Yeager Airport RSS Failure Case History

Ryan Berg, P.E., D.GE., F. ASCE
Ryan R Berg & Associates
ryanberg@att.net

Jim Collin, Ph.D., P.E., D.GE., F. ASCE
The Collin Group, Ltd.
jim@thecollingroup.com

Introduction

- Purpose – Share lessons learned from the forensic evaluation of the largest RSS failure in North America

- The forensic analysis presented in the webinar was performed by The Collin Group and involved an extensive field exploration, and laboratory testing program to supplement the detailed engineering analysis – the entire process took over two years.

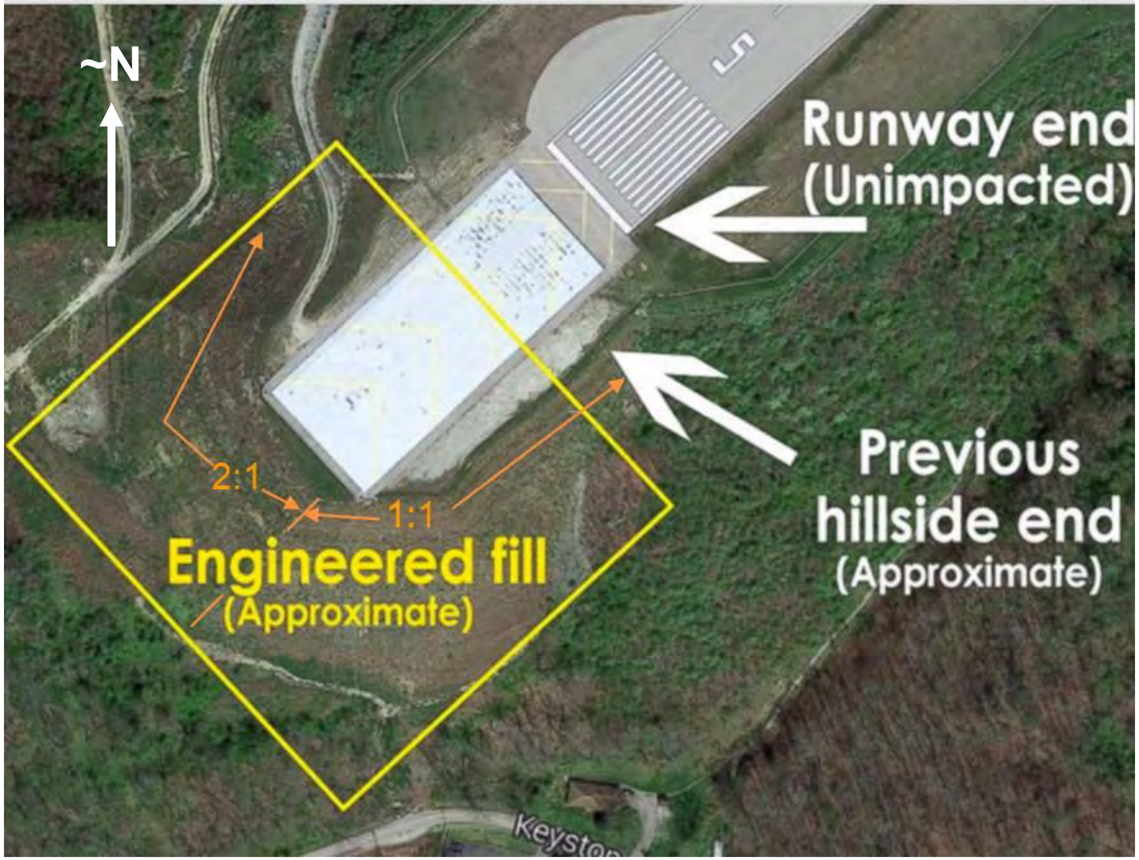
- Our client was the West Virginia Regional Airport Authority

Yeager Airport Background Info

- Original Construction - 1944
  - Excavating 7 hilltops (9,100,000 yd^3) to create a level area for the airport
  - Up to 130 foot cuts and 210 foot high fills were required

Yeager Airport Background Info (cont.)

- 2005 New FAA Regulations required Runway 5 extension for an emergency arrest system
- Runway extension was achieved by utilizing both a reinforced (1H:1V) and unreinforced (2H:1V) soil slope

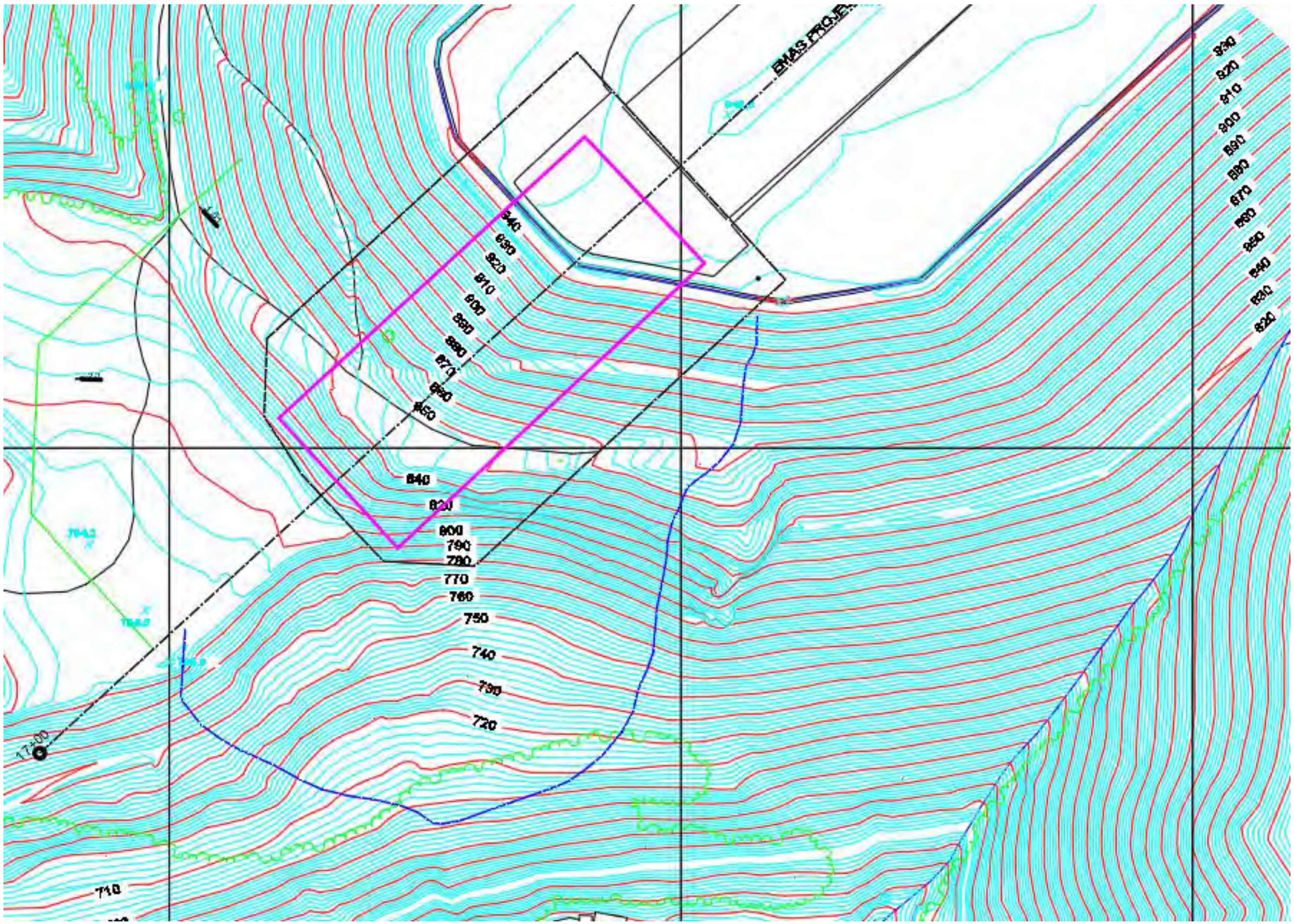


EMAS – Engineered Mass Arresting System

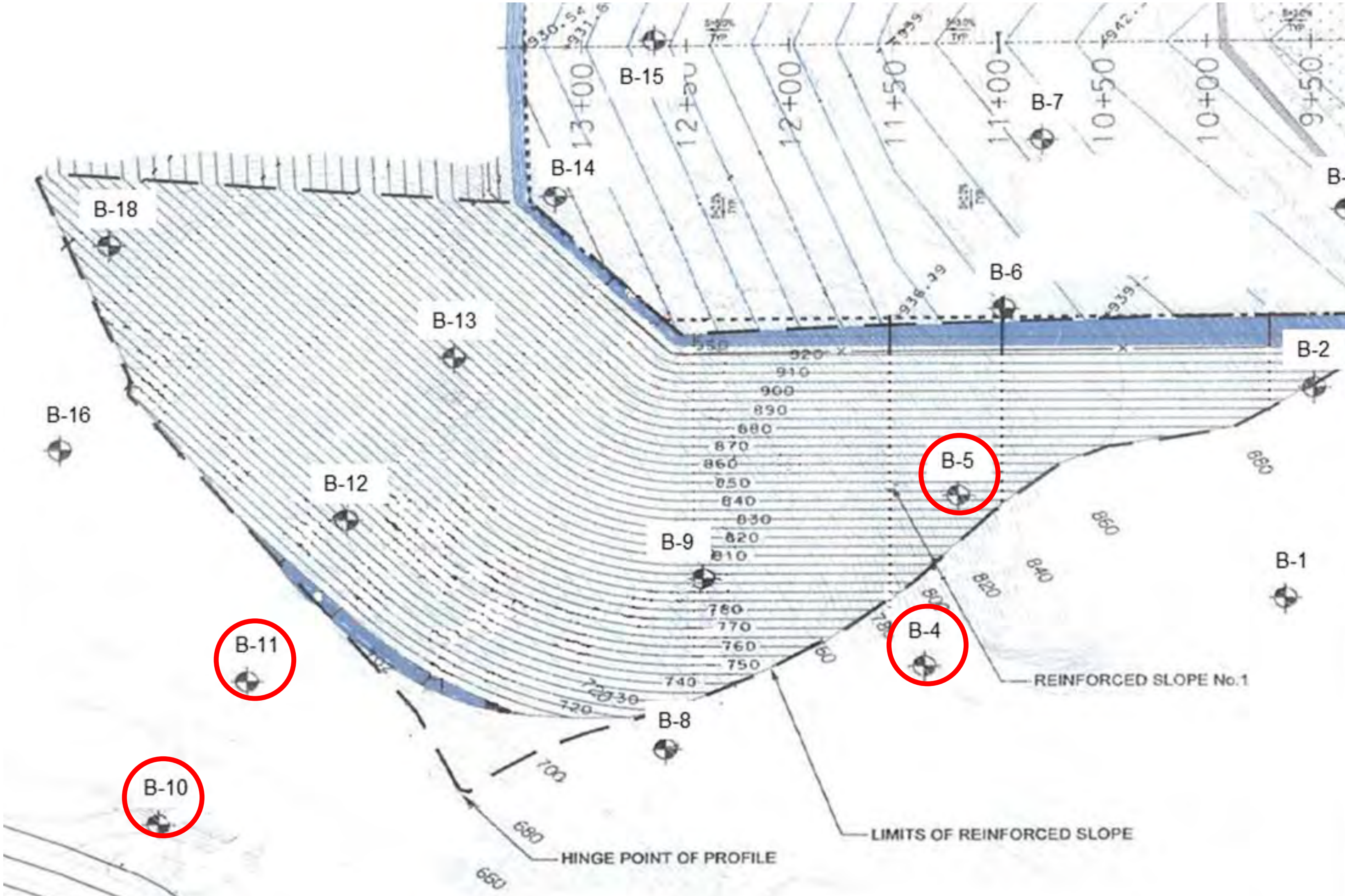
Yeager Airport Background Info (cont.)

