Investigation and Analysis of the Seismic Stability of Mine Waste and Tailings

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LMMG Geotecnia Limitada

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PANAMGEO CHILE 2024
17th PAN-AMERICAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING
2nd LATIN-AMERICAN REGIONAL CONFERENCE OF IAEG
LA SERENA • CHILE / NOVEMBER 12 - 16, 2024

LEARN BY DOING
CAL POLY

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Short Course Objectives

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17th PAN-AMERICAN CONFERENCE ON SOIL MECHANICS AND GEOTECHNICAL ENGINEERING

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This short course explores the best-practices for investigating and analyzing mine waste and mine tailings for seismic stability. The expected outcome is that attendees will be able to;

- properly characterize the subsurface conditions,
- identify if sand-like or clay-like physics control,
- highlight key static and seismic stability concerns,
- perform triggering analysis of liquefiable (sand-like) soils,
- determine post-triggering strength values, and evaluate post-triggering stability and runout distances.

Software that maybe useful during the short course (acquired free via trial versions):

- LiqIT (Geologismiki)
- Slide2 (Rocscience)

Case History of Seismic Induced Tailings Failure

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PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

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

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Pontifica Universidad Catolica de Chile

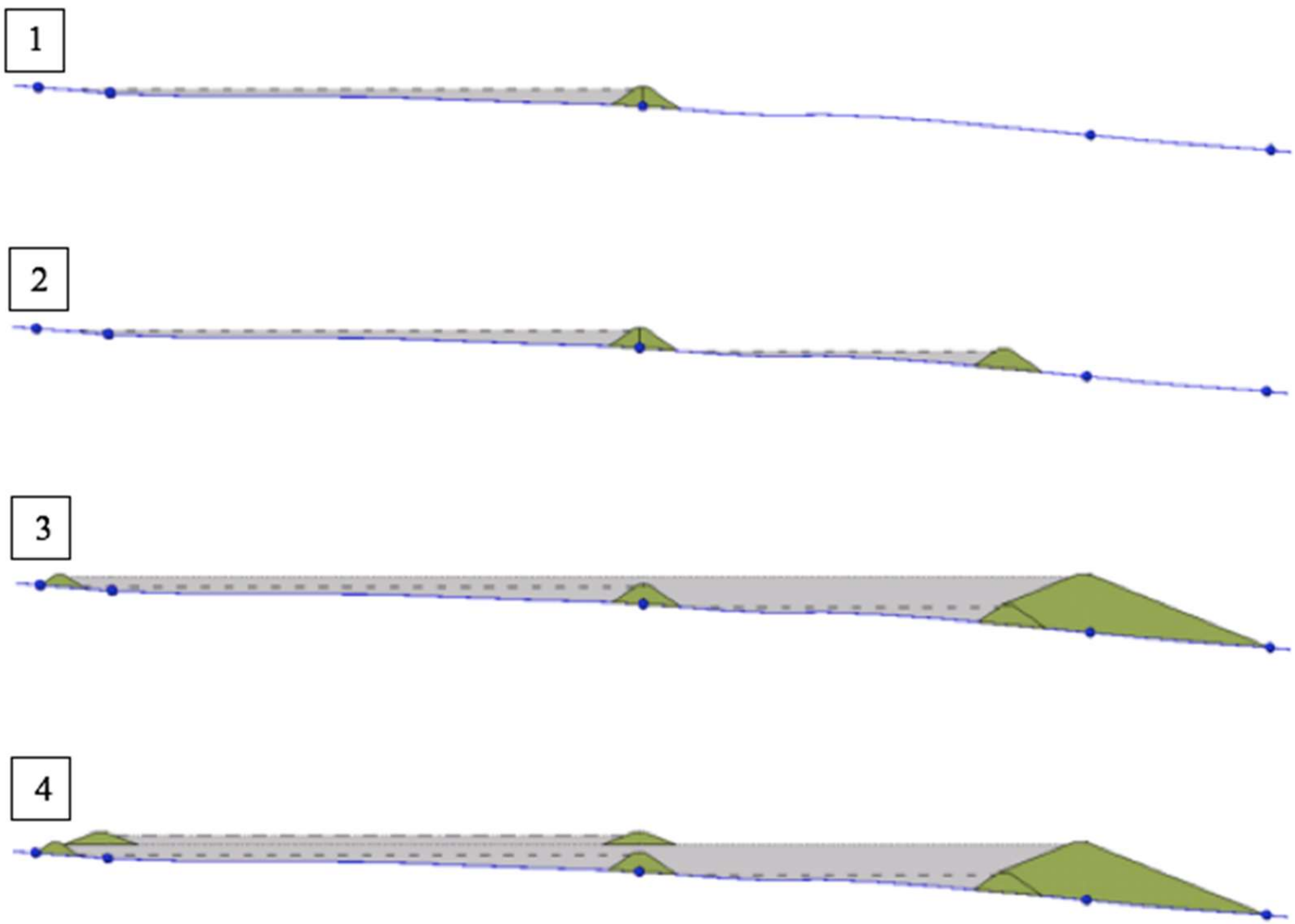
PEER Report No. 2019/01
Pacific Earthquake Engineering Research Center
Headquarters at the University of California, Berkeley

January 2019

PEER 2019/01
January 2019

https://peer.berkeley.edu/publications/2019-01


Algarrobo
Santiago
San Antonio
Maipú
Melipilla
Puente Alt
Navidad
Rancagua
Machalí
Pichilemu
Rengo
San Fernando
Santa Cruz
Lolol
Vichuquén
Alto Huemul
Licantén
Curicó
Curepto
Molina
Talca
San Javier
Colbún
Linares
Parral
Map data ©2017 Google
50 km
K-195
35.184679 S 71.759410 W
© 2017 Google
Image © 2017 DigitalGlobe
Google Earth
Google Earth
feet
2000
meters
800



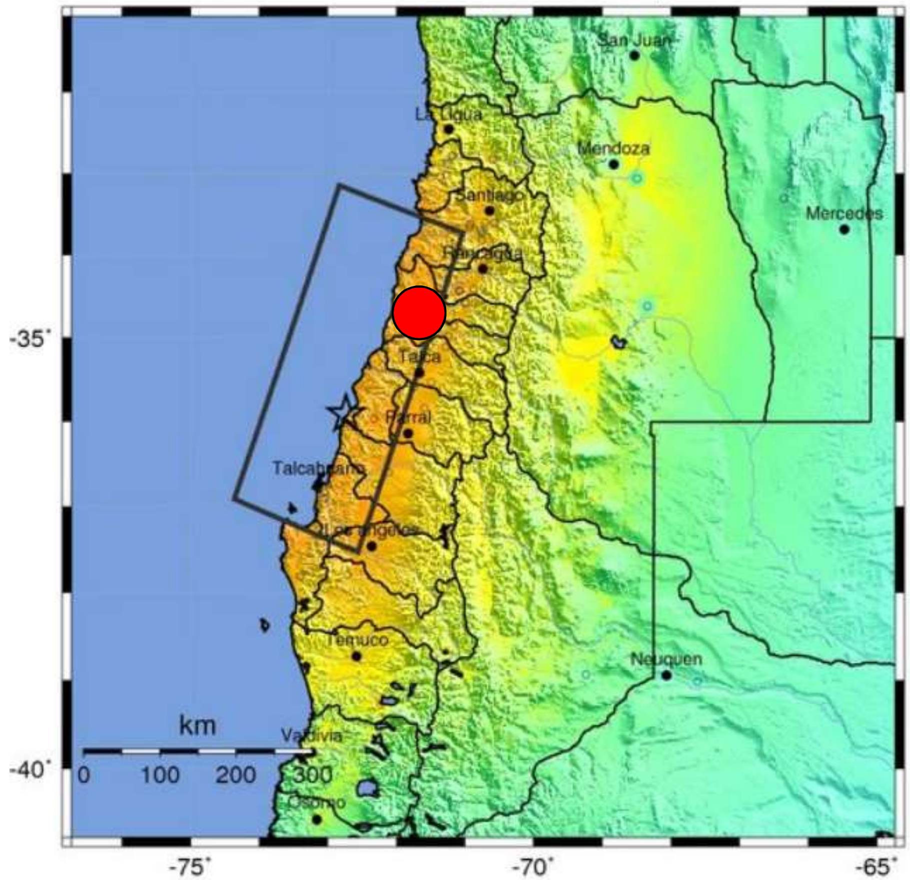
Gebhart, 2016

M8.8 Maule (Chile) Earthquake

- Februrary 27th
- Focal Depth 35km
- Fault Plane 500x100km
- Max slip > 8m
- Approx 600 deaths

USGS ShakeMap : OFFSHORE MAULE, CHILE

FEB 27 2010 06:34:14 AM GMT M 8.8 S35.91 W72.73 Depth: 35.0km ID:2010tfan



Map Version 10 Processed Wed Apr 11, 2012 05:10:42 PM MDT

PERCEIVED SHAKING	Not felt	Weak	Light	Moderate	Strong	Very strong	Severe	Violent	Extreme
POTENTIAL DAMAGE	none	none	none	Very light	Light	Moderate	Mod./Heavy	Heavy	Very Heavy
PEAK ACC.(%g)	<0.1	0.5	2.4	6.7	13	24	44	83	>156
PEAK VEL.(cm/s)	<0.07	0.4	1.9	5.8	11	22	43	83	>160
INSTRUMENTAL INTENSITY	I	II-III	IV	V	VI	VII	VIII	IX	X+

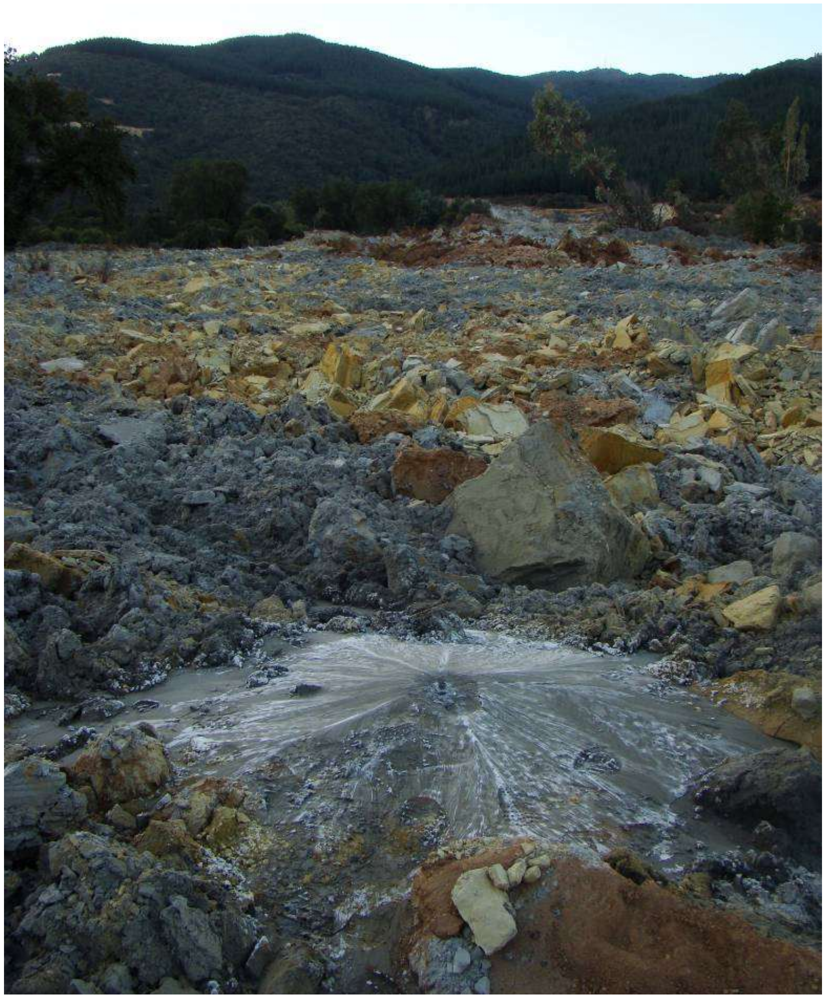
Scale based upon Wald, et al.; 1999

GEER, 2010


GEER, 2010

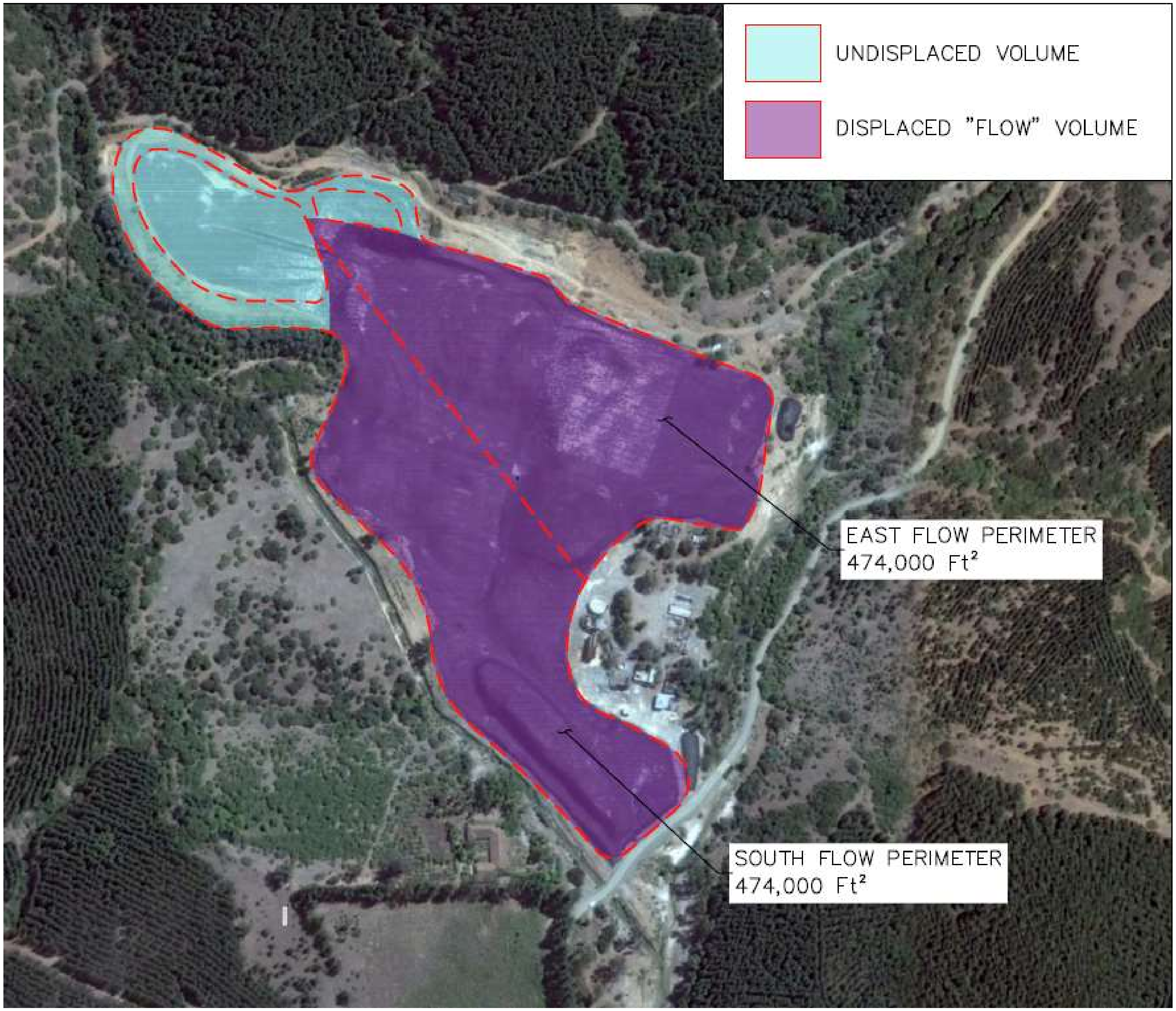
Santa Maria, 2012; from Gebhart, 2016


After
Legend
-35.1847 -71.7594
-35.1847 -71.7594
Google Earth
Image © 2018 DigitalGlobe
800 ft
N



GEER, 2010

GEER, 2010



UNDISPLACED VOLUME
DISPLACED "FLOW" VOLUME
EAST FLOW PERIMETER
474,000 Ft²
SOUTH FLOW PERIMETER
474,000 Ft²

TRIGGERING

Rainfall infiltration

Seepage

Static Liquefaction

Seismic Liquefaction

Landslide

Rock Fall

Rock Slide

Quick Conditions

FLOW

Steady State Shearing
w/ Excess Pore Pressures
(Bingham flow)

