PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

Flow-Failure Case History of the Las Palmas, Chile, Tailings Dam

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PEER Report No. 2019/01
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ABSTRACT

This report documents the flow failure of the Las Palmas tailings dam that was induced by the 27 February 2010 Maule Chile M8.8 earthquake. The Las Palmas site is located in Central Chile in Region VII near the town of Talca. Construction of the tailings dam occurred between 1998 as part of a gold mining operation and was no longer in active use.

The ground shaking from the earthquake induced liquefaction of the saturated tailings material and resulted in a flow failure that ran out upwards of 350 m, flowing downslope in two directions. This report is broken into three sections:

1. A summary of the construction and flow failure of the Las Palmas tailings dam;
2. Details on the field investigations at the site, including the 2010 GEER reconnaissance, 2011 litigation support [DICTUC 2012], and the recent PEER–NGL-funded 2017 investigation; and
3. Back-analysis of the flow failure by Gebhart [2016] to estimate the residual strength.

The goal of this work is to provide a "high-quality" flow-failure case history to augment the existing database. The existing database is composed of roughly thirty case histories of varying quality (e.g., Weber et al. [2015] and Kramer and Wang [2015]). Herein, the term "high-quality" means that the *in situ* measurements were made in a controlled and repeatable manner, and that the back-analysis of the residual strength was performed considering static and dynamic effects of the slide mass. The results from this research indicate that the median back-analyzed residual strength of the liquefied material is ~8.3 kPa (~173 psf) at a pre-earthquake vertical effective stress of 2 atm (~200 kpa or 4000 psf), which is correlated to a median SPT blow count of $N_{1,60}$~2.5, a median CPT tip resistance of q_{c1}~1.3 MPa, and a median shear-wave velocity of VS_1~172 m/sec. The back analyzed residual strength has a nominal coefficient of variation of 5.5% determined using a sensitivity analysis.

ACKNOWLEDGMENTS

This research is a compilation of prior work and more recent field investigations. The initial reconnaissance was performed by GEER members and was funded by the U.S. National Science Foundation (NSF) through the Geotechnical Engineering Program under Grant No. CMMI-1266418. The work by Professor Ledezma and colleagues at Pontificia Universidad Católica de Chile was funded through litigation support. The work by Tristan Gebhart was performed as part of his Master's Degree at Cal Poly, San Luis Obispo. The most recent field investigations were funded through the PEER–NGL program, with support by the California Department of Transportation and the U.S. Nuclear Regulatory Commission.

Thanks go to NSF, the NGL program, Professor Ricardo Moffat and the employees of LMMG, and Professor Joe Weber, with special thanks to Professor Jon Stewart who provided a careful and detailed review of a draft report that helped vastly improve the presentation and communication of this study.

Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the sponsoring organizations, PEER, or the Regents of the University of California.

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1 Introduction

The 27 February 2010 Maule, Chile, earthquake is the seismic event that resulted in the failure for this case history. The sixth largest recorded earthquake since 1900, this event occurred at 3:34 am local time with a moment magnitude of 8.8 and was centered off the coast of Bio Bio, Chile. The hypocenter was located at an approximate depth of 35 km (21.7 miles), 95 km (60 miles) off the coast, and 335 km (210 miles) southwest of the capital of Santiago.

Maximum recorded acceleration was 0.94 g at a station located in the city of Angol, Chile (*www.terremotosuchile.cl/red_archivos/RENAMAULE2010R2.pdf*). Strong earthquake shaking exceeded a minute in some locations for a total duration up to nearly two minutes. The rupture occurred on 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 due to a plate convergence of approximately 7 m (23 ft) per century. The fault rupture at depthplane exceeded 100 km (60 miles) in width and nearly 500 km (300 miles) parallel to the coast [Hayes 2010]. Rupture initiated beneath the coast and propagated westward, northward, and southward, causing tectonic deformation that triggered a tsunami.

The ground shaking resulted in pervasive damage of lifelines, e.g., roadways, bridges, railroads, and road embankments. In total, approximately 523 people were killed, 12,000 injured, and 800,000 displaced, and 370,000 houses, 4000 schools, and 79 hospitals were damaged or destroyed [USGS 2010]. The tailings dam of the Las Palmas gold mine experienced liquefaction-induced flow failure due to strong ground shaking, which produced a run out of a third of a kilometer [GEER 2010]. The flow failure resulted in fatalities and caused environmental degradation of the surrounding area.

This report summarizes the following:

- Construction and subsequent failure of the Las Palmas tailings dam
- Reconnaissance investigations performed by the GEER team [GEER 2010]
- Field investigations performed by Professor C. Ledezma and colleagues [DICTUC 2012] where standard penetration test (SPT) measurements were made
- PEER (Pacific Earthquake Engineering Research Center)-NGL (next generation liquefaction)-funded field investigations where cone penetration tests (CPT) and shear-wave velocity (*VS*) measurements were made

- Back-analysis of the failure by Gebhart [2016] that estimated the liquefied residual strength, and effective stress at the time of failure.

The objective of this report is to provide a "high-quality" flow-failure case history to add to the existing liquefied residual strength database.

2 Las Palmas Tailings Dam Construction and Failure

The construction of the Las Palmas gold mine tailings dam (Figure 2.1) began in 1981 [DICTUC 2012]. The dam was built upon existing ground that was downward sloping 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. The dam was constructed in four phases (Figure 2.2), which occurred between 1981–1998. Each stage included initial construction of containment embankments and hydraulic fill placement of tailings materials. Construction of containment walls or embankments typically utilize the sandy, more granular fraction of the tailings material to provide increased strength

Construction documents indicate use of both upstream and centerline construction methods to build containment embankments. Stage 1 construction spanned from the end of 1981–1986, covering the upper half of the slope. Stage 2 construction spanned from 1986–1992, initiating the covering of the lower half of the slope. Stage 3 construction continued from 1992–1997, containing the largest volume of material and spanning the entire length of the slope, covering both Stages 1 and 2. The final construction of Stage 4 spanned from 1997–1998, covering approximately the same area as Stage 1 [DICTUC 2012]. After closure of the facility, the tailings area was partially covered with a thin 6-in. layer of gravelly material.

Limited available information indicates that during Stage 4, the down-slope embankment was built nearly atop the down-slope embankment built during Stage 1 [DICTUC 2012]. Stage 3 covered the entire area, which resulted in a continuous potentially weak horizontal plane between the lower and upper material. During the earthquake Stage 4 acted as a driving mass at the head of the slope. The boundary between Stage 3 and the material below it became the zone in which liquefaction occurred.

The strong ground shaking resulted in liquefaction of susceptible tailings material. The flow failure took two paths: an easterly and southerly direction. The leading edge of the easterly flow traveled approximately 165 m (540 ft), whereas the southerly flow traveled roughly 350 m (1150 ft) based on air photos rendered into CAD [Gebhart 2016]. Approximately 231,660 m^3 (303,000 yds^3) of material displaced in roughly two equal halves; see the slope stability analyses presented herein.

Strong ground shaking as measured in the town of Talca is shown in Table 2.1 below (after Boroschek et al. [2012]). The Las Palmas site is located roughly 20 km NNW from Talca in a direction closer to the 2010 rupture plane, so ground shaking at the site [ignoring two-dimensional (2D) site response effects] can reasonably be assumed to be in line with that recorded in Talca.

Table 2.1 Ground shaking recorded in Talca (after Boroschek et al. [2012]).

Code Station (Rrup)	Channel	PGA (g)	PGV (cm/s)	PGD (cm)	5% damped PSa (g)					Arias Intensity (m/s)	Significant Duration (sec)	Central Freq. (Hz)
					0.1sec	0.2sec	1sec	2sec	3sec			
TAL	L	0.48	28	4	0.77	1.22	0.31	0.13	0.05	11.61	69.9	5.14
(66 km)	T	0.42	34	7	1.01	1.79	0.38	0.19	0.08	11.06	71.9	5.14

.

Figure 2.1 **Location of Las Palmas tailings dam; regional inset and local with coordinates.**

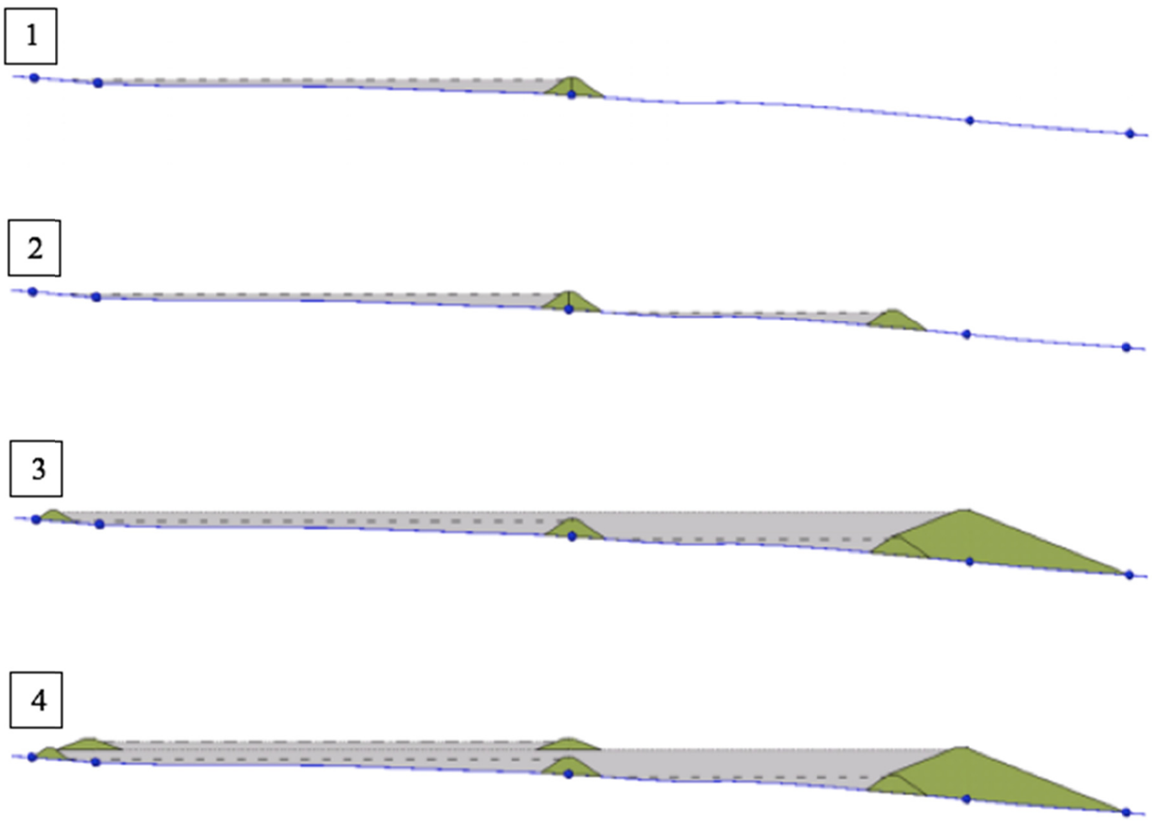


Figure 2.2 Construction Stages 1-4 of tailings embankments (from DICTUC [2012]).

3 GEER Investigations

The Las Palmas site was investigated by GEER personnel on 11 March 2010, with a follow-up visit on 28 March 2010 to collect LIDAR and *VS* measurements. The reconnaissance team found the flow failure marked by a large scarp (Figures 3.1–3.3) and sand boils throughout the failed material (Figure 3.4) and other locations. Figure 3.5 shows the ground cracking and layered nature of the underlying tailings materials. The only eyewitnesses of the flow failure were killed; therefore, it is not known if the event was primarily co-seismic or post-seismic in nature. Because of the observed sand boils and the large distance of runout, the subsequent analysis assumed that this failure could be characterized as liquefaction initiation, followed by post-liquefaction flow-failure deformations that were not seismic-loading dependent.

Figure 3.1 Upper scarp of failed tailings impoundment looking southwest (from GEER [2010]).

Figure 3.2 Upper scarp of failed tailings impoundment looking northeast (from GEER [2010]).

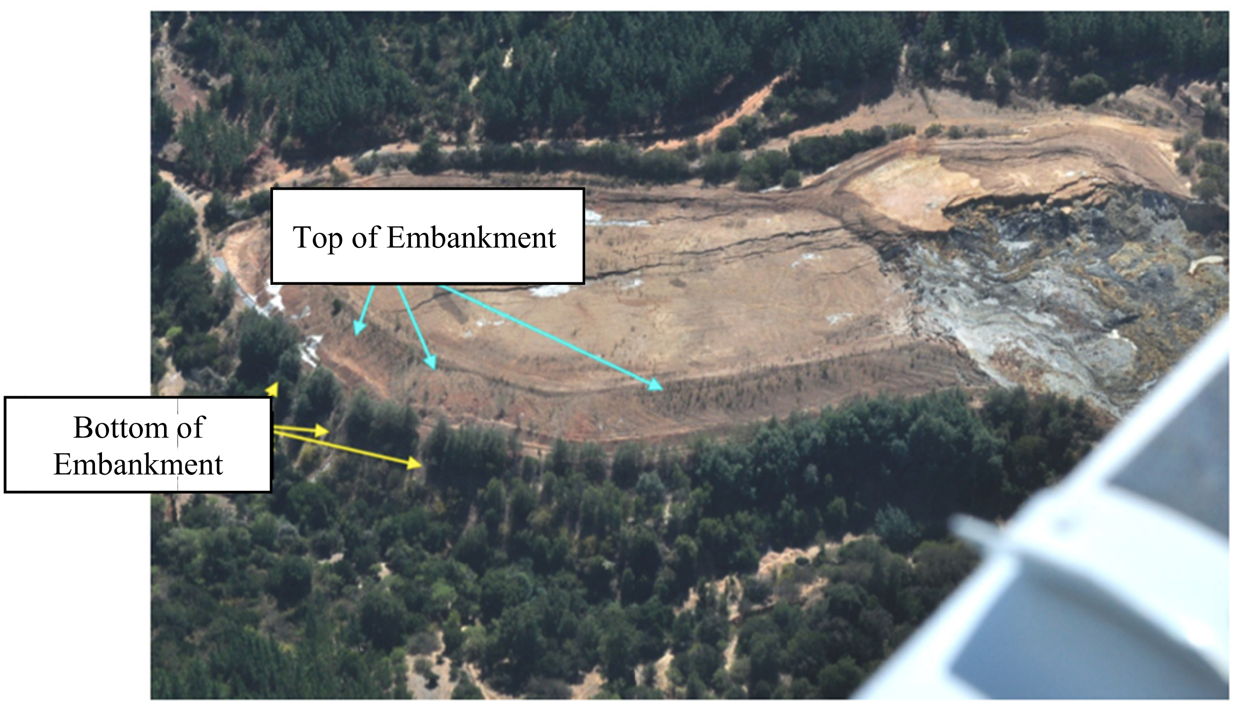


Figure 3.3 Aerial photograph showing scarp, longitudinal cracks, and the location of the tailings dams (after DICTUC [2012] and Gebhart [2016]).

Figure 3.4 Sand-boil tailings along flow path (S35.18872, W71.75777) looking up towards the scarp is shown in Figures 3.2 and 3.3 (from GEER [2010]).

Figure 3.5 **Gravelly cover layer over oxidized and unsaturated tailings, S35.184679 S, W71.759410 (from GEER [2010]).**

LIDAR measurements of the failed slope were performed. The terrestrial LIDAR technique [three-dimensional (3D) laser scanning] consists of sending and receiving laser pulses to build a point file of 3D coordinates of the scanned surface. The time of travel for a single pulse reflection is measured along a known trajectory that computes the distance from the laser and the position of a point of interest. Using this methodology, data collection occurs at rates of thousands of points per second, generating a "point cloud" of 3D coordinates. LIDAR measurements were conducted by Dr. Rob Kayen as part of the reconnaissance efforts, and the point-cloud data was processed by the third author, Professor David Frost.

A laser scanner was used to conduct a tripod-mounted survey (Figure 3.6). Multiple scans were collected during each site survey to fill in "shadow zones" of locations not directly in the line-of-sight of the laser and to expand the range and density of the point data. Data were collected at a rate of 8000 points per second, scanning a range of 360° in the horizontal direction and 80° in the vertical direction. For all sites, a project coordinate system was used to register and reference multiple scan locations into one large project file. No global geo-referencing using differential GPS was performed. Scanning and registration were performed using Riscan-Pro. Point-cloud processing and surface modeling of the data was performed using I-SiTE software specifically designed to handle laser-scan data.

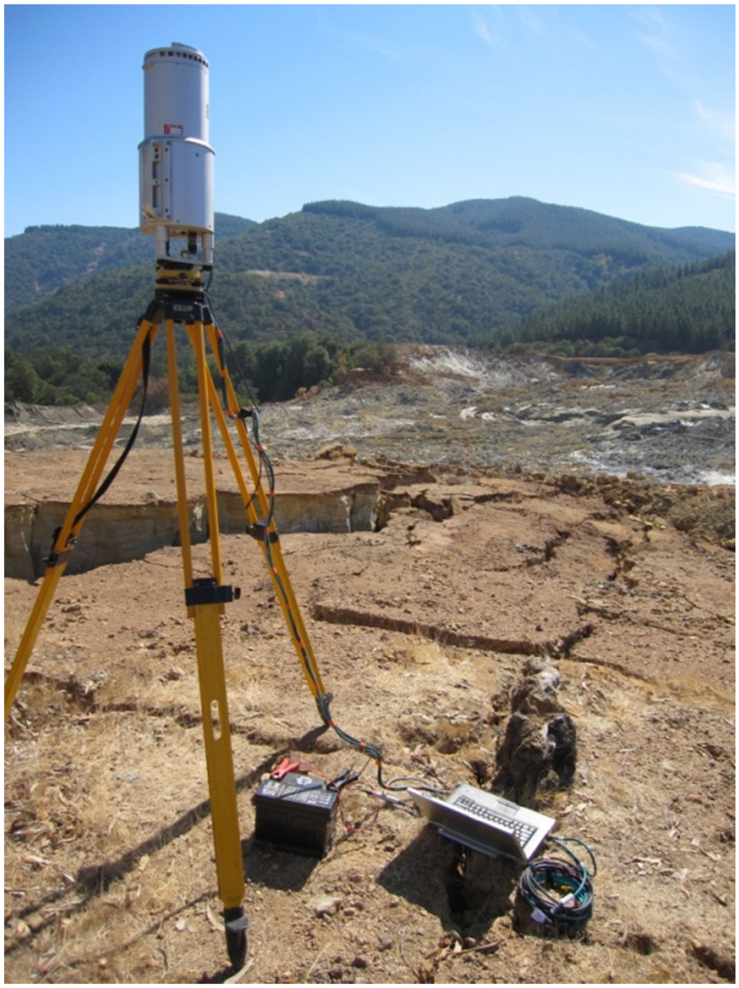


Figure 3.6 Terrestrial laser scanning with a Riegl z420 LIDAR unit at the Las Palmas Mine tailings dam failure. The system uses a PC for data acquisition and a car battery to power the laser (from GEER [2010]).

The registration of LIDAR data involves merging two, or more, individual scans data to form a single model of the reconnaissance site area, termed the "registration process." A best fit translation and rotation registration process—using millions of points—aligns the overlapping data within a pair of point-cloud datascans.

Point data from each set of scans were subjected to a series of filters to remove non-ground surface and extraneous laser returns from the point clouds. Points reflected from vegetation and other non-ground conveyance features were manually cropped from each of the point clouds. Next, an isolated point filter was used to remove single-point instances occurring above the land surface. These isolated points are usually a result of reflections from moisture in the atmosphere. Topographic filters that select the lowest point in the point clouds were used to remove vegetation from point clouds. Here, the entire dataset was divided into 5 to 10 cm square bins, and only the lowest points within the bins were selected.

The final product of LIDAR data processing is 3D surface models. A linear interpolation method is used to process the surface models to generate surface edges of the triangular irregular network (TIN) facets between points. Triangular irregular network models represent a topographic surface of each area. After filtering, a TIN surface was generated from each scan file using either a spherical surface algorithm (curved facets) or a linear topographic algorithm (flat facets); see Figure 3.7.

Figure 3.7 Detailed LIDAR of head scarp of Las Palmas tailings dam failure (from GEER [2010[).

4 Litigation-Supported Field Investigations

In support of ongoing litigation concerning this flow failure, the fourth author (Professor Christian Ledezma) and colleagues performed field investigations to gather subsurface information on the failure material [DICTUC 2012]. Between June 2–26, 2011, they conducted geotechnical exploration including borings with SPT measurements. Five 4-in. diameter exploratory borings were advanced to depths ranging from 8.5–21.0 m (28–70 ft) below ground surface, typically terminating in the competent native material below the tailings. As indicated in Figure 4.1, these locations are within the non-displaced portion of the dam. Borings B-1 and B-5 were located within the containment embankments, and borings B-2, B-3, and B-4 were located within the tailings material. The drill rig was equipped with an automatic safety hammer to obtain blow counts, with an estimated efficiency of approximately 60%. Corrected ($N_{1,60}$) SPT blow counts for fine and coarse-grained materials were between single digits to the lower teens range: see Figure 4.2. The field crew encountered groundwater in four of five SPT borings at depths ranging from 5–13 m (17–43 ft) below the ground surface, which is thought to be not grossly dissimilar to the ground water present at the time of the earthquake. Vane shear testing was also performed in conjunction with some of the borings to measure the peak strength of the intact material (see Table 4.1). Subsequent laboratory testing was performed on samples acquired from the drilling operation. The following bore logs (Figures 4.3–4.7) and laboratory data (see Table 4.1) were compiled by Gebhart [2016] and used to guide subsequent investigations.

Table 4.1 Summary of field and lab data performed by DICTUC [2012]; from Gebhart [2016].

			Depth			Strength		Unit Wt. (pcf)			Lab Strength Testing			
Location	Field Test	Location	(m)	(ft)	Soil Type	Cohesion (psf)	Φ (°)	Total	Dry	Fail Strain (%)	CU	Cyclic Triaxial	UC	Sample Method
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	-	-	-	-	-	-	-	-
B-2	Vane Shear	"Bowl"	7	23	ML	780	-	-	-	-	-	-	-	-
			13	43	ML	818	-	-	-	-	-	-	-	-
B-3	Vane Shear	"Bowl"	5	17	ML	780	-	-	-	-	-	-	-	-
			8	26	ML	502	-	-	-	-	-	-	-	-
B-4	SPT	"Bowl"	-	-	-	-	-	-	-	-	-	-	-	-
B-5	SPT	"Wall"	5.0-5.5	17	ML	5600	-	110	96	8.3	X	X	-	Undisturbed
			10.0-10.5	34	ML	-	-	-	-	-	-	X	-	-

CU: Consolidated Undrained
UC: Unconfined Compressive

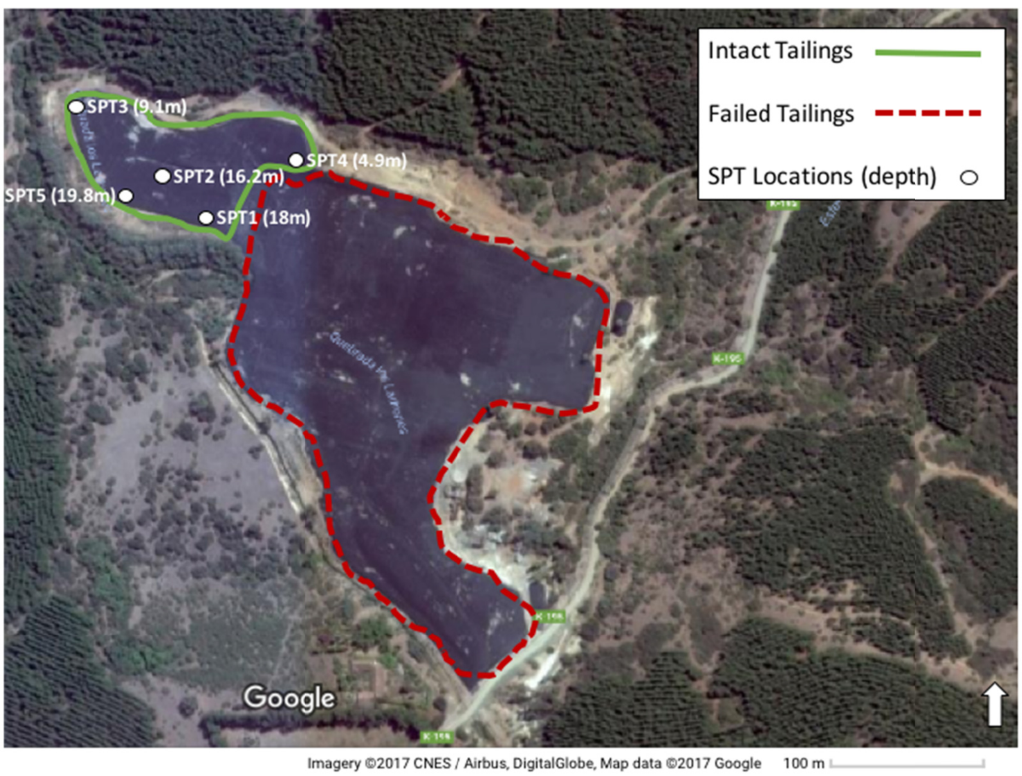


Figure 4.1 Location of SPT borings performed by DICTUC [2012], with the depth of each boring shown in parenthesis. The intact and failed material zones are delineated.