January 29, 2010

Existing Topography Prior to RSS Construction



RSS Design – Subsurface Investigation Pre-Design



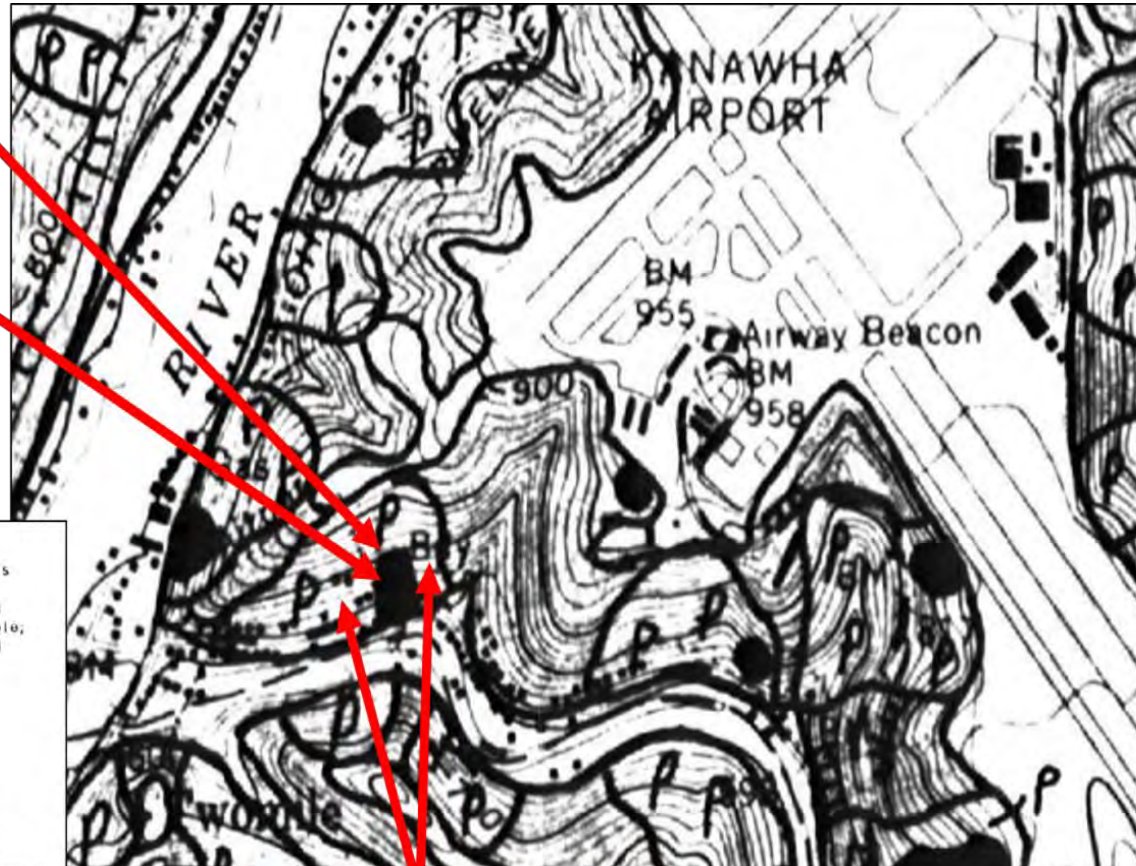
Subsurface Investigation Pre-Design Issues

- No pre-design Geotechnical Investigation Report was prepared for the project.
- No borings in slope below proposed toe of RSS
- Groundwater found in three borings – warranted further investigation or inclusion of drainage in design of RSS
- Shale (fissile) identified in several borings – ranging in strength from very soft, to soft, and to medium hard was not investigated with regards to stability
- Coal seams identified – but not investigated with regards to extent, effect on drainage, etc.

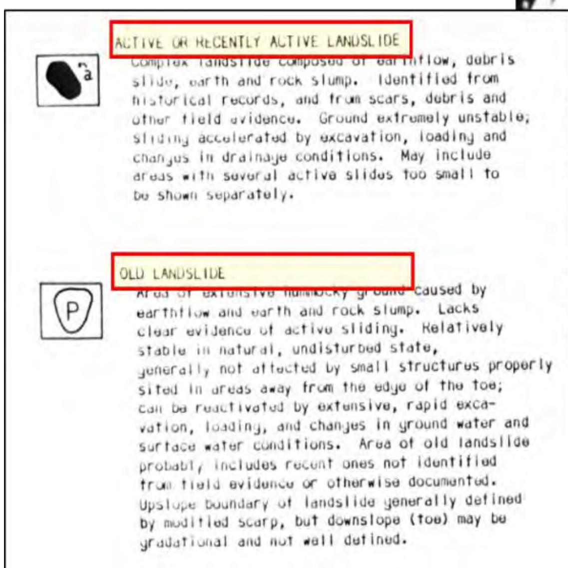
USGS Landslide Map

YEAGER RSS FAILURE, 2015

ACTIVE OR RECENTLY ACTIVE LANDSLIDE, 1983



OLD LANDSLIDE, 1983



ACTIVE OR RECENTLY ACTIVE LANDSLIDE

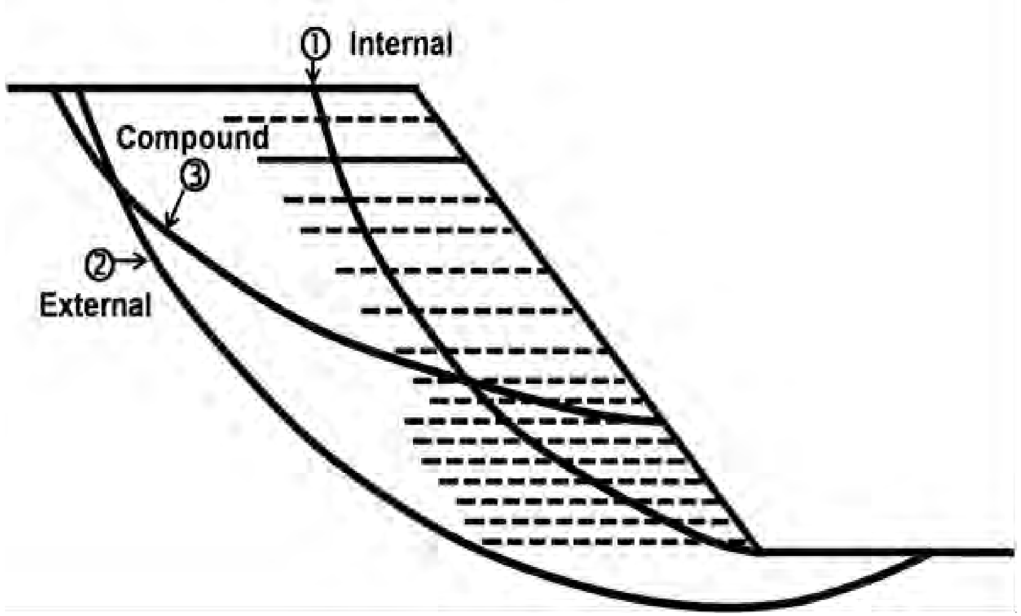
Complex landslide composed of earthflow, debris slide, earth and rock slump. Identified from historical records, and from scars, debris and other field evidence. Ground extremely unstable; sliding accelerated by excavation, loading and changes in drainage conditions. May include areas with several active slides too small to be shown separately.

OLD LANDSLIDE

Area of extensive hummocky ground caused by earthflow and earth and rock slump. Lacks clear evidence of active sliding. Relatively stable in natural, undisturbed state, generally not affected by small structures properly sited in areas away from the edge of the toe; can be reactivated by extensive, rapid excavation, loading, and changes in ground water and surface water conditions. Area of old landslide probably includes recent ones not identified from field evidence or otherwise documented. Upslope boundary of landslide generally defined by modified scarp, but downslope (toe) may be gradational and not well defined.

RSS Design

- An RSS design report was not prepared by the EOR of the RSS design
- Design minimum factor of safety for global and compound stability was set by EOR = 1.3



Industry Standard Recommended FS

Recommended Minimum Values of Factor of Safety
(Table 13.1 of Duncan and Wright, 2005).

Cost and consequences of slope failure	Uncertainty of analysis conditions	
	Small[a]	Large[b]
Cost of repair comparable to incremental cost to construct more conservatively designed slope	1.25	1.5
Cost of repair much greater than incremental cost to construct more conservatively designed slope	1.5	2.0 or greater

[a]The uncertainty regarding analysis conditions is smallest when the geologic setting is well understood, the soil conditions are uniform, and thorough investigations provide a consistent complete, and logical picture of conditions at the site.

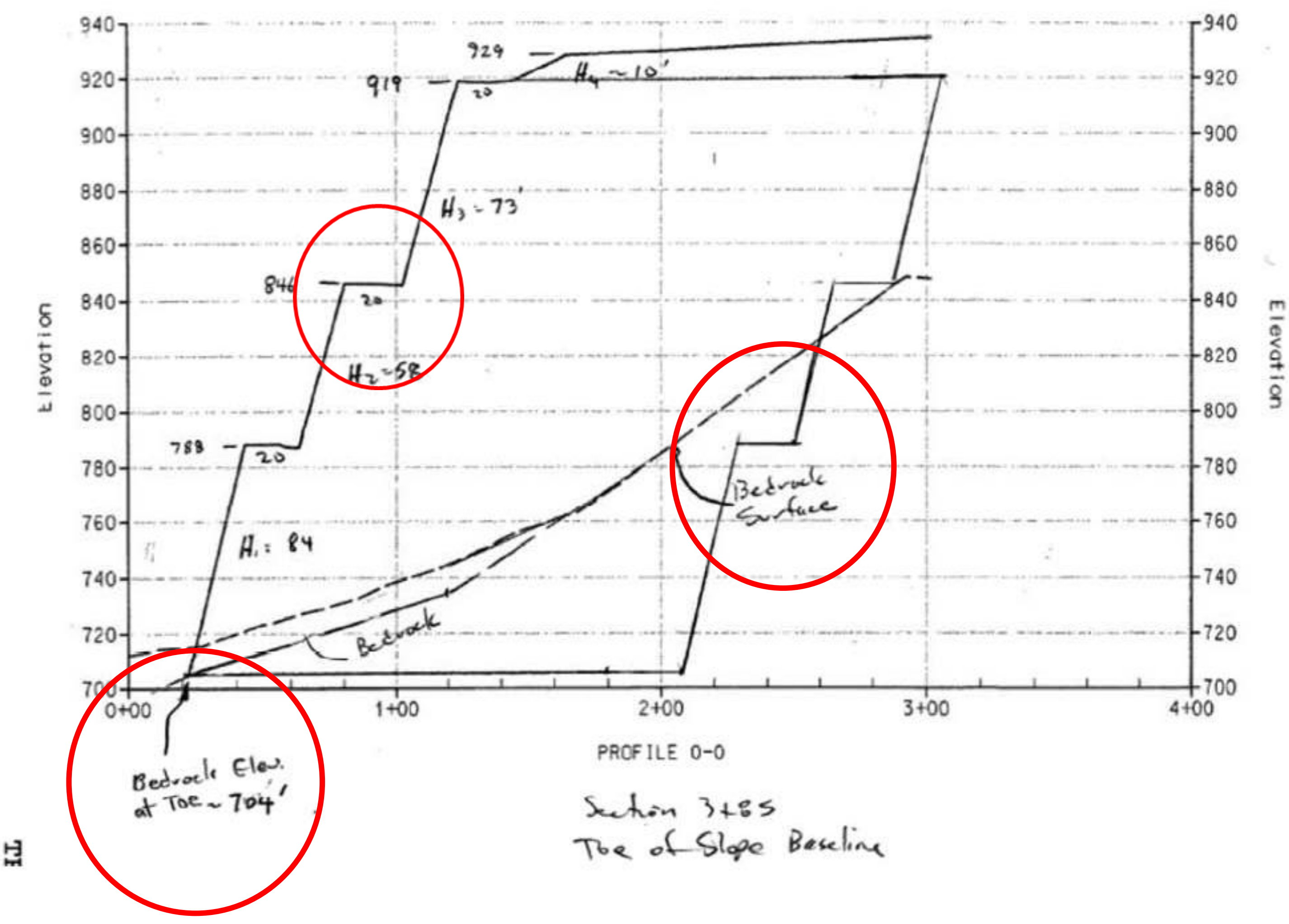
[b]The uncertainty regarding analysis conditions is largest when the geologic setting is complex and poorly understood, soil conditions vary sharply from one location to another, and investigations do not provide a consistent and reliable picture of conditions at the site.