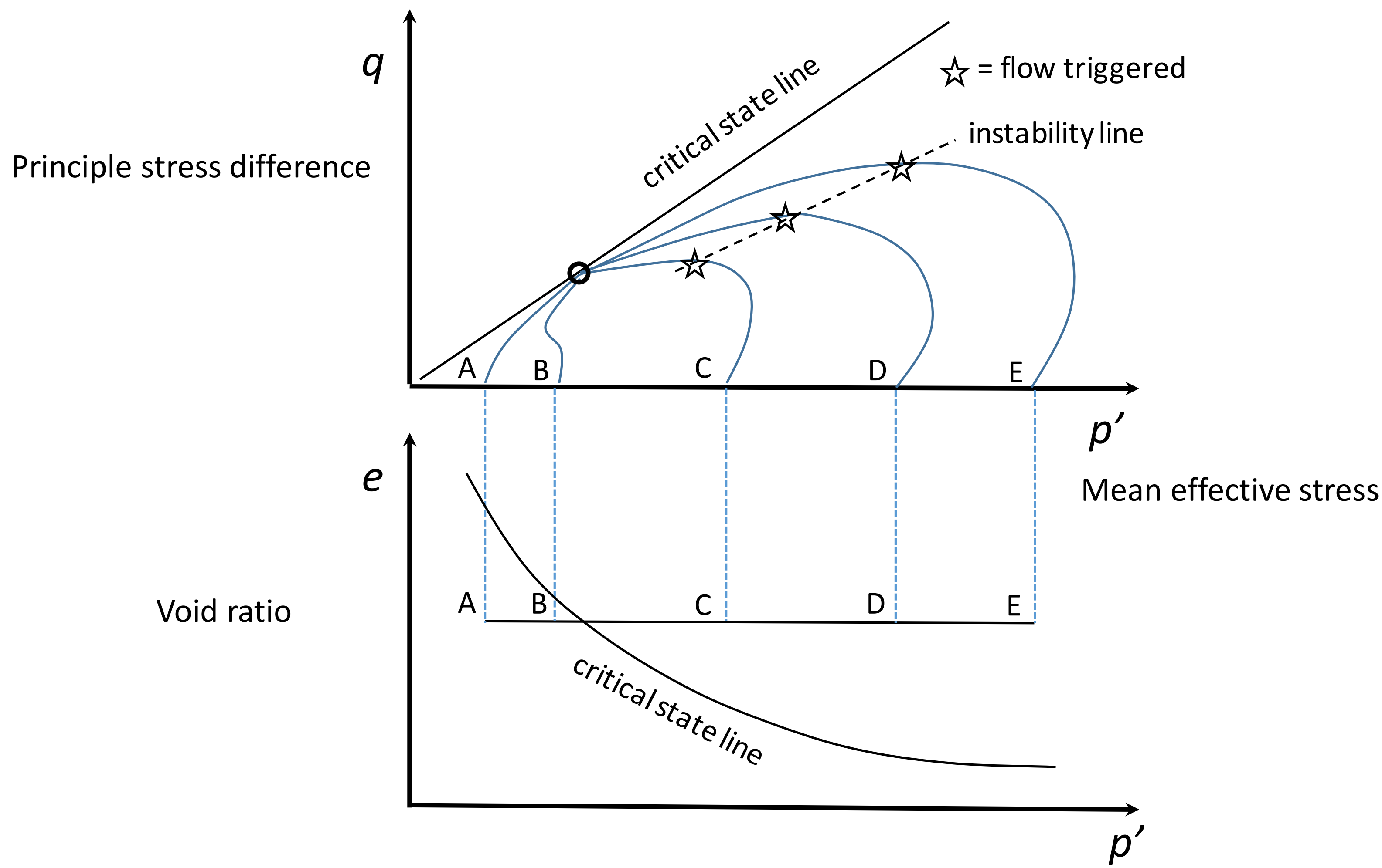
RUNOUT

High vs Low Granular Temp

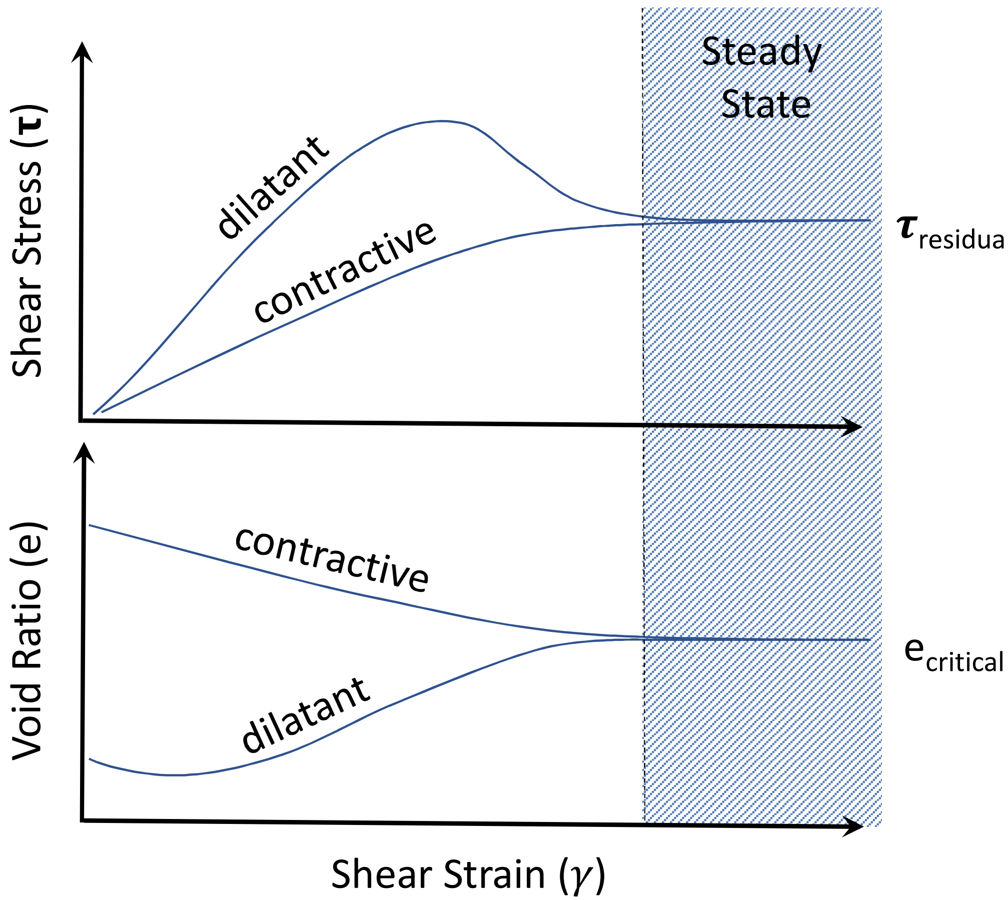
Rapid vs Slow Drainage

Turbulent vs Laminar Flow

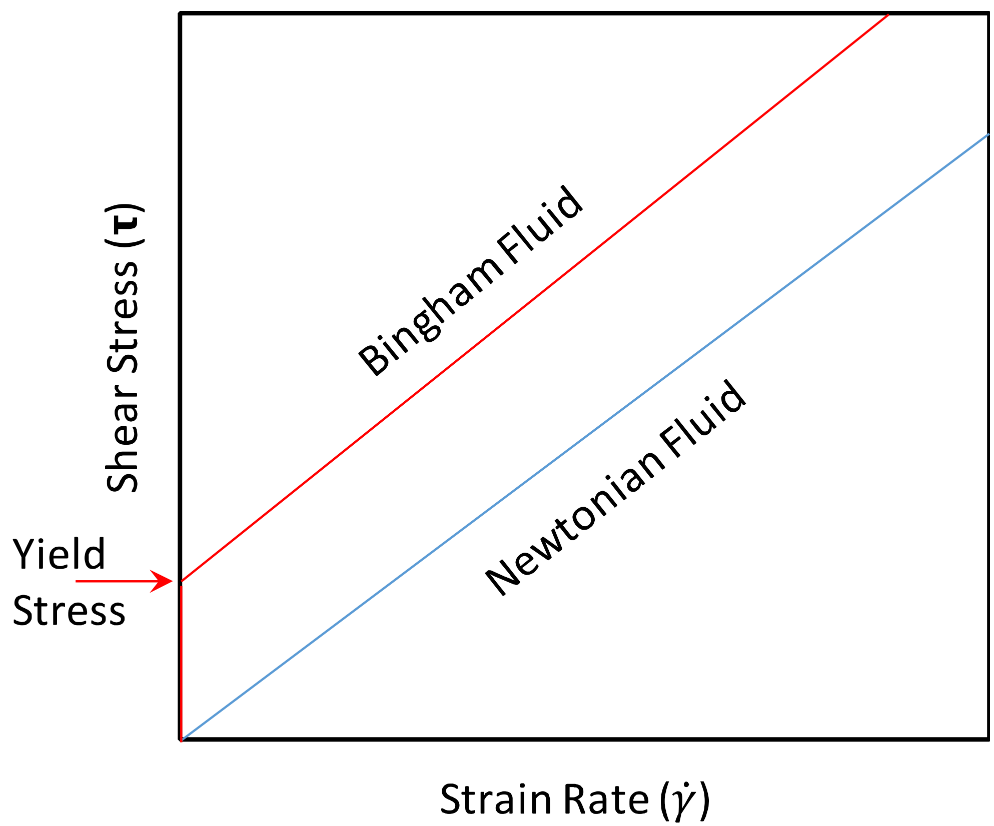
Constant vs Changing Slope



q
Principle stress difference
critical state line
= flow triggered
instability line
A
B
C
D
E
p'
Mean effective stress
e
Void ratio
A
B
C
D
E
critical state line
p'



Steady State
Shear Stress (τ)
dilatant
contractive
$\tau_{residual}$
Void Ratio (e)
contractive
dilatant
$e_{critical}$
Shear Strain (γ)



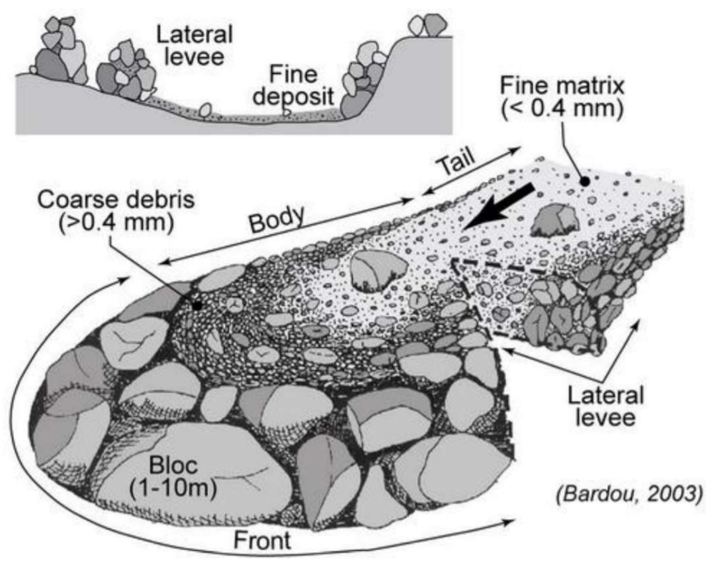
Shear Stress (τ)
Bingham Fluid
Newtonian Fluid
Yield
Stress
Strain Rate ($\dot{\gamma}$)

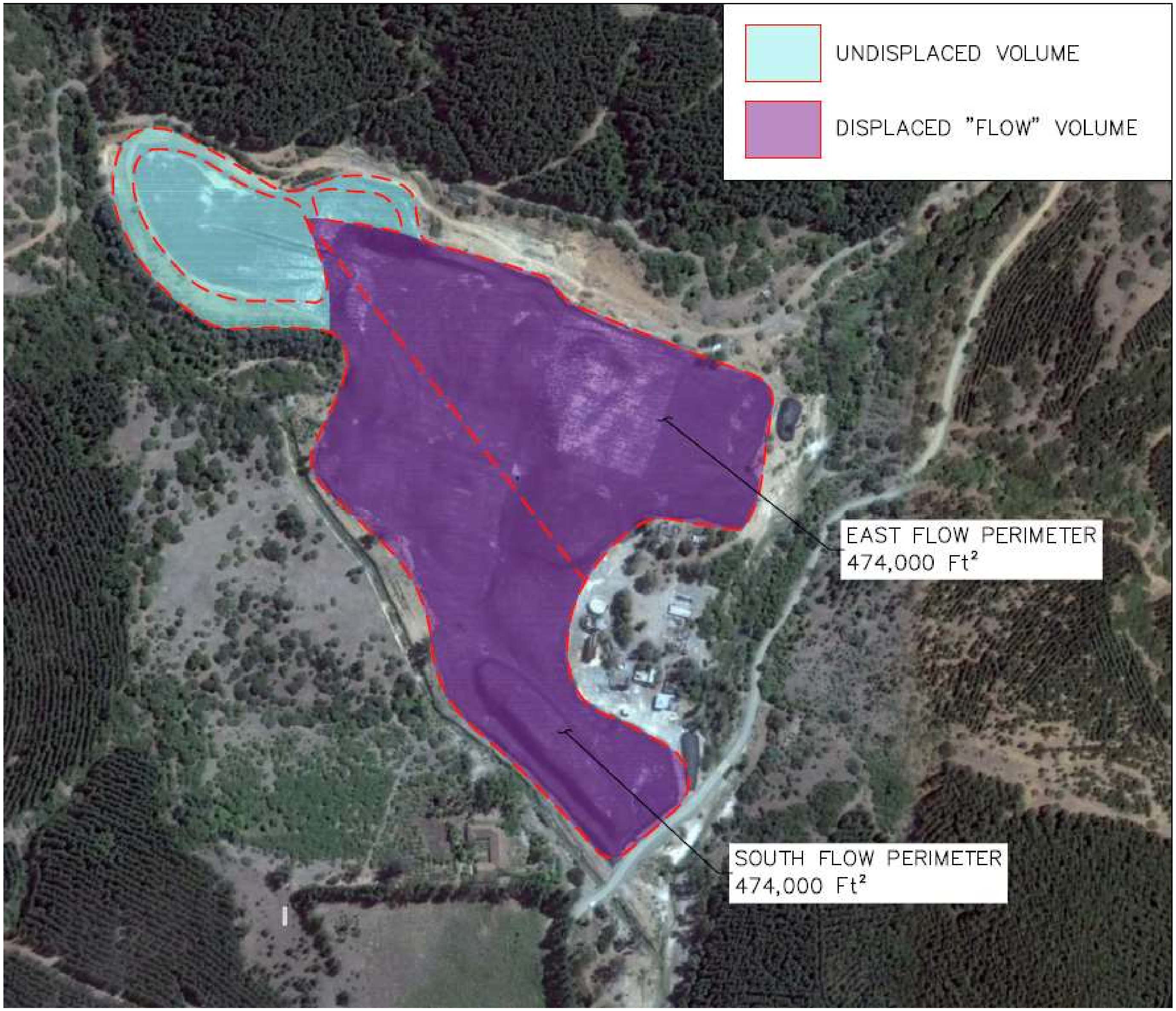
High vs. Low Granular Temperature

Rapid vs. Slow Drainage

Turbulent vs. Laminar Flow

Constant vs. Changing Slope




UNDISPLACED VOLUME
DISPLACED "FLOW" VOLUME
EAST FLOW PERIMETER
474,000 Ft²
SOUTH FLOW PERIMETER
474,000 Ft²

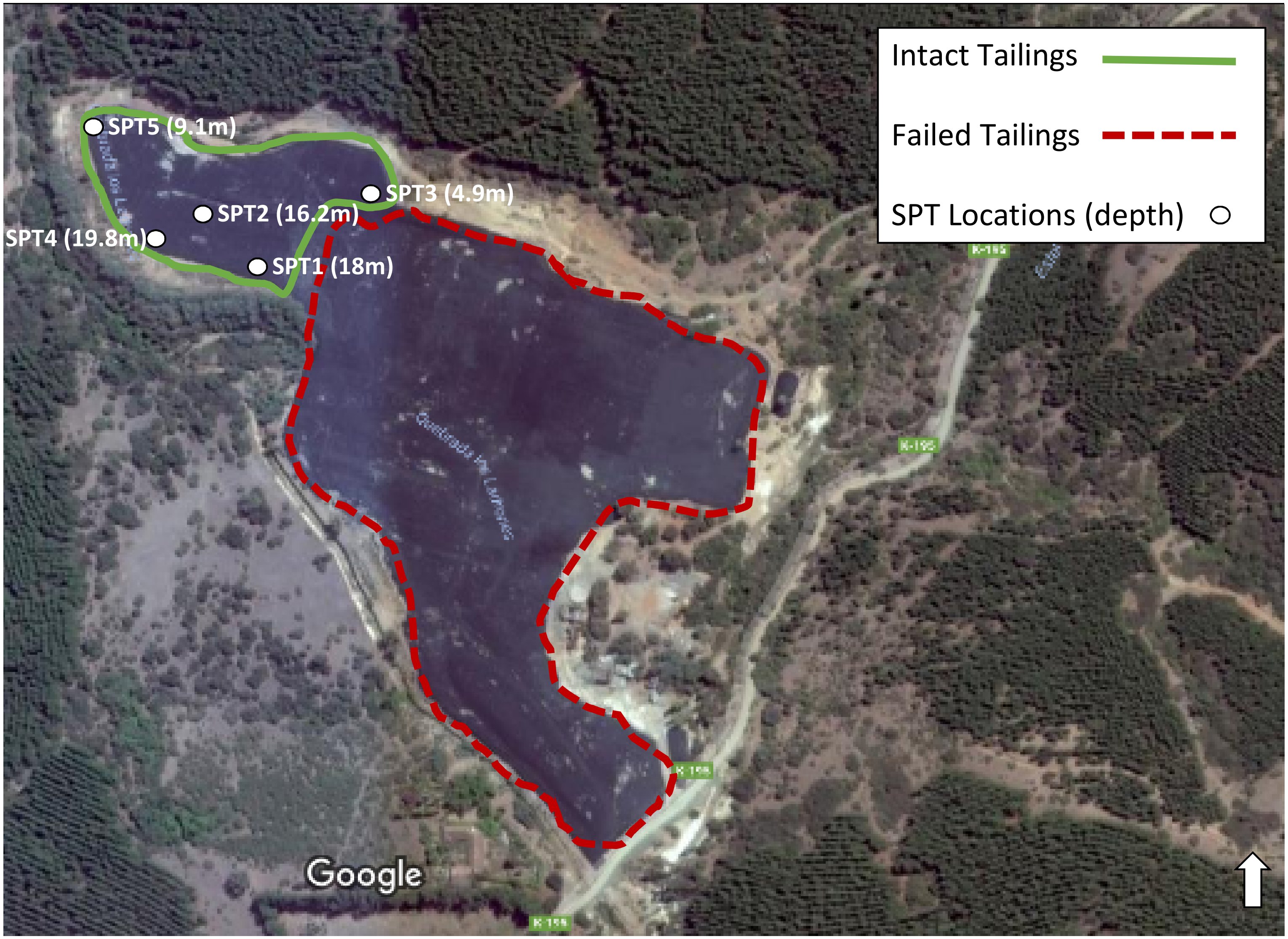
Subsurface investigations were needed to measure the engineering properties, back-analyze the problem, and learn from this failure.

SPT (Blows / 0.3 m + soil samples lab testing) - DICTU

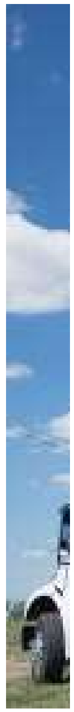
CPT (tip, sleeve, pore pressure) - PEER

V_S (shear wave velocity) - PEER


NO ENTRAR
FAENA
CERRADA
CAMINO
CERRADO



Imagery ©2017 CNES / Airbus, DigitalGlobe, Map data ©2017 Google 100 m



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
-2					10	15						83	
-5					7	11						79	
-8					5	6						76	
-12					6	6						90	
-15					5	5						81	
-20 ▼					2	2						75	
-23					10	9						73	
-28					7	6						59	
-30													

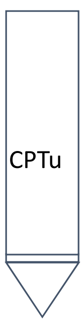

LMMG
C-SZ72

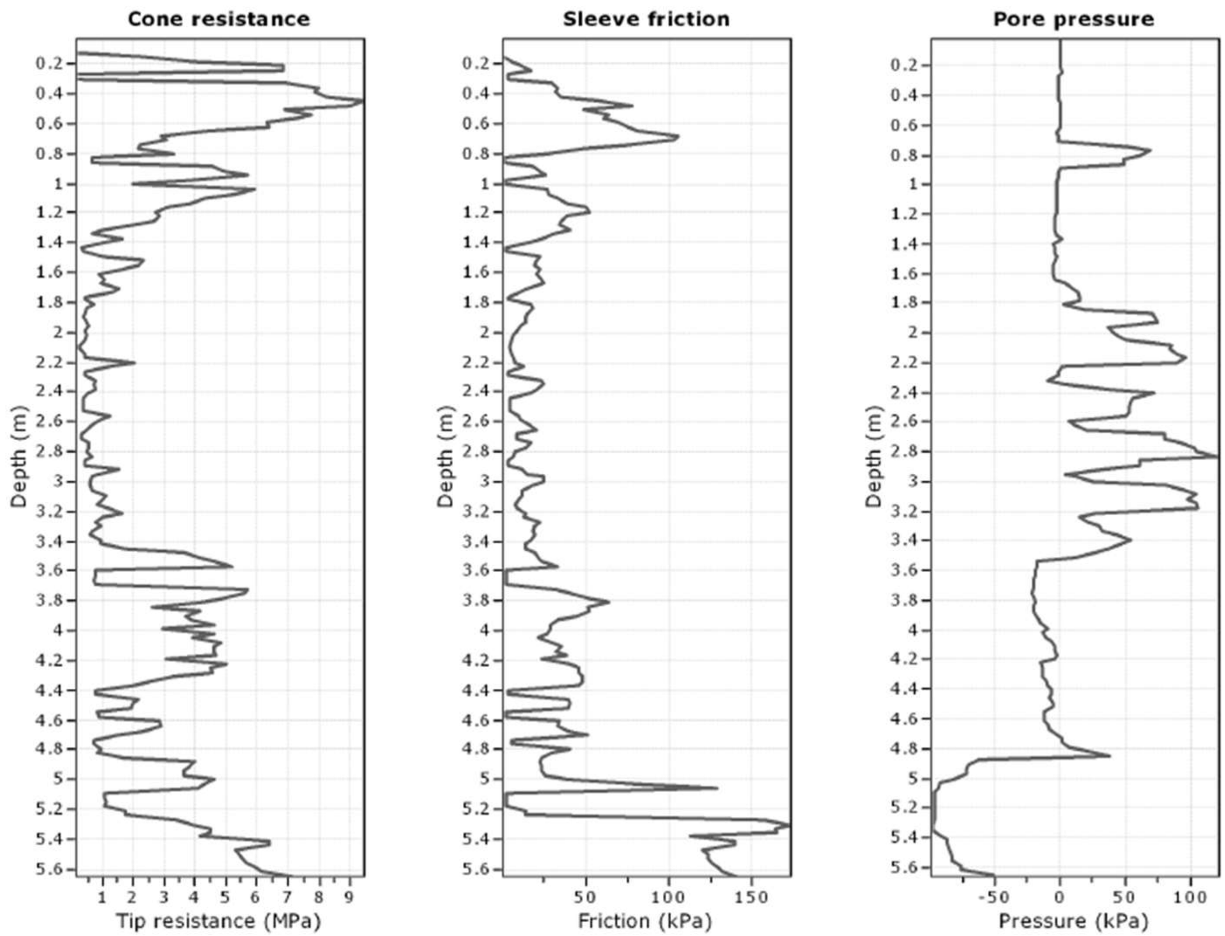

CPTu

LMMG GEOTECNIA LTDA
WWW.LMMG.CL
CONTACTO@LMMG.CL

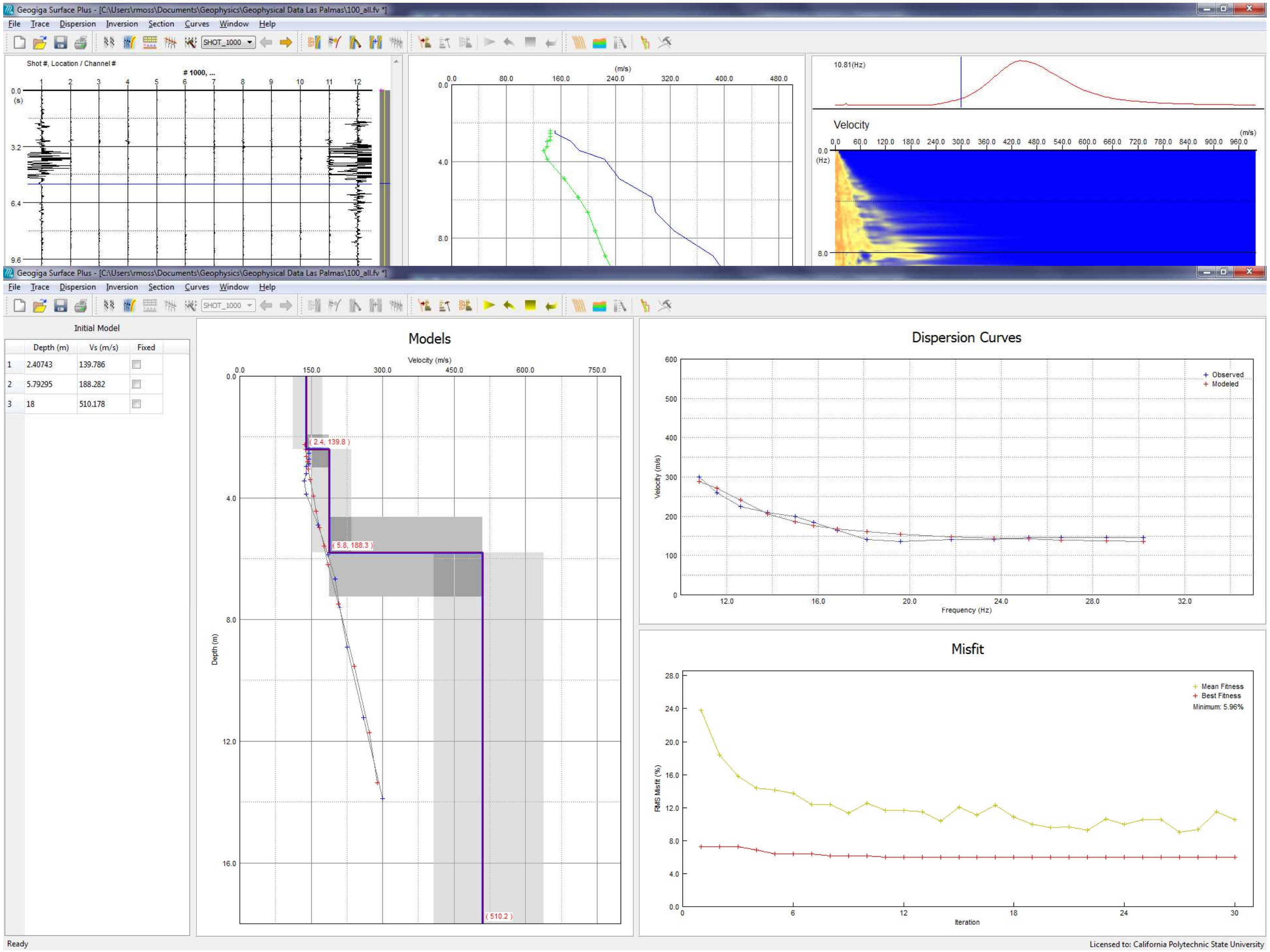
Project:
Location:

