forested and grassy areas, with some short woody shrubs present adjacent to drainages and springs.

2.1.3 Existing Infrastructure

There is an extensive network of existing utilities and infrastructure at the Climax mine site. Where necessary, the OSF designs have been developed to avoid conflicts with these utilities. The OSF designs also consider how to appropriately integrate the proposed surface water management features with the existing channels and infrastructure. A summary list of the key existing utilities and other infrastructure within or adjacent to the OSF footprint areas is provided below:

- East Tailing Delivery Line (ETDL) –The ETDL runs roughly south to north on the east side of Highway 91 to just south of McNulty Gulch, and then remains on the west side of the highway to the sludge densification plant (SDP) (see Drawing 2). The ETDL currently collects contact water from the camp area and several other facilities. It is Golder understanding that the ETDL will remain undisturbed and active throughout operations and during the closure period. The ETDL may be used to transmit contact water collected from the OSF CWCS or perimeter channels to the Climax water treatment system.
- Tailing Delivery Line (TDL) – this line runs adjacent to the ETDL, and delivers tailing and water from the mill to the tailing impoundments. This pipeline also may be used to transmit contact water at closure.
- High pressure natural gas and above ground electrical lines – these utilities run north/south in a utility corridor along Highway 91 (see Drawing 2). These lines are more distant (west) than the ETDL, and will not be impacted by the OSFs.
- Camp drainage line – The camp drainage line is located along the south toe of the North 40 OSF, adjacent to the low grade ore stockpile, and removes contact water from the camp area. The line ties into the ETDL. Climax indicated this line should remain un-buried and in-service throughout operations and during the closure period for transmission of contact water.
- East Side Channel (ESC) – this channel transports contact water from the Camp area, North 40 OSF, Robinson Pond, and McNulty Gulch. The channel runs mostly on the west side of Highway 91, and will not be affected by OSF construction. The ESC can be utilized as an outlet for contact water from the North 40 and McNulty OSFs. At closure portions of the East Side Channel will be converted to a clean water diversion along the

east side of the Tenmile and Mayflower TSFs and can potentially be used to convey non-contact water from the North 40 and McNulty OSFs at that time.

- Miscellaneous decommissioned utilities – it is Golder's understanding that several decommissioned utilities (including gas and potentially communications lines) cross the northern portion of the North 40 and southern portion of the McNulty OSF footprints. Because the utilities are decommissioned, it is Golder's understanding that these utilities may be buried in-place during OSF construction.
- Clinton Ditch – this ditch has its origin in the headwaters of the Clinton Creek, and was used historically to divert non-contact water south towards McNulty gulch. The ditch is not currently in use but may be considered for future water diversions.
- East Interceptor – this channel and pipeline system collects non-contact surface water flows from McNulty Gulch and from the area between McNulty Gulch and Clinton Gulch. The channel and pipeline run east to west along the north side of McNulty gulch and then turns north and runs along Highway 91 to the Clinton Reservoir. The East Interceptor will be used as the discharge point for non-contact water collected on or around the McNulty OSF, including flows from the underdrain system and potentially from non-contact channels at closure.
- McNulty Interceptor – this channel diverts or has diverted flows from the central McNulty drainage north towards the East Interceptor. The McNulty Interceptor will be covered during OSF operations. The function of the McNulty Interceptor will be replaced by the McNulty OSF underdrain system and perimeter channels and berms.
- DSM Interceptors – this series of interceptors collect contact water from the camp and North 40 OSF areas and discharges into the ETDL. It is Golder's understanding that this interceptor collection system will generally remain in place during OSF operations, with some modifications to accommodate the camp area drains and OSF CWCS.
- Storage Tanks – there are several storage tanks along the haul road bounding the east side of the OSF. The OSFs will not impact these tanks during operations. Some tanks may be decommissioned at closure, allowing the OSF toe to move east during reclamation.
- Reclaim Pipeline – The reclaim pipeline is located along the south toe of the North 40 OSF, adjacent to the camp drainage line and along the Colorado Boulevard south of the truck shop. This buried pipeline conveys process water to the mill water storage tank and will remain in-service throughout operations.
- Utility Corridor – there are several pipelines to convey and distribute water and gas utilities to the storage tanks and the mill or appurtenant buildings. These pipelines are run west to east from the Mill building along the road below the Primary Crusher, turn south to north at the Truck Shop bench, turn west to east at the truck wash and run up to the bench above for connection to the storage tanks.
- Truck Shop, Truck Wash, Phillipson Warehouse, Maintenance Shop – these facilities lie on a bench just to the east of the southern North 40 OSF.

2.2 Field Investigation

2.2.1 Overview

The primary purpose of the field investigation was to collect the field data and soil/rock samples necessary to support the stability evaluation of the proposed North 40 and McNulty OSF designs. More specific objectives of the investigation were:

- Classify the foundation soils in accordance with the Unified Soils Classification System (USCS)
- Provide estimates of the depth to bedrock and the thickness of overlying strata (soil/fill/tailing) within and adjacent to the proposed OSF footprint
- Provide disturbed and undisturbed samples of soil and rock for laboratory testing
- Provide data on soil and rock types and distribution throughout the site
- Provide data on surface hydrology features based on field reconnaissance and groundwater elevations within the OSF footprint areas

To meet these goals, a test pit program and a geotechnical drilling program were completed. The field program also included visual observations, strike and dip measurements, point load testing of rock samples, and general site reconnaissance. A detailed description of the field program is included as Appendix A, which includes drilling methods, field testing methods and results, soil and rock sampling, test pit logs, boring logs, and a photographic log. The locations of the test pits and borings are shown on Drawing 2. Summaries of the test pit, drilling, and point load test programs are provided in the following sections.

2.2.2 Test Pit Program

2.2.2.1 Methods

The test pit program consisted of excavating, logging, and sampling 19 test pits between October 10 and October 13, 2011. The test pit program focused mainly on mapping, characterizing, and sampling the shallow materials within the OSF footprint areas. The test pits allow for the collection of larger soil samples and are more suited for sampling and characterizing gravelly and cobbly soils. The characteristics that were logged during the test pit program include density, color, weathering, grain size, angularity, structure, parent/source rock, plasticity and moisture as well as any pertinent observations made during the excavation (e.g., groundwater conditions, soil/fill density, strata strike and dip, etc.).

Moltz Construction was contracted to perform the test-pitting program and a John Deere 240D tracked excavator was utilized to excavate the test pits. All test pits were staked and cleared with the Climax's blue stake crew prior to excavation. Test pits were excavated a minimum of 4 ft (when hard conditions were encountered) and a maximum of 16 ft. Wherever possible, test pits were excavated to bedrock refusal. Representative samples of the various materials encountered were obtained as bulk (pail) samples for testing in Golder's Denver laboratory. All test pits were backfilled after excavation and logging activities were complete and compacted with the excavator bucket and excavator tracks.

2.2.2.2 Conditions Encountered

Four test pits were excavated in mine overburden materials. These pits include GA-11P-04, GA-11P-05, GA-11P-08, and GA-11P-10. The mine overburden in these test pits all classify as a well graded

GRAVEL (GW) with small variations in origin, gradation, color, and overall composition. These test pits varied in depth from about 4 ft BGS to about 10 ft BGS. No groundwater was encountered in any of these test pits.

Four test pits were excavated in non-waste rock fill materials that were generally located on or near mine roads, road embankments, or in the lay-down area. These pits include GA-11P-11, GA-11P-12, GA-11P-14, and GA-11P-19. These pits varied in depth from about 5 ft BGS to approximately 16 ft BGS, and no groundwater was encountered in any of these test pits. These materials were generally classified by the USCS as well graded SAND (SW) or well graded GRAVEL (GW).

Five test pits were excavated in areas containing residual soils weathered from the Lincoln Formation (Lincoln) porphyry. These test pits included GA-11P-01, GA-11P-03, GA-11P-09, GA-11P-15, and GA-11P-16. The total depth of these test pits varied from approximately 5 ft BGS to approximately 13.5 ft BGS, and no groundwater was encountered in any of these test pits. The Lincoln porphyry encountered in the test pits was generally characterized as a highly weathered weak rock to completely weathered very weak rock (i.e., residual soil). The residual soil was well graded SANDY GRAVEL (GW) or well graded GRAVELY SAND (SW) with some very weak cobbles and boulders. The bedrock structure was generally intact and visible, even in the completely weathered/residual soil zones. The in-situ rock was gray and massive with an aphanitic to very coarse crystalline structure. The soils above the Lincoln porphyry were generally sandy and were either colluvial or residual soils.

Four test pits were excavated in areas where the Minturn Formation (Minturn) was present at or near the surface. These test pits include GA-11P-02, GA-11P-06, GA-11P-13, and GA-11P-17. These test pits varied in depth from approximately 5 ft BGS to 15 ft BGS. Groundwater was encountered only in test pit GA-11P-06 where it was clearly seeping into the pit from a depth of approximately 4 ft. Sandstone was encountered at approximately 2 ft BGS in test pit GA-11P-13 and at approximately 4 ft in GA-11P-17. The strike and dip of the Minturn formation was measured as N100E at 40 degrees in GA-11P-13 at a depth of approximately 5 ft. The Minturn encountered in the test pits was characterized as a slightly to moderately weathered, thickly bedded, medium to coarse crystalline micaceous SANDSTONE. The thickly bedded sandstone layers were separated by thinner shale beds or clay infill (approximately 0.5 cm to 3 cm thick). The clay infills were generally red to orange low to highly plastic CLAY (CL-CH). The soils above the Minturn bedrock tended to be low plasticity SILT (ML) and low plasticity CLAY (CL) with some sand and gravel, and were generally a red-brown or maroon color.

Test pit GA-11P-07 was excavated in glacial till material. Directly above and next to the test pit, approximately 5 ft of native material was exposed and logged in addition to the 5 ft of material that was excavated below the ground surface. Seeping groundwater was encountered in this test pit at approximately 3 to 5 ft BGS. Hard digging was encountered at 5 ft BGS. The materials in this test pit were classified as low plasticity SILT (ML) and low plasticity CLAY (CL) with sand, gravel, and cobbles.

Finally, a single test pit was excavated east of Highway 91 and just west of the estimated limits of the historic tailing impoundments. The total depth of test pit GA-11P-18 was approximately 16 ft BGS, and ponded groundwater was noticeable at the base of the test pit prior to backfilling. The materials in the upper 4 ft of the test pit were characterized as poorly graded to well graded SAND (SP) with little gravel and cobbles and trace boulders. From 4 to 6 ft BGS, the material was classified as a SILTY SAND (SM) with gravel and from 6 to 16 ft BGS, the material was classified as a dense, dark gray, SILTY SAND (SM).