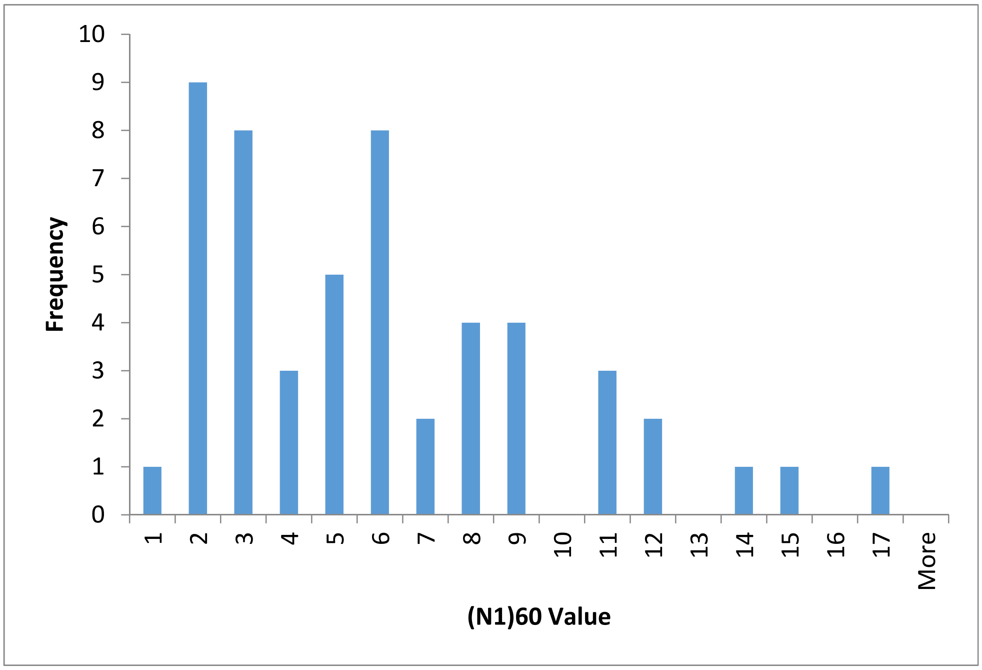


Figure 4.2 **Histogram of corrected SPT blow counts, $(N_1)_{60}$, from the field investigations (from Gebhart [2016]).**

BORING LOG

BORING NO. **B-1**

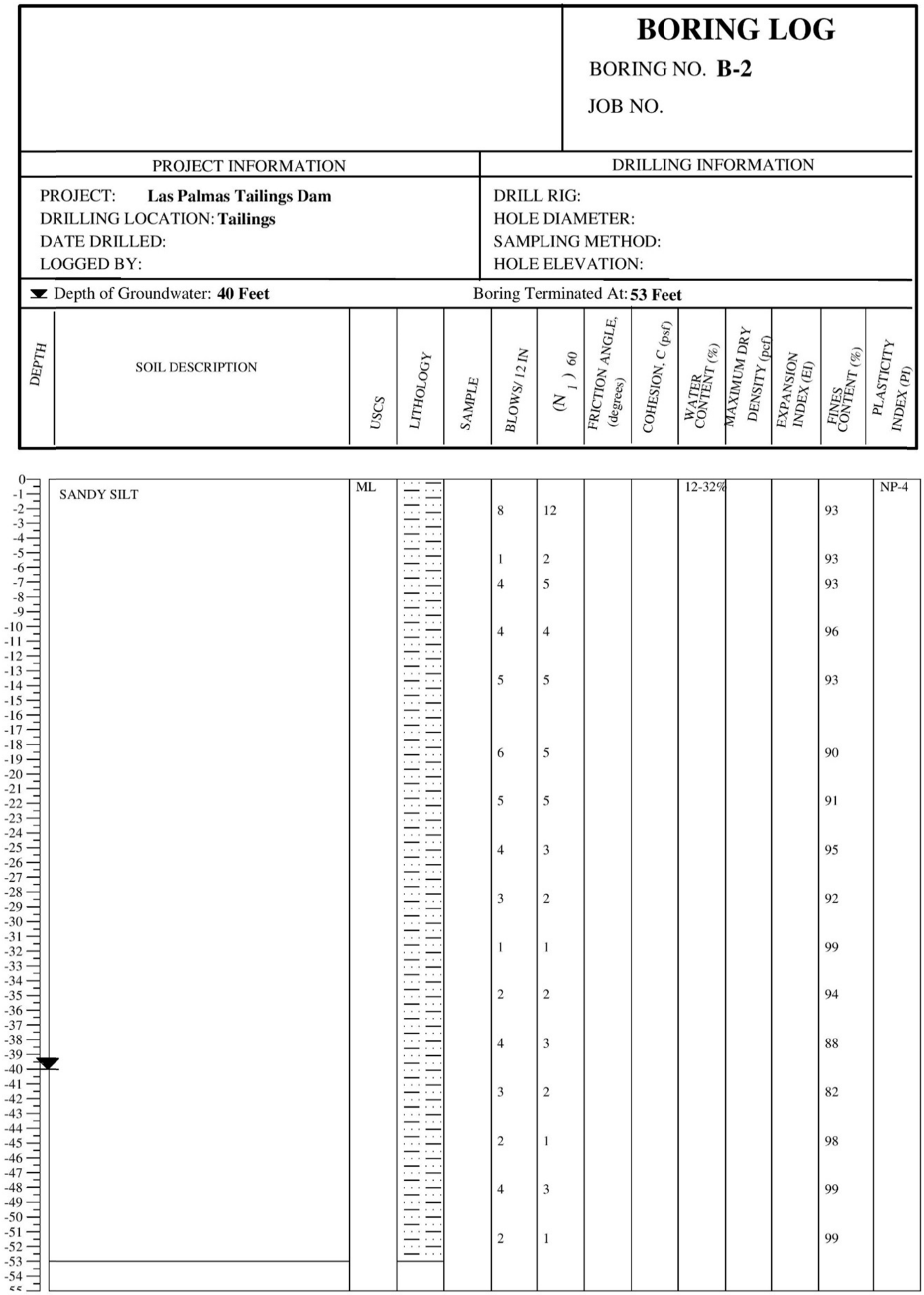
JOB NO.

PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Las Palmas Tailings Dam**	DRILL RIG:
DRILLING LOCATION: **Containment Wall**	HOLE DIAMETER:
DATE DRILLED:	SAMPLING METHOD:
LOGGED BY:	HOLE ELEVATION:

Depth of Groundwater: **43 Feet** Boring Terminated At: **70 Feet**

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)
SILTY SAND	SM							5-11%				NP
				9	14						35	
				7	11						35	
				11	16						33	
				8	10						31	
				4	4						31	
				7	7						37	
				9	8						43	
				7	7						43	
				6	6						43	
				8	7						27	
			Shelby @ 20'	9	8						34	
				9	7						48	
				8	6						48	
SANDY SILT:	ML							20-21%				2-4
				2	2						77	
				6	4						77	
				4	3						82.5	
				4	3						82.5	
Liquefied material leaves the sampler				3	2						88	
				5	3						77	
				4	2						77	
			SH-20 @ 54'	3	2						65	
				3	2						76	
				2	1						76	
SILTY CLAYEY SAND: (BEDROCK)	SC-SM							8-13%				8-12
			UC @ 62'									
			UC @ 67'									
BEDROCK	SC											

Figure 4.3 Boring B1 performed by DICTUC [2012]; from Gebhart [2016].



BORING LOG

BORING NO. **B-2**

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: **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)
	SANDY SILT	ML							12-32%				NP-4
					8	12						93	
					1	2						93	
					4	5						93	
					4	4						96	
					5	5						93	
					6	5						90	
					5	5						91	
					4	3						95	
					3	2						92	
					1	1						99	
					2	2						94	
					4	3						88	
					3	2						82	
					2	1						98	
					4	3						99	
					2	1						99	

Figure 4.4 Boring B2 performed by DICTUC [2012]; from Gebhart [2016].

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
					10	15						83	
					7	11						79	
					5	6						76	
					6	6						90	
					5	5						81	
-20					2	2						75	
					10	9						73	
					7	6						59	

Depth scale: 0, -1, -2, -3, -4, -5, -6, -7, -8, -9, -10, -11, -12, -13, -14, -15, -16, -17, -18, -19, -20, -21, -22, -23, -24, -25, -26, -27, -28, -29, -30

Figure 4.5 Boring B3 performed by DICTUC [2012]; from Gebhart [2016].

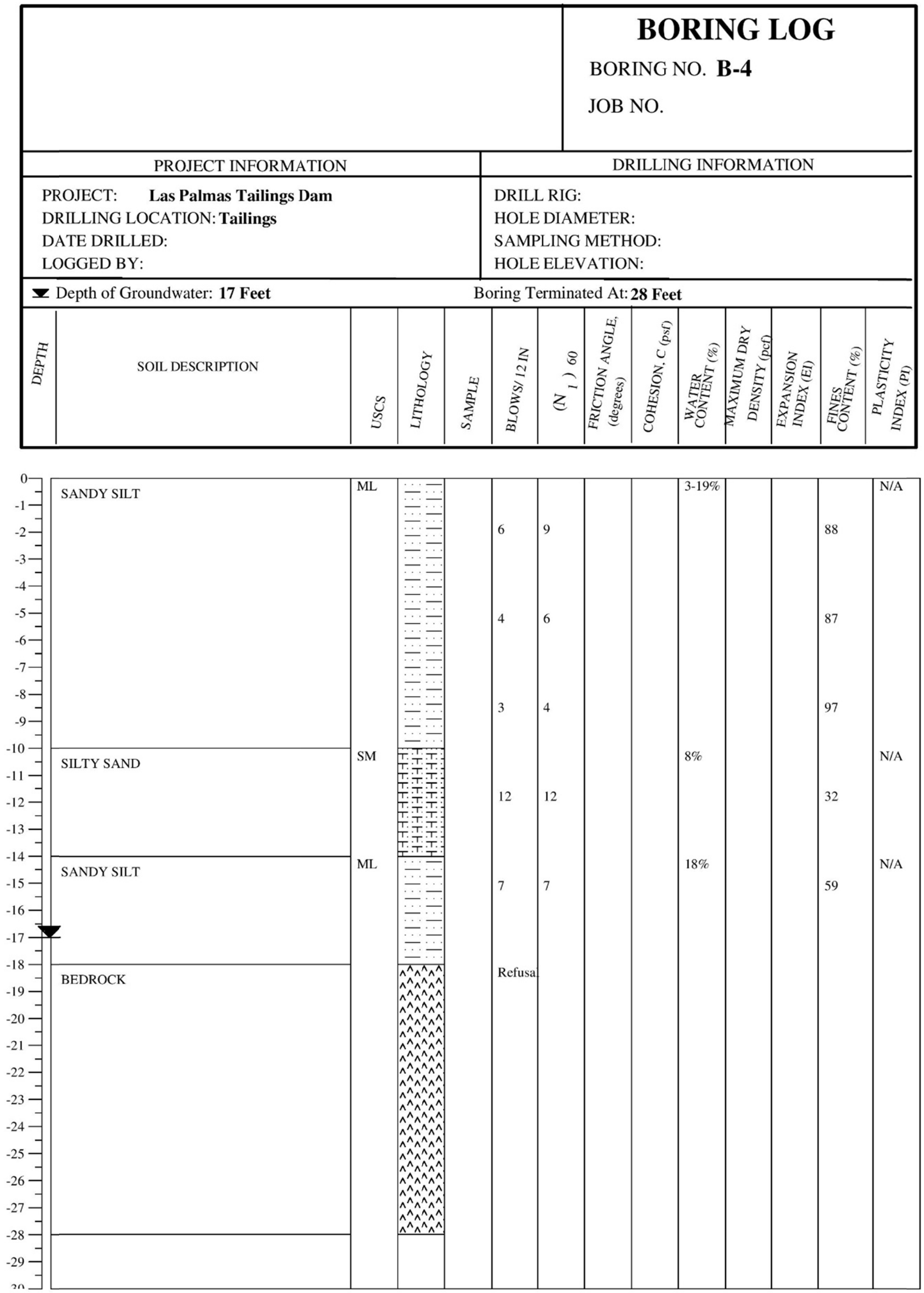


Figure 4.6 Boring B4 performed by DICTUC [2012]; from Gebhart [2016]).

BORING LOG

BORING NO. **B-5**

JOB NO.

PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Las Palmas Tailings Dam**	DRILL RIG:
DRILLING LOCATION: **Containment Wall**	HOLE DIAMETER:
DATE DRILLED:	SAMPLING METHOD:
LOGGED BY:	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
-8	SANDY SILT	ML		Triax @ 17' Cyclic Triax @ 34'					10-16%			62-81%	NP
-40	SANDY SILT	ML							21-26%			65-98%	1-5
-65	BEDROCK												

Figure 4.7 Boring B5 performed by DICTUC [2012]; from Gebhart [2016].

5 PEER NGL-Funded Investigations

The field investigations funded by PEER–NGL occurred over the time frame of 2017 June 20–23. The goal was to augment the existing SPT measurements with CPT and surface-wave measurements. Participants in the field investigations included the four authors of this report.

An initial site visit was conducted on June 20th. The CPT equipment was subsequently mobilized for three full days of field testing and included a portable CPT "ramset" (Figure 5.1), which is deployable in locations not accessible by typical CPT trucks, and a geophysical array of twelve geophones and accompanying seismograph for performing passive surface-wave measurements. Three CPT soundings were performed per ASTM 5778; the locations are shown in Figure 5.2, two of which were collocated with the SPT measurements.

Figure 5.1 Portable "ramset" mobilized for Las Palmas field investigations because of the difficult access conditions. The reaction weight was provided by water tanks and truck. The ramset is the white hydraulic jack in the middle. The black box with orange cord to the right is the hydraulic pump.

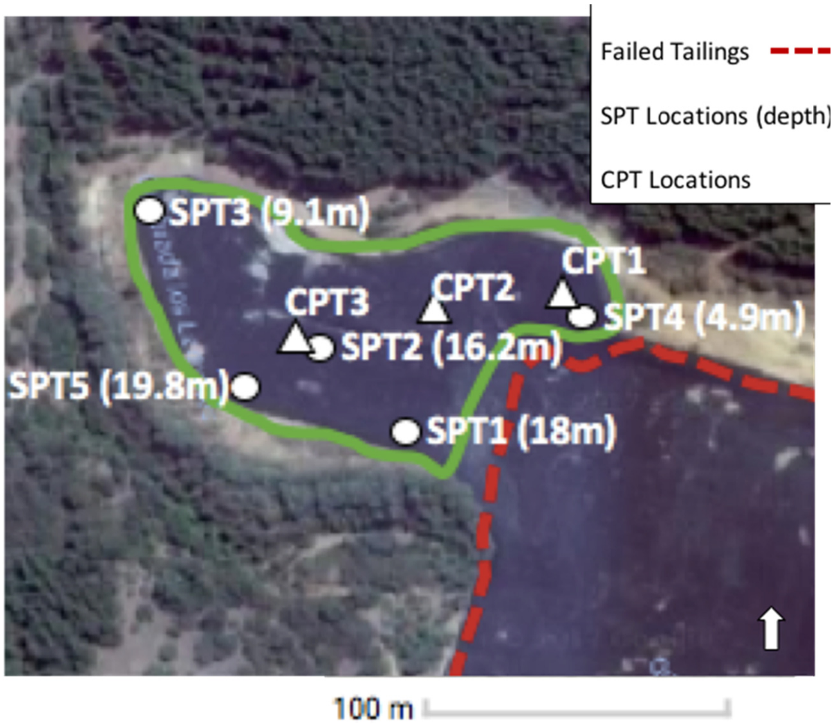


Figure 5.2 Location of CPT soundings with respect to prior SPT borings.

Passive surface-wave measurements were made using circular arrays of twelve 4.5 Hz vertical geophones (Figure 4.10). The locations of the passive measurements are shown in Figure 4.11. G1, G2, and G3 were collocated with the CPT measurements in the intact tailings. G5 and G6 were collocated in the failed portion of the tailings, where G5 was located in the flow failure and G6 in an intact block that was transported in the flow failure. The diameters of the arrays were 5m, 10 m, and 20 m (16.40 ft, 32.81 ft, and 49.21 ft, respectively) ftm, depending on the depth and resolution required. Recordings were made at a 2 m/sec sampling rate for 32 sec, with 10 of these recordings made for each array and concatenated for processing. Different array diameters were combined into single dispersion curves in those cases when it provided clearer resolution. Dispersion curves were arrived at using SPAC [Aki 1957] as coded in Geogiga Surface Plus [Geogiga 2017]. Dispersion curve picking and shear-wave velocity profile fittings were performed within Geogiga. To minimize interpretation uncertainty given the lack of prior knowledge of the stratigraphy, layering within a *VS* profile was typically limited to three layers. The estimated shear-wave velocity profiles are shown in Figures 5.8–5.13, with a nominal coefficient of variation of 10% [Moss 2008]. The dispersion curve picks and profile fitting details are provided in the Appendix. Table 5.1 shows how the SPT, CPT, and *VS* measurements are co-located with respect to each other.

Figures 5.5–5.7 show the results of the CPT, and Figures 5.8-5.13 shown the *VS* data collected at the site. The CPT data was collected and processed by LMMG Geotecnia Limitada. The *VS* data was collected and processed by the first author. Full reporting of the CPT and *VS* measurements can be found in the Appendix.

Table 5.1 Showing co-location of different field tests.

CPT	*VS*	SPT	Coordinates
CPT1	G1	SPT4	S35.184242 W71.759540
CPT2	G2		S35.184297 W71.760284
CPT3	G3	SPT2	S35.184350 W71761197
		SPT1	
		SPT3	
		SPT5	
	G5		S35.185729 W71.758658
	G6		S35.186669 W71.758161

Figure 5.3 Shown is a 10-m circular array at location G5. Passive measurements were made of ambient noise from a generator, foot falls, etc.

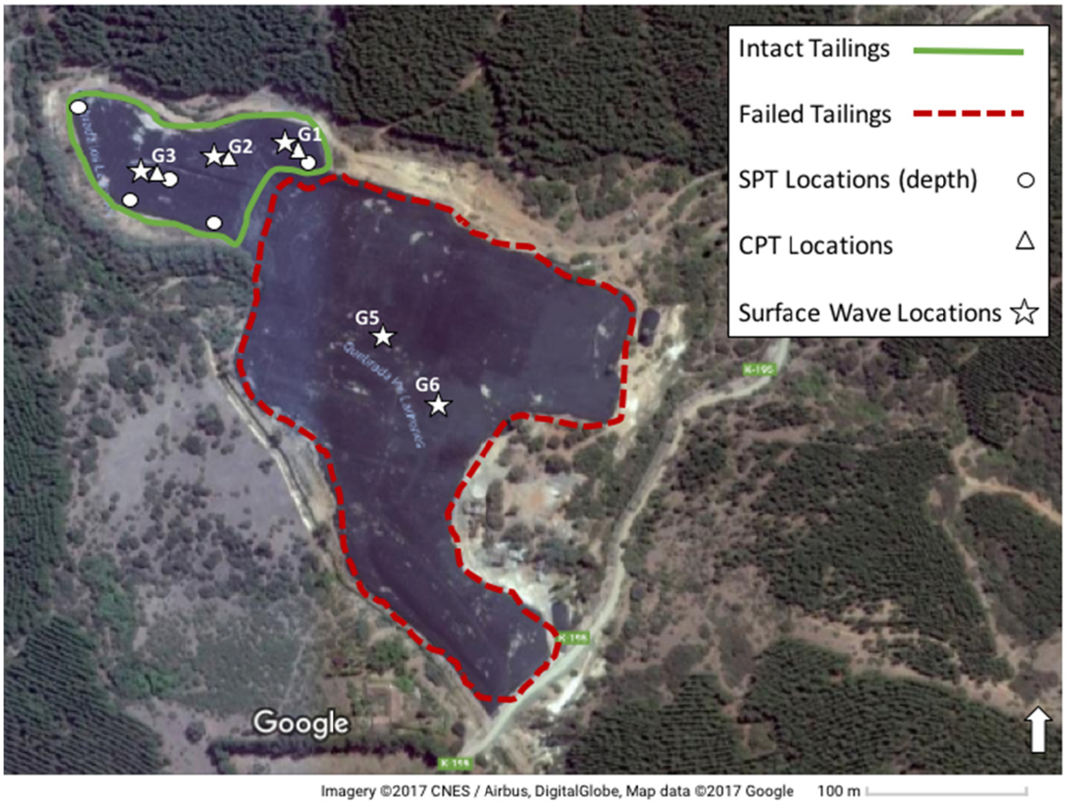


Figure 5.4 **Locations of passive circular array measurements with respect to CPT soundings and SPT borings.**

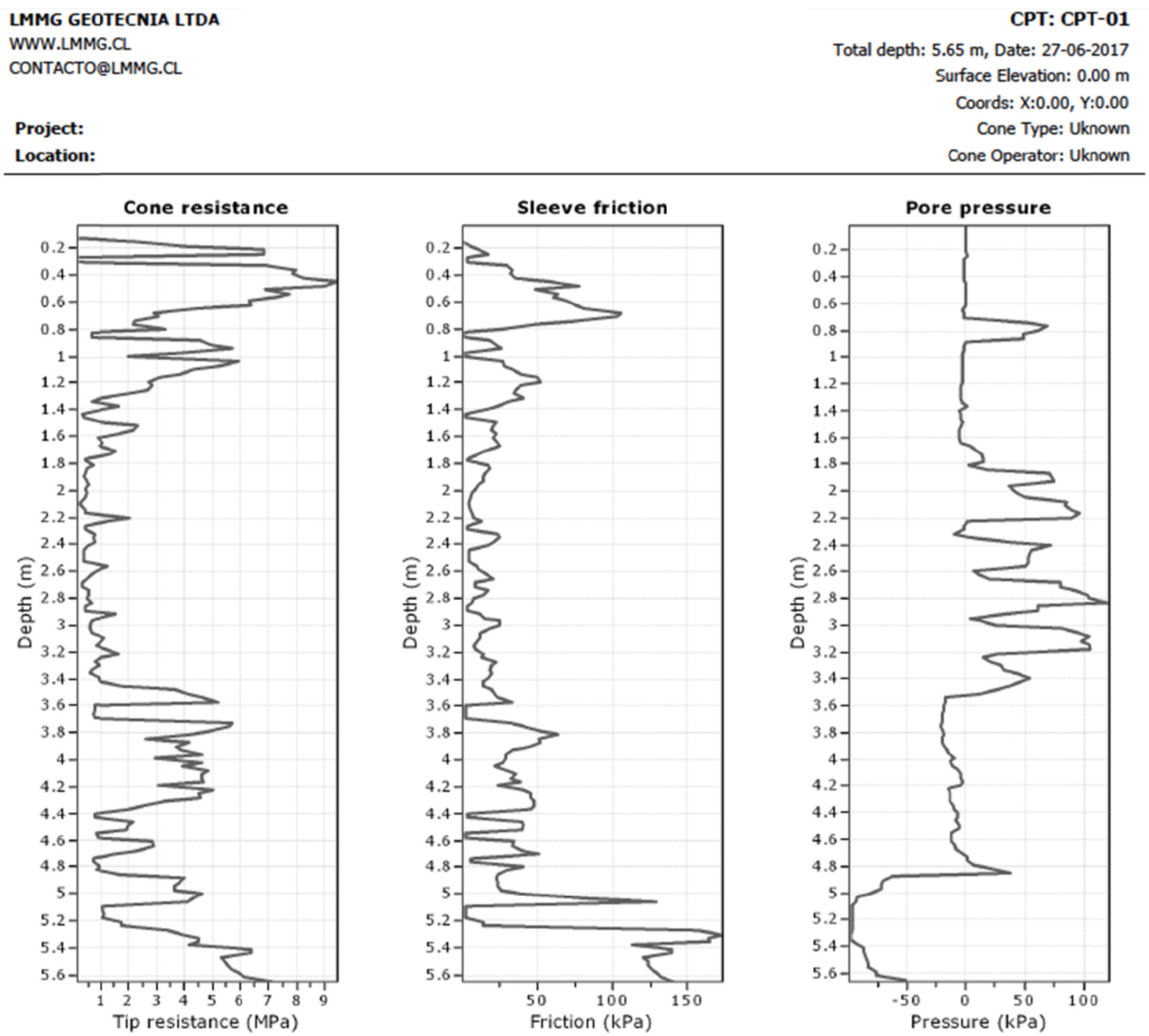


Figure 5.5 **CPT1 cone tip, sleeve, and pore pressure measurements. Location S35.184242 W71.759540; elevation ~155 m.**