RSS Design – Material Properties

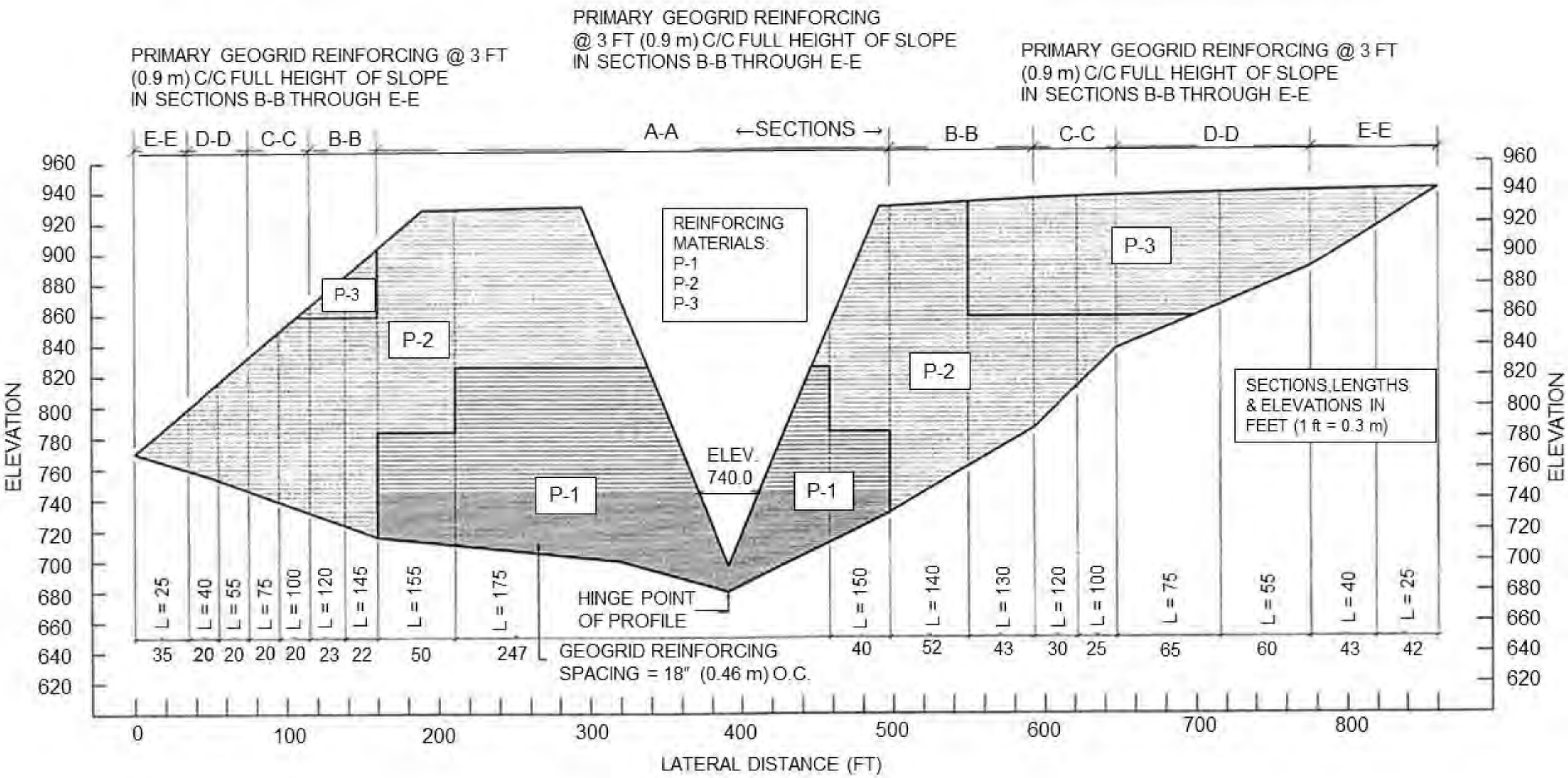
EOR Design Soil Parameters			
Soil Layer	Unit Weight (pcf)	Effective Friction Angle °	Cohesion
Reinforced Fill	115	36	0
Retained Fill	115/140	36/40	0
Foundation	145*/140	40	0

EOR Geosynthetic Reinforcement Design Properties								
Geogrid	T_{ult} (plf)	RF_{ID}	RF_{D}	RF_{CR}	C_{ds}	C_{i}	α	T_{LTDS} (plf)
P-1	13,250	1.2	1.1	2.60	0.8	0.8	0.8	3,861
P-2	12,785	1.2	1.1	2.60	0.8	0.8	0.8	3,725
P-3	10,195	1.2	1.1	2.60	0.8	0.8	0.8	2,970

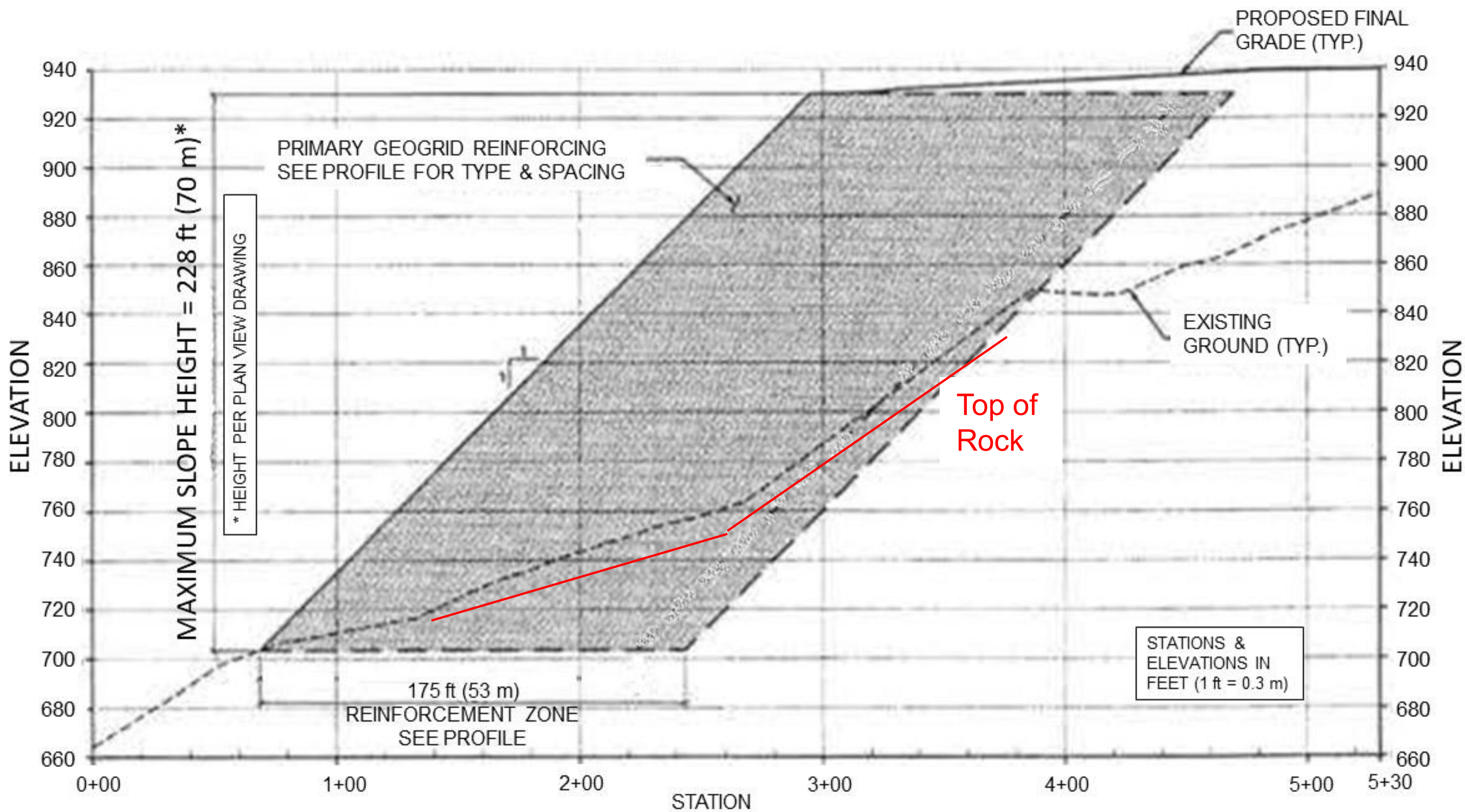
EOR Conceptual Design Cross-Section



EOR Design Elevation View



EOR Design Cross-Section



RSS Construction Processed Rock Fill

RSS Construction Fill Placement Below RSS

RSS Construction

RSS Construction Geogrid Placement

RSS Construction Geogrid Placement

RSS Construction Haul Ramp

RSS Construction

RSS Construction

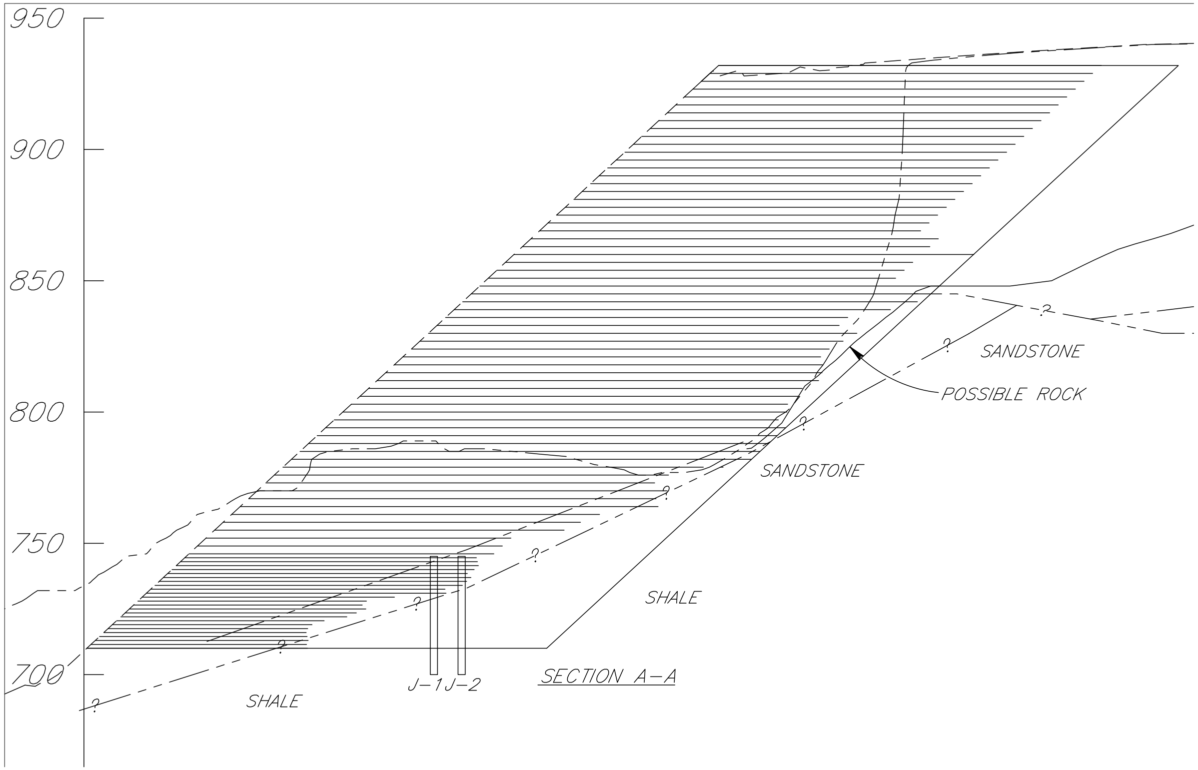
RSS Construction Completed

EMAS System In Place

RSS Construction Completed

As-Built RSS

- Geogrid lengths shortened to 75 ft in bottom of RSS
- Toe of RSS raised 10 feet
- Rock fill placed below the toe of RSS
- Geogrid Reinforcement alternate geogrid approved by EOR