2.2.3 Geotechnical Drilling Program

2.2.3.1 Methods

The drilling program included 7 borings spread strategically throughout the footprint of the proposed OSF footprints. The program began on October 18 and continued through November 4, with a 5-day break during the drilling program. Drilling facilitated obtaining information and samples from deeper strata, in-situ measurement of soil strength and density through the use of standard penetration testing (SPT), and also allowed installation of temporary piezometers for measuring groundwater levels within the OSF foundations. During drilling a Golder engineer logged soil and rock types, moisture, density, color, weathering, strength, grain size, angularity, lithology, plasticity, structure, rate of advance and other characteristics. Disturbed samples of the major soil types were obtained for laboratory index testing to confirm the field characterization and classifications (e.g., grain size and Atterberg limits) and for reconstituting samples for large scale tests (e.g., proctors, direct shear tests, etc.). In addition to the disturbed samples, relatively undisturbed samples were obtained by pushing thin walled Shelby Tubes into fine grained horizons (e.g., silts and clays) for laboratory testing. These samples were utilized for triaxial strength tests, in-situ natural moisture and density measurements, and consolidation tests.

Due to the granular and variable nature of the expected drilling conditions, Golder recommended the use of sonic drilling for this project. Sonic drilling utilizes rotation and high frequency vibration to advance an inner core barrel and an outer casing, thus allowing for a continuous sample of soil or rock to be collected in the inner barrel. Sonic coring is particularly well suited for drilling through hard, coarse soils and soils prone to caving, such as coarse glacial, alluvial, and colluvial sediments as well as man-made fills and coarse mine overburden materials. Sonic drilling is also capable of drilling through silts, clays, and other soft soils. Most sonic rigs can also drill through intact bedrock, although at reduced rates and with some breakage. For this drilling program, a Boart Longyear GP24-300RS sonic drill rig was utilized. The rig was equipped to facilitate SPTs and Shelby Tube sampling at specified intervals. When competent bedrock was reached, the Boart drill rig was capable of switching over to HQ diamond bit triple barrel rock coring (2.5-inch diameter), which allowed for improved rock core recovery.

The depth of the 7 boreholes ranged from a minimum of approximately 30 ft to a maximum of approximately 300 ft with a total drilling depth of approximately 704 ft. Drilling was advanced to bedrock at 5 of the 7 boring locations.

The sonic drill rig provided continuous core samples for each hole with nearly 100% recovery. Soil and rock samples were returned in runs ranging from approximately 5 to 15 feet in length. Non-cohesive soils were generally returned in a disturbed state, however very dense, stiff, and/or cohesive soils were returned relatively intact and only slightly disturbed.

Each borehole location was staked and approved by the Climax blue stake crew prior to drilling. After drilling was completed at each borehole, a 1-inch diameter PVC standpipe was lowered to the base of the borehole prior to removal of the drill casing. The bottom 10 ft of each standpipe was slotted. These standpipes were used to facilitate water level measurements. Water levels were allowed to equilibrate until the piezometric levels stabilized between readings, generally for a minimum of 24 hours, before recording the final static water table elevation. At the end of the drilling program, all borings were backfilled with cuttings to the water table and with bentonite chips above the water table in accordance with Colorado requirements (2 CCR 402-2).

2.2.3.2 Conditions Encountered

The conditions encountered in each of the 7 borings, including approximate location, materials encountered, depth to bedrock, depth to groundwater, and total drilling depth are provided in Appendix A and summarized in Table 1. A more detailed summary is also provided below:

- GA-11B-20: This borehole was located near the base of McNulty Gulch. The total depth of the borehole was approximately 54 ft. Groundwater was measured at a depth of approximately 24.8 ft BGS. The upper 38.5 ft of material in this location consisted of colluvial soil weathered from the Minturn formation. This soil ranged from low plasticity SILT (ML) to SILTY SAND (SM) with gravel and little cobbles (COLLUVIAL – MINTURN). The cobbles and gravel were sub-rounded micaceous sandstone with R3 (medium strong) to R4 (strong) strength. From approximately 36.5 ft to 38.5 ft BGS, a number of porphyritic cobbles were encountered. At 38.5 ft BGS, there was a sharp transition from colluvial Minturn to residual soil weathered from the Lincoln porphyry. This residual porphyry extended from 38.5 ft to the bottom of the hole at 54 ft BGS. The residual porphyry was classified as dense, gray, SANDY GRAVEL (GW) with cobbles (RESIDUAL SOIL – LINCOLN).
- GA-11B-21: This borehole was located on the existing OSF within the southern portion of McNulty Gulch. The total depth of this borehole was approximately 78.5 ft. The static water table was measured at approximately 42.9 ft BGS. The upper 25 ft of material in this location was a loose to compact, brown to red-brown, well-graded, SANDY GRAVEL (GW) (MINE OVERBURDEN). Native ground was reached at 25 ft BGS and consisted initially of a very stiff maroon CLAY (CL) with some sand and trace to little gravel (RESIDUAL SOIL – MINTURN). This material appears to have weathered from the upper Minturn formation. The stiff clay transitioned into a compact, maroon SANDY GRAVEL (GW) with clay (RESIDUAL SOIL – MINTURN) at approximately 33 ft BGS and this material continued to a depth of approximately 43 ft BGS. At approximately 43 ft

BGS, the gravelly material transitioned into a weathered, but more intact SANDSTONE (MINTURN). Due to difficult drilling, Boart drilled with water from 43 to 60 ft BGS. This allowed for the recovery of 4 to 8 inch pieces of intact Minturn sandstone core. This material was moderately to slightly weathered, bedded (thick sandstone beds separated by thinner clay layers approximately 0.5 cm to 3 cm thick), fine to coarse crystalline SANDSTONE (MINTURN). At 60 ft BGS, Boart transitioned to triple barrel HQ rock coring in an attempt to obtain rock core samples for laboratory testing. Boart drilled 5 runs of core that returned a slightly weathered, bedded, maroon, very fine to coarse crystalline SANDSTONE (MINTURN). The Rock Quality Designation (RQD) for this interval ranged from 17 to 48. The strength ranged between R1 (weak) to R4 (strong).

- GA-11B-22: This borehole was located on a hillside on the edge of a mine road near the base of McNulty Gulch. The total depth of this hole was approximately 33.5 ft BGS. Groundwater reached equilibrium at approximately 12.0 ft BGS. The upper 10 ft of material in this location consisted of compact to dense, brown-gray CLAYEY SAND (SC) and SANDY GRAVEL (GW) with cobbles (FILL). The cobbles in the fill were mostly from the Lincoln porphyry. At 10 feet there was a transition to dense gray SANDY GRAVEL (GW) with fines (RESIDUAL SOIL – LINCOLN). This material was a residual soil derived from the Lincoln porphyry. From 10 ft to 28 ft BGS, the material gradually transitioned from residual soil to in-place but completely weathered PORPHYRY (LINCOLN) with strength between R0 (extremely weak) and R3 (medium strong). At 28 ft, Boart transitioned to triple barrel HQ rock coring in an attempt to obtain intact core. Boart drilled 2 runs with limited success. Approximately 1 ft of gravel was returned in run number 1 and the core barrel plugged during run number 2 due to lack of water during drilling.

- GA-11B-23: This borehole was located on top of the existing OSF just north of the lay-down yard and directly above the northern historic tailing impoundment. The total depth of this borehole was approximately 294 ft and the water table reached equilibrium at 179.8 ft BGS. The upper 179 ft of material in this location was classified as mine overburden. The mine overburden varied between well-graded GRAVEL (GW) and well-graded SAND (SW) (MINE OVERBURDEN). This mine overburden material was generally sub-angular to angular, with non-plastic to low-plasticity fines. Tailing was encountered at 179 ft BGS, and ranged from a non-plastic SILTY SAND (SM) to a low plasticity CLAYEY SILT (ML) (TAILING). The thickness of the tailing layer was approximately 56 ft in this location. From approximately 226 to 235 feet BGS there was a short transition zone where the tailing had infiltrated the native glacial till. Below 235 feet BGS, soils encountered consisted entirely of glacial till. The glacial till generally ranged from a well-graded GRAVEL (GW) with silt and clay to a CLAYEY SAND (SC) with gravel and cobbles (TILL). The exception was a low plasticity CLAY (CL) layer present from approximately 273 ft to 287 ft BGS.

- GA-11B-24: This borehole was located in the gravel crushing area directly above the northern most portion of the southern historical tailing impoundment. The total depth of this borehole was approximately 164.5 ft and the water table reached equilibrium at approximately 54.9 ft BGS. The upper 45 ft, at this location, was comprised of mine overburden. The mine overburden generally ranged from well-graded GRAVEL (GW) to CLAYEY GRAVEL (GW) to CLAYEY SAND (SC) (MINE OVERBURDEN), although there were some zones of low plasticity SANDY CLAY (CL). The tailing contact was located at approximately 45 ft BGS. The majority of tailing encountered in this location was comprised of low plasticity CLAYEY SILT (ML) (TAILING) with some thin layers of non-plastic, poorly graded, fine SILTY SAND (SM) (TAILING). The tailing layer was approximately 53 ft thick in this location. Below the tailing layer, drilling encountered glacial till. The glacial till in this location ranged from low plasticity SILT (ML) to SILTY SAND (SM) to CLAYEY SAND (SC) to well graded GRAVEL (GW) (TILL). All layers contained some sub-rounded gravel and cobbles of various origins and parent rock types. This borehole was terminated at 164.5 ft prior to reaching bedrock.

- GA-11B-25: This borehole was located in the lay-down yard near the southernmost toe of the proposed OSF. In addition, this borehole was located near the center of the southern historical tailing impoundment. The total depth of this borehole was approximately 193 ft, with static groundwater levels measured at approximately 33.9 ft BGS. The upper 7 ft of material in this location consisted of a well-graded GRAVEL (GW) (FILL). Mine overburden was encountered from 7 ft to 33.5 ft BGS and consisted of well-graded GRAVEL (GW) and CLAYEY SAND (SC) with gravel and cobbles (MINE OVERBURDEN). The tailing contact was at approximately 33.5 ft BGS. Tailing in this location consisted mainly of non-plastic, poorly graded, fine SILTY SAND (SM) with some thin layers of CLAYEY SILT (ML) (TAILING). The total thickness of the tailing layer in this location was approximately 38.5 ft. Below the tailing layer there was a 4 ft layer of stiff, dark brown, ORGANIC SILT (OL), before glacial till was encountered at 76 ft BGS. The glacial till layer continued for approximately 95 ft. The till in this area consisted mainly of well-graded SAND (SW) with gravel and cobbles and well graded SANDY GRAVEL (GW) (TILL). The Lincoln porphyry was encountered at approximately 171 ft BGS and can generally be described as a completely to moderately weathered bedrock or residual soil. The residual soils weathered from the porphyry in this location ranged from a stiff to hard, gray-brown CLAY (CL) with low plasticity to a dense, gray, poorly graded coarse SAND (SP), slightly moist and non-plastic (LINCOLN).
- GA-11B-26: This borehole was located on native ground above McNulty Gulch, near a mine road. The total depth of this borehole was approximately 36 ft BGS and the groundwater reached equilibrium at a depth of approximately 13.1 ft BGS. The soil in this location was comprised of material weathered from the Lincoln porphyry. This residual soil can generally be described as dense, gray, SANDY GRAVEL (GW) with cobbles. From 31 to 36 feet BGS, the residual material transitioned into a moderately weathered, massive, fine to very coarse crystalline PORPHYRY (LINCOLN) bedrock. The rock was highly fractured, but the larger cobbles returned had strength rated as R4 (strong).