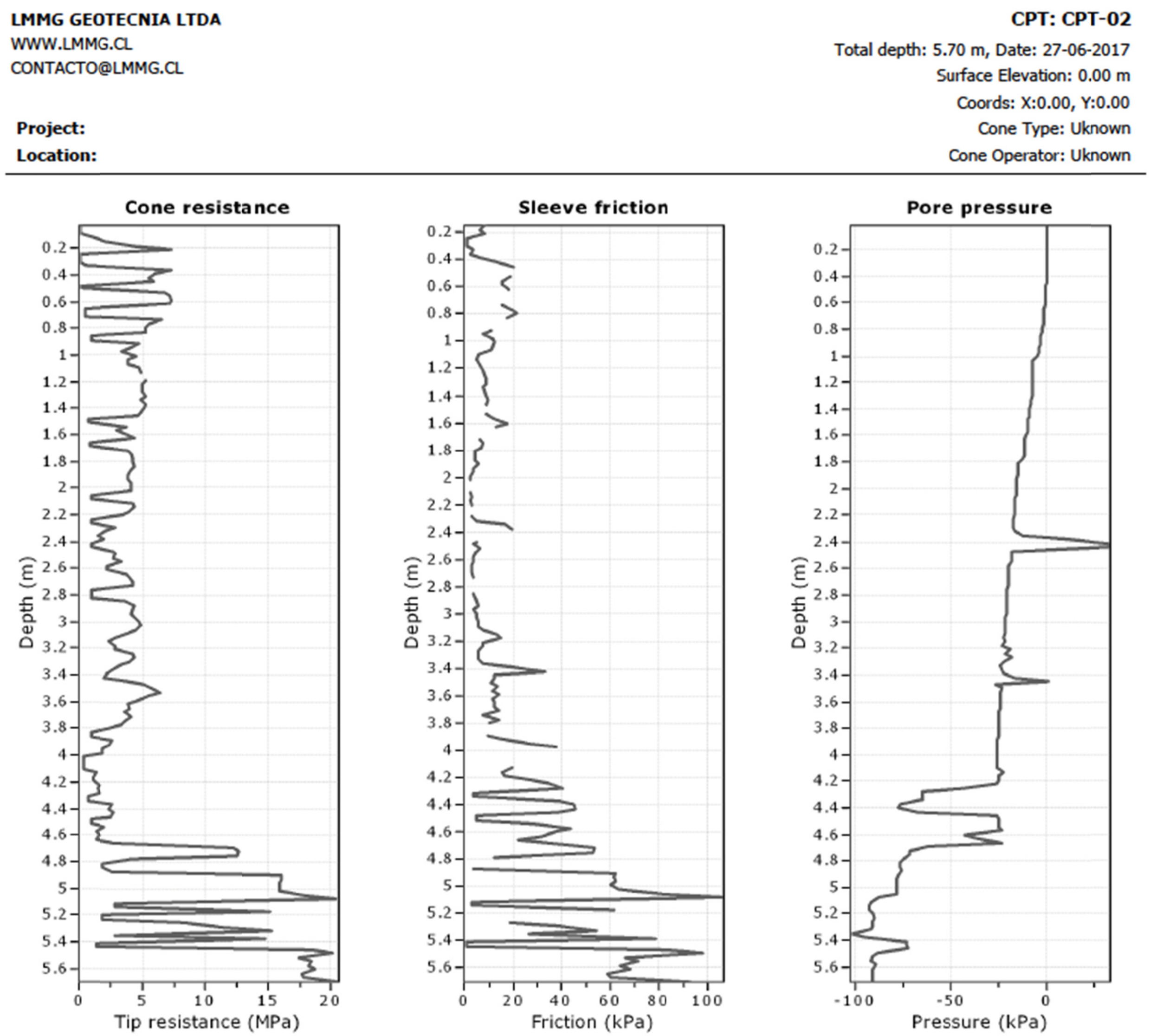


Figure 5.6 **CPT2 cone tip, sleeve, and pore pressure measurements. Location S35.184297 W71.760284; elevation ~155 m.**

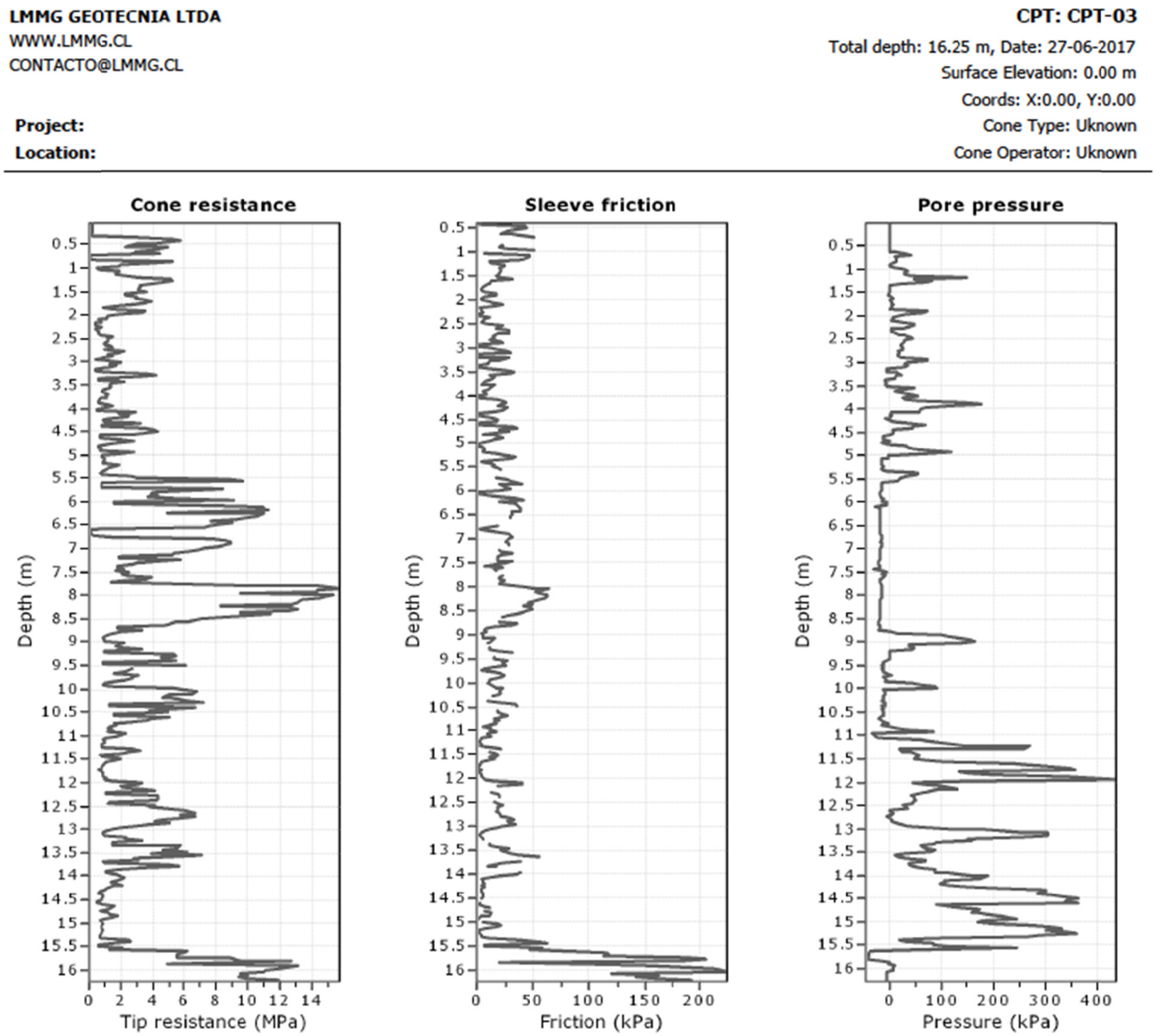


Figure 5.7 CPT3 cone tip, sleeve, and pore pressure measurements. Location S35.184350 W71.761197; elevation ~153 m.

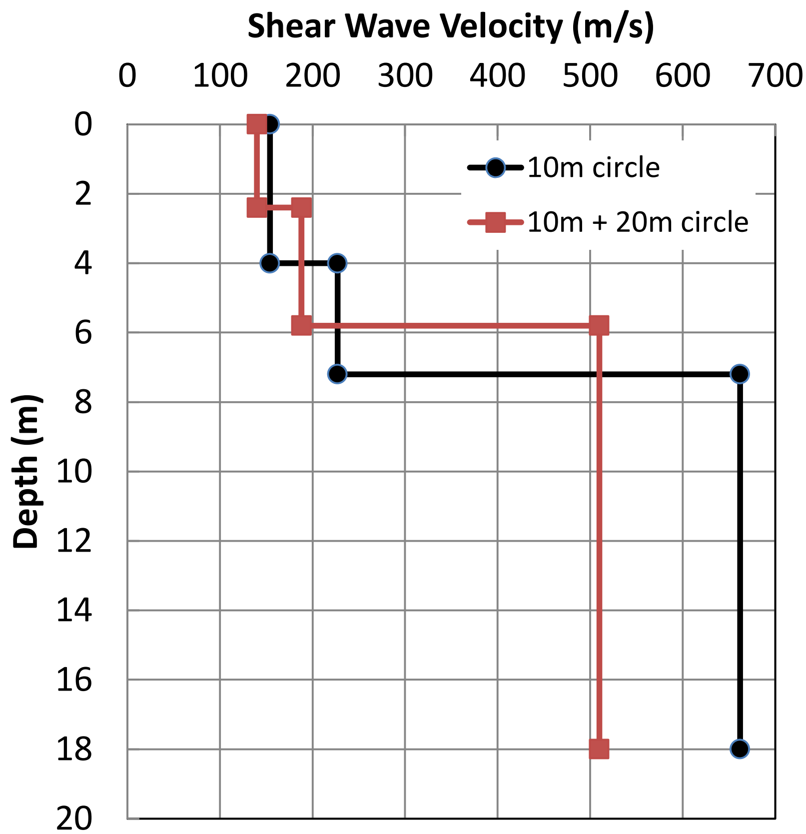


Figure 5.8 **Location G1 *VS* measurements. Location S35.184242 W71.759540; elevation ~155 m.**

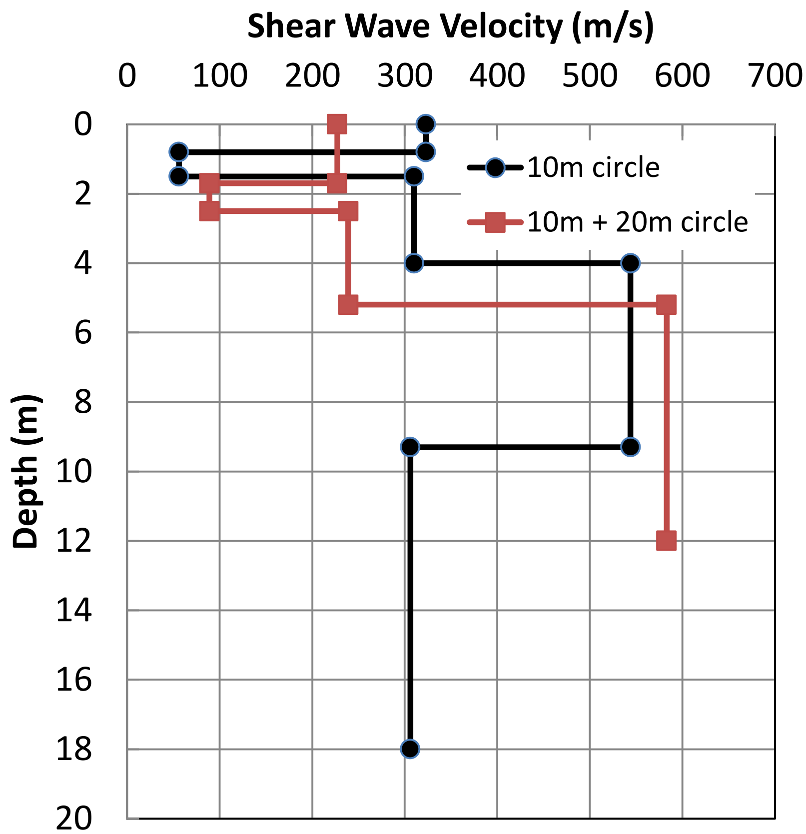


Figure 5.9 **Location G2 *VS* measurements. Location S35.184297 W71.760284; elevation ~155 m.**

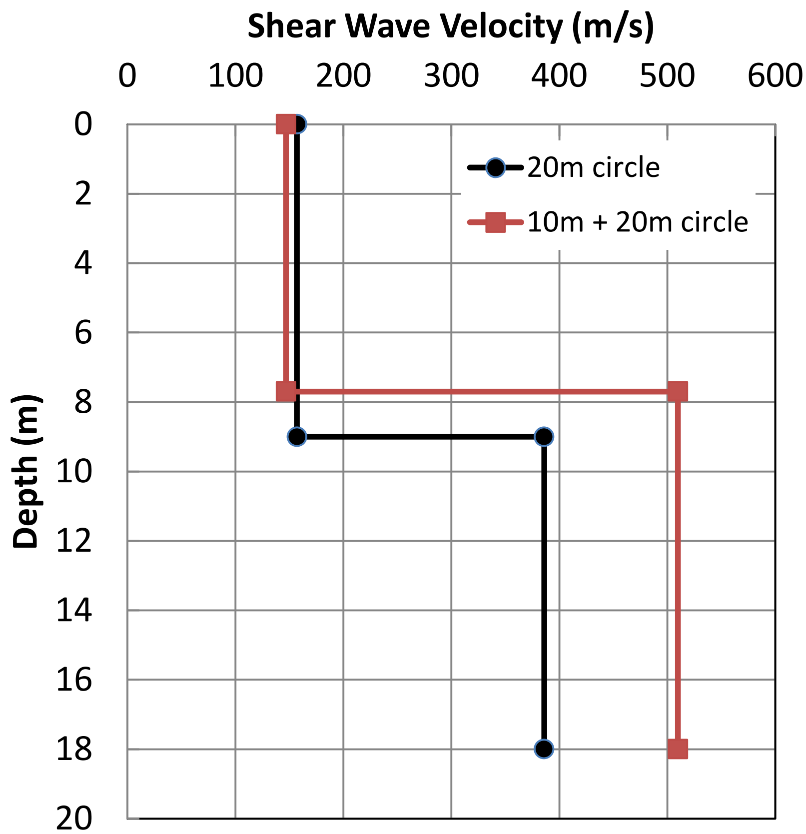


Figure 5.10 **Location G3 *VS* measurements. Location S35.184350 W71.761197; elevation ~153 m.**

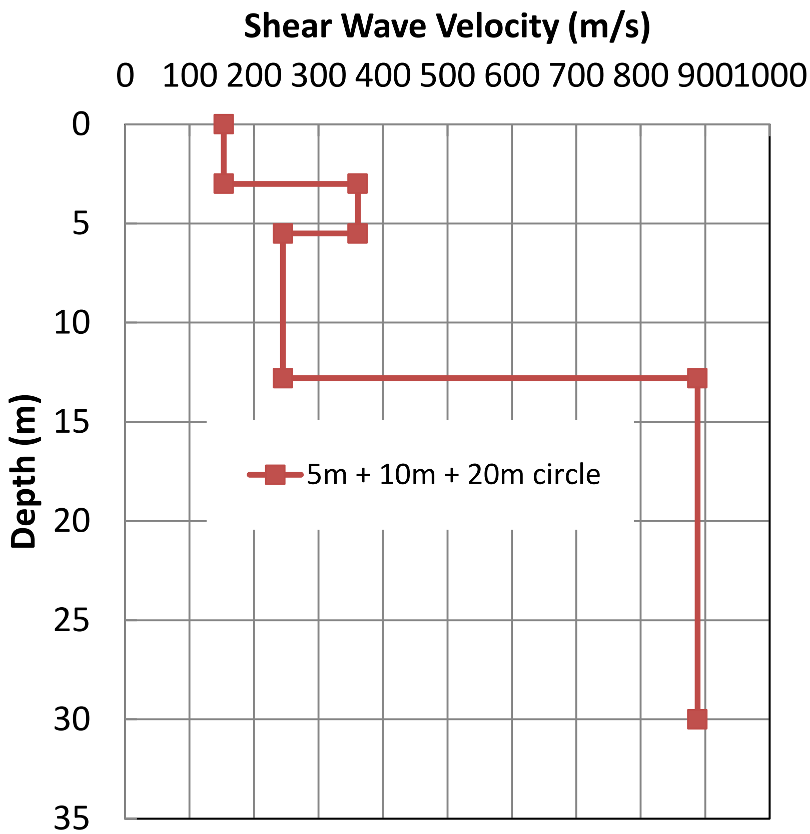


Figure 5.11 Location G5 *VS* measurements of flow slide material. Location S35.185729 W71.758658; elevation ~140 m.

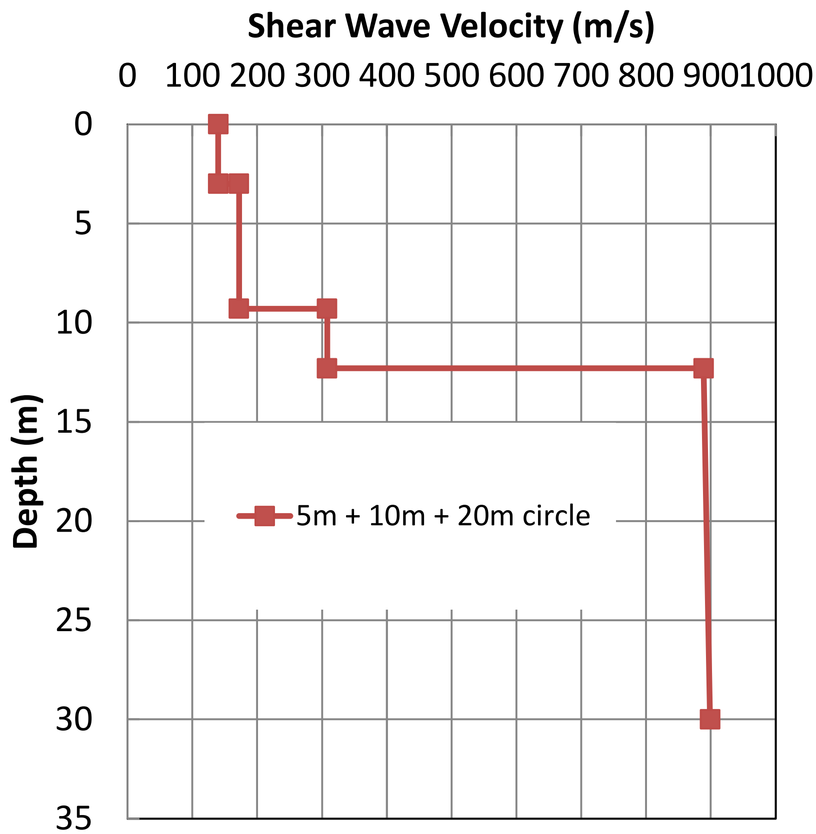


Figure 5.12 **Location G6 *VS* measurements of translated block material. Location S35.186669 W71.758161; elevation ~134 m.**

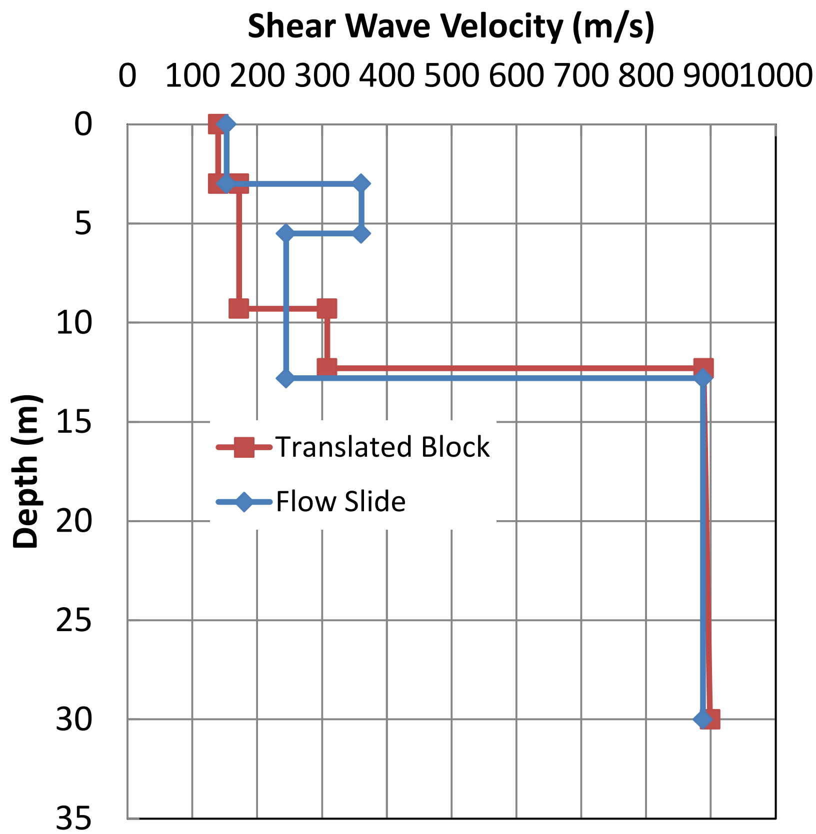


Figure 5.13 Comparison of G5 and G6 *VS* measurements.

6 Back-Analysis of Residual Strength

This section presents a back-analysis of the flow failure to estimate the residual strength of the liquefied material. A full description of this analysis and the details can be found in Gebhart [2016]. The strong ground shaking from the 2010 event resulted in liquefaction of some portion of the tailings material. The liquefied flow failure took two paths: an easterly and southerly direction. The leading edge of the easterly flow traveled approximately 165 m (540 ft), whereas the southerly flow traveled roughly 350 m (1150 ft). Based on the failed geometry plan view and the depth of the failed material, a total of approximately 231,660 m^3 (303,000 $yards^3$) of material displaced in roughly two equal halves; see Figure 6.1. The flow failure debris was approximately 1.5-4.0 m (5-13 ft) thick in some locations.

Boring logs indicate groundwater was located between depths of 5–13 m (17–43 ft) below the ground surface across the undisturbed portion of the tailings. This ground water table at the time of drilling is considered roughly representative of the ground water table at the time of the earthquake. This was corroborated by the location of seepage exiting the exposed failure slope in reconnaissance observations. Sand boils were also observed in numerous locations (Figure 6.2) throughout the failed mass, indicating saturated conditions of that material during failure.

