POST-LIQUEFACTION RESIDUAL STRENGTH ASSESSMENT
OF THE LAS PALMAS, CHILE TAILINGS FAILURE

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Tristan Reyes Gebhart
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COMMITTEE MEMBERSHIP

TITLE: Post-Liquefaction Residual Strength Assessment
of the Las Palmas, Chile Tailings Failure

AUTHOR: Tristan Reyes Gebhart

DATE SUBMITTED: September 2016

COMMITTEE CHAIR: Robb Eric S. Moss, Ph.D., P.E.
Associate Professor of Civil and Environmental
Engineering

COMMITTEE MEMBER: Gregg Fiegel, Ph.D., P.E., G.E.
Professor of Civil and Environmental Engineering

COMMITTEE MEMBER: Nephi Derbidge, P.E.
Lecturer of Civil and Environmental Engineering

ABSTRACT

Post-Liquefaction Residual Strength Assessment of the Las Palmas, Chile Tailings Failure Tristan Reyes Gebhart

Assessment of post-liquefaction residual strength is needed for the development of empirically-based, predictive correlations for earthquake engineering design. Previous practice commonly assigned negligible strengths to liquefied materials for engineering analysis, producing overly-conservative designs. Increasingly available case history data, and improved analytical tools have allowed for more accurate and less overly-conservative estimation of soil residual strength, improving empirical predictive models. This study provides a new case history to the limited suite of (approximately 30) liquefaction failure case histories available for post-liquefaction in-situ strength predictive correlations.

This case history documents the Las Palmas gold mine tailings dam failure, resulting from seismic-induced liquefaction during the moment magnitude 8.8 February 27, 2010 Maule, Chile earthquake; the sixth largest since 1900. Forensic analysis provides reasonably well-constrained values of 1) back-calculated representative post-liquefaction residual strength, 2) representative penetration resistance, and 3) representative vertical effective stress along the suspected liquefied failure surface.

This study employs the incremental momentum method to incorporate momentum effects of a moving soil mass. The incremental momentum method requires a series of cross sections animating the geometry of failure progression from initiation to termination, converging on the observed final geometry. Using interpreted soil strength characteristics, an iterative procedure approximates the back-calculated value of post-liquefaction residual strength.

Findings of this case history plot well with existing empirical deterministic regression charts and are in general agreement with previous, related efforts. Results yield representative, well-constrained values of: 1) post-liquefaction residual strength ≈ 173 psf, 2) penetration resistance of $N_{1,60,CS} \approx 5$ and $N_{1,60} \approx 2.5$, and 3) vertical effective stress $\approx 4,300 \text{ lb/ft}^2$, or $\approx 2.0 \text{ atm}$.

Keywords: Earthquake engineering, liquefaction, residual strength, tailings dam

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1 INTRODUCTION

1.1 Earthquakes

Earth is largely shaped and formed by forces of tectonic movement. The material composition of earth undergoes a constant cycle, as actions of plate tectonics converge, diverge, and translate portions of earth like a giant puzzle. This process occurs at various rates, with relative horizontal movement of plates on the approximate order of 0 to 100 mm annually (Press, F. and Siever, R., 2001). The dynamic interaction of plate boundaries generates internal forces and subsequent movement within the earth's lithosphere, resulting in development and build-up of strain energy. This deformational strain energy occurs primarily within the earth's crust and upper mantle, applying both compressional and tensional forces. Energy buildup transposes to kinetic energy, and sudden rupture, or slippage along fault geometry triggers the event of an earthquake.

Figure 1: Elastic Rebound Theory of an earthquake (from Incorporated Research Institutions for Seismology, www.iris.edu, June 2016)

Earthquakes are naturally occurring phenomena, showing a historic pattern of location primarily aligned with earth's major tectonic plate boundaries. The occurrence of earthquakes in densely populated areas of the world adversely effects the economical and societal aspects of many cultures, disrupting vital lifeline infrastructure in built environments. Failure of engineered facilities is largely a function of the quality of design and construction, governed by compliance with regulatory codes and laws established to mandate a minimum level of competency and safety. The unfortunate reality is many underprivileged societies lack financial stability and adequate judicial action required to enforce such engineering codes.

The destructiveness of earthquakes necessitates advanced knowledge of the physics and science at work, to better design and build for the future. To advance the development of earthquake engineering, lessons of past events are incorporated into current engineering methodology. This is partially accomplished through efforts of case history research; requiring identification of pertinent earthquakes, gathering essential information, cataloging principle findings and lessons, and synthesizing this knowledge to draw conclusions that benefit engineering practice.

1.2 Soil Liquefaction Overview

Liquefaction is one of the costliest earthquake effects, drawing an increase in attention over the past three decades. Liquefaction-induced failure of critical facilities warrants further understanding. Researchers, engineers, and scientists are progressing towards understanding the phenomenon, conditions that lead to its occurrence, the likelihood of

“triggering,” and the resulting consequences. Soil liquefaction engineering is its own sub-specialization.

Figure 2: Liquefaction-induced sand boils, Christchurch, New Zealand (Gebhart, 2011)

Concerted engineering study of liquefaction dates back to the effects of two major earthquakes, occurring in a three month period in 1964: Good Friday earthquake ($M_w=9.2$) in Alaska followed by the Niigata earthquake ($M_w=7.5$) in Japan. Both events saw significant seismically-induced soil liquefaction damage, including bridge and building foundation failures, slope failures and flotation of buried structures. The occurrence of liquefaction was previously known, however no well-established means of addressing it yet existed.

Figure 3: Liquefaction-induced failure of structure, Christchurch, New Zealand (Gebhart, 2011)

Progress ensued over the following decades. Initially, the primary focus was risk assessment of triggering, or the initiation of liquefaction. Soils deemed likely to liquefy during seismic excitation were typically assigned negligible post-liquefaction strength values. This is a conservative approach, as post-liquefaction residual shear strength of soil is often greater than zero. Advancements including improved analytical tools for estimation of displacements and deformations, have allowed more accurate, and subsequently less overly-conservative estimation of residual soil strength.

1.3 Statement of Research

The primary purpose of this research is to provide a case history of the Las Palmas gold mine tailings dam failure, which occurred during the M_w 8.8 earthquake on February 27, 2010, in the greater vicinity of Constitucion, Chile. The constructed tailings dam failed

from seismic-induced liquefaction. Forensic analysis of this event provides useful information regarding liquefaction engineering and residual strength. The author intends for this case history to provide a transparent, complete, and sound assessment of the event, providing information and conclusions to improve earthquake engineering development of empirical-based predictive correlations for in-situ evaluation of post-liquefaction residual strength.

The study of liquefied residual strength is an evolving field, originating three decades ago. As higher-order analyses methods are established and utilized, in combination with an increasing amount of data, assessment of post-liquefied strength and, ultimately, empirical predictive models are improved. Available data for residual strength back-analysis is currently limited to approximately 30 vetted case history studies. These studies are based upon varied amounts and quality of data, with some case histories more informative than others.

This study expands the suite of liquefaction failure case histories for post-liquefaction in-situ strength correlations. A method recently developed by Weber (2015) for back-analysis of residual strength, termed the “incremental momentum method,” allows for accurate and reliable modeling by capturing momentum effects of a moving soil mass. Through comparison of pre-failure tailings dam geometry to post-failure geometry, using interpreted soil strength characteristics, an iterative procedure approximates the back-calculated value of post-liquefaction residual strength.

2 LITERATURE REVIEW

2.1 Overview

This chapter provides a synopsis of previous efforts related to pertinent topics of this study. Much of this report builds on the work of others who established what is known today regarding liquefaction earthquake engineering and post-liquefaction residual strength. This chapter does not attempt to encompass the entirety of prior work, rather discusses the most significant material to inform and prepare the reader for the findings of this report.

Topics discussed include an introduction to soil liquefaction, different soil material type susceptibility to liquefaction (with a focus on low plasticity silts), assessment of in-situ liquefaction “triggering,” post-liquefaction residual strength, and the construction of tailings dams.

2.2 Seismically-Induced Soil Liquefaction

According to Kramer (1996), the term “liquefaction” was originally coined by Mogami and Kubo (1953), and was used in the context of a variety of phenomena occurring in saturated, cohesionless soils under undrained conditions. A simplified description of liquefaction follows: generation of excess pore pressure occurring primarily in saturated, cohesionless soils under undrained conditions, resulting from monotonic (static) or transient (cyclic) loading, causing effective stress in soil to rapidly decrease. When the

capacity of soil effective stress resistance reduces below driving stress demand, soil failure can occur.

Liquefaction phenomena can be grouped into two main categories: local instability and global instability. Local instability refers to situations where static driving shear stress does not exceed liquefied soil strength, producing smaller displacements including rotational, bearing, slumping, volumetric compression, settlement, and sand boil ground loss. Global instability refers to situations where static driving shear stress exceeds liquefied soil strength, producing much larger displacements including flow, lateral spreading, and larger versions of local failure mechanisms. An example of global instability is the flow failure of the Las Palmas tailings dam in Chile, resulting in extensive deformation. Deformations can occur after ceasing of strong ground motions and related cyclic inertial forces; such forces are considered less contributing to “driving” the deformations and much more influential in initial “triggering.” Once triggered, progression of flow failures are driven primarily by static shear stresses.

Development of this topic led to establishing a concise flow chart (Figure 4) by Seed, Cetin & Moss, et al. (2003) of key elements of liquefaction engineering. This case history targets stage 2, assessment of post-liquefaction strength. This assumes that liquefaction “triggering” is satisfied according to stage 1. Stage 3 covers assessment of expected liquefaction-induced deformations and displacements; this is a “gray” area in need of further advancement. This stage can be negated in situations where deformations are simply considered unacceptably “large enough” to warrant mitigation; hence accurate displacement estimates are superfluous. There are limited engineering tools for Stage 4,

assessment of consequences of deformations and displacements. If the outcome of this stage is unsatisfactory, stage 5 may be warranted. Liquefaction mitigation is often expensive and complex. To address this, cost/benefit analysis can rely heavily upon not only initial likelihood of triggering, but also estimation of consequence. Some situations do not warrant the need for engineering mitigation, for example a soil mass deemed likely to fail, but without consequence.

Figure 4: Liquefaction engineering flow chart (from Seed, Cetin & Moss, et al., 2003)

2.3 Liquefaction Susceptibility of Soil Materials

In terms of “classic” liquefaction (significant loss of strength due to cyclic pore pressure generation), it is long understood that clean, sandy soils with low fines content and low plasticity are most susceptible. Discussion of liquefaction potential in fine grain silty/clayey soils increases in complexity, especially with the wide variability of silt behavior. Gravelly material can generate cyclic pore pressure, but due to the mass of

larger particles and depositional characteristics, gravels seldom deposit gently in a loose state and large voids typically allow for rapid dissipation of excess pore pressure. For these reasons, liquefaction in this material type is less problematic, unless a cap of finer, less pervious materials defeats drainage.

Liquefaction susceptibility of fine grain soils is the focus of more recent studies. Soils with significant fines content generally do not develop excess pore pressure as rapidly as granular material, therefore are less prone than “clean” cohesionless soils. However, silty soils of very low plasticity exhibit susceptibility to liquefaction triggering at relatively lower shear strains (3-6%), and soils of higher plasticity may exhibit similar characteristics, although requiring larger shear strains (Seed, Cetin & Moss, et al., 2003). However, pore pressures are typically lower with less loss of strength.

Two key soil parameters in classifying fine-grained material, and subsequently liquefaction triggering potential, are: 1) fines content and 2) plasticity index. The “Modified Chinese Criteria” depicted in Figure 5 from Wang (1979) and Seed & Idriss (1982) was a primary reference for defining liquefaction potential. According to Figure 5, soils plotting above the A-line are considered potentially liquefiable if the following three conditions are met: 1) fines content (<0.005 mm) is less than 15%, 2) liquid limit is less than 35%, and 3) in-situ water content exceeds or equals 90% of liquid limit.

1. Percent Finer than 0.005mm $\leq 15\%$
2. Liquid Limit (LL) $\leq 35\%$
3. Water Content (W) $\geq 0.9 \times LL$

Figure 5: Modified Chinese Criteria (from Wang, 1979, and Seed & Idriss, 1982)

The database presented by Wang (1979) was re-evaluated by Andrews & Martin (2000) including data from additional earthquakes, and changed to agree with U.S. fines content convention (<0.002 mm). Figure 6 presents a summary of their findings.

	Liquid Limit ¹ < 32	Liquid Limit ≥ 32
Clay Content ² < 10%	Susceptible	Further Studies Required <i>(Considering plastic non-clay sized grains – such as Mica)</i>
Clay Content ² $\geq 10\%$	Further Studies Required <i>(Considering non-plastic clay sized grains – such as mine and quarry tailings)</i>	Not Susceptible

Notes:

1. Liquid limit determined by Casagrande-type percussion apparatus.
2. Clay defined as grains finer than 0.002 mm.

Figure 6: Liquefaction susceptibility of silty and clayey sands (from Andrews & Martin, 2000)

Liquefaction potential in fine-grained soils became more evident after two major earthquakes in 1999: Kocaeli (Turkey) earthquake and Chi-Chi (Taiwan) earthquake. Both involved damage in soil materials more cohesive than advised as potentially liquefiable by the Modified Chinese Criteria. This spurred additional field and lab testing to better define potential triggering criteria in fine grain soils.

According to Seed, Cetin & Moss, et al. (2003), there is a greater transition in behavior of material than previously assumed (refer to Figure 7). An increase in fines content and plasticity adds ductility to soil over a larger range of material type, as “classic” liquefaction transitions to potential cyclic softening for sensitive clays. It was found that percent clay-sized particles is not the governing characteristic, rather the percent clay mineral content is responsible for plasticity. Quartz particles, for example, exist in small, fine size, however this mineral type is typically non-plastic. This observation can produce unconservative results, as materials with significant fines are deemed unlikely to liquefy, however significant quartz mineral composition will not combat liquefaction to the degree expected. Case history research shows material composition with sufficient fines content to separate coarser particles (15–35% fines) is controlled by characteristics of the fines (Seed, Cetin & Moss, et al., 2003). Additionally, gradation of soil composition substantially influences liquefaction susceptibility. For example, poorly-graded soils have larger void ratios than well-graded and subsequently require greater fines content to negate liquefaction potential.

Figure 7: Recommendation for assessing liquefiable soil types (from Seed, Cetin & Moss, et al., 2003)

A particular material type challenging to understand regarding soil strength criteria are low to non-plastic silts, typically high in fines content but lacking plasticity. This material composition can have a high potential for liquefaction triggering and slow dissipation of excess pore pressures due to low permeability, and is the material type responsible for failure of the Las Palmas tailings dam.

2.3.1 Low Plasticity Silt Strength Characterization

Behavior of silt materials are commonly considered as middle ground between granular sand and fine grain clay. Such convenient assessment is quickly dispelled when detailed strength characteristics and failure criterion are required. The variability of silts includes a wide range of stress-strain and pore pressure response. Silts are also more prone to compression by static pressures than sands, and more prone to settlement and densification from vibratory motion than clay. A paper by Brandon, Rose & Duncan

(2006) delves into the complexities of silt behavior, providing practical assessment. Work by Duncan, J.M., Wright, S.G., Brandon, T.L. (2014) also addresses the challenges involved with silt material.

Adding to the difficulties of material acquisition and laboratory testing is the ease which samples are disturbed. A study by Fleming & Duncan (1990) of silt from the Alaskan arctic found disturbance reduced undrained strengths measured in UU tests by up to 40% and increased undrained strengths measured in CU tests by up to 40%. Development of strength characteristics and failure criteria are complicated as a result.

Non-plastic silts tend to dilate during shear, initially developing positive pore pressure then transitioning to negative at increasing strains, according to Brandon, Rose & Duncan (2006) and Duncan, J.M., Wright, S.G., Brandon, T.L. (2014). Rose (1994) provides an example of this material response, depicted in Figure 8, bearing similar resemblance to laboratory CU testing of Las Palmas tailings shown later (Figure 46). This occurrence can complicate triaxial testing with low cell pressure in UU testing and low back pressure in CU testing, resulting in loss of saturation and the possibility of cavitation. Cavitation is a function of air dissolved in the pore water, with often erratic results, occurring in some specimens but not others, even though the specimens appear identical. If cavitation occurs, the effective confining pressure remains constant, while the deviator stress is maximized. In the absence of cavitation, pore pressure decreases, effective confining pressure increases, and deviator stress continues to increase. Observation of such behavior has reached strains up to 30%, with deformation of specimens exceeding practical interpretation of the results (Brandon, Rose & Duncan, 2006).

