Salt Lake County
Tailings Impoundment Study (Draft)

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1.0 INTRODUCTION

Tetra Tech was commissioned by Salt Lake County to provide geotechnical analyses related to the seismic stability of the South Tailings Impoundment operated by Kennecott Utah Copper Company (KUCC). This study represents an independent evaluation of the seismic stability of the facility and potential hazard to public safety due to a possible earthquake induced failure.

During the course of the study, Tetra Tech made all requests for information through the Kennecott Tailings Impoundment Committee which was formed by the Salt Lake County Council to provide general control and supervision over the study. An initial meeting with Kennecott mine personnel was arranged by the Committee to initiate data collection and to provide an overview of the project background. Numerous reports were reviewed including field investigations and geotechnical studies conducted by various consultants. A document list is provided in Table 1.1 attached. The document numbering system presented in the table is referenced throughout this report.

1.1 Project Purpose and Objectives

This report presents the results of the data review, evaluation of previous work and analysis performed and Tetra Tech's evaluation of the static and post-earthquake stability of the southern tailings impoundment (Southeast corner 1700 South to 9400 West). The work included liquefaction and post-earthquake stability analyses. The primary objective of the study is to provide an impartial and independent evaluation of the seismic stability of the south tailings facility. The key tasks included:

- A comprehensive review of all available data relevant to the stability of the southern tailings dam
- Preparation of a data summary
- Evaluation of the data adequacy to support the required geotechnical analyses
- Stability analyses for design earthquake conditions
- Review of previous runout analyses
- Conclusions and recommendations

1.2 Project Background

The tailings impoundment is located 10 miles west of Salt Lake City, Utah near the town of Magna. The tailings south tailings impoundment was constructed between 1906 and 2001 when tailings deposition was moved to the north impoundment. Since that time, the south tailings impoundment has undergone a series of seismic upgrades based on extensive geotechnical investigations initiated in about 1988. The tailings impoundment was constructed using upstream method tailings deposition. A "starter" dam was constructed and tailings deposited hydraulically to form a tailings pile with relatively steep outer slopes. The operational sequencing of tailings deposition resulted in inter-layering of material with varying particle size characteristics and associated geomechanical properties.

1.2.1 Seismic Hazard

A comprehensive site seismic hazard evaluation is beyond the scope of this study. Previous studies were used to provide earthquake magnitude and resulting peak ground accelerations (PGA) for the impoundment. The seismic loading conditions considered for these studies reflect the most recent hazard assessment for the Maximum Credible Earthquake (MCE) conditions, which are reported to be a PGA of 0.38g resulting from an earthquake magnitude of 7.25.

1.2.2 Design Criteria

According to Utah dam safety regulations, tailings impoundments are required to meet or exceed a post-earthquake factor of safety of 1.2 for MCE conditions. However, a factor of safety of 1.0 or above is required provided that acceptable deformations are predicted using rigorous dynamic deformation modeling. Acceptable deformations in this case are assumed to be limited to the KUCC property, thereby limiting the threat to public health and safety or damage to public property.

Runout analyses have been performed by several consultants including Woodward Clyde (Document #12), AGRA (Document #15) and URS (Document #21). These methods are not considered "rigorous dynamic deformation modeling" and continue to predict post-earthquake runouts past the KUCC property boundary. However a minimum factor of safety of 1.0 has been reported as the basis for acceptable performance of the structure. Given the historical and ongoing collection of site-specific data for the southeast corner, we feel that a post-earthquake factor of safety of 1.0 for MCE conditions is appropriate; assuming continued monitoring and additional data collection is performed to provide a high level of understanding of the site conditions.

1.2.3 Geotechnical Studies

The risk of liquefaction-induced flow failure of the tailings material was identified in 1988 by KUCC and its consultants (Document #1). Extensive geotechnical studies have been conducted since that time by various consultants focusing on the seismic stability of the south tailings embankment. A wide range of seismic upgrade options were evaluated by KUCC to mitigate this risk. Figure 1.1 presents the timeline of studies and resulting mitigation measures and Attachment A presents capsule summaries of the geotechnical study documents.

1.2.4 Mitigation and Safety Measures

A "tailings modernization program" was implemented by KUCC and included a number of seismic upgrades for the south tailings impoundment, including the following:

- 315 horizontal drains installed at the toe of the southeast embankment (1989-90)
- Installation of wick drains at the crest (1989-90)
- Step back dike constructed along the southeast corner (1991)
- Installation and operation of vertical dewatering wells (1991)

A number of safety measures were implemented to reduce the risk of potential impact to third parties (adjacent Highway 201, telephone utilities, Magna golf course, Meadow Green Estates residential subdivision) in the event of a liquefaction-induced flow slide. Figure 1.2 presents a map of the southeast corner of the impoundment and Figure 1.3 presents a map of the impoundment and the surrounding area. The implemented safety measures included:

- Construction of earthen diverter berms approximately 15-feet high near a neighborhood south of the impoundment on 80th west.
- Installation of warning signs along Highway 201 which are linked to seismic accelerometers located around the facility.
- Public notification efforts in the Magna area to inform the community and agencies of the efforts to improve the safety of the south tailings impoundment.