Development of pre- and post-failure, 2D and 3D models of the Las Palmas tailings dam used AutoCAD Civil 3D (Auto Desk) and Slope/W (Geo-Slope International) was undertaken. These models considered:

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

Pre-failure geometry was estimated using as-built information, aerial images, and existing intact embankments as a guide. Post-failure geometry is based on aerial images of the failed mass. The pre-and post-failure geometries were then used for “time” stepping from the beginning to the end of the failure event, mapping the change in geometry following the incremental momentum methodology (IMM) presented in Weber et al. [2015]. An initial intact slope cross-section area of approximately 33,000 ft^2 (3065 m^2) is stepped through progressively more failed slope geometries converging on the final failed slope cross-section area of approximately 12,000 ft^2 (1115 m^2). This also applies to advancement of the toe of the slope, with displacement constrained to converge on a final observed displacement of approximately 165 m (540 ft). This yields an initial set of "time" steps that were treated as linear between each step because of the lack of prior knowledge.

The mass-property function of AutoCAD was used to calculate the centroid of mass for each step. This helped establish linear trends with respect to the following: (1) area loss and (2) centroid displacement of the failed mass. This process is iterative and requires repeated adjustment of cross-section geometry to converge upon a final solution. A typical step sequence for fully liquefied material can be seen in Figure 6.3. A similar analysis was performed for the situation where a weak layer liquefied, causing blocks of intact material to be carried downslope in the flow.

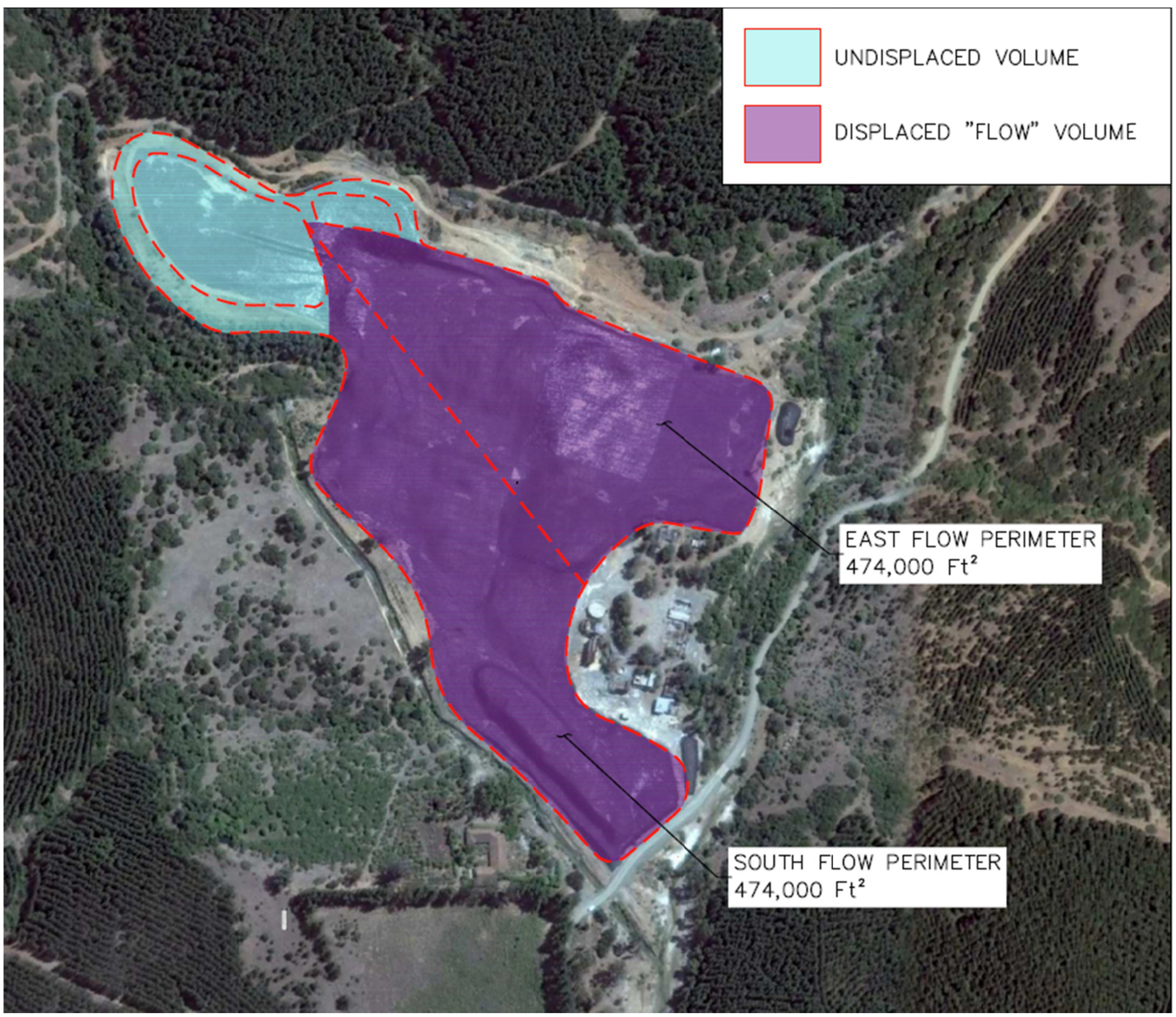


Figure 6.1 Marked image showing flow failure runout following two directions, east and south (from Gebhart [2016]).

Figure 6.2 Evidence of sand boils in the flow-failure debris (from GEER [2010]).

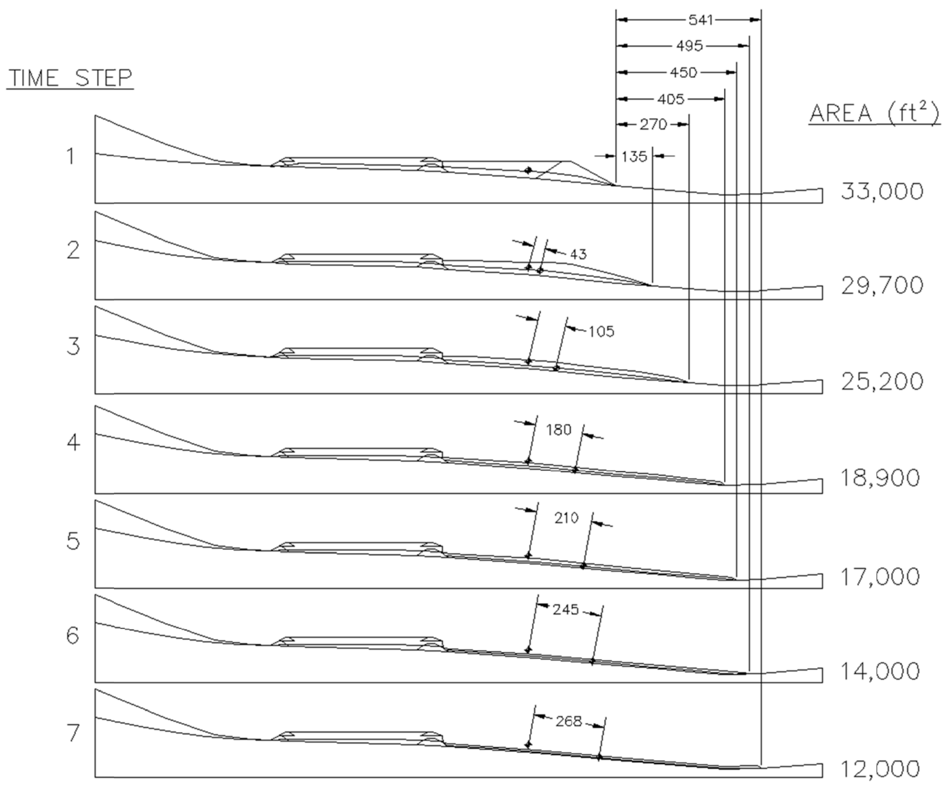


Figure 6.3 Modeled slope failure progression sequence in AutoCAD (from Gebhart [2016].

The material properties used in the Slope/W model, which were based on the subsurface drilling investigations and IMM back-analysis, are shown in Table 6.1. The Morgenstern and Price method was used to solve the limit equilibrium problem with a pre-defined slip surface; see Figure 6.4.

The average of pre- and post-failure strengths are approximated for an initial run. This strength value is used for each step to execute a series of static LEM slope stability analyses without seismic effects. Slope/W produces the following output for further IMM analysis: driving force, resisting force, slide area, slide weight, and factor of safety.

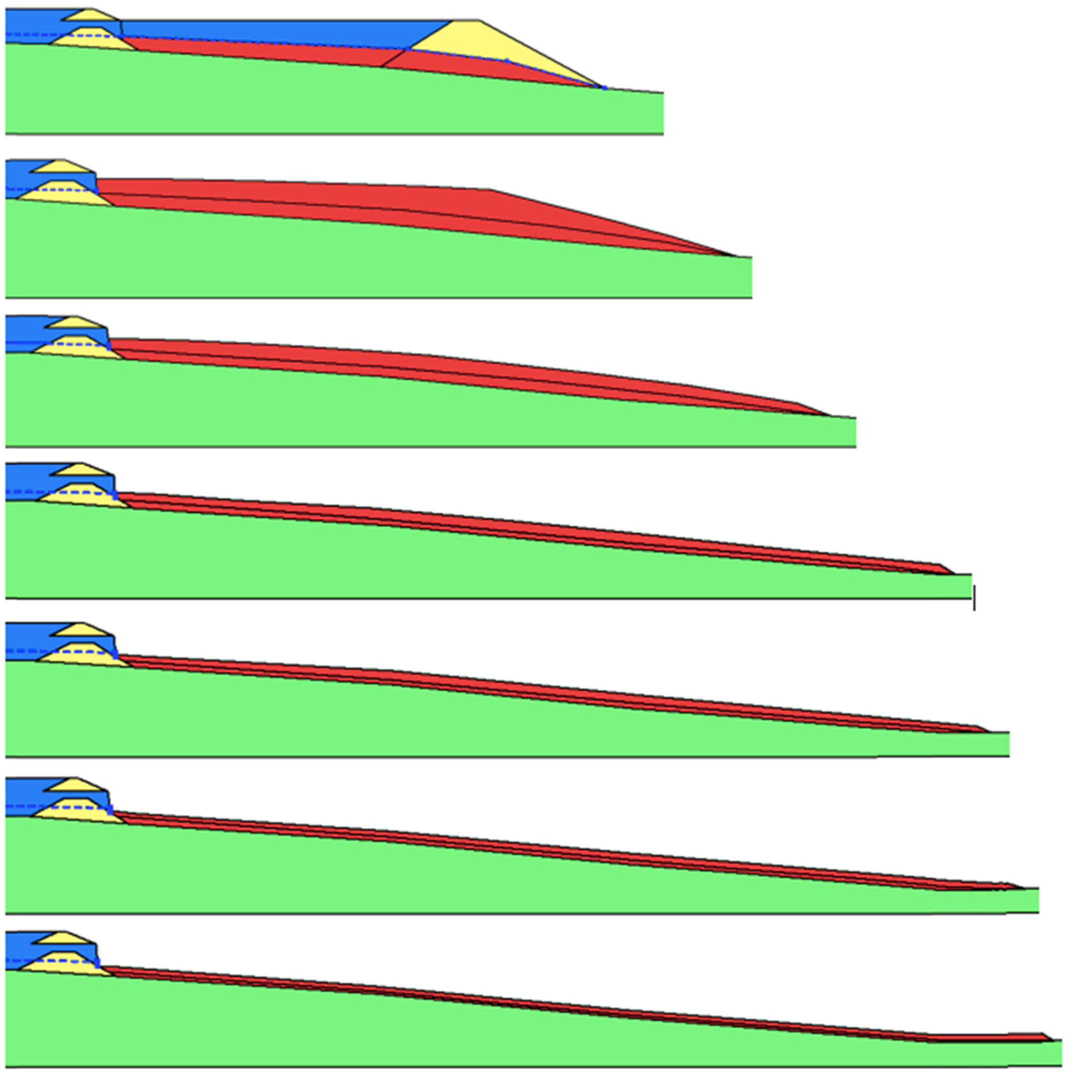


Figure 6.4 Modeled slope failure progression sequence in Slope/W (from Gebhart [2016]).

Table 6.1 Summary of materials and properties used in modeling the tailings failure.

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

A series of iterative calculations are performed, then the slide mass displacement for each step is calculated. This IMM process follows Newton's second law of physics, rearranged to solve for acceleration as a function of the calculated force and calculated mass for each individual step:

$$a = F/m = F/\left(w/\mathrm{g}\right) \tag{6.1}$$

where w is the weight of slide mass, g is gravity, and F is the net force, which equals the driving force–resisting force

Velocity is estimated through integration of the calculated acceleration; subsequently, displacement is determined through integration of the velocity function, both using the trapezoidal rule. These three parameters (acceleration, velocity, and displacement) are functions of time and are plotted on the y-axis; see Figure 6.5.

The parameter time (t) is estimated through the goal seek function in Excel and plotted on the x-axis. To accomplish this, a time value is selected to integrate each function (in order: acceleration, velocity, displacement) yielding a displacement of known value for each individual step. Each step is converged before proceeding to the next. This procedure is performed through all (7) steps, to converge on a final velocity of zero and the known final displacement.

The input residual strength is systematically changed, and the procedure repeated for each step in the analysis. Too large a residual strength value results in a premature reduction in velocity to zero and too small of a displacement; too small a residual strength value produces the opposite. The value best satisfying the final boundary conditions of zero velocity and known displacement represents the estimated post-liquefaction residual strength mobilized within the displaced mass. All of these analyses considered two possible failure modes: full-flow failure and layered liquefaction failure. As shown in Figures 6.3 and 6.4, full-flow failure is where the entire mass of the tailings liquefied. Layered liquefaction failure is where the weakest layer (as identified in the subsurface investigations)—which also "daylights" along the face of the slope—liquefied and carried block of intact material along with it downslope.

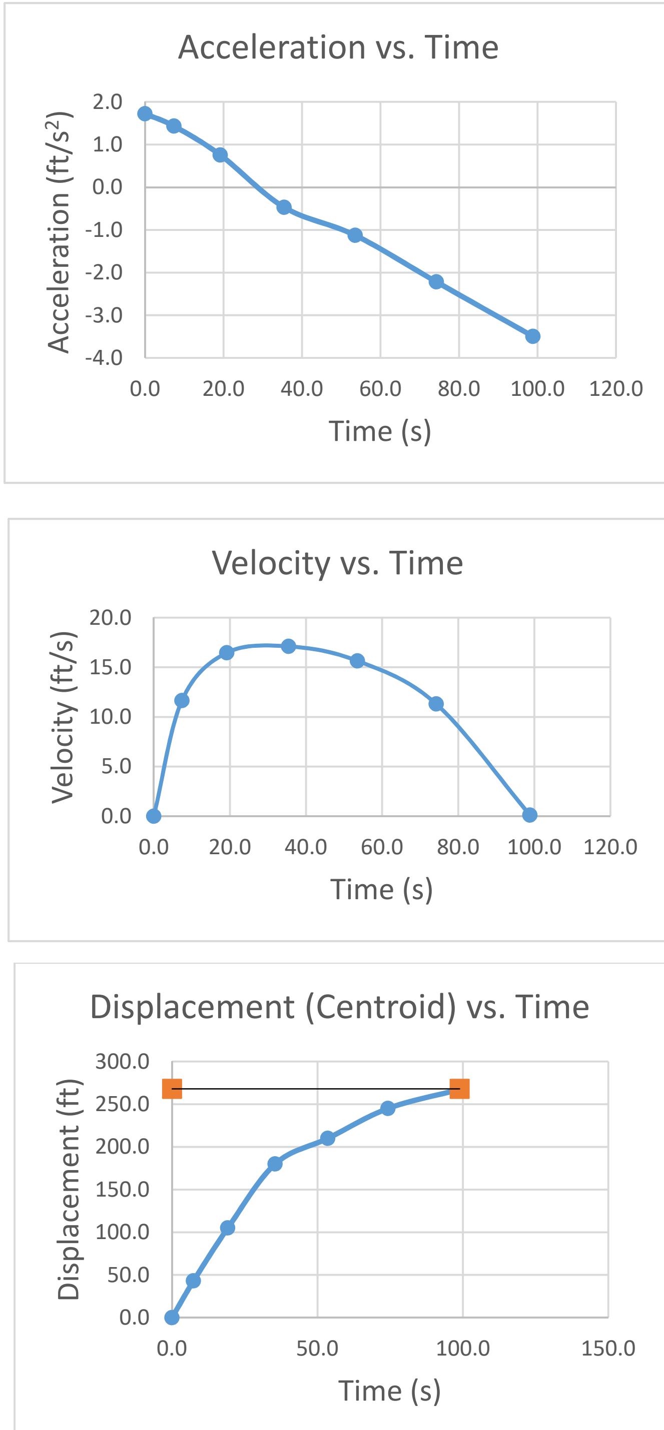


Figure 6.5 **Example trial of incremental momentum method (IMM) after Weber [2015].**

7 Comprehensive Results

The results in this report fall into three categories;

1. Resistance measurements representative of the pre-failure conditions of the material that liquefied and produced the flow failure;
2. Residual strength estimates of the liquefied material; and
3. Effective stress estimates of the pre-failure conditions at the depth of failure.

7.1 RESISTANCE MEASUREMENTS

Selection of a representative penetration resistance value of the Las Palmas dam for establishing post-liquefaction strength predictive correlations considered blow counts within what is interpreted as the saturated portion of the tailings material. Blow counts were corrected for factors related to overburden pressure and fines content; see Figure 7.1. Fines correction was per Equations (7.1) and (7.2) put forth by Cetin et al. [2004] to transform SPT $N_{1,60}$ values to $N_{1,60,CS}$ values. This fines correction accounted for two factors: (1) increased resistance to liquefaction as a function of fines content; and (2) adjustment of blow counts as a function of granular and fine grain material *in situ* resistance.

$$N_{1,60,CS} = N_{1,60} \times C_{fines} \tag{7.1}$$

$$C_{fines} = \left(1 + 0.004 \times FC\right) + 0.05 \times \left(FC / N_{1,60}\right) \tag{7.2}$$

In these equations, FC = percent fines content (by dry weight) expressed in percent (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.

The averaged blow counts from the SPT measurements of intact material representative of liquefiable flow failure tailings were estimated at $N_{1,60} \approx 2.5$ and $N_{1,60,CS} \approx 5$ (Figure 6.6), with an estimated coefficient of variation of 25% [Kulhawy and Mayne 1990].

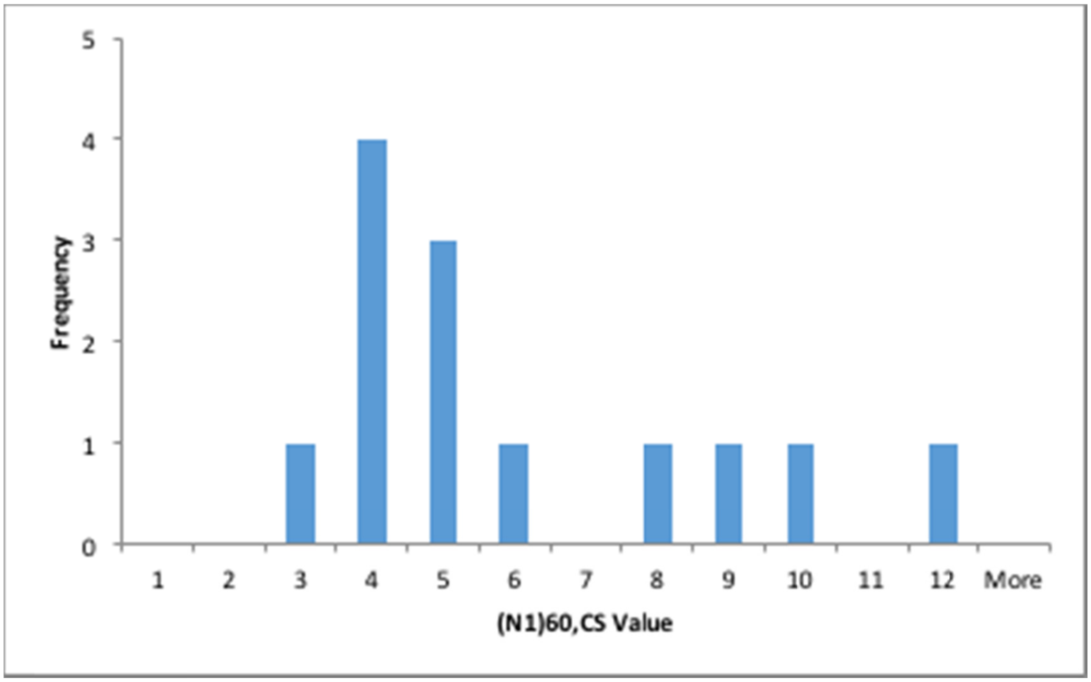


Figure 7.1 Histogram of blow counts in saturated tailings material, with fines correction (borings B-2, 3, and 4).

For the CPT, overburden corrections were performed using the equation below to arrive at qc_1 and also corrected with the subsequent equation to arrive at the dimensionless Q_{tn} [Robertson and Cabal 2015].

$$q_{c1} = q_c \cdot \left(P_a / \sigma_v^{'}\right)^n \; q_{c1} = q_c \cdot \left(P_a / \sigma_v'\right)^n \tag{7.3}$$

where q_{c1} is the overburden stress normalized tip resistance (MPa); q_c is the raw tip resistance; P_a is one atmosphere of pressure ≈ 100 kPa – 0.1 MPa; σ''_{w} is the vertical effective stress (kPA); and n is the normalization exponent ≈ 0.5.

Although the normalization exponent varies by soil type and stress conditions, a median value of 0.5 is typical for young normally consolidated sandy soils [Moss et al. 2006].

$$Q_{tn} \left[\left(q_t - \sigma_v\right)/P_a\right] \cdot \left(P_a/\sigma_v'\right)^n \tag{7.4}$$

where $q_t = q_c + u\left(1 - a\right)$ is the pore pressure corrected tip resistance; u is the pore pressure measurement at the u_2 position; a is the cone factor ≈ 0.8 (typical for Gregg drilling cones; and σ_v is the vertical total stress (kPa).

The VS values have been overburden corrected (VS_1) for Holocene sands using the equation below, which uses a typical median normalization exponent of n = 0.25 [Andrus and Stokoe 2000].

$$VS_1 = VS \cdot \left(P_a/\sigma_v'\right)^n \tag{7.5}$$

The processed cone measurements are shown in Figures 7.2–7.4. Water table depths at the time of the measurements were estimated based on the pore pressure measurements and are shown in the figures with a triangle. The depth ranges thought to represent potentially liquefiable material with continuous stretches of low normalized penetration resistance are boxed. The histograms of the boxed regions are shown below each figure. CPT1 was located closest to the scarp where the material failed, CPT2 was positioned to intercept wall material, and CPT3 was located near the thickest portion of ponded tailings material.

Table 7.1 shows the stress normalized shear-wave velocity for the "weak" layers of the profiles measured. G2 was omitted because it appears to be non-liquefiable based on the CPT and *VS* profiles. Based on analysis of the normalized CPT and *VS* measurements, it appears the CPT3 and G3 are the most representative of intact material that resulted in flow failure in the unrestrained portion of the tailings. The interpreted measurements for this case history are then; q_{c1} ~1.3 MPa (Q_{tn}~11.7) and VS_1~172 m/sec, with an estimated coefficient of variation of 10% for the CPT [Kulhawy and Mayne 1990] and the same for *VS* [Moss 2008].

Table 7.1 Shear-wave velocity of "weak" layers for each profile.

