FINAL REPORT

Tailings & Water Services

Geotechnical Evaluation Summary Report for December 17, 1999 State Engineer's Meeting

December 17, 1999

Prepared for

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Project No. 6800024462

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1.1 PURPOSE AND SCOPE

The purpose of this report is to provide a summary of the Magna Impoundment stability evaluations as of December 17, 1999 for the State Engineers Meeting. Numerous geotechnical reports have been developed in the past, which are summarized in this report.

Items addressed in this summary report include the following:

- Geotechnical site characterization for the various study areas and study sections located around the Magna Tailings Impoundment (Section 2.0).
- Description of the design criteria as applied to the North Expansion and Magna tailings impoundments, based on the State of Utah Statutes and Administrative Rules for Water Retention Dams Safety and modifications agreed between KUCC and the State Engineer's Office (Section 3.0).
- Methodology for the various analyses completed to evaluate the stability of the Magna Tailings Impoundment slopes, including the two North Expansion abutments (Section 3).
- Summary of the Magna Impoundment slope stability analyses results (Section 4).
- Design revisions for the east and west abutments of the North Expansion embankment (Section 5).
- Summary of the geotechnical monitoring program and observational approach as applied to both impoundments (Section 6).
- Conclusions (Section 7).

1.2 FACILITY DESCRIPTION

The Kennecott Utah Copper Corporation (KUCC) Tailings Facility is located approximately 10 miles west of Salt Lake City, near Magna, Utah. The facility is comprised of two tailings impoundments: the Magna Tailings Impoundment, and the new North Expansion Tailings Impoundment. The two tailings impoundments, located adjacent to each other and occupying approximately 9,200 acres, are bounded by State Highway 201 and the Oquirrh Mountains on the south, Highway 202 and the Great Salt Lake on the west, and Interstate I-80 on the north. An aerial photograph of the site is shown as Figure 1.1. A general layout and site plan for the Kennecott Tailings Facility is shown on Figure 1.2.

1.2.1 Magna Impoundment

The Magna Impoundment has been in operation since the early 1900s, and is nearing its operational capacity. Its total area is about 5,700 acres and it is currently about 240 feet high. It has been built using upstream method of construction. The overall side slopes around this 90-plus year impoundment vary approximately from 5H:1V to 7H:1V.

As part of the tailings management strategy, the tailings storage operation is currently being transitioned from the Magna Impoundment to the new North Expansion Impoundment. The transition of tailings is anticipated to be complete by the end of 2004, after which the Magna Impoundment will be completely reclaimed. The reclamation plan, due to the interior surface area of the Magna Impoundment being large (approximately 3,000 acres), is subdivided into six smaller and more manageable areas, as shown on Figure 1.2. These areas will be reclaimed in a systematic and sequential manner, while tailings continue to be deposited into the unreclaimed areas. A series of reclamation dikes constructed across the surface of the impoundment will isolate and delineate each of the reclamation areas. The sequential reduction in active tailings deposition areas will limit susceptibility to wind erosion during the transition period.

The first reclamation dike was completed in the fall of 1998. Approximately 500 acres of impoundment surface were removed from active tailings deposition and reclaimed with vegetation cover. The second reclamation dike is currently being constructed and when complete, will cut-off additional 890 acres of the impoundment surface.

1.2.2 North Expansion Impoundment

The North Expansion Impoundment, located on the north side of the Magna Impoundment, has a total area of approximately 3,500 acres. The tailings within the North Expansion Impoundment will be retained by constructing an embankment along the perimeter of the impoundment. The geotechnical design of the North Expansion embankment was completed in 1995 (Woodward-Clyde 1995) to meet the design criteria established by the Utah State Engineer's office, in addition to Kennecott's requirements. The central focus of the design process was to develop an embankment capable of surviving the Maximum Credible Earthquake (MCE) event without failure of the impoundment and subsequent uncontrolled release of tailings. The North Expansion embankment will be constructed from compacted underflow sands using a center-line method of construction. The projected design geometry of the North Expansion embankment is 245 feet high, with a crest width of 100 feet and an overall side slope of 4H:1V.

The North Expansion embankment will be raised sequentially during mining and processing operations, and is designed to store approximately 1.6 billion tons of tailings. The new embankment will tie into the Magna Impoundment at the east and west abutment locations, as shown on Figure 1.2. Abutment berms will be constructed at both abutment locations to provide adequate seismic stability to the existing impoundment corners.

Whole tailings deposition into the North impoundment began on May 24, 1999 and about 14.3 million tons of tailings have been deposited to date in the embankment and within the new impoundment.

1.3 BACKGROUND AND OTHER COMPANION REPORTS

The Magna Impoundment has been investigated and evaluated on numerous occasions and the results have been summarized in various reports. Each investigation has provided additional data associated with the impoundment that has progressively increased the level of knowledge. This report presents the results of the stability analyses conducted between 1998 and 1999 by URS Greiner Woodward Clyde (URSGWC) based on our up-to-date understanding of the conditions at the Magna Tailings Impoundment. Similar previous evaluations were conducted by Woodward-Clyde (now URSGWC). The past studies that have been utilized in the preparation of this summary report include:

- North Expansion, Geotechnical Site Characterization Report (Woodward-Clyde 1991).
- North Expansion Design Report (Woodward-Clyde 1995).
- North Expansion Geotechnical Detailed Design Report, Appendix G (Woodward-Clyde 1995a).
- Geotechnical Site Characterization, Southeast Corner Seismic Upgrade Design (Woodward-Clyde 1998b).
- Investigations/Remediation of Northeast Corner Toe Slide (Woodward-Clyde 1998a).

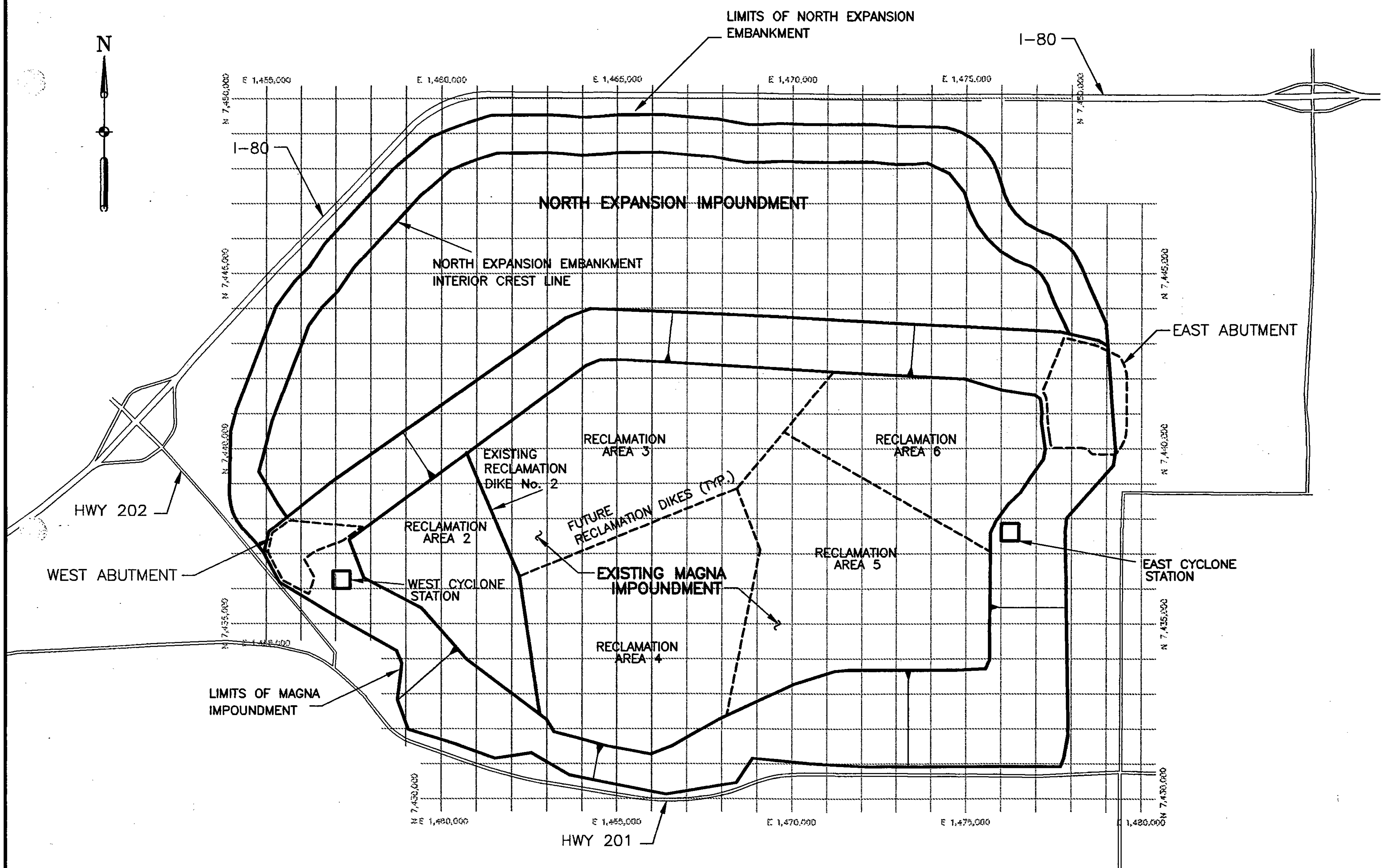
This report is intended as a summary report associated with the stability of the Magna Tailings Impoundment, including the revised designs of the east and west abutments of the North Expansion embankment. Detailed stability evaluations for the different areas around the Magna Tailings Impoundment are presented in the following Reports:

- North Slope Stability Evaluation Report (URSGWC 1999b).
- Revised East Abutment Berm Design Report (URSGWC 1999).
- Northeast Corner Raise Evaluation Report (URSGWC 1999a).
- East Slope Stability Evaluation Report (URSGWC 1999c).
- South Slope Seismic Stability Evaluation Report (URSGWC 1999d).