2.0 DATA REVIEW

Tetra Tech completed a data adequacy review for the Kennecott Utah Copper Company's (KUCC) South Tailings Impoundment. A summary of the documents and data reviewed is presented herein, with an evaluation of the quality of the data and adequacy for detailed seismic stability analyses of the southwest corner of the impoundment. In general, we have found the data to be adequate in terms of quantity and quality to allow the study to be completed. Recommendations for additional data collection to improve the level of confidence of study results are given in Section 5.0.

2.1 Field Data

Numerous reports were reviewed including field investigations conducted on the Kennecott tailings impoundment. Table 2.1 summarizes the types of field investigations conducted for each of the reports reviewed and Table 2.2 presents a detailed summary of field testing and instrumentation installation locations.

2.1.1 Cone Penetrometer Testing

Cone penetrometer testing (CPT) has been the primary method of investigation of the tailings mass. CPT investigations date back approximately 20 years at the site with the most recent data related to the southwest impoundment area collected in 2005.

This method involves pushing a probe into the tailings mass at a controlled rate to measure tip resistance, sleeve friction, and dynamic pore pressure. By using standard engineering correlations, the geotechnical properties of stratigraphic layers can be inferred. This method of investigation is considered appropriate for providing useful data for evaluating the seismic stability of the impoundment.

Several methods of CPT were employed during field investigations at the southwest impoundment area by several contractors, including standard CPT (CPTU), seismic methods (SCPTU), and resistivity methods (RCPTU). Shear wave velocities were measured by SCPTU tests and soil resistivity was measured by RCPTU tests. Occasionally, pore pressure dissipation (PPD) tests were conducted, allowing piezometric pressure to be calculated. Table 2.3 summarizes the various CPT test methods utilized during each field investigation reviewed. Table 2.4 summarizes the number of CPT locations and pore pressure dissipation (PPD) tests conducted.

The CPT is one of the most used and accepted in situ test methods for soil investigation worldwide. CPT for geotechnical applications was standardized in 1986 by ASTM Standard D 3441. The International Society of Soil Mechanics and Geotechnical Engineering (ISSMGE) provides international standards on CPT and CPTU.

The information gleaned from the digital CPT data and PPD tests will directly aid in determining the liquefaction potential of the tailings mass and aid in verifying results of previous studies. Digital log files and raw data for the tests performed to support the most recent studies ("Dewatering and Seismic Stability Evaluation, URS, 2006) will be used to perform liquefaction susceptibility evaluations.

2.1.2 Standard Penetration Tests

The Standard Penetration Test (SPT) aims to determine the SPT N value, which gives an indication of the soil stiffness and can be empirically related to many engineering properties. The SPT was conducted at several locations in the study area. Generally the tests were conducted in conjunction with the installation of stand-pipe piezometers.

SPT blow count values are available for each of the referenced borings. In some instances, the consultants reduced the data to standardized blow count ($N_{1,60}$). SPT blow count data and associated fines contents (if available) can be used to aid in analysis of liquefaction potential and corroborate the liquefaction potential determined from CPT data.

2.1.3 Pressuremeter Tests

Woodward-Clyde (1991) conducted pressuremeter tests to determine "stress, deformation, and strength properties of various foundation soil units under their ambient site conditions." Self-boring and Menard pressuremeters were utilized.

The self-boring pressuremeter test method minimizes hole disturbance by supporting the hole with the body of the instrument during insertion. The pressuremeter is inserted using a jetting system that is described in detail in the referenced report. Menard tests require pre-drilling the borehole and then lowering the pressuremeter to the required depth. A detailed description of the process is provided in the referenced document.

A total of eight pressuremeter tests were conducted in three borings. Two self-boring pressuremeter tests were conducted in boring DH-WC-300. Six Menard pressuremeter tests were conducted in borings DH-WC-113 and DH-WC-300.

2.2 Laboratory Data

A spreadsheet database has been created to summarize the most relevant laboratory testing results for the study and is presented as Table 2.5.

2.2.1 Sampling

Sampling methods included bulk samples, SPTs, Shelby tubes, and "fixed piston" samplers. Bulk samples were used for index testing or reconstituted (Document # 9). Shelby tubes were used to obtain "undisturbed" samples for strength and consolidation tests. Thin-walled piston samples were used for microstratigraphy studies and laboratory testing (Document # 4).

2.2.2 Index Tests

Grain size analyses, along with Atterberg Limits and hydrometer analyses are used to classify the materials (for example silt, clay, sand, etc.) A total of at over 100 grain size analyses were conducted on the tailings and foundation materials at various locations. The fines content obtained in the grain size analyses is used to model susceptibility of the materials to liquefaction.

2.2.3 Strength Tests

Unconsolidated-Undrained (UU) tests were performed on “new tailings grind” to determine its steady state strength over a range of void ratios (Document # 1). The data was used to estimate the minimum shear strength that the tailings would exhibit at a given void ratio during undrained loading (earthquake loading), and ultimately evaluating the extent of the liquefied zone necessary to cause failure of the tailings slope. However, because of difficulties encountered during testing and interpretation of the results, the steady state strength determined from these tests was not used for analysis.