Profile	Depth range (m)	Average VS_1 (m/sec)
G1	0 to 5	211
G2	na	na
G3	0 to 8	172
G5	0 to 3	222
G6	3 to 9	175

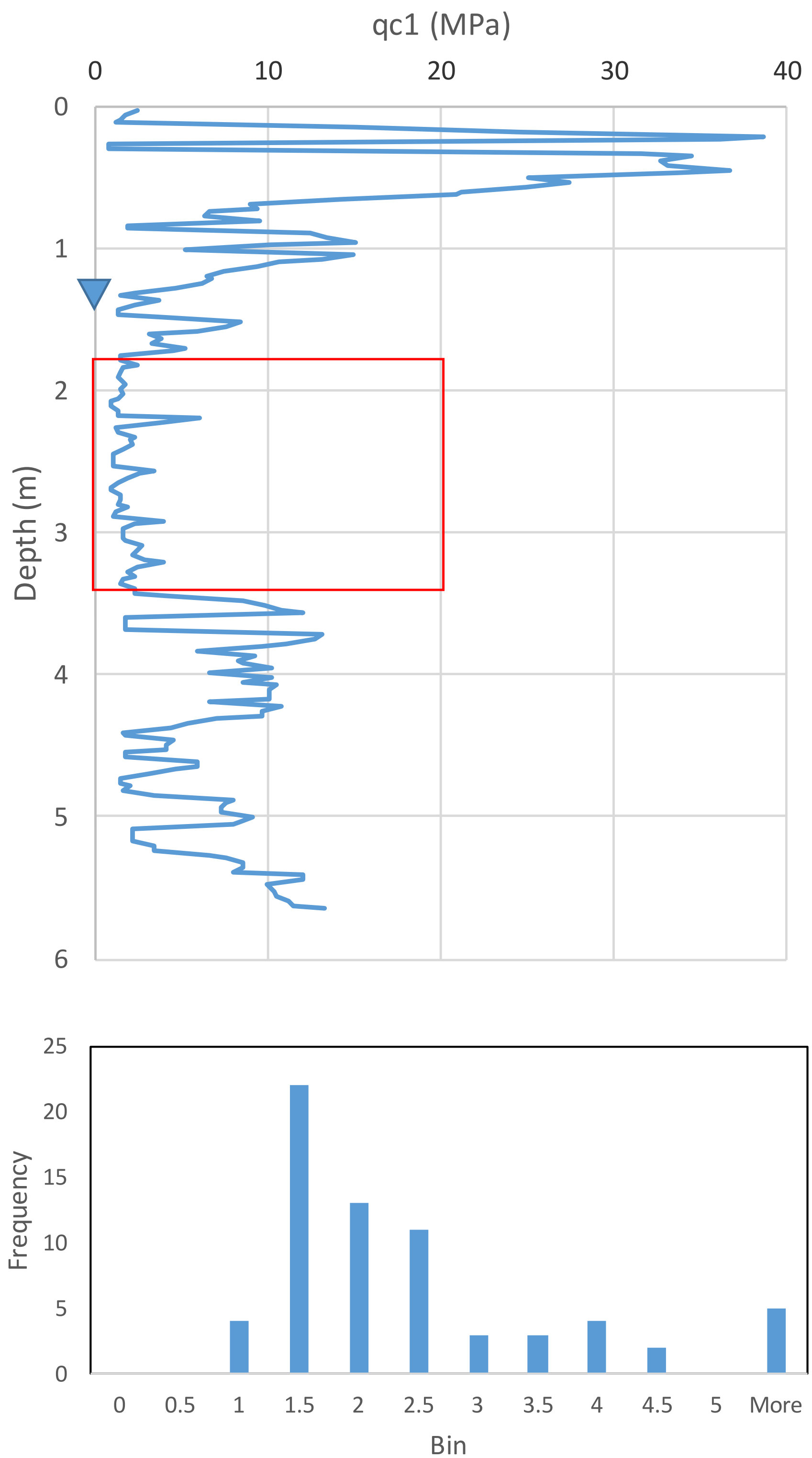
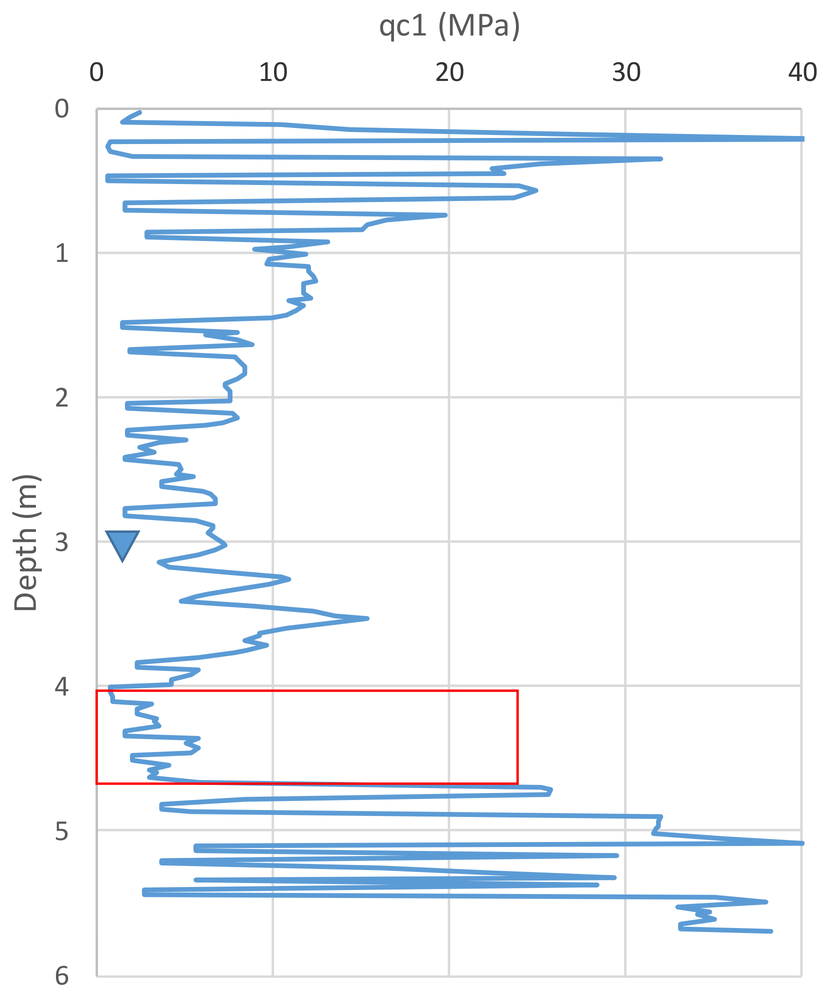


Figure 7.2 **CPT1 overburden corrected tip resistance with histogram of boxed region thought to best represent the tailings material susceptible to liquefaction and flow failure. The mean and median are approximately 1.83 MPa with a CoV of 0.40.**



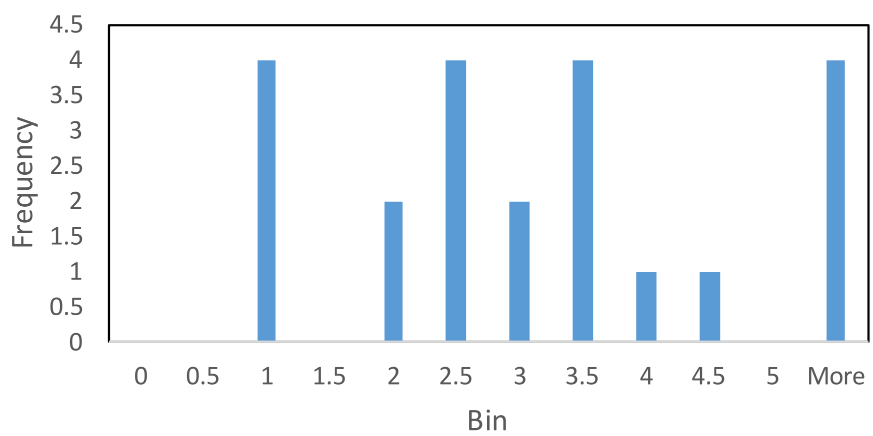


Figure 7.3 CPT2 overburden corrected tip resistance. This sounding is thought to represent material not highly susceptible to liquefaction because of wall material and interlayering. The boxed region mean is 2.94 MPa, median 1.86 MPa, with a CoV of 0.52.

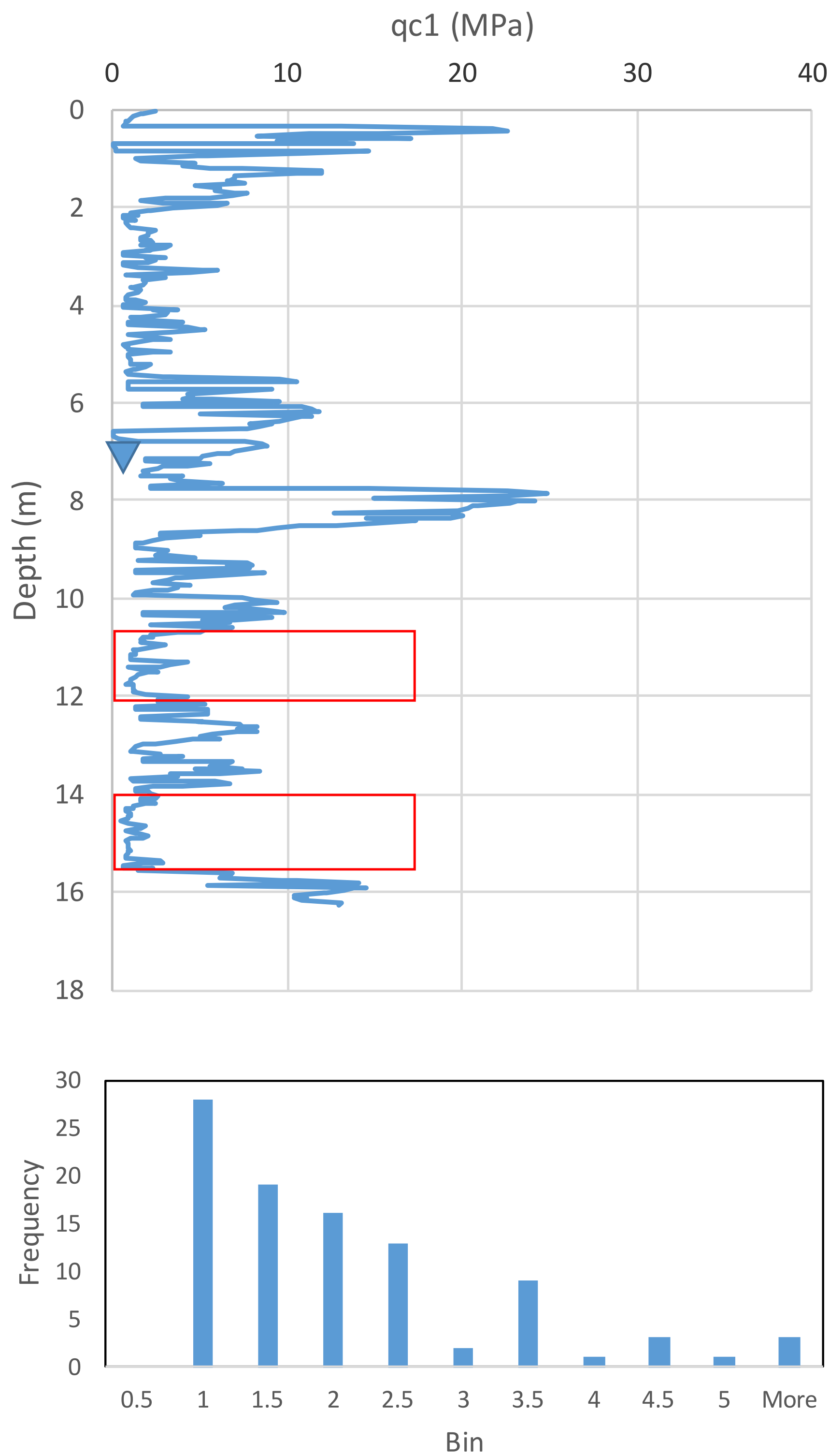


Figure 7.4 **CPT3 overburden corrected tip resistance. This sounding is in the center of a region that experienced liquefaction but did not exhibit failure in a flow failure. The boxed regions have a mean of 1.3 MPa, median 1.4 MPa, with a CoV of 0.06.**

7.2 RESIDUAL STRENGTH

A post-liquefaction residual strength ≈ 7.8 kPa (163 psf) and ≈ 8.7 kPa (181 psf) represent bounds for a layered failure analysis, and a residual strength ≈ 8.4 kPa (175 psf) represents a complete flow-failure analysis. The average of these three residual strengths ≈ 8.3 kPa (173 psf), which represents the estimated mobilized post-liquefaction residual strength for this case study, with a nominal coefficient of variation of 5.5% estimated from the sensitivity analysis.

To provide confidence in this value, a sensitivity analysis was conducted; the results shown in Figure 7.5. The sensitivity analysis included alternate failure modes, differing ground water conditions, and variable unit weight of the uncompacted tailings embankment and ponded material.

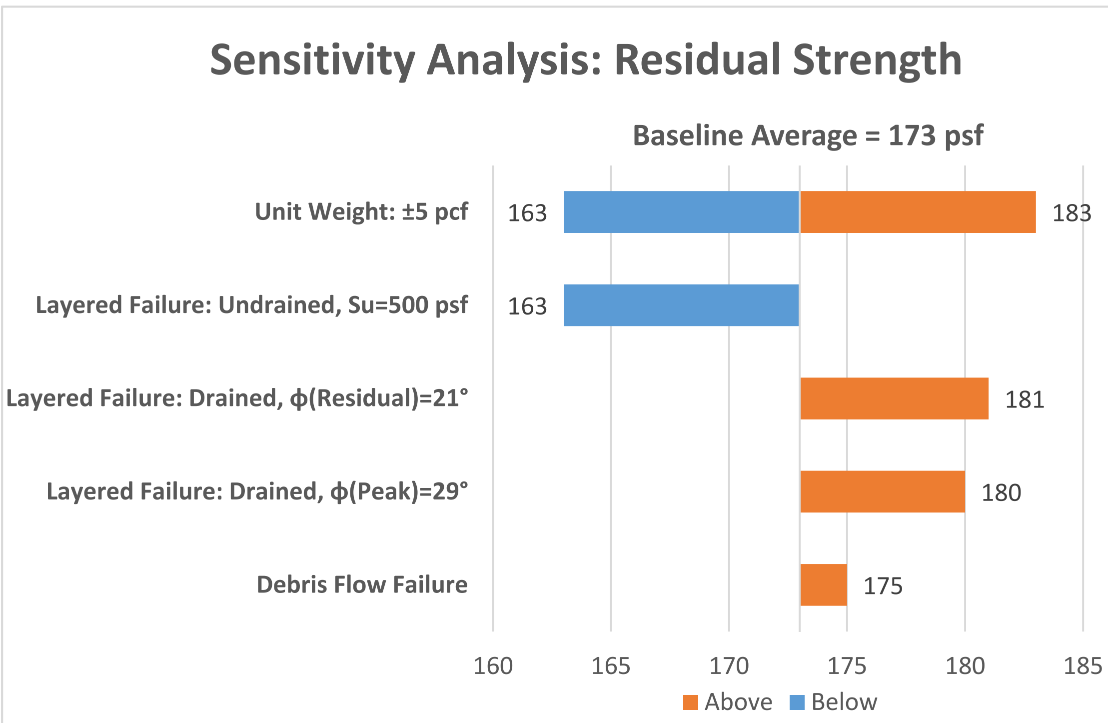


Figure 7.5 Tornado plot sensitivity analysis of post-liquefaction residual strength (from Gebhart [2016]).

7.3 EFFECTIVE STRESS

For this study, the location selected to represent initial vertical effective stress (or soil overburden) was the assumed failure interface between tailings and underlying competent native material. Values and geometry used for calculation are per Figure 7.6. An initial vertical effective stress of approximately 2.0 atmospheres (202.5 kPa or 4,300 psf) was the best estimate for this case history, with an estimated coefficient of variation of 5% [Kulhawy and Mayne 1990].

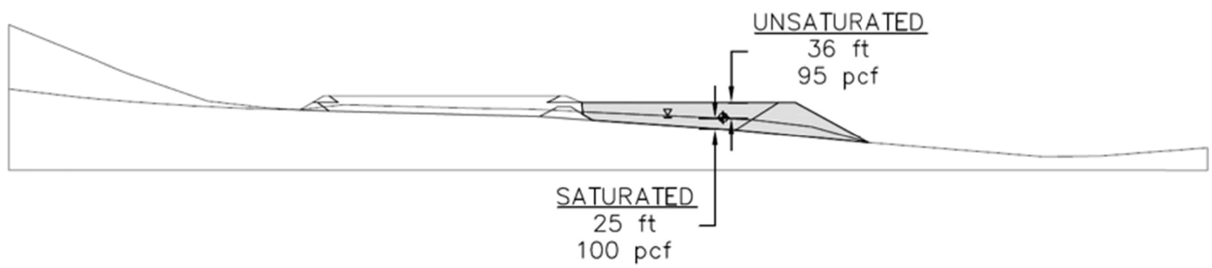


Figure 7.6 **Geometry used to estimate the effective stress on the failure plane at the time of failure (from Gebhart [2016]).**

7.4 COMPARISON WITH EXISTING DATABASE

The results are plotted against prior case histories in the format of residual strength predictive plots: Seed and Harder [1990] (Figure 7.7); Olson and Stark [2002] (Figures 7.8 and 7.9); Kramer and Wang [2015] (Figure 7.10); and Weber et al. [2015] (Figure 7.11). The data from this case history is shown as an ellipse capturing the quantified uncertainty in penetration resistance and residual strength.

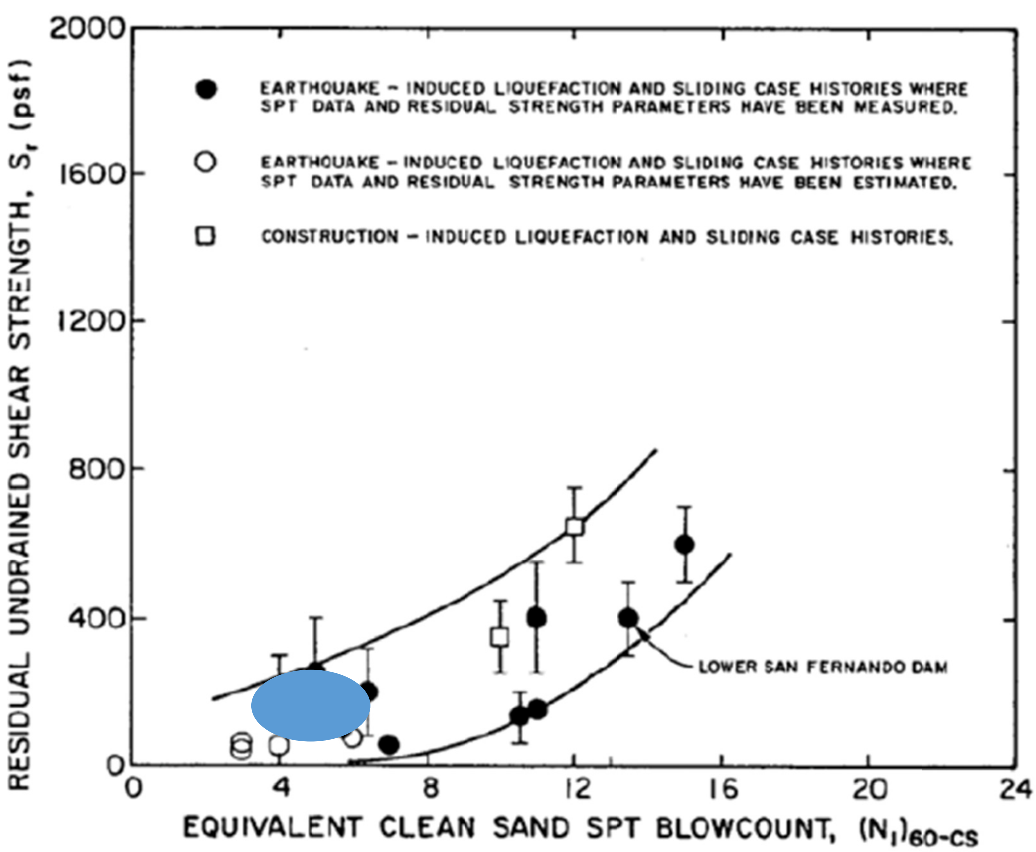


Figure 7.7 **Las Palmas data ellipse with respect to Seed and Harder [1990].**

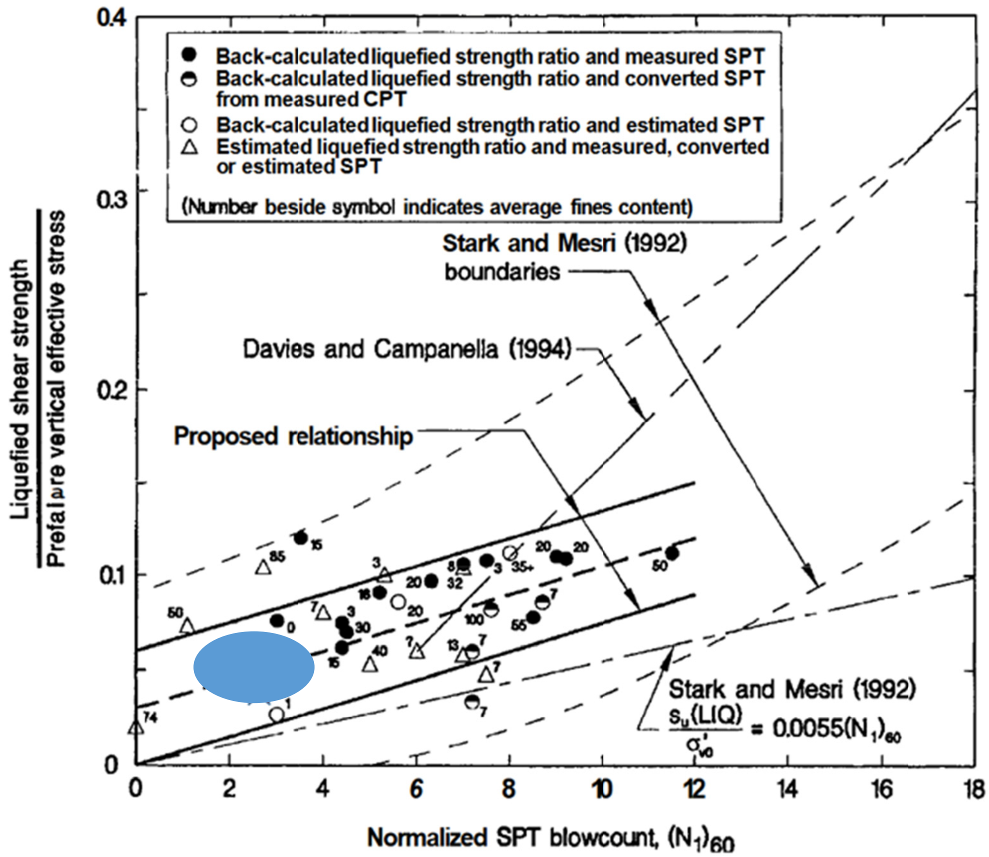


Figure 7.8 Las Palmas data point (SPT) with respect to Olson and Stark [2002].