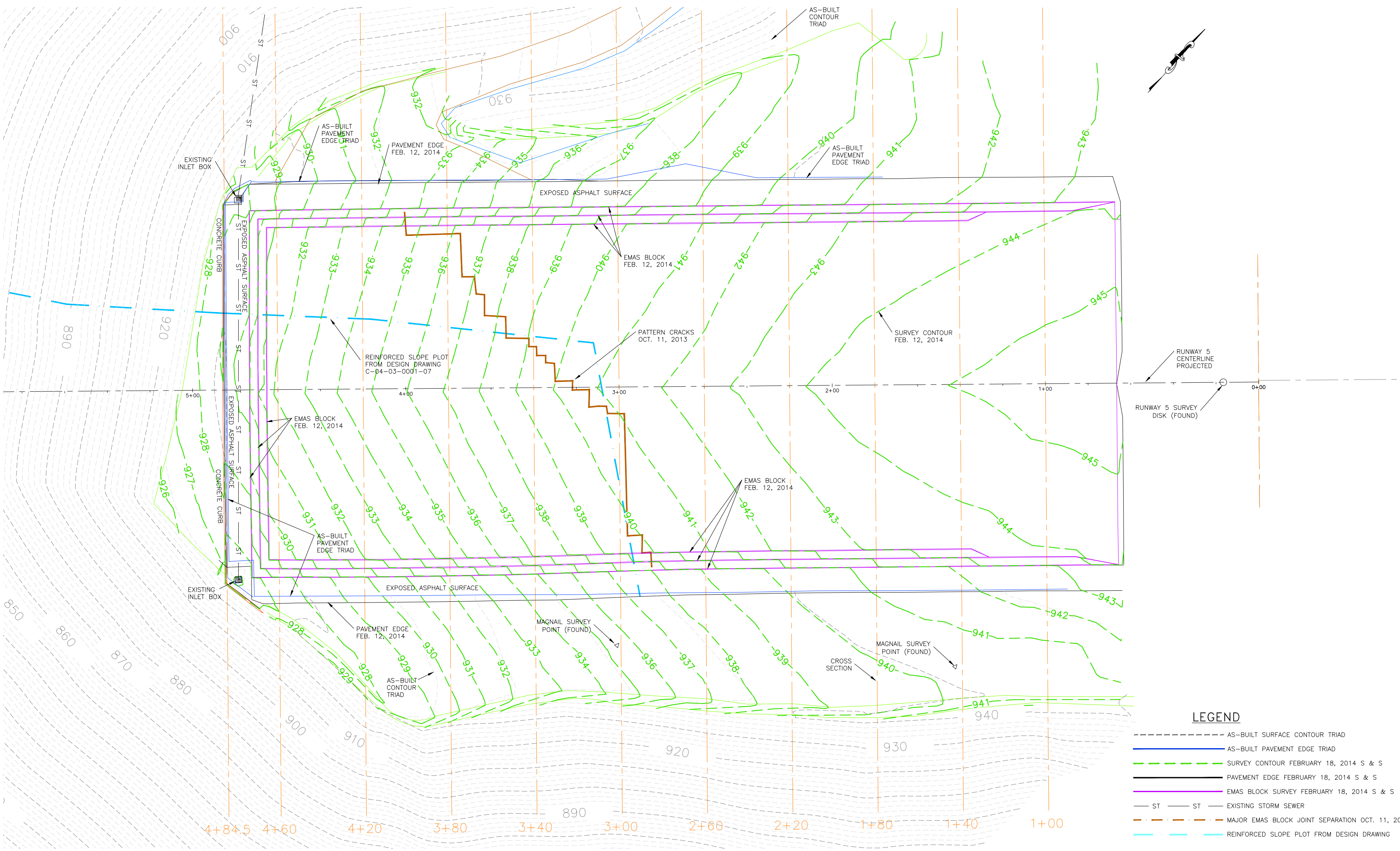
Substitute Geogrid

Geogrid Material Properties							
Geogrid	T_{ult} (plf)	RF_{CR}	RF_{CR}	RF_{D}	RF_{ID}	RF_{ID}	T_a (plf)
A	9,950	1.67	1.9	1.15	1.11	1.3	3,502
B	12,870	1.72	1.9	1.15	1.05	1.3	4,530

Timeline

- August 2005 - RSS Construction started
- December 2006 - RSS Construction Completed
- 2010 through 2014 - Shallow slides at base of RSS
- July 2013 - First cracks in EMAS noted
- January 2015 – Settlement of EMAS observed
- March 12, 2015 - Catastrophic failure