UU tests were also performed on foundation clay collected from beneath the impoundment to check the results of the Consolidated-Undrained test on the clay samples taken at the toe of the impoundment and consolidated to an effective pressure that would exist under the embankment.

UU tests were performed on four “crest” (under embankment) samples and three “free field” samples (Document #4). Undrained strength (S_u) was plotted versus elevation and compared with shear strength data from CPT and pressuremeter tests. Pocket penetrometer and torvane testing were also conducted, but these were considered to be less accurate measures of the undrained shear strength of the materials and the results were omitted from the report.

Undrained triaxial tests on partially saturated, reconstituted samples of Southeast Corner tailings were conducted to characterize their shear strength, cyclic behavior and post-earthquake shear strength (Document # 9).

Consolidated undrained (CU) tests were performed on four samples of foundation clay to determine the effective angle of shearing resistance and to evaluate the undrained strength ratio C_u/p' that would be representative of normally consolidated clay existing beneath the final impoundment (Document # 4). One CU test was performed on a sample of the tailings clay recovered at the Pond section to confirm its properties during shear.

Isotropically consolidated CU (CIU) were conducted on undisturbed samples and reconstituted and saturated specimens of Southeast Corner tailings (Document # 9). Anisotropicially consolidated CU (CAU) were conducted on reconstituted saturated samples to characterize the shear strength, cyclic behavior and post-earthquake shear strength of saturated and partially saturated tailings. The CU test on the tailings correlated with the undrained fiction angles determined by previous investigators. The main focus of this testing was to characterize the behavior of partially saturated (dewatered) SE corner tailings during cyclic loading; and the post-cyclic shear strength of partially saturated tailings relative to saturated tailings.

Cyclical shear tests were conducted on tailings to investigate the response of the cohesive tailings to dynamic loading and to evaluate the effect of anisotropic consolidation on the cyclic resistance of the tailings (Document # 1). The anisotropicially consolidated samples were used to evaluate the effect of sloping ground on cyclic resistance. The tests were used to qualitatively evaluate trends rather than to determine absolute values of dynamic resistance.

Oedometer consolidation tests were performed on the foundation clay (beyond the toe of the impoundment) to evaluate consolidation characteristics under the impoundment. The results were used to predict the void ratio of the clay and to evaluate its dynamic properties (Document # 1). Consolidation tests were performed on the foundation clay (four under the tailings crest, and two in the “free field”) to define the stress history and compressibility characteristics of these soils (Document # 4).

Consolidation tests were also performed on six undisturbed tailings samples from Southeast corner - four performed to gain insight into the stress history (maximum past stress) and compressibility characteristics and two as confirmatory tests for pressuremeter tests within the "STC" layer ("Southeast Tailings Clay" – one of three "idealized material types" referred to in the report). These tests involved standard, incrementally loaded testing in general accordance with ASTM D 2435-80 (Document # 9).

2.3 Montoring Data

Tetra Tech received approximately 8.5 years of vibrating wire piezometer data from KUCC. The period of available monitoring data extends from approximately July 1999 to January 2009. A review of the information from 40 piezometers indicates that phreatic levels within the tailings impoundment have been decreasing consistently over the recorded period of monitoring. These data may be used to forecast future phreatic levels and aid in modeling slope stability and tailings liquefaction potential.

3.0 LIQUEFACTION ANALYSIS

The liquefaction potential of the tailings mass was evaluated using recently published, state-of-practice liquefaction analysis methods (Idriss and Boulanger, 2008) and the most recent CPT data available (ConeTec, 2005). Additional material properties, including unit weight and fines contents, were selected from laboratory test data summarized in various reports as presented in Section 2.0. The liquefaction analysis methodology, tailings materials properties, site seismic conditions, phreatic conditions, and pertinent model assumptions are summarized in the following sections.

3.1 Methodology

The liquefaction analysis methodology is presented in detail in the referenced text by Idriss and Boulanger (2008). This methodology consolidates the most recent advances in liquefaction analysis into a single method and represents the current state-of-the-practice for the evaluation of liquefaction.

The liquefaction analysis was conducted based on recent CPT data (ConeTec, 2005) collected for the southeast corner of the Kennecott tailings facility. Liquefaction resistance of the tailings was calculated as a function of the site seismicity, CPT tip resistance, CPT sleeve friction, CPT resistivity measurements, and tailings material properties. Residual shear strength of liquefied tailings was also calculated using the aforementioned data.

3.2 Liquefaction Analysis Parameters

The parameters and material properties used in the liquefaction analysis are summarized in the Table 3.1. Additional information, where necessary, is provided in the following sections.