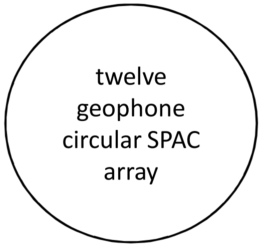
CPT: CPT-01
Total depth: 5.65 m, Date: 27-06-2017
Surface Elevation: 0.00 m
Coords: X:0.00, Y:0.00
Cone Type: Uknown
Cone Operator: Uknown

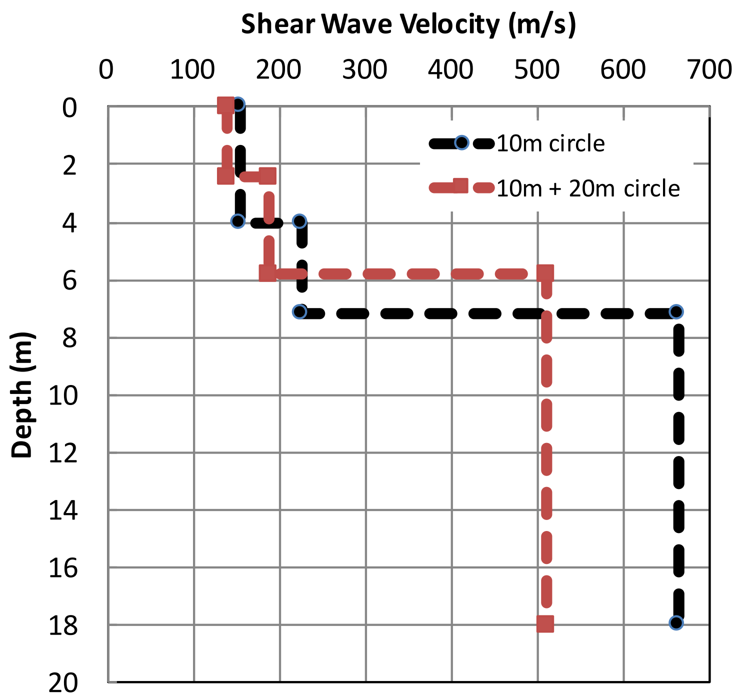


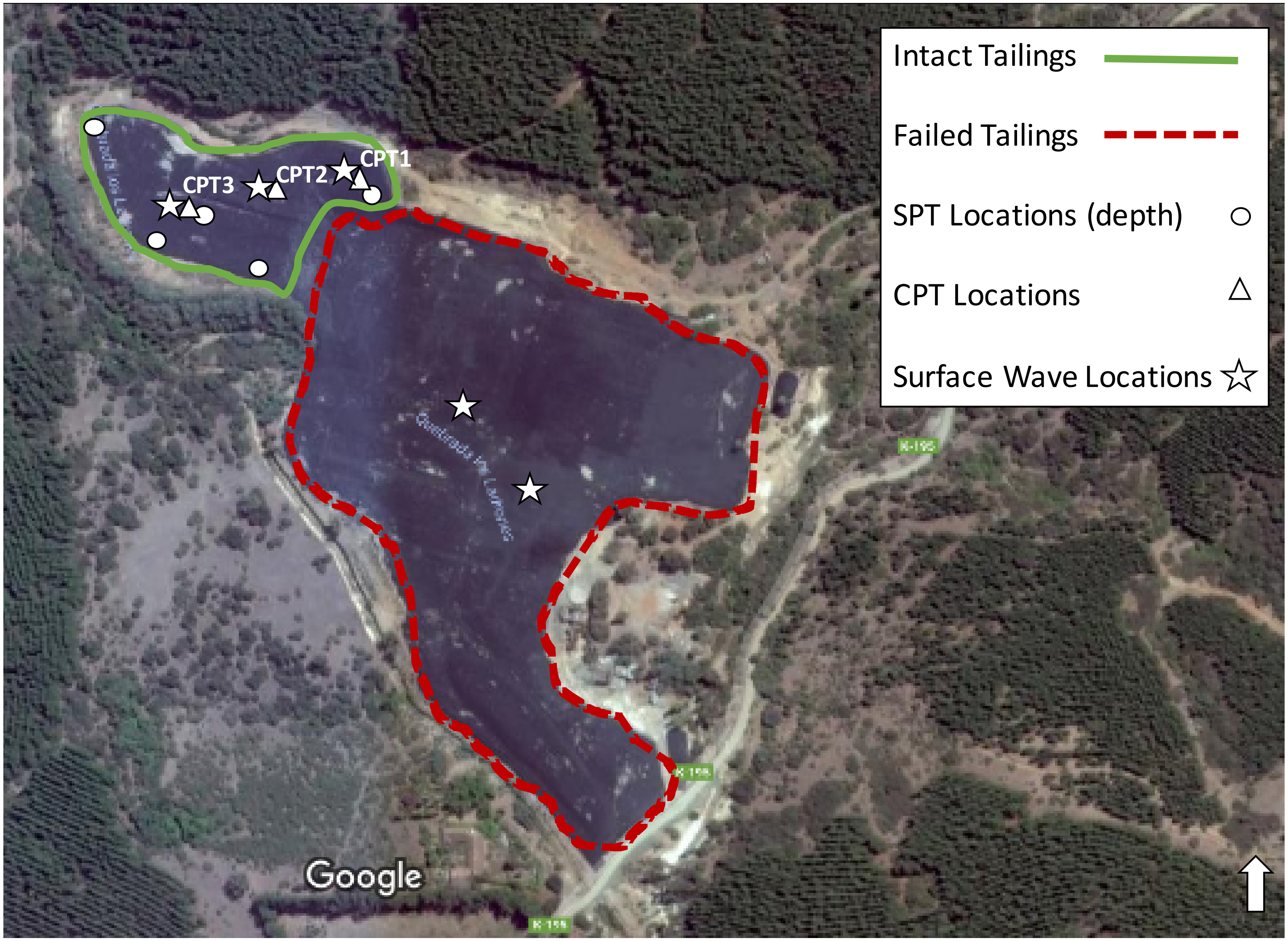



V_S
twelve geophone circular SPAC array


Geogiga Surface Plus - [C:\Users\rmoss\Documents\Geophysics\Geophysical Data Las Palmas\100_all.fv *]
File Trace Dispersion Inversion Section Curves Window Help
SHOT_1000
Shot #, Location / Channel #
# 1000, ...
10.81(Hz)
Velocity
Initial Model
Depth (m) Vs (m/s) Fixed
1 2.40743 139.786
2 5.79295 188.282
3 18 510.178
Models
Velocity (m/s)
Depth (m)
(2.4, 139.8)
(5.8, 188.3)
(510.2)
Dispersion Curves
Observed
Modeled
Velocity (m/s)
Frequency (Hz)
Misfit
Mean Fitness
Best Fitness
Minimum: 5.96%
RMS Misfit (%)
Iteration
Ready
Licensed to: California Polytechnic State University


twelve geophone circular SPAC array


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



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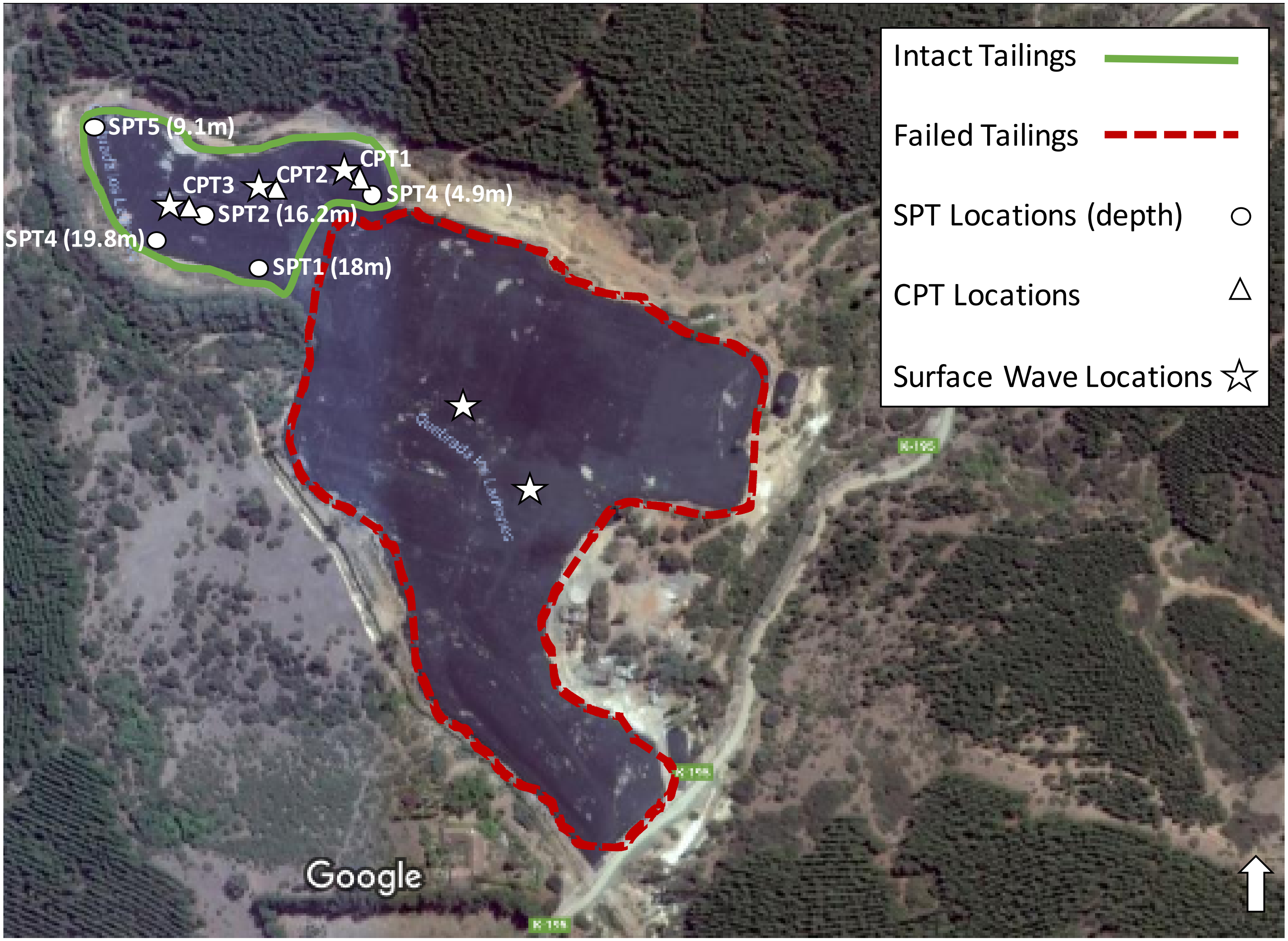

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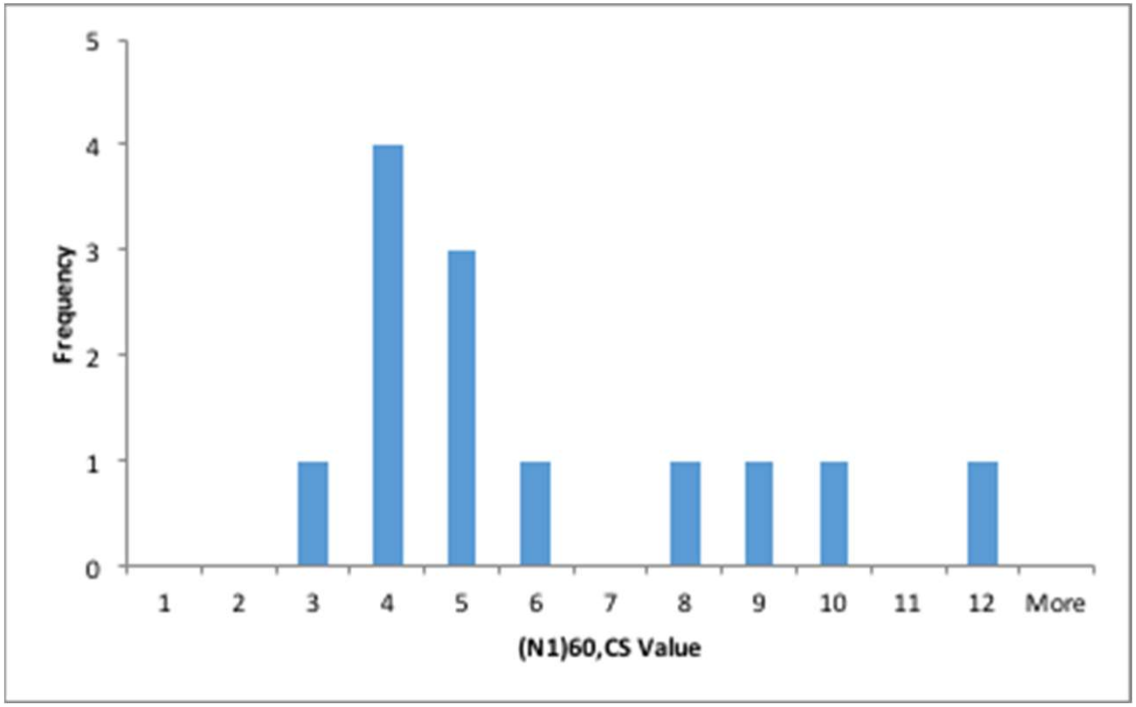
Cummulative Field Investigations on Las Palmas Tailings Dam Failure

- ✓ LIDAR
- ✓ SASW (1)
- ✓ SPT (5) and Lab Testing
- ✓ CPT (3 +1)
- ✓ SPAC (5)

Intended Use of Data

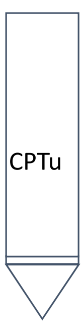
- Post-Liquefaction Residual Strength Database
- Calibration of Flow Failure Numerical Modeling
- New Standard for Flow Failure Case Histories

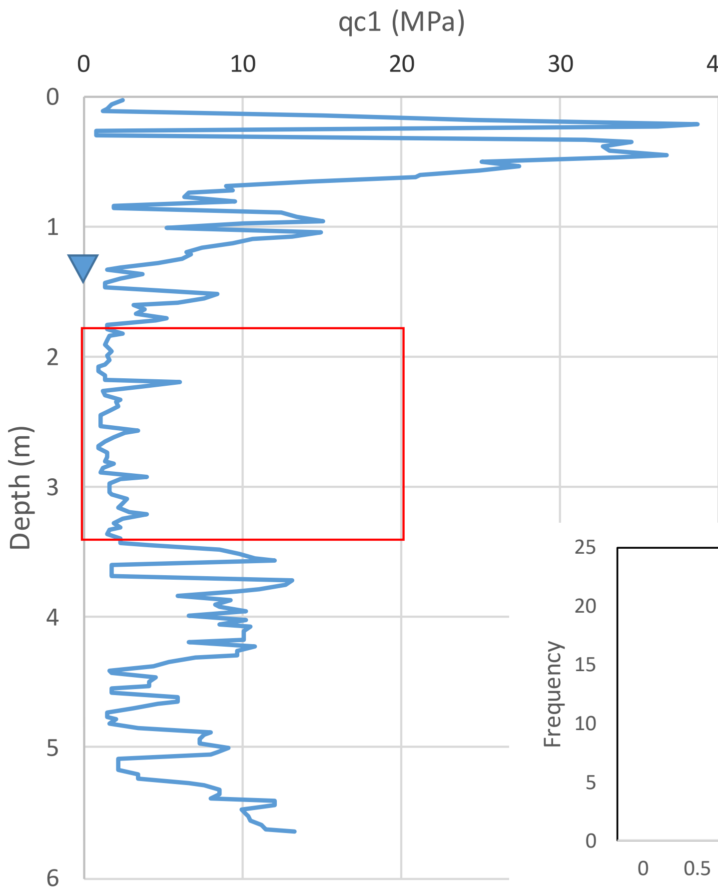
SPT



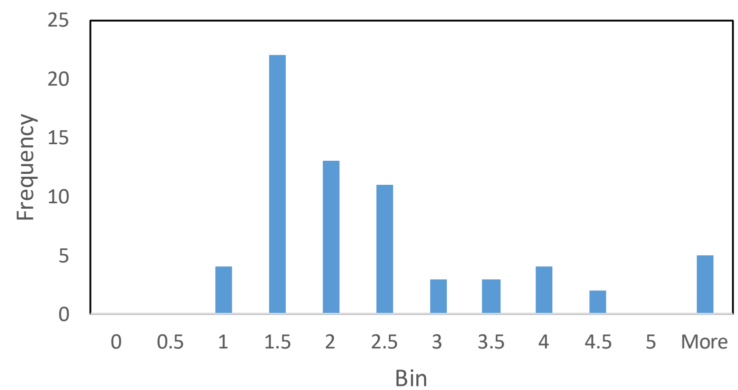
Histogram of blow counts in saturated tailings material, with fines correction (borings B-2,3,4) thought to best represent material susceptible to liquefaction with a median of 5 and a CoV of 25%

CPT





CPT1 overburden corrected tip resistance with histogram of boxed region thought to best represent the tailings material susceptible to liquefaction and flow failure. The median are approximately 1.3 MPa with a CoV of 10%.

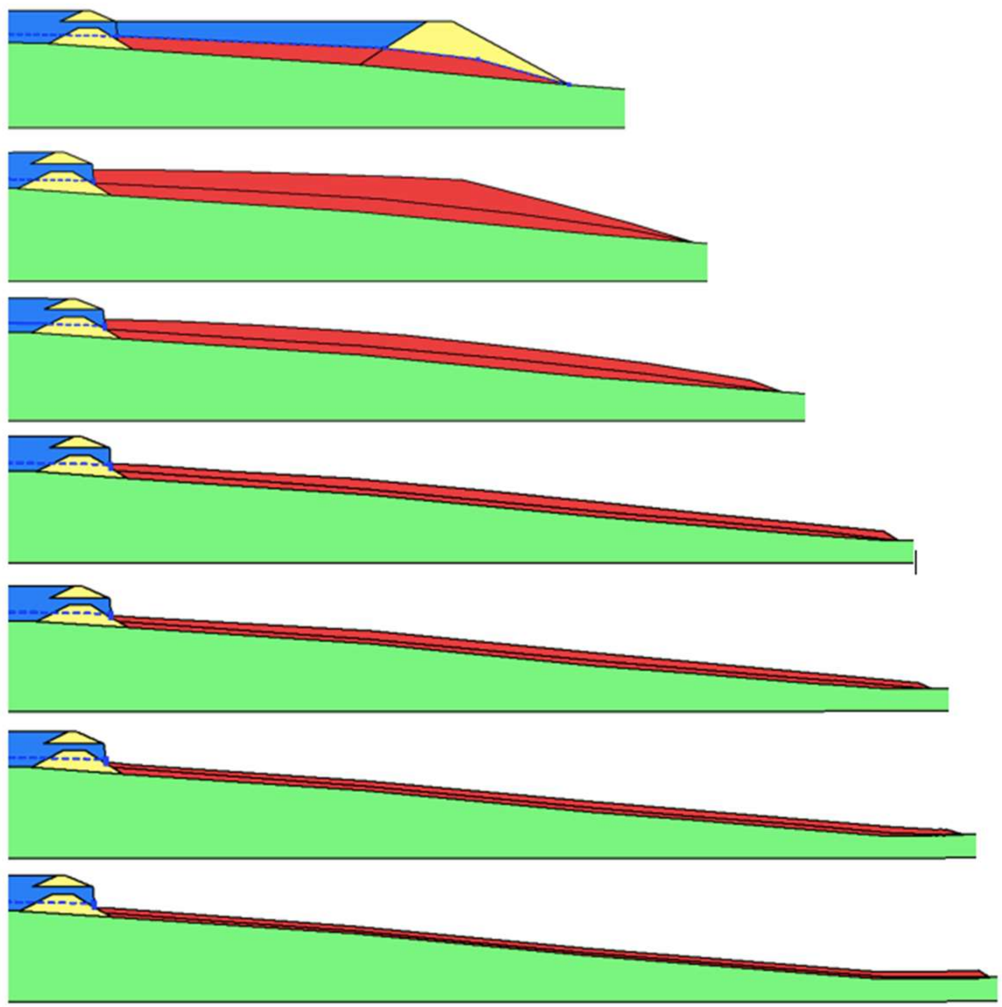


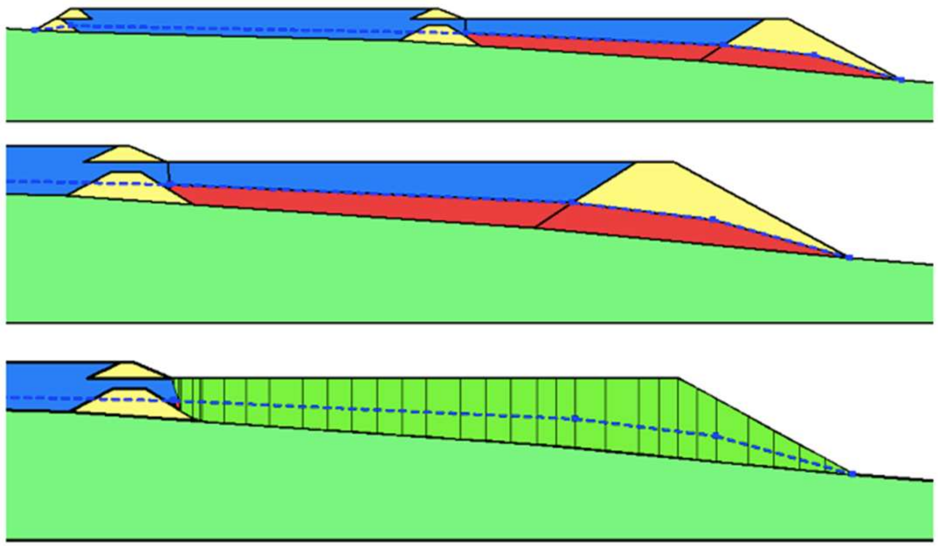
V_S

twelve geophone circular SPAC array

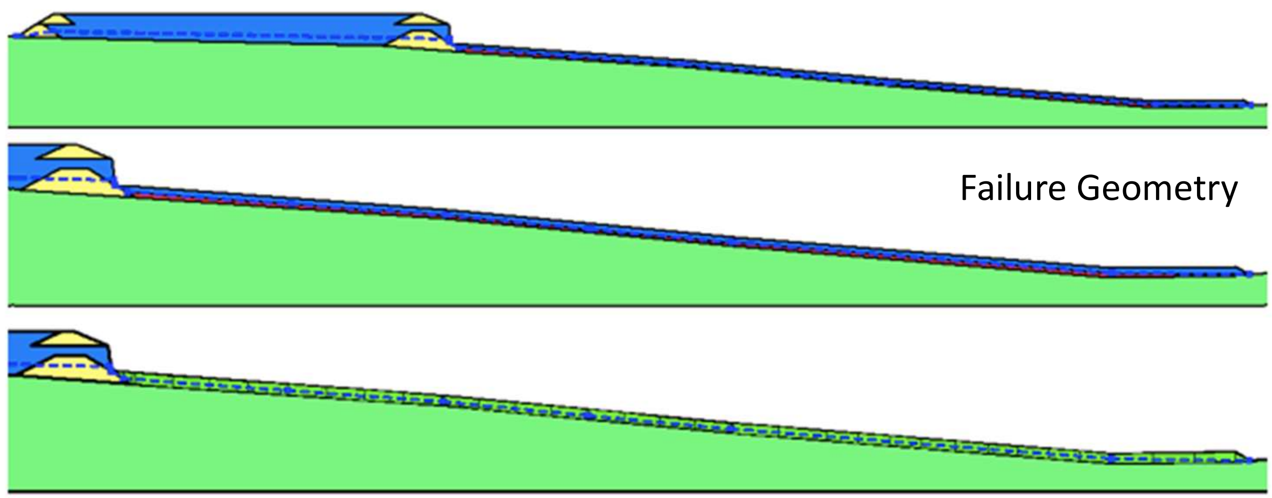
Profile	Depth Range (m)	Average VS1 (m/s)
G1	0 to 5	211
G2	na	na
G3	0 to 8	172
G5	0 to 3	222
G6	3 to 9	175