2.2.3.3 SPT Test Results

Twenty (20) SPTs were performed in various materials during the drilling program. Two tests were performed in soils derived from the Minturn Formation, nine tests were performed in mine overburden materials, five tests were performed in the tailing materials, three tests were performed in the glacial till, and 1 test was performed in soil derived from the Lincoln porphyry.

Table 6 in Appendix A provides a summary of all the SPT tests performed throughout the drilling program. This table presents the field N values, the corrected N_{60} values, and the corrected $(N_1)_{60}$ values. The field N values are corrected for field conditions and normalized to standardized N_{60} values. The N value correction factors include hammer efficiency, borehole diameter, sampler type, and rod length. The N_{60} values are further corrected based on the effective overburden stress to produce normalized $(N_1)_{60}$ values.

$(N_1)_{60}$ values for soils derived from the Minturn formation ranged from 10 to 42. $(N_1)_{60}$ values for mine overburden varied between 6 and 27. $(N_1)_{60}$ values for tailing ranged from 6 to 9. $(N_1)_{60}$ values for the glacial till ranged from 18 to >50. The single test performed in the Lincoln porphyry gave a $(N_1)_{60}$ value of 29.

2.2.4 Point Load Testing

Thirteen point load tests were performed on samples of the Minturn sandstone collected from borehole GA-11B-21, eight point load tests were performed on samples of the Lincoln porphyry collected from borehole GA-11B-26, and thirty-five point load tests were performed on lump samples of mine overburden collected from the ground surface at various locations throughout the existing mine overburden piles in the North 40 and McNulty Gulch areas. All point load tests were performed using International Society for Rock Mechanics (ISRM) guidelines.

The results of the point load tests indicate variability in the strength of the Minturn sandstone, the Lincoln porphyry, and the mine overburden. Weathering, which varies considerably between samples, and preferential failure planes (i.e., bedding planes and/or joints) contribute significantly to the measured strengths. In order to perform a point load test, samples of a minimum size are required. Because weaker strata within the Minturn formation generally weathered to fragments too small for testing, the tests performed were biased towards the stronger rock that more frequently yielded samples of sufficient size for testing. The point load testing results from the Minturn sandstone, the Lincoln porphyry, and the mine overburden piles are presented in Tables 3, 4, and 5 of Appendix A, respectively. In summary, the unconfined compressive strength (UCS) of the Minturn samples ranged from 3 to 15 ksi with an average of 10 ksi. The UCS of the Lincoln porphyry samples ranged from 5 to 21 ksi, with an average of 12 ksi. The UCS of the mine overburden samples ranged from 3 to 27 ksi, with an average of 12 ksi.

2.3 Laboratory Testing Program

2.3.1 Methods

All laboratory testing was performed in either Golder's Denver or Atlanta certified geotechnical laboratories. All laboratory testing procedures are in accordance with the American Society for Testing and Materials (ASTM) standards where applicable. Soil samples were classified using the Unified Soil Classification System (USCS) (ASTM D2487). The laboratory tests performed on various samples and the associated standards are summarized below:

- Sieve Analysis – ASTM C117/C136
- Hydrometer/Sieve/Specific Gravity – ASTM D422
- Atterberg Limits – ASTM D4318
- Specific Gravity – ASTM D854
- Standard Proctor Compaction Testing – ASTM D698
- Modified Proctor Compaction Testing – AASHTO T180 Method A
- Minimum Index Density Determination – ASTM D4254
- Consolidated-Undrained (CU) Triaxial Compression – ASTM D4767
- One-dimensional Consolidation Testing – ASTM D2435

- Natural Density and Moisture Content – ASTM D2937 and D2216
- Large Scale Direct Shear Testing – ASTM D3080 (Modified)
- Jar Slake Durability Testing – Kentucky Method 64-514-02

The methodologies and results of all laboratory tests are presented in Appendix B. A summary of the test results is provided below.

2.3.2 Results Summary

Testing was strategically assigned to each material type in order to define the necessary index and engineering properties. The results of the index tests performed are summarized in Table 1 in Appendix B. Engineering testing consisted of a consolidation test on fine tailing and various strength tests. Strength tests included a staged consolidated undrained (CU) triaxial test on soil weathered from the Minturn Formation, two large scale direct shear tests on mine overburden samples, a large scale direct shear test on glacial till, a large scale direct shear test on soil weathered from the Lincoln Porphyry, and a series of CU triaxial tests on fine tailing samples extruded from Shelby tubes. Summaries of the results of all strength tests and of the consolidation testing results for the fine tailing are presented in Appendix B as Tables 2 and 3, respectively. A brief description of each material type, and summary of the laboratory test results, is provided below.

- Minturn Formation (and soils weathered from the formation) – this formation generally consists of interbedded shales, siltstones, and sandstones. In the North 40 and McNulty Gulch area, the formation tends to dip at approximately 30 to 45 degrees towards the east to north-northeast. Based on laboratory classifications, the formation tends to weather to silty or clayey SAND (SM, SC) near the surface. A staged CU triaxial test was performed on a sample of the finer grained clayey SAND (SC) soil weathered from the formation (obtained from GA-11B-21 at 28 ft). The sample was relatively undisturbed, and was taken from an intact piece of sonic core. The sample had 49% fines (material finer than the #200 sieve, or 0.075mm). The best-fit Mohr-Coulomb residual effective strength obtained from this test was 31 degrees with 0 cohesion.
- Lincoln Porphyry (and soils weathered from the formation) – this formation is a fine to coarsely crystalline volcanic porphyry, which has locally intruded into the host rock of the Minturn Formation. The rock is massive, with few evident foliations or bedding planes. Field classifications show that, depending on the degree of weathering, the formation tends to weather to well graded GRAVEL, SAND and silty or clayey SAND (GW, SW, SM, SC) near the surface. Laboratory testing of a typical sample confirm these findings, and classify the residual soil as a clayey SAND (SC) with 21% fines. A large scale direct shear test was performed on a sample of the clayey sand residual soil weathered from the formation (obtained from GA-11B-20 at 40-43 ft). The best-fit Mohr-Coulomb residual effective strength obtained from this test was 35 degrees with 18 psi cohesion.
- Glacial Till – glacial till is present in the lower parts of the valley containing the ancestral Tenmile Creek. Field classifications show the till tends to consist of a varying mix of compact to dense, well graded GRAVEL and SAND and silty to clayey GRAVEL and SAND (GW, SW, GM, GC, SM, SC). The till also contains some thin layers of stiff to hard low plasticity CLAY (CL). Laboratory tests performed on a sample judged to be representative of the till classified the material as a clayey GRAVEL (GC) with 25% fines. A large scale direct shear test was performed on a representative sample of the till (a

composite sample of material sampled from GA-11B-25 at 104-106 ft and GA-11B-23 at 249-251 ft). The best-fit Mohr-Coulomb residual effective strength obtained from this test was 31 degrees with 14 psi cohesion.

- Tailing – the tailing was created by historic milling of ores at the site, and is located in two historic impoundments on the south west side of the North 40 OSF footprint area. The tailing can be divided into two general facies depending upon the depositional distance from the decant location, beach tailing and fine tailing. The beach tailing consists of fine grained SILTY SAND (SM) with 25-36% fines, while the fine tailing consists of non-plastic SILT (ML) with 97-99% fines.

  A CU triaxial test was performed on undisturbed samples of the fine tailing obtained from Shelby tubes (GA-11B-24 at 51.5-53.6 ft). The best-fit Mohr-Coulomb residual effective strength obtained from this test was 33 degrees with no cohesion. Other testing included a 1D oedometer consolidation test on fine tailing (see Table 3, Appendix B), a specific gravity test, several natural moisture/density measurements, and determination of the minimum and maximum dry density for beach tailing. The median natural dry density for the tailing was approximately 98 pcf. The minimum and maximum dry densities of the beach tailing were measured to be 73.1 and 120.7 pcf, respectively. The specific gravity of the tailing was measured to be 2.75.

- Mine overburden (waste rock) – this material was derived from the overburden material excavated from the pit, which does not contain economic mineralization. This material was sampled from several locations in the North 40 and southern McNulty areas. Based on field classifications, mine overburden tends to consist of well graded GRAVEL, poorly graded GRAVEL, and some silty to clayey GRAVEL (GW, GP, GM, GC). Two laboratory analyses classified the mine overburden as clayey GRAVEL and as a poorly graded silty GRAVEL (GC and GP-GM). The two sieve analysis showed that 66-68% of the material is finer than ¾-inch, and that 9-14% of the material consists of silt and/or clay. However, note that the mine overburden rock samples taken to the laboratory were scalped of oversize material in the field, as transporting and testing representative samples of cobble to boulder sized rock is not practicable. The in-situ mine overburden rock material contains an estimated 15 to 40% oversize material, which should be considered when applying the results of the laboratory analyses.

  Jar slake testing was performed on two different samples of rock and both tests showed no degradation of the mine overburden over the 24 hour test period. Two large scale direct shear tests were performed on different samples of the mine overburden material (one sample from GA-11P-05, the other a composite sample of GA-11B-25 at 16 feet and GA-11B-23 at 38-40 ft). The best-fit Mohr-Coulomb residual effective strengths obtained from these tests, assuming no cohesion, were 35 degrees and 36 degrees, respectively, both with no cohesion.

- Miscellaneous Fill – miscellaneous fills are located throughout the site, with the majority occurring in the areas around the mine buildings and offices, in the vicinity of the laydown yard, and along roads and embankments. Relative to the other material types, the occurrence of miscellaneous fills is minor. No laboratory testing was conducted on miscellaneous fill materials. Based on field classifications, fills at the site consist primarily of well graded SAND and GRAVEL (SW, GW).