This summary report also includes results of stability analyses completed for the West Slope of the Magna Impoundment and the West Abutment Berm. Formal reports documenting these analyses will be issued upon completion in January 2000.

Project No. 24462	Kennecott Tailings Facility Aerial Photograph	Figure 1.1

URS Greiner Woodward Clyde



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GENERAL LAYOUT PLAN

SCALE: 1" = 3000'

PROJECT NO. 6800024462	KENNECOTT UTAH COPPER SUMMARY REPORT	GENERAL LAYOUT AND SITE PLAN KENNECOTT TAILINGS FACILITY	FIGURE 1.2
URS Greiner Woodward Clyde			

The site characterization includes evaluation of the slope surface geometry, sub-surface stratigraphy, engineering material properties, and pore water pressure conditions along the perimeter slope of the Magna Tailings Impoundment. These results were used to develop the slope stability models to evaluate the theoretical factors of safety for the Magna Impoundment.

2.1 STUDY AREAS

The current and future projected geotechnical conditions of the slope vary considerably around the entire Magna Impoundment. To accommodate these variations and to study each area according to its specific current and future conditions, the perimeter slope of the Magna Tailings Impoundment was divided into the following six study areas:

1) North Slope
2) Northeast Corner (including East Abutment)
3) East Slope
4) South Slope
5) West Slope
6) West Abutment

These study areas, graphically identified on Figure 2.1, cover all the important areas around the Magna Impoundment. These six study areas were identified based primarily on:

- Location.
- Current and future construction along the slope.
- Geotechnical conditions.
- Past performance of the slope.

Because the geotechnical conditions may somewhat vary within each study area, each study area was evaluated by locating and developing various study cross-sections within that study area. Table 2.1 presents a list of all the study cross-sections located around the Magna Tailings Impoundment.

Table 2.1
Study Cross-Sections

Study Area	Study Sections	Location (see Figure 2.1 for location in plan view)
North Slope	N1	Eastern Section
	N2	Western Section
	N3	Along the North Expansion decant barges
	N4	Western Section
	N5	Longitudinal section along the slope
	N6	Longitudinal section along the slope
Northeast Corner	J	Along Repaired Section
	NE1	Passing through the East Abutment Berm
East Slope	SE6	Northern Section
	SE4	Passing through the East Cyclone Station
	SE3	South of East Cyclone Station
	SE5	Southern Section
South Slope	SEC	Southeast Corner
	SE2	Eastern Section
	KLC	Western Section
West Slope	W2	South of West Cyclone Station
	W1	Passing through the West Cyclone Station
West Abutment	SLB	Southern Section
	SLD	Northern Section

2.2 CHARACTERIZATION OF STUDY SECTIONS

The surface geometry and the internal stratigraphy along the various study cross-sections were based on previous and recent field investigations, and actual ground surface topography. The field investigations performed to estimate the sub-surface stratigraphy mainly consisted of Cone Penetration Tests (CPTs) in conjunction with pore pressure dissipation tests. Bore holes were also drilled, using hollow stem augers, at key locations to obtain soil samples for laboratory testing and also to perform in-situ pressuremeter testing. In certain areas, especially along the toe of the slope, test pits were excavated to augment the CPT data.

The following section provides a general description of the soils and materials encountered along the slope of the Magna Tailings Impoundment. Details about material characterization are presented in our previous site characterization reports (Woodward-Clyde 1991, 1995, 1997, 1998a).

Embankment Tailings: In general, the tailings can be divided into two categories: *whole tailings and soft tailings clay.* The whole tailings are highly interbedded and relatively coarse-grained in nature and typically classify as a silty sands interbedded with silts and silty clays. The soft tailings clay is fine-grained and typically classifies as a low-to-medium plasticity silty clay. Two units of whole tailings generally exist around the Magna Impoundment: an upper layer of sandy beach deposits and a deep layer underlying the soft tailings clay. The soft tailings clay around the Northeast Corner area is characterized as *decant pond clay.*

Dikes: Three dike units are generally found around the Magna Impoundment: the starter dike, the 1950 dike, and the 1952 dike, as described in various historical documents. The starter dike, actually consisting of 3 to 4 dike raises, was reportedly constructed of dumped mine waste rock ranging in size from gravel to boulder-sized particles. The competency of the dike materials varies around the Magna Impoundment. In places substantial quantities of boulders and cobbles of relatively competent quartzite rock are found, while in other areas the mine waste rock has weathered considerably to silt and sand size matrix.

Foundation: The foundation of the Magna Tailings Impoundment consists of lake clays interbedded with lenses of sands. The top of the Upper Bonneville Clay unit, which comprises the upper approximately 10 to 15 feet of the foundation, is marked by the Gilbert Red Beds, so called because of oxidation stains resulting in a reddish appearance. The clays in this unit are occasionally interrupted by sand beds, typically less than 2 feet thick. Beneath the Upper Bonneville Clay unit are various interbedded clays and sands, referred to as the Interbedded Sediments.

A sample study cross-section along with the profiles of available CPT tip resistance values is shown on Figure 2.2. As seen in this figure, the various slope materials appear to have distinct cone tip resistance signatures. However, cone tip resistance was not the only parameter used to identify the boundary between the various materials. In areas where the tip resistance signatures were not very distinct, the profiles of other cone parameters, such as sleeve friction and dynamic pore pressures, were used to identify the different tailings layers. Additionally, drilling and sampling was used to verify the CPT signatures.

2.3 MATERIAL PROPERTIES

Engineering parameters for tailings and foundation materials used in the analyses presented in this report were generally based on the following previous characterization studies completed for the North Expansion and Magna tailings impoundments:

- Geotechnical Site Characterization Report for North Expansion Project (Woodward-Clyde 1991).
- Geotechnical Detailed Design Report, Appendix G, for North Expansion Project (Woodward-Clyde 1995).
- Geotechnical Site Characterization Report for Southeast Corner Seismic Upgrade Design (Woodward-Clyde 1998).

Based on the pressuremeter testings, conducted in 1998 and 1999, combined with oedometer testing of undisturbed specimens from drill holes, the following three modifications were made to the previous material characterization:

- The older deposits of deep whole tailings were assigned an overconsolidation ratio (OCR) of 1.5. This slight overconsolidation state is attributed to aging and chemical alteration of tailings over time. For these reasons, the undrained shear strength for a zone of deep whole tailings near the toe has been modeled with a slightly higher strength ratio.
- The soft tailings clays were assigned an OCR of 1.5, for reasons similar to the deep whole tailings. The OCR for the tailings clays present around the Northeast Corner area, characterized as decant pond clay, has been kept as 1.0.
- The Upper Bonneville Clay (UBC) layer underneath the 1952 dike has been assigned an OCR = 1.2. For strength characterization purposes, the OCR of the UBC layer was incrementally decreased from 4.0, in the free field, to 1.2, underneath the 1952 dike, and 1.0 beyond the 1952 dike, underneath the embankment.

Table 2.2 presents a summary of the current understanding of material properties based on field and laboratory investigations performed to date.

2.4 PORE PRESSURE CONDITIONS

The pore water pressure conditions along the various study cross-sections were evaluated and characterized by estimating two parameters: depth of the phreatic surface below the slope surface and the gradient of the pore pressure increase within the saturated material. To estimate these parameters in the field, extensive cone dissipation tests were performed typically at 15 to 25 feet depth intervals during the CPT program. The equilibrated dissipation test values were then plotted versus depth to estimate the in-situ pore water pressure conditions. To further monitor the pore water pressure variations with time, electronic vibrating wire piezometers were also installed at selected locations. These are a part of the continuously expanding monitoring program at the Magna Tailings Impoundment.

Along the Magna Impoundment perimeter slope, the depth to the phreatic surface within the tailings is generally fairly shallow - within 10 to 15 feet from the ground surface - except along the South Slope where the phreatic surface has been lowered due to the dewatering and stepback dike program. The gradient of pore pressure increase with depth is typically between 40 to 70 percent of hydrostatic near the crest to about 80 to 100 percent of hydrostatic near the toe. This general trend of increase in pore pressures with depth, from the crest towards the toe, is typical for the entire perimeter slope of the Magna Impoundment.

In certain areas of the Magna Tailings Impoundment, mainly the Northeast Corner and the East Slope, the pore pressure regime is extremely complicated and not adequately characterized by a piezometric surface to model the pore pressure conditions. For these study sections, pore water pressure contours were developed by hand to characterize the pore water pressures along the slope. In situ pore pressures in the embankment were measured using either vibrating wire piezometers and/or cone penetration dissipation tests. These measured pore pressures were then plotted on cross-sections and idealized contours were generated by hand contouring, then digitized and entered into the stability analysis files.