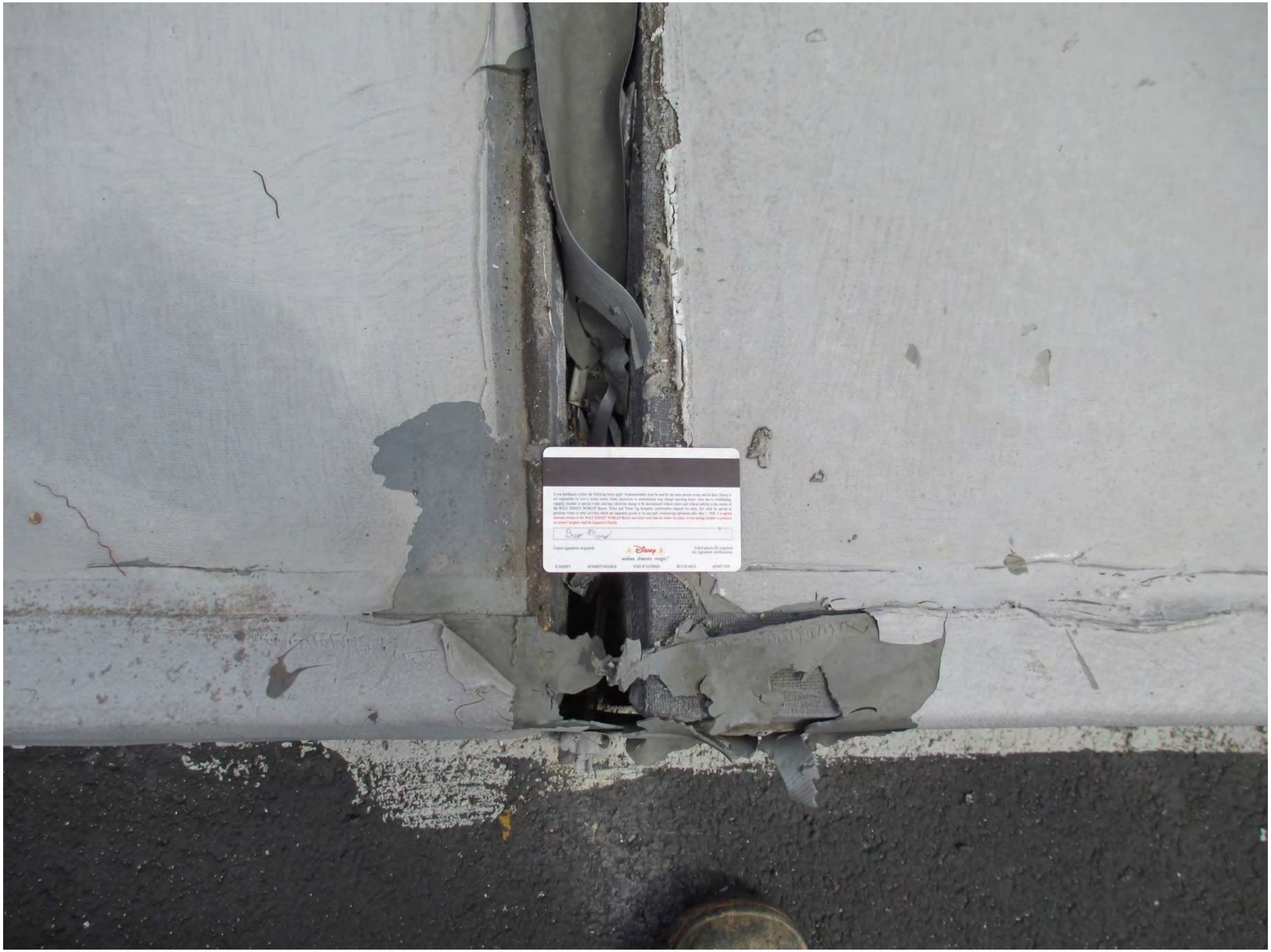
Post Construction Deformation - 2013



Post Construction Surficial Slide at Toe of RSS – May 2014

Post Construction Deformation - Sept 2014

Post Construction Deformation - Sept 2014

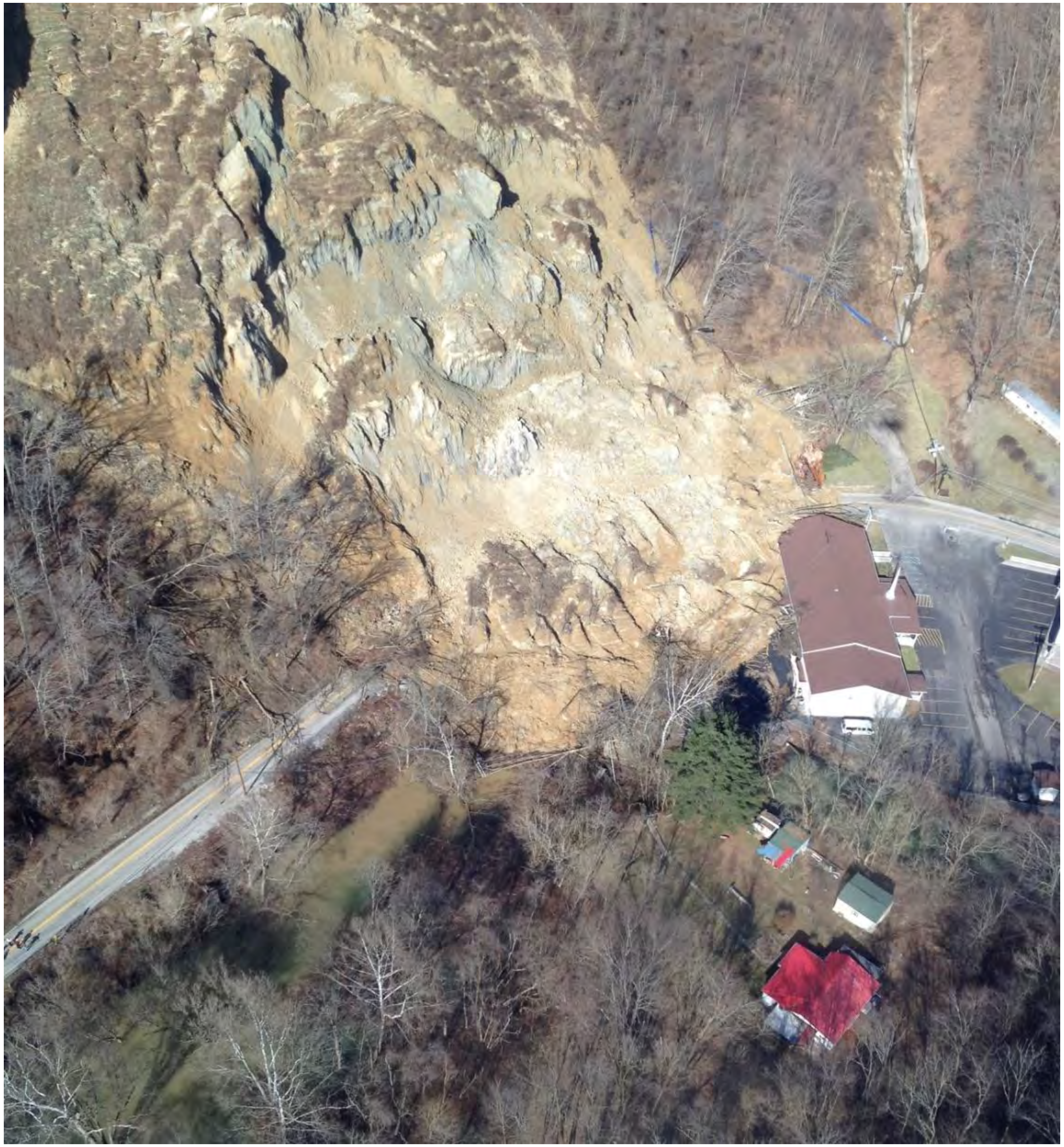


March 2015

March 2015 Prior to Catostrophic Collapse – Ruptured Geogrid

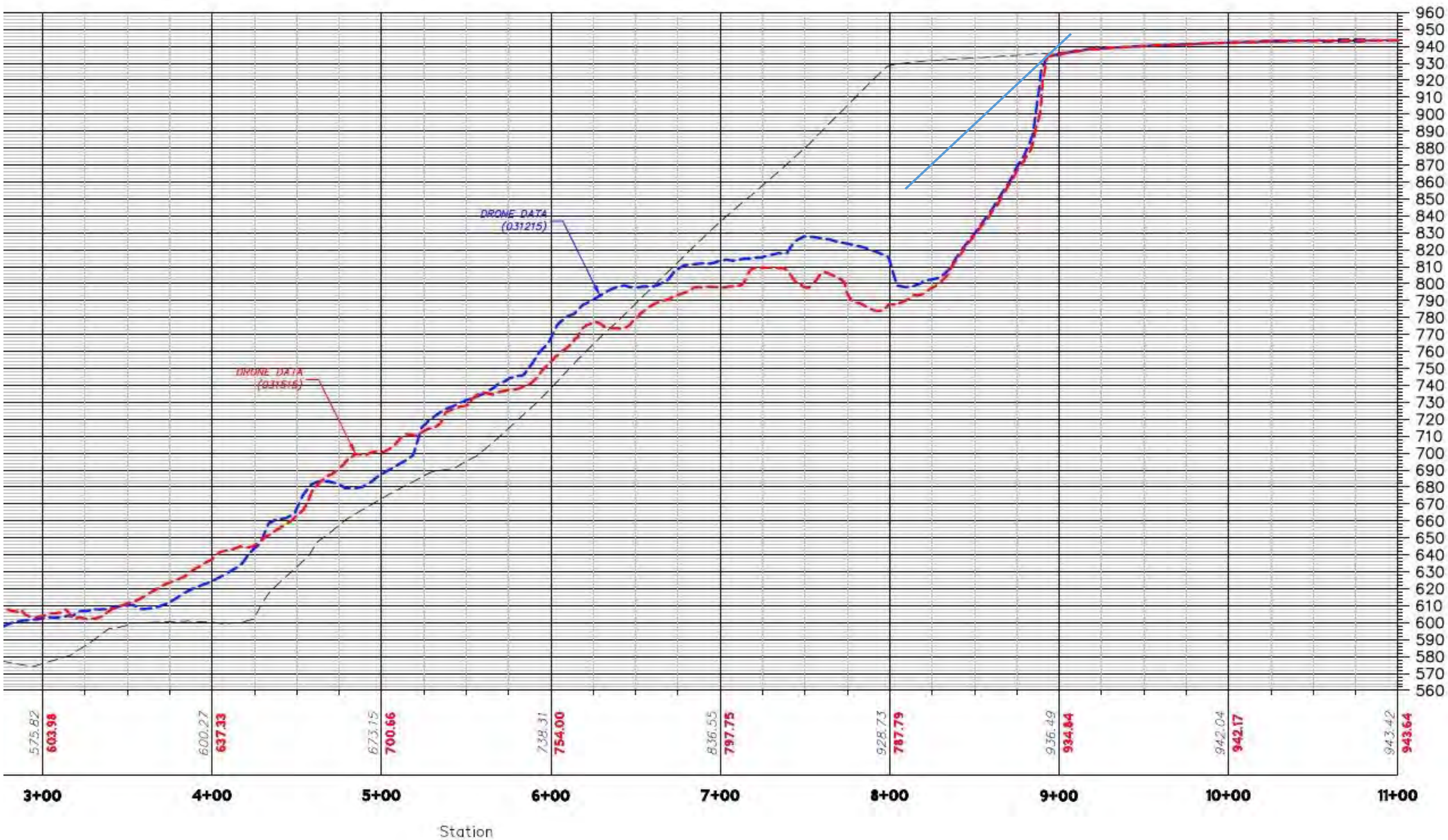
Failure March 2015

Failure Closed Keystone Drive Below RSS



Failure March 2015

Cross Section Showing Head Scarp and Failure Mass

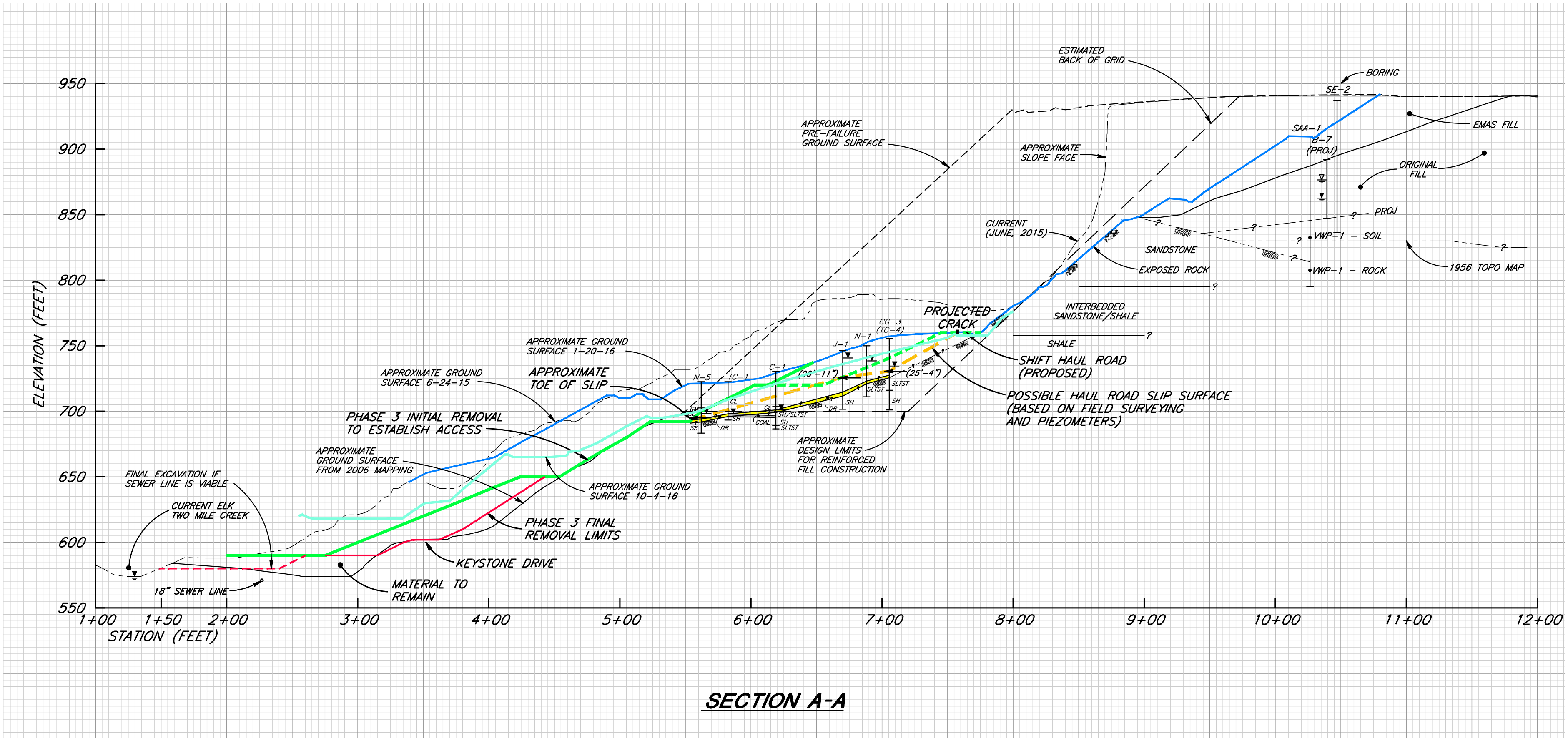


Head Scarp – Rupture of Over 30 Layers of Geogrid

Failure Mass

Failure Mass

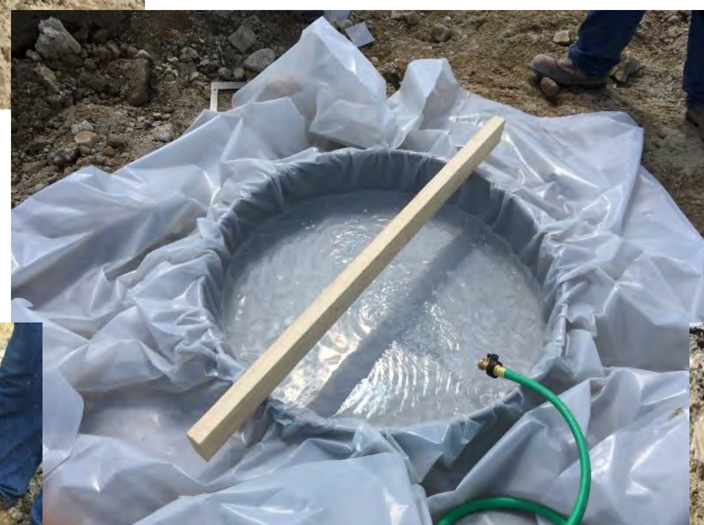
Post Failure Stabilization



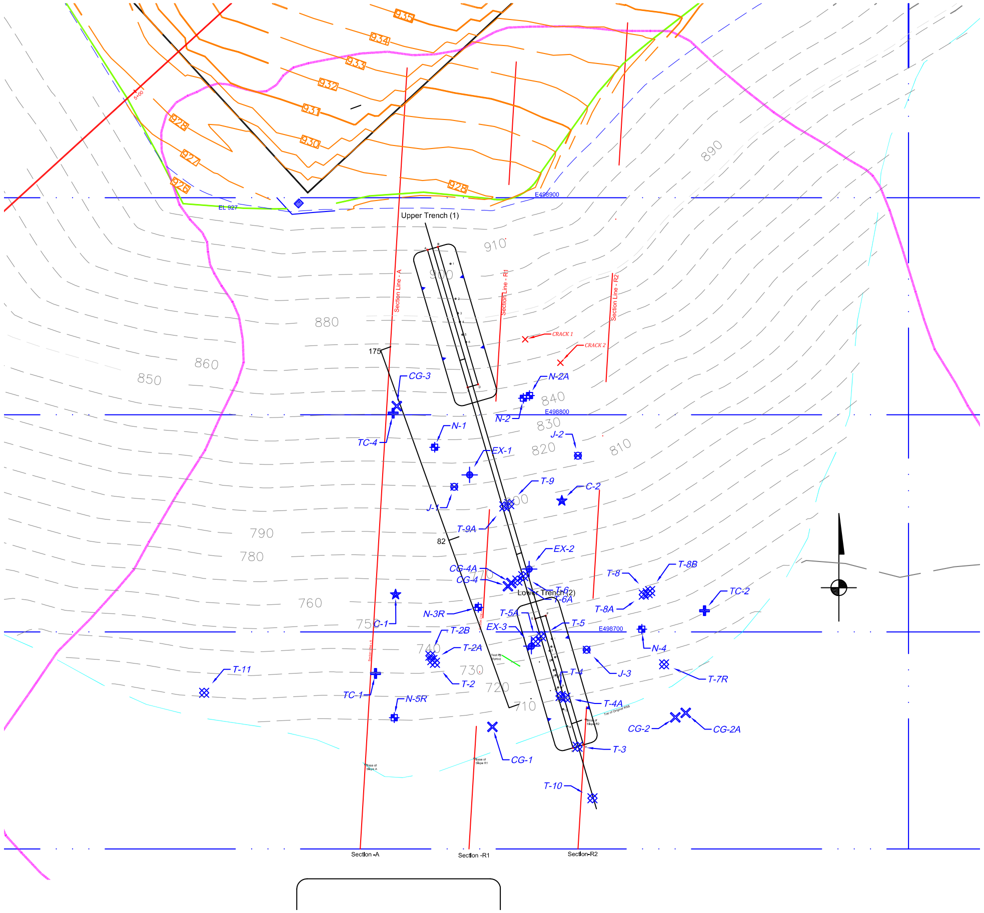