Figure 8: Effect of cavitation on undrained strength of reconstituted silt (from Rose, 1994)

The work of Brandon, Rose & Duncan (2006) focuses on establishment of failure criteria for undrained strength through examination of laboratory testing. Their testing used numerous approaches to define failure (see Figure 9), each with its difficulties. Referencing the peak deviator stress as a failure criterion produces wide scatter, resulting from cavitation. Use of the peak principal stress ratio as a failure criterion also produces wide scatter as the stress path follows closely to effective stress failure envelope. Use of peak pore pressure is overly conservative; note on Figure 9 effective stresses are below the effective stress strength line at the location where pore pressure is maximum. Lastly, Brandon, Rose & Duncan (2006) suggest Skempton's pore pressure parameter ($\bar{A}$), indicative of a soil's inclination for dilation (defined in Eq. 1), should be equated to zero for strength parameter evaluation, to ensure values are not selected from negative changes in pore pressure.

$$\bar{A} = \frac{\Delta u}{(\Delta \sigma_1 - \Delta \sigma_3)} \quad (Eq. 1)$$

(σ_1 = major total principal stress, σ_3 = minor total principal stress, u = pore pressure)

Figure 9: Idealized stress path showing stresses at failure for different failure criteria (from Brandon, Rose & Duncan, 2006)

2.4 In-Situ SPT Liquefaction Triggering Assessment

The first step in the liquefaction engineering flow chart depicted in Figure 4 is assessment of liquefaction “triggering” (initiation). This is the most understood and developed stage of liquefaction engineering. A variety of methods are available to evaluate triggering potential based on 1) laboratory testing of “undisturbed” samples and 2) in-situ field testing, such as Standard Penetration Testing (SPT), Cone Penetration Testing (CPT), Shear Wave Velocity (V_s), and Becker Penetration Test (BPT). Although laboratory testing has its benefits, in this context it is complicated by sample disturbance and scaling effects. Due to its wide-spread, common use in industry practice and the available field data for this report, SPT is the focus of this study for in-situ liquefaction triggering assessment and the development of empirical-based predictive correlations.

Estimating the susceptibility and resistance of soil to liquefaction involves two main parameters: 1) cyclic stress ratio (CSR) representing the seismic demand on soil and 2) cyclic resistance ratio (CRR) representing the capacity of soil to resist liquefaction. Seed and Idriss (1971) established the following simplified equation for calculating CSR:

$$CSR = \frac{\tau_{av}}{\sigma_{vo'}} = 0.65 * \frac{a_{max}}{g} * \frac{\sigma_{vo}}{\sigma_{vo'}} * r_d \quad (Eq. 2)$$

(a_{max} = peak ground acceleration (PGA) generated by the earthquake, g = gravitational acceleration, σ_{vo} and $\sigma_{vo'}$ = total and effective stress, respectively, and r_d = stress reduction coefficient at depth.)

Standard Penetration Testing was first employed for evaluating liquefaction potential after the two major 1964 earthquakes in Alaska and Japan, discussed in chapter 1.2. Seed and Idriss (1971) established initial research into evaluation of liquefaction occurrence as a function of SPT blow count and cyclic stress ratio (CSR). Many efforts contributed to the progression of this topic, including development of one of the most widely accepted and used SPT deterministic correlations by Seed et al. (1985), presented in Figure 10. This figure includes regression as a function of fines content. This relationship is dated, however, and does not include more recent case history data (after 1984) to improve the model. Youd et al. (2001) advanced this research through development of improved SPT based correlations.

Figure 10: SPT clean-sand base curve for Magnitude 7.5 earthquakes from liquefaction case history data (modified from Seed et al., 1985)

Further research by Cetin et al. (2004) developed both probabilistic and deterministic SPT-based correlations. These advanced regression models improved the shortcomings of previous studies by including more recent case histories, implementing advanced knowledge regarding both SPT and site-specific ground motions (including improved evaluation of peak horizontal ground acceleration), utilizing improved methods for calculating in-situ cyclic stress ratio (CSR), selectively screening case histories regarding data quality, and using higher-order probabilistic tools to capture uncertainty. As a result,

the new correlations provided reduction in overall uncertainty, improving the empirical SPT-based liquefaction triggering correlations, presented as Figure 11.

Figure 11: Probabilistic (left) and deterministic (right) SPT-based liquefaction triggering correlation ($M_w=7.5$ and $\sigma_v'=1.0$ atm) (from Cetin et al., 2004)

2.5 Post-Liquefaction Residual Strength Assessment

Assessment of post-liquefaction residual strength is a critical part of the development of empirically-based predictive correlations for earthquake engineering design. For the past few decades, it was common practice to assume negligible strengths assigned to liquefied materials for engineering analysis, producing overly-conservative designs. With the continued occurrence of earthquakes and subsequent accumulation of available case

history data, this field of study is developing; however, this collection of case histories still remain rather limited.

2.5.1 Laboratory Testing

Numerous approaches exist for evaluating post-liquefaction residual strength characteristics of soil, each with its own challenge. Laboratory efforts suffer from scaling issues as full-scale conditions in the native environment are nearly impossible to mimic on small-scale. An example is localized void redistribution under globally undrained shearing, discussed by Park (2013). Past research includes centrifuge testing for modeling upward seepage and water film formation beneath a lesser-permeable soil, restricting excess pore pressure dissipation initiated during seismic excitation (see Figure 12). This occurrence is responsible for delayed strain localization failures and subsequent deformations, as was exemplified during flow failure of the Lower San Fernando Dam in 1971. Large deformation of the dam is not believed to be co-seismic, rather a 20 to 30 second failure delay indicates post-seismic void redistribution from upward water migration. A similar occurrence of post-earthquake induced void redistribution is suspected during the Niigata earthquake in 1964. Lab testing cannot effectively capture this phenomenon, further supporting other justifications for use of full-scale case history forensics.

Figure 12: Void redistribution induced strain localization failure (from Park, 2013)

2.5.2 Full-Scale Case History Evaluation

An alternative to scaled laboratory research is back analysis of case history events, capturing full-scale physics in the native environment. The evolvement of earthquake engineering employed numerous methods to capture the post-liquefaction residual strength for development of useful correlations. Currently there are approximately 30 case histories available for use in this realm, with varying levels of detail and completeness of recorded information warranting different analytical approaches.

Seed (1987) and Seed & Harder (1990) performed the original post-liquefaction residual strength research. Back-calculated residual strengths from 12 case histories were correlated with SPT resistance, with 5 cases later added (Figure 13). To (implicitly) incorporate momentum effects, selection of residual strength values were between those back-calculated using pre-failure and post-failure geometry. Seed (1987) corrected SPT $N_{1,60}$ blow counts for fines content using Table 1 and Equation 3.

$$(N_1)_{60-CS} = (N_1)_{60} + \Delta(N_1)_{60} \quad (Eq. 3)$$

Table 1: Fines content adjustment (from Seed, 1987)

Fines Content (%)	SPT Correction, $\Delta(N_1)_{60}$ in blows/ft
0	0
10	1
25	2
50	4
75	5

Figure 13: Relationship between residual undrained shear strength (S_r) and equivalent clean sand SPT blow count (from Seed & Harder, 1990)

Stark & Mesri (1992) concluded post-liquefaction strength was likely linearly dependent upon the initial vertical effective stress ratio along the failure surface. An additional 3 case histories expanded the original suite of 17, and calculation of the residual strength to average initial vertical effective stress along the failure surface provided new correlations. This approach evoked contrast between two diverging theories; the first is

classical critical state theory (depicted in Figure 14), based on Casagrande's (1936) original work, showing that when sheared to large strains all similar specimens approached the same density and continued to shear with constant shearing resistance. Further work by Castro and Poulos (1977) and Poulos (1981) defined the *steady state of deformation*, proposing this at large strains would depend only on relative density of the soil. The second theory is based on assumed constant strength ratio, analogous to the concept of SHANSEP for clay material. Ultimately, regression analysis on correlations put forth by Stark & Mesri (1992) yielded weaker trends, however the relationship of these two parameters remain an important concept.

Figure 14: Critical state theory (from Park, 2013)

Olson & Stark (2002) further expanded the group of case histories to 33 (Figure 15). To explicitly account for momentum effects a “kinetics” method was employed, based on an approach implemented by Davis, et al. (1988), for the 10 field case histories deemed most suitable. This method addresses that progression of a failing mass will initially accelerate to a point, after which the mass will decelerate and eventually stop (Figure 16). Initially, the static driving shear stress demand must exceed the undrained steady state (residual) strength resistance for large deformation failure to occur (post-earthquake shaking). The failing mass will accelerate under a net positive static driving shear stress, until the undrained steady state (residual) strength mobilized within the liquefied material overcomes driving stresses. At this instant only, inertial forces are negated. Deceleration continues until inertia of the failing mass is entirely overcome by the mobilized residual strength, thereby halting movement. The inertial influence of momentum is only absent at the transition from acceleration to deceleration, at which point a static limit equilibrium method (LEM) analysis would theoretically calculate the post-liquefied residual strength. The caveat here is the subjectivity and difficulty in reliably establishing the geometry at this zero inertia transition point, which is subsequently discussed.

Figure 15: Empirical relationship for estimation of normalized residual strength ratio as a function of SPT blow count (from Olson & Stark, 2002)

Figure 16: Failure dynamics of moving mass (from Davis, et al., 1988)

Wang (2003) with the assistance of Kramer (2008) reconsidered previous available case histories for developing new probabilistic relationships between post-liquefaction strength and SPT resistance (Figure 17). A new approach termed the “zero inertial factor” (or ZIF) method, based on the principles implemented by Davis, et al. (1988), attempts to model geometry at the zero inertial transition between acceleration and deceleration. Although convenient, the zero inertial point cannot be assumed at the midpoint of progressed failure, as the trend is not symmetrical. Through these studies, the zero inertial point appears to occur before half of the overall displacement. This is explained through observation: driving static shear stresses diminish through development of failure, as geometry reduces in height and flattens to accommodate progression toward equilibrium of imbalanced forces. The nine highest quality cases were back-analyzed using this ZIF method, with results generally in good agreement with other methods to follow.

Figure 17: Recommended deterministic residual strength values (S_r) as a function of SPT resistance and initial effective vertical stress (from Kramer, 2008)

Kramer, S.L., and Wang C. (2015) then presented a third type of residual strength estimation model, in addition to previous approaches assuming residual strength is either: 1) a function of penetration resistance alone, or 2) proportional to initial vertical effective stress. This new model predicts residual strength as a nonlinear function of both penetration resistance and initial vertical effective stress, depicted in Figure 18. Resulting correlations allow estimation of the probability distribution of residual strength, thereby avoiding conservative estimation imposed by many deterministic models. A fines correction is not included in the model, as this did not result in improved residual strength predictions. Also, including a void redistribution potential indicator term accounting for the likelihood of void redistribution did not improve the predictive model, supporting the theory that void redistribution effects occur to some degree in most flow failure case histories. Lastly, this new model predicts greater residual strengths at low initial vertical effective stresses than previously correlated by normalized models (the opposite also holds true) and similar to values predicted by direct models (Figure 18).

Figure 18: Predicted residual strength as a function of initial vertical effective stress for (a) $(N_1)_{60}=5$, (b) $(N_1)_{60}=10$, (c) $(N_1)_{60}=15$, and (d) $(N_1)_{60}=5$ and 10 with minimum strengths implied by lateral spreading case histories and flow failure case histories with low initial vertical effective stresses (from Kramer, S.L., and Wang C., 2015)

Weber (2015) developed the “incremental momentum method” explicitly including momentum effects and requiring a combination of engineering judgement and geometric rendering, to generate a series of cross sections animating progressive failure from initiation to termination. This method was selected for use in this current study. Initial and final slope geometries are first established, however unlike previous methods, the difficult and subjective task of crafting a zero inertial force section is avoided. Instead, a selected number of estimated incremental time steps are generated to produce an

animated sequence. The sequence must provide a reasonable path throughout the entirety of the failure progression, converging on the established final geometry. Producing this animation is a highly iterative procedure, accomplished through a series of trials.

Back-calculation of residual strength using limit equilibrium methods (LEM) with initial and final geometry cross sections assist to bracket feasible upper and lower bound values. The average of these values is often a reasonable approximation for selection of an initial trial strength value assigned to (saturated) material deemed to have liquefied. This strength value is input for each time step to execute a series of static LEM slope stability analyses. A series of iterative calculations then utilize known slide mass parameters for each time step, and must converge on two key principles: 1) the *calculated* displacement of the sliding mass must converge at the *observed* displacement for each time step, and 2) the velocity of the sliding mass must converge at zero at the final time step. Each time step is converged before proceeding to the next. If the two key principles are not converged upon, the input residual strength is systematically changed, and the procedure repeated. Too large a residual strength value is exemplified in a premature reduction in velocity to zero and too small a displacement; the opposite holds true for too small a residual strength value. The value is selected which best satisfies the established criteria to represent the post-liquefaction residual strength mobilized within the displaced mass. Chapter 6.4 further details this procedure.

Weber (2015) employed the incremental momentum method for re-analysis of previous case histories, implementing lessons provided by previous research efforts, producing a suite of empirical-based probabilistic and deterministic regression analysis (Figure 19 and

Figure 20). These results of unprecedented reliability were based on an improved back-analysis procedure (incremental momentum method), internal cross-checking within the framework of the empirical relationships developed, and external cross-checking against previous investigation results with the knowledge of their strengths and shortcomings.

Figure 19: Deterministic regression showing post-liquefaction strength (S_r) as a function of both penetration resistance and initial effective vertical stress (from Weber, et al., 2015)

Figure 20: Deterministic regression showing post-liquefaction strength ratio (S_r/P) as a function of both penetration resistance and initial effective vertical stress (from Weber, et al., 2015)

2.6 Tailings Dam Material & Construction

Mineral processing operations and subsequent production of mine waste around the world require the disposal of a special type of processed material, termed “tailings.” Placement of such manufactured material presents a special case of slope stability analysis and engineering consideration. A common method for tailings disposal is constructed deposits of tailings dams. The practices often employed for dam construction lack methods of typical construction, including compaction, to establish sufficient engineered soil strength.

Tailings material is typically comprised of sand, silt, or clay-sized particles, processed and broken down from crushing and grinding of parent ore material. According to Turner & Schuster (1996), maximum particle size is commonly between 0.1 to 1.0 mm and a function of processing required for optimum mineral extraction efficiency. This typically produces highly angular particles. In general, clay material is contained within tailings only if present in the parent ore, and typically only for tailings processed through washing operations.

Construction of a tailings impoundment begins with containment wall(s) to corral the placement of tailings material. Although not always the case, walls are ideally built with available granular material and placed with compactive effort to reduce air voids and strengthen construction. Various methods of containment wall construction are employed, including the upstream, downstream, and centerline procedure, depicted in Figure 21. The downstream method requires the greatest material, providing the most

sound and competent containment as each dike extends further away from the pond, extending full height down to solid ground. Upstream method requires the least material, providing the weakest containment as each dike is founded on the previous and not solid ground, extending towards the pond. Centerline method is between these two procedures, as each dike is built vertically on top of the previous, extending away from the pond down to solid ground.

Upstream dam construction historically shows poor performance with greater susceptibility to liquefaction flow events and are solely responsible for all major static liquefaction events worldwide (Davies, 2002). In response, this method is banned in numerous countries to prevent future disasters. In many cases however, abandoned dams built in upstream style remain and pose hazard to surrounding populations.

Figure 21: Methods of containment wall construction (from Chamber of Mines South Africa, 1996)

Placement of tailings material within built containment walls is often via hydraulic fill, with discharge occurring from the embankment perimeter. When granular fractions are present in slurry, heavier sand materials typically deposit close to point of discharge, while finer and lighter “slime” materials carry further away. This can produce a separation or gradation of material types. According to Turner & Schuster (1996), sand tailings are typically loose with relative densities of approximately 30-50%, and corrected SPT $N_{1,60}$ blow counts ranging from 3-5. Aging effects can consolidate and strengthen tailings, yielding increase in blow counts. Slime material typically consists of low-to-non-plastic silts with void ratios of 0.8-1.3, and high specific water retention. This is a significant characteristic of this material, as slimes have remained soft and saturated for decades after initial placement. The slime constituent of tailings provides the greatest problems with slope engineering (Turner & Schuster, 1996).

Figure 22: Material layering of Las Palmas tailings dam (from Bray, et al., 2010)

The nature of construction and material characteristics of tailings dams poses a significant seismic-induced liquefaction hazard to many built facilities around the world. Adding to this issue is the stewardship of these facilities during and after their active use, to help insure an intended design remains in good standing in “perpetuity.” Although somewhat dated, according to USCOLD (1994) no flow failures are known to have occurred (up to that point) for abandoned tailings deposits that did not retain impounded water, under earthquake loadings or otherwise. The history of tailings dams is populated with failures, and through study of case histories, valuable lessons from past should be applied to reduce error in future engineering practice.