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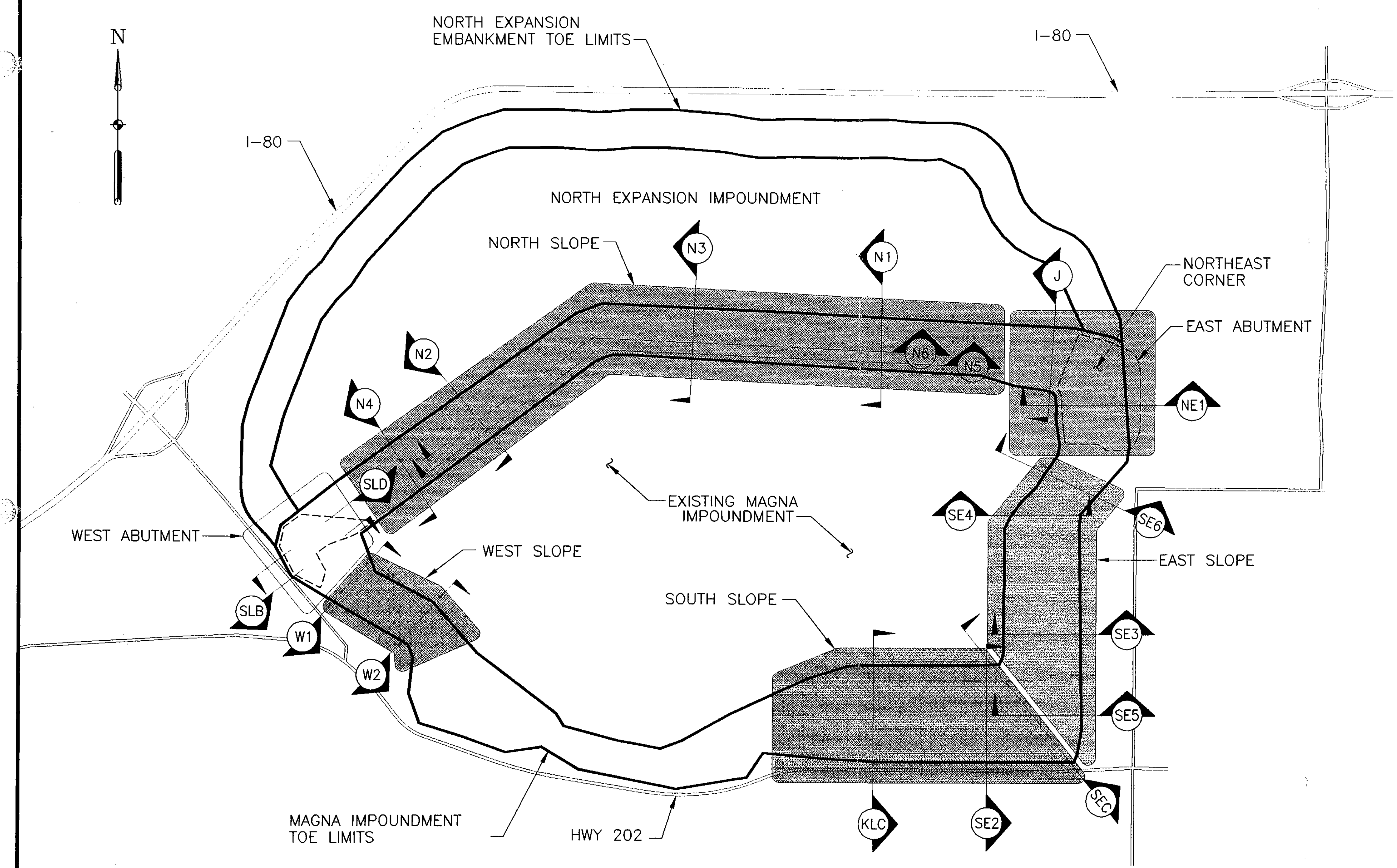

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GENERAL LAYOUT PLAN

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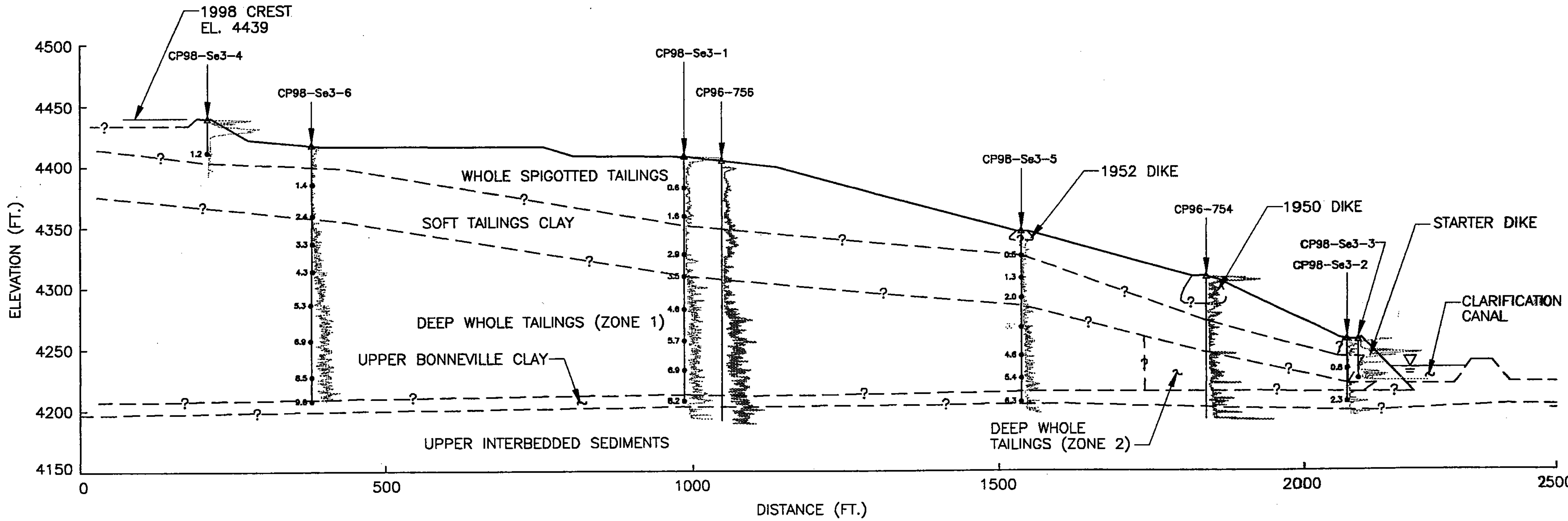
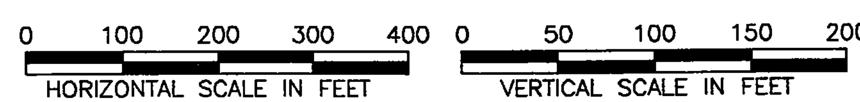
PROJECT NO. 6800024462	KENNECOTT UTAH COPPER SUMMARY REPORT	GENERAL LAYOUT OF STUDY AREAS AND STUDY SECTIONS AROUND MAGNA TAILING IMPOUNDMENT	FIGURE 2.1
URS Greiner Woodward Clyde			



STUDY SECTION SE3



LEGEND

- SOUNDING CONE TIP RESISTANCE PLOTTED AT 1"=800 TSF
- 1.4 — STABILIZED PORE PRESSURE OF 1.4 KSF FROM LINE DISSIPATION TEST
- △ CONE PENETRATION TEST

NOTES:

1. INTERNAL GEOMETRY IS APPROXIMATE.

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PROJECT NO. 6800024462	KENNECOTT UTAH COPPER SUMMARY REPORT	SAMPLE MAGNA IMPOUNDMENT STUDY SECTION INTERNAL GEOMETRY CHARACTERIZATION	FIGURE 2.2
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Table 2.2
SUMMARY OF ESTIMATED MATERIAL PROPERTIES USED IN STABILITY ANALYSES

MATERIAL	TOTAL UNIT WEIGHT (pcf)	PEAK UNDRAINED SHEAR STRENGTH		EFFECTIVE STRESS SHEAR STRENGTH	POST-EARTHQUAKE (MCE) SHEAR STRENGTH
		Direct Simple Shear (DSS)	Triaxial Compression Shear (TXL)		
TAILINGS					
Whole Spigotted Tailings (saturated)	119	$C_u/\sigma'_{v,c} = 0.23$	$C_u/\sigma'_{v,c} = 0.35$	$\phi'=34^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.12$
Whole Spigotted Tailings (unsaturated)	117	$\phi'=34^\circ$, c=0		$\phi'=34^\circ$, c=0	$\phi'=34^\circ$, c=0
Soft Tailings Clay	107	$C_u/\sigma'_{v,c} = 0.23(OCR)^{0.8}$ Estimated OCR =1.5	$C_u/\sigma'_{v,c} = 0.29(OCR)^{0.8}$ Estimated OCR =1.5	$\phi'=28^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.20$
Decant Pond Clay	107	$C_u/\sigma'_{v,c} = 0.23(OCR)^{0.8}$ Estimated OCR =1	$C_u/\sigma'_{v,c} = 0.32(OCR)^{0.8}$ Estimated OCR =1	$\phi'=28^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.20$
Deep Whole Tailings (Zone 1)	116	$C_u/\sigma'_{v,c} = 0.23(OCR)^{0.8}$ Estimated OCR =1.0	$C_u/\sigma'_{v,c} = 0.35(OCR)^{0.8}$ Estimated OCR =1.0	$\phi'=34^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.12$
Deep Whole Tailings, (Zone 2)	116	$C_u/\sigma'_{v,c} = 0.23(OCR)^{0.8}$ Estimated OCR =1.5	$C_u/\sigma'_{v,c} = 0.35(OCR)^{0.8}$ Estimated OCR =1.5	$\phi'=34^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.12$
Dikes (Starter, 1950, and 1952)	115	$\phi'=35^\circ$, c=0		$\phi'=35^\circ$, c=0	$\phi'=35^\circ$, c=0
FOUNDATION					
Upper Bonneville Clay	118	$C_u/\sigma'_{v,c} = 0.26(OCR)^{0.8}$ Max OCR=4, Min Cu=625psf		$\phi'=29^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.208$ Min S_{ur}=500 psf
Upper Interbedded Sediments	129	$C_u/\sigma'_{v,c} = 0.26(OCR)^{0.8}$ Max OCR=4, Min Cu=625psf		$\phi'=31^\circ$, c=0	$S_{ur}/\sigma'_{v,c}=0.208$ Min S_{ur}=500 psf

Notes:
1) $S_{ur}/\sigma'_{v,c}$=Residual Shear Strength Ratio.
2) $\sigma'_{v,c}$=Vertical Effective Stress During Consolidation.
4) Post-earthquake (OBE) strengths are equal to the peak undrained strengths.
3) Foundation undrained strenths are reduced by 20% for post-earthquake (MCE) strengths.

