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REPORT ON

IMPLICATION OF INCREASES IN THE HEIGHT OF THE FIMISTON AND KALTAILS TAILINGS STORAGE FACILITIES ON STRUCTURAL STABILITY

Submitted to:

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May 2006 05641521-R02

EXECUTIVE SUMMARY

Kalgoorlie Consolidated Gold Mines (KCGM) operate and manage Super Pit and Fimiston Mill, which are projected to continue in operation until 2017. Managing the storage of the mill tailings forms an important component of the operation and KCGM is currently in the process of developing a life of mine solution to the management of tailings. This will require expansion of the existing tailings storage facilities (TSFs), which comprise the Fimiston I and Fimiston II TSFs.

There are currently applications before the regulatory authorities to increase the heights of the Fimiston I and Fimiston II TSFs to provide storage capacity for tailings through to 2012. To provide the life of mine storage requirement through to 2017, further expansion of the facilities will be required. The most robust options under consideration are the following:

- further increase the height of the Fimiston I and Fimiston II TSFs to provide the additional storage capacity; or
- recommission and raise the Kaltails TSF and use in conjunction with the Fimiston TSFs to minimise the necessary increase in the height of the Fimiston TSF embankments.

With these options in mind, the stability of the TSFs have been modelled under the maximum height conditions likely to be achieved under either of the options. Modelling of the embankments was carried out under both static (no seismic) and dynamic (earthquake) conditions of loading. Shear strength parameters used in the modelling are derived from field and laboratory testwork programmes carried out over the life of the operations and on recent piezoprobe testing carried out on both the Fimiston and Kaltails TSFs

The results of the modelling indicate that the TSF embankments would remain stable under the conditions modelled and at the maximum heights considered for the expansion.

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1.0 INTRODUCTION

Kalgoorlie Consolidated Gold Mines (KCGM) Operate the Fimiston gold operations, which are projected to reach the end of the operational life in 2017. A significant element of the Fimiston operations is the need to manage the waste products from the Fimiston mill, in particular, the storage of the mine tailings. KCGM is currently engaged in planning to meet the life of mine tailings storage capacity requirements. In order to accomplish this KCGM will need to expand the capacity of the existing facilities, which comprise the following two tailings storage facilities (TSFs):

- Fimiston I TSF, originally comprising six paddocks, which have since been amalgamated and operated as a single paddock storage. The Fimiston I TSF currently has a functional storage area of approximately 104 ha and maximum perimeter embankment height of approximately 31 m; and

- Fimiston II TSF, which comprises three Paddocks of which Paddock A/B is an amalgamation of original Paddocks A and B. Fimiston II Paddocks A/B, C and D currently have functional storage areas of approximately 110 ha, 89 ha and 96 ha respectively and maximum perimeter embankment heights of approximately 28 m, 27 m and 24 m respectively.

Applications have been submitted to increase the maximum embankment heights from the currently licensed heights of 30 m (32 m on Paddock C of Fimiston II). This will provide sufficient capacity to carry the operation through to 2012. Further increases in storage capacity are required to meet the 2017 storage requirement. The most robust solutions that are being pursued by KCGM are the following:

- further increase in the heights of the Fimiston I and II paddocks to provide the additional storage capacity through to 2017; or

- recommission and raise the existing Kaltails TSF and operate is conjunction with the Fimiston TSFs to minimise the additional increase in height required on those TSFs.

This study assesses the impact on the stability of the respective TSFs that may occur as a result of increases in height of the paddocks. The model geometry represents the TSFs at the maximum height that would be required under either of the scenarios.

The general layout and locations of the Fimiston and Kaltails TSFs relative to the Fimiston Mill are shown on Figure 1.

2.0 OVERVIEW OF STABILITY MODELLING

2.1 Maximum Heights of TSF Cells

The proposed maximum heights of the paddocks envisaged under the alternative scenarios are summarised in Table 1 below.

Table 1: Maximum Embankment Heights under Alternative Scenarios

Tailings Storage Facility	Maximum Embankment Height (m)*		
	Fimiston I & Fimiston II	Fimiston I, Fimiston II and Kaltails	Maximum Height for Modelling
Fimiston I	50	40	50
Fimiston II A/B Paddock	59.2	45.2	59.2
Fimiston II C Paddock	57.9	43.9	57.9
Fimiston II D Paddock	55.7	40.7	55.7
Kaltails	-	43.5	43.5

*Based on Golder Associates report, 2006

The stability of the TSFs has been modelled on a number of representative sections through each paddock with the TSFs at the maximum heights envisaged under the different options.

2.2 Modelling Software and Model Geometry

The stabilities of the perimeter embankments of the Fimiston and Kaltails TSFs have been assessed using the limit equilibrium computer software package SLIDE. The geometries used in the analyses for the Fimiston I and II TSFs are based on recent survey data provided by KCGM. The cross sections were analysed using the Morgenstern-Price method under both static and pseudo-static (earthquake) conditions. Superficial failures of less than 1 m depth were ignored in this study.

2.3 Minimum Factors of Safety

The following minimum Factors of Safety (FoS) are based on the requirements set down by ANCOLD (ANCOLD, 1999) and are considered appropriate for the Fimiston and Kaltails TSFs:

- Steady state static loading conditions, FoS = 1.5
- Operating Base Earthquake (OBE) conditions, FoS = 1.2.
- Maximum Design Earthquake (MDE), FoS = 1.0

These minimum values are also consistent with other published values for earth dams.

2.4 Seismicity

A site specific probabilistic assessment (Golder Associates, 2004c) was carried out to determine the appropriate Maximum Design Earthquake (MDE) for the TSFs. The study drew upon a catalogue of crustal earthquakes, spanning 50 years from 1954 to 2004, in a subset extending 600 km east, west, north and south from the Fimiston site. In addition, seismic data from the Mt Charlotte mine and Super Pit seismic monitoring system from 1994 to 2004 were considered.

The most critical results from the seismic study referenced above in terms of anticipated ground accelerations are summarised in Table 2 below.

Table 2: Summary of Likely Severest Earthquake Loadings on Fimiston TSFs

Return Period (years)	Peak Ground Acceleration (PGA)	Corresponding Earthquake Magnitude (M_L)
50	0.05g	1.1
100	0.06g	1.3
200	0.10g	1.6
475*	0.08g	1.9
1,000	0.14g	2.3
MCE	0.28g	3.2

*Extracted from AS1170.4 – 1993 Minimum Design Loads on Structures, Part 4 Earthquake Loads, Standards Australia.
NOTE: a 475 year return period corresponds to a 10% likelihood of exceedence in 50 years

The seismic study indicates that earthquake magnitudes of up to 7.3 are possible. However, the peak ground accelerations associated with these events are significantly less than those given in Table 2 (Kramer, 1996).

The selection of an appropriate acceleration coefficient for use in MDE pseudo-static limit equilibrium analyses of embankments such as at the Fimiston TSFs normally recognises that the slope is not rigid and that the peak acceleration due to earthquake loading only lasts for a very short period of time. Several recognised authorities in this field have recommended that an appropriate pseudo-static coefficient should correspond to between one half and one third of the peak maximum anticipated ground acceleration (Kramer, 1996). The revised analyses presented in this report have therefore used reasonably conservative acceleration coefficients of 0.5 × PGA.

Assuming a “High” hazard rating applies to the Fimiston I, Fimiston II and Kaltails TSFs, the design earthquake for the TSFs, according to ANCOLD, should be 1:1,000 years. Accordingly, the corresponding horizontal acceleration for the operating base earthquake (OBE) is estimated at $0.5 \times 0.14g = 0.07g$ and the horizontal acceleration for the maximum design earthquake (MDE) is estimated at 0.5 x MCE = $0.5 \times 0.28g = 0.14g$. These coefficients have been used in the pseudo-static model analyses.

3.0 STABILITY ASSESSMENT

3.1 Fimiston I TSF

3.1.1 Material Parameters

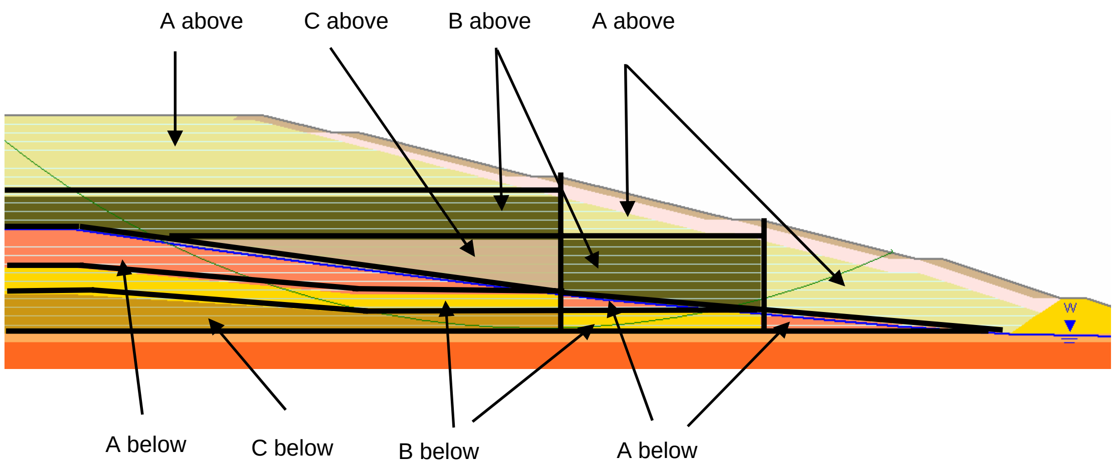
The material parameters and phreatic surface adopted for the analyses are based on interpretation of the piezoprobe results and supported by previous stability analyses (Golder Associates, 2003a, 2003c, 2004b, 2004d). Parameters adopted for the effective stress analyses are supported by past laboratory results. The value adopted for the unit weight of the tailings at the Fimiston TSFs is equivalent to the dry density of the tailings and is a conservative estimate.

To represent the layered nature of the tailings, the material has been divided into “coarser” and “finer” layers. Based on the piezoprobe data, it is judged that a reasonable representation of layering within the TSF is one 200 mm fines layer for each 2 m of deposited tailings. The material parameters for the “finer” and “coarser” layers were obtained from examination and analysis of the piezoprobe measurements applicable to the cross-section under examination. Three zones A, B and C have been introduced so that the effect of consolidation, reduced pore pressure and increased overburden weights can be represented in the form of increased undrained shear strength (refer illustration below). These assumptions have been incorporated into the stability analyses and the adopted parameters are summarised in Table 3.