Table 3.1: Liquefaction Analysis Parameter Summary

Parameter	Value	Source
Tailings unit weight (above water table)	17.3 kN/m^3	Based on data from Document #1, 11, 15, and 16a
Tailings unit weight (below water table)	18.5 kN/m^3	Based on data from Document #1, 11, 15, and 16a
Tailings fines content	Variable. Based on a generalized section created from laboratory test data.	Based on data from Documents #1, 4, 6, and 9
Tailings degree of saturation	Value based on relationship developed by ConeTec.	ConeTec, 1997
Water table depth	Variable	ConeTec pore pressure dissipation tests and field monitoring data collected from piezometers

Design earthquake peak ground acceleration	0.38g	URS, 2006
Design earthquake magnitude	7.25	URS, 2000

3.3 Tailings Fines Content

Previous methodologies of liquefaction analysis used by URS and others did not take into account the effect of fines content (percent passing the No. 200 sieve) on the liquefaction resistance of a soil mass. The current state of the practice accounts for the effect of fines content; therefore, a generalized section of fines contents versus depth was generated using available laboratory test data from a series of reports. A graphical example of the fines content data reduction is presented in Figure 3.1.

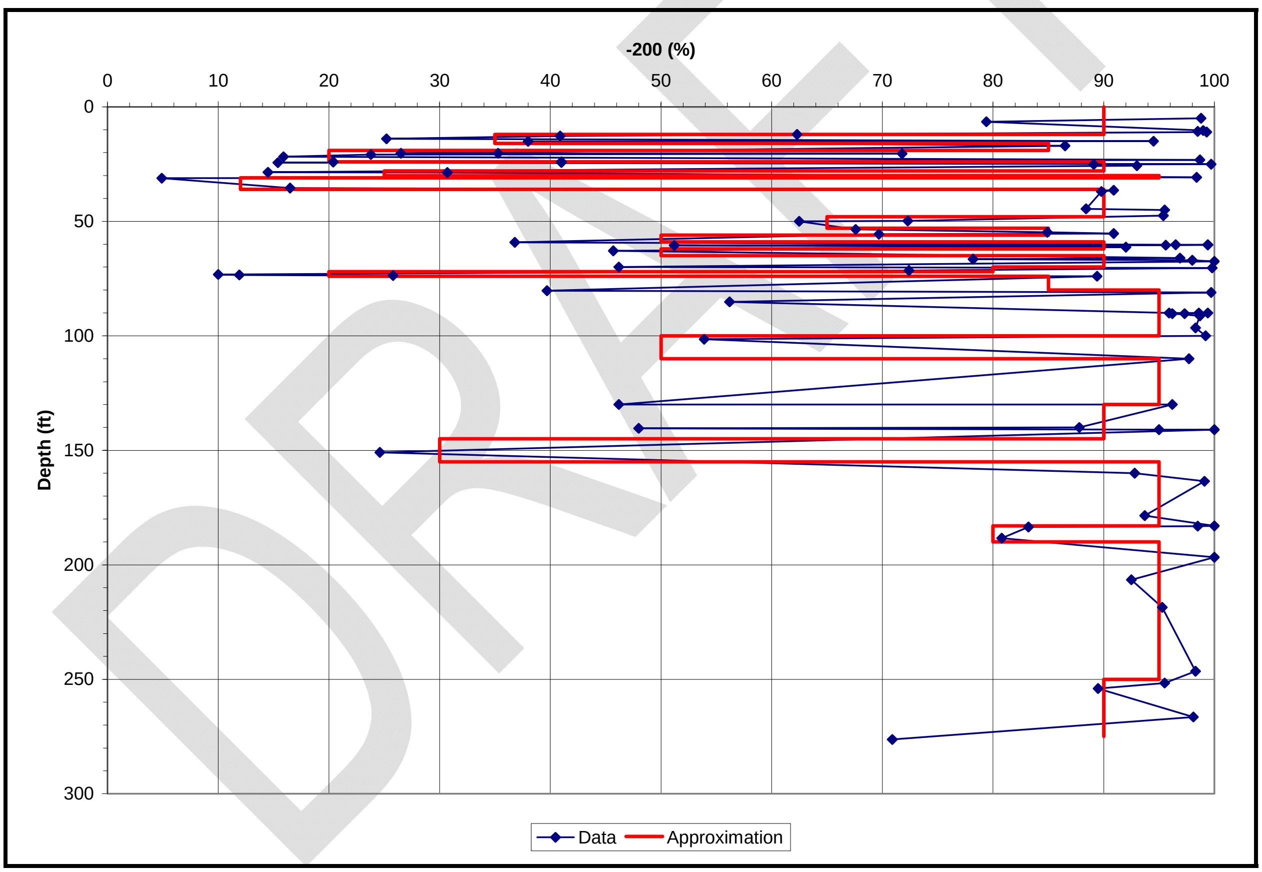

Figure 3.1 – Example of CPT Data Reduction Results

The fines content profile was applied to each section with a depth of zero equating to the ground surface at each CPT location. A limited parametric analysis was conducted to allow some observation of the sensitivity of fines content to the results. The analysis indicates that the delineation of the zone of liquefied tailings is somewhat sensitive to changes in fines content, and the post-liquefaction shear strength of the liquefied tailings is strongly influenced by tailings fines content. The estimated shear strength of a material with a fines content of 10 percent is approximately 30 percent less than that of a material with a fines content of 90 percent.