Post Failure Forensic Investigation

NDT – 340 tests – 122.2 pcf
SC – 71 tests – 121.7 pcf
WT – 32 tests – 123.8 pcf

Weighted Ave – 122pcf at 9.3% moisture



Post Failure Forensics



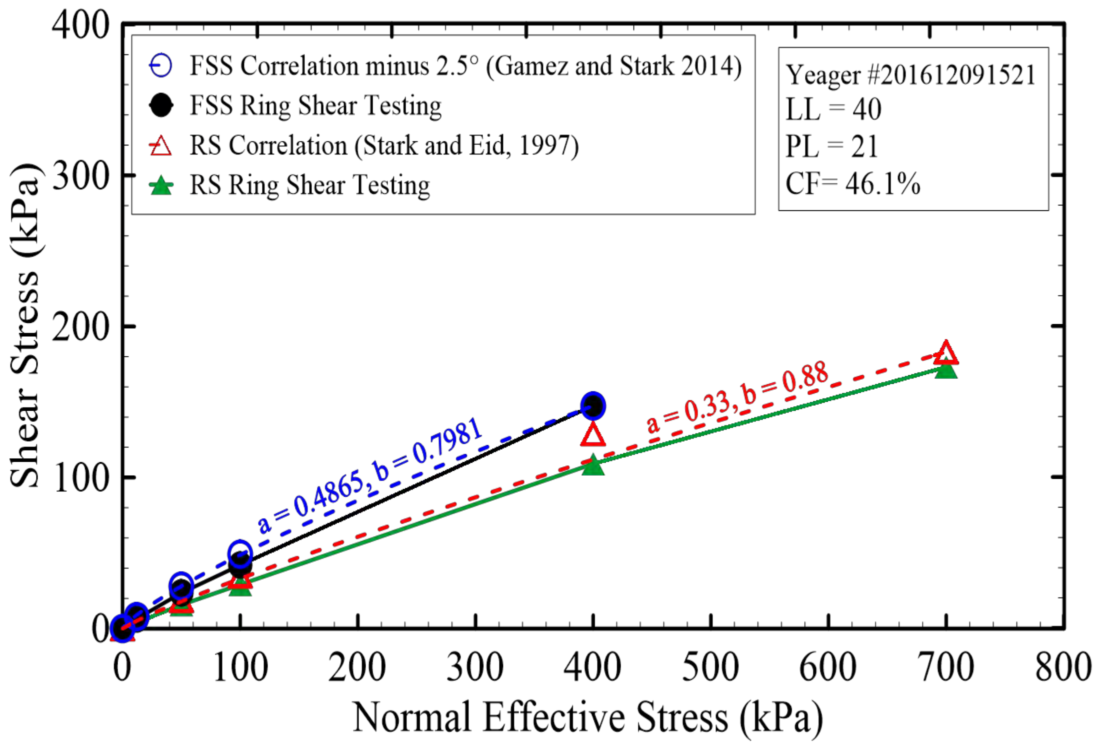
Failure Surface Identified

Soil-Rock Interface Fully Softened and Residual Strength

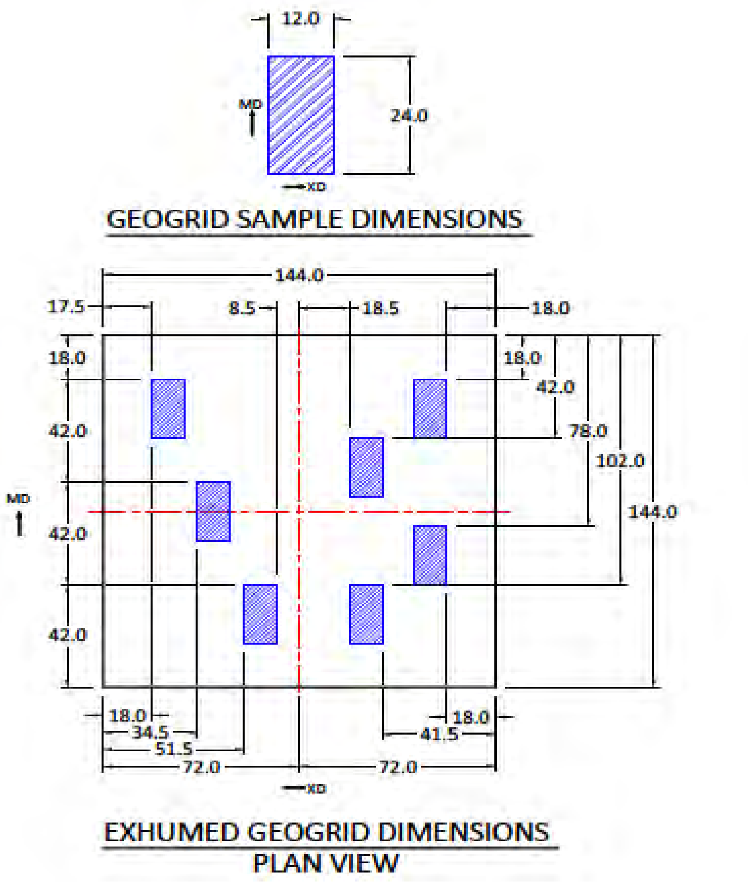
FSS strength 19.7° – 25.8°

RSS 14.3° – 20.2°

Normal Stress Range 50 to 400 *kN/m*^{2} (1050 – 8350 psf)



Exhumed Geogrid



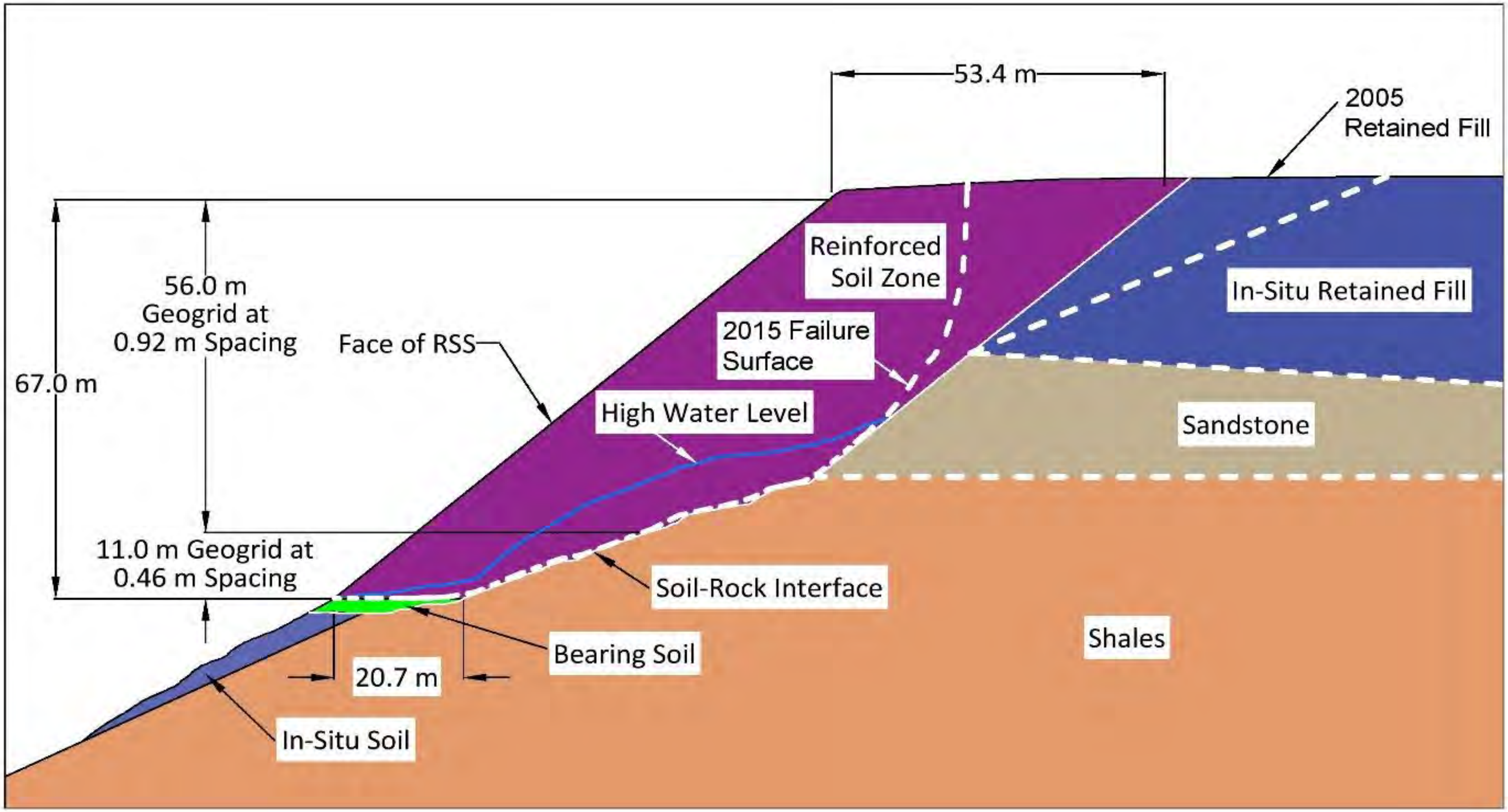
Geogrid Testing Results

Geogrid Wide Width and Single Rib Test Results			
Geogrid Type	Wide Width strength (lbs/ft)	Single Rib Strength (lbs/ft)	Strength Used in Analysis (lbs/ft)
A	7,511	9,165	9,000
B	9,037	9,848	10,000