Table 3: Parameters used in Slope Stability Analyses – Fimiston I

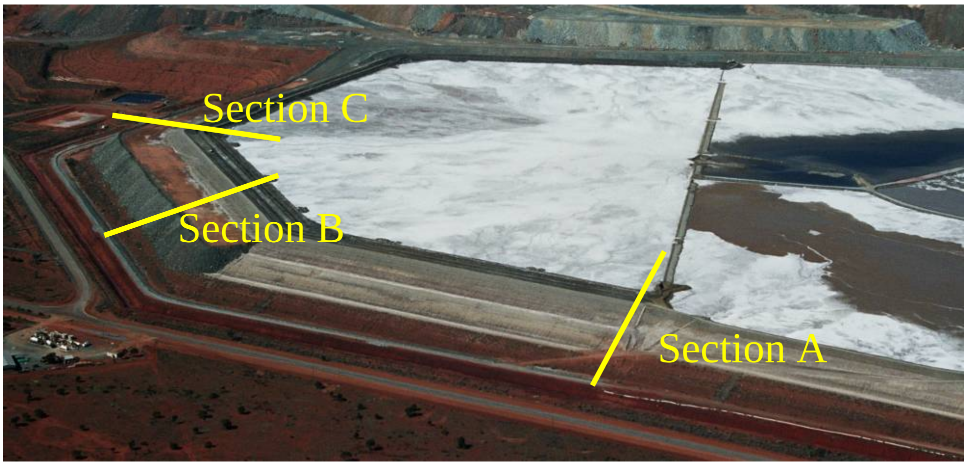
Material	Zone	Unit Weight (γ_m) (kN/m^3)	Friction Angle (ϕ') (degrees)	Cohesion (c') (kPa)	Undrained Shear Strength (s_u) (kPa)	
					Above Phreatic Surface	Below Phreatic Surface
Coarse Tailings	A	16	35 (33)	0	200	100 (80)
Fine Tailings		16	27 (25)	0	20	20
Coarse Tailings	B	16	35 (33)	0	240	130 (110)
Fine Tailings		16	27 (25)	0	25	25
Coarse Tailings	C	16	35 (33)	0	280	160 (140)
Fine Tailings		16	27 (25)	0	30	30
Tailings in Borrow		16	30	0	-	-
Embankment Raises		19	35	7	-	-
Starter Embankment		19	30	17	-	-
Upper Foundation		22	29	25	-	-
Lower Foundation		22	30	40	-	-
Rock Cover		20	38	0	-	-

Note: Figures in parenthesis relate to some weaker zones in Section B only.



3.1.2 Representative sections

Three representative sections through the Fimiston I perimeter embankment at the locations shown on the aerial photograph below have been analysed. Each section has been modelled with the crest at the maximum elevation required to provide the life of mine storage as shown in Table 2. The geometry of the sections reflect the intention to raise the perimeter embankments in an upstream direction with an external batter slope of 1V:4H. It has been assumed that a 6 m wide step-in will be incorporated into the outer slope profile at 10m vertical intervals.



Approximate Locations of Section Taken at the Fimiston I TSF

Figures 2 to 4 represent cross sections taken at Sections A, B and C, respectively. The Figures show the different zones of material and locations of the phreatic surface through each section. The relative positions of the phreatic surface in the model geometry have been interpolated from the piezoprobe testing, piezometers and observation.

3.1.3 Effective Stress Analyses

Effective stress stability analyses (using the computer software code SLIDE) have been carried out on the three sections, assuming static loading conditions. The results are presented in Table 4 below and are shown on Figures 2 to 4, for Sections A to C, respectively.

Table 4: Results of Effective Stress Slope Stability Analyses – Fimiston I

Section	Minimum Factor of Safety	
	Current Height	At Proposed Maximum Heights
A	2.0	2.0
B	2.1	2.0
C	2.1	2.1

Under static loading, there is very little difference in the slope Factor of Safety after raising the TSF, as the critical failure surfaces remain in the lower benches/starter embankment.

Supplementary analyses, adopting a larger unit weight of tailings (19 kN/m^3) were also carried out to check the sensitivity of the models to this parameter. The results indicated a slightly improved factor of safety.

3.1.4 Total Stress (Undrained) Analyses

To analyse the stability of the representative sections under earthquake (dynamic) loading, it is appropriate to utilise undrained strength parameters (refer to Table 3) for the “coarser” tailings below the phreatic surface and for the “finer” tailings throughout. The results of the total stress analyses are summarised in Table 5 and presented on Figures 2 to 4 for Sections A to C, respectively.

Table 5: Results of Total Stress (Undrained) Slope Stability Analyses – Fimiston I

Section	Minimum Factor of Safety			
	Current Height		At Proposed Maximum Heights	
	OBE (0.07g)	MDE (0.14g)	OBE (0.07g)	MDE (0.14g)
A	1.7	1.2	1.5	1.1
B	1.2	1.1	1.2	1.0
C	1.5	1.1	1.5	1.1

The above results indicate that the Factor of Safety of the outer slope of all of the Sections in Fimiston I satisfy the minimum factor of safety requirement of 1.2 for the OBE and 1.0 for MDE. A more detailed assessment of the potential seismic response of the TSF is recommended in view of the closeness of the analysed result to the guideline minimum factors of safety for the OBE and MDE cases.

3.2 Fimiston II TSF

3.2.1 Material Parameters

The material parameters and phreatic surface adopted for the analyses are based on interpretation of the piezoprobe results and supported by previous stability analyses (Golder Associates, 2003c, 2004b, 2004d, 2005). Parameters adopted for the effective stress analyses are based on past laboratory results and are consistent with previous analyses.

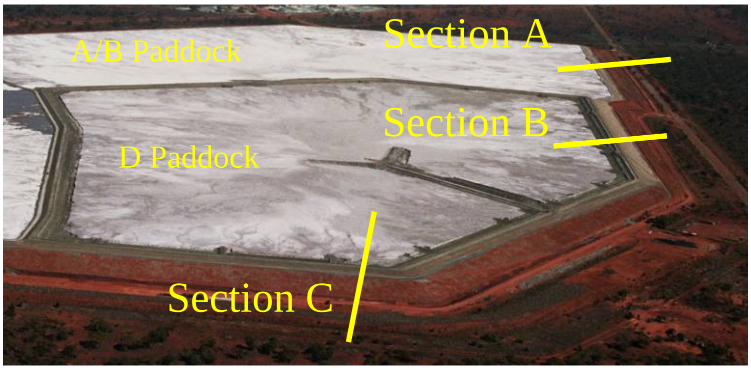
To represent the layered nature of the tailings, the material has been divided into eight zones based on strength. The location and thickness of each zone was estimated from examination and analysis of the piezoprobe measurements applicable to the relevant cross-section being analysed. These assumptions have been incorporated into the stability analyses and the adopted parameters are summarised in Table 6.

Table 6: Parameters used in Slope Stability Analyses – Fimiston II

Material	Unit Weight (γ_m) (kN/m^3)	Friction Angle (ϕ') (degrees)	Cohesion (c') (kPa)	Undrained Shear Strength (s_u) (kPa)
Tailings 1	16	36	0	500
Tailings 2	16	35	0	400
Tailings 3	16	33	0	250
Tailings 4	16	31	0	200
Tailings 5	16	30	0	150
Tailings 6	16	29	0	100
Tailings 7	16	28	0	80
Tailings 8	16	27	0	50
Tailings in Borrow	16	30	0	-
Embankment Raises	19	35	7	-
Starter Embankment	19	30	17	-
Upper Foundation	22	29	25	-
Lower Foundation	22	30	40	-
Rock Cover	20	38	0	-

3.2.2 Representative sections

Five representative sections have been analysed, as shown below. Each section has been analysed using two elevations for the perimeter wall (current and final elevations).



Approximate Locations of Sections A to C at the Fimiston II TSF

Approximate Locations of Sections D and E at the Fimiston II TSF

Figures 5 to 9 present cross sections taken at Sections A, B, C, D and E, respectively. The Figures show the different zones of material and locations of the phreatic surface through each section. The relative positions of the phreatic surface in the model geometry have been interpolated from the piezoprobe testing, piezometers and observation.

3.2.3 Effective Stress Analyses

Effective stress stability analyses have been carried out on the three sections, assuming static loading conditions. The results are presented in Table 7 below and are shown on Figures 5 to 9, for Sections A to E, respectively.

Table 7: Results of Effective Stress Slope Stability Analyses – Fimiston II

Section	Minimum Factor of Safety	
	Current Height	At Proposed Maximum Heights
A	2.1	2.2
B	2.5	2.3
C	2.6	2.6
D	2.3	2.5
E	1.7	1.9

Under static loading, it is evident that there is unlikely to be slope instability. Supplementary analyses, similar to that used for the Fimiston I sections, using a larger unit weight (19 kN/m^3) showed a marginal improvement in the factor of safety.

3.2.4 Total Stress (Undrained) Analyses

To analyse the stability of the representative sections under earthquake (dynamic) loading, it is considered appropriate to utilise undrained strength parameters (refer to Table 6) for the zones of tailings identified. The results of the total stress analyses are summarised in Table 8 and presented on Figures 5 to 9 for Sections A to E, respectively.

Table 8: Results of Total Stress (Undrained) Slope Stability Analyses – Fimiston II

Section	Minimum Factor of Safety			
	Current Height		At Proposed Maximum Heights	
	OBE (0.07g)	MCE (0.14g)	OBE (0.07g)	MCE (0.14g)
A	1.8	1.4	1.8	1.4
B	1.9	1.6	1.7	1.4
C	2.0	1.6	1.6	1.2
D	1.9	1.5	1.6	1.3
E	1.5	1.3	1.3	1.1

The above results indicate that slope instability at Fimiston II is unlikely to occur under the current or the proposed maximum height conditions, even under MDE loading.

3.3 Kaltails TSF

3.3.1 Material Parameters

The material parameters and phreatic surface adopted for the analysis are based on interpretation of the piezoprobe results and supported by experience with tailings similar in nature (Golder Associates, 2004a). Parameters used in the effective stress analyses are based on empirical relationships between pore pressure ratio and friction angle (Lunne et al, 1997).

To represent the layered nature of the material, the tailings has been divided into coarse and fine layers, based on the piezoprobe data. It is judged that six 200 mm layers over the depth of the TSF are sufficient to represent the variation in the tailings. Additionally, the piezoprobe data indicates the presence of weaker, saturated zones of thickness in the order of 5 m immediately overlying natural ground. The strength of the material in the weaker zone is likely to be influenced by overburden stress, and hence may decrease in strength closer to the TSF wall (where there is less overburden and hence a lower normal stress on the material).