3 CASE HISTORY

3.1 Overview

This chapter provides a synopsis of this case history, providing the reader with background knowledge of the earthquake event resulting in the failure of the Las Palmas tailings dam, the methods and procedures employed for construction of the dam, and both observed and inferred characteristics of liquefaction-induced failure of the dam. Detailed information regarding this case history is somewhat limited, therefore requiring engineering intuition to solve missing pieces of the puzzle.

3.2 Earthquake Event

The seismic event for this case history is the February 27, 2010 Maule, Chile earthquake, the sixth largest since 1900, releasing with a moment magnitude 8.8 centered off the coast of Bio Bio, Chile at 3:34 am local time. The hypocenter was located an approximate depth of 21.7 miles (35 km), 60 miles (95 km) off the coast, and 210 miles (335 km) southwest of the capital, Santiago. Maximum recorded acceleration was 0.65g (see Appendix D) at Concepcion, Chile (USGS).

Strong earthquake shaking exceeded a minute in some locations with total duration up to nearly two minutes, commencing at the shallow-inclined fault conveying the Nazca plate eastward and downward beneath the South American plate. Thrust-faulting occurred on the interface between both plates, each converging at approximately 23 ft (7 m) per century. At depth the fault rupture plane exceeded 60 miles (100 km) in width and nearly

300 miles (500 km) parallel to the coast. Rupture initiated beneath the coast and propagated westward, northward, and southward, warping the ocean floor and triggering a tsunami (USGS).

This seismic event loaded many structures, resulting in the pervasive destruction of critical lifeline facilities such as roadways, bridges, railroads and road embankments. In total, approximately 523 people were killed, 12,000 injured, 800,000 displaced and 370,000 houses, 4,000 schools, and 79 hospitals were damaged or destroyed (USGS). Figure 23 depicts an intensity scale of the seismic event.

Figure 23: USGS ShakeMap: Offshore Maule, Chile
 (USGS: <http://earthquake.usgs.gov/earthquakes/eqinthenews/2010/us2010tfan/#maps>, June 2016)

3.3 Construction of Las Palmas Dam

Construction of the Las Palmas gold mine tailings dam began late 1981 (Santa Maria, Ledezma & Bahamondes, 2012). The dam was built upon existing ground sloping down toward the south and east, with approximate maximum upper slope of 4:1 (horizontal to vertical) above the dam and maximum lower slope of 15:1 (horizontal to vertical) below

(see Figure 24). Construction sequence comprised of 4 individual stages, occurring approximately 17 years between 1981 and 1998. Each stage included initial construction of containment walls and hydraulic fill placement of processed earth. Construction of containment walls or embankments typically utilize the sandy, more granular fraction of the tailings material to provide increased strength capacity.

Figure 24: Pre-failure aerial image of Las Palmas tailings dam (Google Earth: www.google.com/earth/, April 2007 image)

Information regarding the construction details of the containment walls are limited, subsequently some level of uncertainty and corollary assumption is understood. Varying sources of information indicate probable use of both upstream and centerline construction methods to build containment walls (Verdugo, R., 2012 and Santa Maria, Ledezma & Bahamondes, 2012).

Stage 1 construction spanned from the end of 1981 to 1986, initiating coverage of the upper-slope half of the final shape. Stage 2 construction spanned from 1986 to 1992, initiating coverage of the down-slope half. Stage 3 construction continued from 1992 to 1997, containing the largest volume of material and spanning the entire length of the final shape, covering both stages 1 and 2. The final construction of stage 4 spanned from 1997 to 1998, covering the top of approximately the same area as stage 1 (Santa Maria, Ledezma & Bahamondes, 2012). Figure 25 depicts the individual construction stages. After closure of the dam, the tailings area were reported as partially covered with a thin 6 inch layer of gravelly material, as depicted in Figure 22 (Bray, J., et al., 2010).

Figure 25: Construction stages 1 through 4 (top through bottom) of tailings dam (from Santa Maria, Ledezma & Bahamondes, 2012)

Figure 26 depicts evidence of separate construction of upper-slope walls for stages 3 and 4, with a referenced aerial view in Figure 27. This establishes two distinct, separate containment walls, further evidenced by the apparent variation in vegetation size (Figure 26), likely indicative of several years between material placement.

Figure 26: Stages 3 & 4 upper-slope containment walls (March 24, 2010). Arrow indicates photo location. (modified after Santa Maria, Ledezma & Bahamondes, 2012)

Figure 27: Aerial view of stages 3 & 4 upper-slope containment walls with failed mass (modified after Santa Maria, Ledezma & Bahamondes, 2012)

Figure 28 delineates the perimeter of the final stage 4 construction.

Figure 28: Perimeter of the final stage 4 construction with failed mass (from Santa Maria, Ledezma & Bahamondes, 2012)

Evidence of the division and elevation difference between construction stages 3 and 4 is visible in Figure 29. The down-slope containment wall for stage 4 was built somewhat vertically aligned with and atop the wall for stage 1 (see Figure 25), depicted as the “step” between stages 3 & 4 shown in Figure 29. This particular location and construction sequence is paramount to understanding and characterizing the dam failure, further discussed next.

Figure 29: Separation of construction stages 3 & 4 shown with arrows 2 & 1, respectively, with failed mass (from Santa Maria, Ledezma & Bahamondes, 2012)

3.4 Failure of Las Palmas Dam

The probable sequence of construction resulted in a weakened boundary of the dam, in particular the vertically-aligned, down-slope walls containing stages 1 and 4 (see Figure 25). Limited available information indicates the down-slope wall containing stage 4 was built nearly atop the down-slope wall containing stage 1 (Santa Maria, Ledezma & Bahamondes, 2012). Stage 3 covered the entire dam between placement of downslope walls 1 and 4, providing a discontinuous and potential weakened horizontal plane between the two vertically-aligned walls. Ultimately, this location (shown in Figure 29 through Figure 31) is a dividing boundary, delineating the remaining portion of the dam from that which experienced liquefaction-induced flow failure.

Figure 30: Edge of flow failure (from Bray, et al., 2010)

Figure 31: Edge of flow failure (2) (from Bray, et al., 2010)

The liquefied flow failure took two paths: an easterly and southerly direction. The leading edge (toe) of the easterly flow traveled approximately 540 ft (165 m), whereas the

southerly flow traveled roughly 1150 ft (350 m) (Google Maps and Santa Maria, Ledezma & Bahamondes, 2012). A total approximated 303,000 yd³ (231,660 m³) of material displaced in roughly two equal halves in both directions (Figure 32). The easterly trajectory flowed in-plane with the length of pre-failure condition, and the southerly trajectory flowed out-of-plane. This “three-dimensional effect” of material movement in two directions, both in-plane and out-of-plane of the orientated length of pre-failure condition, is pivotal in subsequent analysis.

Figure 32: East and South flow directions with approximate equal flow volume

The observed post-failure runout condition, depicted in Figure 33 through Figure 36, was approximately 5-13 ft (1.5-4 m) thick in some locations, according to Santa Maria, H., Ledezma, C., & Bahamondes, F. (2012). Available detailed post-failure condition

geometry is limited, therefore the final displaced shape is subject to interpretation and engineering judgement. Considering the large runout distance, variation in post-failure height is estimated and averaged as approximately uniform throughout. Chapter 5 describes this further.

Figure 33: Tailings flow failure runout (from Bray, et al., 2010)

Figure 34: Tailings flow failure runout (2) (from Bray, et al., 2010)

Figure 35: Tailings flow failure runout (3) (from Bray, et al., 2010)

Figure 36: Tailings flow failure runout (4) (from Yasuda, et al., 2010)

Boring logs indicate groundwater was located between depths of 17-40 ft (5-12 m) below ground surface across the undisturbed portion of the tailings, investigated during post-failure condition (Santa Maria, Ledezma & Bahamondes, 2012). The presence of groundwater is further evidenced in Figure 37, depicting groundwater exiting the bottom

of the exposed failed slope. This saturated, lower layer of tailings material is likely the location of seismic-induced liquefaction triggering, and further discussed in chapter 5.7.

Figure 37: Groundwater seepage from base of failed slope at Las Palmas dam (from Villavicencio, et al., 2013)

The appearance of sand boils is common evidence of the occurrence of liquefaction, as can be seen in Figure 38 & Figure 39. This phenomenon was observed on site in numerous locations as a result of the fluidization of saturated tailings and the resulting migration and release of excess pore pressure upward to the ground surface.

Figure 38: Evidence of sand boils (from Bray, et al., 2010)

Figure 39: Evidence of sand boils (2) (from Bray, et al., 2010)

4 FIELD INVESTIGATION, LABORATORY TESTING, MATERIAL CLASSIFICATION

4.1 Overview

An initial, on-site field investigation performed in-situ testing to characterize sub-surface materials and obtain (as best as possible) representative samples, both disturbed and relatively undisturbed. Laboratory testing of these samples assessed soil characteristics, providing essential information for synthesis of engineering analysis, design and recommendations.

A desk study of previous efforts included all available field investigation and laboratory testing data acquired from report by Santa Maria, Ledezma, & Bahamondes (2012). The majority of this report was translated to English language for this study. In addition, this report includes field investigation information provided by the Geo-Engineering Extreme Events Reconnaissance (GEER) Association report by Bray, et al. (2010).

4.2 Field Investigation

The field reports referenced by Bray, et al. (2010) and Santa Maria, Ledezma, & Bahamondes (2012) were conducted in March 2010 and June 2011, respectively. Use of multiple in-situ techniques characterized existing site conditions and obtained relatively undisturbed samples for analysis. Field testing included the Standard Penetration Test (SPT), (Dynamic) Cone Penetration Test (CPT), Vane Shear Test (VST), and Spectral Analysis of Surface Waves (SASW) method.

4.2.1 Standardized Field Test Methods

The Standard Test Method for Penetration Test and Split-Barrel Sampling of Soils (ASTM D1586-11), known as the Standard Penetration Test (SPT), obtains field indication of the in-situ density of the soil and provides visual observation of a portion of the soil column. Samples are obtained with a split spoon sampler, driven in the ground by a 140-pound hammer free-falling 30 inches. The sampler is initially seated six inches to penetrate loose cuttings at the base of the test boring, then driven an additional 12 inches. The blow count per foot to advance the sampler the final 12 inches, known as N-value, is recorded.

The Standard Test Method for Performing Electronic Friction Cone and Piezocone Penetration Testing of Soils (ASTM D5778-12), known as the Cone Penetration Test (CPT), uses an electric cone pushed into the ground to obtain a near continuous soil behavior profile and in-situ measurement of the cone bearing tip resistance, sleeve friction, and pore water pressure using a pore pressure transducer.

The Standard Test Method for Field Vane Shear Test in Saturated Fine-Grained Soils (ASTM D2573/2573M-15), known as the Vane Shear Test (VST), involves placing a four-bladed vane into the soil and rotating the device to determine the required torque to shear a cylindrically-shaped surface. This value is converted to a unit shearing resistance of the failure surface. The intention of this field method is for estimation of undrained shear strength of soil, valid only for fine-grained soils exhibiting an undrained shear response.

The Spectral Analysis of Surface Waves (SASW) method measures in-situ shear wave velocity profile for a column of soil and/or rock. This method involves dispersive characteristics of surface waves to estimate variation of shear wave velocity with depth. This information typically applies to seismic site class characterization and liquefaction analysis.

4.2.2 Standardized Field Test Results

The field investigation in Santa Maria, Ledezma, & Bahamondes (2012) conducted Standard Penetration Testing between June 2-26, 2011. Five, 4 inch diameter exploratory borings were advanced to depths ranging from 28 to 70 ft (8.5 to 21 m) below ground surface, typically terminating in the competent native material below the tailings dam. These locations are within the non-displaced portion of the dam, indicated on Figure 40. Borings B-1 & B-5 were located within the containment walls, and borings B-2, B-3 & B-4 were located within the tailings material. The drill rig was equipped with a safety hammer utilized to obtain blow counts, with an estimated efficiency of approximately 60 percent. Corrected ($N_{1,60}$) SPT blow counts for fine and coarse-grained materials were between single digits to the lower teens range, depicted in Figure 41. The field crew encountered groundwater in 4 of 5 SPT borings, at depths ranging from 17 to 43 ft (5 to 13 m) below ground surface.

Figure 40: SPT & CPT field investigation locations (values indicate depth to competent native material)

Figure 41: Histogram of all SPT blow counts (all borings)

The field investigation team performed Cone Penetration Testing at 5 locations within the non-displaced portion of the dam, indicated on Figure 40. The advancement of 5 soundings ranged from depths of 33 to 65 ft (10 to 20 m) below ground surface, typically terminating in the competent native material below the tailings dam. Soundings CPT-1, CPT-2, CPT-3 & CPT-5 were located within the containment walls, and the location of CPT-4 is currently unknown. At the time of research, field investigation dates and electronic data files are unavailable for further detailed analysis.

The field investigation team conducted in-situ Vane Shear Testing at intermittent depths in the same borings as B-1, B-2 & B-3, correlating to both containment wall and tailings material locations. Testing occurred both above and below the observed ground water table, primarily in sandy silt (ML) and some silty sand (SM) material. All tailings (non-wall) material test locations were classified as sandy silt (ML) with high fines content, and it is assumed that measurement of undrained shear strength below the ground water table is valid. To address uncertainty of the validity of undrained shear response in the silt material above (and below) the phreatic surface, drained response is estimated through back-calculation of an approximate peak and residual friction angle with use of the following soil shear strength equation:

$$\tau = \sigma' \tan(\varphi') + c' \quad (Eq. 4)$$

(τ = soil shear strength, σ' = normal effective stress, φ' = effective friction angle, c' = effective cohesion).

For this analysis, cohesion is assumed zero (0). In-situ Vane Shear Testing produces a vertical, cylindrically-shaped failure surface, therefore the normal effective stress is the

horizontal stress component at the center of the test depth. It is difficult to accurately assess this condition, and for all practical purposes the normal effective stress is assumed somewhere between the vertical effective stress and half of this value (with at-rest lateral earth pressure coefficient, K_0 , assumed 0.50). The average of these, 0.75, is the (approximate) multiplier of the vertical effective stress used to estimate the normal effective stress. This is a conservative approach, as the use of a horizontal coefficient of 0.50 yields unreasonably large drained frictional strength.

Table 2 presents the test results. Undrained shear strength obtained within *wall* material ranged from approximately 520-1490 psf (peak) and 330-1490 psf (residual). Undrained shear strength obtained within *tailings* material *above* the water table was approximately 780 psf (peak) and 410-670 psf (residual), and *below* the water table ranged from approximately 500-820 psf (peak) and 370-670 psf (residual).

Back-calculated drained frictional strength of the *tailings* material *above* the water table ranged from approximately 25-34° (peak) and 20-22° (residual), and *below* the water table ranged from approximately 16-17° (peak) and 13° (residual). The ratio of peak to residual strength (an indicator of soil sensitivity) within *wall* material ranged from 1.0-1.6 and within *tailings* material ranged from 1.2-1.9.

From this data, only Vane Shear values from testing in the sandy silt (ML) *tailings* material *above* the ground water table are considered for subsequent analysis to model non-liquefied (unsaturated) tailings material.

Table 2: Vane Shear Field Test results

Boring	1 (Wall)					Boring	2 (Tailings)		Boring	3 (Tailings)	
Test	1	2	3	4	5	Test	1	2	Test	1	2
Depth (m)	12	14	5	7	9	Depth (m)	7	13	Depth (m)	5	8
Depth (ft)	39	46	16	23	30	Depth (ft)	23	43	Depth (ft)	16	26
Degrees	Ton/m2					Degrees	Ton/m2		Degrees	Ton/m2	
11.25	2.8	2.2	-	-	-	11.25	2.8	3.0	11.25	2.2	2.6
22.5	2.4	3.4	2.8	3.2	2.8	22.5	4.0	4.4	22.5	4.2	2.7
45	1.8	2.6	2.8	4.6	4.8	33.75	4.2	4.0	33.75	3.8	2.4
67.5	-	-	-	6.8	6.4	45	4.2	4.0	45	2.2	2.0
90	-	-	-	8.0	7.4	56.25	3.6	3.6	-	-	-
101.25	-	-	-	-	8.0	-	-	-	-	-	-
Max (tsm)	2.8	3.4	2.8	8.0	8.0	Max (tsm)	4.2	4.4	Max (tsm)	4.2	2.7
Max (psf)	520	632	520	1486	1486	Max (psf)	780	818	Max (psf)	780	502
Max ϕ (°)	-	-	-	-	-	Max ϕ (°)	25	16	Max ϕ (°)	34	17
Final (tsm)	1.8	2.6	2.8	8.0	8.0	Final (tsm)	3.6	3.6	Final (tsm)	2.2	2.0
Final (psf)	334	483	520	1486	1486	Final (psf)	669	669	Final (psf)	409	372
Final ϕ (°)	-	-	-	-	-	Final ϕ (°)	22	13	Final ϕ (°)	20	13
Max/Final	1.6	1.3	1.0	1.0	1.0	Max/Final	1.2	1.2	Max/Final	1.9	1.4

tsm: tons per square meter

psf: pounds per square foot

The field investigation team from Bray, et al. (2010) performed the Spectral Analysis of Surface Waves (SASW) method, yielding the shear wave velocity profile with depth depicted in Figure 42. The plot shows depth on the y-axis in meters, and shear wave velocity on the x-axis in meters per second. Based on the profile, the site would classify as a NEHRP site class D.