Soil Properties

Slope Material	Moist Unit Weight γ_{moist} (pcf)	Effective Stress Friction Angle ϕ' (deg)	Effective Stress Cohesion (c')
Reinforced Soil Zone	135	36^{o}	0
Retained Soil Zone	135	36^{o}	0
Bearing Soil Zone	135	36^{o}	0
Soil/Rock Interface	Shear Function		0

Forensic Cross-Section

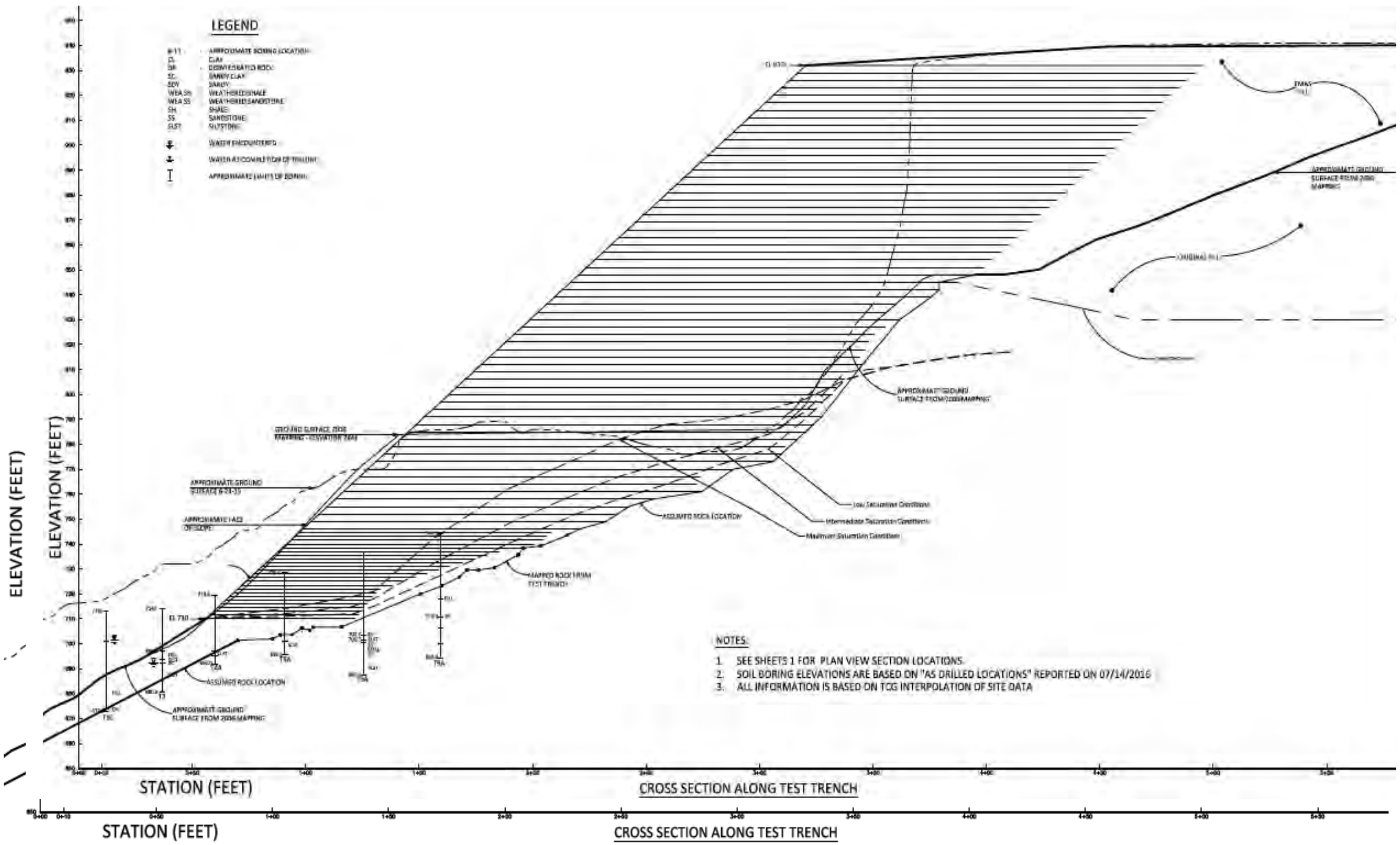


Exposed Sandstone Rock Face

Stability Analysis 2D

Design Cases			
Case	Name	Scenario	Notes
1	Initial Design	L1+G1+S1+Drained	Geogrid LTDS - Uniform length
2	Revised Design	L2+G1+S1+Drained	Geogrid LTDS - Variable geogrid length
3	End of Construction	L2+G2+S1+Drained	Exhumed geogrid - Variable length
4	End of Construction	L2+G2+S3+GW	Exhumed geogrid - Variable length
5	Failure	L2+G3+S3+GW	Exhumed values reduced for creep - Variable length

Groundwater Parametric Evaluation



Our estimate for a maximum groundwater potentiometric surface (groundwater table) is based on Chapter Four of Hoek and Bray.

Stability Analysis 2D Results

Factors of Safety for Cases 1-3 Fully Drained		
Design Case	Water Condition	2D FS
1. Initial Design Case	Dry	1.54
2. Revised Design Case	Dry	1.45
3. End of Construction Case (Peak)	Dry	1.70

Stability Analysis 2D Results

Factors of Safety for Cases 4-5 with Groundwater			
Design Case	Water Condition	Geogrid Tensile Resistance Model	2D FS
4. End of Construction Case (FSS)	Dry	Isotropic	1.15
	Low	Isotropic	1.15
	Medium	Isotropic	1.13
	High	Isotropic	1.13
5. Failure	Dry	Isotropic	1.03
	Low	Isotropic	1.01
	Medium	Isotropic	0.99
	High	Isotropic	0.95

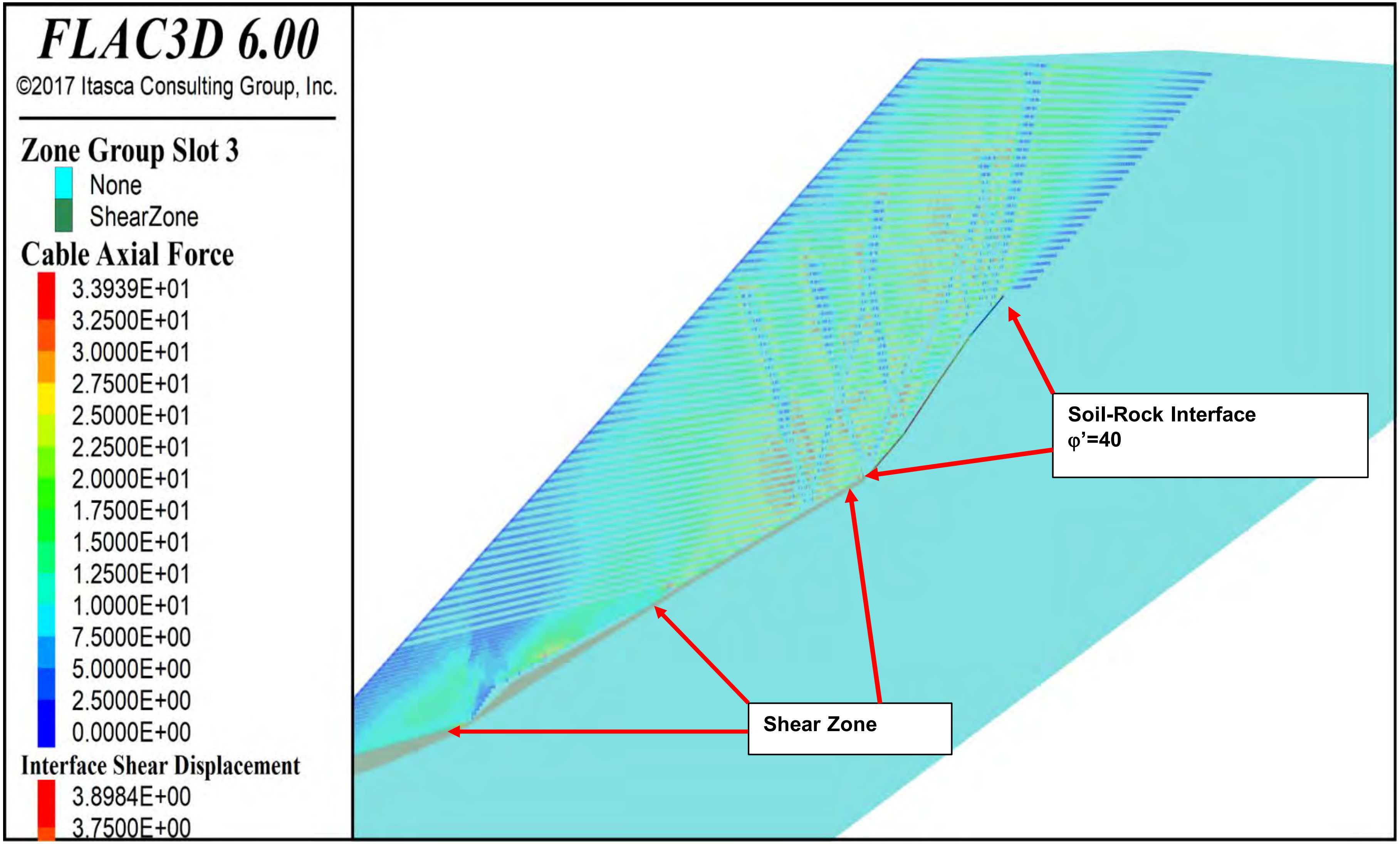
2D Stability Analysis Summary

- Reducing the reinforcement lengths from the original 175 feet to the as-built lengths reduced the factor of safety of the slope.
- Failure to design and construct the RSS on sound rock reduced the factor of safety of the further.
- Not including an internal drainage system in the RSS and the development of groundwater within the RSS further decreased the stability of the slope.
- When the FS was marginal (i.e., less than 1.15) the strength of the geogrid was reduced by creep prior to collapse, as post-peak soil shear strengths were mobilized, resulting in factors of safety of approximately 1 when collapse of the RSS occurred.