Therefore, the presence of this weaker, saturated zone has been modelled as a zone of fine tailings of thickness 5 m, decreasing in strength from 100 kPa at the probe location to 30 kPa near the starter embankment. These assumptions have been incorporated into the stability analyses. The adopted parameters are summarised in Table 9.

Table 9: Parameters used in Slope Stability Analyses - Kaltails

Material	Unit Weight (γ_m) (kN/m^3)	Friction Angle (ϕ') (degrees)	Cohesion (c') (kPa)	Undrained Shear Strength (s_u) (kPa)	
				Above Phreatic Surface	Below Phreatic Surface
Coarse Tailings	14.9	34	0	400	150
Fine Tailings	14.9	27	0	100	30 - 100
Embankment Raises	18	35	0/10	600	-
Starter Embankment	19	34	5	600	200
Foundation	22	35	100	1000	800

3.3.2 Analysis under Static (No Seismic) Condition

An initial slope stability analysis of the Kaltails TSF was carried out under static conditions adopting effective stress parameters (Figure 10). The results indicate a Factor of Safety of 1.7, which is above the ANCOLD minimum guideline factor of safety of 1.5 outlined in Section 2.3.

An analysis using total stress parameters for the tailings below the phreatic surface was also carried out, which returned a minimum factor of safety of 1.7, consistent with the effective stress analysis, and indicates that realistic undrained shear strength parameters have been derived for the layered tailings below the phreatic surface.

The results are summarised in Table 10.

Table 10: Results of Kaltails Analyses Under Static Conditions

Section	Minimum Factor of Safety At Maximum Height	
	Effective Stress Parameters	Total Stress Parameters
Section A	1.7	1.7

3.3.3 Analyses under Dynamic (Pseudo-static) Conditions

Analyses were carried out on the model under dynamic (earthquake) loading conditions using total stress parameters for all layers of fine tailings, as well as the zones of coarse tailings that are below the assumed phreatic surface.

The results, shown on Figure 10, indicate a minimum FoS of 1.3 (OBE) and 1.1 (MDE). These values are above the ANCOLD guidelines for minimum factors of safety for both the OBE and the MDE case. It is therefore judged to be unlikely that major slope instability would occur within the Kaltails TSF embankment under dynamic loading conditions. Nevertheless, some superficial instability may occur in the top 1 m of the soil column during earthquake events. Shake-down of the slope has not been modelled, and may occur during significant earth tremors.

4.0 CONCLUSIONS

Stability modelling has been carried out on the Fimiston I and Fimiston II TSFs with embankments at the maximum crest elevation required to provide the life of mine tailings storage capacity through to 2017, without recourse to an alternative TSF. The stability of the Kaltails TSF has been modelled at the maximum embankment crest elevation that would be necessary to provide the Fimiston life of mine tailings storage capacity if Kaltails were to be used in conjunction with the Fimiston I and Fimiston II TSFs.

Material strength parameters used in the analyses are based on the results of field and laboratory testing programmes carried out over the life of the operation and on recent piezoprobe work carried out on the Fimiston and Kaltails TSFs.

The results of the analyses are summarised as follows:

- The stability analyses produced no incidents of model slope failure.

- While the factors of safety obtained meet the ANCOLD guidelines for minimum factors of safety under static loading conditions and OBE and MDE earthquake conditions, the results obtained for the Fimiston I TSF are sufficiently close to the minimum factor of safety benchmarks that we recommend a more detailed assessment to guage the potential seismic response of the TSF.

It is also recommended that settlement (displacement) analysis be carried out using a finite element programme such as Plaxis, which incorporates the effect of consolidation, to model the deformations and impacts on stability that may occur as the height of the TSFs increase.

If the proposal to increase the height of the TSFs is implemented, then a programme of *in situ testing* should be carried out at regular intervals coupled with reassessment of the stability to continue to validate the data used in the current analyses and results of the analyses.

5.0 IMPORTANT INFORMATION

Your attention is drawn to the document - “Important Information About Your Geotechnical Engineering Report”, which is included in Appendix A of this report. This document has been prepared by the ASFE *(Professional Firms Practicing in the Geosciences)*, of which Golder Associates is a member. The statements presented in this document are intended to advise you of what your realistic expectations of this report should be, and to present you with recommendations on how to minimise the risks associated with the groundworks for this project. The document is not intended to reduce the level of responsibility accepted by Golder Associates, but rather to ensure that all parties who may rely on this report are aware of the responsibilities each assumes in so doing.

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REFERENCES

ANCOLD (1999) "*Guidelines on Tailings Dam Design, Construction and Operation*". Australian National Committee on Large Dams, October 1999.

Australian Standards:AS 1289.6.5.1-1999. "*Determination of the static cone penetration resistance of a soil - Field test using a mechanical and electrical cone or friction-cone penetrometer*".

Golder Associates (1999). "*1999 Annual Operational Audit of the Tailings Storage Facilities at KCGM*". Consultants' report (99640202) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, December1999.

Golder Associates (2003a). "*Addendum to Notice of Intent for Increasing the Capacity of the Fimiston I Tailings Storage Facility at KCGM, Kalgoorlie, Western Australia*". Consultants' report (03641063) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, April 2003.

Golder Associates (2003c). "*2003 Annual Operational Audit of the Tailings Storage Facilities at KCGM*". Consultants' report (03641296) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, December 2003.

Golder Associates (2004a). "*Piezoprobe Investigations carried out on the Kaltails Tailings Storage Facility*". Consultants' report (04641304-R03) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, November 2004.

Golder Associates (2004b). "*Piezoprobe Investigations carried out on the Fimiston Tailings Storage Facilities*". Consultants' report (04641304-R01) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, October 2004.

Golder Associates NZ (2004c). "*Seismic Risk Assessment KCGM Fimiston I and II Tailings Dams*". Consultants' report (04652022/R003) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, November 2004.

Golder Associates (2004d). "*Stability Analyses and Freeboard Estimations for Fimiston Tailings Storage Facilities*". Consultants' letter (04641119_L03_rev1) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, November 2004.

Golder Associates (2005). "*Addendum to Notice of Intent:Proposed Increase in the Storage Capacity of the Fimiston II Tailings Storage Facility at KCGM.*" Consultants' report (05641089-R01) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, September 2005.

Golder Associates (2006). "*Tailings Management Options at Kalgoorlie Consolidated Gold Mines for Life of Mine to 2017*." Consultants' report (05641521-R01) submitted to Kalgoorlie Consolidated Gold Mines Pty Ltd, May 2006.

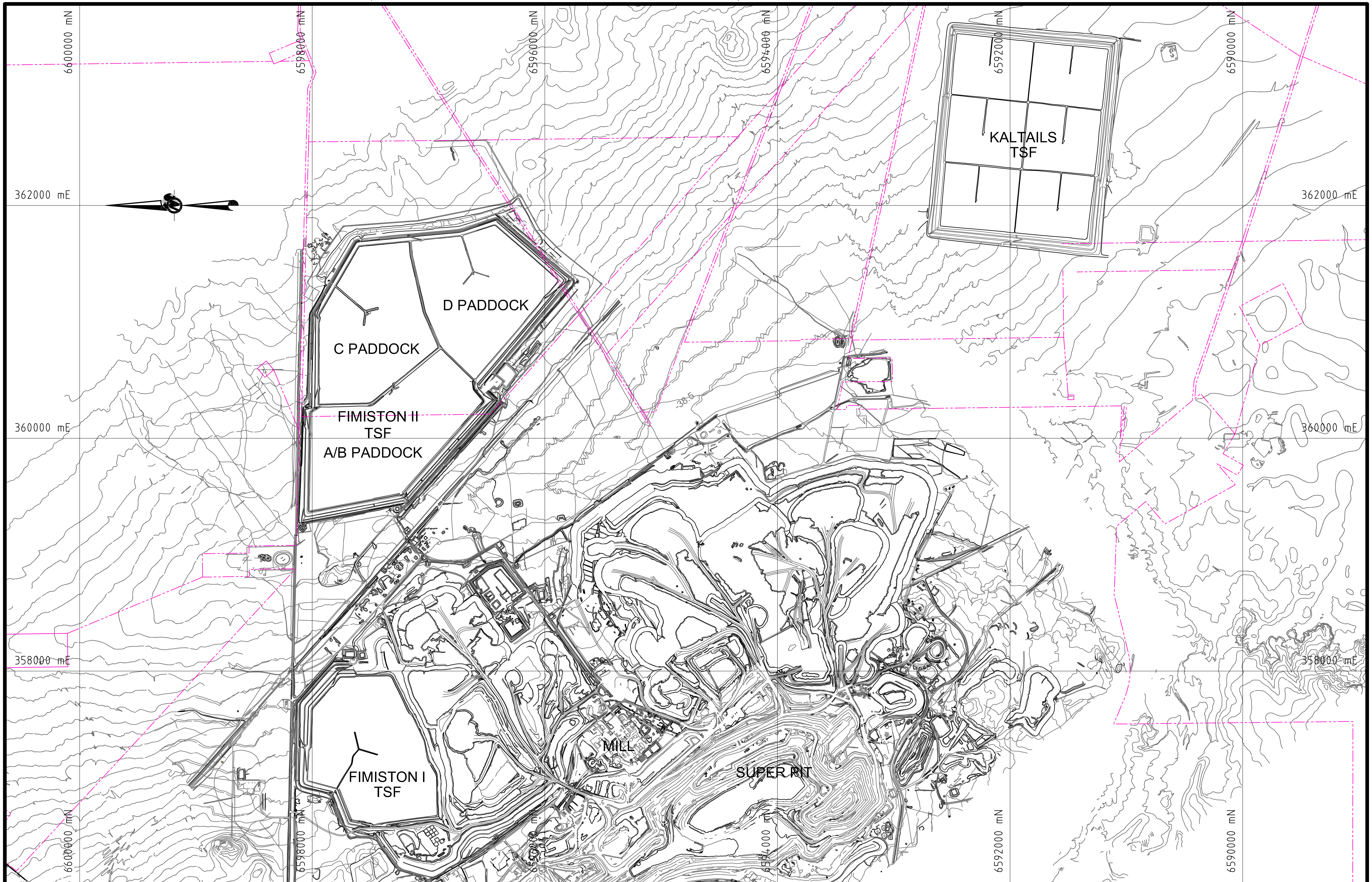
Kramer, SL (1996) *Geotechnical Earthquake Engineering* University of Washington, Prentice-Hall Inc, pp436-7.

Institution of Engineers, Australia, *Australian Rainfall and Runoff, Vol. 1, Book 6,* Institution of Engineers, Australia, Australian Capital Territory, 2001.