This section presents the design criteria, loading conditions, and a general methodology of the analyses performed to evaluate the stability and deformation of the Magna Impoundment slope.

3.1 DESIGN CRITERIA

The design criteria for the Kennecott Utah Copper North Tailings Expansion was developed with modifications to the State of Utah Statute, UAC 73-5a-502 and the Administrative Rules for Water Retention Dam Safety, UAC R655-11-6A, adopted June 1993. These modifications were summarized in a KUCC letter to Mr. Richard Hall, Directing Engineer, of the State of Utah, dated August 26, 1992, and the confirmation letter from Mr. Richard Hall dated September 4, 1992. (Both letters are attached in Appendix A) The modifications are due to the fact that the Utah Statues were developed for structures impounding water, not for structures impounding tailings.

For static stability analyses, UAC R655-11-6A requires the factor of safety under the Steady State Seepage (long-term) to be 1.5 and the factor of safety under the End of Construction (short-term) to be 1.3.

For the seismic stability analyses, UAC R655-11-5C requires both the Operating Basis Earthquake (OBE) and Maximum Credible Earthquake (MCE) conditions to be addressed as part of the design criteria. The OBE would be estimated based on a site-specific probabilistic seismic hazard analysis and the MCE would be estimated in a deterministic manner.

If portions of a water retention dam liquefy, the post-earthquake factor of safety (i.e., the factor of safety after liquefaction is triggered) should be greater than or equal to 1.2 under both the OBE and MCE conditions. For the North Expansion tailings embankment this requirement has been modified by the State Engineer to a post-earthquake factor of safety of 1.0 under the OBE and MCE conditions, provided that a "state-of-the-art" dynamic deformation analysis demonstrates that a "catastrophic failure" will not occur.

A catastrophic failure is defined as a "major tailings release resulting in a threat to public health, property and/or present a public safety risk." The potential for catastrophic failure would be precluded if the dynamic analysis results indicate that the likely displacement and deformation are manageable without threat of life or extensive property damage. The August 26, 1992 letter by KUCC also allows for the calculated deformation to exceed the normal freeboard as defined within the Utah Statutes "provided that the operating decant pond was kept at an adequate distance from the impoundment perimeter such that 'catastrophic' tailings release was precluded."

The above design criteria govern and are satisfied by the design and construction of the new North Expansion tailings embankment including the east and west abutments. It is Kennecott's intent to meet the same modified design criteria at the existing Magna tailings impoundment. Consequently, efforts are actively progressing to seismically upgrade the 90-plus year old structure.

3.2 LOADING CONDITIONS

The North Expansion embankment slopes and the Magna Impoundment slopes are collectively referred to as the "Kennecott tailings slopes." The stability and deformation of the Kennecott tailings slopes were, in general, evaluated for the following loading conditions based on the modified design criteria as discussed in Section 3.1:

- Static End of Construction conditions
- Static Steady State Seepage conditions
- Seismic loading conditions:
  - Post-MCE loading conditions
  - Post-OBE loading conditions

The following discussions of these loading conditions also include general types of analysis used in the evaluations. The analyses results however, are discussed in the subsequent sections.

3.2.1 End of Construction Conditions

The End of Construction loading conditions for the Kennecott tailings slopes apply to the stability of slopes during or immediately after tailings deposition, or the construction of berms near the slopes. Only the areas of slopes affected by such construction are evaluated for the End of Construction conditions.

The shear strength used in the End of Construction conditions is typically undrained shear strength corresponding to the pre-construction conditions. The subsequent assumptions made were as follows:

- The construction loading is fast enough to preclude migration of pore water pressures during shearing.
- The construction is completed fast enough that the foundation soils would not have time to consolidate under the new loading conditions.

Therefore, the stability of slopes under the End of Construction conditions was evaluated using the Undrained Strength Analysis (USA) approach described by Ladd (1991). Alternatively, an effective stress analysis can be performed using appropriate pore water pressures that reflect the effects of both the ambient and shear loading due to construction.

During any construction in the vicinity of the slopes, the above mentioned assumptions are judged to be reasonably satisfied with some level of conservatism. For the cases involving the deposition of tailings, the two assumptions listed above are considered to lead to excessive conservatism. For these areas we believe that, the rate of loading increase due to tailings deposition is slow enough that excess pore water pressures induced by the deposition would be dissipated in the foundation tailings. Therefore, for these cases, the effective stress analysis (Steady State Seepage) with pore water pressures that appropriately reflect the effects of loading and consolidation was performed to evaluate the static conditions.

For large slopes, such as those involved in the North Expansion embankment and the Magna Tailing Impoundment, the End of Construction conditions should apply only to those areas directly affected by the construction loading. In order to obtain an adequate level of confidence in this regard, a detailed two-dimensional stress analysis using the computer program FLAC (Itasca 1995) was performed. The objective of the analysis was to calculate the stress and associated excess pore pressure increase, within the affected slope caused by the deposition of tailings at the crest. The results of the FLAC analysis was used as guidance in evaluating the portions of the embankment slope affected by the End of Construction loading conditions. On the basis of these results, potential failure surfaces that lie outside the zones affected by deposition, were not addressed in the slope stability analysis for the End of Construction conditions, but were evaluated for Steady State Seepage conditions.

3.2.2 Steady State Seepage Conditions

The Steady State Seepage conditions represent the long-term loading conditions where all the transient excess pore water pressures have been dissipated. For this loading condition, the pore pressures in the impoundment are assumed to have reached their steady-state values. The slope stability analyses of the Magna Tailings Impoundment under the Steady State Seepage conditions were performed using an effective stress analysis with the steady state pore water pressures and effective shear strength (drained) stress parameters.

3.2.3 Seismic Loading Conditions

A detailed seismic hazard assessment has been conducted at the Magna Tailings Impoundment in 1995 for the design of the North Expansion and in 1996 for the Southeast Corner evaluations. Currently, the seismic loading conditions for the North Expansion impoundment and the Magna Tailings Impoundment are being updated to reflect the impacts of the ongoing regional seismic hazard study of the Salt Lake City region (Wong et al., pending publication). The updated seismic loading conditions consist of evaluation of the Maximum Credible Earthquake (MCE) and the Operating Basis Earthquake (OBE). The MCE peak ground accelerations (PGA) at Magna Tailings Impoundment were estimated based on the highest median deterministic values using the maximum moment magnitude associated with various faults. However, since the OBE

PGAs are currently under evaluation, using a probabilistic seismic hazard analysis (PSHA), a range of valves were assumed for the Magna Tailings Impoundment. A report detailing the seismic site conditions will be issued early in 2000.

For both conditions (MCE and OBE), as discussed in Section 3.1, if the post-earthquake slope stability analysis results in the computed factor of safety of 1.2 or greater, the slope is considered to meet the water retention dam design criteria. Such slope stability analysis would use post-earthquake undrained shear strength of the materials, as shown on Table 2.2, assuming liquefaction has occurred. The post-earthquake undrained shear strength would reflect the strength reduction induced by cyclically induced excess pore water pressures and the residual liquefied shear strength for those materials assumed to liquefy under the postulated seismic loading conditions.

Also, as discussed in Section 3.1, when the results of the post-earthquake slope stability analysis do not satisfy the 1.2 factor of safety criteria, the slope can be considered to meet the tailings embankment design criteria, provided the following two conditions are satisfied:

- The computed post-earthquake factor of safety is 1.0 or greater, and
- The dynamic deformation analysis shows that the expected deformation would not result in catastrophic failure and uncontrolled tailings release.

For the areas of the Magna Tailings Impoundment that do not satisfy these two conditions, run-out analyses were performed to evaluate the potential consequence of tailings release.

Dynamic deformation analysis was performed using the undrained shear strength of materials reflecting the following effects:

- the rate of loading effects.
- cyclically induced reduction in strength.
- residual shear strength when liquefaction occurs.
- large-strain induced reduction in strength for clayey soils.

The seismic design criteria as discussed in Section 3.1 allows for the slopes to be deformed in the event of an MCE without a catastrophic failure (i.e., tailings would not be released). Therefore, slopes that satisfy the MCE conditions would automatically satisfy the less intense OBE conditions.

The estimated values of PGA for the MCE were updated at the four site locations shown on Figure 3.1, in accordance with the ongoing regional seismic hazard study of Salt Lake City region. However, the current PGA values for the OBE cannot be estimated at this time, since this portion of the regional study (Wong et al., pending publication) has not been completed.