Back-Analysis



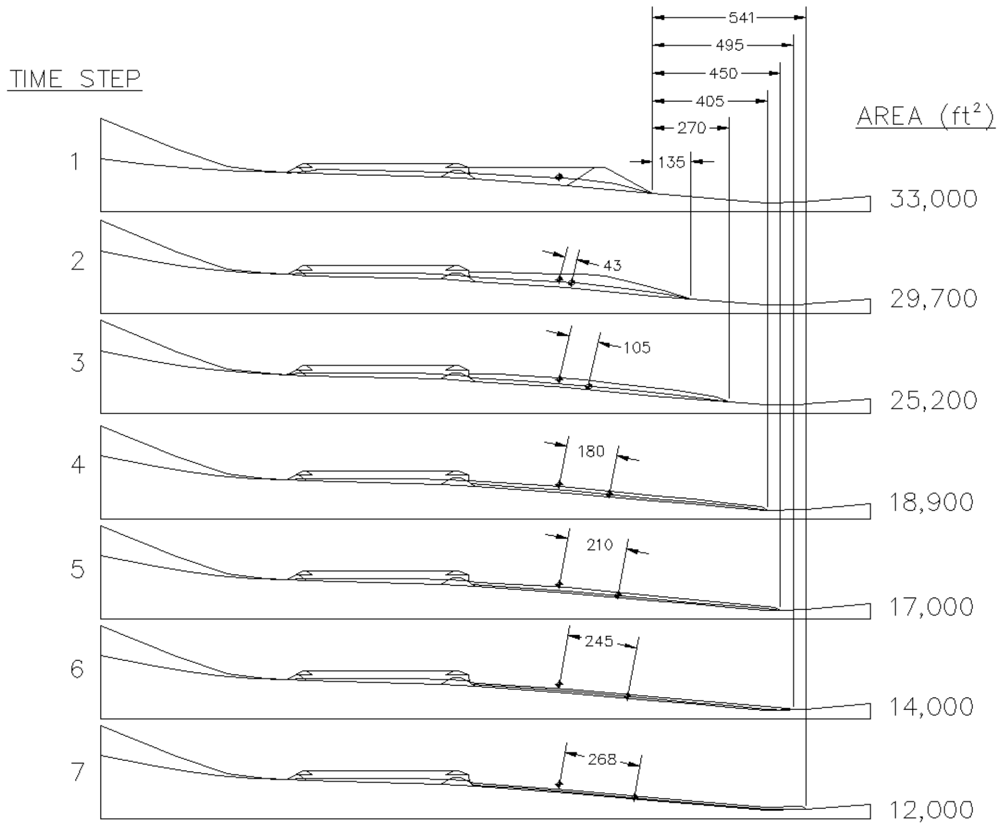


Original Geometry

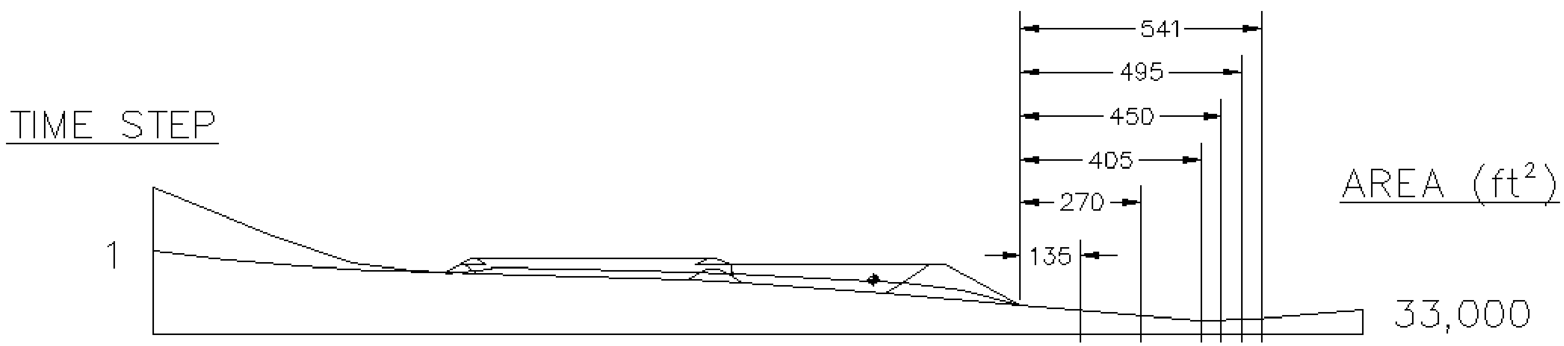


Failure Geometry

Residual Strength Back Analysis using the Incremental Momentum Method (Weber et al., 2015)

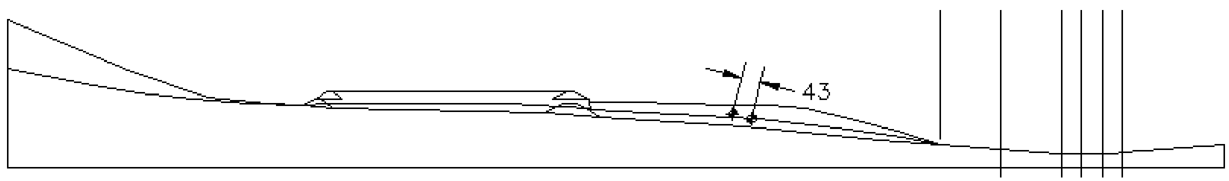


Gebhart, 2016

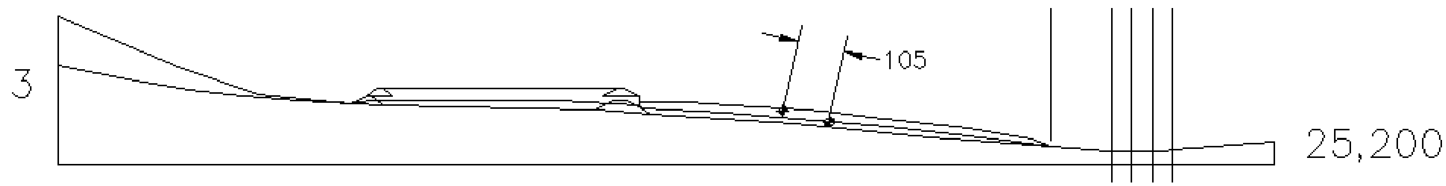


TIME STEP
1
541
495
450
405
270
135
AREA (ft²)
33,000

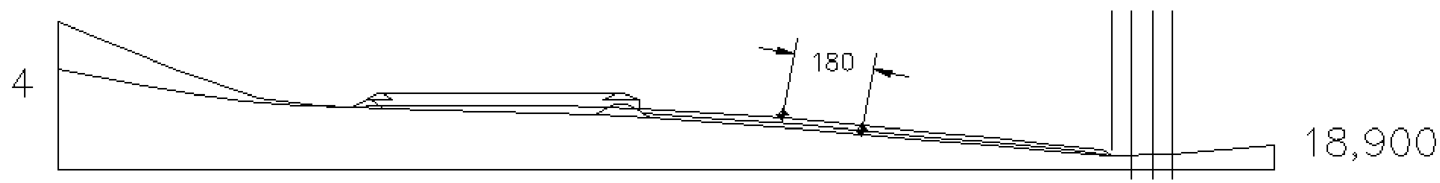
2



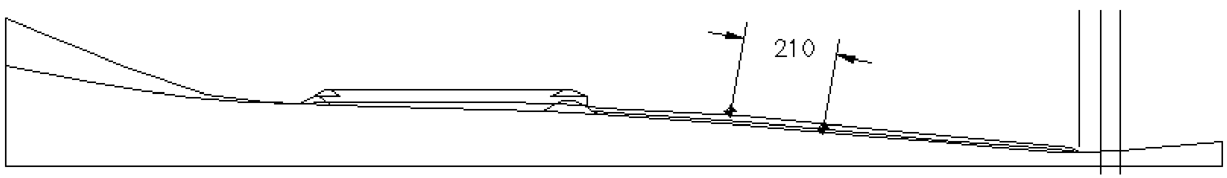
29,700



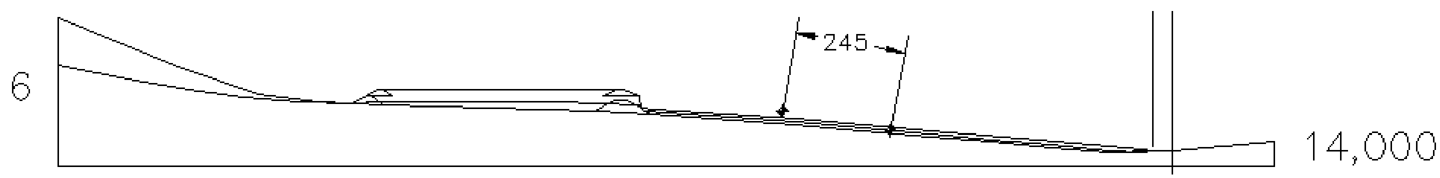
3
105
25,200


4
180
18,900

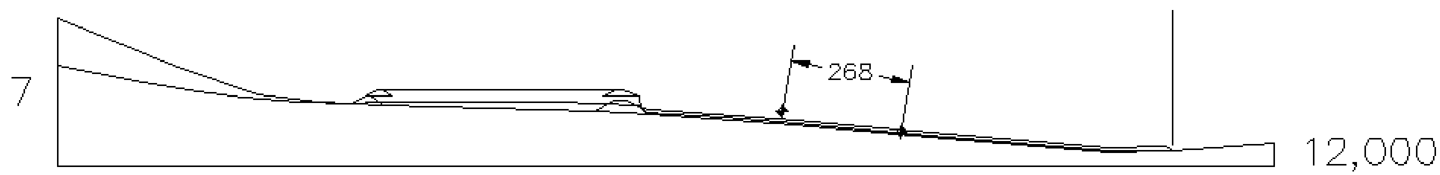
5



17,000

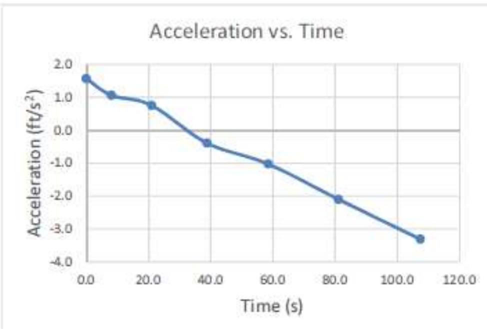


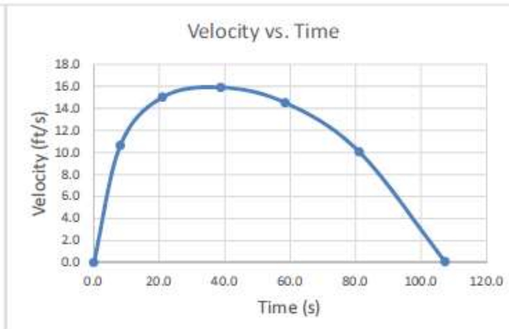
6
245
14,000

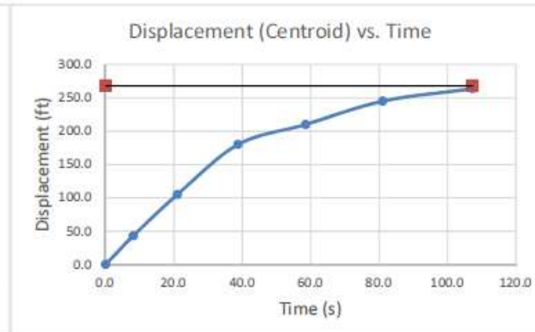

7
268
12,000

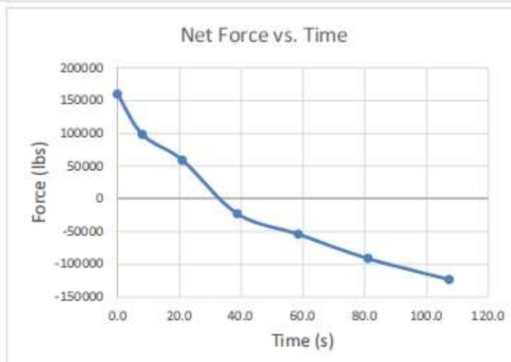
Incremental Momentum Analysis: Trial 7
Residual Strength = 180 psf
Debris Flow

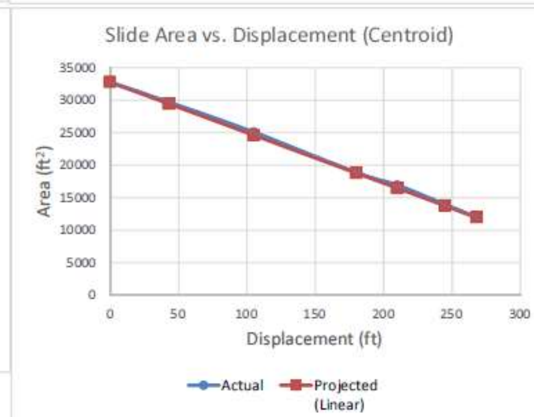
		Displacement (ft)		Area (ft²)	Weight (lb)	Force (lb)			Accel. (ft/s2)	Time (s)		Velocity (ft/s)		Displacement (ft)			Area Check (ft²)		
Time Step	FS	Total	Increment	Increment	Increment	Driving	Resisting	Net	Increment	Increment (Goal Seek)	Total	Increment	Total	Increment	Total	Difference (Goal Seek)	Projected (Linear)	Difference (Proj - Act)	% Diff
1	0.44	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%

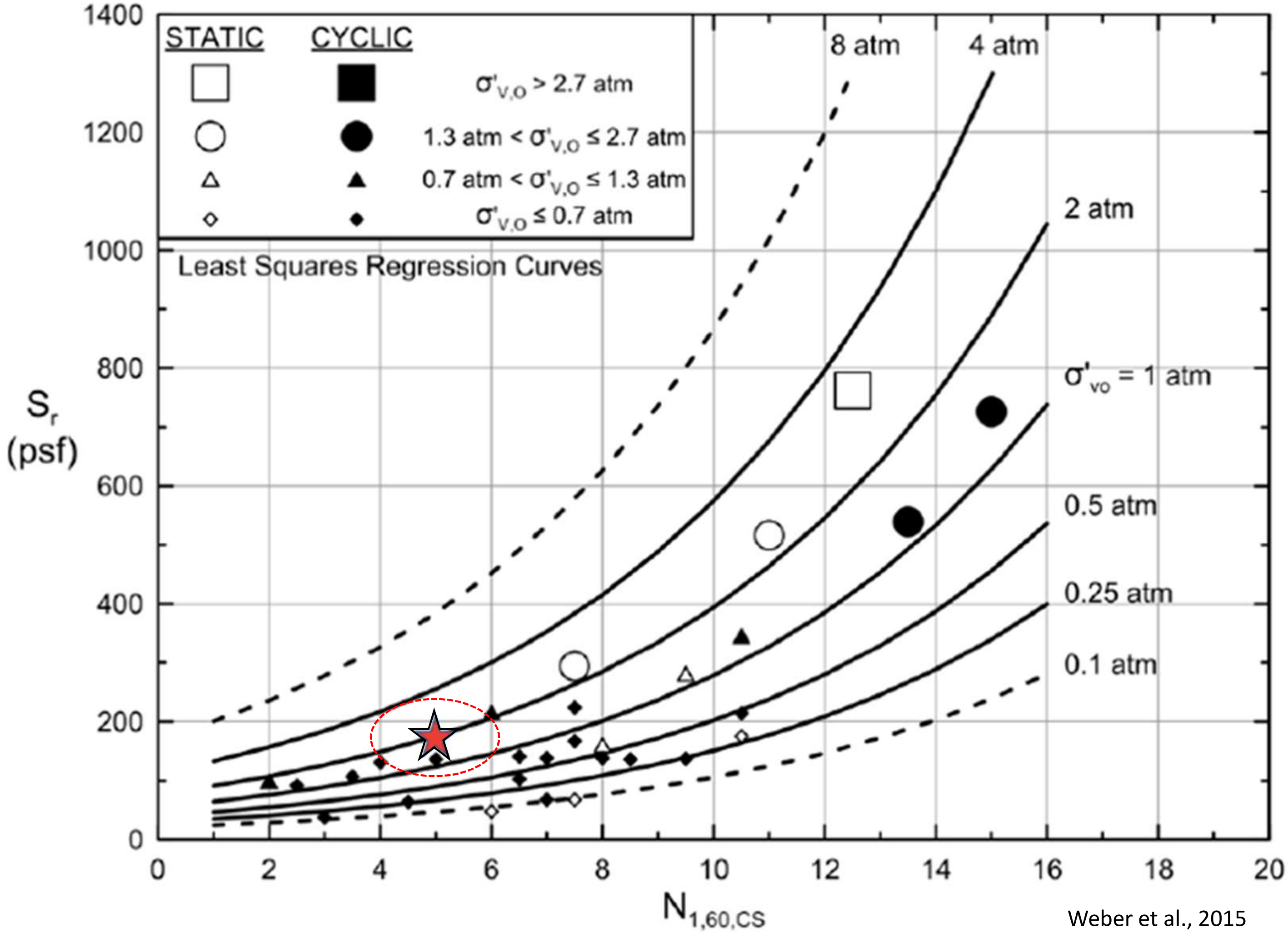












Weber et al., 2015

Acknowledgments

- Tristan Gebhart, PE, CalTrans
- Prof. Christian Ledezma, PUC
- Prof. David Frost, Georgia Tech
- Prof. Joe Weber, Loyola Marymount
- Prof. Jon Stewart, UCLA

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Caltrans

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Protecting People and the Environment

SwRI

Forward Analysis

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<u>Liquefaction flow failure engineering analysis</u>

1. Subsurface investigation (sCPTu + SPT preferred) for tailings/dam strength measurements

2. Measure mean and max water table conditions

3. Susceptiblilty assessment of materials

4. Triggering assessment of weak layers/foundation

5. Post-liquefaction psuedo-static stability analysis using residual strength

6. If FS<1.0 then address runout and/or consequences

Susceptibility

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Sand-like → Liquefaction → Post-Seismic

vs

Clay-like → Cyclic Failure → Co-Seismic

Cyclic failure of sensitive clays in Nepal 2015

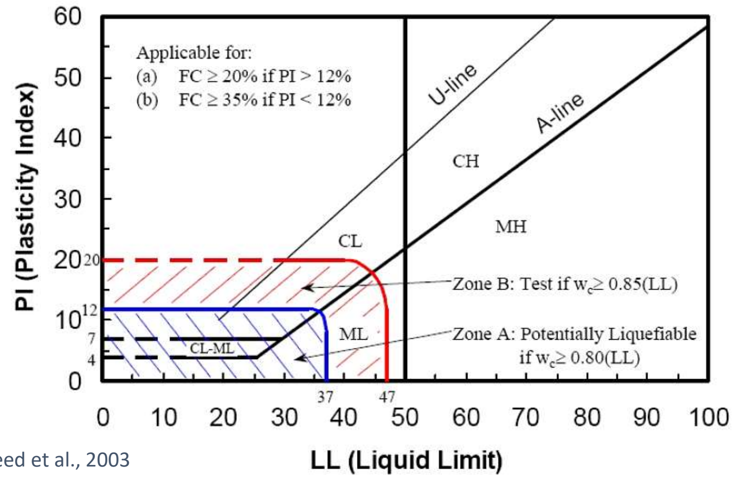
Co-seismic: Shaking stops = deformations stop

Liquefaction flow failure of tailings in Chile 2010

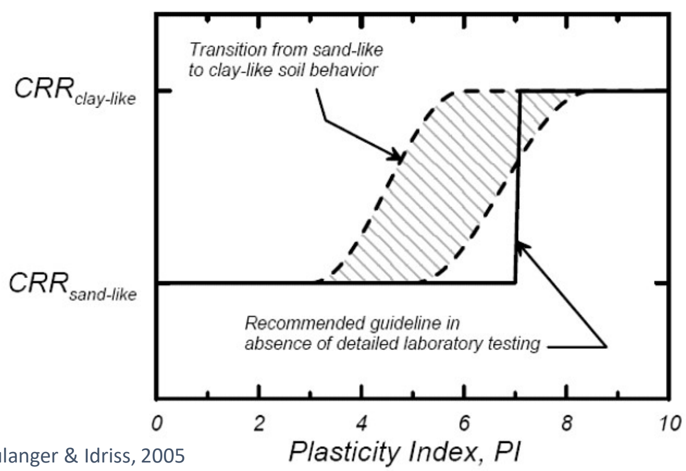
Post-seismic: Shaking ”breaks” the slope and deformations continue until no further momentum, and/or excess pore pressures dissipate independent ground shaking.