International Commission on Large Dams (ICOLD), 1989. *Tailings Dam Safety Guidelines*, Bulletin 74, 1989.

Lunne, T., Robertson, P.K., & Powell, J.J.M. 1997. "*Cone Penetration Testing in Geotechnical Practice*", Blackie Academic/Rutledge Publishers, New York.

Youd et. al. 2001, "*Liquefaction Resistance of Soils*": Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils', Journal of Geotechnical and Geoenvironmental Engineering, October 2001, pp. 817-831.



MAP PROJECTION - MGA 94

Golder Associates

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CHECKED	DATE 30/05/06		
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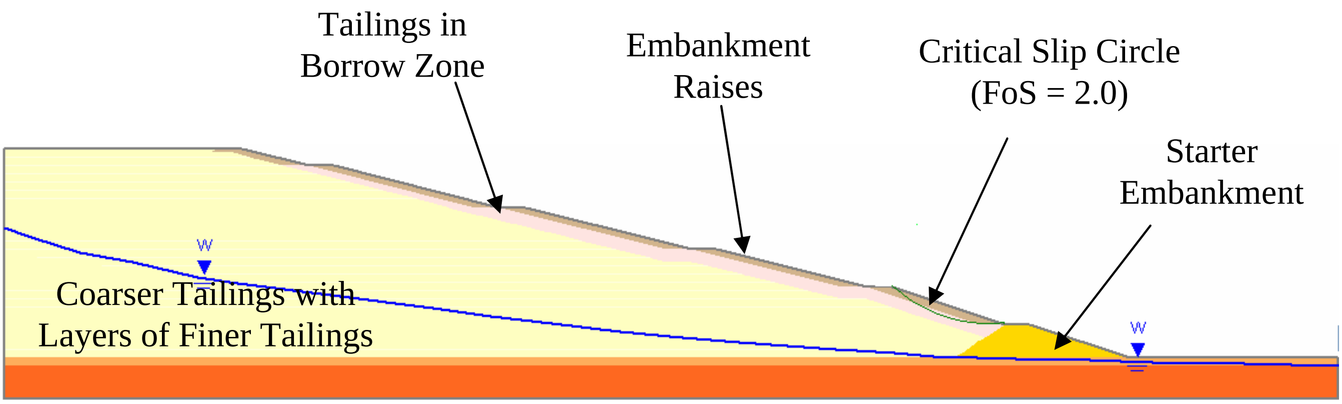
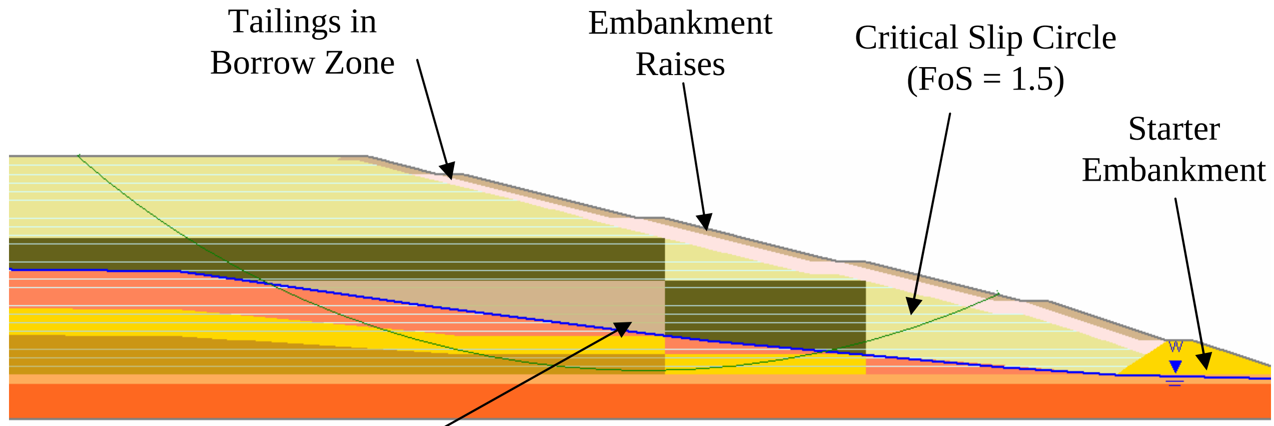
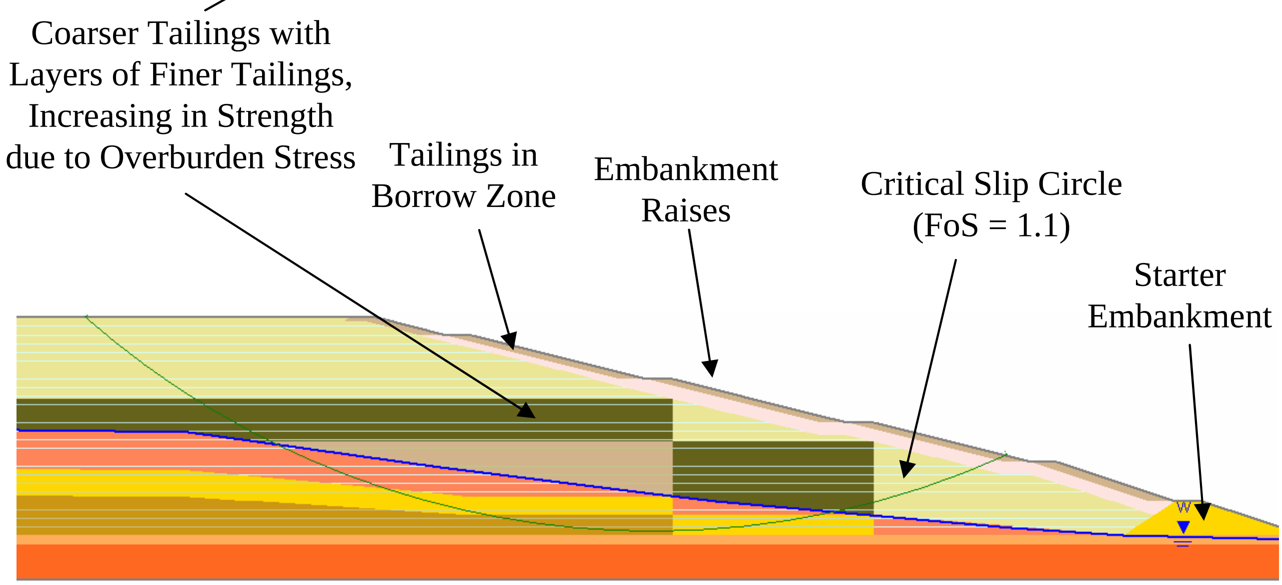


Section A under Static Conditions



Section A under OBE Conditions



Section A under MDE Conditions

Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment
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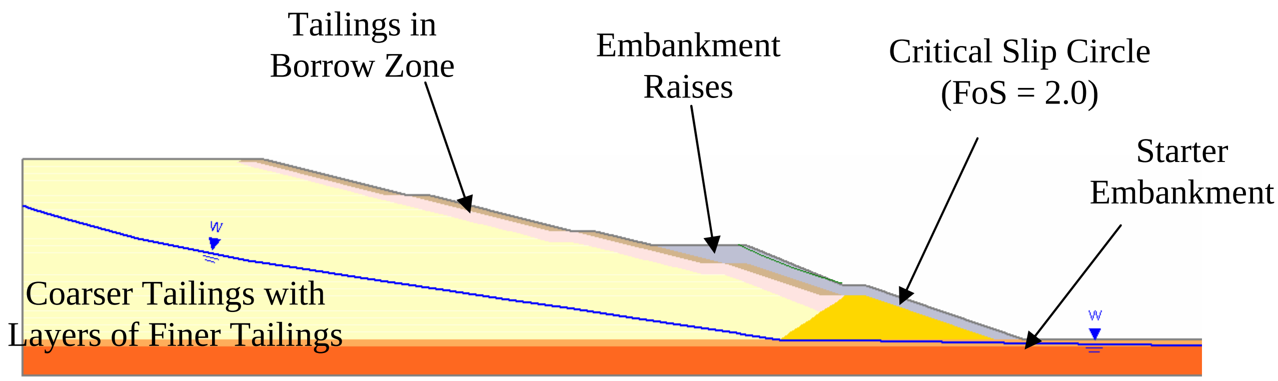
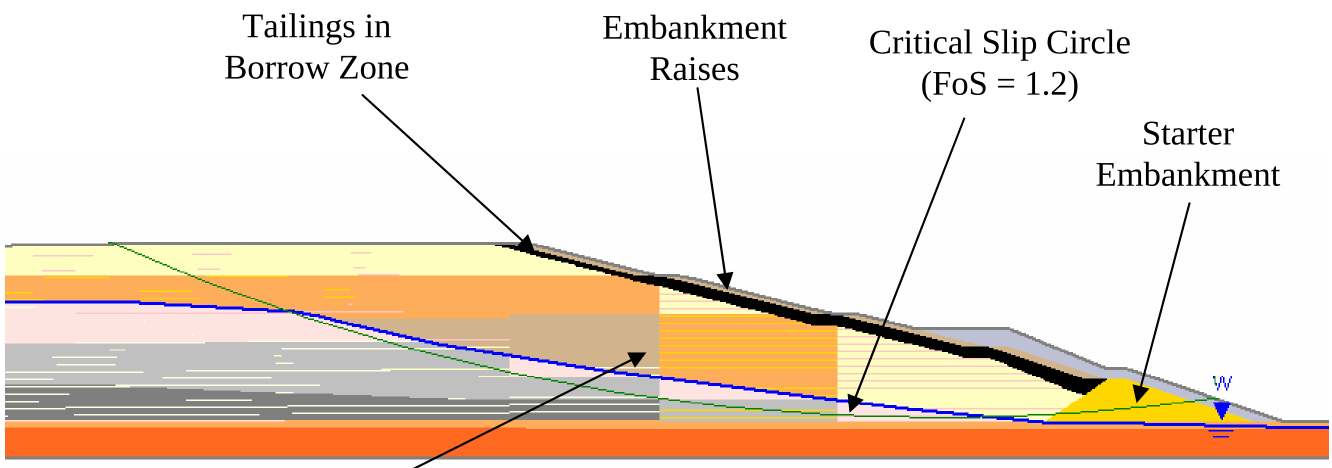
	CHECKED	DATE 03/05/06	
	SCALE Not To Scale	A4	PROJECT No **05641521-R02** / FIGURE No. **FIGURE 2**