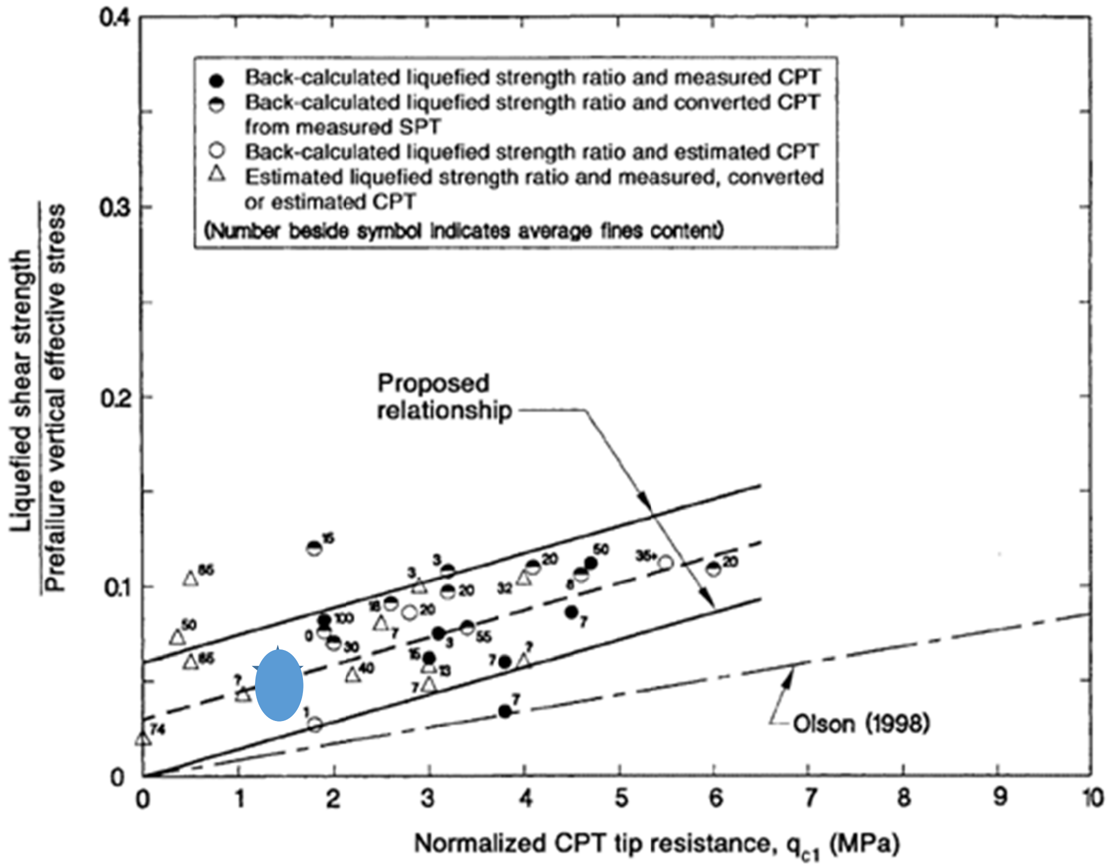


Figure 7.9 Las Palmas data point (CPT) with respect to Olson and Stark [2002].

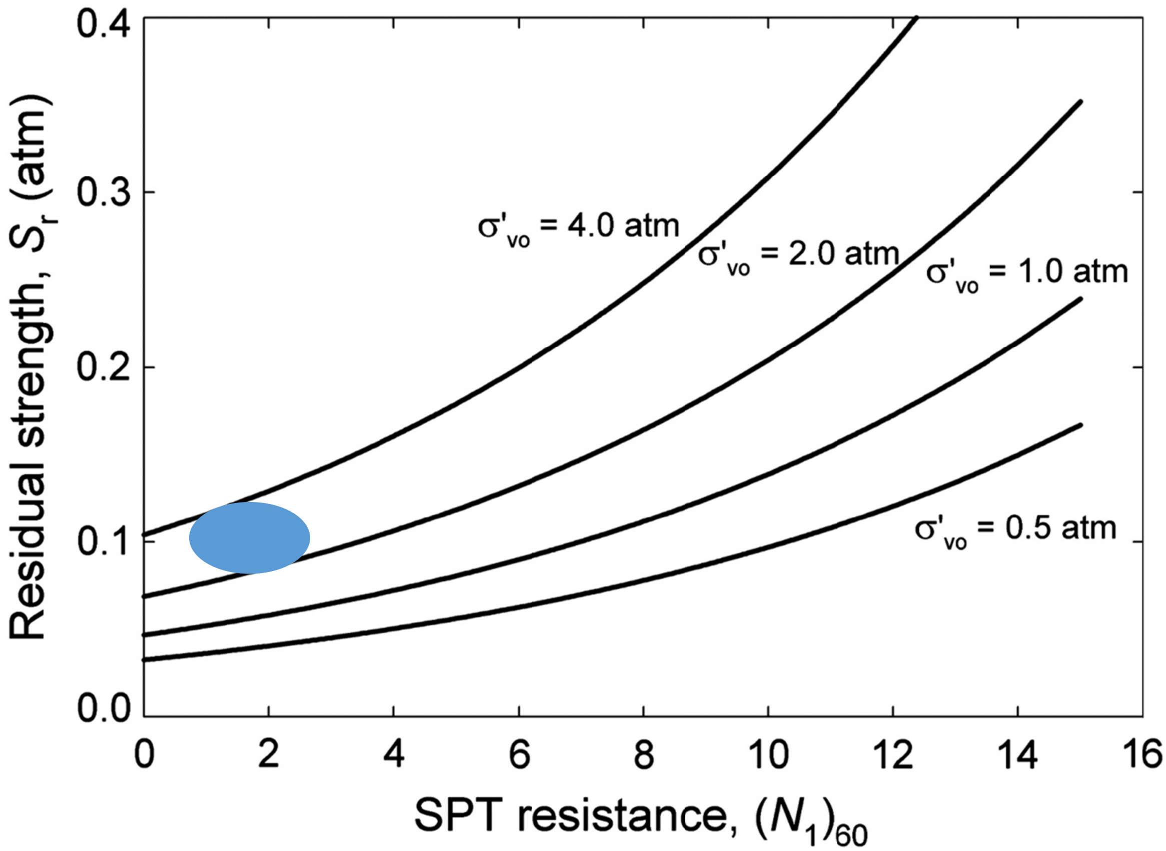


Figure 7.10 Las Palmas data point with respect to Kramer and Wang [2015].

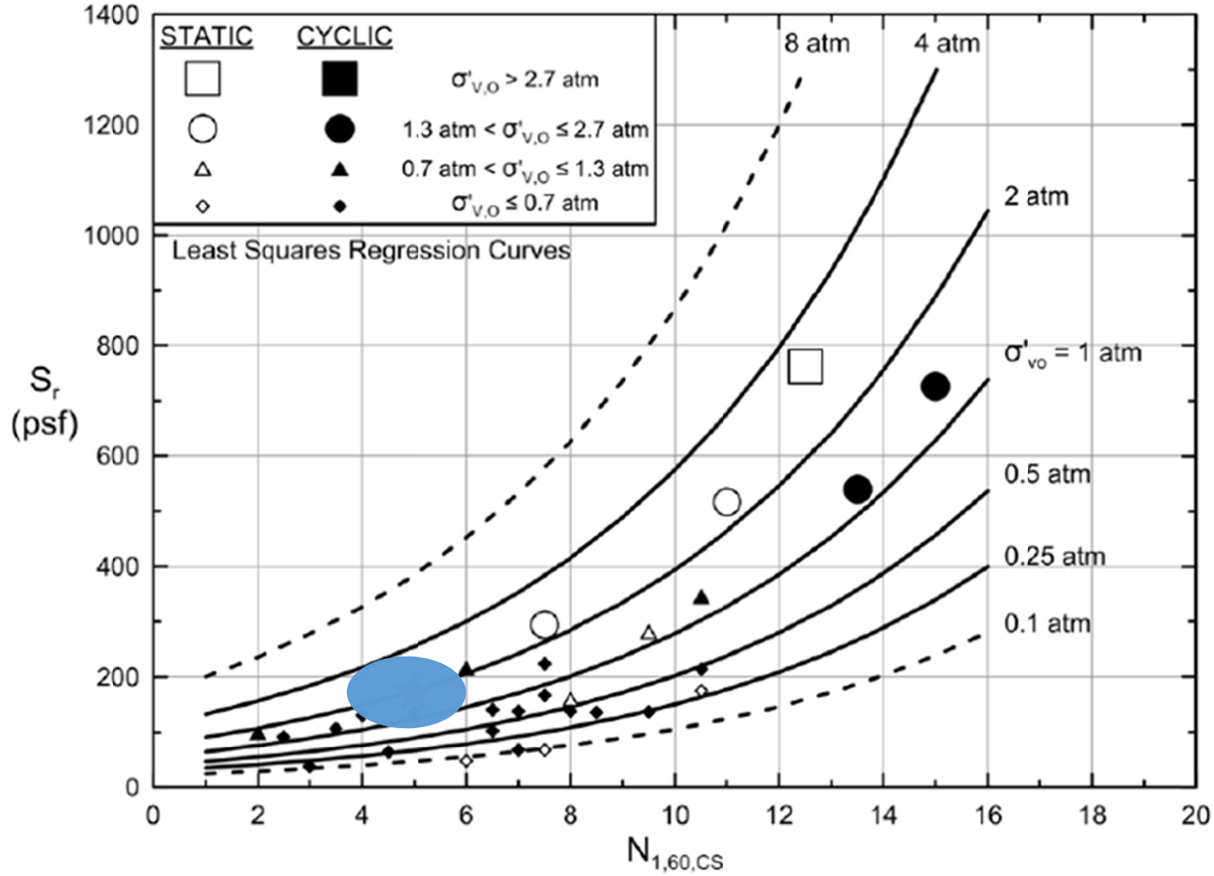


Figure 7.11 Las Palmas data point with respect to Weber et al. [2015].

8 Summary and Conclusions

The goal of this study was to provide a well-documented case history of a liquefaction flow failure. The 2010 Maule, Chile, event triggered the flow failure in the Las Palmas tailings dam, resulting in a run out of upwards of 350 m. The documentation herein includes; reconnaissance information by GEER [2010], drilling and standard penetration measurements (SPT) by DICTUC [2012], and back analysis of the residual strength by Gebhart [2016], as well as the results of cone penetration tests (CPT) and shear-wave velocity (*VS*) measurements presented here for the first time.

Based on the information evaluated, analyzed, and measured, the Las Palmas flow failure can be summarized with the following mean values and estimated coefficients of variation.

Table 8.1 Summary mean and coefficient of variation.

Resistance	SPT	$N_{1,60} \approx 2.5$ and $N_{1,60,CS} \approx 5$	COV~25%
	CPT	$q_{c1} \approx 1.3$ MPa and $Q_{tn} \approx 11.7$	10%
	VS	$VS_1 \approx 172$ m/sec	10%
Residual strength	Sur ≈ 8.3 kPa (173 psf)		5.5%
Effective stress	σ'_v ≈ 2.0 atmospheres (202.5 kPa, 4300 psf)		5.0%

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Appendix Geophysical Data Processing

This appendix shows the passive surface wave geophysical data analysis used to arrive at the shear-wave velocity profiles. The data processing was carried out using the Surface Plus module in the Geogiga software suite. Measured and estimated dispersion curves are shown. All data was recorded using;

- circular arrays
- 12 geophones (4.5Hz)
- 2 m/sec sampling rate
- 32 sec recording length x 10 recordings all concatenated

CPT1 (G1) s35.184242 w71.759540 (coincident with SPT4)
1000-1009 is a 10-m array
1100-1109 is a 20-m array

CPT2 (G2) s35184297 w71.760284
2000-2009 is a 20-m array
2100-2109 is a 10-m array

CPT3 (G3) s35.184350 w71.761196 (coincident with SPT2)
3000-3009 is a 20-m array
3100-3109 is a 10-m array

G5 (debris flow material) s35.185729 w71.758658
5000-5009 is a 10-m array
5100-5109 is a 20-m array
5200-5209 is a 5-m array

G6 (translated mass) s35.186669 w71.758161
6000-6009 is a 10-m array
6100-6104 is a 20-m array
6200-6209 is a 5-m array

CPT1 (G1) s35.184242 w71.759540 (coincident with SPT4)

BORING LOG

BORING NO. **B-4**

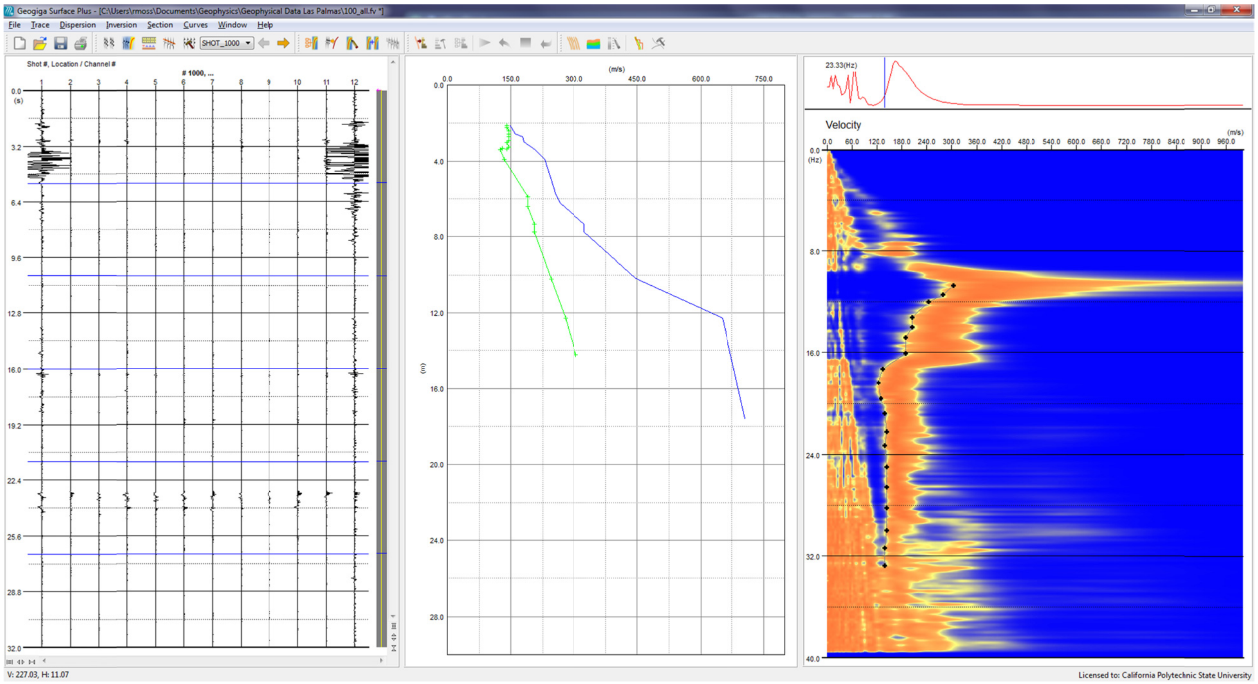
JOB NO.

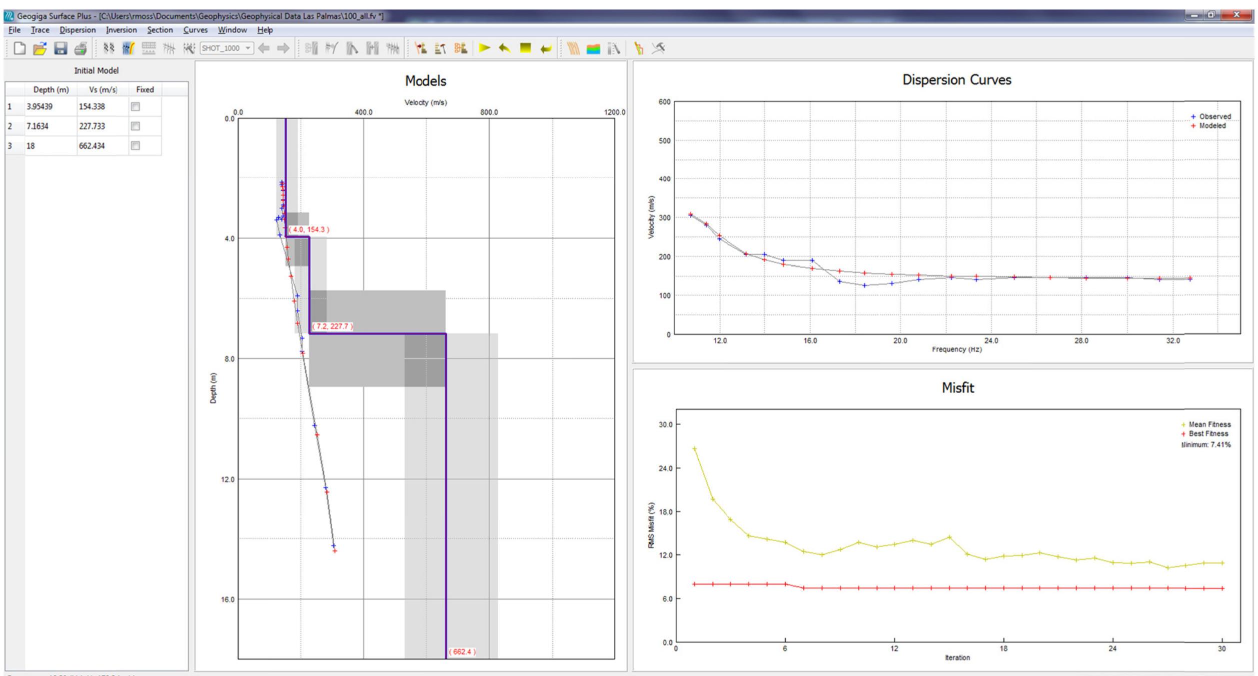
PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Las Palmas Tailings Dam**	DRILL RIG:
DRILLING LOCATION: **Tailings**	HOLE DIAMETER:
DATE DRILLED:	SAMPLING METHOD:
LOGGED BY:	HOLE ELEVATION:

Depth of Groundwater: **17 Feet** Boring Terminated At: **28 Feet**

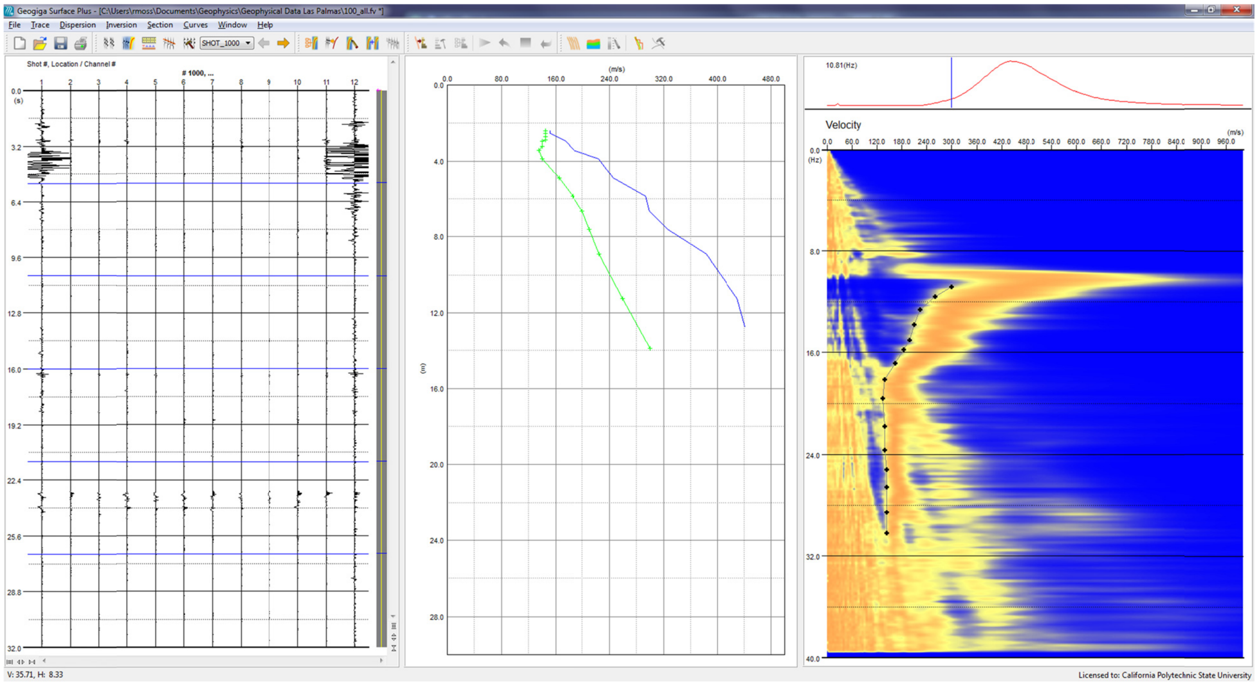
DEPTH	SOIL DESCRIPTION	USCS	LITHOLOGY	SAMPLE	BLOWS/ 12 IN	$(N_1)_{60}$	FRICTION ANGLE, (degrees)	COHESION, C (psf)	WATER CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	EXPANSION INDEX (EI)	FINES CONTENT (%)	PLASTICITY INDEX (PI)
0	SANDY SILT	ML							3-19%				N/A
-2					6	9						88	
-5					4	6						87	
-8					3	4						97	
-10	SILTY SAND	SM							8%				N/A
-12					12	12						32	
-14	SANDY SILT	ML							18%				N/A
-15					7	7						59	
-18	BEDROCK				Refusal								
-28													