Consequently, the following paragraphs describe our assumptions associated with MCE and OBE loading conditions.

MCE Loading Conditions

A detailed seismic hazard and ground motion assessment was performed in 1995 for the design of the North Expansion structures (Woodward-Clyde 1995, Appendix A). Based on the results of the 1995 study, a design MCE PGA value of 0.52g was selected for the design of the North Expansion embankment and abutments. In 1996, the seismic hazard and ground motions were re-evaluated for the Magna Impoundment and a design MCE PGA value of 0.38g was selected for the Southeast corner of the Magna Impoundment. The recently updated MCE for the North Expansion impoundment and the Magna Tailings Impoundment is a Mw 7-1/4 earthquake on the East Great Salt Lake Fault resulting in the PGA values shown on Table 3.1:

Table 3.1

Recently Updated PGA Values for MCE

Site	Seismogenic (km)	Distances Horizontal (km)	Rupture (km)	Abrahamson & Silva (1997) (g)	Spudich et al. (1999) (g)	Campbell (1997) (g)	Sadigh et al. (1997) (g)	Weighted Mean (g)
1	13.0	10.5	10.5	0.27	0.34	0.32	0.34	0.31
2	12.5	10.0	10.0	0.28	0.35	0.33	0.34	0.32
3	6.5	3.8	3.8	0.42	0.54	0.38	0.47	0.46
4	6.9	4.2	4.2	0.41	0.53	0.37	0.46	0.45

Notes:

(1) See Figure 3.1 for Locations 1 through 4.

(2) The weighted mean value is based on assigning weights of 0.40, 0.30, 0.15 and 0.15 for the above attenuation relationships, respectively.

(3) Seismogenic distance used by Campbell (1997). The top of crystalline rock is assumed to be at 2 km.

(4) Horizontal distance used by Spudich *et al.* (1999).

(5) Rupture distance used by Abrahamson and Silva (1997) and Sadigh *et al.* (1997).

The Imperial Valley array 5 motion, scaled to an appropriate MCE PGA value was selected to represent the acceleration time history of the MCE for use in dynamic response and deformation analyses. Figure 3.2 shows the MCE response spectrum (specified at the free-field ground surface) at 5 percent damping, scaled to an example PGA value of 0.46g along with the corresponding acceleration time history used for the West Abutment analyses.

OBE Loading Condition

Based on the 1995 seismic hazard and ground motion assessment, a design OBE PGA value of 0.29g was selected for the design of the North Expansion embankment and abutments. In 1996, the seismic hazard and ground motions were re-evaluated for the Magna Impoundment and a design OBE PGA value of 0.14g was selected for the Southeast corner of the Magna Impoundment. In accordance with the ongoing regional seismic hazard study of Salt Lake City region, the design OBE PGA values are being re-evaluated and the results have not been finalized. Therefore, the OBE PGAs for the North Expansion impoundment and the Magna Impoundment have not been updated at this time. However, the OBE magnitude would be a Mw 6.5 earthquake with the PGA values selected from the results of the updated probabilistic seismic hazard analysis corresponding to an average return period (ARP) of 500 years for the North Expansion impoundment and 200 years for the Magna Impoundment. For the purposes of evaluating possible deformations of the North and East slopes of the Magna Impoundment under the OBE, a possible range of OBE PGA values, from 0.14g to 0.2g, was selected in the absence of updated regional study results.

The resulting OBE would be postulated as a random-source event occurring beneath the impoundment for the purposes of selecting an acceleration time history. The 1987 Loma Prieta Gilroy 3 motion, scaled to the appropriate PGA value was selected to represent the acceleration time history of the OBE ground motion. Figure 3.3 shows the OBE response spectrum (specified at the ground surface) at 5 percent damping, scaled to an example PGA value of 0.2g along with the corresponding accelerogram used in the dynamic analyses.

3.3 SLOPE STABILITY ANALYSES

The slope stability analyses were performed using the computer program UTEXAS3 (Wright 1991). Spencer's method of slices was used for computing the theoretical factors of safety. Both non-circular and circular search routines were used to identify the critical factors of safety and the corresponding potential failure surfaces for the slopes under various loading conditions, using the appropriate shear strengths discussed previously.

3.4 LIQUEFACTION POTENTIAL

Site response analyses were performed to assess the seismically induced shear stresses in the tailings and in the foundation soils in response to the MCE and OBE shaking conditions. The seismically induced shear stresses from these analyses were used together with the cyclic strength properties of various materials to evaluate the liquefaction potential of these materials under the MCE and OBE loading conditions.

The site response analysis consisted of one-dimensional (1-D) and two-dimensional (2-D) analyses. The purpose of the one-dimensional analyses was to develop input accelerograms for the two-dimensional site response analyses. Using the accelerograms computed from the 1-D analyses, the 2-D analyses estimated the cyclic shear stresses induced in the impounded tailings and foundation soils. The computed cyclic shear stresses were then used to estimate the post-earthquake shear strengths. The one-dimensional analyses were completed using the computer program SHAKE and the two-dimensional analyses were completed using the computer program QUAD4. Further details of SHAKE and QUAD4 analyses are presented in Appendix A of the "South Slope Seismic Stability Evaluation Report" (URSGWC 1999d).

3.5 DYNAMIC DEFORMATION ANALYSES

The computer code FLAC (Itasca 1995) was used to perform the seismic deformation evaluation of the Kennecott tailing slopes. FLAC is a two-dimensional explicit finite difference computer program for geotechnical applications.

The approach to a dynamic deformation analysis using FLAC typically consists of the following three parts:

1) A turn-on gravity analysis is completed to evaluate the initial stresses in a tailings slope before an input earthquake motion is applied. At the end of the turn-on gravity analysis, the calculated stresses (horizontal, vertical, and shear stresses) should satisfy the horizontal and vertical force equilibrium conditions, and strain compatibility conditions as well.

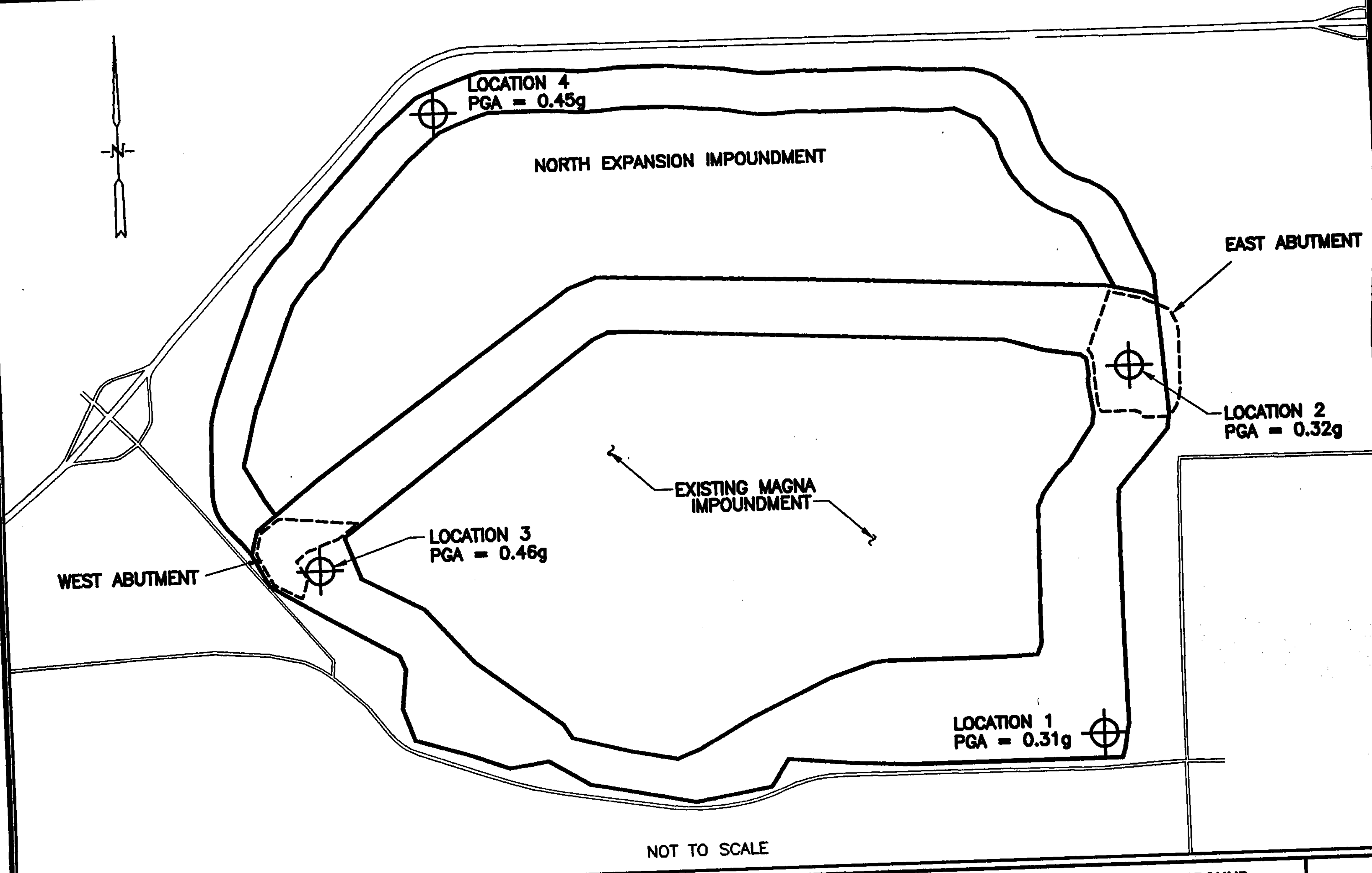
2) A dynamic analysis is then performed to evaluate the seismic response and deformations of the tailing slope due to gravity and the input earthquake motion. The dynamic stresses are added to the static stresses, and the earth structure is allowed to distort (translate, rotate, compress, and expand).