Section B under Static Conditions



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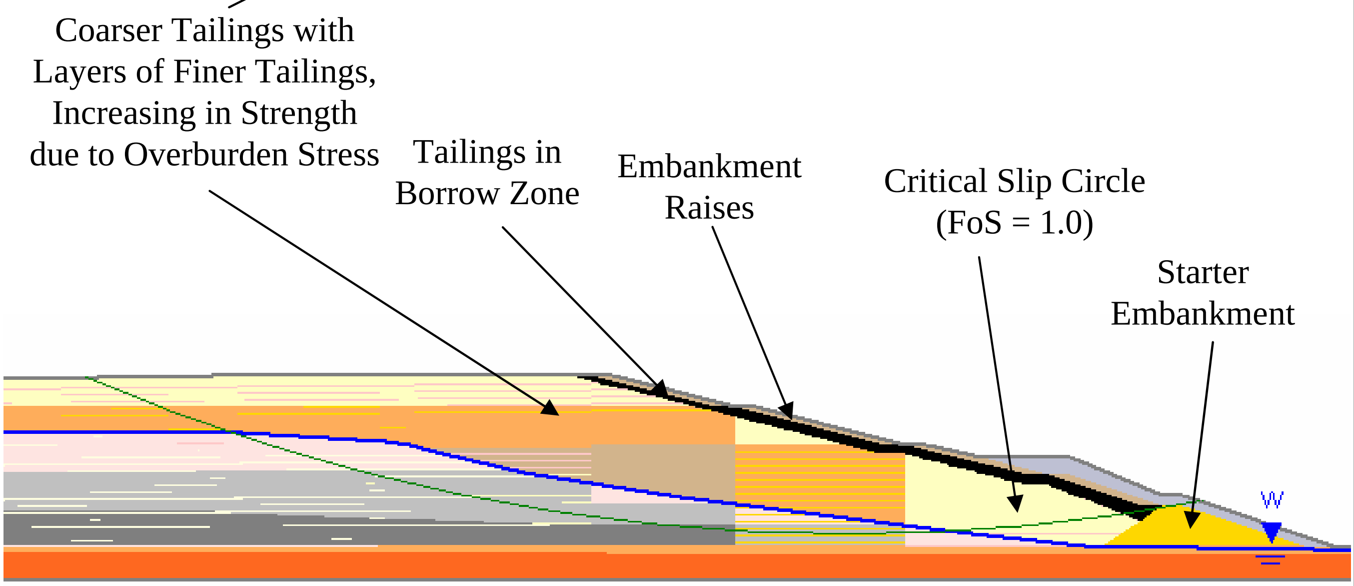
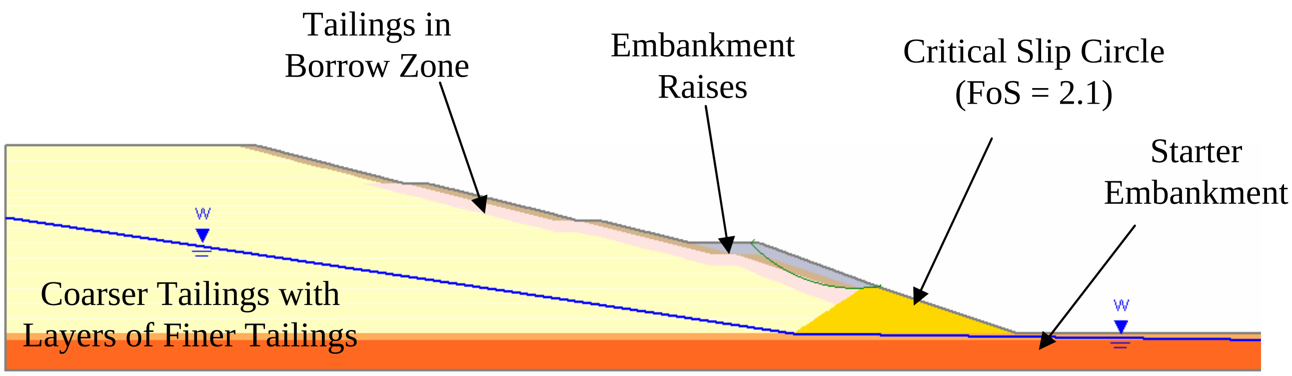


Section B under MDE Conditions

Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06	TITLE **FIMISTON I SECTION B**	
	CHECKED	DATE 03/05/06		
	SCALE Not To Scale	A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 3**



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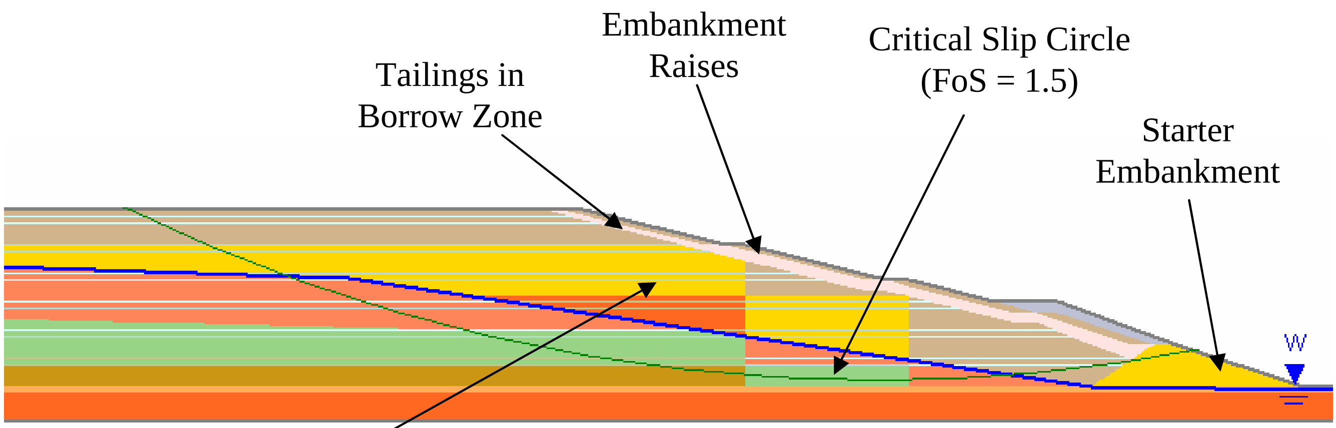

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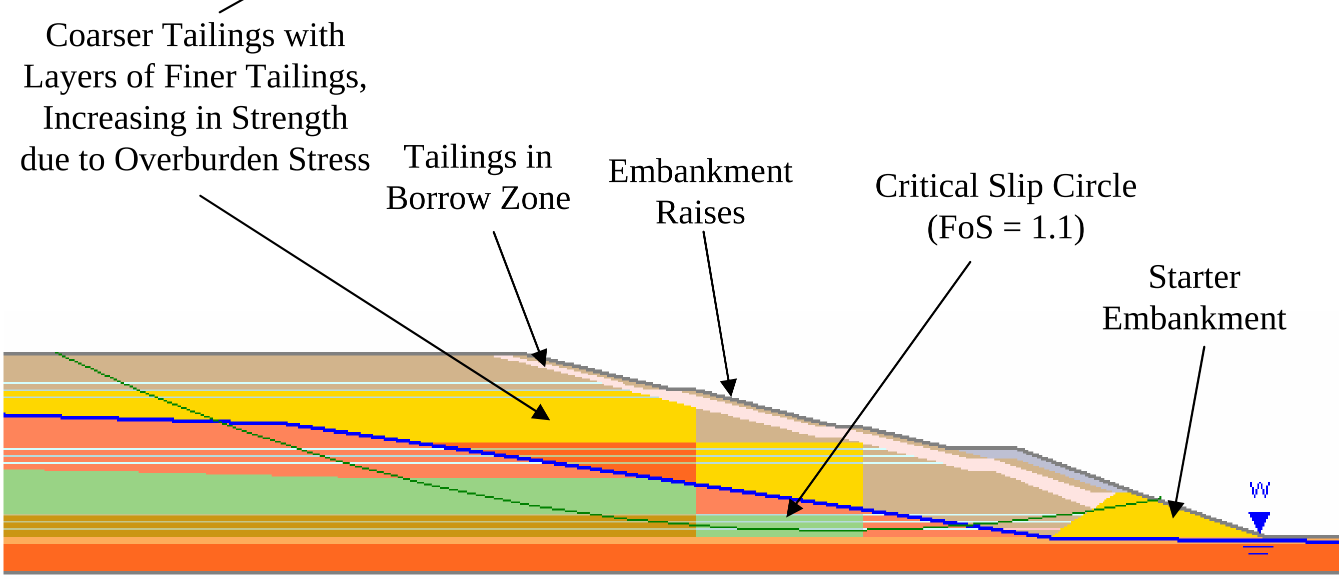
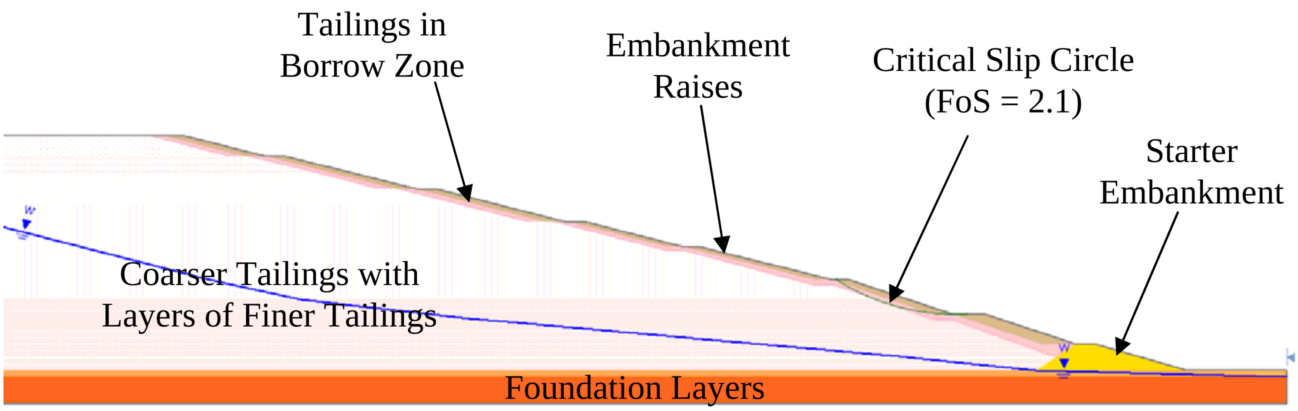
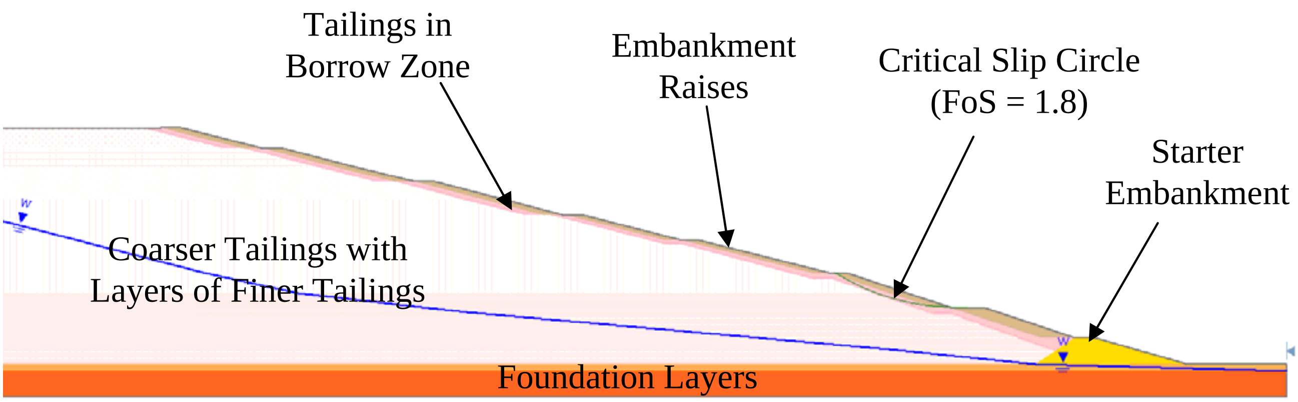