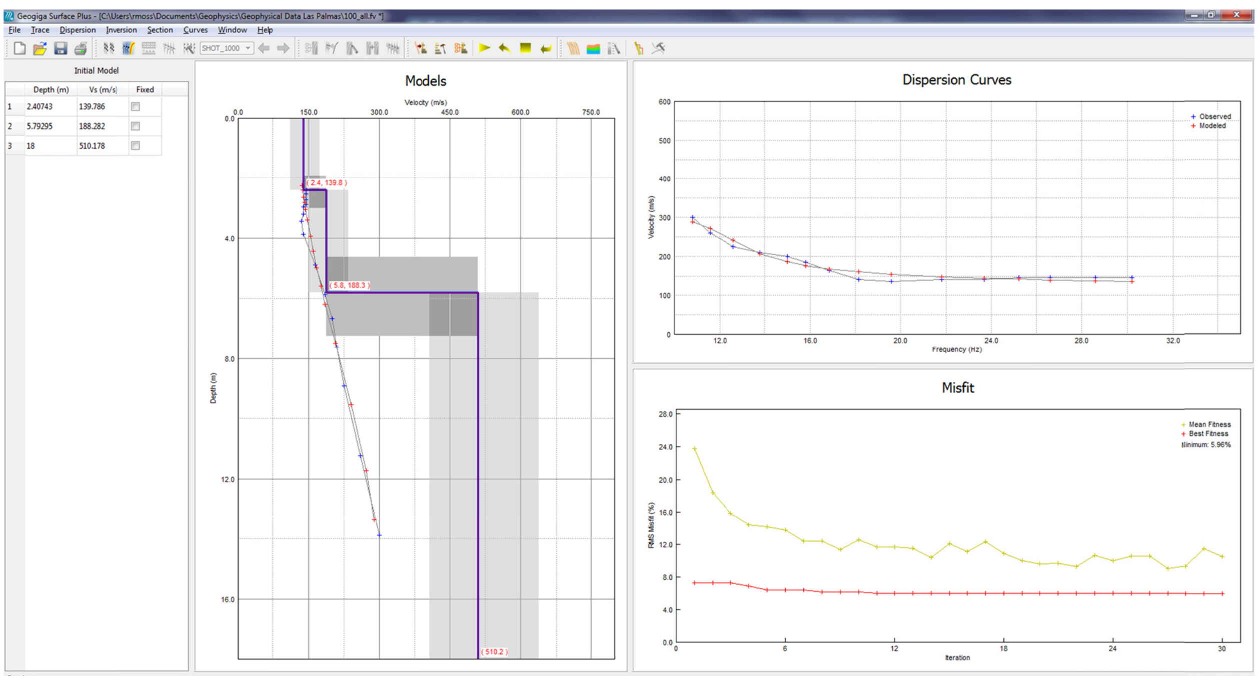
10-m circle only

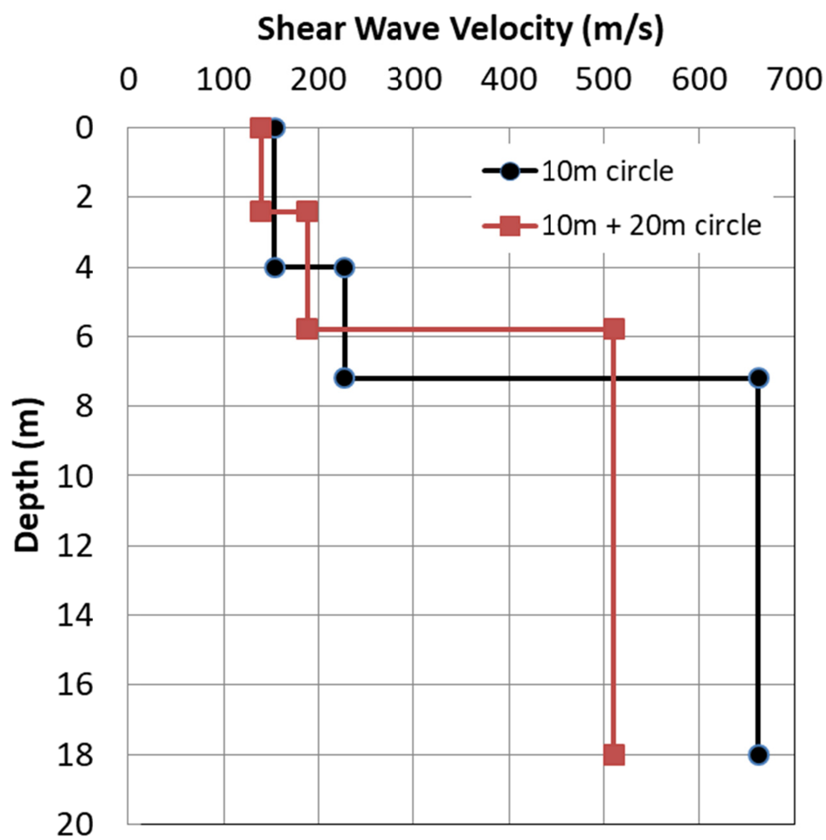




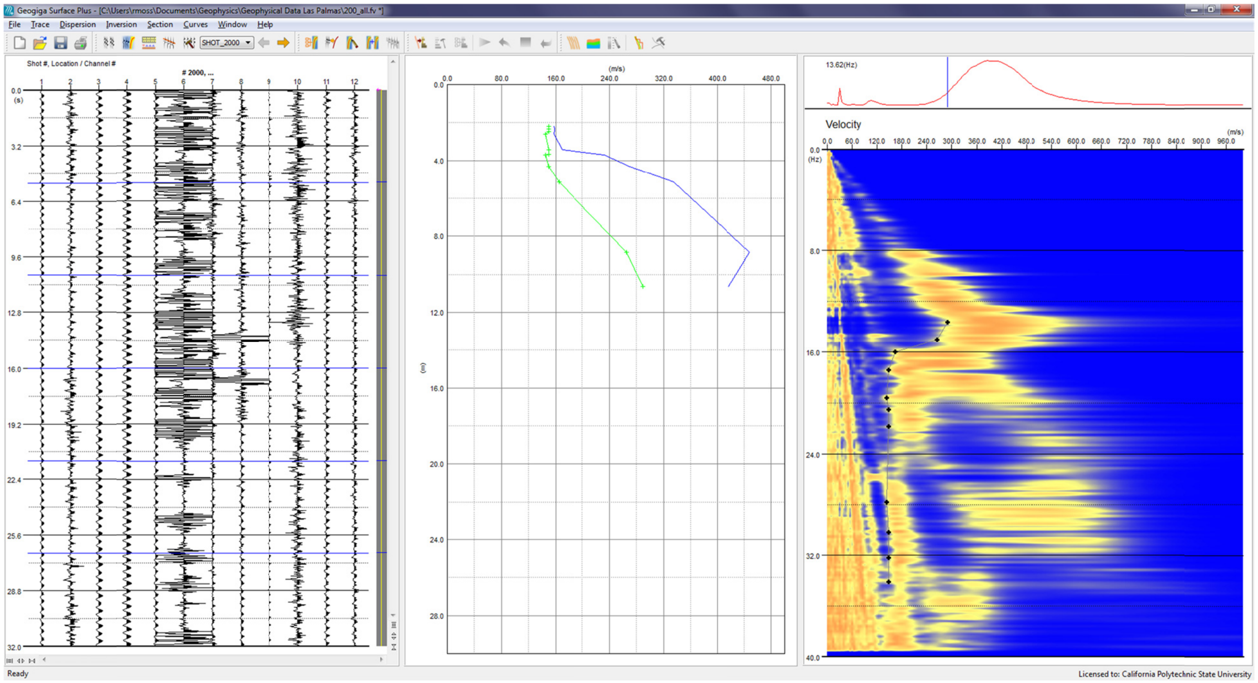
10-m + 20-m circle combined

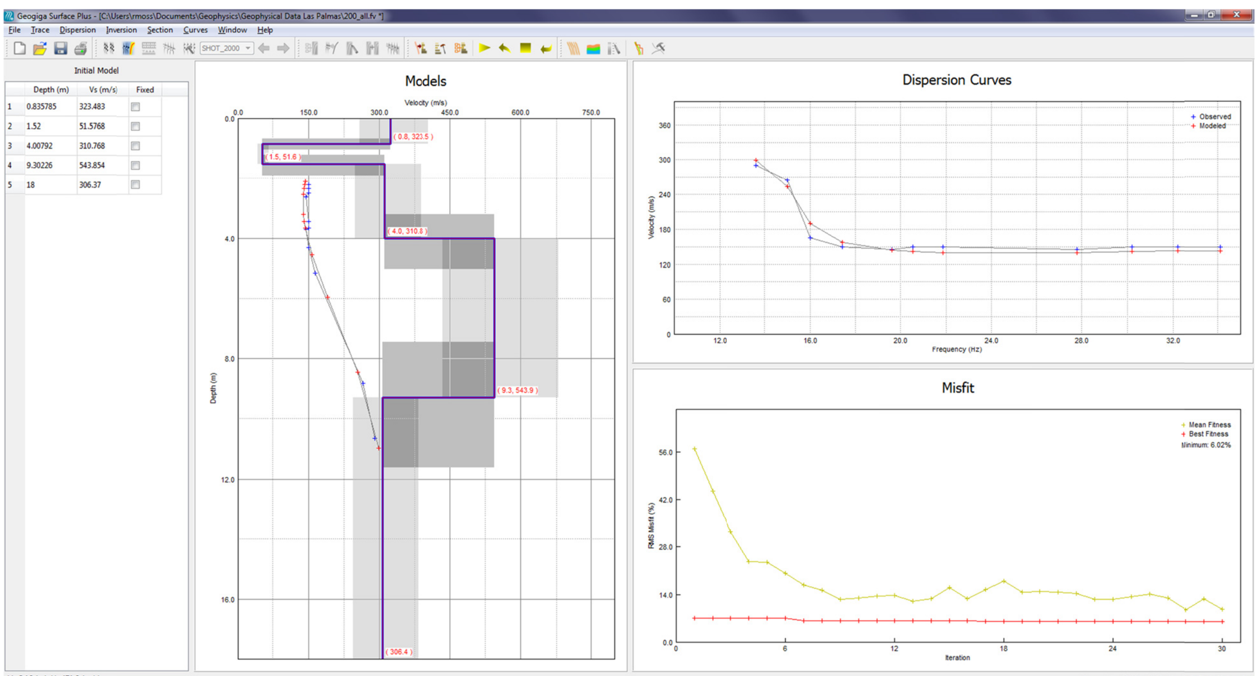




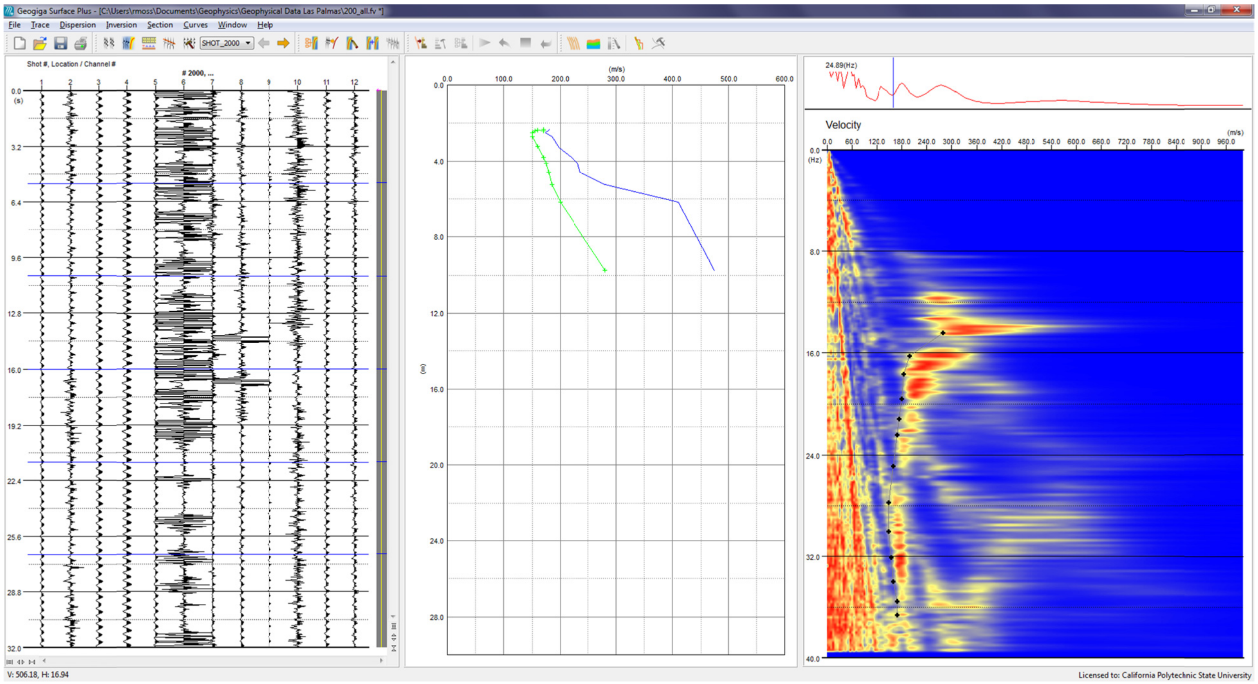

Shear Wave Velocity (m/s)
0
100
200
300
400
500
600
700
0
2
4
6
8
10
12
14
16
18
20
Depth (m)
10m circle
10m + 20m circle

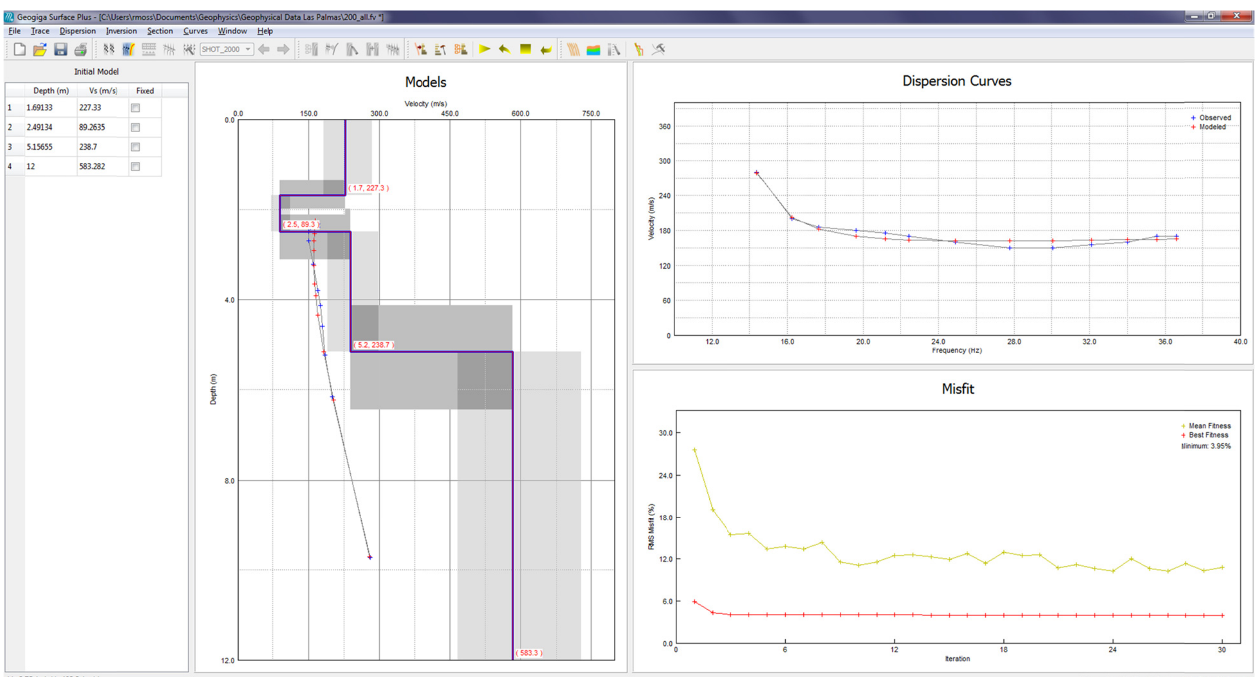
CPT2 (G2) s35184297 w71.760284
10-m circle

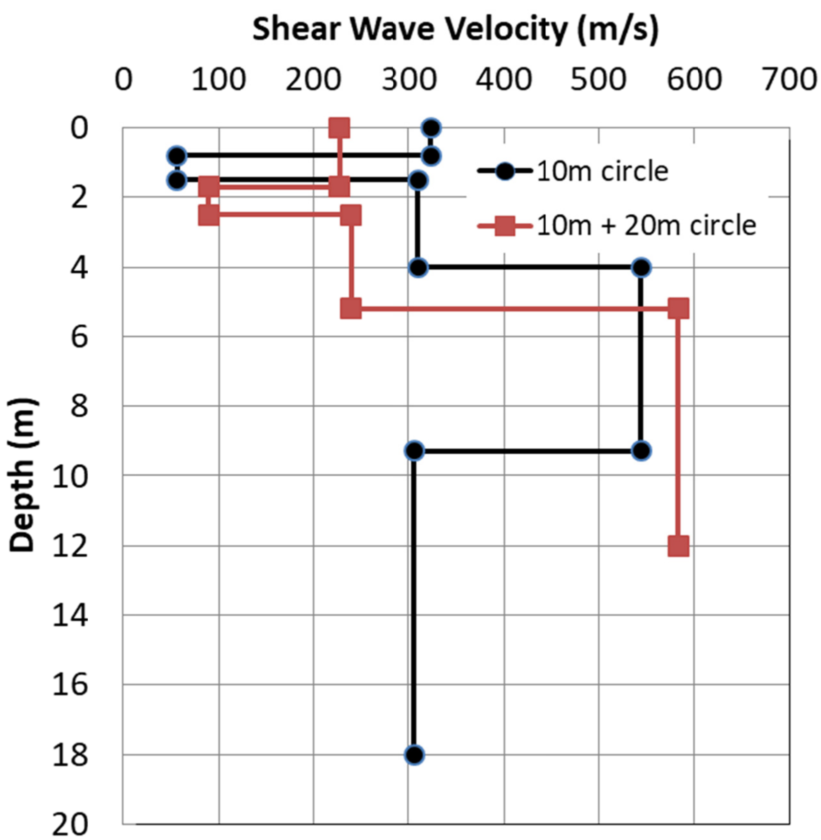




10-m + 20-m circle






Shear Wave Velocity (m/s)
0
100
200
300
400
500
600
700
0
2
4
6
8
10
12
14
16
18
20
Depth (m)
10m circle
10m + 20m circle

CPT3 (G3) s35.184350 w71.761196 (coincident with SPT2)

BORING LOG

BORING NO. **B-2**

JOB NO.

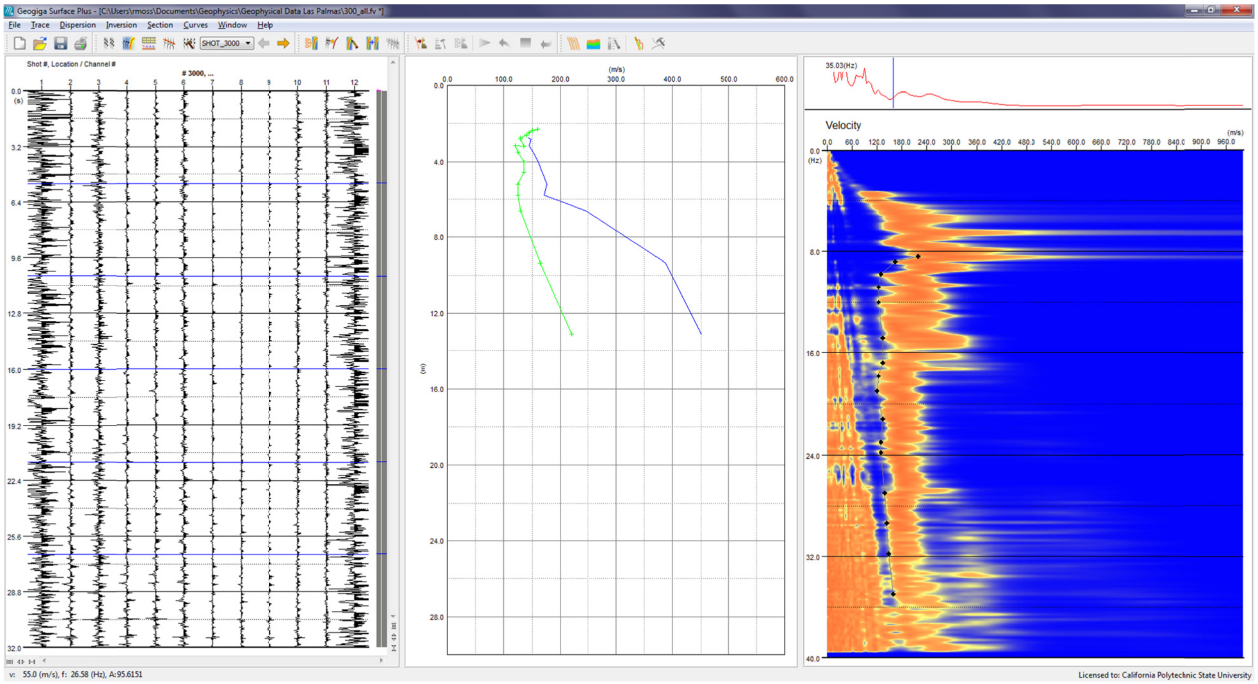
PROJECT INFORMATION	DRILLING INFORMATION
PROJECT: **Las Palmas Tailings Dam**	DRILL RIG:
DRILLING LOCATION: **Tailings**	HOLE DIAMETER:
DATE DRILLED:	SAMPLING METHOD:
LOGGED BY:	HOLE ELEVATION:

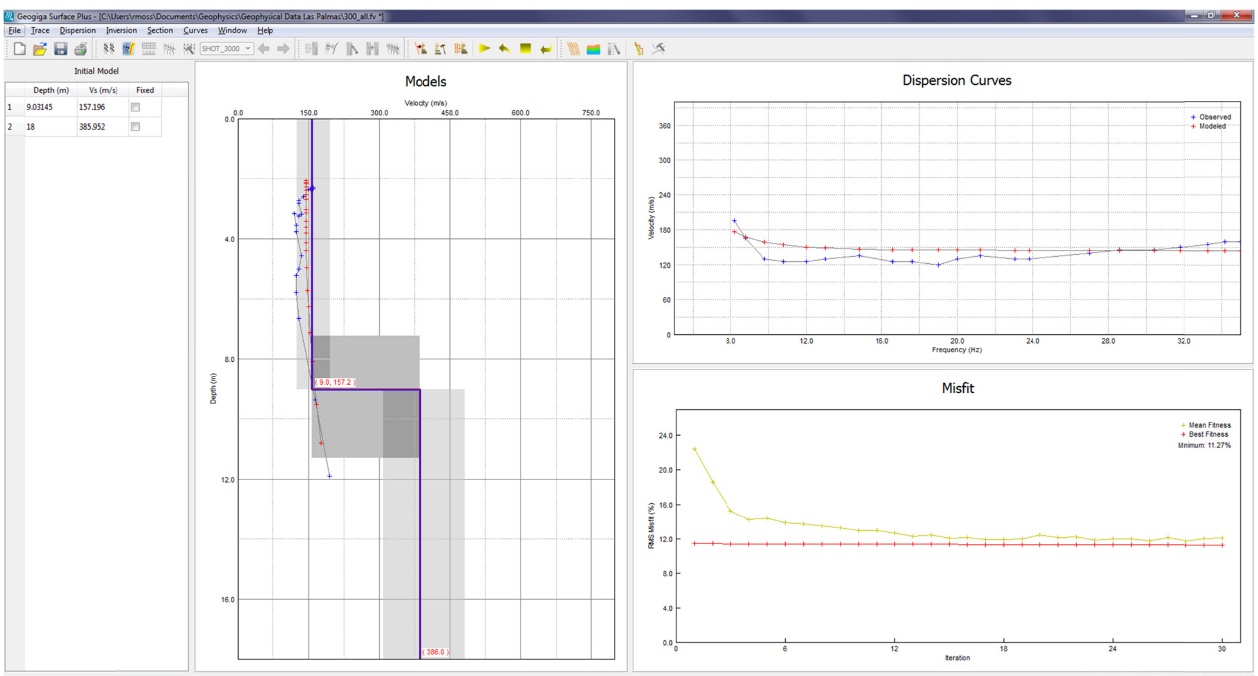
Depth of Groundwater: **40 Feet** Boring Terminated At: **53 Feet**

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)
	SANDY SILT	ML							12-32%				NP-4
					8	12						93	
					1	2						93	
					4	5						93	
					4	4						96	
					5	5						93	
					6	5						90	
					5	5						91	
					4	3						95	
					3	2						92	
					1	1						99	
					2	2						94	
					4	3						88	
					3	2						82	
					2	1						98	
					4	3						99	
					2	1						99	

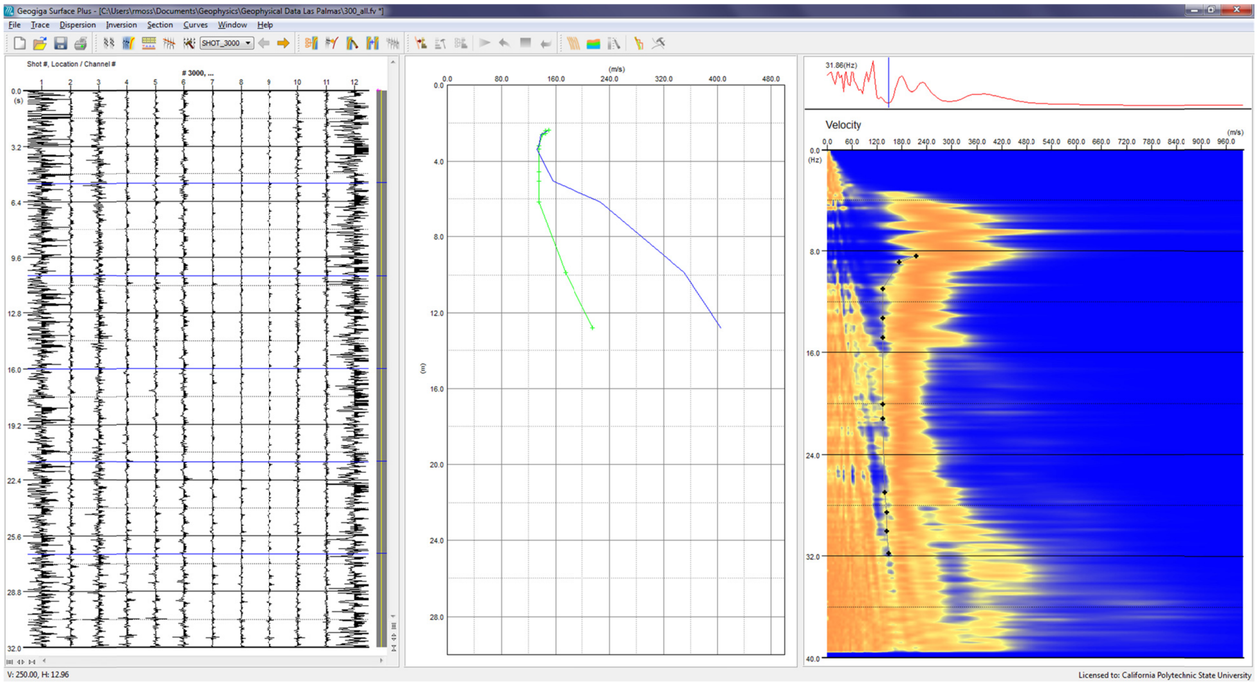
Depth scale: 0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11 -12 -13 -14 -15 -16 -17 -18 -19 -20 -21 -22 -23 -24 -25 -26 -27 -28 -29 -30 -31 -32 -33 -34 -35 -36 -37 -38 -39 -40 -41 -42 -43 -44 -45 -46 -47 -48 -49 -50 -51 -52 -53 -54 -55