3) Finally, a post-earthquake analysis is completed to evaluate deformations of the tailings slope under gravity loading alone following the input earthquake shaking. The input earthquake motion is stopped, but gravity load is maintained to evaluate deformations that may be induced by readjustment of stresses and strains developed during earthquake shaking. The gravity force is applied until a final, static equilibrium is achieved. However, if the slope is "unstable" under post-earthquake loading, a static force imbalance will be maintained, and the slope will continue to deform, meaning that a "flow failure" of the slope may be considered as a likely scenario.

3.6 RUN-OUT ANALYSES

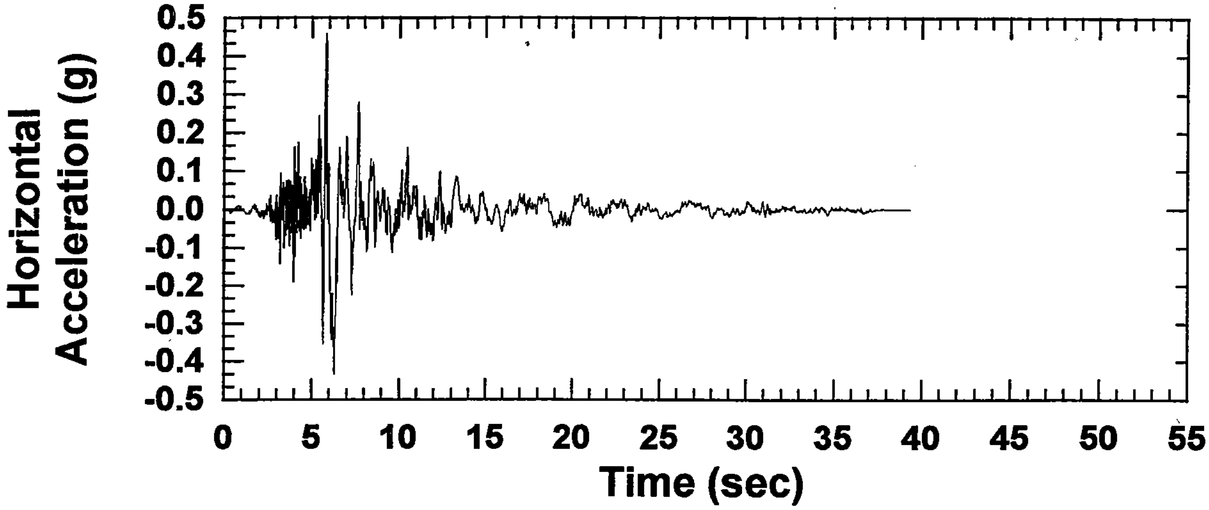
When enough tailing materials liquefy under seismic loading conditions, the factor of safety for some potential failure surfaces may fall below one. This condition could initiate a release of tailing materials from the impoundment, resulting in a tailings flow slide. The ensuing slide mass would initially gain speed with time; however, the mass would eventually slow down and reach a stable configuration and would come to rest in a post-liquefaction state. An analysis to evaluate such post-liquefaction configurations is termed "run-out" analysis.

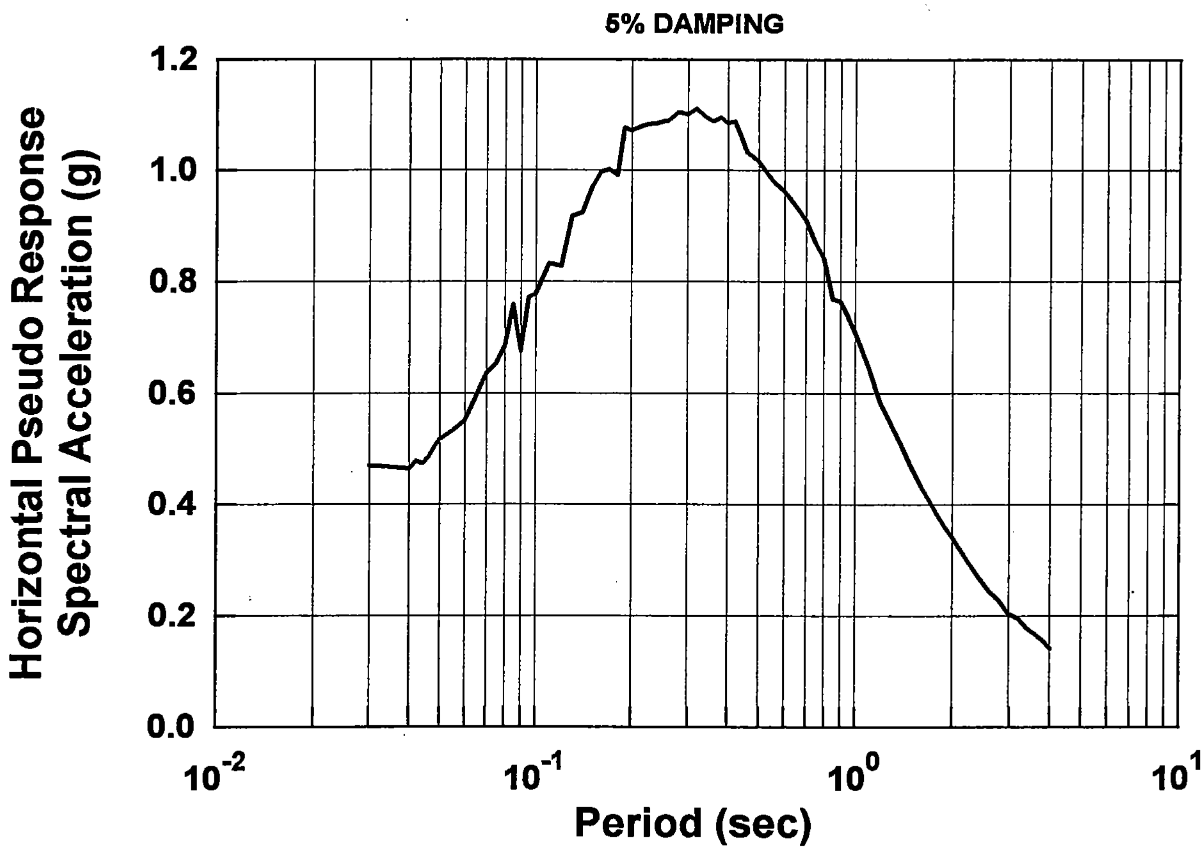
The run-out analysis was performed using the computer DRUM (Dynamic Run-out Method) (Tan et al., pending publication) program, as described in East Slope Stability Evaluation Report. In DRUM, the dynamic nature of the run-out process is modeled by first identifying a failure surface (mass) and analyzing downslope movement by translating the unbalanced forces acting on the sliding mass into acceleration using Newton's "F = ma" equation. Eventually the sliding mass comes to a stop because of the resistance from shear strength of the liquefied material, and the "dynamically stable" configuration of the sliding mass is computed. The distance between the initial and the final configuration of the failure mass is estimated to be the run-out distance.



NOT FOR CONSTRUCTION	PROJECT NO. 6800024462 / KENNECOTT UTAH COPPER SUMMARY REPORT / URS Greiner Woodward Clyde	ESTIMATED VALUES OF PEAK GROUND ACCELERATION (PGA) FOR MAXIMUM CREDIBLE EARTHQUAKE (Based on 1999 "Update" Study)	FIGURE 3.1

EXEC–F1.6





ACCELERATION TIME HISTORY AND ACCELERATION RESPONSE SPECTRA OF INPUT GROUND MOTION 1979 IMPERIAL VALLEY EARTHQUAKE - IMPERIAL VALLEY ARRAY No. 5 - MCE			
Project No. 24462	Date: DEC 99	Project: KENNECOTT	Fig. 3.2