Section C under MDE Conditions

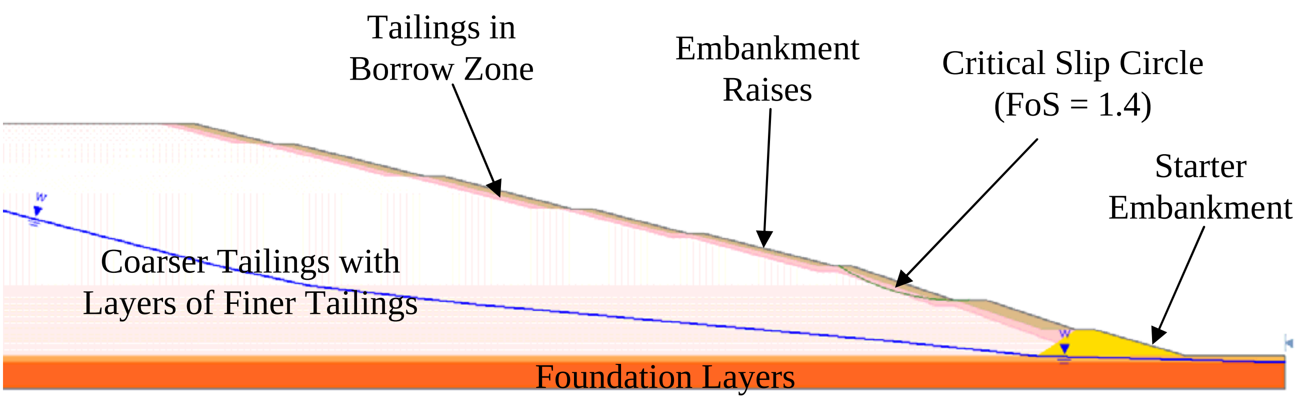
Golder Associates					
	CLIENT KCGM			PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06		TITLE **FIMISTON I SECTION C**	
	CHECKED	DATE 03/05/06			
	SCALE Not To Scale		A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 4**



Section A under Static Conditions

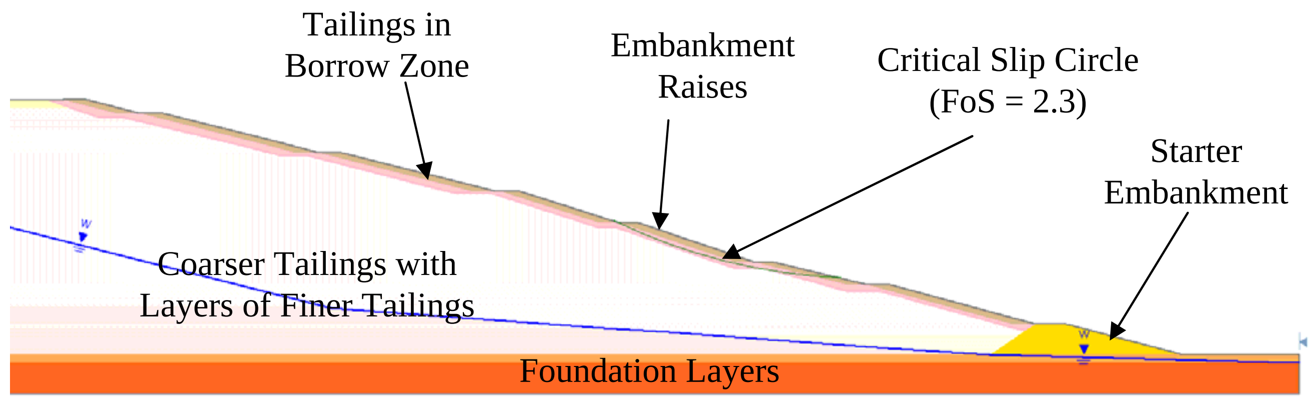


Section A under OBE Conditions



Section A under MDE Conditions

Golder Associates	CLIENT KCGM			PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06		TITLE **FIMISTON II SECTION A**	
	CHECKED	DATE 03/05/06			
	SCALE Not To Scale		A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 5**



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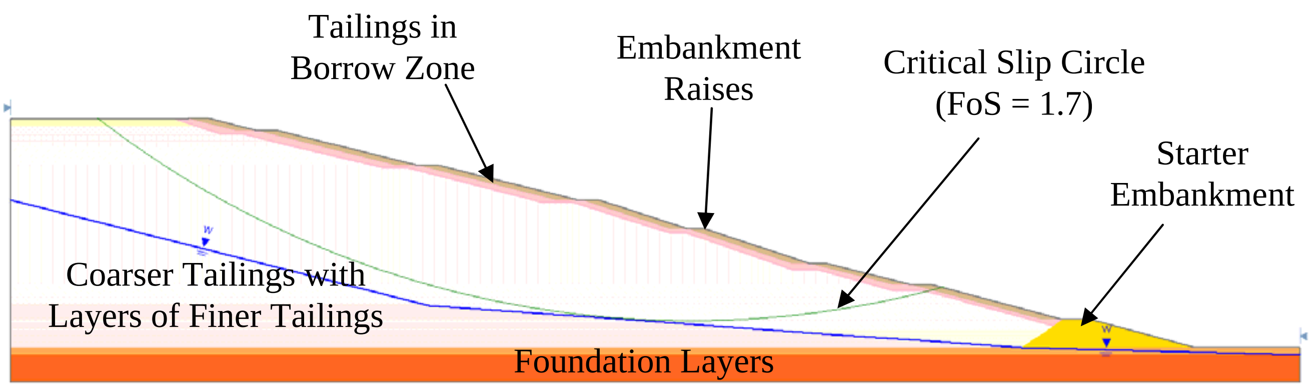

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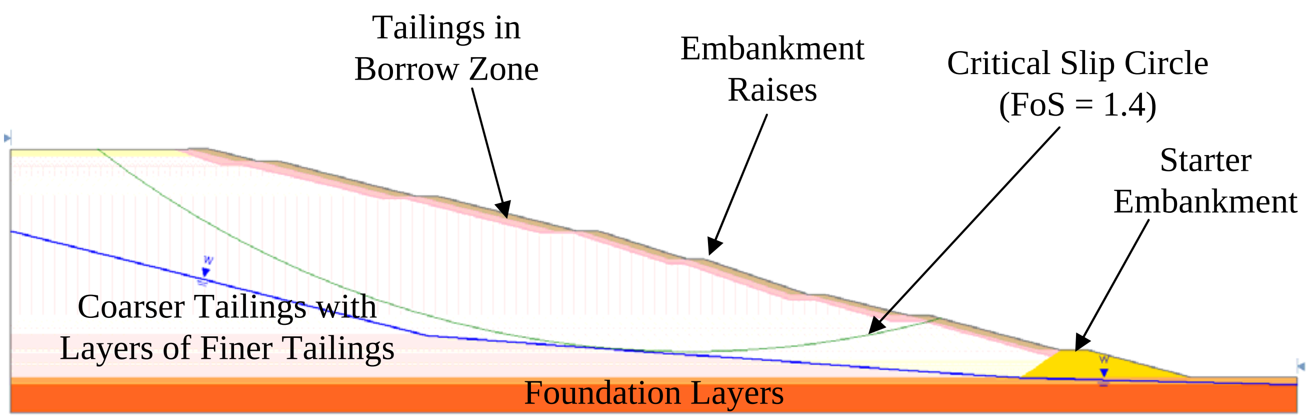
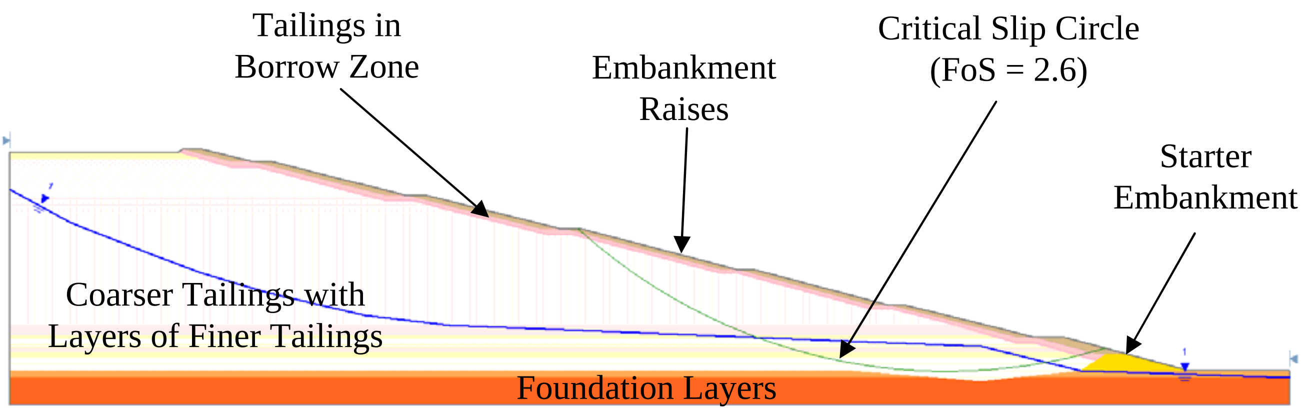
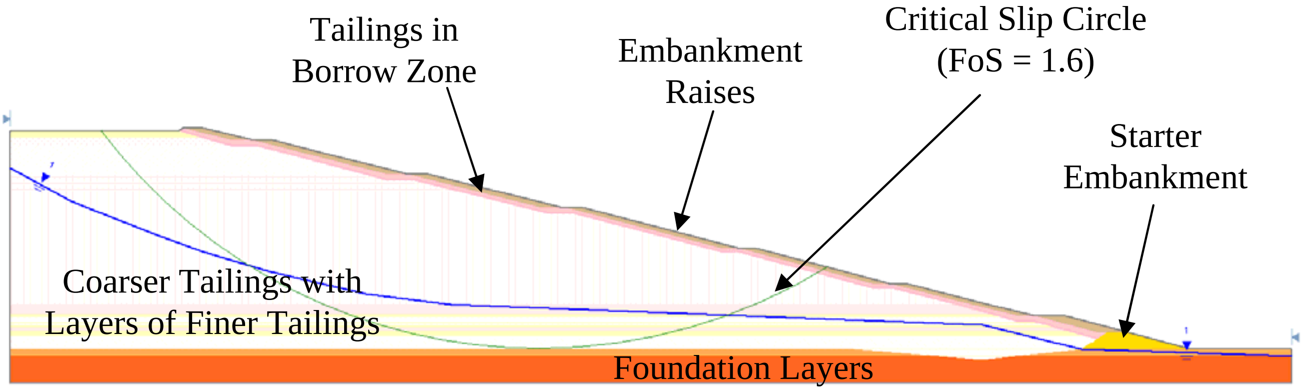