Fully Saturated (below WT)

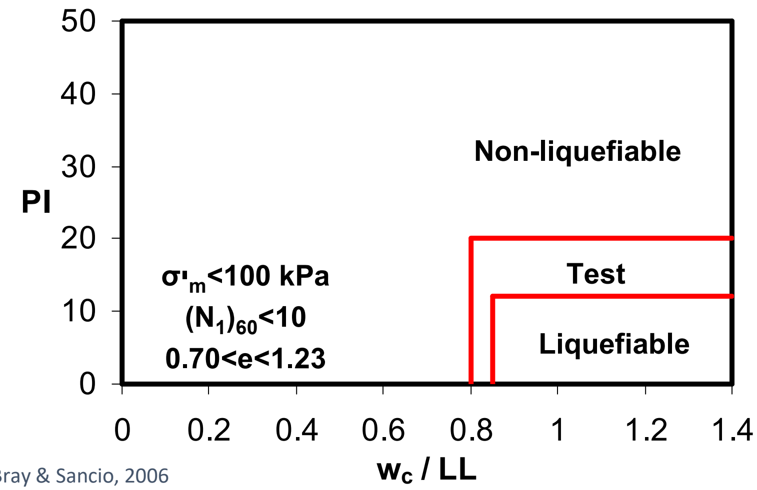
20% – 30% Fines
Control Threshold



Seed et al., 2003



Boulanger & Idriss, 2005



Bray & Sancio, 2006

Moss et al., 2006 – No evidence of liquefaction for Rf>5%

Robertson & Wride 1998 – No evidence of liquefaction for Ic>2.6

Triggering

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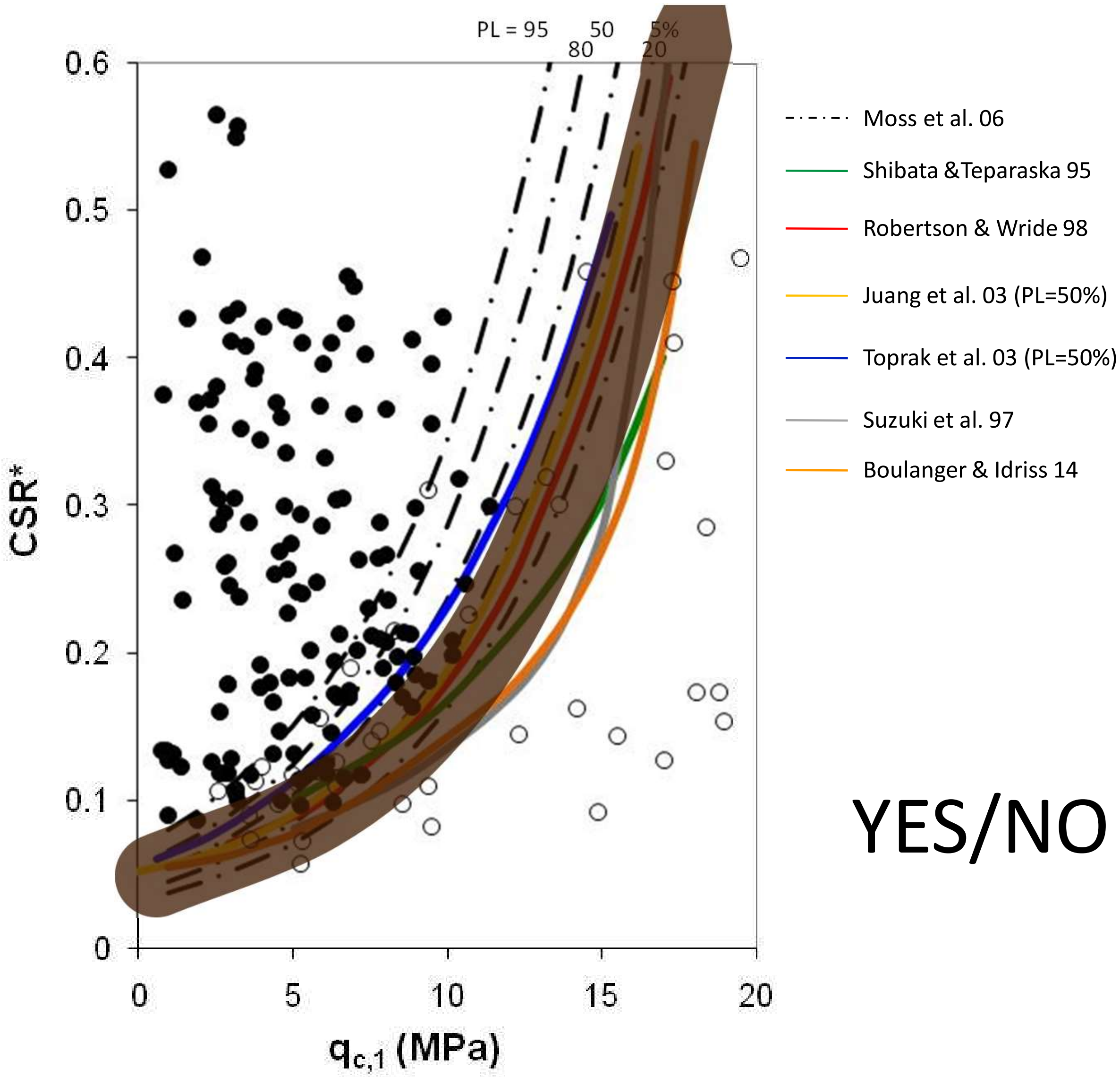
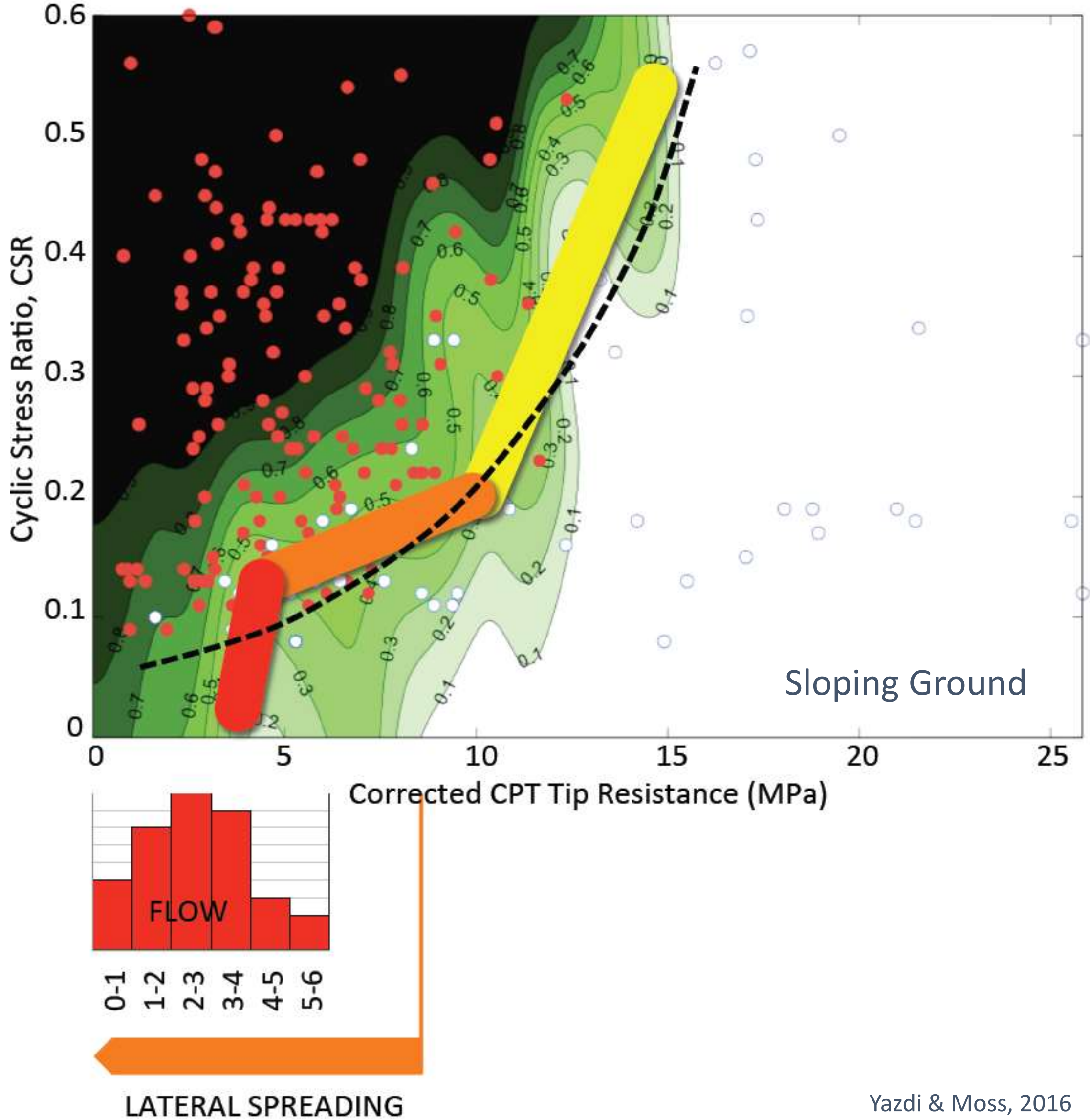

PL = 95
80
50
20
5%
0.6
0.5
0.4
0.3
0.2
0.1
0
CSR*
0
5
10
15
20
qc,1 (MPa)
Moss et al. 06
Shibata &Teparaska 95
Robertson & Wride 98
Juang et al. 03 (PL=50%)
Toprak et al. 03 (PL=50%)
Suzuki et al. 97
Boulanger & Idriss 14

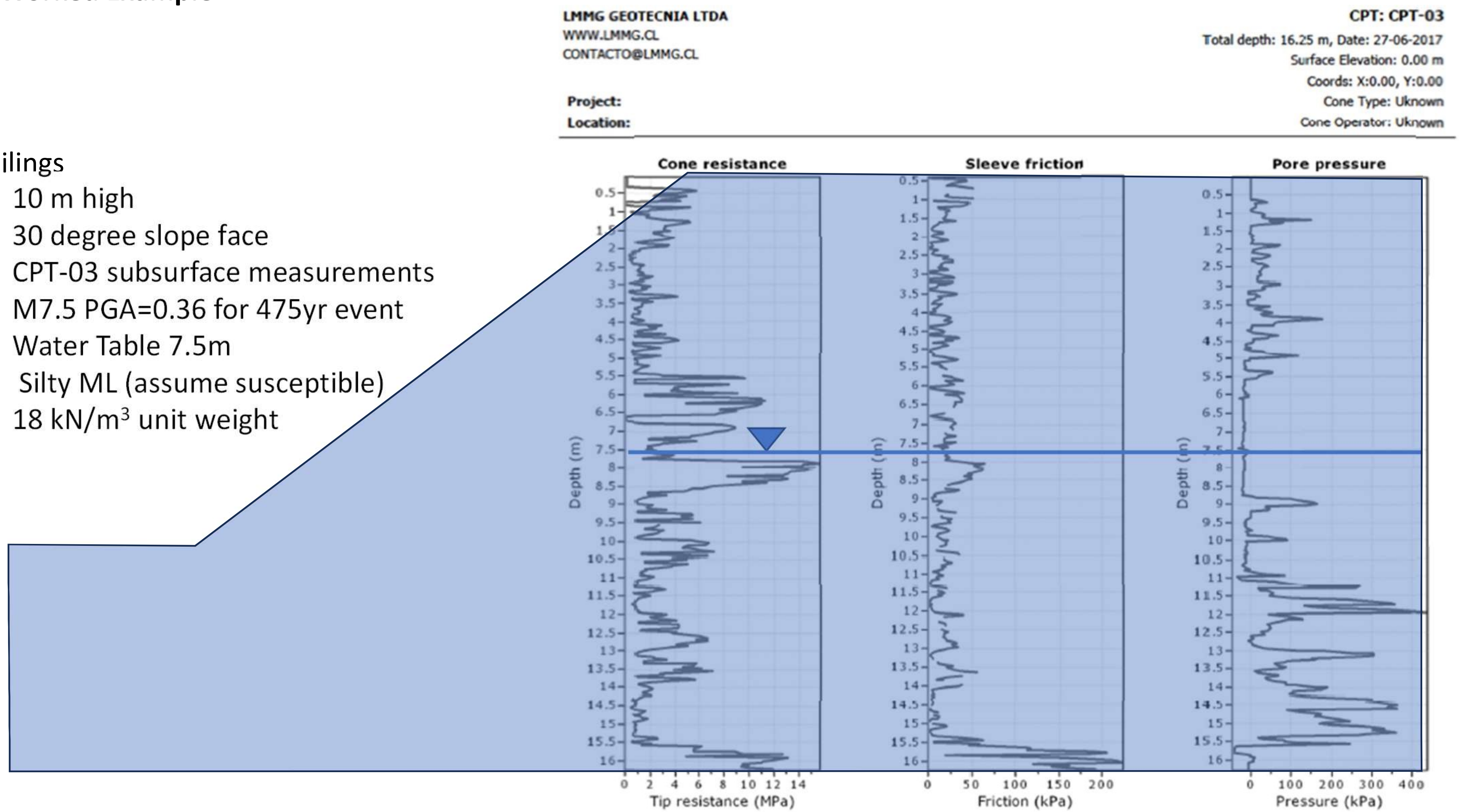
YES/NO



Yazdi & Moss, 2016

In-Class Worked Example

Tailings
- 10 m high
- 30 degree slope face
- CPT-03 subsurface measurements
- M7.5 PGA=0.36 for 475yr event
- Water Table 7.5m
- Silty ML (assume susceptible)
- 18 kN/m³ unit weight



$$CSR = \frac{\tau_{liq}}{\sigma_v'} = 0.65\, a_{max} \frac{\sigma_v}{\sigma_v'} r_d$$

Solution: hand-calcs and/or LiqIT

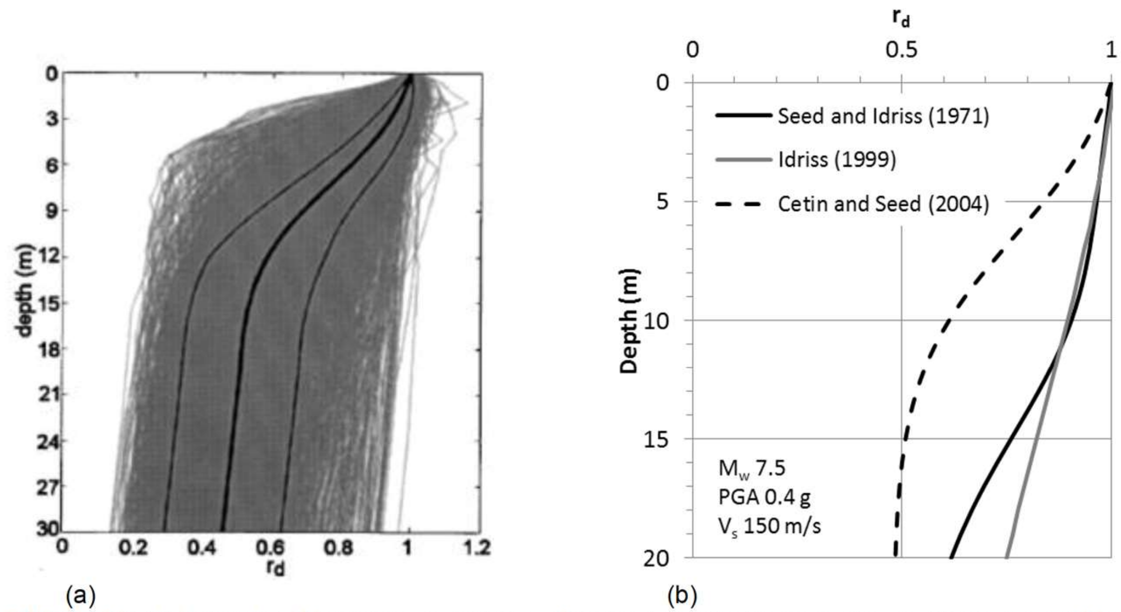
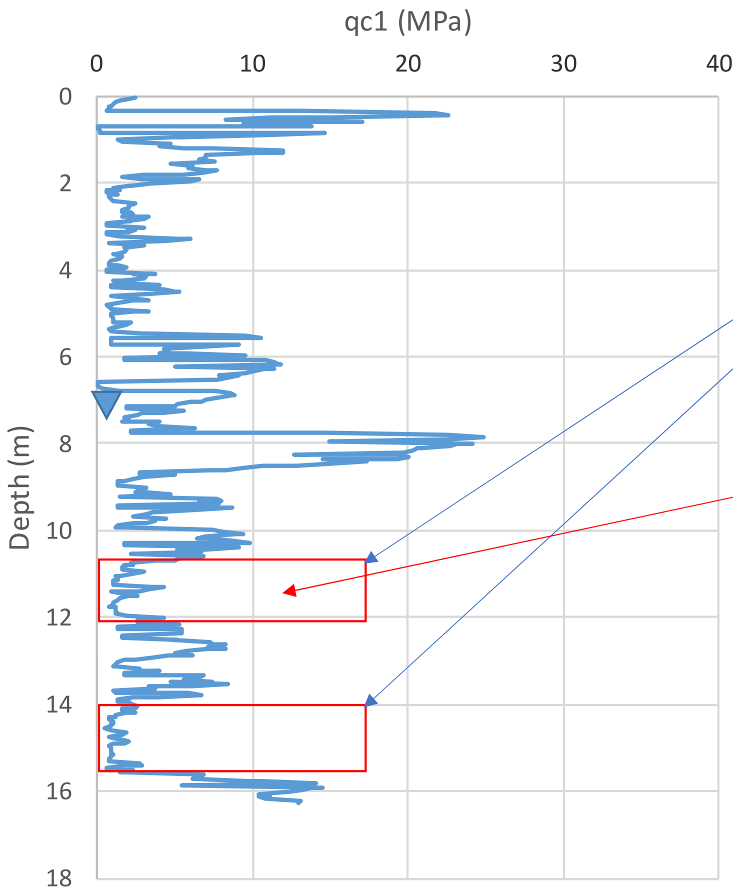


FIGURE 4.2 (a) Computed shear stress reduction coefficients (r_d) for a range of site conditions and input motions. The solid curves indicate the mean and standard deviations in calculated values of r_d for different site conditions and input motions (gray lines). (b) Proposed r_d relationships from different researchers. SOURCE: (a) Cetin, K.O., and R.B. Seed. 2004. Nonlinear shear mass participation factor (rd) for cyclic shear stress ratio evaluation. *Soil Dynamics and Earthquake Engineering* 24(2):103–113. With permission from Elsevier. (b) Courtesy of E. Rathje.