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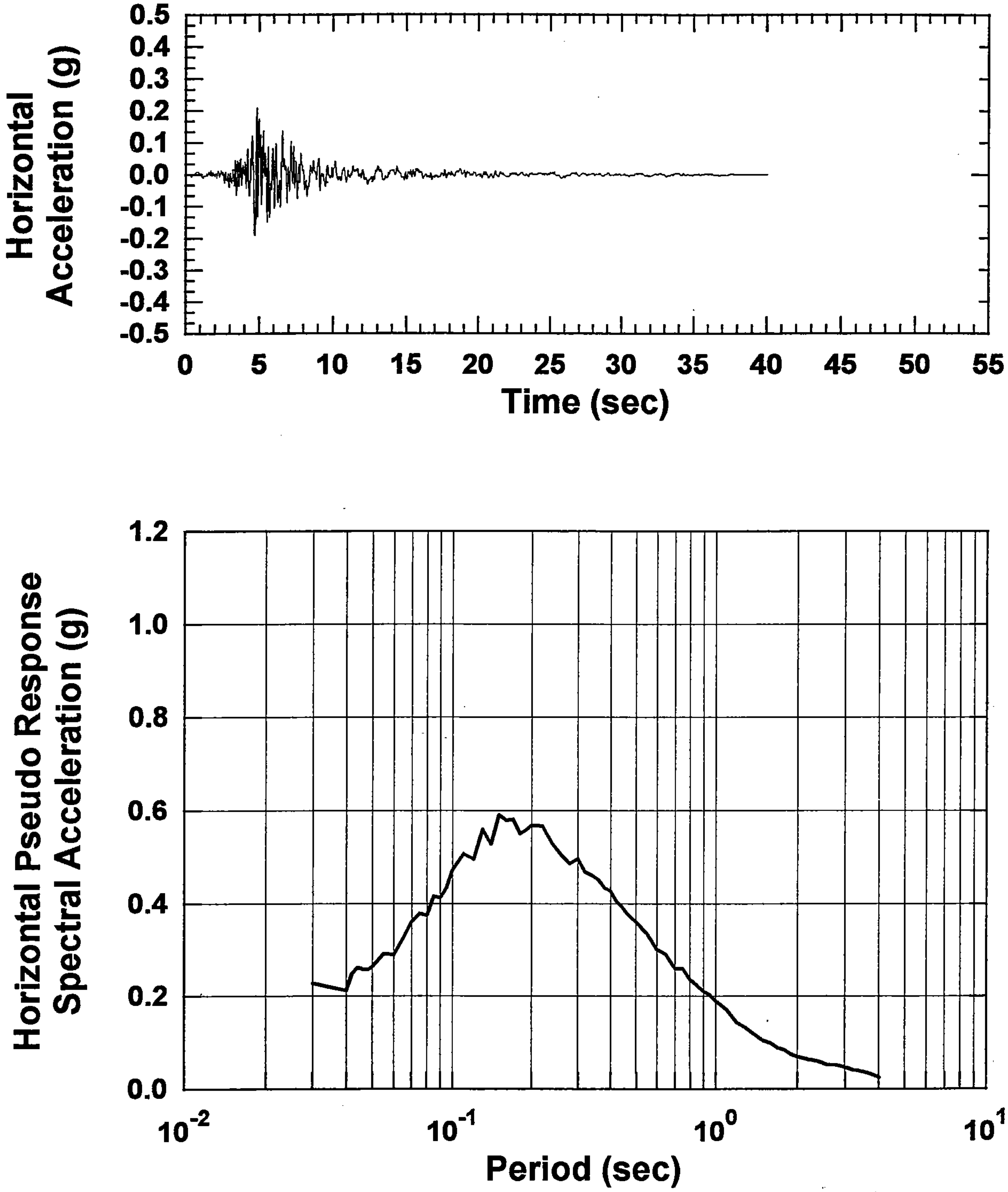


ACCELERATION TIME HISTORY AND ACCELERATION RESPONSE SPECTRA OF INPUT GROUND MOTION 1989 LOMA PRIETA EARTHQUAKE - GILROY NO. 3 - OBE (0.2g)			
Project No. 24462	Date: DEC 99	Project: KENNECOTT	Fig. 3.3

4.1 GENERAL

Slope stability analyses were performed to assess the static and seismic stability of the Magna Tailings Impoundment slope for the current (1999) and future embankment heights, and associated loading conditions. The future conditions consist of additional tailings deposition on top of the Magna Impoundment and the North Expansion tailings deposition against the North Slope of the Magna Impoundment.

4.2 SUMMARY OF RESULTS

The computed theoretical factors of safety (FS) values for the various study sections are summarized in Table 4.1. The following paragraphs describe a brief overview of the stability analyses results.

4.2.1 North Slope

For static stability, the North Slope was analyzed for Steady State Seepage conditions. The current static stability of the North Slope satisfies the design criteria (computed FS range from 1.5 to 3.1). The future static stability of North Slope will further improve due to the buttressing from the North Expansion tailings and hence will also satisfy the design criteria.

The critical post-OBE FS for the North Slope was calculated as 1.1 assuming no liquefaction. To evaluate the deformation potential of the North Slope under OBE loading conditions, a dynamic deformation analysis was performed using a range of OBE events (PGAs=0.14g to 0.2g), as discussed in Section 3.2.3. The results from the dynamic deformation analysis indicate that during the assumed OBE event the liquefaction potential is not significant and the magnitude of deformations will be acceptable (less than about 1 to 3 feet) - thus satisfying the design criteria.

The North Slope is not expected to be stable for the post-MCE loading conditions until the North Expansion elevation reaches the top of the Magna Impoundment. Estimates of run-out distances indicate that in the event of a failure due to an MCE event, the tailings flow is expected to be contained within the North Expansion impoundment, as shown on Figure 4.1.

4.2.2 Northeast Corner and East Abutment

Due to the Northeast Corner remediation completed in July 1998, the static stability of this part of the North Slope was evaluated for the End of Construction conditions. The current and future static stability of the Northeast Corner is estimated to satisfy the design criteria (the minimum computed FS is 1.4).

The post-OBE FS for the Northeast Corner was calculated to range from 1.3 to 1.6 assuming no liquefaction. Similar to North Slope, we believe that the potential for liquefaction under the assumed OBE PGAs is slight with associated deformations within the acceptable range. The post-MCE FS for the eastern side of the Northeast Corner (Section NE1), is presently below 1.0. However, it is expected to improve as the future phases of the East Abutment berm are completed. When the entire East Abutment Berm is complete, the projected post-MCE FS along the east side of the Northeast Corner is estimated to be above 1.2, thus satisfying the design criteria. The seismic stability of the northern side of the Northeast Corner is expected to be similar to the North Slope, as described above.

The overall size of the East Abutment Berm was dictated by the design MCE estimated at the time of the North Expansion embankment design (PGA=0.52g, as estimated in 1995). Based on the latest MCE evaluations as a part of the Salt Lake City regional study, as discussed in Section 3.2.3, the PGA for the MCE event in the Northeast Corner area is estimated to be 0.32g. Since this latest estimate of PGA is significantly less than the previous design value, the East Abutment Berm may be optimized in the future.

4.2.3 East Slope

For static stability, the East Slope was analyzed for Steady State Seepage conditions. The current static stability of the East Slope satisfies the design criteria (computed FS range from 1.6 to 2.6). The critical shear surfaces identified for the current conditions are considerably away from the crest and are not expected to be influenced by the future tailings deposition based on the results of the FLAC analysis. Therefore, it is expected that the future static stability of the East Slope will also satisfy the design criteria.

The post-OBE FS ranges from 1.1 to 1.2. To evaluate the potential deformations of the East Slope under OBE loading conditions, a dynamic deformation analysis was performed using a range of the OBE events (PGA=0.14g to 0.2g), as discussed in Section 3.2.3. The results from the dynamic deformation analysis indicate that the East Slope tailings deformations will be acceptable (less than 1.0 foot) - thus satisfying the design criteria.

The East Slope is not expected to be stable for the post-MCE loading conditions (computed FS range from 0.5 to 0.7). Estimates of run-out distances indicate that, in the event of a failure due to an MCE event, the tailings flow will not impact any public roads or buildings, as shown on Figure 4.1.

4.2.4 South Slope

For static stability, the South Slope was also analyzed for Steady State Seepage conditions. The current static stability of the South Slope satisfies the design criteria (computed FS of 2.3). Since the tailing deposition along the South Slope has been considerably stepped back (about 2,000 feet), the future static stability of the South Slope is not expected to be impacted by the tailings deposition, and therefore is also expected to satisfy the design criteria.

The post-OBE FS for the South Slope is greater than 1.2 assuming no liquefaction, therefore, satisfying the design criteria. The potential for liquefaction of the South Slope is slight in the event of the assumed OBE PGAs, and the associated deformations within the acceptable range.

Since the South Slope is close to State Highway 201 and other public facilities, KUCC implemented a dewatering program in 1989 to seismically upgrade the South Slope and Southeast Corner. The intent of the dewatering program is to lower the phreatic surface within the slope so that the liquefaction potential of the South Slope tailings is reduced. The dewatering system implemented by KUCC along the South Slope and the Southeast Corner of the Magna Impoundment include:

- Moving active deposition away from the crest by a series of step-back dikes
- Installing horizontal drains along the toe
- Installing dewatering wells along the 1991 step-back
- Installing vertical wick drains at the crest and around the dewatering wells
- Construction of potential tailings flow deflection berms
- Installation of seismic hazard road warning system

Current field data has shown that significant progress has been made in lowering the phreatic surface within the tailings along the South Slope since 1993, as shown on Figure 4.2. Under the current pore water pressure conditions, the post-MCE FS of the South Slope is estimated to be 0.9. Target phreatic surfaces, corresponding to post-MCE FS values of 1.0 and 1.2, have been established in order to monitor compliance with the design criteria, as shown on Figure 4.2.

4.2.5 West Slope

For static stability, the West Slope was also analyzed for Steady State Seepage conditions. The current static stability of the West Slope satisfies the design criteria (computed FS range from 2.1 to 2.3). The western portion of the Magna Impoundment was reclaimed in the beginning of 1999. Therefore, the future raise and pore pressure conditions along the West Slope are not expected to negatively impact the stability in the future, and the slope is expected to continue to satisfy the design criteria.

The post-OBE FS range from 1.1 to 1.7 assuming no liquefaction. Based on the comparison of the geotechnical conditions along the West Slope to other parts of the Magna Impoundment, we believe the West Slope is more resistant liquefaction and associated deformations. Therefore, based on the results of the OBE deformation analyses completed for the East and North Slopes, we believe that the potential post-OBE deformations along the West Slope would be small - thus satisfying the design criteria.