Future liquefaction analyses may take into account the segregating effects of tailings deposition and apply differing fines content profiles depending on the CPT location. These analyses will require that soil samples be collected at each CPT location in order to generate unique fines content profiles. As an alternative to such a study, a detailed parametric analysis may be conducted to determine the effect of fines content on liquefaction potential and associated post-liquefaction shear strength of the tailings within the tailings mass.

3.4 Degree of Saturation

The degree of saturation of the tailings was calculated using a unique relationship for the Kennecott tailings developed by ConeTec based on field and laboratory testing (ConeTec, 1997). A correlation for calculating the degree of saturation from resistivity measurements was then developed based on controlled laboratory testing of the tailings at various moisture contents for calibration of field resistivity measurements.

Previous liquefaction studies have assumed that tailings saturation level of 85 percent or greater would results in potentially liquefiable conditions. References supporting this saturation cut-off value are absent from the reviewed text. This assumption is considered reasonable based on published studies. The liquefaction resistance of partially saturated sand indicates that liquefaction resistance decreases rapidly once saturation exceeds 70 percent (Yoshimi et al., 1989).

A parametric analysis was used to evaluate the effect of the saturation on liquefaction resistance. Liquefaction analyses were conducted on each CPT data file using tailings saturation levels of 70 and 85 percent to evaluate the sensitivity of this parameter to liquefaction potential.

3.5 Liquefaction Analysis Results

The available CPT data was analyzed to determine discrete intervals of potential liquefaction under the design earthquake and calculate the approximate post-earthquake shear strength of the liquefied tailings mass. A summary of potentially liquefied tailings are summarized in Table 3.2.

Post-earthquake shear strength of liquefied tailings ranging from 0.11 to 0.35 (S/P) were calculated with an average value was 0.16. A conservative value of 0.13 was selected for use in the post-earthquake stability analyses. This value is similar to the values of 0.12 used by URS in previous analyses.

Table 3.2: Liquefaction Analysis Results Summary

CPT Location	Total CPT Depth (m)	URS Modeled Thickness of Liquefiable Tailings (m)	85% Saturation Cut-Off		70% Saturation Cut-Off	
			Total Thickness of Liquefiable Tailings (m)	Modeled Thickness of Liquefiable Tailings (m)	Total Thickness of Liquefiable Tailings (m)	Modeled Thickness of Liquefiable Tailings (m)
CPT-06	54.90	21.03	25.75	33.0	34.55	33.0
CPT-07	58.15	16.01	16.80	10.0	21.50	17.0
CPT-08	18.45	5.62	4.95	3.5	7.15	6.0
CPT-12	45.75	31.56	33.65	40.0	42.25	41.5
CPT-13	30.75	15.33	19.70	20.5	22.60	22.0
CPT-14a	19.75	8.79	12.40	11.0	15.10	15.0
CPT-17	51.85	7.99	37.10	40.5	46.70	46.0
CPT-18	49.70	13.74	26.55	27.0	39.65	39.0
CPT-19	7.40	5.83	2.65	3.0	5.25	6.0
CPT-21	34.45	5.27	10.05	6.0	21.15	9.0

The variances between the total thickness of liquefied tailings and the modeled thickness of liquefied tailings are a function of engineering judgment based on a review of the available data. The variances between the thicknesses of liquefied tailings for the two saturation cut-off values are largely influenced by interlayered zones of liquefiable and non-liquefiable tailings at the 85 percent cut-off that tend to completely liquefy once the cut-off value is reduced to 70 percent. Effectively, the zone of liquefied tailings modeled in the stability analyses remains unchanged regardless of which saturation cut-off value is used, indicating that sensitivity to saturation level is low and would not significantly affect post-earthquake stability results.

URS (2006) delineated zones of potential liquefaction using two criteria: cone penetrometer tip resistance and the degree of saturation of the tailings. The criteria selected required tip resistance to be less than 50 tons per square foot (tsf) and saturation to exceed 85 percent in order to result in liquefaction. If an investigated interval failed to meet one of the criteria then the interval was deemed ‘non-liquefiable.’ The updated liquefaction analysis does not exclude an interval from liquefaction based solely on tip resistance.

In most cases, the calculated zones of liquefaction correlated well with those delineated by URS. Variances are attributed to the following:

- Liquefaction of layers delineated by URS as ‘non-liquefiable’ due to measured CPT tip resistances in excess of 50 tsf.

- Liquefaction of discrete layers with adequate saturation located above the 85 percent saturation zone delineated by URS.
- The more refined and intensive liquefaction calculation outlined in the recently published liquefaction analysis method (Idriss and Boulanger, 2008).

Section SE-2

Four CPT locations are associated with section SE-2: CPT-06, CPT-07, CPT-08, and CPT-12. Our analysis results in liquefaction thicknesses ranging from approximately 88 to 122 percent of the thicknesses calculated by URS. With the exception of CPT-06, the zones of liquefaction calculated correlate well with those calculated by URS. The large variance at CPT-06, located within the impoundment, resulted from an increase in the depth of tailings liquefaction calculated into a zone delineated by URS as ‘non-liquefiable’ due to CPT tip resistances in excess of 50 tsf.