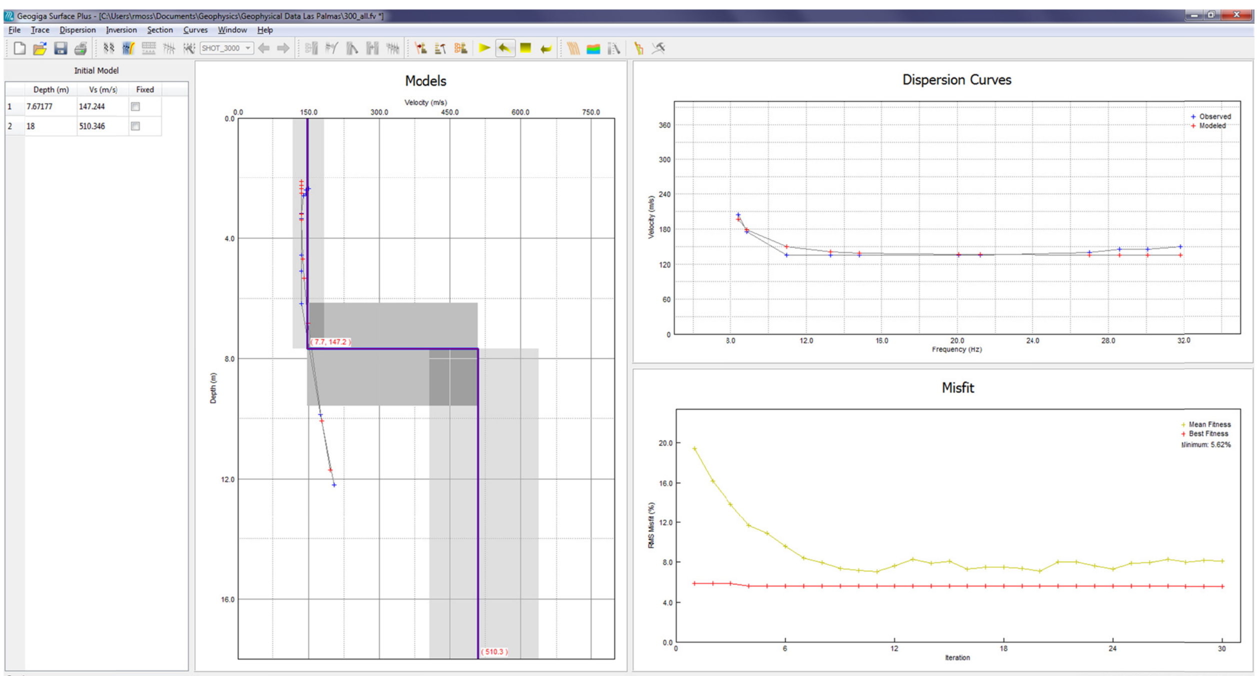
20-m circle

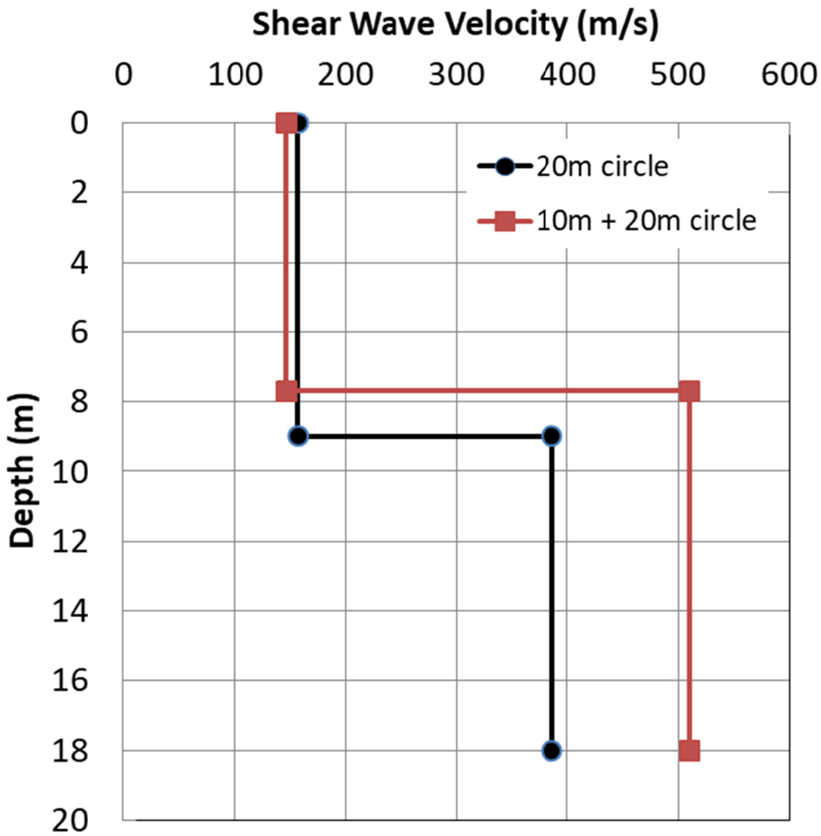

Geogiga Surface Plus - [C:\Users\rmoss\Documents\Geophysics\Geophysical Data Las Palmas\300_all.fv *]
File Trace Dispersion Inversion Section Curves Window Help
SHOT_3000
Shot #, Location / Channel #
# 3000, ...
35.03(Hz)
Velocity
(m/s)
(Hz)
v: 55.0 (m/s), f: 26.58 (Hz), A:95.6151
Licensed to: California Polytechnic State University


Geogiga Surface Plus - [C:\Users\rmoss\Documents\Geophysics\Geophysical Data Las Palmas\300_all.fv *]
File Trace Dispersion Inversion Section Curves Window Help
Initial Model
Depth (m) Vs (m/s) Fixed
1 9.03145 157.196
2 18 385.952
Models
Velocity (m/s)
Depth (m)
(9.0, 157.2)
(386.0)
Dispersion Curves
Observed
Modeled
Velocity (m/s)
Frequency (Hz)
Misfit
Mean Fitness
Best Fitness
Minimum: 11.27%
RMS Misfit (%)
Iteration
Licensed to: California Polytechnic State University

10-m + 20-m circle

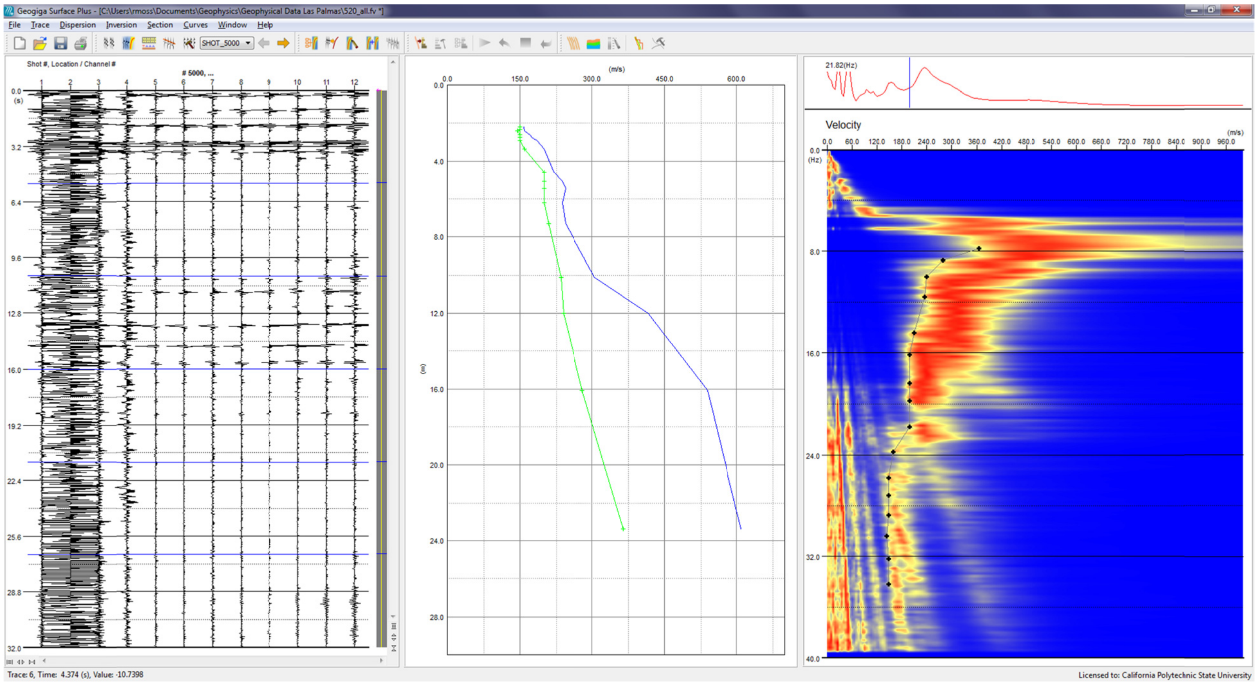


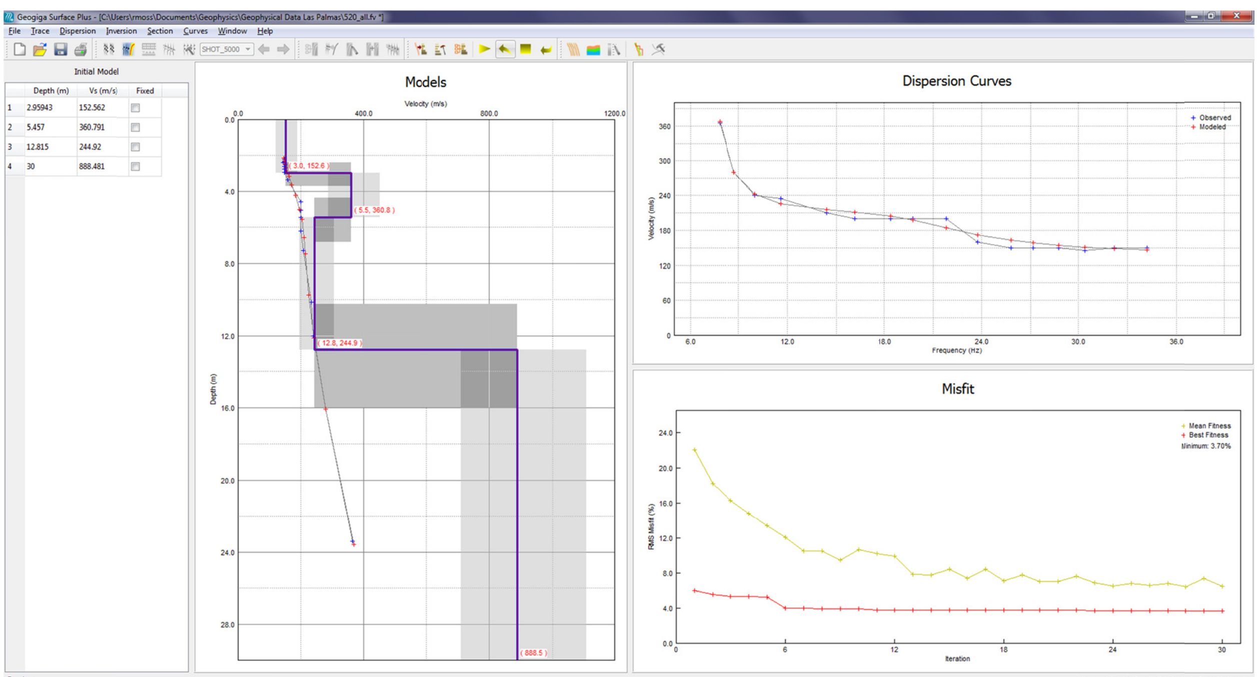


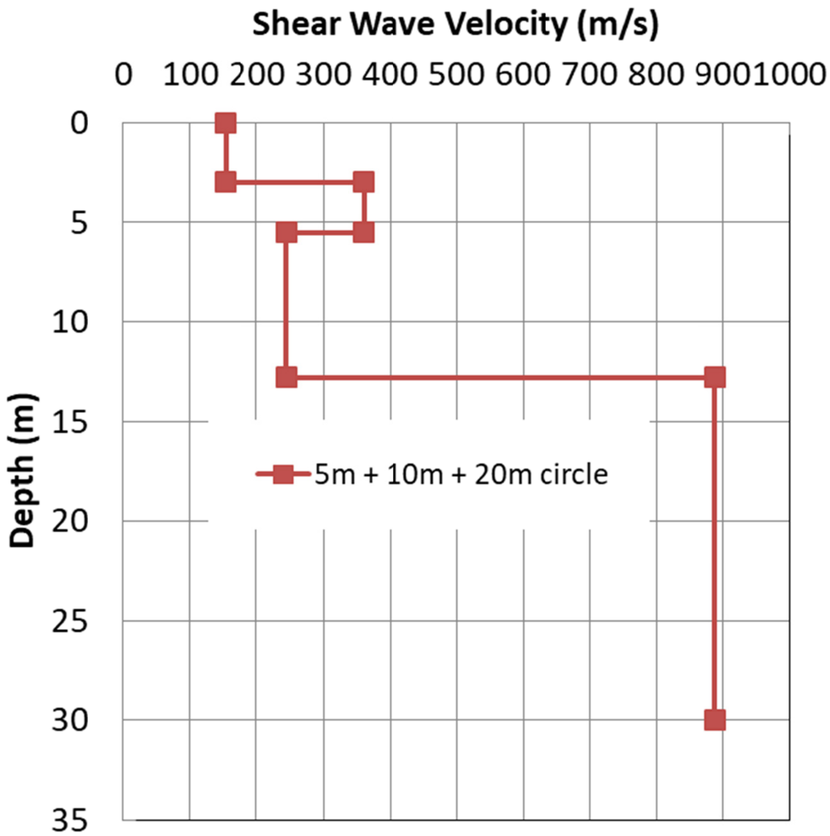


Shear Wave Velocity (m/s)
0
100
200
300
400
500
600
0
2
4
6
8
10
12
14
16
18
20
Depth (m)
20m circle
10m + 20m circle

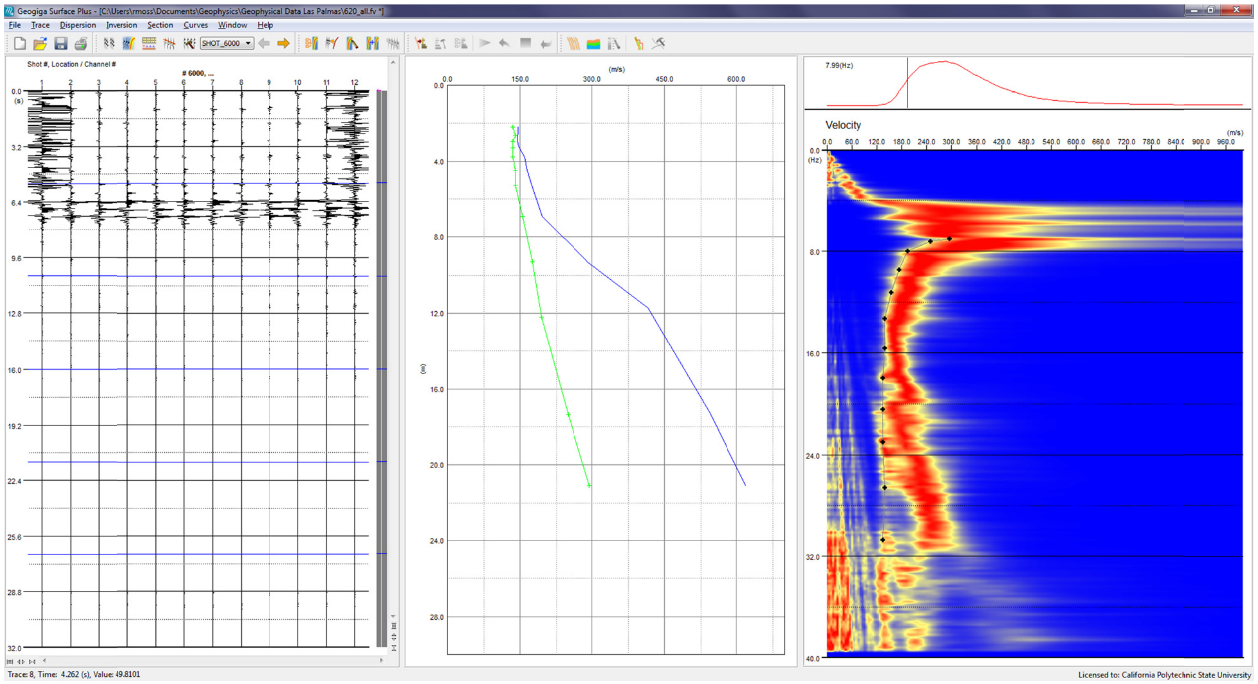
G5 (flow failure material) s35.185729 w71.758658
5000-5009 is a 10-m array
5100-5109 is a 20-m array
5200-5209 is a 5-m array

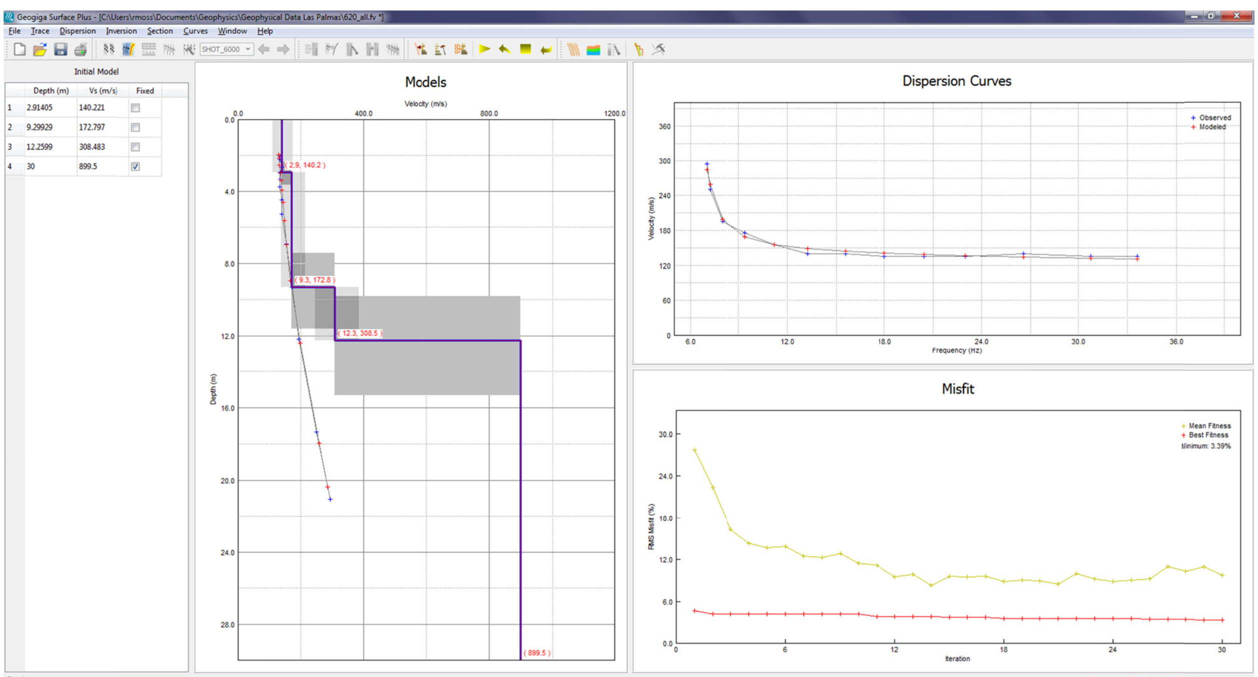


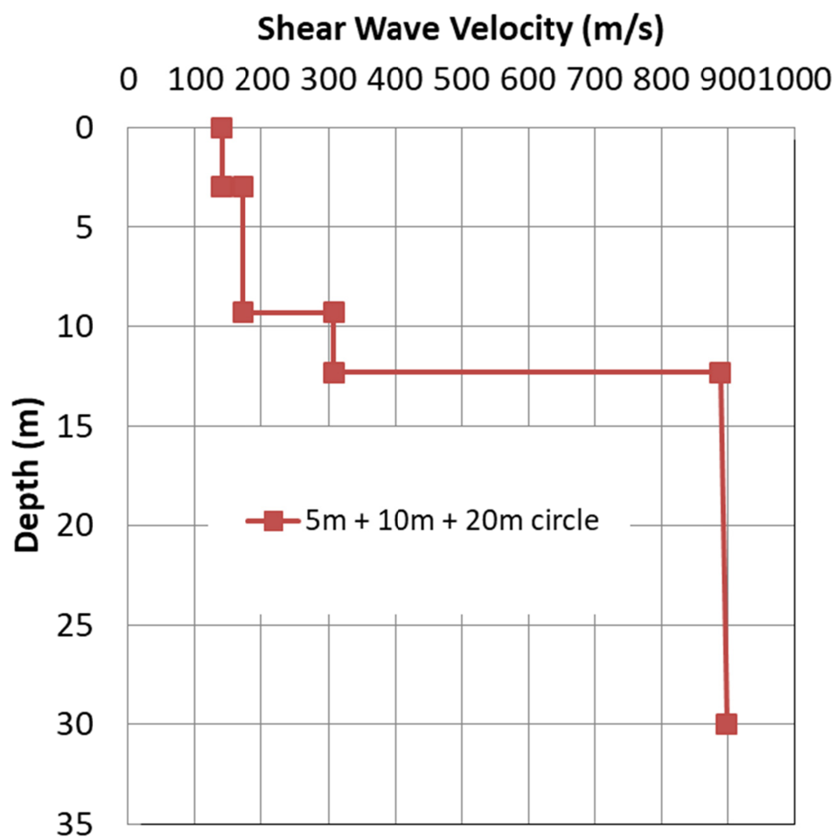


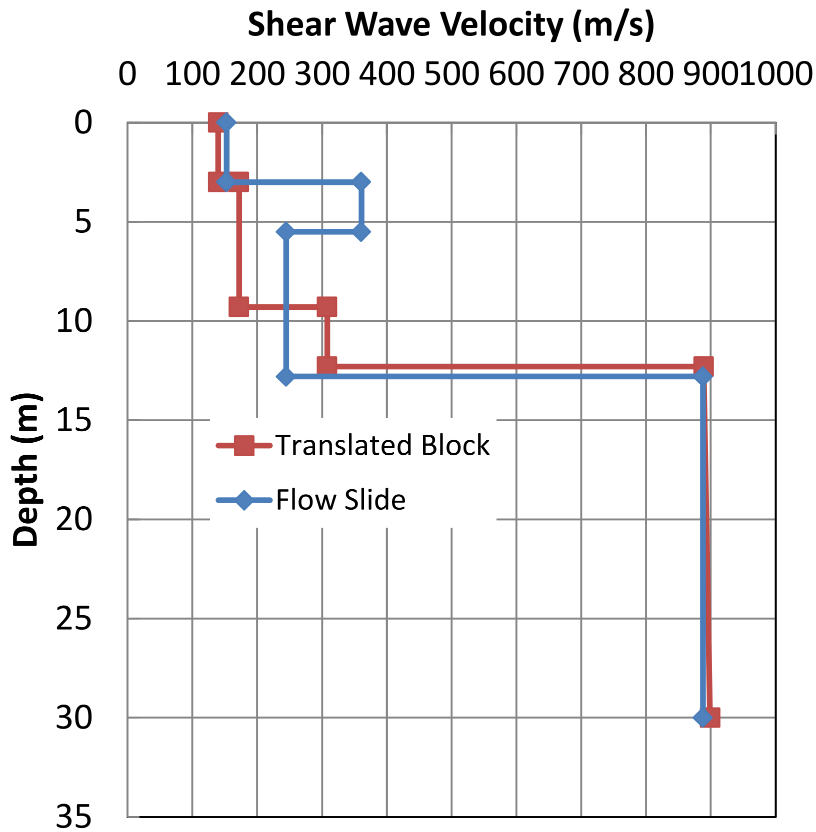

Shear Wave Velocity (m/s)
0 100 200 300 400 500 600 700 800 900 1000
0
5
10
15
20
25
30
35
Depth (m)
5m + 10m + 20m circle

G6 (translated block) s35.186669 w71.758161
6000-6009 is a 10-m array
6100-6104 is a 20-m array
6200-6209 is a 5-m array






Shear Wave Velocity (m/s)
0 100 200 300 400 500 600 700 800 9001000
0
5
10
15
20
25
30
35
Depth (m)
5m + 10m + 20m circle



Shear Wave Velocity (m/s)
0
100
200
300
400
500
600
700
800
900
1000
0
5
10
15
20
25
30
35
Depth (m)
Translated Block
Flow Slide

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