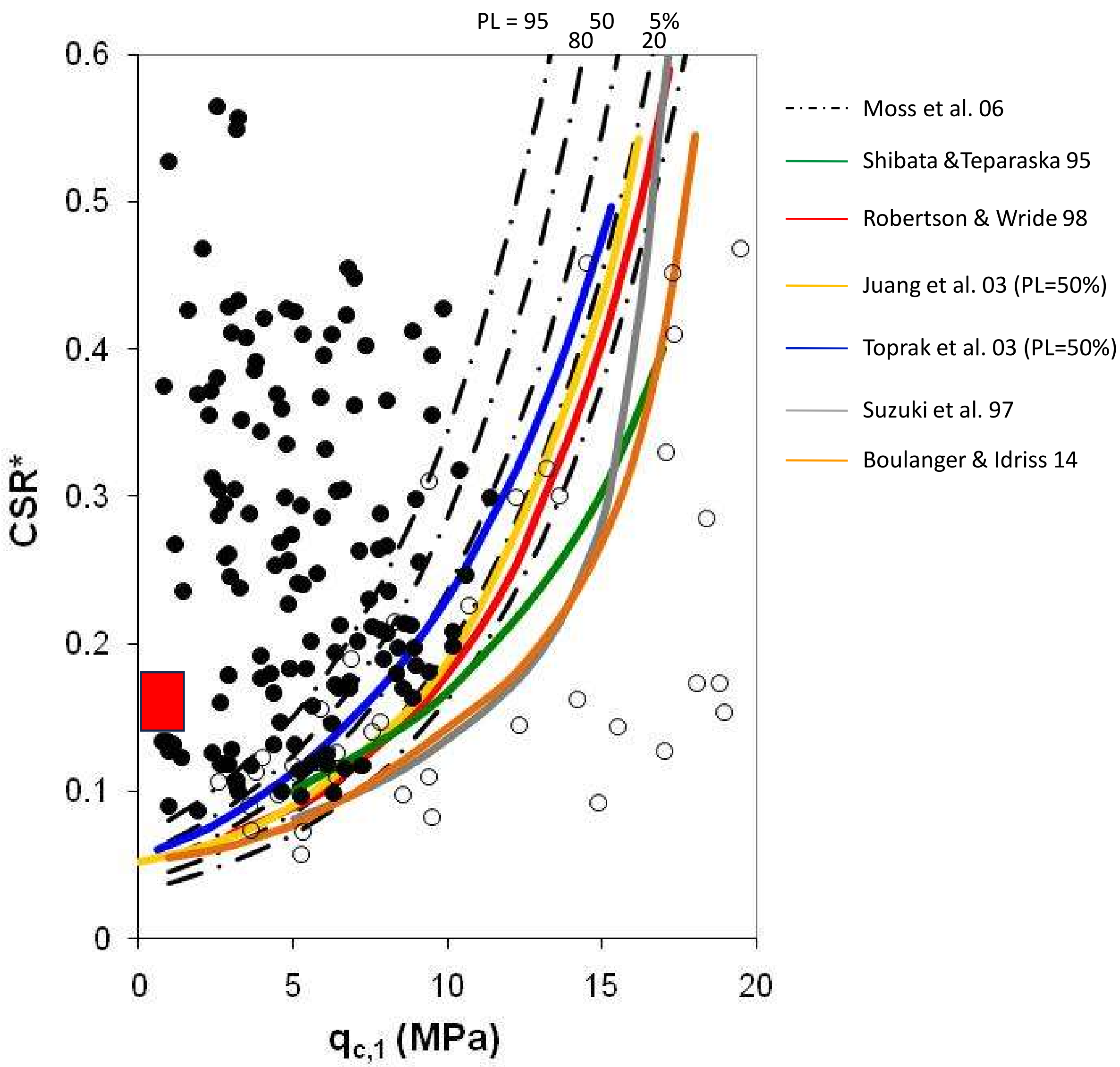
Critical Layer = continuous stretch (>0.5m) of low tip resistance and low friction ratio below the water table.

11 to 12 m layer

qc	qc1
1.29	1.01

CSR ~ 0.65(1.2)0.36(0.6) = 0.17

Shallow vs Deep vs Both



PL = 95
80
50
20
5%
0.6
0.5
0.4
0.3
0.2
0.1
0
CSR*
0
5
10
15
20
qc,1 (MPa)
Moss et al. 06
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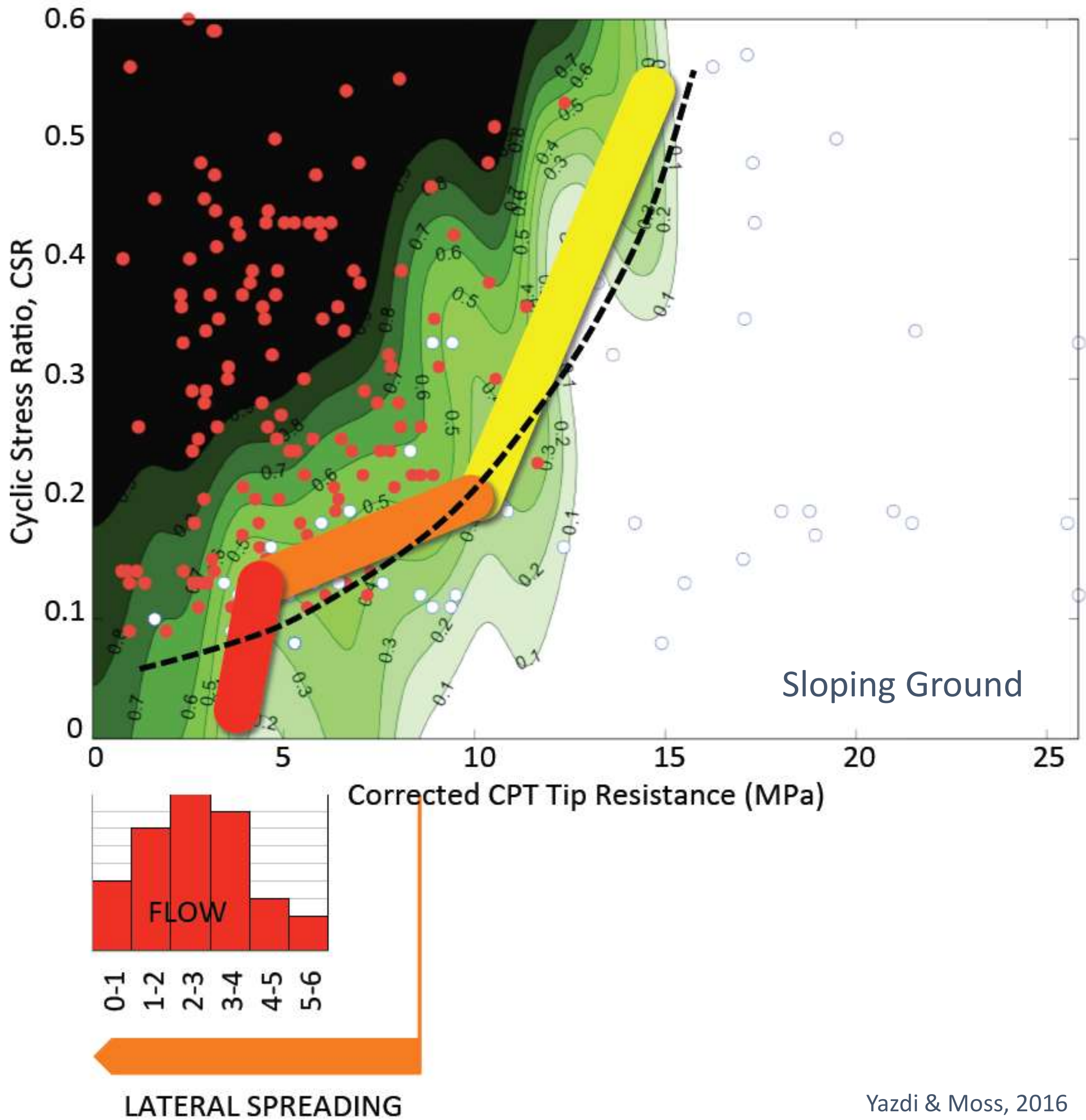

Yazdi & Moss, 2016

Post-Liquefaction Strength and Stability

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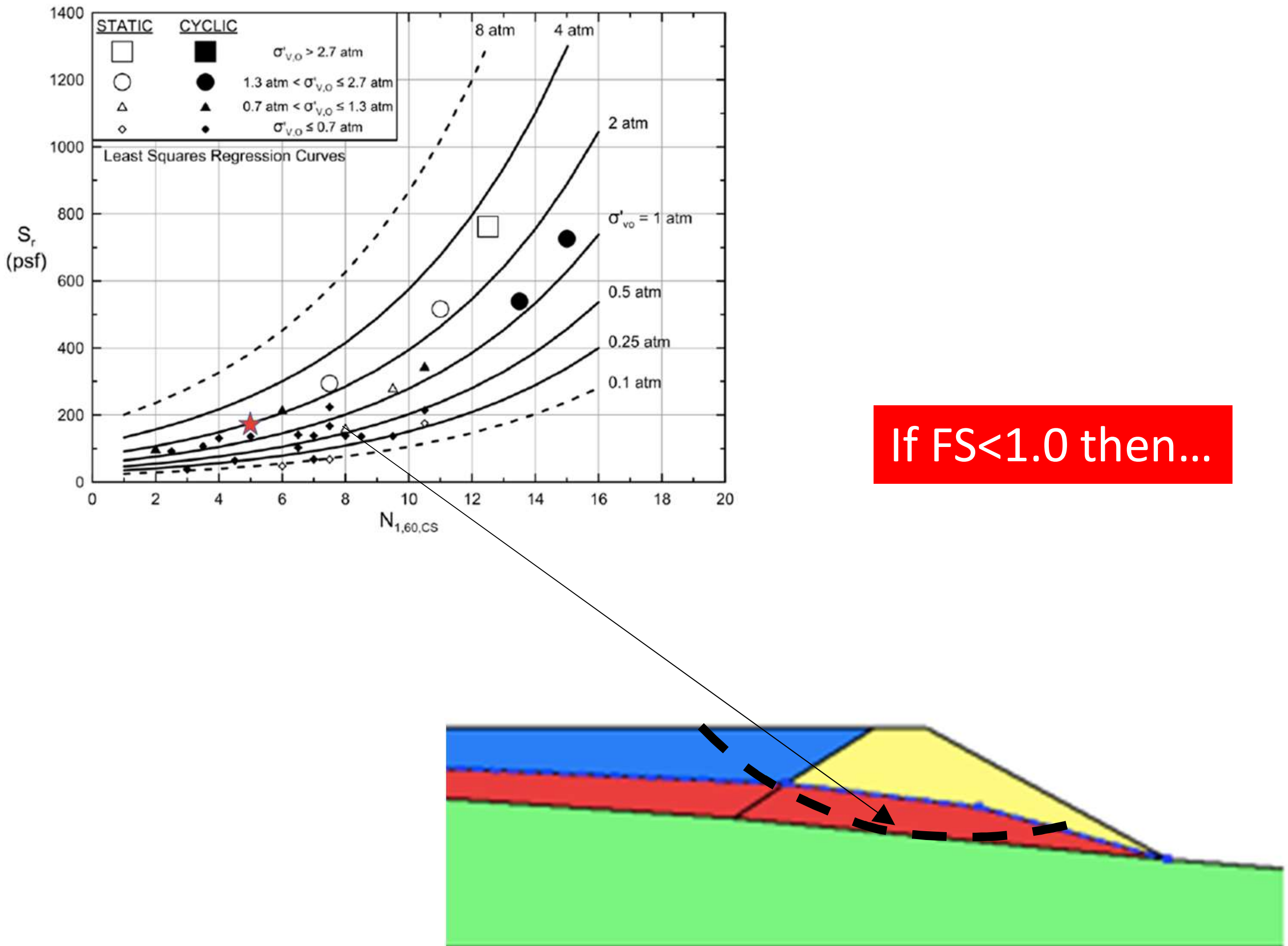
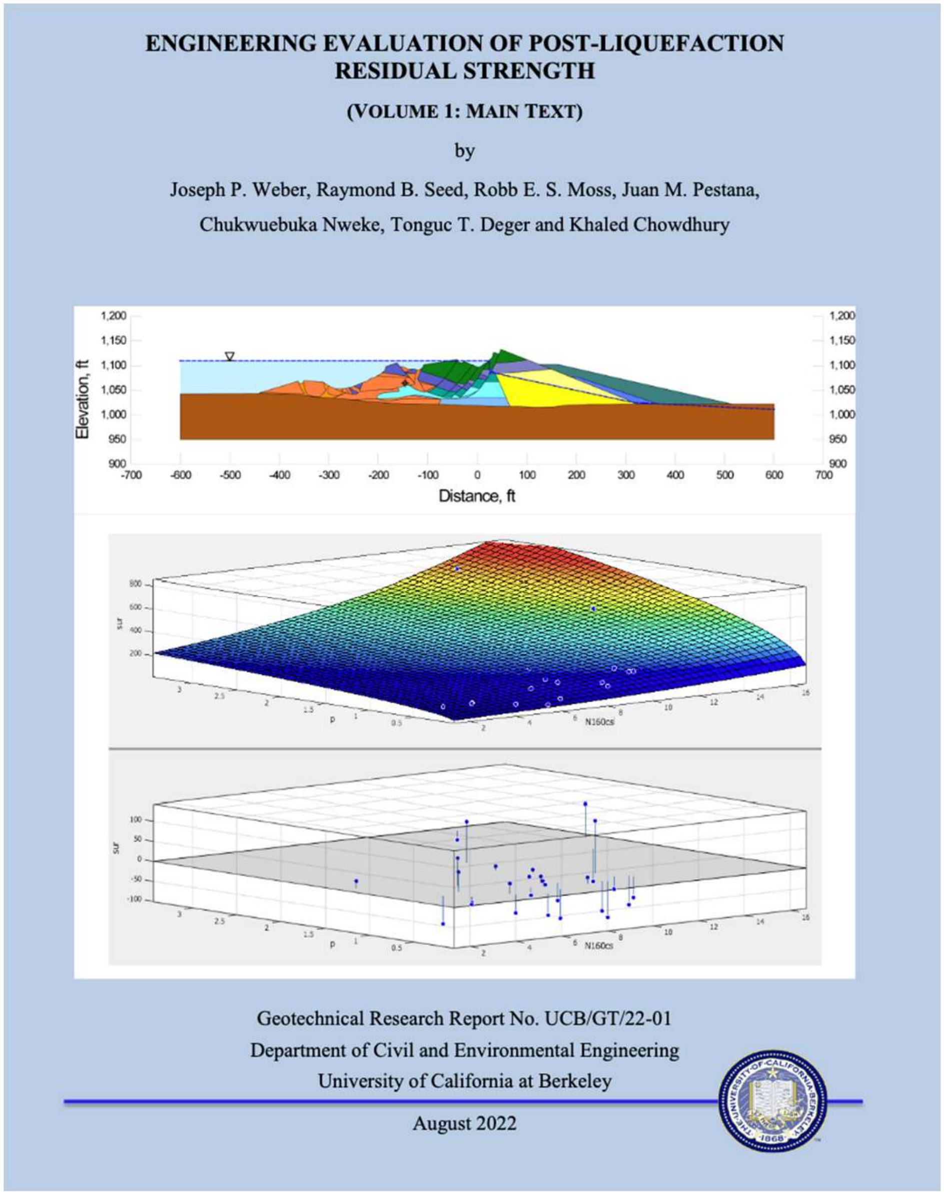

STATIC
CYCLIC
σ'v,o > 2.7 atm
1.3 atm < σ'v,o ≤ 2.7 atm
0.7 atm < σ'v,o ≤ 1.3 atm
σ'v,o ≤ 0.7 atm
Least Squares Regression Curves
8 atm
4 atm
2 atm
σ'vo = 1 atm
0.5 atm
0.25 atm
0.1 atm
Sr (psf)
N1,60,CS
0
200
400
600
800
1000
1200
1400
0
2
4
6
8
10
12
14
16
18
20

If FS<1.0 then…



ENGINEERING EVALUATION OF POST-LIQUEFACTION RESIDUAL STRENGTH

(VOLUME 1: MAIN TEXT)

by

Joseph P. Weber, Raymond B. Seed, Robb E. S. Moss, Juan M. Pestana, Chukwuebuka Nweke, Tonguc T. Deger and Khaled Chowdhury

Geotechnical Research Report No. UCB/GT/22-01
Department of Civil and Environmental Engineering
University of California at Berkeley

August 2022

https://geotechnical.berkeley.edu/sites/default/files/UCB-GT_22-01_Vol1.pdf

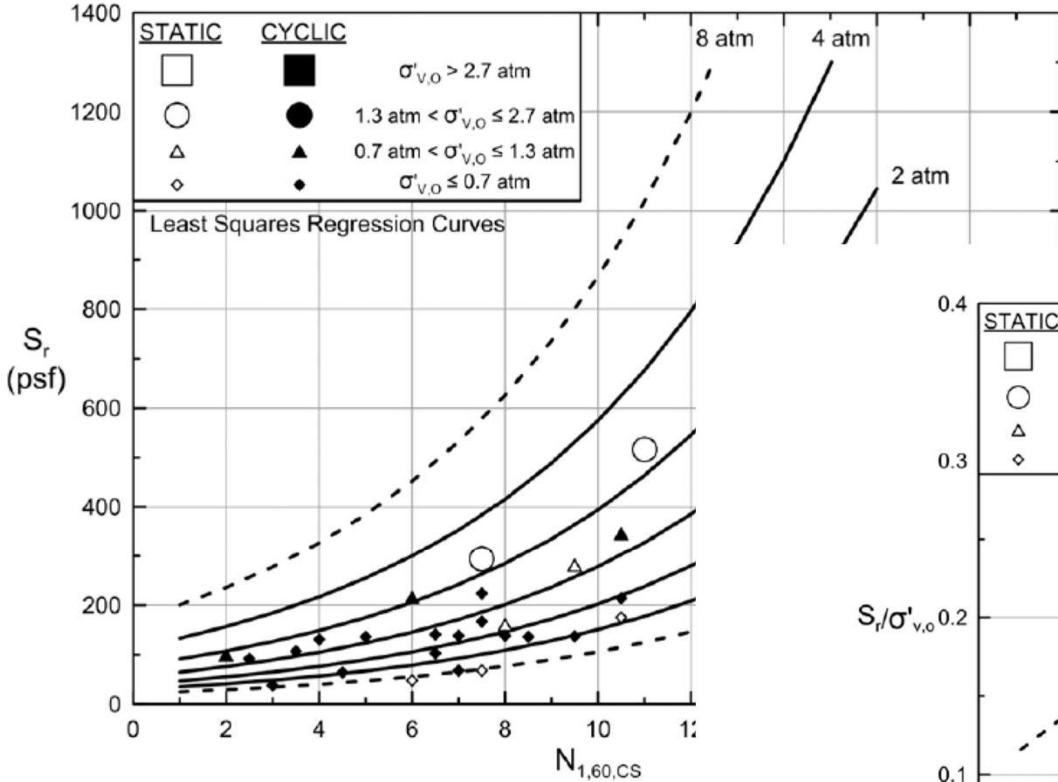


Figure 18: Deterministic regression showing post-liquefaction stren
resistance and initial effective vertical stress (fro

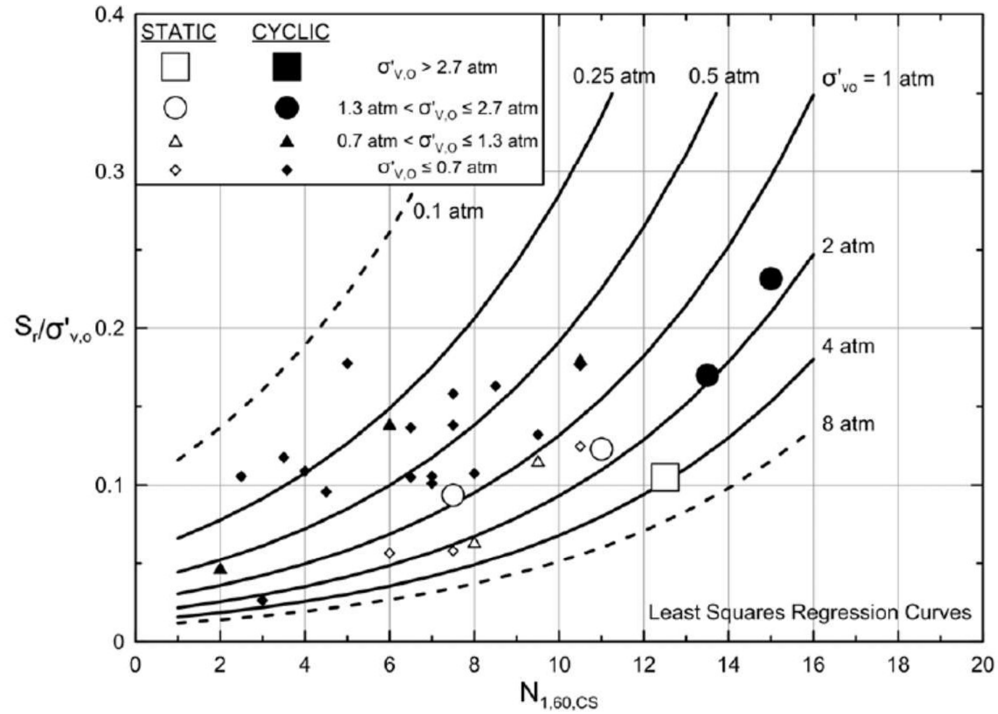
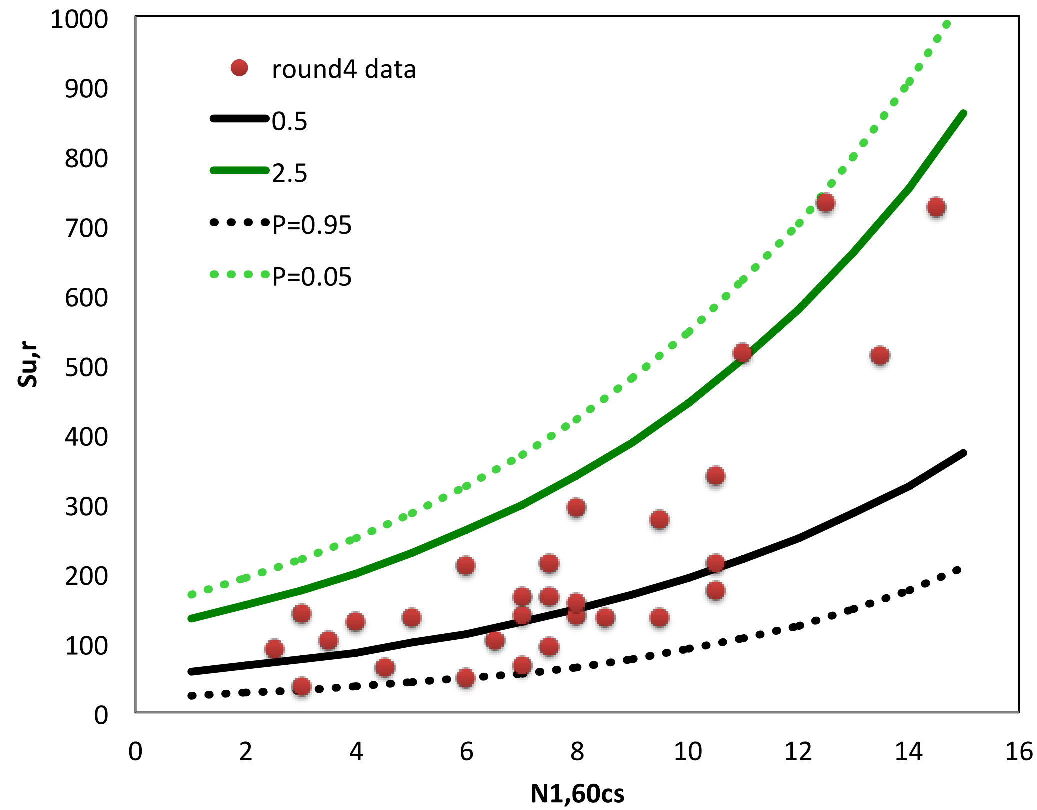
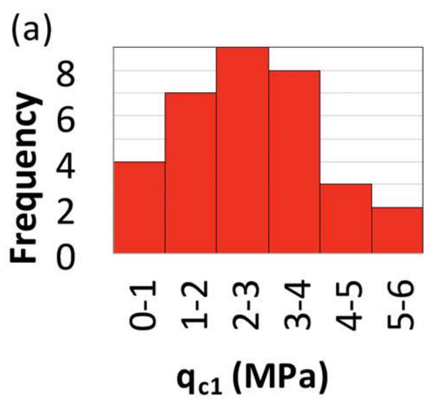