Section SE-3

Four CPT locations are associated with section SE-3: CPT-06, CPT-12, CPT-13, and CPT-14. Our analysis results in liquefaction thicknesses ranging from approximately 87 to 129 percent of the thicknesses calculated by URS. The increased liquefaction thicknesses calculated for CPT locations CPT-06, CPT-12, and CPT-13 resulted from an increase in the depth of tailings liquefaction calculated into a zone delineated by URS as ‘non-liquefiable’ due to CPT tip resistances in excess of 50 tsf. The zone of liquefaction calculated for CPT-14 correlated well with the zone delineated by URS.

4.0 STABILITY ANALYSES

This section summarizes the post-earthquake stability analyses that have been performed for the study. Material properties were determined based on a review of previous studies with modifications based on results of the liquefaction evaluation discussed in Section 3.0. The study sections referred to as SE2 and SE3 were selected for these analyses (see Figure 1.1). Study Section SE2 is oriented north-south and is located near the southeast corner has been used to evaluate the progress of the dewatering efforts and resulting improvement in seismic stability and potential runout distance. Data from regular monitoring of ten piezometers at three verticle profiles along the SE2 study section was reviewed. Study Section SE3 is oriented east-west and is located near the southeast corner has been monitored with five piezoemters.

4.1 Methodology

Slope stability analyses were performed using the Slope/W component of GeoStudio 2007 (version 7.14) by Geo-Slope International, Ltd. The analyses utilized limiting equilibrium analyses using both the Spencer and general limit equilibrium (GLE) methods, which satisfy both force and moment equilibrium. The Spencer method was used mainly to duplicate and verify the results from the 2006 URS report, and the GLE method was used as a more rigorous approach that allows for a range of interslice shear-normal force conditions.

To determine the position of the lowest factor of safety, a fully specified, entry and exit, block specified and auto search slip surfaces were used. The fully specified slip surface was used to reproduce the work done by URS in the 2006 report. The entry and exit slip surface defines a range of possible locations to search for the most critical failure surface. The block specified slip surface was mainly used in flat side slopes with a potential mode of sliding. The auto search method combines the entry and exit method along with an optimization procedure in a series of trial analyses based on the geometry.

Previous studies have estimated the location of the phreatic surface based on CPT dissipation testing, field resistivity measurements and piezometer monitoring data. The results of these studies indicate the presence of a downward seepage gradient in the tailings mass resulting in pore pressures less than hydrostatic below the interpreted phreatic level. The methods used provide a reasonable Phreatic conditions were modeled based on measured pore pressures from piezometer monitoring data for 2005 conditions and 2008 conditions.

4.2 Material Properties

The material properties represent post-liquefaction conditions and are shown in Table 4.1. The interval of liquefaction susceptible tailings was defined based on the evaluation discussed in Section 3.0. Average values between direct simple shear and triaxial compression properties were used in the analyses.

Table 4.1 - Post Liquefaction Material Properties

Material	Total Unit Weight (pcf)	Shear Strength	
		Direct Simple Shear	Triaxial Compression
Unsaturated whole tailings spigotted tailings	117	ϕ =34°, c = 0	
Liquefiable whole spigotted tailings	119	$S_{Ur}/s_{v,c}$ = 0.13	
Saturated whole spigotted tailings	119	$C_U/s_{v,c}$ = 0.8 * 0.23	$C_U/s_{v,c}$ = 0.8 * 0.35
Unsaturated soft tailings clay	107	ϕ =28°, c = 0	
Saturated soft tailings clay	107	$C_U/s_{v,c}$ = 0.23 $(OCR)^{0.8}$ Estimated OCR = 1.5	$C_U/s_{v,c}$ = 0.35 $(OCR)^{0.8}$ Estimated OCR = 1.5
Liquefiable deep whole tailings (zone 1 and 2)	116	$S_{Ur}/s_{v,c}$ = 0.13	
Saturated deep whole tailings (zone 1)	116	$C_U/s_{v,c}$ = 0.23 $(OCR)^{0.8}$ Estimated OCR = 1.0	$C_U/s_{v,c}$ = 0.35 $(OCR)^{0.8}$ Estimated OCR = 1.0
Saturated deep whole tailings (zone 2)	116	$C_U/s_{v,c}$ = 0.23 $(OCR)^{0.8}$ Estimated OCR = 1.5	$C_U/s_{v,c}$ = 0.35 $(OCR)^{0.8}$ Estimated OCR = 1.5
Dikes (starter, 1950 and 1952)	115	ϕ =35°, c = 0	
Upper Bonneville clay	118	$S_{Ur}/s_{v,c}$ = 0.208; Min S_{ur} = 500 psf	
Upper interbedded sediments	129	$S_{Ur}/s_{v,c}$ = 0.208; Min S_{ur} = 500 psf	

4.3 Results

The following analyses were performed using the GLE method and the fully specified failure surfaces matching previous results by URS. Additionally, the auto-search function was utilized to identify the most critical failure surfaces. Figures presenting the results from this analyses and previous analyses are contained in Attachment A.