Under current pore pressure conditions, the West Slope is not expected to be stable for the post-MCE loading conditions (computed FS range from 0.6 to 0.7). However, run-out analyses and deformation analyses along study Section W1 indicate that the extent of the seismic run-out scarp will not impact the North Expansion impounded tailings. A seismic hazard road warning system has been installed in this area to protect the public. Furthermore, as a result of the reclamation of the western portion of the Magna Impoundment, it is expected that the pore pressures along the West Slope will decrease with time (natural drainage under gravity), and subsequently, the seismic stability along the West Slope will improve. Observational approach, similar to the South Slope, has been developed for the West Slope to actively monitor the pore pressures and evaluate the stability improvement with time.

4.2.6 West Abutment

The stability of the Magna Impoundment at the West Abutment area was checked both before and after completion of the revised West Abutment Berm (revised West Abutment Berm design is discussed in Section 5.0). The current stability of the Magna Impoundment in the West Abutment area is similar to the West Slope stability results, as discussed above. However, after completion of the revised West Abutment Berm, the post-MCE FS is anticipated to range between 1.2 and 1.4. The accompanying dynamic deformation analysis, completed for the final geometry of the revised West Abutment Berm, indicate that the West Abutment would be stable during and after an MCE event. The dynamic deformation analyses were completed using the latest estimate of PGA=0.46g for the West Abutment area, as discussed in Section 3.2.3.

Table 4.1
Summary of Slope Stability Analyses Results

Study Area	Study Section	Static Conditions			Seismic Conditions	
		Loading	Current FS	Future FS	Current FS	Future FS
North Slope	N1	Steady State Seepage	Toe = 1.48 Global = 2.20	See note 1	OBE[6] = 1.08 MCE = 0.60	MCE ~ 0.6 [2]
	N2	Steady State Seepage	Toe = 1.74 Global = 2.63	See note 1	OBE[6] = 1.10 MCE = 0.63	See note 8
	N3	Steady State Seepage	Toe = 3.10 Global = 2.85	See note 1	OBE[6] = 1.30 MCE = 0.65	See note 8
	N4	Steady State Seepage	See note 8	See note 1	See note 8	See note 8
Northeast Corner	J	End of Construction	Toe = See note 8 Global[7] = 1.43	Toe = See note 8 Global = 1.27	OBE[6] = 1.43 MCE = See note 8	OBE = 1.27 MCE = See Note 2
	NE1	End of Construction	Toe = 1.26 Global[7] = 1.34	Toe = See note 8 Global = 1.37	OBE[6] = 1.26 MCE = See Note 3	OBE = 1.37 MCE = See Note 3
East Slope	SE6	Steady State Seepage	Toe = 1.93 Global = 2.49	See note 4	OBE[6] = 1.18 MCE = 0.70	See note 8
	SE4	Steady State Seepage	Toe = 1.66 Global = 2.26	See note 4	OBE[6] = 1.15 MCE = 0.54	See note 8
	SE3	Steady State Seepage	Toe = 1.60 Global = 2.57	See note 4	OBE[6] = 1.12 MCE = 0.68	See note 8
	SE5	Steady State Seepage	Toe = 1.79 Global = 2.14	See note 4	OBE[6] = 1.17 MCE = 0.55	See note 8

Study Area	Study Section	Static Conditions			Seismic Conditions	
		Loading	Current FS	Future FS	Current FS	Future FS
South Slope	SEC	Steady State Seepage	See note 8	See note 8	See note 8	See note 8
	SE2	Steady State Seepage	2.30	See note 8	$OBE^6 = 1.49$ MCE = 0.87	MCE = 1.2 [5]
	KLC	Steady State Seepage	See note 8	See note 8	None	See note 8
West Slope	W2	Steady State Seepage	2.12	See note 8	$OBE^6 = 1.74$ MCE = 0.61	See note 8
	W1	Steady State Seepage	2.33	See note 8	$OBE^6 = 1.12$ MCE = 0.71	See note 8
West Abutment	SLB		See note 8	See note 8	See note 8	See note 8
	SLD	Steady State Seepage	See note 8	2.31 [9]	See note 8	$OBE^{6,9} = 2.03$ $MCE^9 = 1.15$

[1] The static stability of the North Slope will improve in the future due to the buttressing effect of the North Expansion tailings. Since the design criteria for current static conditions is satisfied, additional analyses for future conditions were not performed.

[2] The seismic stability of North Slope, for MCE loading condition, was evaluated for various North Expansion tailing elevations. The results indicate that the North Slope is not expected to be seismically stable before the North Expansion elevation reaches the top of the Magna Impoundment.

[3] The seismic stability of East Abutment, for MCE loading condition, is expected to improve as the various phases of the abutment berm are completed. When the entire East Abutment Berm is complete, the projected FS is estimated to be above 1.2 – satisfying the design criteria.

[4] The critical shear surfaces identified for the current conditions are considerably away from the crest and are not expected to be influenced by the future tailings deposition. Since the design criteria for current static conditions is satisfied, additional analyses for future conditions were not performed.

[5] The post-seismic FS of South Slope, for MCE loading, is expected to improve to 1.2, provided the established target phreatic surface is achieved.

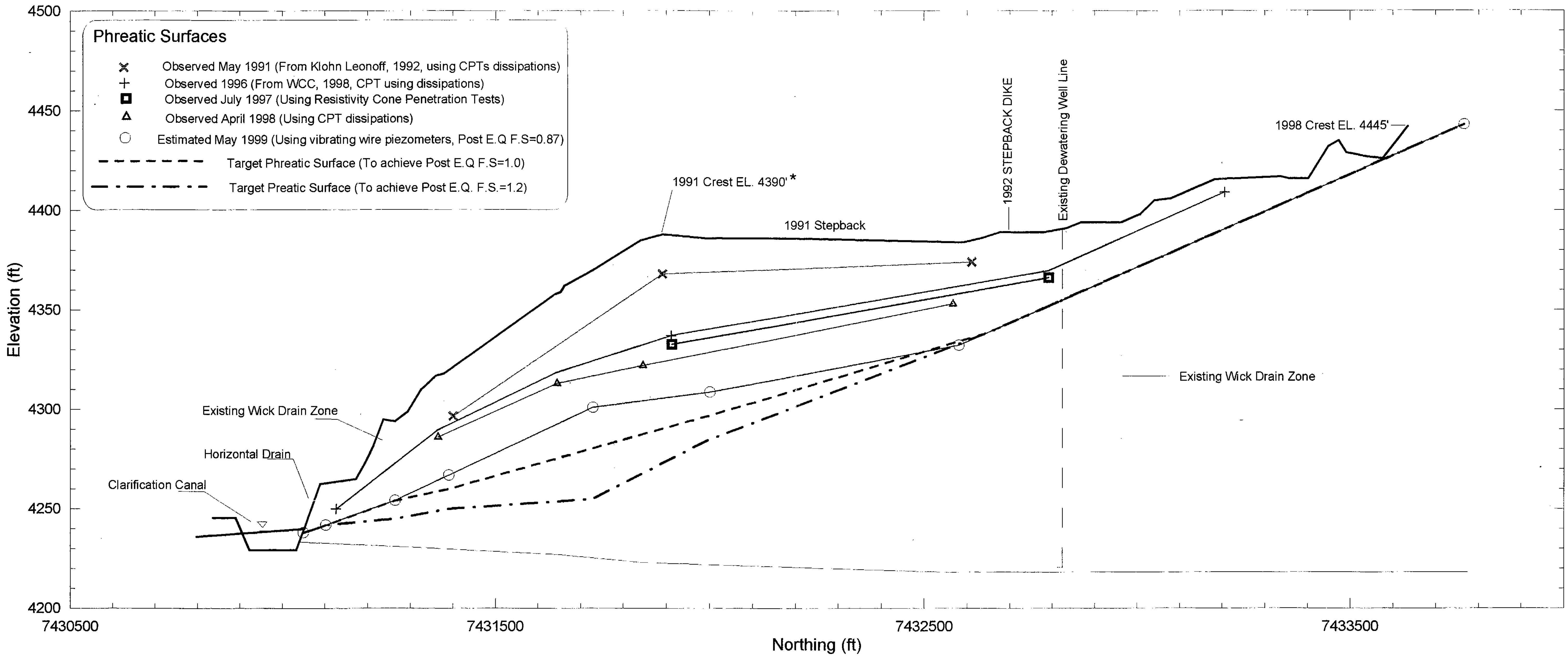
[6] Assuming no liquefaction (PGA varies from 0.14g to 0.17g)

[7] Based on an ultimate height of 4467 feet

[8] Analyses not performed

[9] With West Abutment Berm complete.

Study Section SE2



Note:

1) Depth of existing dewatering wells and wick drains were provided by KUC.
2) Phreatic surfaces for May 1991 and 1996 were observed along cross-section KLD, 800' west of SE2, and projected onto SE2.
3) Slope geometry is taken from the April 1998 topo map.
4) Variation of the phreatic surfaces between the observed locations was assumed to match the general trend of the phreatic surface.
5) Foundation contact is based on 1998 cone penetration tests.

* NOTE: Slope around 1991 crest has been regraded since 1998 topo.

Project No. 24462	Kennecott Utah Copper South Slope	PROGRESS OF DEWATERING ALONG SOUTH SLOPE STUDY SECTION SE2	Figure 4.2
URS Greiner Woodward Clyde			

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