2.4 Material Design Strength Selection

This section describes the process used to select strength parameters for slope stability analysis. In general, strengths were selected based on an evaluation of data from several different sources, including:

- The results of the Golder laboratory test program

- Strengths developed in previous geotechnical reports
- Hoek-Brown strength envelopes (applicable for mine overburden, Minturn Formation, and Lincoln Porphyry). These strength envelopes were developed based on Golder point load data, laboratory UCS test data provided in previous reports, and on rock quality characteristics as encountered in the Golder field investigation
- Comparison with typical strength values published in the technical and scientific literature, such as the typical strength values published by Leps (1971) for rockfill (e.g., waste rock or mine overburden)

The strength envelopes for each material type were selected based on evaluation of the above information and engineering judgment, with the intent of realistically representing the actual strength, while with sufficient conservatism to account for inherent natural variability in material parameters and for potentially locally unfavorable conditions within the geologic units. In all cases, residual strengths were used for design rather than peak strengths. Peak strengths are determined by measuring the maximum resistance to shear provided by the sample during testing. This maximum resistance generally occurs at a relatively low strain (i.e., after relatively little shearing has occurred). For over consolidated clay or dense sand, the shear resistance generally decreases slightly after the peak and eventually reaches a steady residual value at large strains. For the OSF stability analysis, Golder considers residual strengths to be appropriate for design of the Climax OSFs. This is because some creep or movement of the mine overburden and shallow foundation soils may occur during initial loading of the OSF. Additional considerations for applying residual strengths include accounting for the potential of seismic deformations and/or previous slope displacements. However, because the Climax mine is not located in a highly seismic area, and no historical slides were identified or are known in the North 40 and McNulty areas, the use of residual strengths is not strictly necessary to counter these site factors. Nonetheless, residual strength were used for the OSF stability evaluations, which are believed to provide an inherent level of conservatism in the design.

The Call & Nicholas (2007) report contains summaries of historical laboratory test data and material strengths back-calculated from slope failures within the pit. The report is focused on samples and data collected from the Climax open pit area. The majority of the pit is excavated in the ore zone. The Mosquito Fault runs north/south through the western side of the pit, with the portion of the pit west of the fault excavated through the Minturn Formation and Lincoln Porphyry. Although the same formations outcrop along the pit west wall and within the OSF foundation areas, the portions of the formations (i.e., strata/beds) outcropping are different in both locations. As a result, although the data presented in the Call & Nicholas report was considered in the development of the design shear strength parameters for the OSF stability analysis, more weight was given to the laboratory test results and field data developed by Golder in the current OSF specific study. A table presenting a summary of the relevant data presented in the Call & Nicholas report is presented in Table 2. Tables showing the strength parameters selected by

Golder for the current static and pseudo-static stability analyses are shown in Tables 3 and 4, respectively.

2.4.1 Minturn Formation

The strength of the Minturn Formation, as determined by a 1984 back analysis of a pit slope failure, was reported as 29 degrees with 4.9 psi cohesion (Call & Nicholas, 2007). An additional rock-mass strength estimation performed by Call & Nicholas yielded a friction angle of 19 degrees with a cohesion of 33 psi. In their pit stability analysis, Call & Nicholas used a rock mass strength of 30.8 degrees with 895 psi cohesion.

Golder also considered the following additional factors in the determination of the Minturn Formation design shear strengths for the OSF stability analysis:

- One potential OSF failure mechanism is shearing through the shallow foundation soils or upper few feet of bedrock. The weathering of the bedrock is expected to decrease somewhat with depth, leading to increased shear strength. Also, as discussed below, the overlying mine overburden material is generally stronger than soils weathered from the Minturn Formation.
- Along the majority of the North 40 and McNulty OSF toes, the dip of the Minturn Formation is favorable. Specifically, the formation dips northeast, into the OSF facilities, in the opposite direction of the OSF outslopes. This means that any failure plane would cross through multiple beds of the formation, and could not preferentially fail along a single bedding plane or through an individual bed.

As previously reported in Section 2.3.2, Golder performed a CU triaxial test on an intact sample of clayey sand weathered from the Minturn Formation. The best-fit failure envelope for this test was found to be a standard Mohr-Coulomb envelope with a friction angle of 30.8 degrees, with a small magnitude of cohesion which was conservatively neglected.

For evaluation of the OSF stability under static conditions, Golder elected to use a standard Mohr-Coulomb failure envelope with a friction angle of 31 degrees and no cohesion. This value is considered conservative because the soil sample on which this strength is based is considered to represent “worst-case” strengths for the Minturn Formation materials encountered. The sample used in the CU test was a soil (completely weathered Minturn Formation material) with a high clay content relative to other Minturn-derived soils encountered during the field investigation. The field investigation suggests that beds with lower clay content and/or weathered to a lesser degree, which are expected to have higher strength, are common. Also, due to the dip of the bedding planes, critical slip surfaces must pass through multiple beds of the Minturn formation. Therefore, in the event that isolated, weaker beds exist, it is still highly unlikely that the average shear strength along the failure plane will be lower than the 31 degrees used in the analysis.

For the seismic stability analyses, strengths were reduced by 20% in accordance with the recommendations provided by Hynes-Griffin and Franklin (1984) for performing seismic analyses using the pseudo-static method. As a result, the strength of the Minturn during seismic events was modeled using a Mohr-Coulomb envelope with a friction angle of 24 degrees and no cohesion. Both the static and pseudo-static strength envelopes for the Minturn are shown in Figure 1.

2.4.2 Lincoln Porphyry

Similar to the Minturn Formation, Call & Nicholas (2007) presents a range of strength values derived from both back-analysis of historic slides within the pit and from laboratory testing. To summarize, the back-calculated strength for weathered porphyry along the west wall of the pit is reported as 27 degrees with 1.4 psi cohesion. The values developed by Call & Nicholas for use in the mine pit stability analysis vary, and range from 23.1 degrees with 21.9 psi cohesion for highly altered porphyry to 53.5 degrees with 1717 psi cohesion for deeper, less altered rock.

As discussed reported in Section 2.3.2, Golder performed a large scale direct shear test on a sample of clayey sand weathered from the porphyry. A standard Mohr-Coulomb failure envelope fit to the large scale direct shear test data provides a friction angle of 35 degrees with 18 psi cohesion. Based on Golder's evaluation of the laboratory test data and engineering judgment, a bi-linear Mohr-Coulomb envelope was determined to provide the best fit to the data and the best representation in-situ material behavior. The best-fit strength envelope can be described as follows:

- Friction angle of 50 degrees with no cohesion for vertical effective stresses below 50 psi
- Friction angle of 33 degrees with 29.2 psi cohesion for vertical effective stresses above 50 psi

Evidence shows that weathering of the porphyry generally decreases with depth (i.e., strength increases). As a result, the strengths provided by the direct shear test should be representative of the residual soils weathered from the porphyry near the surface, but are increasingly conservative for deeper potential failure planes. Also, because the porphyry is generally massive without preferentially oriented joint sets, there is little likelihood of failure through a continuous weak zone at depth. After weighing these factors, Golder elected to use the bi-linear Mohr-Coulomb envelope described above in the stability analysis.

For the seismic stability analyses, strengths were reduced by 20% in accordance with the recommendations provided by Hynes-Griffin and Franklin (1984) for performing seismic analyses using the pseudo-static method. As a result, the strength of the porphyry during seismic events was modeled using a bi-linear Mohr-Coulomb envelope described as follows:

- Friction angle of 44 degrees with no cohesion for vertical effective stresses below 50 psi

- Friction angle of 28 degrees with 23.3 psf cohesion for vertical effective stresses above 50 psi

Both the static and pseudo-static strength envelopes for the porphyry are shown in Figure 2.

2.4.3 Glacial Till

As discussed above, Golder performed a large scale direct shear test on a representative sample of glacial till obtained from the drilling program. A standard Mohr-Coulomb failure envelope fit to the large scale direct shear test data provides a friction angle of 31 degrees with 14 psi cohesion. However, similar to the porphyry, Golder believes that a bi-linear Mohr-Coulomb envelope provides both a more realistic representation of actual material behavior, and the best fit to the data. The bi-linear strength envelope can be described as follows:

- Friction angle of 44 degrees with no cohesion for vertical effective stresses below 50 psi
- Friction angle of 30 degrees with 18.7 psi cohesion for vertical effective stresses above 50 psi

Golder elected to use the bi-linear Mohr-Coulomb strength envelope for modeling the strength of the glacial till under static conditions. For the seismic stability analyses, strengths were reduced by 20% in accordance with the recommendations provided by Hynes-Griffin and Franklin (1984) for performing seismic analyses using the pseudo-static method. As a result, the strength of the glacial till during seismic events was modeled using a bi-linear Mohr-Coulomb envelope described as follows:

- Friction angle of 38 degrees with no cohesion for vertical effective stresses below 50 psi
- Friction angle of 25 degrees with 14.9 psi cohesion for vertical effective stresses above 50 psi

Both the static and pseudo-static strength envelopes for the glacial till are shown in Figure 3.

2.4.4 Mine Overburden

2.4.4.1 Theoretical Background

The distribution of the various-sized particles plays a significant role in determining the physical properties of the mine overburden materials. In general, the value of this friction angle will be a result of the following:

- Particle size distribution (increasing with increasing particle size);
- Particle shape (increasing with angularity);
- Strength and specific gravity of individual particles (increasing with degree of silicification); and
- Applied stress level (decreasing with increasing normal stress, resulting in a curvilinear envelope passing through the origin).

Research conducted by Fragaszy, et al. (1992) suggests that the strength of a soil with oversize particles may conservatively be characterized by the strength of the matrix material if the oversize particles are truly in a floating state. Conversely, the strength of the soil may be characterized by the properties of the oversize material if there is sufficient oversize particle to particle contact. Various researchers suggest that the shear strength properties of a soil having less than 40 percent oversize material are controlled largely by the soil matrix and that the strength properties of a soil with over 65 percent oversize material are controlled primarily by the properties of the oversize material. The strength properties of soils having between 40 and 65 percent oversize material are controlled by both the soil matrix and the oversize material. It has long been recognized (Holtz and Gibbs, 1956; Holtz, 1960) that an increase in the proportion of coarse material in an otherwise fine-grained granular soil results in an increased shear strength. Simons and Albertson (1960) present data that show, for instance, that the effect of scalping to allow laboratory testing may reduce the indicated angle of repose for the scalped material by 6 degrees compared with the field value for the full-sized material. Alternatively, when the voids in a coarse grained rock fill are filled with fines (i.e., a well graded material), the friction angle can be increased by as much as 10 degrees. The amount of granular fines required to have a significant beneficial effect on the shear strength of mine overburden is relatively small (Stratham, 1974). Leps (1970) presented friction angle data based on triaxial strength testing of large size (up to 200 mm) rockfill particles. This data suggests that the friction angle of durable compacted rock fill could be as high as 60 degrees at low normal stress levels and is likely to be at least 45 degrees at moderate stress levels. Given the lower densities of mine overburden materials, the above data suggests that more durable well graded mined overburden materials could be expected to have peak shear strengths on the order of 40 to 45 degrees at moderate stress levels.