4.3.1 2005 Conditions

These analyses reflect 2005 phreatic conditions as reported by URS, with modified liquefiable zones and post-earthquake material strengths. Table 4.2 presents a summary of the results.

Table 4.2 - Slope Stability Results Summary for 2005 Conditions

Section	Failure Surface	Factor of Safety		
		URS (2006)	Tetra Tech (fully specified)	Tetra Tech (auto-search)
SE2	A	1.95	1.73	
	B	1.75	1.81	
	C	0.80	0.75	0.68 (1)
	D	0.71	0.60	
SE3	A	1.10	1.00	
	B	0.86	0.90	0.87 (2)
	C	0.74	1.07	

1) Failure mass size is somewhat smaller than URS failure surface SE2-C

2) Failure mass size is somewhat smaller than URS failure surface SE3-B

The results indicate good agreement with previous analyses performed by URS for study section SE2. It should be noted that although the results for fully-specified failure surfaces indicate higher factors of safety for study section SE3, this is likely due to the modified liquefaction zones and material properties causing a shift in the location of the most critical (lowest factor of safety) failure surfaces. The auto-search results reflect the most critical failure surfaces, as the analyses do not force a failure surface through a user-defined path.

4.3.2 2008 Conditions

These analyses reflect 2008 phreatic conditions as interpreted by Tetra Tech, with modified liquefiable zones and post-earthquake material strengths. All analyses reflect December 2008 phreatic and pore pressure conditions taken from the piezometer monitoring data. Phreatic surfaces for each section were estimated based on historical data including CPT pore pressure dissipation tests, field resistivity testing and piezometer monitoring. Pore pressure profiles were evaluated and adjusted to reflect a linear best fit line below the interpreted phreatic surface.

Using this method for study section SE2, pore pressure profiles ranged from 95% of hydrostatic near the slope toe to 54% of hydrostatic for the series of pizometers located approximately 1600 feet north of the toe. For study section SE3, pore pressure profiles ranged from 29% to 81% of hydrostatic. For this analysis it was assumed that the tailings above the interpreted phreatic line would generally not be prone to liquefaction, therefore zone of liquefiable tailings was also adjusted to reflect the expected reduction of saturation levels associated with dewatering. Table 4.3 presents a summary of the results.

Table 4.3 - Slope Stability Results Summary for 2008 Conditions

Section	Failure Surface	Factor of Safety		
		URS (2006)	Tetra Tech (fully specified)	Tetra Tech (auto-search)
SE2	B	1.75	2.09	
	C	1.07	1.07	0.93 (1)
	D	0.82	0.82	
SE3	A	NA	1.05	
	B	NA	0.98	0.79 (2)
	C	NA	0.97	

1) Failure mass size is somewhat smaller than URS failure surface SE2-C and deeper seated.

2) Failure mass size is significantly smaller than URS failure surface SE3-B, but larger that SE3-C from 2005 analyses.

The results indicate very good agreement with previous analyses performed by URS for study section SE2. The results also indicate significant improvement of the seismic performance of the embankment since 2005. However, factors of safety remain below 1.0 for full height failure surfaces.

4.3.3 Target Phreatic Levels

Stability analyses were performed to determine a target phreatic level for achieving the seismic performance desired with factors of safety of greater than 1.0 for each study section. For study section SE2, the phreatic surface was lowered by about 5 below current conditions in the area upslope of the 1950 dike. For study section SE3, the phreatic surface was lowered by approximately feet 35 below current levels in the area upslope of the 1950 dike.

The results indicate acceptable factors of safety of 1.0 or above can be achieved with continued dewatering and reduction of the phreatic levels. Pore pressure trends were reviewed based on available piezometer monitoring data. For study section SE2, pore pressure dissipation trends have leveled off and indicate a gradual future reduction of less than 0.4 psi per year. For study section SE3, piezometric trends since 2002 indicate an average reduction in pore pressure of 0.75 psi per year. This would equate to phreatic level reduction of less than 14 inches per year for SE2 and 21 inches per year for SE3, assuming steady state conditions. Although the performance of the dewatering systems is difficult to predict, the target phreatic levels indicated to achieve the desired performance may be attainable with continued operation of the current systems however the time required to reach these level is unclear. Based on current trends, the target phreatic levels may be achieved in approximately 7 years and 20 years respectively for study sections SE2 and SE3 respectively.

4.3.4 Implications to Runout Estimates

A detailed runout analyses is beyond the scope of this study, however the implications of the results of the stability analyses can be judged based on a review of previous estimates. Estimates of slope runout during post-earthquake failure have been developed by URS and others. Original studies estimated runout lengths of up to 690 feet in the area of study section SE2 (Document #12) using Dynamic Run-out Method (DRUM), which is a momentum based analysis. This estimate has been revised over time by URS with the most recent estimate performed in 2008 (Document #21) resulting in 190 feet of runout at section SE2 using similar methodology.

Since similar factors of safety were found for study section SE2 compared to the most recent runout analysis, a similar runout distance would result assuming the same methodology is applied. Applying this runout estimation methodology, and assuming further reduction in phreatic levels, a gradual reduction in the total runout distance can be expected. If the target phreatic level is reached in the area of study section SE2, resulting in factors of safety 1.0 or higher, minimal to no runout can be expected, although crest and slope deformations may occur with possible blockage of the clarification canal.