Figure 19: Deterministic regression showing post-liquefaction strength ratio (Sr/P) as a function of both penetration resistance and initial effective vertical stress (from Weber, et al., 2015)

Probabilistic including the parameter uncertainty and modeling uncertainty



Using 5 kPa in subsequent calculations



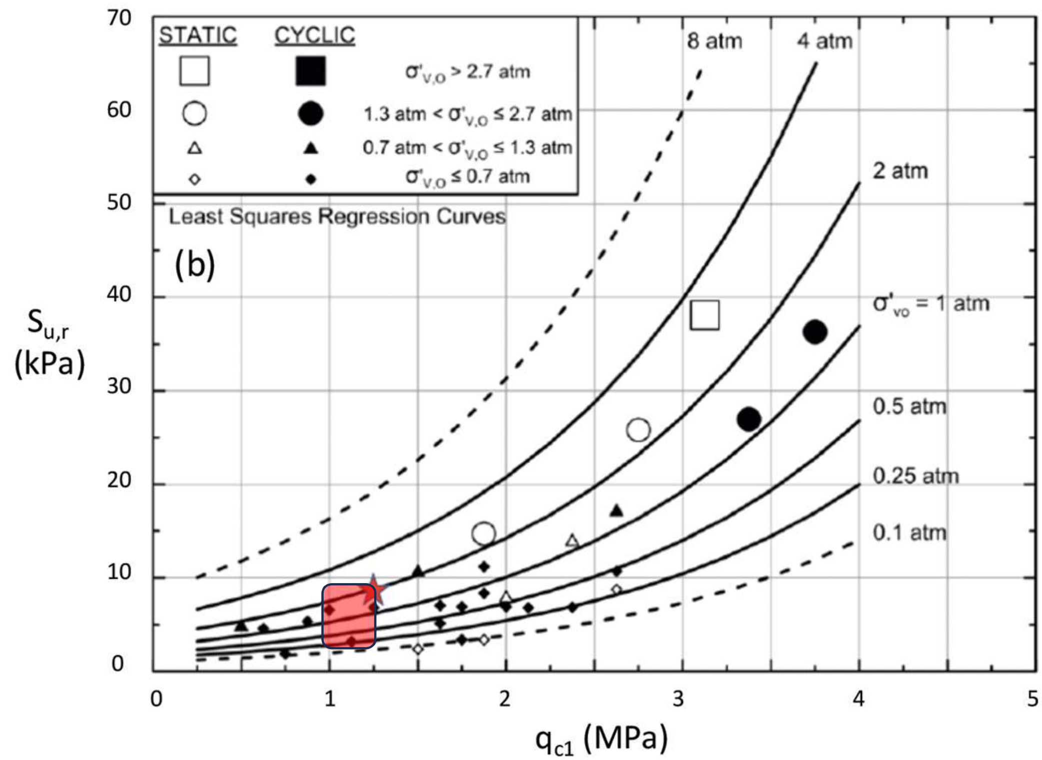
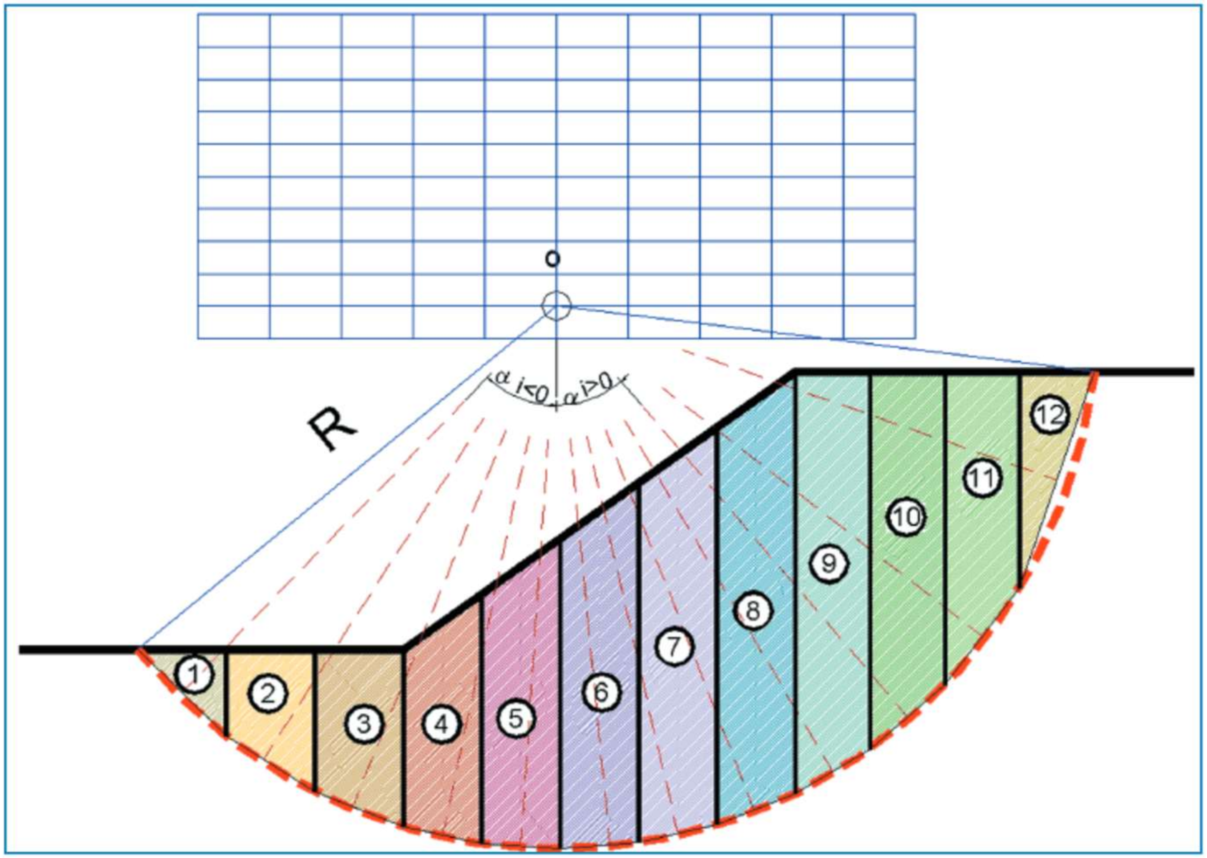
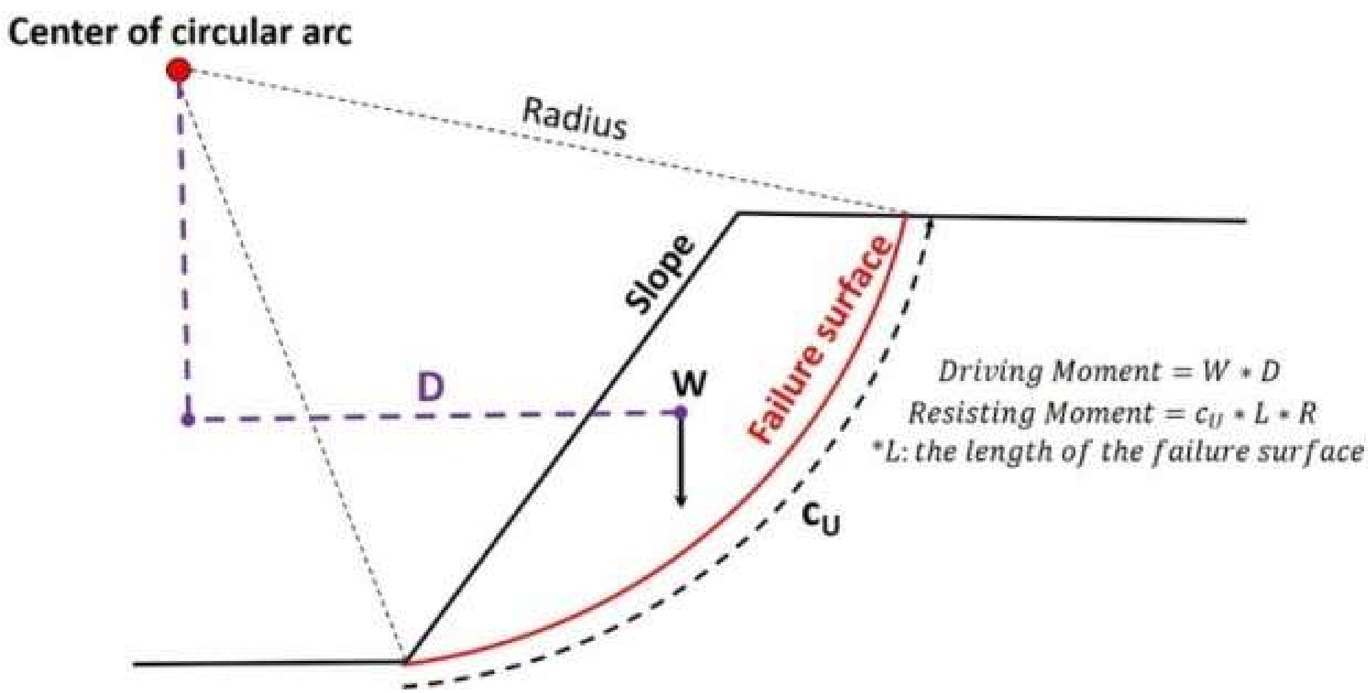


Figure 4. (a) Histogram of cone penetration resistance (q_{c1}) values of flow failure case histories from the Olson & Stark (2002) database (after Yazdi and Moss, 2016). (b) Plot (revised after Weber et al., 2015) correlating penetration resistance to the liquefied residual strength. Red star shows the location of the Las Palmas tailings dam flow failure.

Method of Slices (e.g., using Slide2 from RocScience)



Swedish Circle



Culmann Planar

$$FS = \frac{N + cos\theta tan\phi}{sin\theta}$$

$$N = \frac{2c \cdot sin\psi}{\gamma H \cdot sin(\psi - \theta)}$$

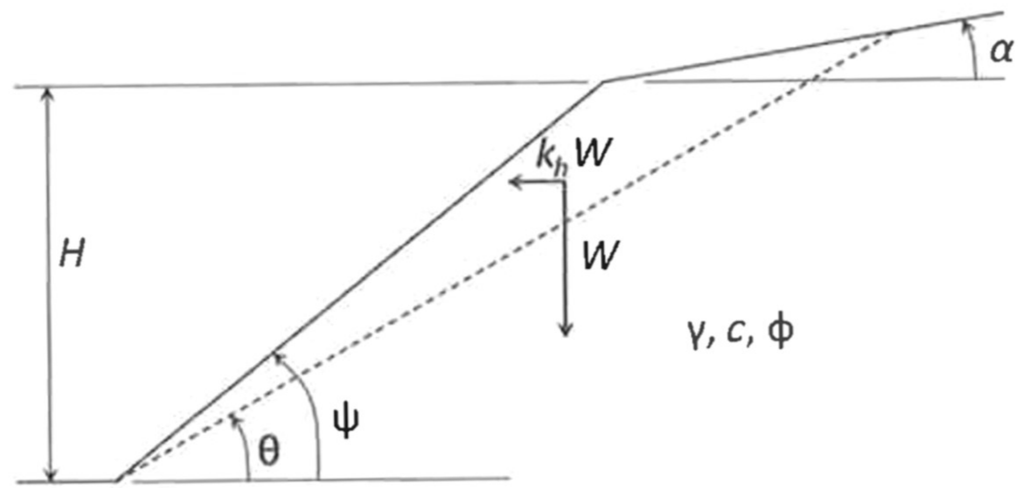


Fig. 1. Culmann single plane failure mechanism notation

$k_y = k_{h,crit} = seismic\ coefficient$
$FS = static\ factor\ of\ safety$
$\phi = friction\ angle$
$\theta = angle\ of\ failure\ plane\ from\ horizontal$
$N = stability\ number$
$c = cohesion$
$\psi = angle\ of\ slope\ face\ from\ horizontal$
$H = height\ of\ slope$
$\gamma = unit\ weight\ of\ the\ soil$

$$k_y = k_{h,crit} = \frac{FS - 1}{tan\phi + 1/tan\theta}$$

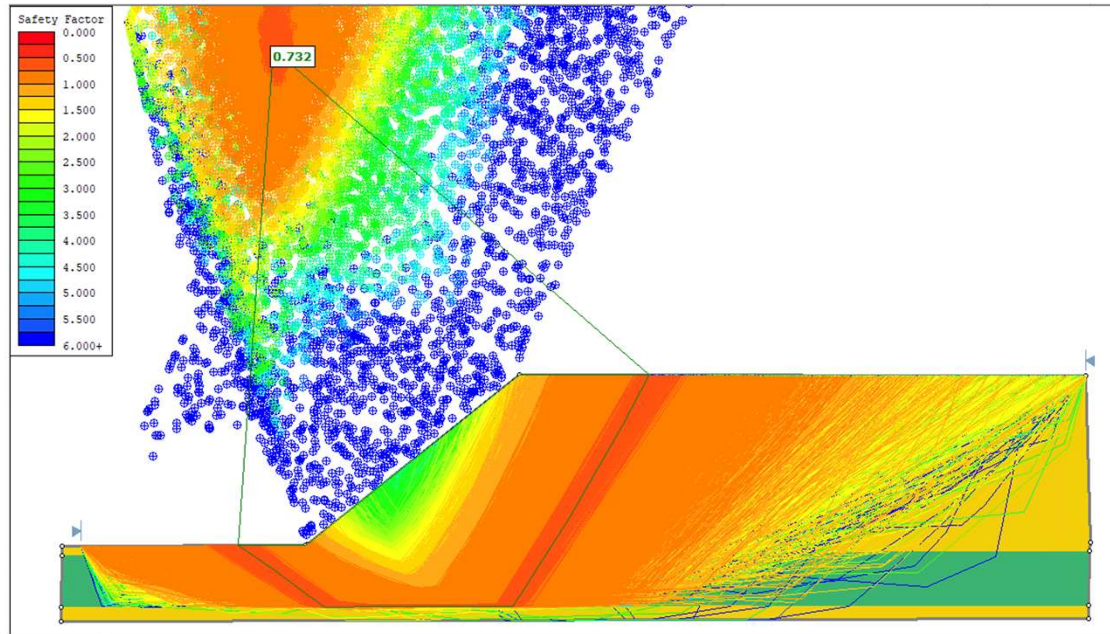
from Christian & Urzua (2017)

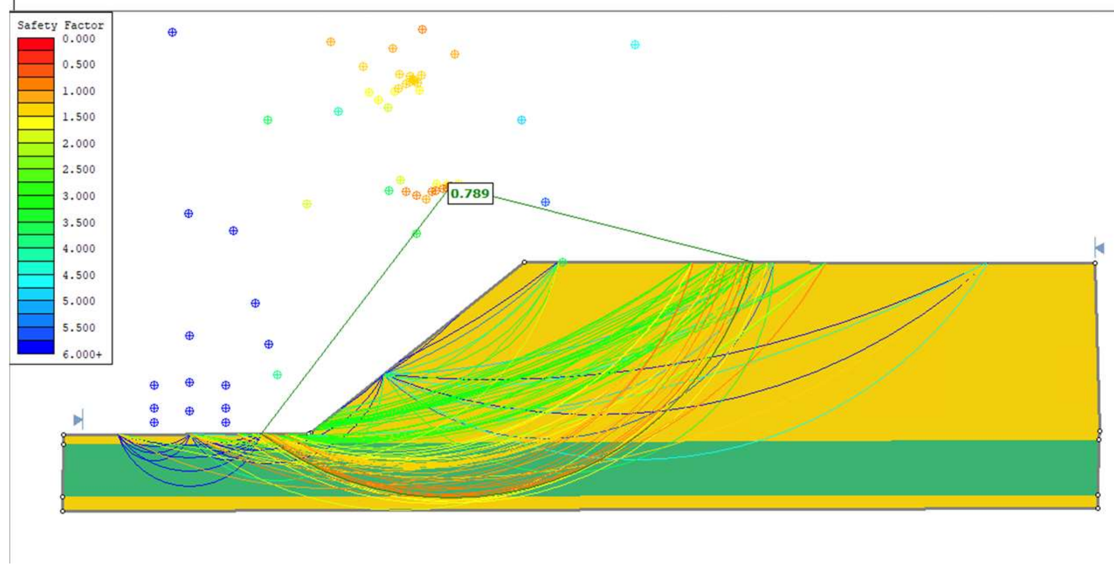
Post-Liquefaction Stability Results

Swedish Circle ~ 0.6-ish

Culman Planar ~ 0.5-ish

MOS ~ 0.73 (non-circular) to 0.78





FS<1.0 then:
- est. displacements
- est. consequences
- implement mitigations

Deformations Analysis

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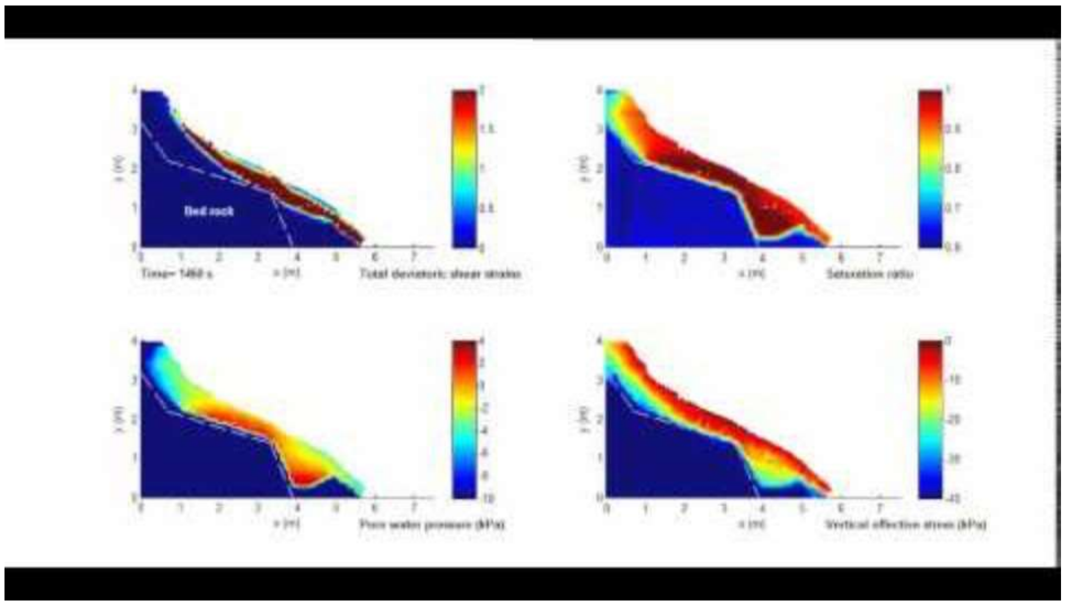
Numerical Modeling for Deformations?