Figure 42: Spectral Analysis of Surface Waves (SASW) method shear wave velocity profile (from Bray, et al., 2010)

4.3 Laboratory Testing

The laboratory testing by Santa Maria, Ledezma, & Bahamondes (2012) included samples obtained during the field investigation using multiple techniques and methods typical in current practice. Characterization of soil parameters such as strength, hydraulic conductivity, and compressibility provide data for further analysis. Ex-situ laboratory testing includes the Standard Test Method for Particle-Size Analysis of Soils (ASTM D422-63), Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils (ASTM D4318-05), Standard Test Methods for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass (ASTM D2216-10), Standard Practice for Classification of Soils for Engineering Purposes (Unified Soil Classification System) (ASTM D2487-11), The Standard Test Method for Unconfined Compressive Strength of Cohesive Soil (ASTM D2166), and The Standard Test Method for Consolidated Undrained Triaxial Compression Test for Cohesive Soils (ASTM D4767-11).

4.3.1 Standardized Laboratory Test Results

Selection of representative soil strength parameters derives from results of laboratory testing performed on relatively undisturbed samples obtained in the field. For this study, selection of lab-based strength parameters are from unconfined compressive strength (UC) testing and consolidated undrained triaxial compression (CU) testing.

Unconfined compressive strength testing of specimens from competent native clayey/silty sand material at location B-1 and depths of 62 ft (19 m) and 67 ft (20.5 m), yield total stress cohesive strengths of approximately 7,000 psf and 19,500 psf,

respectively (Figure 43). Both of these tests failed at significantly low strains of 0.8% and 1.0%, indicative of the brittle behavior of the material. Chapter 5.9.1 discusses this further.

Figure 43: Unconfined Compressive (UC) Strength Test results (SPT-1)

Consolidated undrained triaxial compression (CU) testing of specimens obtained from the silty sand (SM) containment wall at location B-1 yielded a *total* stress friction angle of 26° and cohesive strength of 800 psf, and *effective* stress friction angle of 34° and cohesive strength of 200 psf, depicted in Figure 44. Failure occurred at 5% strain, with deviator stress reaching a relatively constant peak strength with increasing strain, shown in Figure 46 (left side). Testing of the sandy silt (ML) containment wall at location B-5 yields a total stress cohesive strength of 5,600 psf, depicted in Figure 45. Failure occurred at 8.3% strain, with deviator stress reaching a peak strength then decreasing towards a residual strength with increasing strain, shown in Figure 46 (right side).

Figure 44: Consolidated Undrained (CU) Triaxial Test results (SPT-1 @ 20 ft (6 m) depth)

Figure 45: Consolidated Undrained (CU) Triaxial Test results (SPT-5 @ 17 ft (5 m) depth)

Figure 46 depicts measurement of pore pressure development for both CU tests. Testing of both silty sand (SM) and sandy silt (ML) materials exhibit an initial development of positive excess pore water pressure, ultimately reducing with increasing strain to negative pore pressure. This pattern is consistent for both materials, exemplifying common

behavior of silty materials, as described in chapter 2.3.1: Low Plasticity Silt Strength Characterization. However, some concern may be warranted with these results due to potential for cavitation occurring in certain material, when tested under insufficient back pressures. This issue is addressed through close inspection of pore pressure development in Figure 46.

Figure 46: Consolidated Undrained (CU) Triaxial Test results: development of positive & negative pore pressures (Santa Maria, Ledezma, & Bahamondes, 2012)

Cyclic triaxial testing of three specimens from sandy silt (ML) containment wall material at locations B-1 & B-5 exhibit contractive response with development of positive pore pressure under cyclic loading, shown in Figure 47 and Figure 48.

Figure 47: Cyclic triaxial test: excess pore pressure (modified after Santa Maria, Ledezma, & Bahamondes, 2012)

Figure 48: Cyclic triaxial test: excess pore pressure (modified after Santa Maria, Ledezma, & Bahamondes, 2012)

To compare pore pressure response from cyclic triaxial testing of Figure 47 and Figure 48 with CU triaxial testing of Figure 46, Figure 49 compares test results from two specimens from the same location and depth (B-5 at 17 ft or 5 m). Inspection of pore pressure with respect to strain % observes the following: excess pore pressures from CU triaxial testing (Figure 49, top) remain positive until reaching strains of approximately 4%, while excess pore pressures from cyclic triaxial testing (Figure 49, bottom) remain positive. Note that maximum strain from cyclic triaxial testing does not exceed approximately 2%, therefore the two results agree for *lower* strain response, as cyclic triaxial testing did not reach large enough strains to induce negative pore pressure.

Figure 49: Pore water pressure response comparison (location B-5 @ 17 ft) (modified after Santa Maria, Ledezma, & Bahamondes, 2012)

Table 3 provides a summary of all field and lab testing results.

Table 3: Field and lab soil strength test results

Location	Field Test	Location	Depth		Soil Type	Strength		Unit Wt. (pcf)		Fail Strain (%)	Lab Strength Testing			Sample Method
			(m)	(ft)		Cohesion (psf)	Φ (°)	Total	Dry		CU	Cyclic Triaxial	UC	
B-1	SPT	"Wall"	6-6.5	20	SM	800	26	100	95	5.5	X	-	-	Shelby
			16.0-16.5	54	ML	-	-	-	-	-	-	X	-	-
		Bedrock	18.5-19.25	62	SC-SM	7000	-	138	127	0.8	-	-	X	HQ3 cutting tip
			20.0-21.0	67	SC	19500	-	131	116	1	-	-	X	HQ3 cutting tip
	Vane Shear	"Wall"	5	17	SM	520	-	-	-	-	-	-	-	-
			7	23	SM	1486	-	-	-	-	-	-	-	-
			9	30	SM	1486	-	-	-	-	-	-	-	-
			12	40	ML	520	-	-	-	-	-	-	-	-
			14	46	ML	632	-	-	-	-	-	-	-	-
			7	23	ML	780	-	-	-	-	-	-	-	-
			13	43	ML	818	-	-	-	-	-	-	-	-
B-2	Vane Shear	"Bowl"	5	17	ML	780	-	-	-	-	-	-	-	-
	Vane Shear	"Bowl"	8	26	ML	502	-	-	-	-	-	-	-	-
B-3	SPT	"Bowl"	-	-	-	-	-	-	-	-	-	-	-	-
B-4	SPT	"Bowl"	5.0-5.5	17	ML	5600	-	110	96	8.3	X	X	-	Undisturbed
B-5	SPT	"Wall"	10.0-10.5	34	ML	-	-	-	-	-	-	X	-	-

CU: Consolidated Undrained
UC: Unconfined Compressive

4.4 Material Classification

According to the Unified Soil Classification System, representative soil samples obtained during the field investigation and tested in the laboratory are primarily sandy silt (ML) and silty sand (SM). These material types are typical of tailings deposits, with maximum particle size between 0.1 to 1.0 mm, governed by the processing required to extract mineral from ore (Turner & Schuster, 1996). Mechanical grinding can produce highly angular particles, while clay content in tailings is a function of the parent ore composition, and usually only significant if produced by washing operations.

4.4.1 Sandy Silt Material

The presence of high fines content, low plasticity, sandy silt (ML) dominates the composition of the Las Palmas tailings material. Typical fines content varies between 59-

99%, with water content ranging primarily from 10-30%, and a plasticity index between non-plastic to 5. This sandy silt constitutes the entirety of material encountered in boring locations B-2 & B-3, and the majority of material in locations B-4 & B-5, correlating to both the containment walls and tailings, respectfully. Corrected ($N_{1,60}$) Standard Penetration Test blow counts in this material are nearly entirely within single digits range, indicative of very soft to medium stiff consistency. The interface of this material type with occasionally interbedded, low fines content, non-plastic, silty sand (SM) is defined through stark changes in material properties (water content, fines content, plasticity index), evidenced in boring logs B-1, B-4 & B-5 (Appendix A).

4.4.2 Silty Sand Material

Classification of the silty sand (SM) is distinguished by its granular nature, yielding low fines content primarily between 30-50%, with low water content typically 10% and below, and entirely non-plastic. This silty sand constitutes the upper-half of material encountered in boring B-1, and small, singular bands of material encountered in borings B-4 and B-5; this correlates primarily to the containment walls, with some present in the tailings. Corrected ($N_{1,60}$) Standard Penetration Test blow counts are mostly within upper single digits to lower teens range, indicative of loose to medium dense consistency.

4.4.3 Silty Sand Competent Native Material

The pre-existing, underlying competent native material comprises of primarily silty sand, exhibiting a well-defined, unyielding interface with the overlying tailings material. A plasticity index range of 8-12 is significant compared to all other non-to-low-plastic

material encountered and tested, which is likely a characteristic of the material's natural in-situ, un-processed nature. Fines content of the granular material is low, ranging 22-34% and water content between 8-13%.

5 MODELING OF TAILINGS DAM

5.1 Overview

Development of pre and post-failure, 2 and 3-dimensional digital models of the Las Palmas tailings dam utilize AutoCAD Civil 3D (by Auto Desk) and Slope/W (by Geo-Slope International). Advantages of these software packages include, but are not limited to the following:

- 2 and 3-D detailed modeling of the dam geometry, used for earthwork quantities
- Flow failure runout length estimation
- Creation of tailings dam cross-sections
- Static and pseudo-static slope stability analysis

This model serves to depict and clarify the spatial characteristics of the dam construction and ultimately map the progression of its failure.

After establishing a functioning digital model for analysis, the next step is vetting it against available case history data. This is accomplished by comparing height of the tailings dam (or depth to competent native soil) between the digital model and field investigation data. SPT and CPT tests are first located in two-dimensional aerial plan view on the model, and a cross section is “cut” intersecting and connecting SPT and CPT test locations. The depth of the model from the surface of the dam to underlying competent native soil is compared to the depth encountered during the corresponding SPT and CPT field investigations (See Figure 50). The model is adjusted, if necessary, until values are deemed agreeable within a few feet. This digital surface adjustment is not

trivial and can require challenging visual interpretation. Ultimately, each set of measured SPT, CPT, and generated digital model values are converged to establish a representative three-dimensional digital model.

Figure 50: Geometric vetting of computer model against field investigation

5.2 Existing Ground Geometry

Modeling the existing ground, or competent native soil beneath the tailings dam, uses topographic contour mapping provided by Santa Maria, Ledezma, & Bahamondes (2012) and Google Earth. Native topography slopes down toward the south and east, with an approximate maximum upper slope of 4:1 (horizontal to vertical) transitioning to an approximate maximum lower slope of 15:1 (horizontal to vertical). Based on historic aerial photos, the vicinity appears to be farm land preceding construction of the dam.

5.3 Pre-Failure Geometry

Creation of a three-dimensional model of the pre-failure geometry uses historic google earth photos and in-situ SPT and CPT field measurements of depth to competent native material. Selection of a critical cross section for analysis bisects the length of and parallel to the pre-failure condition, shown in Figure 51 and Figure 52. The pre-failure volume contains approximately 537,000 yd³ (410,570 m³), with maximum down-slope height of 74 ft (22.5 m), average middle height of 44 ft (13.5 m), minimum upper-slope height of 34 ft (10 m), total length of 1290 ft (393 m), and approximate slope of 5 degrees along the base, depicted in Figure 52.

Figure 51: Pre-failure geometry aerial view

Figure 52: Pre-failure geometric cross section

5.4 Post-Failure Geometry

Generating a CAD model for post-failure geometry is a similar process as for pre-failure. Conservation of mass is the controlling physics, trying to achieve a post-failure model in agreement with pre-failure condition. Total volume of post-failure material should closely match that of pre-failure. To accomplish this, systematic adjustment of the final displaced shape continues until post-failure volume is within close agreement of pre-failure volume, quantified as less than 5% difference. Ultimately, an average of the two values represents total volume of the Las Palmas tailings dam, included in Table 4.

Table 4: Pre & post-failure tailings geometry

	Volume		Area	Height
	yd ³	ft ³	ft ²	ft
Pre-Failure	524,000	14,148,000		
Post-Failure	550,500	14,863,500		
<i>Difference</i>	<i>26,500</i>	<i>715,500</i>		
<i>% Difference</i>	<i>4.9</i>	<i>4.9</i>		
Total (Average Pre/Post)	537,000	14,499,000	948,000	9.0
No-Flow (Undisplaced)	234,000	6,318,000		
Flow (Displaced)	303,000	8,181,000		

As discussed previously, the observed post-failure runout condition, depicted in Figure 33 to Figure 36, was approximately 5-13 ft (1.5-4 m) thick in some locations (Santa Maria, Ledezma, & Bahamondes, 2012). Documentation of detailed post-failure geometry is limited, therefore the final displaced shape is subject to interpretation and engineering judgement. Considering the large runout distance, variation in post-failure height is averaged as uniform throughout. To approximate this final displaced thickness, conservation of mass is again employed. First, the *displaced* volume is estimated by subtracting the (known) *undisplaced* volume (calculated as the remaining portion of the pre-failure model) from (known) *total* volume, illustrated in Figure 32. By principle of volumetric calculation (area x height), and with assumption the displaced height is approximately uniform, displaced volume is divided by displaced area, to yield a displaced height of approximately 9 ft (3 m) (see Table 4). Note this value agrees with the average of the observed final displaced heights of 5-13 ft (1.5-4 m).

The averaged pre/post-failure volume yields approximately 537,000 yd³ (410,570 m³), no-flow volume of 234,000 yd³ (178,000 m³), flow volume of 303,000 yd³ (231,660 m³),

approximate uniform flow height of 9 ft (3 m), total length of 1830 ft (558 m), and rough slope of 5 degrees along the base (see Figure 53 and Figure 54).

Figure 53: Post-failure geometry

Figure 54: Post-failure geometric cross section

5.5 Three Dimensional Effect

The liquefied flow failure bifurcated into an easterly and southerly direction. The leading edge (toe) of the easterly flow traveled approximately 540 ft (165 m), while the southerly flow traveled approximately 1150 ft (350.5 m). A total approximated 303,000 yd³ (231,660 m³) of material displaced in roughly two equal halves in both directions, depicted in Figure 53. The easterly path flowed in-plane with the length of pre-failure condition, and the southerly path flowed out-of-plane. This “three-dimensional effect” of bifurcating material movement in two directions, both in and out-of-plane of pre-failure orientation, is a critical consideration. To address this, the concept of conservation of mass is again considered. As the easterly and southerly flow paths contain approximately equal volumes of displaced material, half the displaced material remained in-plane, and half out-of-plane. Therefore, mass balance is not strictly valid for cross-sectional analysis of in-plane flow. Rather, the average displaced flow height sets the criteria for final, post-failure cross-section geometric convergence.

5.6 Slope Failure Progression

Resulting movement of tailings flow out-of-plane of the (in-plane) cross section selected for analysis governs the progression of the Las Palmas tailings failure. This dynamic manner (i.e. linear versus non-linear) by which material is displaced out-of-plane warrants specific consideration. To institute a reasonable analytical approach, 2 failure mechanisms are initially assumed linear from inception to termination: 1) out-of-plane migration of material volume and 2) in-plane advancement of the toe of slope.

To begin, the design of initial and final sections are per previous discussion. An initial cross-section area of approximately 33,000 ft² (3,065 m²) linearly reduces to converge on the final cross section area of approximately 12,000 ft² (1,115 m²). The same (linear) relationship also applies to advancement of the toe of slope, with displacement linearly advancing to converge on a final observed displacement of approximately 540 ft (165 m). Implementation of these two principles yields an initial set of time steps following a linear pattern.

This initial pattern, used to create each incremental section, was then adjusted to re-establish this linear relationship of area loss (out-of-plane) with centroid displacement (in-plane) of the failed mass. The goal is to prioritize focus on the *average* movement of the failing mass, not the toe of slope. The mass-property function of AutoCAD calculates the centroid for each time step. Correlation of centroid displacement with section area allows comparison of a *linear* trend with *actual* values. Under the guidance of Weber (2015), the actual area for each time step was then systematically adjusted to within 5% difference of the linear projected area, for each centroid displacement. This ultimately establishes linear trending with respect to: 1) progression of area loss and 2) centroid displacement of the failed mass.

This process of designing a sequence of cross sections each representing a particular stage of the dam failure progression requires a hybrid of geometric rendering and engineering judgement (Figure 55 and Appendix B). The process is highly iterative and requires systematic adjustment of cross-section geometry to converge upon the final solution, further discussed in chapter 6.