Due to the granular nature of mine overburden materials, there should not be any cohesive strength at low stress levels (i.e., the strength envelope should pass through the origin of the normal stress vs. shear stress space). In some cases, attempting to fit a linear Mohr-Coulomb envelope to a given set of data will produce an apparent “cohesion”. It is generally understood that this “cohesion” is an artifact of attempting to fit a linear line to a curvilinear data set. As a result, mine overburden material strength is either modeled using a curvilinear envelope, the Mohr-Coulomb line is forced through the origin (resulting in a poorer fit to the data), or the best-fit Mohr-Coulomb line is used for the analysis with the understanding that the “cohesion” is an artifact of the curve-fitting process, realizing that the assumed strength will not be realistic at low effective stresses.

Curvilinear strength envelopes for mine overburden are commonly modeled in several ways. Leps (1970) presents three different curvilinear envelopes for high, medium, and low strength rockfill. Mine overburden can be modeled by assuming one of these envelopes, when appropriate. Alternately, the mine overburden strength can be modeled using a Hoek-Brown criteria (Hoek et al, 2002). The benefit of these first two methods is that they do not require site-specific laboratory testing, although knowledge of the source rock type, particle size, weathering characteristics, and engineering experience is necessary to utilize these methods appropriately. A third method is to perform laboratory shear testing and then best fit a strength envelope to the data (typically a power curve). The main drawback to laboratory testing is that it is not practicable to test a representative sample of mine overburden rock due to the extremely large particle sizes and the limited scale of most laboratory equipment. A common standard of care approach is to test a sample where oversize material has been screened off, with the understanding that the strengths indicated by the test are representative of the mine overburden matrix material only, and likely underestimate the field strength of the material by a significant degree.

2.4.4.2 Climax Mine Overburden Strength Development

Mine overburden material consists of blasted overburden rock. At the Climax mine, there are abundant fill areas where aged mine overburden is available for logging and sampling. The current mine overburden fills were produced using similar mining methods as are expected to be utilized for the future phases of OSF expansion at the mine, so the geotechnical properties of existing fills are expected to be representative of future fills.

Four test pits were excavated into mine overburden fills for the purpose of classification and sampling. Visual field estimates place the percent of oversize material between 15 and 40 percent. These test pits were excavated on the mine overburden fill top surfaces. The mine overburden fills are expected to be constructed using haulage truck placement methods. Placing mine overburden from a high face results in segregation of the material, with the coarsest and most durable rock preferentially being deposited near the toe of the slope. Therefore, the percentage of oversize material estimated in the test pit logs is expected to represent the lower end of values expected for the site.

As discussed above, two large scale direct shear tests were performed on samples of mine overburden collected from the site. The shear box was 12 inches by 12 inches, and as a result only the sampled material finer than 2 inches was used in the test. Assuming zero cohesion, the results indicate residual strengths of 35 to 36 degrees (linear Mohr-Coulomb). Figure 4 shows the results of the laboratory testing overlain with the Leps (1970) curves for low, average, and high strength rockfill. Also shown are linear Mohr-Coulomb and power curves which best fit the laboratory data. These two best-fit lines lie approximately midway between the Leps curves for low and average strength rockfill.

For the Climax mine overburden, the curvilinear power curve fit to the large scale direct shear test data was selected for use in stability modeling. This curve is considered representative of expected worst-case conditions within the OSFs for areas where overburden derived from igneous and/or metamorphic rock makes up the majority of the OSF fill. For the majority of the OSF this strength envelope is considered conservative, as the tests were performed only on the finer-grained matrix material, and was not corrected to account for the large amount of oversize material present in the OSFs. Note that approximately 30% of the overburden is expected to consist of sedimentary rock. The power curve described above is also considered representative for areas of the OSF containing average quantities of sedimentary rock derived overburden (i.e., approximately 30%). Golder considers the low strength Leps curve an appropriate “lower bound” strength envelope for the OSF. The low strength Leps envelope is suitable for evaluating the stability of the OSF in the event that a significant contiguous portion of the facility is constructed primarily from sedimentary overburden. The strength envelopes for the mine overburden are shown in Figure 4.

2.4.5 Tailing

Golder based the strength envelope for the tailing on the results of the series of 3 CU triaxial shear tests performed on undisturbed Shelby tube samples obtained during the 2011 field investigation. The tests were performed on samples of tailing fines. Although it is possible that the coarser beach tailing have a greater strength, Golder conservatively applied the same shear strength envelopes to both fine and beach tailing. To Golder’s knowledge, there is no other laboratory data specific to the historic tailing impoundments within the North 40 OSF footprint. A standard Mohr-Coulomb envelope provided the best fit to the Golder test data. Although the best-fit envelope shows a cohesive intercept, Golder conservatively neglected cohesion for the determination of the design shear strength. The resulting shear strength envelope is described by a friction angle of 33 degrees with no cohesion.

For the seismic stability analyses, Golder utilized the total stress strength parameters, also provided by the Golder laboratory tests. The tests show a total stress Mohr-Coulomb envelope characterized by a friction angle of 18 degrees and 12 psi cohesion. For the analysis, Golder conservatively neglected the cohesion, and simply used a friction angle of 18 degrees. Both the static and pseudo-static strength envelopes for the tailing are shown in Figure 5.

3.0 OSF STABILITY ANALYSES

The purpose of this analysis is to evaluate the global stability of the North 40 and McNulty OSFs. Calculations were performed to assess both the operational and post-closure OSF configurations under both static and seismic loading conditions. A liquefaction assessment was performed to verify that the historic tailing deposits will not liquefy (i.e., lose a significant amount of their strength) during earthquake events. In addition, a rockfall hazard evaluation was performed to evaluate potential hazards to Highway 91 resulting from rocks falling from the OSFs. The details of the stability evaluation and the liquefaction assessment are presented in Appendices C and D, respectively. The evaluations are summarized below.

3.1 Liquefaction Assessment

The objective of this analysis is to determine the potential for the historic tailing deposits to liquefy when subjected to the project maximum design earthquake (MDE) seismic event. The MDE has a peak bedrock acceleration of 0.14g and a reoccurrence interval of 1 in 2,475 years. Liquefaction is defined as a loss in strength due to a build-up of excess pore water pressure. The generation of this excess pore water pressure is most commonly attributed to cyclic undrained loading, such as that applied by an earthquake.

The design condition that was evaluated for liquefaction of the buried historic tailing impoundments is the existing condition, which is considered to represent the most critical condition for liquefaction to occur. This is because the potential for liquefaction of the historic tailing, where they are located within the footprint of the North 40 OSE, will decrease once the OSF is constructed as a result of the additional confining stresses and densification that will occur.

As a preliminary means of evaluating the nature of the historic tailing deposits three screening level methods that correlate index properties with liquefaction susceptibility were considered, including the Chinese Criteria (Wang 1979, Youd et al 2001), the Andrews and Martin Criteria (2000), and the Bray and Sancio Criteria (2006). These three screening methods provide criteria are that based on index properties including in-situ moisture content, grain size (notably fines content) of the materials, and Atterberg limits.

The Chinese screening criteria indicated that the tailing beach materials are potentially liquefiable and the tailing fines are likely not liquefiable. The Andrews and Martin screening criteria also indicated that the tailing beach materials are potentially liquefiable and that the tailing fines require more rigorous testing and analysis to determine their liquefaction potential. Finally, the Bray and Sancio screening criteria indicates that both the tailing beach and tailing fines may be potentially liquefiable.

Because these three simplified screening procedures did not eliminate the historic tailing impoundments from being classified as potentially liquefiable, a more rigorous liquefaction assessment was performed.

Two different approaches were used for the rigorous assessment. The critical stress ratio (CSR) predicted from the maximum design earthquake (MDE) was determined by the method proposed by Youd et al. (2001). After the determination of the CSR, two methods were used to estimate the cyclic resistance ratio (CRR) of the historic tailing deposits. Both methods are based on the in-situ state of the deposits, however, this in-situ state is measured by independent methods.

The first method utilizes the state parameter as recommended by Jefferies and Been (2006). The state parameter corresponding to no-liquefaction was determined from the three CU triaxial tests (see Section 2.3). The in-situ state of the soil was then developed by a continuous void ratio-effective stress relationship based on one-dimensional laboratory consolidation and by natural density results from Shelby tube samples obtained during the field investigation. Based on this state parameter analysis, the in-situ state of the soil indicates that shearing will cause dilation and thus strain hardening. Factors of safety calculated from this analysis range from 1.6 to 3.6.

The second rigorous method estimates the in-situ state of the tailing deposits through evaluation of SPT results. SPT values were recorded during the October-November 2011 field investigation and these values were correlated with CRR. The relationship between SPT blow counts and CRR was recommended by Idriss and Boulanger (2008) and is based on a database of SPT values recorded in locations subjected to earthquake loading where liquefaction has either occurred or not occurred. The blow counts recorded in the field were corrected for overburden stress, rod length, and hammer efficiency to obtain the $(N_1)_{60}$ blow count value. Next, these blow counts were corrected for fines content to determine the equivalent clean sand blow count values. Finally, CRR was determined by the Idriss and Boulanger (2008) relationship and this CRR was corrected for the earthquake magnitude and overburden stress. The factor of safety against liquefaction determined by this method ranges from 2.1 to 2.9. As a result, both rigorous liquefaction evaluation methods provide relatively high factor of safeties against liquefaction for the critical existing conditions. Additional stress confinement that will occur within the footprint limits of the North 40 OSF, once it is constructed, will further reduce the potential for liquefaction and any potential impacts to the North 40 OSF.

3.2 Global Stability

3.2.1 Method of Global Stability Analysis

Limit equilibrium stability analyses were performed with Rocscience's 2-D program, Slide 6.0. Factors of safety were computed based on Spencer's Method of Slices (Spencer 1967). The program uses various search algorithms to calculate factors of safety against failure for thousands of potential failure surfaces in order to find the most critical failure surface (or kinematic mechanism), and then computes the factor of safety for that surface. The program was used to evaluate both circular and non-circular (i.e., translational or block) failure surfaces. In addition, both deep and shallow failure surfaces were investigated. However, surficial veneer (infinite slope) slip surfaces were excluded from the results by constraining the failure surfaces to a minimum depth of 15 feet.