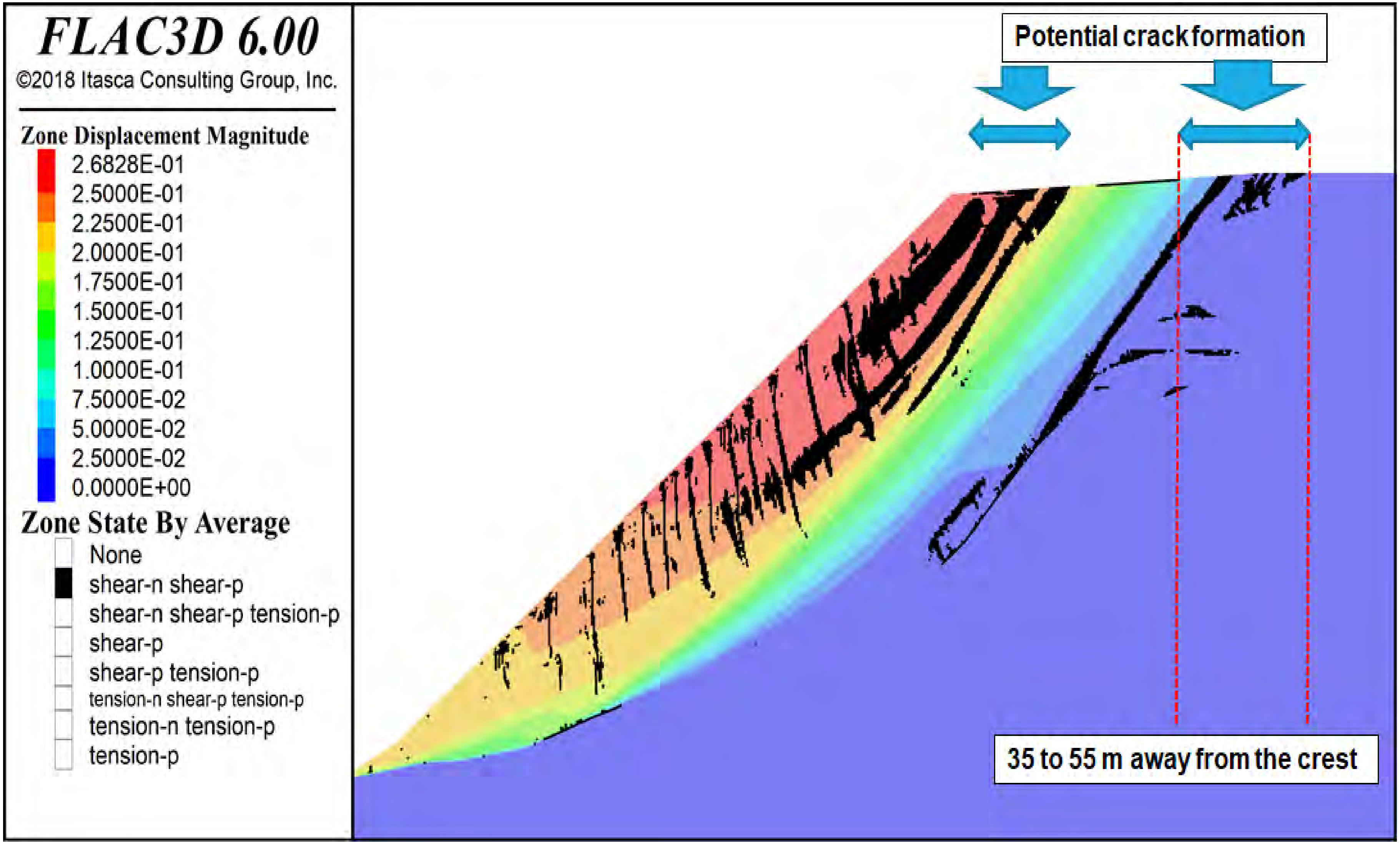
RSS As-built 2D Finite Difference Analysis

- LE performed to evaluate FS of RSS under various loading conditions
- LE not capable of evaluating stress and deformation prior to failure
- FLAC3D slice model selected to further understand the failure kinematics
- FLAC3D analysis performed for Case 4 End of Construction with strength reduction of soil-rock interface
- Geogrid Ultimate strength used to model the construction stages
- Once the activation the geogrid layer was completed the exhumed strength was used
- Geogrids are modeled with non-linear cable elements that can yield and rupture if the strain limit is reached and its contribution to the model was removed.

Soil-Rock interface

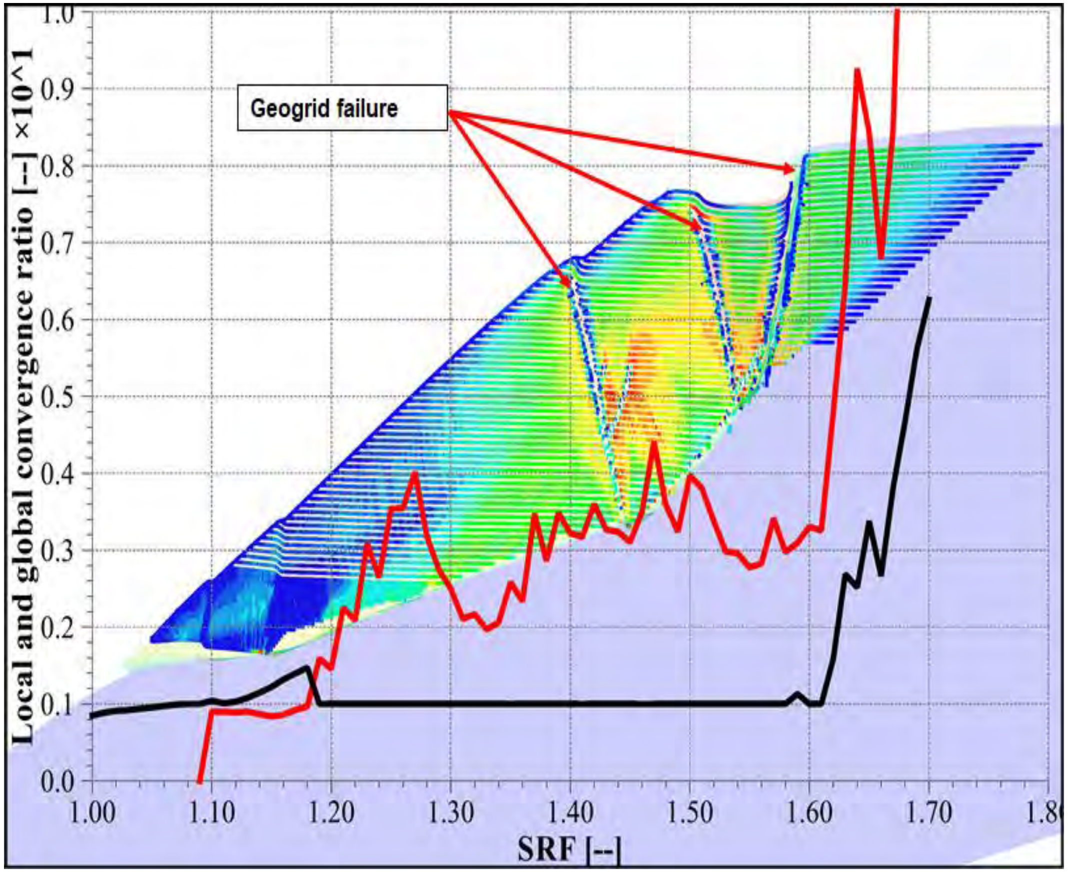


Shear Band development inside RSS at failure using FLAC3D



Failure Mechanism

- Geogrid failure in head scarp and two downslope areas
- Movement occurring behind and below the geogrids in lower portion of RSS where field changes were made to shorten the geogrids
- Deformation analysis results consistent with field observations



Failure Mechanism

- Compound failure mode
- Failure surface below RSS was along a shale-claystone interface
- RSS collapse occurred after 8 years in-service as shear strength of shale-claystone interface decreased from peak towards the fully softened strength

Contributing Factors to the Failure

- Insufficient subsurface exploration program
- 3D aspects of uniaxial geogrids not addressed in design
- Insufficient foundation prep and rock excavation and benching due to inadequate specifications
- Founding RSS on compacted fill instead of freshly excavated bedrock
- Significantly shortening the geogrid reinforcement in the lower portion of RSS
- Deterioration of the soil-rock interface shear strength from the peak towards FSS strength due to the high applied shear stresses and the presence of groundwater at the interface

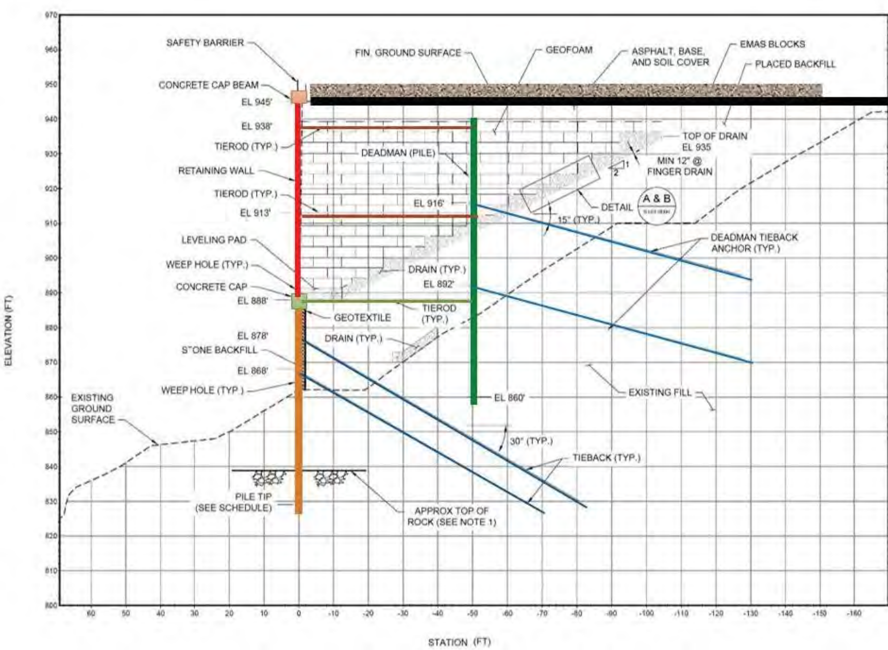
Summary

- The results of the 2D limit equilibrium analyses and the 3D permanent deformation analyses are consistent with the failure mechanism identified in the post forensic subsurface investigation
- Finite difference deformation analyses confirmed that a reduction in strength occurred along the soil-rock interface during the eight (8) year service life of the RSS due to deformations induced by the applied shear stresses and available groundwater.
- The deformation analyses also identified that the failure surface propagated from below the reinforced zone near the slope toe, behind the geogrids in the lower portion of the slope, and through the geogrids in the upper portion of the slope.

Summary

- Tension cracks observed approximately two (2) years before the failure also appeared in the stress-deformation analyses when the tensile geogrid strains reached about 2%. This analysis suggests that stress-deformation analyses can be used to predict the applied shear stresses and strains and possible development of a failure surface through an RSS for future projects.
- As the shear strength of the soil-rock interface decreased from the peak strength towards the FSS, the lower portion of the slope underwent shear deformations, which transferred the shear stresses to the geogrid reinforcement, which resulted in its deformation and creep strength reduction resulting in the failure of this RSS.

Summary


SAFETY BARRIER
FIN. GROUND SURFACE
GEOFOAM
ASPHALT, BASE, AND SOIL COVER
EMAS BLOCKS
PLACED BACKFILL
CONCRETE CAP BEAM
EL 945'
EL 938'
TIEROD (TYP.)
DEADMAN (PILE)
TOP OF DRAIN EL 935
MIN 12" @ FINGER DRAIN
RETAINING WALL
TIEROD (TYP.)
EL 916'
DETAIL
A & B
EL 913'
15° (TYP.)
LEVELING PAD
DEADMAN TIEBACK ANCHOR (TYP.)
WEEP HOLE (TYP.)
DRAIN (TYP.)
EL 892'
CONCRETE CAP
EL 888'
GEOTEXTILE
TIEROD (TYP.)
EL 878'
DRAIN (TYP.)
S'ONE BACKFILL
EL 868'
WEEP HOLE (TYP.)
EL 860'
EXISTING FILL
EXISTING GROUND SURFACE
30° (TYP.)
TIEBACK (TYP.)
PILE TIP (SEE SCHEDULE)
APPROX TOP OF ROCK (SEE NOTE 1)
ELEVATION (FT)
STATION (FT)
970
960
950
940
930
920
910
900
890
880
870
860
850
840
830
820
810
800
60
50
40
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-140
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Contact Information

Ryan Berg, P.E., D.GE., F. ASCE
President
Ryan R Berg & Associates
ryanberg@att.net

Jim Collin, Ph.D., P.E., D.GE., F. ASCE
President
The Collin Group, Ltd.
jim@thecollingroup.com

Timothy D. Stark Ph.D., P.E.
Professor of Civil & Environmental Engineering
University of Illinois at Urbana-Champaign
Technical Director
Fabricated Geomembrane Institute
tstark@Illinois.edu

Jennifer Miller, M.S.
Coordinator
Fabricated Geomembrane Institute
University of Illinois at Urbana-Champaign
fabricatedgeomembrane@gmail.com

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