Progression of dam failure includes development of the failure surface along the rear (up-hill) heel of the failing slope. As material movement advances both downhill and out-of-plane, the failure mass reduces in height, exposing the flow/no-flow failure boundary, depicted in Figure 30 and Figure 31. Development of the rear, heel failure surface is also assumed and modeled to progress linearly.

Figure 55: Slope failure progression sequence

5.7 Groundwater

Understanding and modeling the groundwater present in the dam from inception to termination of failure is difficult. The selected location of the initial, approximate pre-failure phreatic surface is based on observed depth to groundwater during the field investigation, performed in June 2011. This is during the southern hemisphere winter season at the site, approximately a year and 4 months after failure occurred in February 2010. Although opportunity for variation in groundwater depth exists, including perched water, various case reports discussed the likelihood of lower saturated material contributing to liquefaction-induced failure. In rough agreement, field boring logs indicated observation of groundwater between depths of 17 to 40 ft (5 to 12 m) below ground surface across the undisturbed portion of the tailings, investigated during post-failure condition.

Native terrain surrounding the dam slopes upward toward the north and west, indicating groundwater flow likely enters the dam from these directions and continues down-slope toward the south and east, to exit the dam at its toe. Location of the pre-failure condition phreatic surface is based on observed field data, yielding saturated and unsaturated portions of 40% and 60% of the failed mass, respectively.

5.8 Slope W Model

5.8.1 Overview

Modeling and analysis of the Las Palmas tailings dam with use of Slope/W software uses four material types: 1) underlying competent native material, 2) containment walls, 3) non-liquefied/unsaturated tailings, and 4) liquefied/saturated tailings. These four materials are assigned soil strength characteristics based on data observed in the field and tested in the laboratory (Table 3 and Table 5), using methods and techniques described in chapter 4.

Figure 56 depicts the four material types for the pre-failure condition: 1) underlying competent native material shown in green, 2) containment walls shown in yellow, 3) non-liquefied/unsaturated tailings shown in blue, and 4) liquefied/saturated tailings shown in red. Static, constant strength values are assigned to non-liquefied material constituents 1-3 (green, yellow, blue) for the entirety of the slope failure progression (discussed next in chapter 5.9). The post-liquefaction residual strength is estimated through systematic, iterative back-analysis of strength parameters assigned to liquefied/saturated tailings material constituent 4 (red). Chapter 6 further describes this process.

Figure 56: Pre-failure Slope/W material constituents

The containment wall within the displaced mass is only included in the pre-failure model, depicted in Figure 57 and Appendix B. It is assumed that once liquefaction-induced failure is initiated, the flowing mass rapidly mobilizes large strains, resulting in the dislocation and breaking apart of the containment walls. Subsequently, at the arrival of the second time step, the containment wall is no longer coherent and modeled as uniform with the tailings material.

Two different “schools of thought” represent two feasible failure mechanisms to explain the dynamic progression of liquefied material during slope failure: 1) layered failure and 2) debris flow. The actual failure mechanism is likely somewhere between these two, and therefore each approach will “book end” and thereby include the true failure progression. For both approaches, the phreatic surface remains constant within the non-displaced mass for the duration of slope failure. Further explanation of each failure mechanism follows.

5.8.2 Layered Failure

For layered failure analysis, only the saturated material beneath the phreatic surface is modeled as liquefied. The overlying non-saturated material then “goes along for the ride.” Following liquefaction-induced failure triggering, pre-failure percentages of saturated and unsaturated material (40% and 60%, respectively) are assumed to remain constant. In other words, during the progression of failure, both in and out-of-plane, external groundwater neither enters nor exits the flowing mass as a whole, but remains in constant proportion of 40% saturated and 60% unsaturated material. Figure 57 illustrates this layered failure progression.

Figure 57: Layered failure time steps 1-7

5.8.3 Debris Flow

For debris flow analysis, liquefaction triggers only in the saturated tailings, however once displaced, large strains are mobilized and the failed material achieves homogeneity. Both non-saturated and saturated material mixes throughout. Pre-failure condition is the same as for layered failure, however, for each subsequent time step the entire failed mass is

modeled as homogeneous liquefied material and assigned a trial residual strength. Figure 58 illustrates this debris flow progression.

Figure 58: Debris flow time steps 1-7

5.9 Soil Strength

Table 5 summarizes soil strength values implemented for analysis. Description of the selection of these values follows.

Table 5: Assigned soil strength parameters

Material Type	Condition	Φ (°)	Cohesion (psf)	Unit Wt. (pcf)
Competent Native Material	-	(Unyielding)	(Unyielding)	(Unyielding)
Containment Walls	-	26	500	100
Non-Liquefied / Unsaturated Tailings	Undrained	0	500	95
	Drained	21	0	95
Liquefied / Saturated Tailings	-	0	(Varies)	100

5.9.1 Competent Native Material

The competent native material underlying the tailings dam was classified as primarily clayey sand (SC) according to the Unified Soil Classification System (ASTM D2487-11). This material exhibited greater strength and consistency/density than the overlying tailings material, evidenced by increased SPT blow counts and CPT tip resistance (see Appendix A) during the field investigation. In addition, unconfined compressive strengths of 7,000 psf and 19,500 psf with failure occurring at low strains of 0.8% and 1.0% indicate this material was brittle and stronger than the overlying tailings material. These field and lab results support the likelihood that failure did not occur within the competent native material. Subsequently, **this material is modeled in Slope/W as an unyielding failure surface.**

5.9.2 *Containment Walls*

The material utilized to construct the containment walls was classified as silty sand (SM) and sandy silt (ML), according to the Unified Soil Classification System (ASTM D2487-11); although this material was comprised of more granular sand than the primarily fine-grained silty tailings material. This material was tested using consolidated undrained (CU) triaxial and cyclic triaxial lab methods, and vane shear field testing. Vane shear testing is only valid for measuring the undrained shear strength of fine-grained soils, subsequently this method was not considered when assessing strength for the granular containment walls. Instead, selected total strength parameters from CU triaxial results yields: **friction angle $\approx 26^\circ$, cohesion ≈ 500 psf, and unit weight ≈ 100 pcf** (Table 3).

5.9.3 *Non-Liquefied / Unsaturated Tailings*

The non-liquefied/unsaturated tailings material was classified as primarily fine-grained sandy silt (ML) with some granular silty sand (SM), according to the Unified Soil Classification System (ASTM D2487-11). Vane shear field testing within the silty material was performed both above and below the observed ground water table. Measurement of undrained shear strength below the water table is assumed valid, however validity of undrained shear condition in the silt material above the phreatic surface is questionable. To address this, approximate peak and residual drained friction angles are estimated through back-calculation, using equation 4 from chapter 4.2.2. Table 2 of chapter 4.2.2 presents the results.

Modeling the non-liquefied/unsaturated tailings material includes two conditions: 1) undrained, and 2) drained. For the undrained condition: vane shear testing of undrained shear strength within silty tailings material above the water table was approximately 780 psf (peak) and 410-670 psf (residual). The *residual* condition most accurately represents soil strength mobilized within the flow failure at significant strains; therefore, the approximate **average residual strength $\approx$ 500 psf represents undrained conditions.** (Note: Per Table 5.4 of Coduto, 2011 this value corresponds well with observed N_{60} blow counts in this material).

For the drained condition: back-calculated drained frictional strength of the silty tailings material above the water table ranged from approximately 25-34° (peak) and 20-22° (residual). The *residual* condition most accurately represents soil strength mobilized within the flow failure at significant strains; therefore the approximate **average residual friction angle $\approx$ 21° represents drained conditions.**

A relatively low **unit weight $\approx$ 95 pcf** represents the unsaturated tailings material deposited loosely as hydraulic fill.

5.9.4 *Liquefied / Saturated Tailings*

The liquefied tailings material was classified as primarily fine-grained sandy silt (ML) with some granular silty sand (SM), according to the Unified Soil Classification System (ASTM D2487-11). Estimation of the undrained shear strength mobilized within the liquefied material is one of the primary goals of this case history. Specifically, it is the

post-liquefied *residual* strength, mobilized within the flow failure at significant strains, estimated through iterative back-analysis. Chapter 6 further describes this process.

A saturated **unit weight** ≈ 100 **pcf** represents the saturated tailings material, including a slight increase in water content.

6 METHOD OF ANALYSIS

6.1 Overview

This case history synthesizes acquired data with an observed outcome to provide analysis and results for improving engineering practice. The quality and completeness of this forensic study builds on the work of others acquiring, sampling, and testing; results are only as good as the data they derive from.

Modeling the progression of a failing slope for back-calculating the post-liquefied residual strength is an iterative, highly dynamic procedure, requiring synthesis of engineering and geometric judgement. Artistic rendering of cross sections representing individual selected time steps of a progressing mass attempts to animate the failing slope from inception to convergence upon an observed final shape. Soil strength values are assigned to material constituents based on field and lab testing, and limit equilibrium slope stability analysis provides engineering evaluation. Adjustment of a single variable influences others, and optimizing multiple variables in concert is challenging; therefore, analysis must be founded on sound assumptions and systematic in approach.

6.2 Background of Methods

A number of selected methods are available for back-analyses of liquefaction failure case histories for assessing post-liquefaction residual strength (chapter 2.5.2). These methods vary significantly in their approach, complexity, and case history data required to warrant their use. This study utilizes a standard (static) slope stability limit equilibrium method

(LEM) with use of the incremental momentum method, developed by Weber (2015). The incremental momentum method is not appropriate for use with limited observed and recorded data; sufficient information is needed to produce back-analyzed results based on an observed failure. For events without sufficient data, other less involved procedures are available.

Perhaps the single most important difference between various methods is the ability to incorporate momentum effects. In classical mechanics, momentum is the product of mass and velocity of an object. In the context of slope stability analysis, movement of a failing earth mass will initially accelerate to a point, after which the mass will decelerate and eventually stop. The influence of momentum is only absent at the instant of transition from acceleration to deceleration (Figure 16); this moment in time is challenging to capture, and can only be estimated at best. Although standard LEM analysis ignores momentum effects, this is inclusive of the incremental momentum method.

6.3 Limit Equilibrium Method

6.3.1 Overview

Stability of slopes are most commonly analyzed by methods of limit equilibrium. Required information includes soil strength parameters, but not stress-strain behavior. This method provides no information regarding the magnitude of slope movement, rather the stability is calculated as a factor of safety against slope failure.

Limit equilibrium methods estimate the approximate upper and lower limits of the post-liquefied residual strength, serving to “book-end” a range of probable residual strength

values to initiate the incremental momentum method. LEM provides a static calculation assuming brittle failure, excluding momentum effects developed by inertial movement of a sliding mass (Figure 16).

Slopes which undergo non-significant strength loss (i.e. $< 20\%$) during an earthquake are sometimes best modeled as co-seismic, occurring simultaneously with cyclic seismic displacement. During failure of the Las Palmas dam, significant strength loss occurred (i.e. $> 20\%$) as a result of liquefaction failure. Therefore, after liquefaction-triggered a brittle failure initiation, analysis of slope failure progression is independent of seismic strong ground motions.

6.3.2 Pre-Failure Analysis

Pre-failure LEM analysis yields what is termed the “post-liquefaction initial yield strength.” This is not an actual soil strength value, rather an estimated value applied to the portion of liquefied tailings to produce a (stable) static factor of safety of 1.0, for pre-failure geometry. This applies to the condition after assumed liquefaction and before significant displacement occurs. This back-calculated value will over-estimate the actual strength within the mass. If such strength existed within the soil the slope would not have failed. This value marks the upper limit of the plausible range of residual strength values.

6.3.3 Post-Failure Analysis

Post-failure LEM analysis yields what is termed the “post-liquefaction residual geometry strength.” Again, this is not an actual soil strength value, rather an estimated value

applied to the portion of tailings deemed liquefied to produce a (stable) static factor of safety of 1.0, for post-failure geometry. This back-calculated value will under-estimate actual strength mobilized within the mass, as it neglects momentum effects that require additional strength to decelerate the moving mass to equilibrium. This value marks the lower limit of the plausible range of residual strength values.

6.4 Incremental Momentum Method

6.4.1 Overview

This current study employs the incremental momentum method developed by Weber (2015) to incorporate momentum effects for a more complete and thorough back-analysis of post-liquefaction residual strength. This method involves combination of engineering judgement and geometric rendering required to generate a series of feasible cross sections to model the failure progression from initiation to termination. Initial and final geometries are first established, and generation of a selected number of estimated incremental time steps produce an animated sequence. The sequence must provide a reasonable path throughout the entirety of the failure progression, converging on the observed final geometry.

6.4.2 Procedure

The average of pre and post-failure strengths is a reasonable approximation for an initial trial strength value assigned to the saturated, liquefied material. This strength value is used for each time step to execute a series of static LEM slope stability analyses, without

seismic effects. Slope/W software outputs the following slide mass parameters as a function of each time step, used for further analysis: driving force, resisting force, slide area, slide weight, and factor of safety.

A series of iterative calculations then utilize (known) slide mass parameters for each time step; these calculations must converge on two key points: 1) the *calculated* displacement of the sliding mass must converge at the *observed* displacement for each time step, and 2) the velocity of the sliding mass must converge at zero at the final time step. This process follows Newton's second law of physics: F (force) = m (mass) $\times$ a (acceleration), rearranged to solve for acceleration, as a function of (known) force and (known) mass, for each individual time step (Eq. 5).

$$a = \frac{F}{m} = \frac{F}{w/g} \quad (\text{Eq. 5})$$

(w = weight, g = gravity, and F = *net* force = driving force – resisting force.)

Velocity is determined through integration of the (known) acceleration function, subsequently displacement is determined through integration of the velocity function; both integrated using the trapezoidal rule. These three parameters (acceleration, velocity, displacement) are functions of time, and plotted on the y-axis (Figure 59).

The parameter time (t) is estimated through the *goal seek* function in Microsoft Excel and plotted on the x-axis (Figure 59). To accomplish this, the optimum time value is auto-selected which ultimately integrates each function (in order: acceleration, velocity, displacement) yielding a displacement of known value, for each individual time step. Each time step is converged before proceeding to the next. This procedure is performed

through all (7) time steps, to converge (as best as possible) through numerical analysis, on a final velocity of zero and known final displacement (Figure 59).

Incremental Momentum Analysis: Trial 7
Residual Strength = 163 psf
Layered Failure: Undrained Shear Strength = 500 psf

Figure 59: Incremental Momentum Method example trial

If the two key principles (1: the *calculated* displacement of the sliding mass must converge at the *observed* displacement for each time step, and 2: the velocity of the sliding mass must converge at zero at the final time step) are not converged upon, the input residual strength is systematically changed, and the procedure repeated. Too large a residual strength value results in a premature reduction in velocity to zero and too small of a displacement; the opposite holds true for too small a residual strength value. The value best satisfying the established criteria represents the post-liquefaction residual strength mobilized within the displaced mass.

6.5 Total vs. Effective Stress

“In principle, it is always possible to analyze stability by using effective stress methods because the strengths of soils are governed by effective stresses under both undrained and

drained conditions. In practice, however, it is virtually impossible to determine accurately what excess pore pressures will result from changes in external loading on a slope.” (Turner & Schuster, 1996). Total stress is the selected governing method of analysis for this study. This follows the principle that development of excess pore pressures are difficult to assess and correct evaluation of equilibrium conditions must include both soil and water forces. To satisfy equilibrium, total unit weights (moist unit weights above the water table and saturated unit weights below) and external boundary water pressures are included in the model. In situations involving seepage forces, these dynamic forces are usually challenging to evaluate, and therefore easier to include for total stress analysis than exclude for effective stress analysis, with use of buoyant unit weights.

This post-liquefaction residual strength analysis captures the undrained shear strengths mobilized within the failing mass, subsequently a total stress analysis utilizes parameters selected from appropriate field and laboratory testing (vane shear test, consolidated undrained test without pore pressure correction, and unconfined compressive test).

6.6 Slope/W Analysis

This report chapter serves to provide transparency to Slope/W software analysis. There are many user input variables required for computation; some are less trivial and have a greater influence on the results. Often, influential variables are further investigated through sensitivity analysis to better capture uncertainty.

Slope/W software can employ numerous limit equilibrium methods for slope stability analysis. Morgenstern-Price method was selected for this analysis. According to Turner

& Schuster (1996), Morgenstern-Price method satisfies all conditions of equilibrium, is applicable to any shape of slip surface, and assumes inclination of side forces follow a prescribed pattern. Side force inclination is calculated in the process of solution so that all conditions of equilibrium are satisfied. This method involves 3 equations and 3 unknowns.

Implementation of a user-defined failure surface along the interface of the tailings material and underlying competent native material ensures mobilization of the entire (observed) failure mass, while preventing failure within the native material. Inclusion of the piezometric surface and groundwater seepage forces satisfy equilibrium for total stress analysis. Mohr-Coulomb theory governs soil shear strength.