Earthquake (seismic) loading conditions were simulated using a pseudo-static approach. In an actual seismic event, the peak acceleration would be sustained for only a fraction of a second. Actual seismic time histories are characterized by multiple frequency attenuating motions. The accelerations produced by seismic events rapidly reverse motion and, generally, tend to build to a peak acceleration which quickly decays to lesser accelerations. Consequently, the duration during which a mass is actually subjected to a uni-directional, peak seismic acceleration is finite, rather than infinite. The pseudo-static analyses conservatively models seismic events as a force with constant acceleration and direction, i.e., an infinitely long seismic pulse. As a result, the standard of practice for geotechnical engineers is to take only a fraction of the predicted peak ground acceleration (PGA) when modeling seismic events using a pseudo-static analyses. A pseudo-static factor of safety of 1.0 is considered appropriate for water retention structures, when the structures are modeled using one-half the peak ground acceleration generated from the design earthquake (Hynes-Griffin and Franklin, 1984), with a strength reduction of 0.2 (80% of the strength parameters) applied to any potential strain softening materials. The Climax OSE earthquake loading conditions were evaluated consistent with the Hynes-Griffin and Franklin methodology (1984).

The minimum allowable factors of safety and the design seismic events are described in the project design criteria in Appendix F. The design earthquakes were developed using the 2008 National Seismic Hazard Maps developed by the USGS. The return intervals for the design earthquakes were selected based on standards of engineering practice for these types of facilities. These criteria are summarized below:

- Active Operations Criteria:
  - Minimum allowable static factor of safety is ≥1.4
  - Minimum allowable seismic (pseudo-static) factor of safety is ≥1.0
  - Operational basis earthquake (OBE) PGA is 0.06g (representing the 1-in-475-years event).
- Closure and Post-Closure Criteria:
  - Minimum allowable static factor of safety is ≥1.5
  - Minimum allowable seismic factor of safety is ≥1.0
  - Maximum design earthquake (MDE) PGA is 0.14g (representing the 1-in-2,475-years event)

The stability was evaluated with five cross-sections, the locations of which were selected to represent the most critical or worst case stability conditions (i.e., steepest foundations, highest stockpile locations, etc. These design sections are generally oriented perpendicular to the foundation and OSF slopes, in areas with steep grades and the greatest fill height. The cross-section locations are shown on Drawings 3 and 4 with cross-sections illustrated on Drawings 5 through 9.

3.2.2 Global Stability Analysis Assumptions

It is routine practice for mines to update the mine plan, and corresponding OSF loading plans, throughout the mine life cycle. These routine mine plan and OSF updates occur within the general framework established by the project design criteria. Therefore, the relevant parameters in the design criteria were used to construct the OSF outslopes for use in the stability design sections. Sections constructed in this manner are considered to represent the "worst-case" section geometry (i.e., steepest slopes) possible within the constraints of the design criteria. The current operational OSF grading plans were developed by Climax, and provided to Golder on February 27, 2012. Golder evaluated the plan, and found it to be consistent with the project design criteria (see Appendix F). The relevant design criteria are listed below:

- OSF toe limits used were defined following the procedure discussed in Section 7 (Closure Considerations)
- Operational scenario:
  - Inter-bench angle of repose slopes were modeled as 1.4H:1V (or 36 degrees)
  - Operational benches were modeled as 200 feet wide
  - The maximum height between benches was modeled as 200 feet
  - Overall operational slopes were thus approximately 2.4H:1V
- Closure scenario:
  - Inter-bench reclamation slopes were modeled as 2H:1V
  - Closure reclamation benches were modeled as 20 feet wide
  - The maximum height between benches was modeled as 56 feet (125 feet slope length)
  - Overall closure slopes were thus approximately 2.4H:1V

The distribution of various geologic materials was modeled based on the findings of the field investigation performed in October and November 2011. A geologic map of the site (USGS, 1971) was also used to support the interpretation of the geology between borings and test pits. The material strength parameters defined in Section 2.4 were used in the analysis.

Piezometric surfaces were modeled based on water levels measured in temporary piezometers installed in the 2011 borings. In areas without tailing deposits, the existing piezometric surface was measured an average of 14 feet below the native ground surface (i.e., 14 feet below the base of existing fills, or 14 feet below the present ground surface in areas with no fill). No perched water was encountered within any of the existing mine overburden fills. In areas with historic tailing deposits, the piezometric surface was located within the upper 10 feet of tailing deposits.

A sensitivity analysis was performed in order to evaluate the effect of varying piezometric levels on OSF stability. For the sensitivity analysis, a conservative, worst-case piezometric surface was assumed to exist at the top of native ground and at the surface of the historic tailing impoundments. Stability was evaluated along the two most critical cross-sections under static conditions.

3.2.3 Global Stability Analysis Results

Based on the analyses performed for this study, all computed factors of safety meet or exceed the factors of safety established by the Project Design Criteria, for both the maximum operational and closure slope scenarios. Factors of safety for the operational and post-closure OSF configurations are presented in Tables 5 and 6, respectively.

The sensitivity analysis showed that the factor of safety is relatively insensitive to changes in the phreatic surface. When conservative elevated phreatic levels were modeled at the base of the OSF, static factors of safety decreased by only 0.02 to 0.08 from the base case. The OSF underdrain system has been designed to prevent elevated phreatic levels at the base of the OSFs, primarily to minimize the potential to develop hydraulic head above the non-contact water underdrain system (see details 2 and 3 on Drawing 10). Therefore, while elevated phreatic levels at the base of the OSF are not anticipated to occur, increased piezometric levels beneath the OSFs are not predicted to create unstable conditions. As a result, installation of piezometers and regular monitoring of groundwater levels are not required as a component of the O&M plan.

3.3 Rockfall Hazard Evaluation

A rockfall evaluation was performed using the Colorado Rockfall Simulation Program (CRSP), Version 4.0. The purpose of the evaluation was to identify potential hazards to Highway 91 resulting from rocks falling from the OSFs. The rockfall run-out potential was evaluated along five (5) sections through the North 40 OSF and low grade ore stockpile area, where the distance between the OSF and Highway 91 is the least. At each cross-section location, CRSP was used to roll 1000 simulated 3-foot diameter rocks. The results show that none of the rocks will reach Highway 91.

4.0 WATER MANAGEMENT AND HYDROLOGIC DESIGN

4.1 Contact Water Collection System Conceptual Design

The purpose of the CWCS is to collect water that has contacted the McNulty OSF and reports to the toe. The CWCS components described below are conceptual in that they represent a general approach to collect and convey water at the OSFs. As the OSFs are constructed, final CWCS designs may be modified based on field conditions or if other construction materials are deemed to be more appropriate.

The CWCS piping system conceptual design consists of primary and secondary perforated corrugated polyethylene (PCPE) pipes placed within the McNulty Gulch drainages. At this time, it is envisioned that there would be two 18-inch diameter primary pipes, one within the main McNulty Gulch north fork, and one within the main McNulty Gulch south fork. Secondary CWCS collector pipes will be smaller (e.g., 10-inch diameter), and will be placed, if needed, in all side drainages reporting to the two main forks of the McNulty Gulch. CWCS Pipes will be protected with drain gravel or other suitable material, which will be wrapped with 12-oz/yd^2 non-woven geotextile or other suitable filter material.

At the toe limit of the OSF, berms would be constructed to direct flows from the PCPE pipes and drain rock into solid-wall high density polyethylene (HDPE) pipes. The solid pipe will exit the toe of the OSF and convey flows to the Climax contact water circuit via the ETDL and/or East Side Channel. The conceptual locations of the CWCS pipes are shown on Drawings 3 and 4, and conceptual level details of the system are shown on Drawing 10. As noted on the Drawings, the conceptual CWCS design will be advanced to a construction level following regulatory acceptance of the CWCS conceptual design approach.

Golder has designed the CWCS to collect and convey 100% of the 100-year, 24-hour design storm which, for design purposes, is conservatively assumed to infiltrate entirely into the OSF and report to the base. It is recognized that in actuality this will not occur as most of the precipitation that falls on the OSFs will runoff, evaporate, sublimate or be retained by the overburden. Also, once a reclamation cover is placed, there would be significantly less infiltration occurring. To provide for a conservative worst case scenario, the CWCS pipes were conservatively sized assuming the entire design storm will report to the CWCS pipes within a period of 48 hours. The CWCS pipes were also designed with a factor of safety of 2. The flow capacity of various pipe sizes was evaluated using Manning's equation. It should also be noted that the CWCS flow capacity conservatively neglects the very high permeability of the overburden that will occur at the base of the OSFs. This very high permeability is the result of the coarsest rocks being deposited at the base of the OSFs as material is placed from the crest. The detailed calculation for sizing the CWCS pipes is presented in Appendix G.

4.2 Underdrain Design

4.2.1 Description of Conceptual Underdrain System

The purpose of the underdrain system is to capture non-contact water entering McNulty Gulch through springs/shallow groundwater, and to convey those flows to the toe of the OSF while preventing contact with OSF material. Like the CWCS described previously, the underdrain components described below are conceptual in that they represent a general approach to collect and convey water at the OSFs. As the OSFs are constructed, final underdrain designs may be modified based on field conditions or if other construction materials are deemed to be more appropriate.

The underdrain system will conceptually consist of primary, secondary, and tertiary underdrains. At this time, it is envisioned that each underdrain will consist of PCPE pipes embedded in drainage rock and wrapped with 12-oz/yd^2 non-woven geotextile for protection. The dimensions, extent, and pipe sizes of the primary and secondary underdrains will be determined based on the results of baseline flow rate monitoring of springs, as further described in the following section. The drainage gravel will covered with a 1 foot thick layer of low permeability liner bedding fill, on top of which an 80-mil linear low density polyethylene (LLDPE) geomembrane will be installed. The geomembrane will extend approximately 20-feet on either side of primary and secondary underdrain centerlines (i.e., two roll widths will be fusion welded). Similarly, a full roll width (approximately 20 feet wide) will be installed over each tertiary underdrain. Geomembranes will be anchored with anchor trenches and capped with a layer of overliner fill, which is provided to protect the geomembrane from damage during mine overburden placement. Conceptual level underdrain details are provided on Drawing 10.

The primary and secondary underdrains will be constructed within the McNulty Gulch drainages. There will be two primary underdrains, one within the main McNulty Gulch north fork, and one within the main McNulty Gulch south fork. Secondary underdrains will be smaller, and will be placed, where needed, in secondary drainages reporting to the two main forks of the McNulty Gulch. Tertiary underdrains collect and convey flows from springs located outside of the drainages to the primary and secondary underdrains. It is also anticipated that tertiary underdrains will convey non-contact flows from presently unknown small springs that will be encountered during clearing and grubbing of the foundation soils (for future reclamation growth medium). Preliminary primary and secondary underdrain locations are shown on Drawings 3 and 4.