Section B under MDE Conditions

Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06	TITLE **FIMISTON II SECTION B**	
	CHECKED	DATE 03/05/06		
	SCALE Not To Scale A4		PROJECT No **05641521-R02**	FIGURE No. **FIGURE 6**



Section C under Static Conditions



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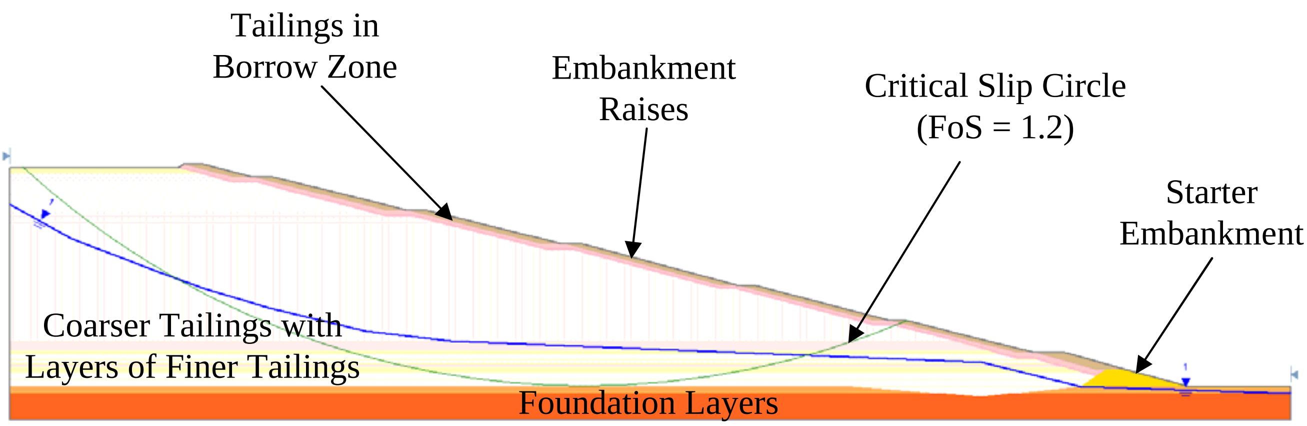
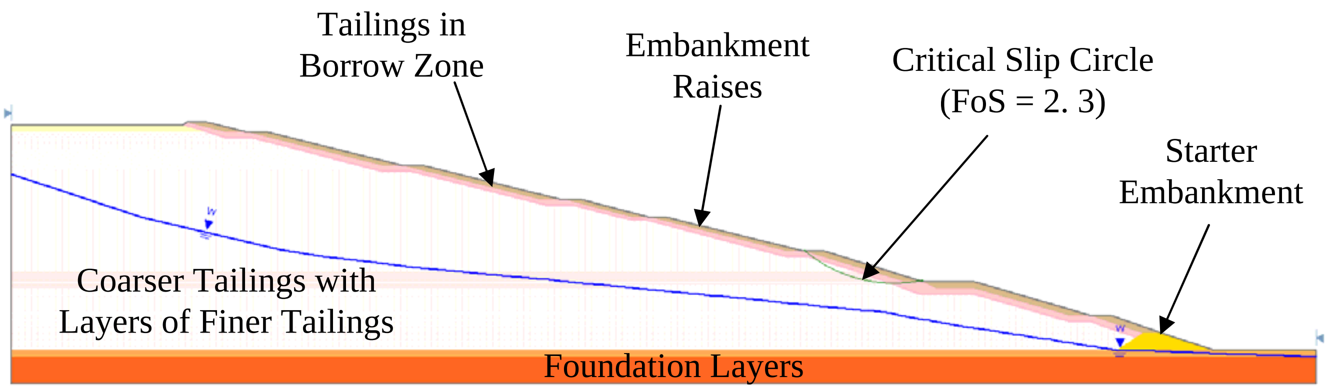
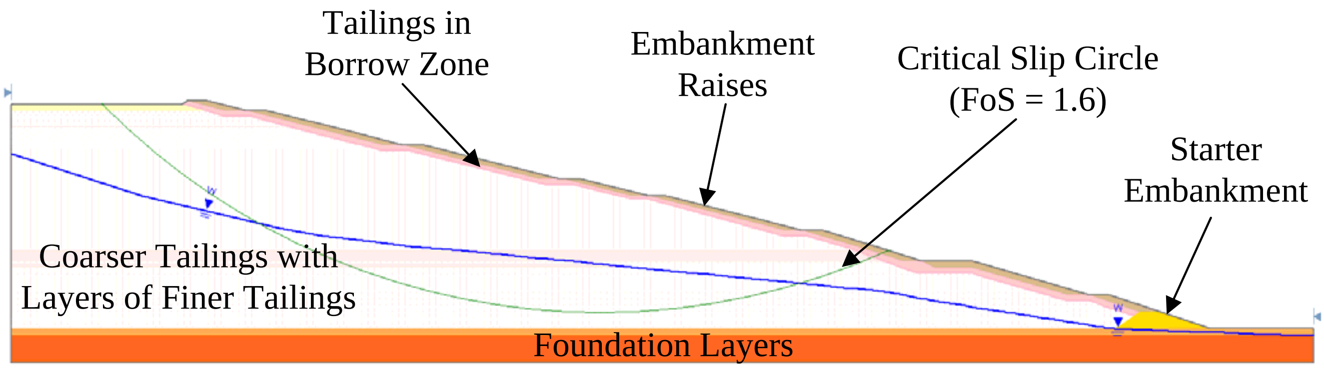


Section C under MDE Conditions

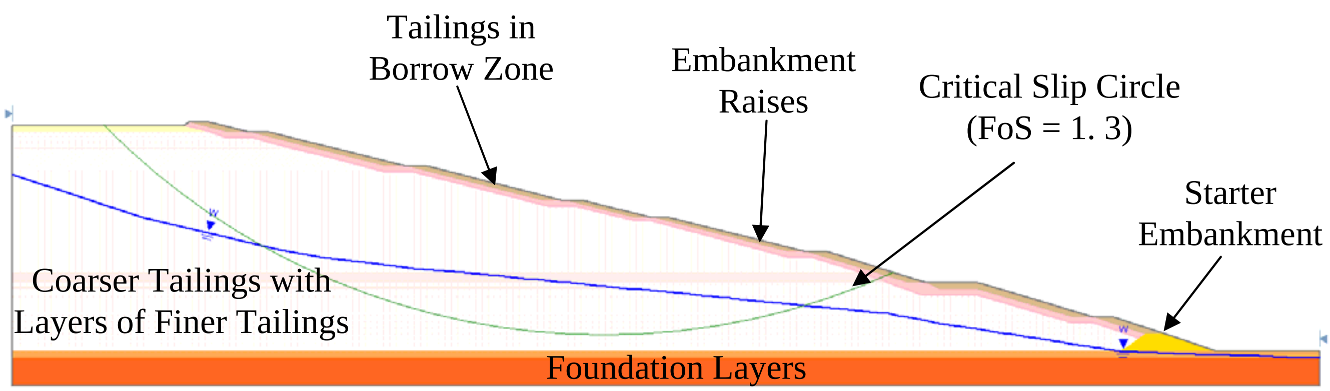
Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06	TITLE FIMISTON II SECTION C	
	CHECKED	DATE 03/05/06		
	SCALE Not To Scale	A4	PROJECT No 05641521-R02	FIGURE No. FIGURE 7