6.7 Evaluation of Representative Penetration Resistance

An important piece of developing empirically-based predictive correlations for current engineering practice is evaluation of in-situ soil strength, typically assessed with SPT or CPT methods. This information provides a quantitative evaluation of site-specific in-situ soil strength to correlate with associated post-liquefaction residual strength. Due to its popularity in current practice, the SPT method with use of blow counts (also known as N-value) quantifies in-situ strength. Selection of a representative SPT blow count for a large site can require synthesis of a large collection of varying numbers, depicted in Figure 60.

Historically, researchers took different approaches to evaluating the “representative” penetration resistance. A common accepted principle is that lower than median values of penetration resistance are likely to control failure within a mass (Weber, 2015). Similar to

the concept of the weakest link in a chain, soil failure seeks out the weakest point within a zone of heterogeneity of strengths. Another approach is selective filtering of penetration resistance values, eliminating outlying values deemed likely to skew statistical trending unfavorably. Irregularities in soil stratum, such as an isolated gravel lens or soil hard pan are potential sources for outlier blow counts. For sparse data sets, median values can provide a more stable characterization, buffered from volatile irregularity.

Raw SPT blow counts determined in the field are corrected for adjustment factors and overburden per Cetin, et al. (2004) to establish SPT $N_{1,60}$ values (Figure 60 through Figure 62). These values are then corrected for fines content per Equations 6 & 7 put forth by Cetin, et al. (2004) to transform SPT $N_{1,60}$ values to $N_{1,60,CS}$ values (Figure 63 through Figure 65). The “CS” subscript stands for “clean sand” equivalent. This fines correction accounts for two factors: 1) increased resistance to liquefaction as a function of fines content, as discussed in chapter 2.3, and 2) adjustment of blow counts as a function of granular and fine grain material in-situ resistance.

$$N_{1,60,CS} = N_{1,60} * C_{FINES} \quad (Eq. 6)$$

$$C_{FINES} = (1 + 0.004 * FC) + 0.05 * \left(\frac{FC}{N_{1,60}}\right) \quad (Eq. 7)$$

In these equations, FC = percent fines content (by dry weight) expressed as an integer (e.g., 20% fines is represented as FC = 20.0). Fines content less than 5% are represented as FC = 0, and fines content exceeding 35% are represented as FC = 35.0.

To select a representative blow count for the Las Palmas dam, only the boring locations within the tailings material are considered, and not within the containment walls (Figure 64). Justification of this follows that liquefaction failure most probably initiated in the

loose portions of the tailings. In addition, a reduced data subset contains only the saturated portion of the tailings, further narrowing the range of blow counts to the specific (saturated, tailings) material most probable to have triggered liquefaction (Figure 65).

Figure 60: Histogram of all blow counts, without fines correction (all borings)

Figure 61: Histogram of blow counts in tailings material, without fines correction (borings B-2,3,4)

Figure 62: Histogram of blow counts in saturated tailings material, without fines correction (borings B-2,3,4)

Figure 63: Histogram of all blow counts, with fines correction (all borings)

Figure 64: Histogram of blow counts in tailings material, with fines correction (borings B-2,3,4)

Figure 65: Histogram of blow counts in saturated tailings material, with fines correction (borings B-2,3,4)

6.8 Evaluation of Representative Initial Vertical Effective Stress

Some empirically-based predictive models for post-liquefaction residual strength are related to initial vertical effective stress, as discussed in chapter 2.5.2. This approach includes effects of overburden in a soil column, incorporating depth and unit weight as contributing parameters to the analysis. Selection of a representative vertical effective stress concentrates on the area(s) of materials most likely to trigger liquefaction, based on the pre-failure condition. An approximated average value follows along the length of the suspected failure plane. For this study, the location selected to represent the initial vertical effective stress is the (assumed) failure interface of saturated tailings and

underlying competent native material, beneath the centroid of the displaced mass, for pre-failure condition (Figure 66).

Figure 66: Initial vertical effective stress calculation

7 RESULTS

7.1 Overview

The findings of this case history are in general agreement with previous, related efforts to study post-liquefaction residual strength. Various methods employed through previous research (covered in chapter 2.5.2) allows cross-comparison of results via different approaches, assessing strengths and weakness of each.

Results of this case history are the product of numerous and time-consuming iterations, producing a data set of reasonably well-constrained values of 1) back-calculated representative post-liquefaction strength, 2) representative penetration resistance, and 3) representative vertical effective stress along the suspected liquefied failure surface. These results are evaluated against deterministic regression charts developed by others for providing empirically derived predictive relationships for engineering practice. Lastly, a sensitivity analysis captures uncertainty in results of this current study to improve transparency and direct focus to influential parameters warranting special consideration.

7.2 Limit Equilibrium Method

Two methods utilized for analysis in this study are the limit equilibrium method and the incremental momentum method. Limit equilibrium method provides initial, static analysis of pre and post-failure geometry conditions. This intends to “book-end” the upper and lower feasible post-liquefaction residual strength values. These results help guide initial estimate of a trial residual strength to commence incremental momentum

method analysis, explicitly inclusive of momentum effects, for both layered and debris flow analysis.

7.2.1 Pre-Failure Analysis

To estimate dam stability excluding earthquake loading before liquefaction triggering, **pre-failure static LEM analysis yields a factor of safety (FS) of 1.5, indicative of a stable slope.** (Note: FS = 1.0 indicates minimum stability, however industry standard commonly requires FS = 1.5 for engineered facilities). This result supports the observation that the dam was in a statically stable condition preceding the earthquake.

To estimate dam stability excluding earthquake loading after liquefaction triggering and before significant displacement, additional results of the pre-failure static LEM analysis yield what is termed the “post-liquefaction initial yield strength.” This is not an actual soil strength, rather an estimated value applied to the portion of tailings deemed liquefied to produce a (stable) static factor of safety of 1.0, for pre-failure geometry. This value over-estimates the actual strength mobilized within the mass, observing that slope failure resulted. **A back-calculated, post-liquefaction initial yield strength $\approx$ 410 psf and 430 psf represents the upper limit of residual strength for layered and debris flow analysis, respectfully. The average of these two strengths $\approx$ 420 psf.**

To estimate dam stability including earthquake loading, pre-failure “pseudo-static” LEM analysis yields an FS = 1.0 for a horizontal seismic load of 0.20 g. This indicates a **peak ground acceleration of 0.20 g would theoretically destabilize the slope.** This only applies to pre-failure geometry, to estimate the slope stability during co-seismic

earthquake loading. The Maule, Chile M_w 8.8 earthquake produced much larger accelerations near the Las Palmas tailings dam, exceeding double this value (see Appendix D) and further supporting the observation of seismic dam failure.

7.2.2 Post-Failure Analysis

To estimate dam stability excluding earthquake loading as the slope came to equilibrium, post-failure static LEM analysis yields what is termed the “post-liquefaction residual geometry strength.” This is not an actual soil strength, rather an estimated value applied to the portion of tailings deemed liquefied to produce a static factor of safety of 1.0, for post-failure geometry. This value under-estimates the actual strength mobilized within the mass, as it neglects momentum effects requiring additional strength to decelerate the moving mass to equilibrium. **A back-calculated, post-liquefaction residual geometry strength $\approx$ 68 psf and 76 psf represents the lower limit of residual strength for layered and debris flow analysis, respectfully. The average of these two strengths $\approx$ 72 psf.**

7.3 Incremental Momentum Method

The incremental momentum method involved a series of systematic trials, included as Appendix C and further described next. Improvements were implemented based on results from each trial, until the model was judged to adequately capture the dynamics of the failing slope. After optimization of analysis, a sensitivity analysis then captures uncertainty of the residual strength as a function of variability in the primary parameters.

This promotes transparency and provides insight into which variables are most influential and therefore warrant special consideration.

7.3.1 Trial Analysis

To explain the progression of the incremental momentum method, the following description details the evolution of 7 trial analyses.

An initial set of 5 time steps were crafted to first establish a working model, yielding an optimized residual strength of 152 psf (Appendix C). The addition of a section between time steps 4 and 5 better defined the resulting acceleration/velocity/displacement graphs. This slightly increased the residual strength to 158 psf (Appendix C).

For trial 3, development of the failure surface along the upper-slope heel of each section resulted in a progressive lowering of the heel. Also, the phreatic surface was generated for each section, and layered failure modeling (described in chapter 5.8.2) applied residual strength values only to saturated material deemed liquefied. As a result, optimized residual strength decreased to 131 psf (Appendix C).

Trial 4 involved slight adjustment of time step 2 to better reflect the initial displaced shape of pre-failure condition. In addition, the progressing phreatic surfaces were updated to preserve saturated/unsaturated ratios of 40/60% respectively, throughout failure, and unit weights were increased from 90 to 95 pcf for saturated conditions. These changes produced a decreased residual strength of 126 psf (Appendix C).

A pivotal observation influenced trial 5. The assumed linear relationship with respect to 1) progression of area loss (out-of-plane) and 2) advancement of toe of slope (in-plane) was modified; adjustment refocused this linear relationship (with area loss) from toe of slope displacement to centroid displacement, to capture the “average” of the mass (described in chapter 5.6). Correlation of centroid displacement with section area allowed comparison of a linear trend projection with actual values. To promote this linear trend, the actual area for each time step was systematically adjusted to within 5% difference of the linear projected area, while maintaining each centroid displacement. In addition, Slope/W software analysis changed from block failure to fully-specified failure, to improve modeling of the displaced mass. This yielded significant improvement in resulting acceleration/velocity/displacement charts, and optimized residual strength increased to 151 psf (Appendix C).

Trial 6 split time step 5 into two steps, before and after the current condition, for a total of 7 time steps. This located time step 4 near the top of the velocity-time graph, establishing a more symmetrical shape and improving trapezoidal rule integration. Optimized residual strength remained the same at 151 psf (Appendix C).

The final trial 7 involved a slight adjustment to unit weights, added debris flow failure analysis (described in chapter 5.8.3), and added a drained condition in non-saturated tailings material (described in chapter 5.9.3). Conservation of mass was implemented to balance the average unit weight of materials across all time steps, specifically time step 1. To explain this, note the containment walls are only modeled in time step 1 (explained in chapter 5.8.1), and this (slightly heavier) material yielded an increase in average cross

section unit weight. To reduce the average unit weight in balance with other time steps, a 5 pcf reduction in containment wall unit weight and 5 pcf increase in tailings material unit weight improved the model. These adjusted values target the only reliable unit weight measurements available from results of CU testing of containment wall material, presented in Table 3.

As a result, optimized residual strength increased to 163 psf for layered failure representing *undrained* conditions (in non-saturated tailings material), as depicted in Figure 67. The introduction of layered failure representing *drained* conditions (in non-saturated tailings material) yielded a residual strength of 181 psf, as depicted in Figure 68. Debris flow analysis yielded a residual strength of 175 psf, depicted in Figure 69. (Note: debris flow strength is near average of layered analysis).

7.3.2 Trial Results

The actual failure mechanism of the Las Palmas tailings dam is challenging to assess, and was probably somewhere between layered failure and debris flow; therefore the (3) strength values are averaged for a representative post-liquefaction residual strength.

A post-liquefaction residual strength ≈ 163 psf and ≈ 181 psf represents bounds for layered failure (undrained and drained, respectively), and a residual strength ≈ 175 psf represents debris flow analysis. **The average of these (3) residual strengths ≈ 173 psf, represents the best estimate of mobilized post-liquefaction residual strength for this case study.**

Figure 67: Layered failure, undrained conditions; final acceleration, velocity and displacement curves

Figure 68: Layered failure, drained conditions: final acceleration, velocity and displacement curves

Figure 69: Debris flow: final acceleration, velocity and displacement curves

7.4 Evaluation of Representative Penetration Resistance

To select a representative penetration resistance value for the Las Palmas dam for establishing post-liquefaction strength predictive correlations, only blow counts within the saturated portion of the tailings material are considered (Figure 65). As described in chapter 7.4, the blow counts are initially corrected for adjustment factors and overburden, then corrected again for fines content. Fines correction increased $N_{1,60}$ values an average of 3 blows.

Basic statistical analysis of the corrected $N_{1,60,CS}$ blow count subset of saturated tailings material (deemed most likely to trigger liquefaction) yields a mean of 5, median of 5, and mode of 4. These values are low compared to the entire (unsaturated & saturated) tailings material. This observation supports the likelihood that liquefaction triggered within the weaker, saturated tailings material. **An $N_{1,60,CS}$ blow count ≈ 5 and $N_{1,60}$ blow count ≈ 2.5 represents this case study.**

7.5 Evaluation of Representative Initial Vertical Effective Stress

For this study, the location selected to represent initial vertical effective stress (or soil overburden) is the assumed failure interface between tailings and underlying competent native material, beneath the mid-point of the displaced mass, for pre-failure condition. Values and geometry used for calculation are per Figure 66. **An initial vertical effective stress of approximately 4,300 lb/ft², or 2.0 atmosphere represents this case study.**

7.6 Comparison of Current Study with Existing Empirical Design Charts

This current study expands the suite of (approximately 30) liquefaction failure case histories utilized for post-liquefaction, in-situ strength predictive correlations. Results of this study are evaluated against deterministic regression charts developed by others (Figure 70 through Figure 74) for providing empirically derived predictive relationships for engineering practice. Figure 70 through Figure 74 present either post-liquefied residual strength or the normalized ratio of post-liquefied residual strength to initial vertical effective stress, as a function of SPT blow count (resistance).

A red star on the following figures compares the results of this current study with existing empirical design charts. Placement of the red star is a function of x-axis and y-axis values only, and the atmospheric trend lines (when present) provide comparative reference between the representative vertical effective stress for this case history (2 atm) and existing regression trends. Close inspection shows the residual strength, a function of both $N_{1,60,CS}$ and $N_{1,60}$ and occasionally vertical effective stress, plot well with existing deterministic regression.

Figure 70: Deterministic regression showing post-liquefaction strength (S_r) as a function of both penetration resistance and initial effective vertical stress (Weber, 2015)

Figure 71: Deterministic regression showing post-liquefaction strength ratio (S_r/P) as a function of both penetration resistance and initial effective vertical stress (Weber, 2015)

Figure 72: Recommended deterministic residual strength values (S_r) as a function of SPT resistance and initial effective vertical stress (Kramer, 2008)

Figure 73: Empirical relationship for estimation of normalized residual strength ratio as a function of SPT blow count (Olson and Stark, 2002)

Figure 74: Relationship between residual undrained shear strength (S_r) and equivalent clean sand SPT blow count (Seed and Harder, 1990)

Research by Weber (2015) re-evaluated the post-liquefaction strength of approximately 30 case histories, each of varying levels of quality and completeness. Findings from that report include empirical-based Figure 70 and Figure 71. Results of the incremental momentum method introduced by Weber were cross-referenced with the findings of this report. Methodology between both research efforts are similar, however an important distinction is observed. The dynamics of the Las Palmas tailings flow failure produced a greater geometric spread of displaced material than most events included in previous case histories. This “three dimensional effect” of material displaced out-of-plane of the cross section selected for analysis, described in chapter 5.5, is likely responsible for minor observed differences in acceleration/velocity/displacement charts between previous and

current studies. Conservation of mass cannot be upheld strictly in terms of cross-sectional analysis, rather through volumetric calculation of slope mass as a whole. The dispersion of flowing material increased the surface area contact between tailings and underlying competent native material, altering flow dynamics and increasing frictional resistance. Such occurrence is recognized and addressed in this research, and through refinement of successive analytical trials, these discrepancies are systematically reduced. Resulting acceleration/velocity/displacement charts are deemed in common agreement with the work of Weber (2015) and justified for subsequent use.

7.7 Sensitivity Analysis

In geotechnical engineering forensics, uncertainty exists in many forms; some easier to identify and quantify than others. Uncertainty divides into two types: 1) aleatory and 2) epistemic (Moss, 2013). Aleatory uncertainty represents the irreducible, natural randomness and inherent variability in natural things and events. Epistemic uncertainty represents reducible, inaccurate estimation or prediction as a function of insufficient / poor / biased data, or improper modeling. These two types of uncertainty are often difficult to separate, however, developing an understanding of influential sources of uncertainty and associated consequences are the first step towards effectively reducing probability of failure in engineering practice.

The main sources of uncertainty, or variability, in back-calculation of residual strength for this case history include (in no particular order): 1) ground water condition, 2) “three-dimensional effect” progression of material out of plane 3) potential for retrogressive

failure, 4) soil unit weights, 5) non-liquefied/unsaturated tailings material strength (for layered failure) and 6) layered failure versus debris flow. Some of these uncertainties are a function of limitations of case history data quality and completeness.