The two primary underdrains will terminate in concrete manholes. The purpose of these manholes is to capture flows from the underdrain systems and transfer the flows to solid wall HDPE pipes. The pipes carrying the underdrain flows will terminate in a third concrete manhole immediately below the toe of the McNulty OSF. The third manhole, which will also receive non-contact surface water flows from north of the OSF, will discharge to the East Interceptor or other clean water conveyance at closure. The concrete vaults will be constructed with overflow outlets located near the top of the manhole. In the event that upset conditions cause flows that exceed the capacity of the system, non-contact water will overflow into the CWCS. The primary non-contact underdrain pipelines also will be plumbed to allow bi-pass to the contact water circuit manhole if ever needed.

4.2.2 Underdrain System Sizing Calculations

The underdrain system will be designed with an appropriate factor of safety for the peak flows determined from baseline monitoring of surface water runoff in McNulty Gulch (per the design criteria included in Appendix F). It is anticipated that the final design will provide a safety factor of 5 for both the piping and granular drain components of the underdrain.

A baseline monitoring plan for obtaining flow data for sizing the primary and secondary underdrains is described in Section 6. Tertiary underdrains will be sized in accordance with the project design criteria based on spring flow rates observed in the field during construction.

4.3 Operational Surface Water Management

4.3.1 Overview of Conceptual Surface Water Management System

The primary component of the operational surface management system is a series of perimeter channels and/or water management berms designed to maintain separation of contact and non-contact stormwater occurring on and adjacent to the OSFs. In addition, four energy dissipaters and three concrete manholes are anticipated. The layout of the berms and/or channels, energy dissipaters, and concrete manholes is shown on Drawings 3 and 12, and is summarized below. Similar to the other water collection systems, the surface water management system described below is conceptual in that it represents an overall approach to collect and convey water at the OSFs. As the OSFs are constructed, final designs may be modified based on field conditions, updates to the mine plan, or if other construction materials are deemed to be more appropriate.

In this report, “energy dissipater” refers to a section of revetment (e.g., articulated concrete block (ACB) lined channel section with a flat, wide base designed to promote and contain a hydraulic jump. The purpose of the energy dissipater is to provide for a controlled transition of supercritical to subcritical flow at the base of the downdrains and other steep channel segments. “Manholes” are concrete vaults with vertical sides. Water can be transferred to the manhole, either as surface water flow entering from the top or from pipes discharging into the manhole. The advantages of concrete manholes are:

- Do not require a large area
- Effective at changing the flow direction of water, especially in a tight spaces
- Can be used to transfer water from surface channels to pipes

- Can be designed to capture water from multiple sources (i.e., underdrains and surface water channels)

Conceptual level details for energy dissipaters and manholes are shown on Drawings 11 and 12. As noted on the Drawings, it is anticipated that the conceptual surface water management details will be advanced to a final level of design once baseline flow data has been compiled and the conceptual water management strategy has been approved by DRMS.

The operational water management strategy for the North 40 OSF is to provide several perimeter channels and/or water management berms, each designed to collect contact water runoff from the OSF. Details of the operations perimeter channels and water management berm geometry are shown on Drawing 12. These features will be constructed when needed during OSF expansion at Climax. Drawings 13, 14, and 15 show the OSF plans at the end of years 5, 10, and 15 of operations, respectively. These drawings also show the proposed staged development of the operational water management systems.

An energy dissipater will is anticipated for the south side of the North 40 OSF, where water flowing east to west will undergo a sharp reduction in gradient. The water from this energy dissipater will be discharged into the camp drain system. A second energy dissipater may be located on the west side of the North 40 OSF, adjacent to the existing DSM interceptor, where the contact water flowing south along the OSF perimeter undergoes a sharp reduction in grade. The energy dissipater would discharge water to a concrete manhole. A second perimeter channel or water management berm, running south to north along the west side of the southern North 40 OSF and low grade stockpile area, will also discharge to the second concrete manhole. Water entering the concrete manhole will be transferred to a pipe, which will tie into the ETDL, East Side Channel, or other conveyance. Where feasible, the tops of OSFs also will be back-sloped to promote drainage off of the tops and into channels.

Note that water management berms and run-on collection channels are not proposed for the northeast side of the North 40 OSF. Water in this area will be managed using temporary channels and berms constructed along haul roads crossing through this area. Because haul road locations will change during the operational period, so will the location of the temporary water management structures.

The McNulty OSF will have two perimeter channels or water management berms. Both channels or berms will direct water to the low point at the base of the OSF, adjacent to the East Interceptor. Collected water will be directed into two energy dissipaters. Two additional concrete manholes will be used at the base of the McNulty OSF. One manhole will be used to capture contact water and outlet the flows to a pipe, which will then connect to the contact water circuit via the ETDL and/or East Side Channel. The second McNulty OSF manhole will capture non-contact water, and transfer the water to the East Interceptor. The non-contact manhole will also collect piped flows from the underdrain system.

4.3.2 Surface Water Design Methods and Assumptions

The details of the hydrologic and hydraulic calculations are provided in Appendix E, and are summarized below. Watersheds were delineated using the existing 2010 site topography and the current OSF grading plans provided by Climax. Additional hydrologic parameters (i.e., precipitation data, design methodology, SCS Curve Numbers (CN), etc.) are as specified in the project design criteria (Appendix F). The hydrologic analysis was completed using HEC-HMS modeling that incorporated SCS methodology to determine the peak discharges and runoff volumes generated by the design storm event. The design storms were defined based on climate data presented in Exhibit K of the AM-06 permit amendment. For operations, the design storm for use in designing temporary run-on/run-off channels was defined as the 10-year, 24-hour storm event (1.40 inches). For perimeter channels which will be utilized at closure in addition to during operations, the post-closure criteria were used. The post-closure channel design criteria consider both potential rainfall and potential snowmelt. The post-closure design storm was defined to be the more conservative (more severe) of either the 100-year, 24-hour storm event or the 10-year, 24-hour storm event superimposed with the estimated flow produced by the 100-year, 24-hour snowmelt event. Based on the available climate data, the 100-year, 24-hour storm event will produce a larger peak runoff, and will therefore be used as the basis for channel design. Based on data from the Climax weather station, the 100-year, 24-hour storm event is 1.99 inches.

The operations perimeter channels have been conceptually designed with 3H:1V side slopes. For constructability, and also to allow cleanout of sediment and snow, the base of the operations channels will be 5-ft wide. The channel will be 3.5 ft deep in order to convey the design storm while maintaining freeboard requirements. Alternately, water management berms have been designed with a height of 4 ft and 2.5H:1V slopes. Per the design criteria riprap or other revetment will be provided for permanent channels or other water conveyances (i.e., conveyances to be used post-closure, in addition to during operations) where velocities were calculated to exceed 5 ft/s during the design storm event. Temporary channels will be unlined, with Climax performing repairs as required. Riprap will primarily be used in perimeter channels and on the water management berms, while ACB will be used for the energy dissipaters required at major grade breaks between steeper and shallower segments.

4.4 Post-Closure Surface Water Management Strategy

4.4.1 Summary of Post-Closure Strategy

The post-closure surface water channels, water management berms, energy dissipaters, and manholes were designed using the same methodology used for the operational system. The details of the design calculations are provided in Appendix E. A summary of the post-closure water management strategy is provided below.

As discussed in Section 7, the outslopes of the OSFs will be regraded at closure from 200 feet high angle of repose interbench lifts to 2H:1V interbench slopes separated by 20 wide horizontal benches, that will provide a nominal 125 foot long slope length. Outslope channels will be constructed on each bench, which will collect sheet flow from the slope above it and convey the water to a downdrain. Closure and reclamation (C&R) of the OSFs will include a reclamation cover as described in Exhibit E to AM-06. Therefore, the post-closure flows are considered non-contact stormwater. Details showing typical bench channel and downdrain geometry are shown on Drawing 11.

A conceptual closure design illustrating the post-closure surface water channels and structures is shown on Drawing 4. At closure, the top surfaces of the OSFs will be backsloped and constructed or graded to promote runoff. Channels will then be constructed on the OSF top surfaces to prevent water from flowing over the crest and convey the non-contact runoff to downdrains that will flow to energy dissipaters at the base of the channels. Flows will generally exit the energy dissipaters into perimeter channels or along water management berms for conveyance to the East Interceptor or other conveyance for discharge from the site.

The North 40 OSF post-closure surface water management will conceptually include 5 downdrains. Each of the downdrains will have an energy dissipater at the toe, which will transfer water to non-contact perimeter channels or water management berms. One perimeter channel and/or berm will convey water around the southeast corner of the OSF, and then tie into the camp drain system (expected to be reclaimed for non-contact use). A second perimeter channel and/or berm will run south to north along the west side of the North 40 OSF and low grade ore stockpile area to the concrete manhole on the west side of the OSF, near the DSM interceptor. Where practicable, perimeter channels, berms, energy dissipaters, concrete manholes, and other water management structures constructed during operations will also be used during closure. The existing structures will be modified or upgraded, as necessary. Additional non-contact channels and/or berms will also be constructed where required.

As shown on Drawing 4, it is anticipated that the North 40 OSF closure strategy will require a non-contact water conveyance to transport non-contact flows north from the concrete manhole (near the DSM Interceptor) towards the East Interceptor or other conveyance. This function could be served by the reclaimed East Side Channel, a pipeline, or an additional channel constructed at closure.

There is a limited area on the north side of the North 40 OSF, between the OSF and the pit. During operations, run-off from this area will be managed with temporary channels and ditches constructed along haul roads. At closure, it is anticipated that run-off collection channels will collect and convey this water to the pit, where it will be managed with other pit inflows.

The conceptual closure plan for the McNulty OSF includes 5 downdrains, with four of the downdrains flowing to new energy dissipaters. The largest downdrain will flow into an energy dissipater at the base of McNulty Gulch constructed for operations, and then connect to the non-contact water manhole constructed for operations. The operations contact water manhole at the toe of McNulty OSF will be converted to a second non-contact water manhole to accommodate flow from the western perimeter channel or water management berm.