Section D under Static Conditions



Section D under OBE Conditions



Section D under MDE Conditions

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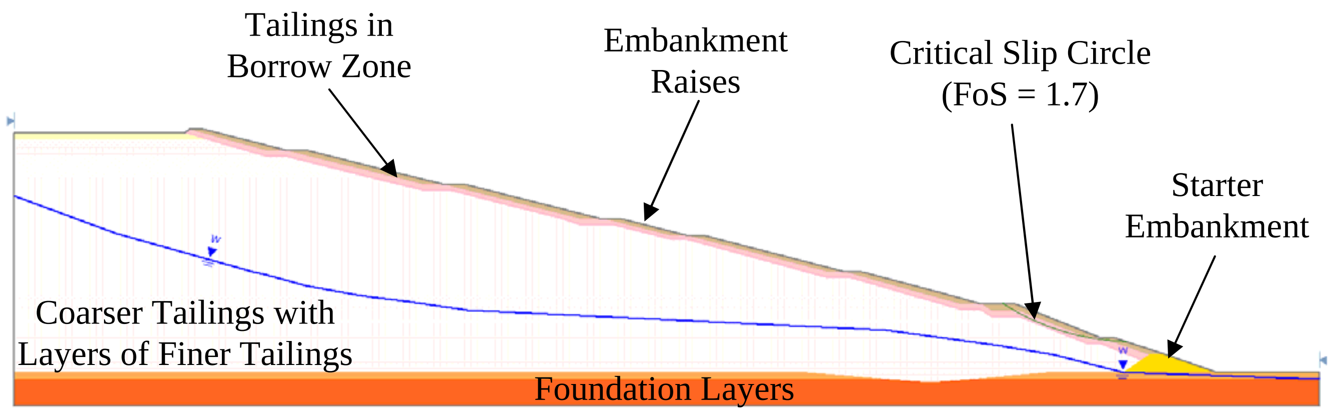
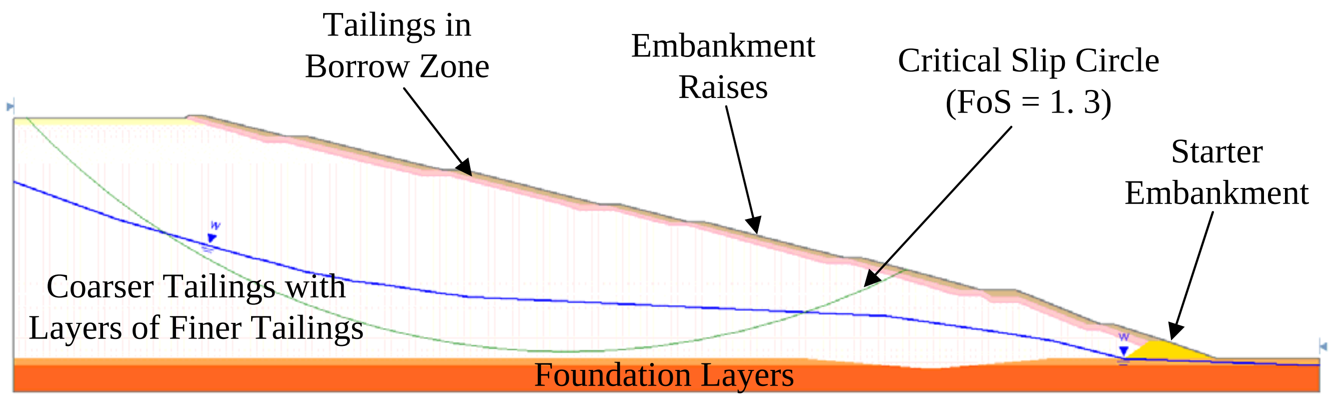
	CHECKED	DATE 03/05/06		
	SCALE Not To Scale	A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 8**



Section E under Static Conditions



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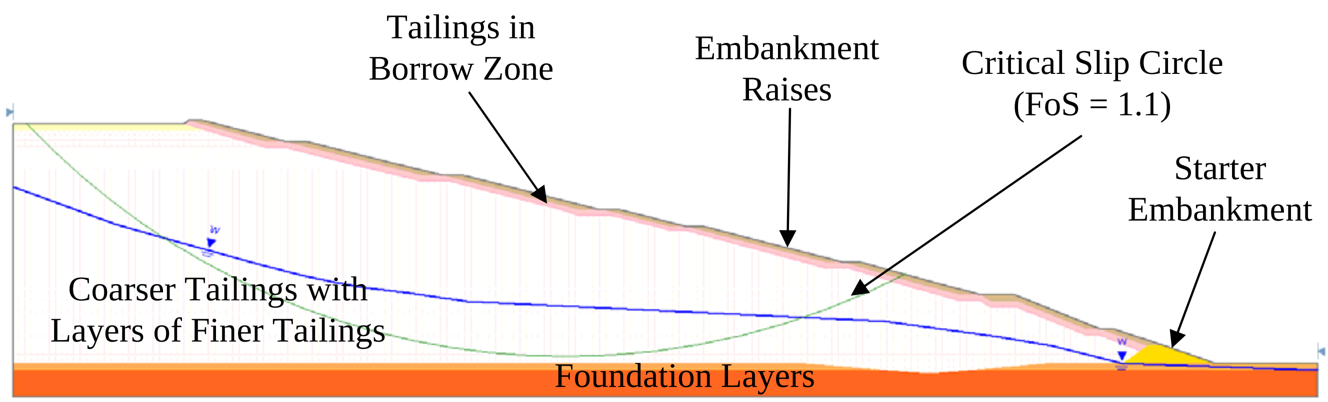
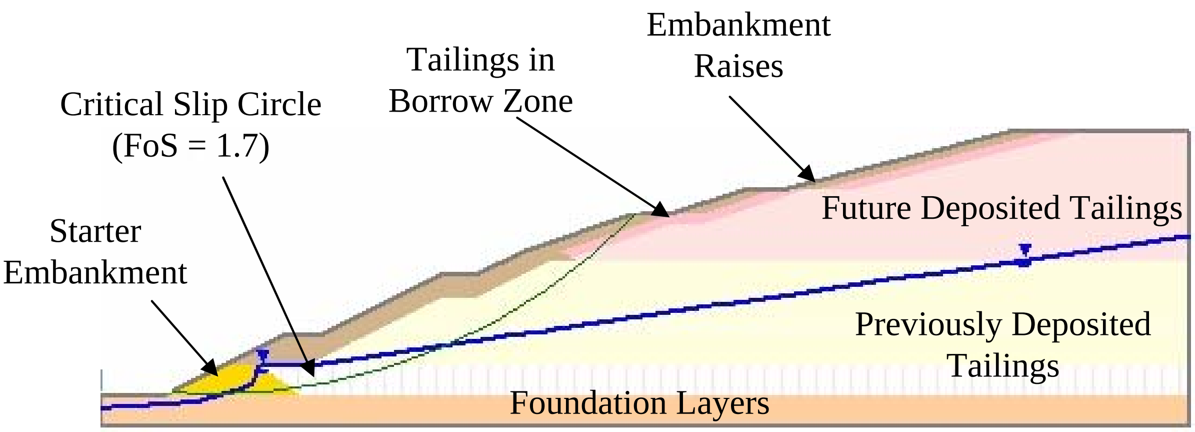
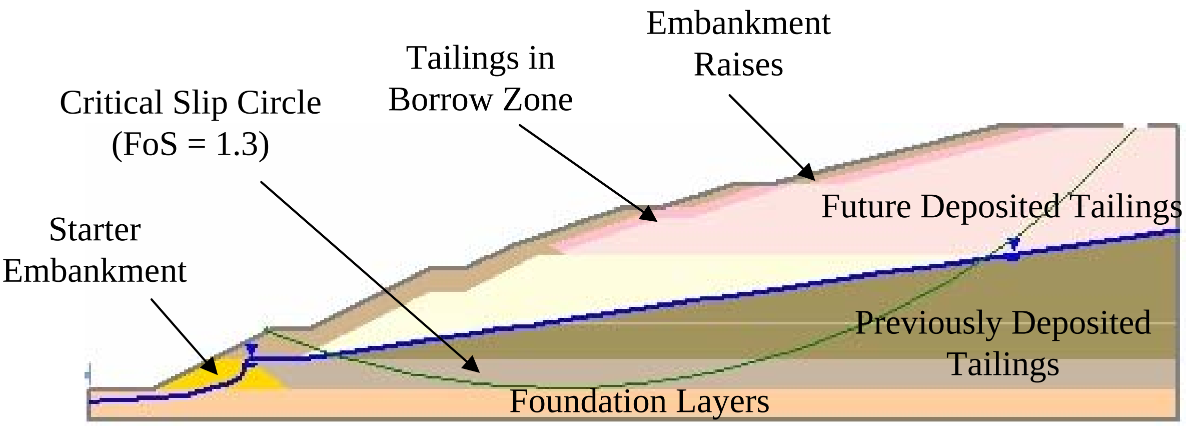


Section E under MDE Conditions

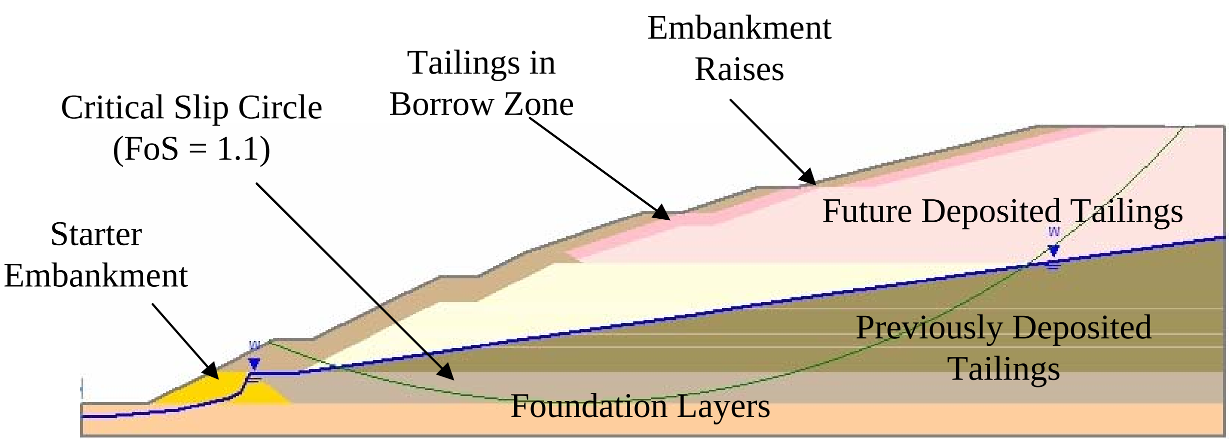
Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE Mar. 06	TITLE **FIMISTON II SECTION E**	
	CHECKED	DATE 03/05/06		
	SCALE Not To Scale	A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 9**



Section A under Static Conditions



Section A under OBE Conditions



Section A under MCE Conditions

Golder Associates	CLIENT KCGM		PROJECT Life of Mine Study – TSF Stability Assessment	
	DRAWN PJC	DATE May 06	TITLE **KALTAILS TSF SLOPE STABILITY**	
	CHECKED	DATE May 06		
	SCALE Not To Scale	A4	PROJECT No **05641521-R02**	FIGURE No. **FIGURE 10**

APPENDIX A

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL ENGINEERING REPORT

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfil the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. *No one except you* should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one – not even you* – should apply the report for any purpose or project except the one originally contemplated.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include : the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, *do not rely on a geotechnical engineering report* that was :

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical change that can erode the reliability of an existing geotechnical engineering report include those that affect :

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes – even minor ones – and request an assessment of their impact. *Geotechnical Engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by : the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions *only* at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgement to render an *opinion* about subsurface conditions throughout the site. Actual subsurface conditions may differ – sometimes significantly – from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final*, because geotechnical engineers develop them principally from judgement and opinion. Geotechnical engineers can finalise their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for* *the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognise that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognise that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce such risks, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labelled "limitations", many of these provisions indicate where geotechnical engineers responsibilities begin and end, to help others recognise their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Rely on Your Geotechnical Engineer for Additional Assistance

Membership in ASFE exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with your ASFE member geotechnical engineer for more information.

ASFE PROFESSIONAL FIRMS PRACTICING IN THE GEOSCIENCES
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