To include uncertainty in the results of this case history, a sensitivity analysis captures variability in residual strength as a function of the primary variables. Each parameter is systematically adjusted and the resulting variations in incremental momentum residual strength are illustrated as a “tornado plot” in Figure 75 to display sensitivity to fluctuations in that condition/parameter. This provides insight into which variables are most influential and therefore warrant special consideration.

To begin sensitivity analysis, the average, representative post-liquefaction residual strength of 173 psf establishes the baseline value for conditions during seismic slope failure. All variations in post-liquefaction residual strength are compared to this baseline as a reference. Although not an exhaustive list, the conditions/parameters selected for sensitivity analysis include:

- 1) Unit weight: +/- 5 pcf variation (5%)
- 2) Layered failure: undrained condition, non-liquefied undrained shear strength = 500 psf
- 3) Layered failure: drained condition, non-liquefied residual friction angle = 21°
- 4) Layered failure: drained condition, non-liquefied peak friction angle = 29°
- 5) Debris flow
- 6) Phreatic surface

Figure 75: Tornado plot sensitivity analysis of post-liquefaction residual strength

Results of the sensitivity analysis (Figure 75) are telling. Despite the understood variability and uncertainty inherent with residual strength back-calculation, the post-liquefaction residual strength model proves to be robust and relatively insensitive. This outcome instills confidence in results representative of this case history.

A ± 5 pcf (5%) fluctuation in unit weight yields a ± 10 psf (6%) variation; an undrained condition with non-liquefied tailings undrained shear strength of 500 psf yields a 10 psf reduction; a drained condition with non-liquefied tailings residual friction angle of 21° yields an 8 psf increase; a drained condition with non-liquefied tailings peak friction angle of 29° yields a 7 psf increase; and debris flow analysis yields a 2 psf increase. The entire variation is within ± 10 psf (6%) of baseline residual strength.

Variation of ground water condition was addressed through comparison of layered failure versus debris flow analysis. For layered failure, the phreatic surface was modeled per chapter 5.7. For debris flow condition, any existing groundwater present within the dam before seismic failure was modeled as homogeneous liquefied material throughout failure progression. Both analyses “book-end” a wide spectrum of ground water conditions, yielding very similar results: an average layered failure of 172 psf and debris flow failure of 175 psf.

7.7.1 Synopsis

The entire post-liquefaction residual strength variation is within ± 10 psf, less than 6% of the baseline 173 psf residual strength. The results of this model prove relatively insensitive, with a narrow range of variability as a function of the conditions/parameters selected for analysis.

Throughout this case history it has been the intention of the author to remain considerate of the multitude of assumptions required for analysis, and aware of compounding uncertainty. It is through the results of this sensitivity analysis that some warranted concern regarding uncertainty within this case history is reduced, and some potentially dubious assumptions are proven trivial. Although there are additional uncertainties to scrutinize within the model, confidence remains in well-constrained, representative values.

8 SUMMARY AND CONCLUSIONS

Liquefaction is responsible for some of the costliest earthquake consequences around the world. This has drawn an increase in attention over the past three decades, as widespread liquefaction-induced failure of critical facilities continues to warrant the necessity to advance our knowledge and practice of earthquake engineering. Researchers, engineers, and scientists are gaining significant progress toward understanding the phenomenon, conditions conducive to its occurrence, the likelihood of “triggering,” and resulting consequences.

Initially, the primary focus of liquefaction engineering was the risk assessment of liquefaction “triggering” (initiation). Soils deemed likely to liquefy during seismic excitation were typically assigned negligible post-liquefaction strength values. This is a conservative approach, as post-liquefaction residual shear strength of soil is often greater than zero. Advancements in this field, including improved analytical tools, have allowed more accurate and subsequently less conservative estimates of soil residual strength.

The primary purpose of this research is to provide a case history of the Las Palmas, Chile gold mine tailings failure and estimate reasonably well-constrained values of 1) back-calculated representative post-liquefaction residual strength, 2) representative penetration resistance, and 3) representative vertical effective stress along the suspected liquefied failure surface. Results of this study plot well with existing deterministic regression charts developed by others for providing empirically derived predictive correlations for engineering practice. The intent for this case history is to provide a transparent, complete,

and sound assessment of the event, including information and conclusions regarding liquefaction engineering and residual strength.

Construction of the tailings dam began late 1981 (Santa Maria, Ledezma & Bahamondes, 2012). Original documented information regarding construction methodology of the containment walls are limited, therefore some level of uncertainty and assumption is understood. Construction sequence was comprised of 4 individual stages, occurring over approximately 17 years between 1981 and 1998. The sequence of construction resulted in a weakened boundary of the dam, in particular the vertically-aligned, down-slope walls containing stages 1 and 4 (see Figure 25). Ultimately, this location was a dividing boundary, delineating the remaining portion of the dam from that which experienced liquefaction-induced flow failure.

The seismic event of this case history is the February 27, 2010 Maule, Chile earthquake; the sixth largest since 1900, a M_w 8.8 generating a maximum acceleration of 0.65g, recorded at Concepcion, Chile. Cyclic loading from the earthquake triggered liquefaction-induced failure within the loosely deposited saturated tailings material at the base of the Las Palmas tailings dam. Once triggered, very significant strength loss occurred within the constructed slope, resulting in brittle failure. The ensuing slope failure is therefore modeled independent of seismic strong ground motions, driven primarily from imbalanced inertial forces developed within the sliding mass.

The liquefied flow failure took two paths: an easterly and southerly direction. The leading edge (toe) of the easterly flow traveled approximately 540 ft (165 m), whereas the southerly flow traveled roughly 1150 ft (350 m) (Google Maps and Santa Maria,

Ledezma & Bahamondes, 2012). A total approximated 303,000 yd³ (231,660 m³) of material displaced in approximately two equal halves in both directions. The easterly trajectory flowed in-plane with the length of pre-failure condition, and the southerly trajectory flowed out-of-plane. This “three-dimensional effect” of material movement in two directions, both in and out-of-plane of pre-failure orientation, is an important consideration for subsequent analysis.

This study uses the incremental momentum method, developed by Weber (2015), to incorporate momentum effects for a more complete and thorough back-analysis of post-liquefaction residual strength. This method requires a combination of engineering judgement and geometric rendering to generate a series of feasible cross sections to model the geometry of failure progression from inception to termination. Initial and final geometries are first established, and a selected number of estimated incremental time steps produce an animated sequence. The sequence must provide a reasonable path throughout the entirety of the failure progression, converging on the established final geometry.

Two different “schools of thought” represent two feasible failure mechanisms to explain the dynamic progression of liquefied material during slope failure: 1) layered failure and 2) debris flow. The actual failure mechanism is likely somewhere between these two, and therefore each approach should “book end” and thereby include the true failure progression. A post-liquefaction residual strength ≈ 163 psf and ≈ 181 psf represents bounds for layered failure (undrained and drained, respectively), and a residual strength ≈ 175 psf represents debris flow analysis. **The average of these (3) residual strengths $\approx$**

173 psf, represents the mobilized post-liquefaction residual strength for this case study.

Selection of a representative penetration resistance value of the Las Palmas dam for establishing post-liquefaction strength predictive correlations only considers blow counts within the saturated portion of the tailings material. Blow counts are corrected for adjustment factors, overburden, and fines content. **An $N_{1,60,CS}$ blow count ≈ 5 and $N_{1,60}$ blow count ≈ 2.5 represents this case study.**

The location selected to represent initial vertical effective stress (or soil overburden) is the assumed failure interface between tailings and underlying competent native material, beneath the mid-point of the displaced mass, for pre-failure condition. **An initial vertical effective stress of approximately 4,300 lb/ft², or 2.0 atmosphere represents this case study.**

Results of a sensitivity analysis show the post-liquefaction residual strength model proves to be relatively insensitive to fluctuations in data variation. The entire post-liquefaction residual strength variation is within ± 10 psf, less than 6% of the baseline 173 psf residual strength. This outcome instills confidence in well-constrained values representative of this case history.

In conclusion, the results of this study expand the suite of (approximately 30) liquefaction failure case histories utilized for post-liquefaction, in-situ strength predictive correlations. Results are well-constrained and in agreement with previous deterministic regression

charts developed by others for providing empirically derived residual strength relationships for engineering practice.

8.1 Limitations of Study

Much of this forensic study builds on the work of others acquiring, sampling, and testing; results are only as good as the data they derive from. A significant amount of epistemic uncertainty is inherent in field investigation activities, data acquisition, sample disturbance, and controlled laboratory testing. These processes only represent the initial phase of most geotechnical engineering endeavors, and such uncertainty is likely to propagate through following analysis. Significant effort was made to reduce and address uncertainty through transparent documentation of this case history and a sensitivity analysis of influential variables. When possible, consideration of a spread of plausible values or outcomes aimed to “book-end” the range of most probable, representative conclusions.

The back-analysis performed in this current study does not account for the effects of cyclic lurching contributing to the overall liquefaction-induced failure displacements; although considering the significant runout distance this may be deemed negligible.

Pre and post-failure geometry modeled with AutoCAD software used limited available topographic information, google earth imagery, and approximate dimensions. Although modeling was completed with attention to detail, without a professional aerial or ground survey some geometric uncertainty exists.

Slope/W modeling included Morgenstern-Price method and Spencer's method. Use of Morgenstern-Price method required limited and minor adjustments to satisfy model convergence. These solutions, however, did not resolve all issues with use of Spencer's method, subsequently this method was omitted from the study. Cross-comparison with numerous limit equilibrium methods is preferred, however it is unlikely that results would differ significantly.

8.2 Future Research

The findings of this case history are limited to available resources, and thus a few important areas of future research are welcomed and warranted. Although not an exhaustive list, the primary suggested topics include: 1) incremental retrogressive failure progression, 2) analysis of the southerly flow corridor, and 3) further field data acquisition, including CPT and shallow subsurface geophysical testing.

Common debate when modeling back-analysis stability of failed slopes is the consideration of monolithic failure or retrogressive failure progression. Such dynamic failure mechanisms are often difficult to discern. Due to the time-intensive iterations required to complete the incremental momentum method, retrogressive failure was not included in this study; however, the possibility of this failure mechanism should be addressed.

As mentioned in previous discussion, the failed mass displaced in two main corridors: 1) east and 2) south. It would be useful to complete the same residual strength back-analysis of the southerly flow direction, for comparison with the results of this study.

Lastly, additional field investigations could provide improved in-situ soil strength characterization, including shallow subsurface geophysical testing and seismic CPT. Shallow subsurface geophysical testing would enable improved characterization of subsurface materials including shear wave velocity, and identification of soil layering thickness and associated dip angle(s). In addition, this portable method can access difficult locations. Seismic CPT performed within the tailings material would provide an opportunity to advance the findings of this study through repeatable and continuous subsurface data analysis; providing accurate and greater detailed soil profiling and in-situ soil characterization than existing SPT, including shear wave velocity. Also, software is available for use with CPT output data for detailed liquefaction analysis. Finally, the CPT post-liquefaction residual strength database is limited, and this case history would provide an opportunity to obtain CPT research data valuable for future earthquake engineering.

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APPENDICES

APPENDIX A: Boring Logs

						<div>BORING LOG</div> <div>BORING NO. B-1</div> <div>JOB NO.</div>											
PROJECT INFORMATION						DRILLING INFORMATION											
PROJECT: Las Palmas Tailings Dam DRILLING LOCATION: Containment Wall DATE DRILLED: LOGGED BY:						DRILL RIG: HOLE DIAMETER: SAMPLING METHOD: HOLE ELEVATION:											
▼ Depth of Groundwater: 43 Feet <div>Boring Terminated At: 70 Feet</div>																	
DEPTH	SOIL DESCRIPTION					USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	(N ₁) 60	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)

0	SILTY SAND	SM			9	14			5-11%			35	NP
1					7	11						35	
2					11	16						33	
3					8	10						31	
4					4	4						31	
5					7	7						37	
6					9	8						43	
7					7	7						43	
8					6	6						43	
9					8	7						27	
10					9	8						34	
11					9	7						48	
12					8	6						48	
13					2	2			20-21%			77	2-4
14					6	4						77	
15					4	3						82.5	
16					4	3						82.5	
17					3	2						88	
18					5	3						77	
19					4	2						77	
20					3	2						65	
21					3	2						76	
22					2	1						76	
23													
24													
25													
26													
27													
28													
29													
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74													

				<h2 style="text-align: center;">BORING LOG</h2> <p>BORING NO. B-2</p> <p>JOB NO.</p>								
PROJECT INFORMATION				DRILLING INFORMATION								
PROJECT: Las Palmas Tailings Dam DRILLING LOCATION: Tailings DATE DRILLED: LOGGED BY:				DRILL RIG: HOLE DIAMETER: SAMPLING METHOD: HOLE ELEVATION:								
▼ Depth of Groundwater: 40 Feet				Boring Terminated At: 53 Feet								
DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN (N ₁) 60	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)

0												
-1	SANDY SILT	ML			8	12			12-32%		93	NP-4
-2												
-3					1	2					93	
-4					4	5					93	
-5												
-6					4	4					96	
-7												
-8					5	5					93	
-9												
-10					6	5					90	
-11					5	5					91	
-12					4	3					95	
-13					3	2					92	
-14					1	1					99	
-15					2	2					94	
-16					4	3					88	
-17					3	2					82	
-18					2	1					98	
-19					4	3					99	
-20					2	1					99	
-21												
-22												
-23												
-24												
-25												
-26												
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-47												
-48												
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-51												
-52												
-53												
-54												

					BORING LOG								
					BORING NO. B-3								
					JOB NO.								
PROJECT INFORMATION					DRILLING INFORMATION								
PROJECT: Las Palmas Tailings Dam					DRILL RIG:								
DRILLING LOCATION: Tailings					HOLE DIAMETER:								
DATE DRILLED:					SAMPLING METHOD:								
LOGGED BY:					HOLE ELEVATION:								
▼ Depth of Groundwater: 20 Feet					Boring Terminated At: 30 Feet								
DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	(N ₁) 60	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)

0	SANDY SILT	ML							5-19%				NP
-1													
-2					10	15						83	
-3													
-4													
-5					7	11						79	
-6													
-7													
-8					5	6						76	
-9													
-10					6	6						90	
-11													
-12													
-13					5	5						81	
-14													
-15													
-16													
-17													
-18													
-19													
-20					2	2						75	
-21													
-22													
-23					10	9						73	
-24													
-25													
-26													
-27													
-28					7	6						59	
-29													
-30													

					<h2 style="text-align: center;">BORING LOG</h2> <p>BORING NO. B-4</p> <p>JOB NO.</p>								
PROJECT INFORMATION					DRILLING INFORMATION								
PROJECT: Las Palmas Tailings Dam DRILLING LOCATION: Tailings DATE DRILLED: LOGGED BY:					DRILL RIG: HOLE DIAMETER: SAMPLING METHOD: HOLE ELEVATION:								
▼ Depth of Groundwater: 17 Feet					Boring Terminated At: 28 Feet								
DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/12 IN	(N ₁) 60	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)

0	SANDY SILT	ML							3-19%				N/A
-1					6	9						88	
-2													
-3													
-4													
-5					4	6						87	
-6													
-7													
-8													
-9					3	4						97	
-10													
-11	SILTY SAND	SM							8%				N/A
-12					12	12						32	
-13													
-14													
-15	SANDY SILT	ML							18%				N/A
-16					7	7						59	
-17													
-18	BEDROCK												
-19					Refusal								
-20													
-21													
-22													
-23													
-24													
-25													
-26													
-27													
-28													
-29													
-30													

					<h2 style="text-align: center;">BORING LOG</h2> <p>BORING NO. B-5</p> <p>JOB NO.</p>								
PROJECT INFORMATION					DRILLING INFORMATION								
PROJECT: Las Palmas Tailings Dam DRILLING LOCATION: Containment Wall DATE DRILLED: LOGGED BY:					DRILL RIG: HOLE DIAMETER: SAMPLING METHOD: HOLE ELEVATION:								
▼ Depth of Groundwater: Not Encountered					Boring Terminated At: 65 Feet								
DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	(N ₁) 60	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)

0	SILTY SAND	SM							4-5%			31-33%	NP
1													
2													
3													
4													
5													
6													
7													
8													
9													
10	SANDY SILT	ML							10-16%			62-81%	NP
11													
12													
13													
14													
15													
16													
17													
18													
19													
20													
21													
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35													
36													
37													
38													
39													
40													
41													
42	SANDY SILT	ML							21-26%			65-98%	1-5
43													
44													
45													
46													
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64													
65													
66	BEDROCK												
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70													