The operational perimeter channels and/or water management berms on the east and west sides of the gulch will be used to collect non-contact run-off and flows from the OSF, and may be upgraded if needed. Two of the downdrains will be constructed on the north side of the OSF. These downdrains will direct flows to energy dissipaters, and then to the perimeter channel or water management berm along the north side of the OSF, which will report to the energy dissipater at the toe of McNulty Gulch,

The remaining two downdrains flow to the toe of the OSF on the southeast side. After being collected in energy dissipaters, the water will flow to the southwest through a non-contact perimeter channel or along a water management berm. This perimeter channel or berm will connect to the North 40 OSF perimeter channel or berm, which continues around the southeast corner of the North 40 OSF before tying into the camp drain system.

4.4.2 Additional Toe Drain Design

The McNulty OSF CWCS will continue to operate at closure. As needed, additional CWCS toe drains will be constructed at select locations along the perimeter of the OSFs where there is potential for contact water to exit the toe of the OSFs as seepage, where operational perimeter channels or berms are no longer required. The additional toe drains are anticipated to consist of a perforated pipe placed in a drain rock filled trench oriented generally parallel to the OSF toe. As discussed above, there will be a number of non-contact surface water channels or water management berms along the perimeter of the OSFs during post-closure. The toe drain system will be designed to capture potential seepage flows before they can exit the OSF slopes and enter these conveyances. The additional toe drains will convey the collected water and transfer it to the ETDL or other conveyance for treatment. The flows captured by the toe drain system are anticipated to be small, as the majority of the contact water internal to the OSF will be captured and managed by the CWCS (see Section 4.1).

5.0 OSF OPERATION AND MONITORING

The investigations and analyses that have been conducted for the Climax OSFs demonstrate that after the final configuration for these facilities are completed, they will have adequate long-term stability. Given the magnitude of the size of the OSFs, it is not practical or realistic to evaluate all the potential intermediate development phases that will occur as the OSFs are developed. Rather, stability of the intermediate development stages will be managed by Climax based on the overall design criteria and an active monitoring program. As a result, Golder has developed an operation and monitoring (O&M) Plan to be used by Climax to support safe development of the OSFs during operations. It is anticipated that the O&M Plan will be a “living document” that is continually updated and improved upon to allow safe development of the OSFs to occur, if limited failures occur during early and intermediate stages of development.

The operation and monitoring plan is presented in Appendix H. The plan includes a discussion of performance and operational considerations, including:

- Construction on steep foundations
- Direction of OSF crest advance
- Selective placement of mine overburden based on material type
- Establishment of restricted access areas
- Water management and monitoring
- Winter Operations

It is well established that failures of mine OSFs are preceded by warning signals such as an increased rate of deformation, increased rate of cracking of the OSF platform, bulging of the OSF face, cracking and bulging at the OSF toe or increased rate of pore water pressure buildup in the OSF foundation. The OSF monitoring program has been developed to:

- Provide early warning of conditions that could lead to failure so that preventative measures can be taken;
- Provide early warning of impending failure so that personnel and equipment can be removed from the area at risk; and,
- Collect and assess data that will confirm or negate the assumptions made during the design studies and to provide data that will allow the design of the OSF to be modified during the life of the mine to improve the performance of the OSF.

A comprehensive OSF monitoring program consists of regular (each shift) visual inspection and of the OSF by the operating personnel, and periodic inspection and ongoing assessment of the accumulated data by the responsible mine engineer. Climax mine will institute a program for monitoring movements of the foundation downgradient of the OSFs, adjacent to the Highway 91. The operation and monitoring plan includes specific requirements regarding the types of monitoring, frequency of monitoring, reporting requirements, and steps to be taken in the event routine monitoring reveals failure warning signs. Monitoring requirements include

- Visual Inspections
- Crest displacement monitoring if indicated by the visual inspections
- Foundation displacement monitoring
- Engineer inspections

The O&M Plan also includes procedures for failure reporting and recommendations for back analysis and design parameter refinement, which can be used to improve monitoring procedures and update the OSF design to decrease the likelihood of future problems.

6.0 RECOMMENDATIONS TO FINALIZE OSF DESIGNS

The designs presented herein were developed using the available site and laboratory information, supplemented by the results of the 2011 Golder field and laboratory investigations performed in support of this report. Where site or laboratory data was not available, Golder used assumptions that are consistent with the current state of practice in the mining industry. It is anticipated that the Climax OSF designs will be finalized pending review by DRMS.

A conceptual design has been developed and design criteria have been established for the McNulty OSF underdrain system (Appendix F). However, data concerning the baseline runoff flows from the various drainages, springs and seeps has yet to be collected. There is no underdrain required for the North 40 OSF. Golder recommends monitoring the McNulty Gulch drainage network at several locations to provide data to appropriately size the underdrain system in each of the main drainages and sources of the watershed. Proposed monitoring locations are shown on Figure 6. Baseline flows should be collected monthly (when not covered with snow). Flows through the system are expected to be greatest during the spring snowmelt season, and it is most important to capture measurements of the peak flows during this time. Therefore, to the extent that access is available, flow measurements should be collected at a minimum bi-weekly frequency during the spring snowmelt. Measurements should be recorded for at least one full season prior to finalizing the underdrain design. Baseline measurements should include quarterly water quality indicators for the major springs and seeps to verify suitability to inclusion in the non-contact water circuit.

7.0 CLOSURE CONSIDERATIONS

Climax and Golder have developed closure design criteria (Appendix F), an OSF design that is compatible with the operational and post-closure OSF configurations, and a conceptual closure design, as shown in plan on Drawing 4.

As previously discussed, the operational OSF outslope will be constructed at an overall slope of 2.4H:1V by constructing a series of 200 ft high angle of repose lifts separated by 200 ft wide horizontal benches. At closure, the overall slope of the OSF outslopes will remain at 2.4H:1V. The interbench slopes will be regraded for compatibility with the closure configuration. Regrading of the angle of repose slopes will reduce maximum slope lengths to 125 feet (measured parallel to the slope), and reduce the interbench slope angle to a maximum of 2H:1V. Outslope drainage channels will be constructed in the reclamation benches. Regrading of the OSF outslopes will produce a more erosion resistant slope that will be stable in the long-term and facilitate cover soil placement, reclamation, and management of storm water falling on the OSF.

Dozers will perform cut-to-fill pushes to regrade the operational outslopes from angle of repose to a maximum of 2H:1V. This will result in an extension of the OSF outslope toe limits as material near the crest of the operational lifts is pushed down and placed as fill near the toe.

Golder has evaluated the proposed operational OSF toe for compatibility with closure. The evaluation included verifying that extension of the OSF outslope will not conflict with existing utilities, extend beyond property boundaries, extend into adjacent watersheds, or interfere with other features which may not be relocated at closure. Golder also verified that a sufficient offset will exist post-closure between the extended ultimate OSF closure slope and the critical features that have been identified (e.g., utilities, property boundaries, etc.), in order to allow sufficient space for perimeter channels, berms, and access roads. The results of this evaluation identified the following limitations to extension of the final closure slope that were considered in the development of the maximum operational OSF footprints:

- On the east side of the North 40 OSF, the OSF extents are limited by the truck shop, haul roads, utility corridor, and the open pit;
- The southern extents of the North 40 OSF are limited by several mine buildings and the camp drain system;
- On the west side of the North 40 OSF, the ETDL is the limiting feature;
- The north side of the North 40 is generally unlimited, and abuts directly with the southern sector of the McNulty OSF;
- The east and north sides of the McNulty OSF are limited by Climax property limits and by the hydrologic divide separating the McNulty Gulch and Clinton Creek drainages; and,
- On the west side of the McNulty OSF, the extents are ultimately limited by the ETDL.

The ultimate operational OSF grading plan is provided as Drawing 3 with the conceptual post-closure layout provided on Drawing 4. The ultimate OSF closure footprint covers more area than the operational OSF footprint, with the closure limits generally extending beyond the operational limits. However, the additional area covered by the post-closure OSF is minimized due to the compatibility between the overall operational and post-closure OSF slopes (i.e., both are 2.4H:1V).

Other aspects of the conceptual closure plan include placement of a reclamation cover on the OSF to facilitate revegetation, and continued management of surface water in order to maintain segregation of contact and non-contact flows. The objective of the reclamation cover will be to control erosion, reduce infiltration, prevent contact between stormwater and mine overburden, and promote revegetation. Revegetation of the OSF will follow the plan presented in Exhibit E of the AM-06 permit amendment.

Water management post-closure will be maintained through a network of surface water channels, water management berms, and other systems constructed during operations and after mining ceases. Where practicable, perimeter channels, berms, energy dissipaters, concrete manholes, and other water management structures constructed during operations will also be used during closure. The existing structures will be modified or upgraded, as necessary. Additional non-contact channels and/or berms will also be constructed where required.

The conceptual closure grading and water management plan illustrated on Drawing 4 provides for a “barber-pole” channel layout, e.g., top-surface and outslope channels that will convey runoff to steeper downdrains and then to perimeter channels or water management berms. While stormwater falling on the OSF during operations is considered contact water, stormwater falling on the OSF post-closure will remain non-contact stormwater due to the reclamation cover. As shown on Drawing 4 and discussed above in Section 4.3, the post-closure surface water network will include:

- Top surface channels will be constructed to collect and convey storm water runoff from the top surfaces of the OSF and direct it to downdrains.
- Channels will be constructed in the 20 ft wide benches on the regraded OSF outslopes to convey runoff from the reclaimed interbench slopes to the downdrains.
- Downdrain channels will collect water from the top surface and outslope bench channels and transmit the water down the OSF outslopes to energy dissipaters, located at the toe of the OSF, and then on to perimeter channels.
- Perimeter channels and/or water management berms will collect water from downdrains and convey the water along the toe of the OSF to the East Interceptor or other conveyance, where the non-contact water will be discharged.
- Run-on diversion channels/berms are anticipated to be constructed on the east sides of the North 40 and McNulty OSFs, as shown on Drawing 4.
- A toe drain network would be constructed along the toe of the OSFs in areas where water internal to the OSFs has potential to exit the OSFs as seepage near the toe of the slope.

The toe drain network will collect this contact water and transmit it to the ETDL or other contact water conveyance.

- The underdrain system will remain in service at closure to convey flows from springs within the McNulty OSF footprint, and transmit them to the toe of the slope while maintaining separation between the collected water and the overlying OSF material.

8.0 CLOSING

The analyses, conclusions, and recommendations presented in this report were prepared in accordance with the generally accepted standard of practice and standard of care for professional geotechnical engineering principles and practices at the time this report was prepared.

This report was prepared for the exclusive use of Climax for evaluating potential OSF designs and for supporting permit documents. The data and report may be provided to appropriate government agencies and/or prospective contractors for their information; however, our report, conclusions, and interpretations should not be construed as a warranty of actual subsurface conditions.

Golder appreciates the opportunity to provide support for the McNulty and North 40 OSF project. If you have questions regarding the information contained herein, please contact us at (303) 980-0540.

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