It should be noted that the runout distance is sensitive to the selection of the critical failure surface using the DRUM analysis. In previous studies (Document #12), the analysis considered surfaces with post-earthquake factors of safety of up to 1.0. However, in the most recent estimate (Document #21), factors of safety of greater than 0.85 were not considered.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The seismic upgrade goal set by KUCC is improvement of the factors of safety against MCE post-earthquake failure to above 1.0 by the year 2018. The measures instituted to achieve this goal have consisted of installation of horizontal drains and wick drains, slope flattening by construction of step back dikes, and operation of active dewatering by pumping of vertical dewatering wells. These measures have resulted in gradual lowering of the phreatic surface thereby improving the stability of the embankment and tailings pile.

Upon review of the previous studies and available data, we have found the methods used to collect data to be appropriate and generally meet industry standards. Previous analyses performed to predict the post-earthquake stability of the southeast corner of the facility were also found be appropriate in methodology and appear to be based on a generally high level of understanding of the site conditions and application of reasonable engineering judgment.

The liquefaction analysis was updated to reflect the state-of-practice methods resulting in a revised delineation of the liquefiable tailings zone. The methods used are considered slightly more conservative and resulted in a thicker liquefaction zone in some areas of the study sections. However, this did not result in significant reduction of the factors of safety against post-earthquake failure.

The study results indicate good agreement with previous studies for post-earthquake stability of the southeast corner of the embankment. Factors of safety against post-earthquake failure have gradually improved with lower phreatic levels and should be expected to continue to improve at a gradual rate assuming current dewatering trends continue.

A target phreatic level was estimated for study section SE2 to achieve the performance goal of post-earthquake factor of safety of 1.0 or greater. The analyses indicate that a drop in phreatic levels of approximately 5 feet and 35 feet below current conditions for study sections SE2 and SE3, respectively will result in sufficient stability improvement to meet the stated performance goal. The time required to reach these phreatic levels is difficult to predict given the complex seepage regime. Our best estimate based on projection of currently pore pressure dissipation trends is 7 years for SE2 and 20 years for SE3.

Seismic performance of the embankment and tailings pile is sensitive to the phreatic level; however interpretation of the phreatic surface based on piezometer data is difficult due to the apparent variable downward gradient in the tailings mass. The long-term performance of the existing dewatering systems is difficult to predict due to the complex nature of the seepage regime. Although seepage modeling can be performed to provide estimates of future phreatic conditions and associated seismic performance of the southeast corner of the facility, such a model would be very complex and would not predict future behavior with a high level of confidence. Therefore, the observational approach taken by KUCC and its consultants is appropriate, assuming additional data is collected as needed through field investigations or installation of additional instrumentation at strategic locations. Future field investigations should include CPT with sampling of the tailings at regular depths to provide useful data regarding level of saturation. Results from pore pressure dissipation testing should be correlated with nearby piezometer readings to provide a more accurate determination of the phreatic level. Tetra Tech concurs with URS's recommendation of performing additional field investigations and/or instrumentation installations to verify the location of the phreatic surfaces used in the analyses.

The calculated runout distance using the DRUM methodology is sensitive to the failure mass size which is dictated by selection of the minimum factor of safety for mobilization of the mass. The most recent URS runout analysis did not consider failure surfaces with factors of safety above 0.85, while previous analyses considered surfaces with factors of safety up to 1.0.

Assuming the flow failure is retrogressive, a smaller failure mass corresponding to a factor of safety of 0.85 may represent the initial flow event, however this initial event may be followed by progressive flow sliding resulting in a longer total runout distance than currently predicted. We do not consider this to be a flaw in previous analyses, however we consider this to be a matter of engineering judgment and recommend an independent runout analysis be performed to provide a higher level of confidence in the most likely post-earthquake runout distance and configuration of the tailings pile and its relation to public facilities.

A review of the KUCC Emergency Action Plan (EAP) for the facility was conducted. Specific safety measures implemented by KUCC include construction of earthen diverter berms to protect adjacent properties and installation of warning signs along Highway 201 which are linked to three seismic accelerometers located around the facility. In the event of an earthquake, the seismic warning system will trigger at ground accelerations above 0.01g at two of three accelerometers. The activated signs will then advise motorists to take an alternative route and warning horns will sound. In general, we found the EAP to meet industry standards and the safety measures implemented to provide an appropriate degree of risk reduction based on the potential hazard.

6.0 GENERAL INFORMATION

Recommendations presented herein are based on an evaluation of the findings of the site investigations and monitoring data noted. This report has been prepared for the exclusive use of Salt Lake County for specific application to the area within this report. Any use which a third party makes of this report, or any reliance on or decisions made based on it, are the responsibility of such third parties. Tetra Tech accepts no responsibility for damages, if any, suffered by any third party as a result of decisions made or actions based on this report. It has been prepared in accordance with generally accepted engineering practices. No other warranty, expressed or implied, is made.

7.0 REFERENCES

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