APPENDIX B: Slope/W Animation Time Steps 1-7

Layered Failure

Time Step 1

Time Step 2

Time Step 3

Time Step 4

Time Step 5

Time Step 6

Time Step 7

Debris Flow

Time Step 1

Time Step 2

Time Step 3

Time Step 4

Time Step 5

Time Step 6

Time Step 7

APPENDIX C: Incremental Momentum Trials

Incremental Momentum Analysis: Trial 1 Residual strength = 152 psf

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)	Force (lb)			Acceleration (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)		
		Total	Increment	Increment	Increment	Driving	Resisting	Net	Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Difference (Goal Seek)
1	0.39	0	0	32840	3096100	279820	108100	171720	1.8	0.0	0.0	0	0	0	0	0
2	0.49	35	35	27631	2486800	244070	118830	125240	1.6	6.4	6.4	10.9	10.9	35.0	35.0	0
3	0.81	94	59	22566	2031000	173600	139720	33880	0.5	10.8	17.3	4.8	15.7	59.0	94.0	0
4	1.11	182	88	17309	1557800	144690	160270	-15580	-0.3	16.3	33.6	0.6	16.3	88.0	182.0	0
5	1.92	272	90	11952	1075700	94081	180860	-86779	-2.6	26.4	60.0	-14.6	1.7	90.0	272.0	0

Incremental Momentum Analysis: Trial 2 Residual Strength = 158 psf

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)	Force (lb)			Acceleration (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)		
		Total	Increment	Increment	Increment	Driving	Resisting	Net	Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Difference (Goal Seek)
1	0.39	0	0	32840	3096100	280280	110040	170240	1.8	0.0	0.0	0	0.0	0	0.0	0
2	0.50	35	35	27631	2486800	244070	121170	122900	1.6	6.5	6.5	10.8	10.8	35.0	35.0	0
3	0.82	94	59	22566	2031000	173600	142480	31120	0.5	10.9	17.4	4.7	15.5	59.0	94.0	0
4	1.13	182	88	17309	1557800	144690	163440	-18750	-0.4	16.5	33.9	0.3	15.8	88.0	182.0	0
5	1.51	236	54	14621	1315900	117080	177320	-60240	-1.5	20.4	54.3	-3.6	12.2	54.0	236.0	0
6	1.96	272	36	11952	1075700	94081	184430	-90349	-2.7	26.2	80.6	-12.2	0.06	35.8	271.8	0

Incremental Momentum Analysis: Trial 3
Residual Strength = 131 psf

		Displacement (ft)		Area (ft²)	Weight (lb)		Force (lb)			Acceleration (ft/s²)		Time (s)		Velocity (ft/s)		Displacement (ft)		
Time Step	FS	Total	Increment	Increment	Increment	Driving	Resisting	Net	Increment	Increment	Increment	Total	Increment	Total	Increment	Total	Difference (Goal Seek)	
1	0.34	0	0		32841	3096200	276070	94536	181534	1.9	1.5	0.0	0.0	0.0	0	0.0	0	
2	0.51	50	50		27850	2508200	231660	117840	113820									
3	0.97	99	49		22556	2030100	123890	120730	3160									
4	1.27	188	89		17288	1555900	110350	139900	-29550									
5	1.49	242	54		14640	1317600	99522	148530	-49008									
6	1.95	270	28		12026	1082300	80467	157270	-76803

Incremental Momentum Analysis: Trial 4
Residual Strength = 126 psf

		Displacement (ft)		Area (ft²)	Weight (lb)	Force (lb)			Accel. (ft/s²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft²)			
Time Step	FS	Total	Increment	Increment	Increment	Driving	Resisting	Net	Increment	Increment	Goal Seek	Total	Increment	Total	Increment	Total	Difference (Goal Seek)	Projected (linear)	Difference (Proj - Act)	% Diff
1	0.33	0	0	32841	3162200	280290	91288	189002	1.9	0.0	0.0	0	0.0	0.0	0	0.0	0	32841	0	0%
2	0.49	40	40	27402	2521100	205960	100950	105010	1.3	7.0	7.0	11.4	11.4	40.0	40.0	0	29731	2329	8%	
3	0.93	99	59	22524	2072300	124480	116150	8330	0.1	11.5	18.5	3.3	14.7	59.0	99.0	0	25143	2619	12%	
4	1.19	186	87	17259	1587800	113180	134630	-21450	-0.4	17.6	36.1	-0.9	13.8	87.0	186.0	0	18377	1118	6%	
5	1.45	227	41	14627	1345700	99646	144410	-44764	-1.1	20.9	57.0	-2.5	11.4	41.0	227.0	0	15189	562	4%	
6	1.89	268	41	12001	1104200	81737	154340	-72603	-2.1	28.0	84.9	-11.3	0.0	40.5	267.5	0.5	12001	0	0%

Incremental Momentum Analysis: Trial 5
Residual Strength = 151 psf

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)	Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment			Driving	Resisting	Net		Increment	Total	Increment	Total	Increment	Total	Difference (Goal/Seek)	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.47	0	0	32099	3091700	235730	110610	125120	1.3	0.0	0.0	0	0.0	0	0.0	0	32099	0	0%
2	0.57	43	43	29680	2730500	211830	120110	91720	1.1	8.5	8.5	10.1	10.1	43.0	43.0	0	28871	-809	-3%
3	0.74	105	62	25125	2311400	188830	140310	48520	0.7	13.5	22.0	4.4	14.5	62.0	105.0	0	24218	-907	-4%
4	1.09	180	75	18895	1738500	147900	160810	-12910	-0.2	18.5	40.5	1.1	15.6	75.0	180.0	0	18588	-307	-2%
5	1.51	228	48	15166	1395200	113860	172180	-58320	-1.3	21.8	62.3	-2.7	13.0	48.0	228.0	0	14985	-181	-1%
6	2.23	268	40	11983	1102500	81160	180930	-99770	-2.9	27.8	90.1	-12.6	0.3	39.5	267.5	0.5	11983	0	0%

Incremental Momentum Analysis: Trial 6
Residual Strength = 151 psf

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)	Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment			Driving	Resisting	Net		Increment	Total	Increment	Total	Increment	Total	Difference (Goal/Seek)	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.47	0	0	32099	3091700	235730	110610	125120	1.3	0.0	0.0	0	0.0	0	0.0	0	32099	0	0%
2	0.57	43	43	29680	2730500	211830	120110	91720	1.1	8.5	8.5	10.1	10.1	43.0	43.0	0	28871	-809	-3%
3	0.74	105	62	25125	2311400	188830	140310	48520	0.7	13.5	22.0	4.4	14.5	62.0	105.0	0	24218	-907	-4%
4	1.09	180	75	18895	1738500	147900	160810	-12910	-0.2	18.5	40.5	1.1	15.6	75.0	180.0	0	18588	-307	-2%
5	1.35	210	30	16562	1523100	121840	164450	-42610	-0.9	20.5	61.0	-1.1	14.5	30.0	210.0	0	16336	-226	-1%
6	1.83	245	35	14001	1290400	99760	183050	-83290	-2.1	23.3	84.3	-4.2	10.3	35.0	245.0	0	13709	-292	-2%
7	2.23	268	23	11983	1102500	81160	180930	-99770	-2.9	27.4	111.7	-10.2	0.1	21.2	266.2	1.8	11983	0	0%

Incremental Momentum Analysis: Trial 7
Residual Strength = 163 psf
Layered Failure: Undrained Shear Strength = 500 psf

Time Step	FS	Displacement (ft)		Area (ft ²)		Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment	Increment	Increment	Driving	Resisting	Net		Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.49	0	0	32788	3216200	242400	119150	123250	1.2	0.0	0.0	0	0.0	0	0.0	0	32788	0	0%
2	0.56	43	43	29672	2878200	229700	129630	100070	1.1	8.5	8.5	10.1	10.1	43.0	43.0	0	29450	-222	-1%
3	0.75	105	62	25124	2437000	200840	151260	49580	0.7	13.6	22.1	4.5	14.5	62.0	105.0	0	24637	-487	-2%
4	1.11	180	75	18897	1833100	156520	173370	-16850	-0.3	18.6	40.7	0.9	15.4	75.0	180.0	0	18814	-83	0%
5	1.33	210	30	16954	1645600	136730	181600	-44870	-0.9	20.6	61.4	-1.2	14.2	30.0	210.0	0	16486	-478	-3%
6	1.73	245	35	13988	1359400	108780	187970	-79190	-1.9	23.5	84.8	-3.9	10.3	35.0	245.0	0	13769	-219	-2%
7	2.28	268	23	11983	1162400	85694	195020	-1E+05	-3.0	27.7	112.5	-10.3	0.0	21.7	266.7	1.3	11983	0	0%

Incremental Momentum Analysis: Trial 7
Residual Strength = 181 psf
Layered Failure: Residual Friction Angle = 21°

Time Step	FS	Displacement (ft)		Area (ft ²)		Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment	Increment	Increment	Driving	Resisting	Net		Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.43	0	0	32836	3220800	298350	127160	171190	1.7	0.0	0.0	0	0.0	0	0.0	0	32836	0	0%
2	0.53	43	43	29703	2881100	268730	141610	127120	1.4	7.4	7.4	11.6	11.6	43.0	43.0	0	29491	-212	-1%
3	0.74	105	62	25100	2435400	220140	163670	56470	0.7	11.8	19.2	4.8	16.4	62.0	105.0	0	24668	-432	-2%
4	1.17	180	75	18897	1833100	162420	189570	-27150	-0.5	16.3	35.6	0.6	17.0	75.0	180.0	0	18834	-63	0%
5	1.42	210	30	16953	1644500	138780	196690	-57910	-1.1	18.2	53.7	-1.5	15.5	30.0	210.0	0	16500	-453	-3%
6	1.85	245	35	13977	1358500	110030	203940	-93910	-2.2	20.8	74.5	-4.4	11.1	35.0	245.0	0	13777	-200	-1%
7	2.45	268	23	11988	1162900	87078	213540	-1E+05	-3.5	24.7	99.2	-11.1	0.0	21.5	266.5	1.5	11988	0	0%

Incremental Momentum Analysis: Trial 7
Residual Strength = 180 psf
Layered Failure: Peak Friction Angle = 29°

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)		Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment		Increment	Increment	Driving	Resisting	Net		Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.43	0	0	32805	3217700	297770	126580	171190		1.7	0.0	0.0	0	0.0	0	0.0	0	32805	0	0%
2	0.53	43	43	29703	2881100	267740	140840	126900		1.4	7.4	7.4	11.6	11.6	43.0	43.0	0	29464	-239	-1%
3	0.74	105	62	25120	2436500	219760	163390	56370		0.7	11.8	19.3	4.8	16.4	62.0	105.0	0	24647	-471	-2%
4	1.16	180	75	18897	1833100	162100	188790	-29690		-0.5	16.3	35.6	0.6	17.0	75.0	180.0	0	18819	-78	0%
5	1.41	210	30	16949	1644100	138470	195790	-57320		-1.1	18.2	53.8	-1.5	15.5	30.0	210.0	0	16488	-461	-3%
6	1.85	245	35	13977	1358200	109900	203150	-93250		-2.2	20.8	74.5	-4.4	11.2	35.0	245.0	0	13769	-208	-1%
7	2.45	268	23	11982	1162400	86869	212430	-1E+05		-3.5	24.7	99.3	-11.2	0.1	22.0	267.0	1.0	11982	0	0%

Incremental Momentum Analysis: Trial 7
Residual Strength = 180 psf
Debris Flow

Time Step	FS	Displacement (ft)		Area (ft ²)	Weight (lb)		Force (lb)			Accel. (ft/s ²)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft ²)		
		Total	Increment		Increment	Increment	Driving	Resisting	Net		Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.44	0	0	32796	3279600	286820	127150	159670		1.6	0.0	0.0	0	0.0	0	0.0	0	32796	0	0%
2	0.59	43	43	29733	2973300	239190	141040	98150		1.1	8.1	8.1	10.6	10.6	43.0	43.0	0	29457	-276	-1%
3	0.74	105	62	25114	2511400	223310	164480	58830		0.8	12.9	21.0	4.4	15.0	62.0	105.0	0	24643	-471	-2%
4	1.14	180	75	18897	1889800	166690	189440	-22750		-0.4	17.8	38.8	0.9	15.9	75.0	180.0	0	18820	-77	0%
5	1.38	210	30	16964	1696300	143250	197440	-54190		-1.0	19.7	58.5	-1.4	14.5	30.0	210.0	0	16490	-474	-3%
6	1.81	245	35	13988	1398600	113100	204380	-91280		-2.1	22.6	81.1	-4.5	10.1	35.0	245.0	0	13773	-215	-2%
7	2.38	268	23	11987	1198700	89568	212950	-1E+05		-3.3	26.3	107.4	-10.0	0.1	18.7	263.7	4.3	11987	0	0%

APPENDIX D: Strong Motion Stations

Strong Motion Stations for 8.8M Chile Earthquake of 27 Feb 2010, 0634 UTC

Station	Code /ID	Network	Distance (km)		Horiz. Apk (g)		View	Download
			Epic.	Fault	Ground	Struct.		
Angol	ANGO	UCS	209.3	65.6	0.928	--		
Concepcion San Pedro	CCSP	NMI	109.1	36.2	0.652	--		
Constitucion	COT	UCS	69.7	38.6	0.640	--		
Llolleo	LLO	UCS	274.3	41.8	0.564	--		
Santiago Maipo	CRMA	UCS	320.7	68.9	0.562	--		
CRS MAIPU RM	CMR	UCS	322.0	69.3	0.560	--	--	--
Municip. San Jose de Maipo	SJCH	UCS	332.8	87.3	0.480	--		
Talca	TAL	UCS	113.1	59.1	0.477	--		
Curico	CUR	UCS	170.5	65.1	0.471	--		
Hosp. Curico	HCUR	UCS	169.9	65.2	0.470	--	--	--
Hualane	HUA	UCS	136.0	50.0	0.461	--

APPENDIX E: AutoCAD Plots

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
PRE-FAILURE PHOTO
APRIL 2007 IMAGE

Developer:

Project Number:

Project Location:
CHILE
-35.185995, -71.758554

Date:

Scale: 1"=300'

Drawn by:

Checked by:

Sheet: of

C:\Users\Tristan\Desktop\Thesis Slope Modeling\Las Palmas Case History.dwg 8/28/16 02:59 PM Tristan

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
POST-FAILURE PHOTO
2010 IMAGE

Developer:

Project Number:

Project Location:
CHILE
-35.185995, -71.758554

Date:
Scale: 1"=300'

Drawn by:
Checked by:

Sheet: of

PRE-FAILURE STATISTICS:

PERIMETER: 459,000 Ft²

VOLUME: 537,000 Yd³

UNDISPLACED VOLUME

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
PRE-FAILURE GEOMETRY

Project Number:

Project Location:
CHILE
-35.185995, -71.758554

Date:
Scale: 1"=300'

Drawn by:
Checked by:

Sheet:
of

POST-FAILURE STATISTICS:

PERIMETER:
221,000 Ft² UNDISPLACED
948,000 Ft² DISPLACED

VOLUME:
537,000 Yd³ TOTAL
234,000 Yd³ NO FLOW
303,000 Yd³ FLOW

- UNDISPLACED VOLUME
- DISPLACED "FLOW" VOLUME

EAST FLOW PERIMETER
474,000 Ft²

SOUTH FLOW PERIMETER
474,000 Ft²

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
POST-FAILURE GEOMETRY

Project Number:

Project Location:
CHILE
-35.185995, -71.758554

Date:
Scale: 1"=300'

Drawn by:
Checked by:

Sheet: of

C:\Users\Tristan\Desktop\Thesis Slope Modeling\Las Palmas Case History.dwg 8/28/16 03:00 PM Tristan

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
PRE-FAILURE PROFILE 1

Developer:

Project Number:

Project Location:

CHILE
-35.185995, -71.758554

Date:

Scale: 1"=200' H
1"=100' V

Drawn by:

Checked by:

Sheet:

of

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CASE HISTORY: LAS PALMAS MINE TAILINGS DAM FAILURE

Sheet Title:
POST-FAILURE PROFILE 1

Developer:

Project Number:

Project Location:

CHILE
-35.185995, -71.758554

Date:

Scale: 1"=200' H
1"=100' V

Drawn by:

Checked by:

Sheet:

of

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CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
SPT & CPT BORING LOCATIONS

Developer:

Project Number:

Project Location:
CHILE
-35.185995, -71.758554

Date:

Scale: 1"=300'

Drawn by:

Checked by:

Sheet: of

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
GEOTECHNICAL BORING
PROFILE

Developer:

Project Number:

Project Location:

CHILE
-35.185995, -71.758554

Date:

Scale: 1"=200' H
1"=100' V

Drawn by:

Checked by:

Sheet:

of

CASE HISTORY:
LAS PALMAS
MINE TAILINGS DAM FAILURE

Sheet Title:
GEOTECHNICAL SPT/CPT
BORING PROFILES

Developer:

Project Number:

Project Location:

CHILE
-35.185995, -71.758554

Date:

Scale: 1"=200' H
1"=100' V

Drawn by:

Checked by:

Sheet:

of