FE/FD, DEM, MPM



Samila Bandara (http://uk.linkedin.com/pub/samila-ban...)
http://onlinelibrary.wiley.com/doi/10... EPFL (Swiss Federal Institute of Technology Lausanne)

Dam Break

Plastic fluid flow that assumes;

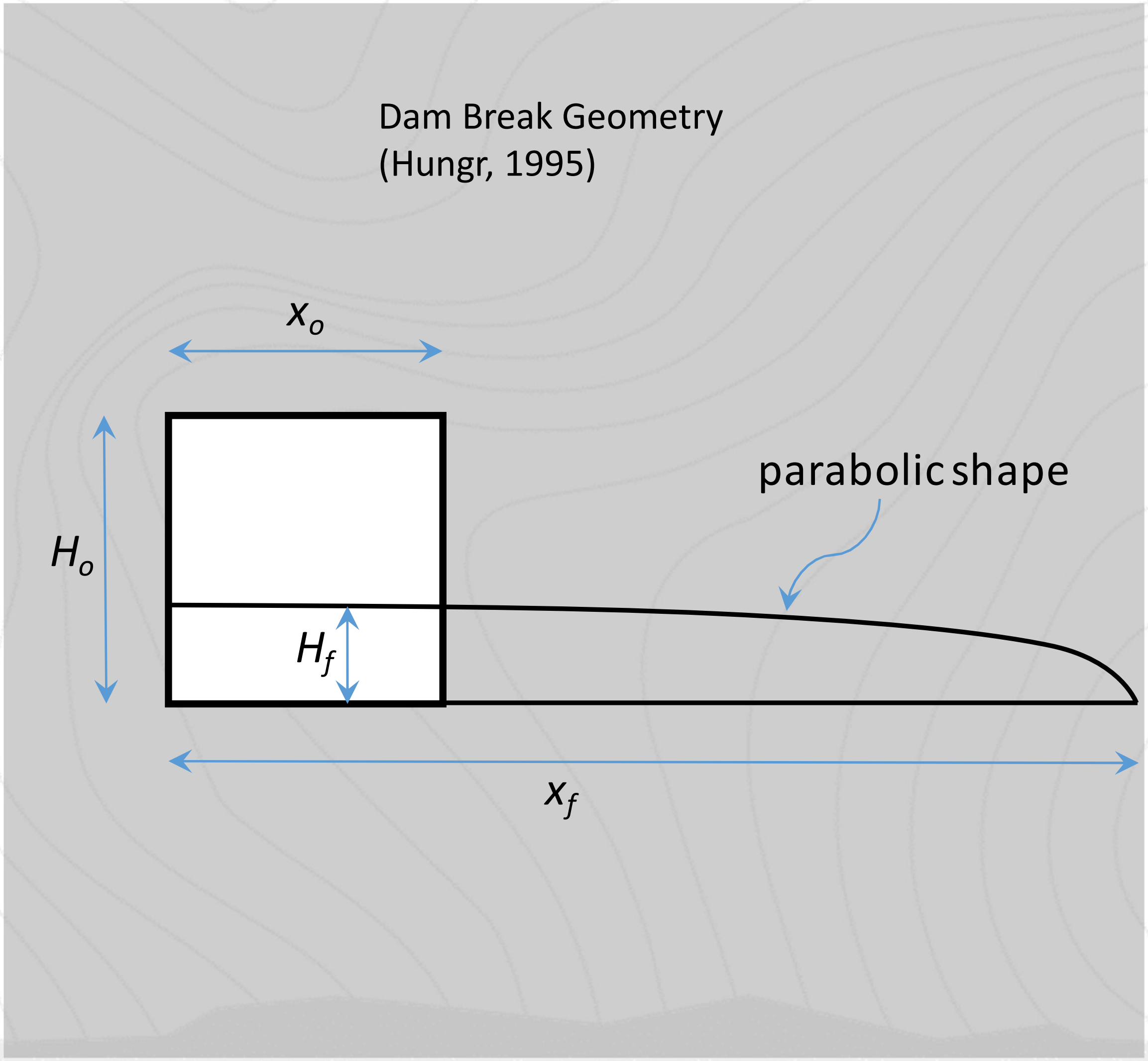
- Conservation of mass, initial to final,
- Translated center of mass, rectangle to parabola,
- Potential energy converted to kinetic energy,
- Work done by shear stress acting on the base,

$$\frac{c}{4}x_f - \left(\frac{c}{4}x_o^2 + \gamma H_o^2 \frac{x_o}{2}\right) x_f + \frac{9}{16}\gamma H_o^2 x_o^2 = 0$$

Runout distance is $x_f - x_o$

Rearranging gives the steady state strength (c)

$$c = 4\left(\frac{\gamma H_o^2 \frac{x_o}{2} x_f - \frac{9}{16}\gamma H_o^2 x_o^2}{x_o^2 x_f - x_f^3}\right)$$



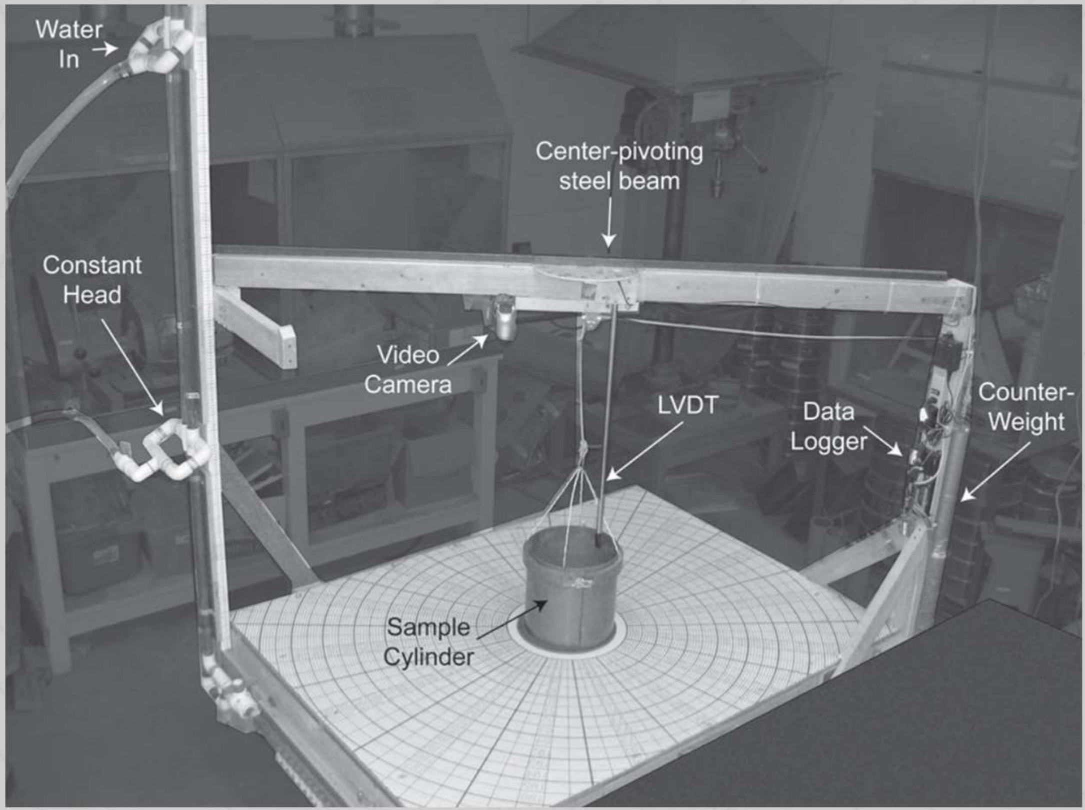
McKenna et al. 2014 **lab testing** experimentally mimics the same geometry.

To tease out which variables are useful, a steady-state strength range of 1.5 to 12.0 kPa was assumed as reasonable target results. This is based on prior studies of steady-state strength in the field (e.g., Weber et al., 2022; Seed and Harder, 1990; Olson and Stark, 2003; Moss et al., 2019) and in the lab (e.g., Dewoolkar et al., 2016; Moss et al., 2020).

What was found is that the following variables show a trend with the predicted steady-state strength:

- fines content was less than approximately 20% (FC<20%),
- water content was less than approximately 200% (w_c<200%),
- Darcy number was less than roughly 5 E+08

Lab Data

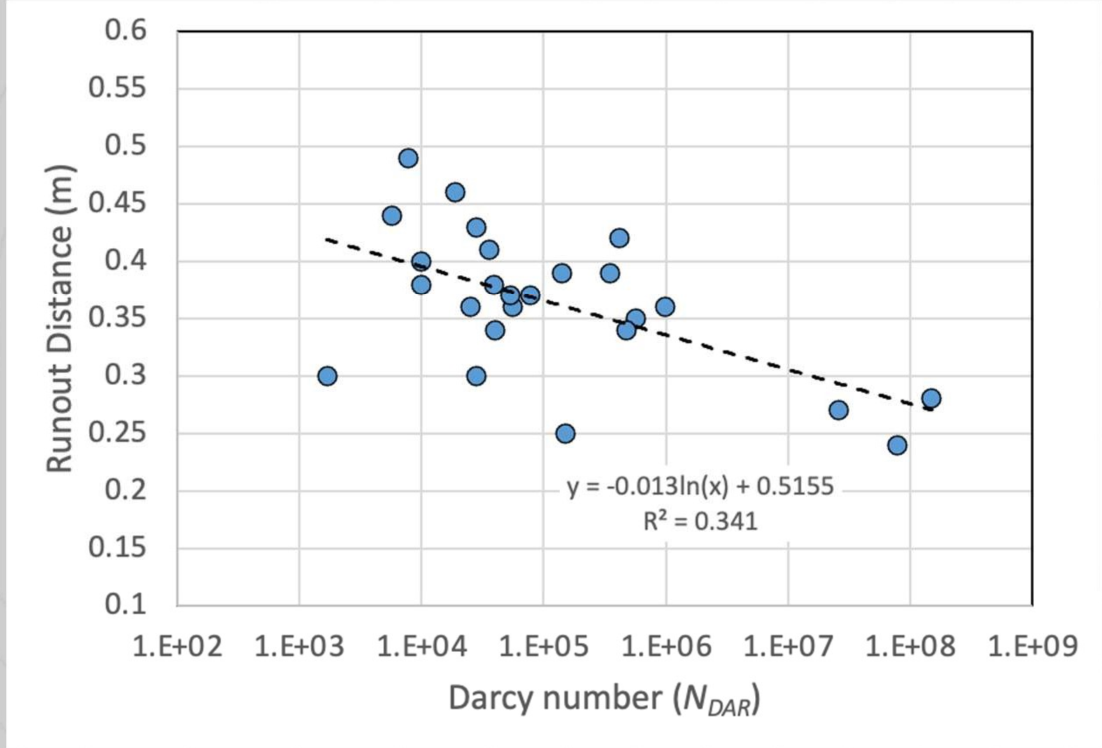


The Darcy Number among all other variables correlated best with runout distances in the lab. The Darcy number is a dimensionless parameter which is the ratio of the solid-fluid interaction stress to the solid inertial stress.

$$N_{DAR} = \frac{\mu}{V_s \rho_s \dot{\gamma} k}$$

$\mu = viscosity (Pa\ s)$,
$V_s = volume\ of\ the\ solids\ (m^3)$
$\rho_s = density\ of\ solids\ (kg/m^3)$
$\dot{\gamma} = shear\ strain\ rate\ (1/s)$
$k = intrinsic\ permeability\ (m/s)$

Lab Data



$$N_{DAR} = \frac{\mu}{V_s \rho_s \dot{\gamma} k}$$

The viscosity of the fluidized soil is a key variable in determining how likely a slope is to achieve flow when triggered. As described in McKenna et al., (2014) it is a function of how much fines are entrained in the fluid during failure.

As the fines are entrained the density of the fluid increases accordingly (Iverson, 1997). A semi-empirical relationship by Thomas (1965) was used in McKenna et al., (2014) to estimate the viscosity of the flowing material.

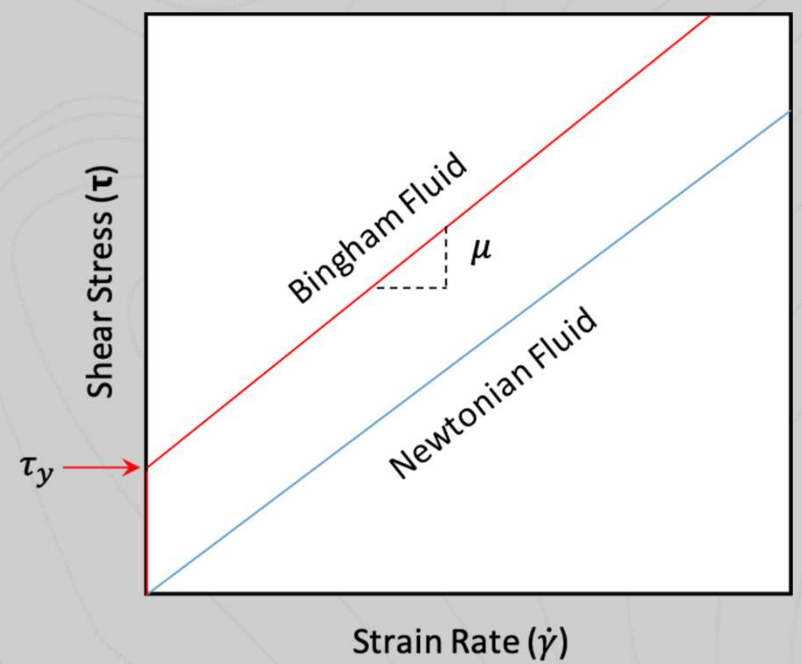
We next examined field data to determine reasonable viscosities at full scale from failure case histories.

Field Data

Bryant et al., (1983) studied dam and embankment flow failures to isolate the failure characteristics of the material that resulted in soil fluidization. Flow material was treated as a Bingham plastic with a yield stress/strength and strain rate dependent strength. The yield strength (τ_y) is the intercept for the Bingham fluid at negligible strain rate and identical to the steady-state strength (c) or liquefied/residual strength. The slope of the line (μ) with an increase in strain rate is the viscosity.

Dimensionless Parameters

Material	τ_y/σ'_v	$\mu * \dot{\gamma}/\tau_y$
Banding Sand #6	~0.05	--
Morenci	~0.15	~0.01
Coeur	~0.15	~0.01
Mission	~0.20	~0.006
Lornex	~0.22	--
Bunker Hill	~0.30	~0.01 to 0.02
Star Morning	~0.30	~0.009
Climax	~0.50	~0.002 to 0.008
Lucky Friday	~1.00	~0.006 to 0.015
Galena	~0.001 to 0.25	~0.03 to 0.04



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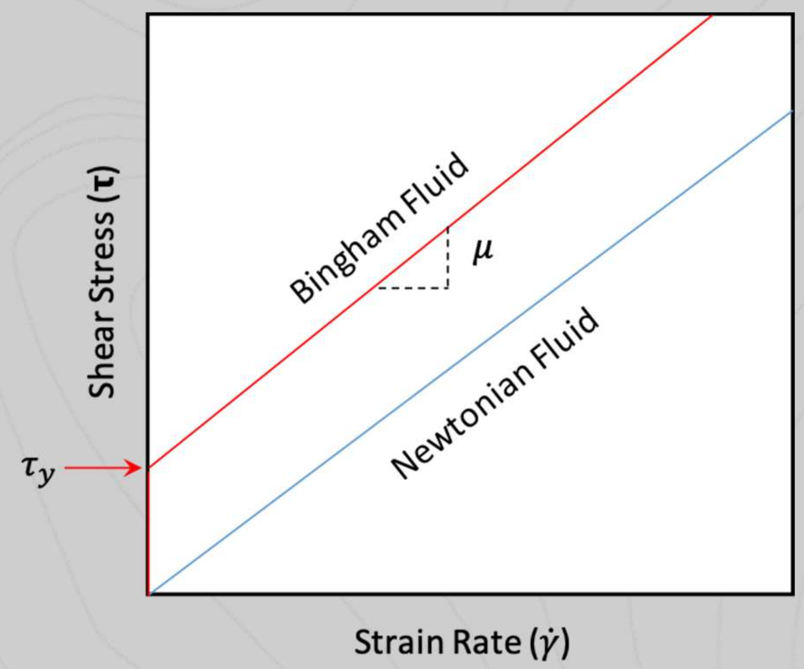
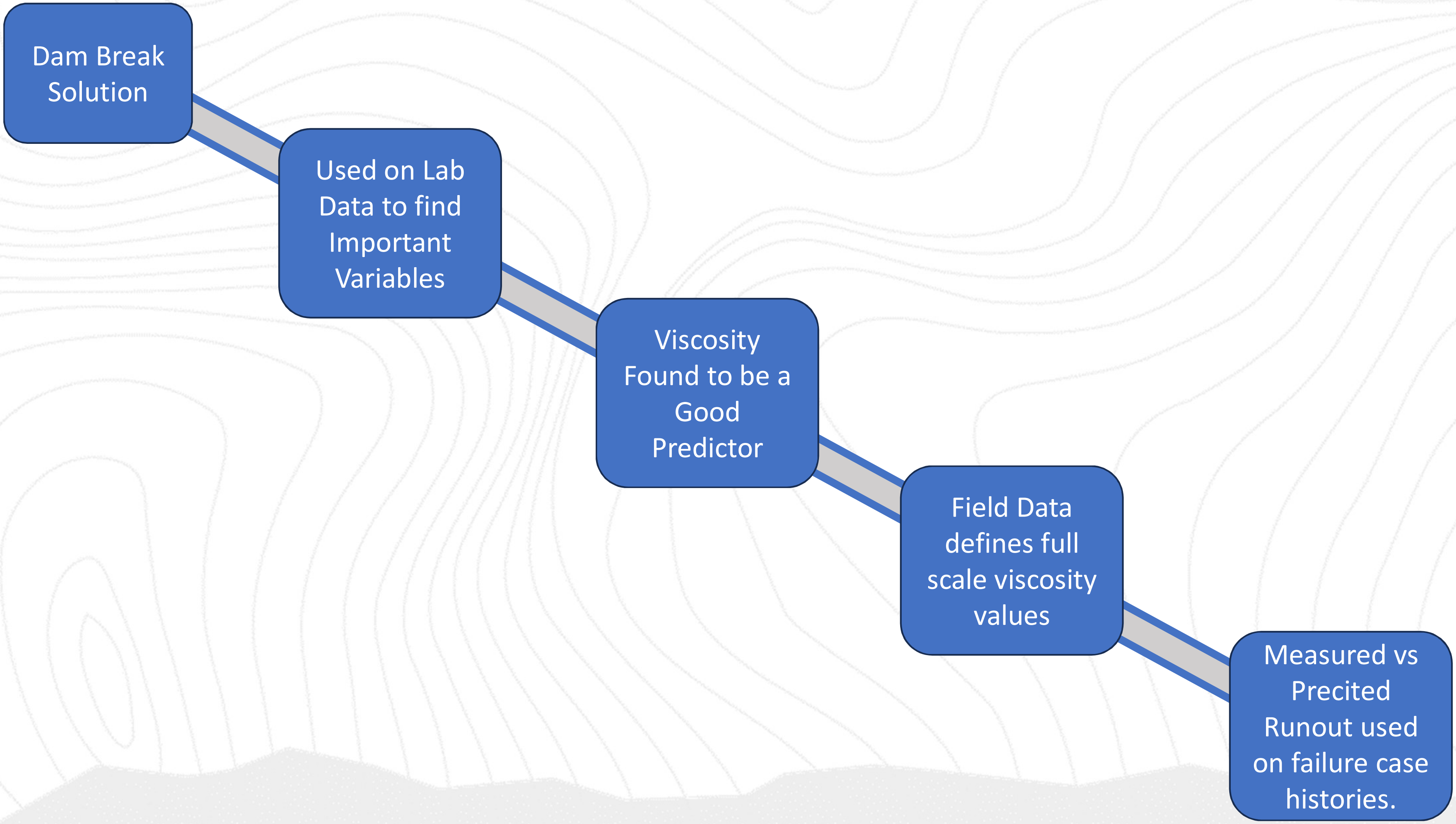
Lucky Friday	~1.00	~0.006 to 0.015
Galena	~0.001 to 0.25	~0.03 to 0.04

Mean dimensionless Viscosity = 0.013 with a CoV = 75% for low confining stress conditions

This is then used in forward modeling an independent set of embankment/tailings failures.

Field Data





Dam Break Solution
Used on Lab Data to find Important Variables
Viscosity Found to be a Good Predictor
Field Data defines full scale viscosity values
Measured vs Precited Runout used on failure case histories.

Dam Break Results

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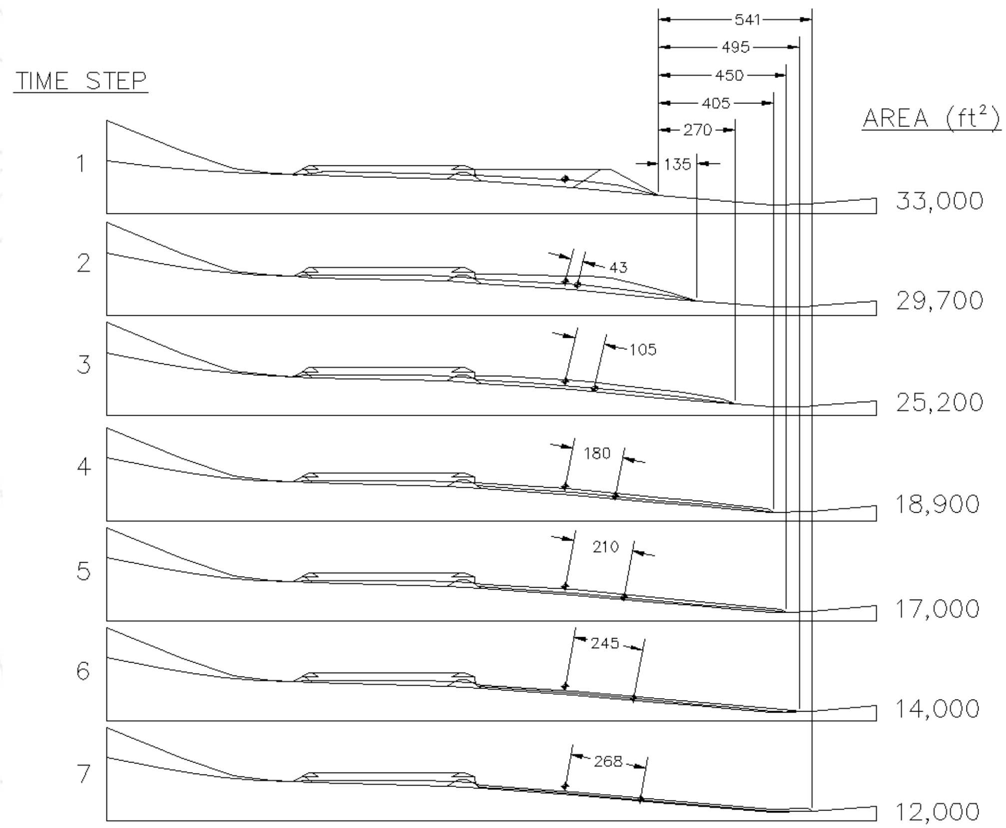
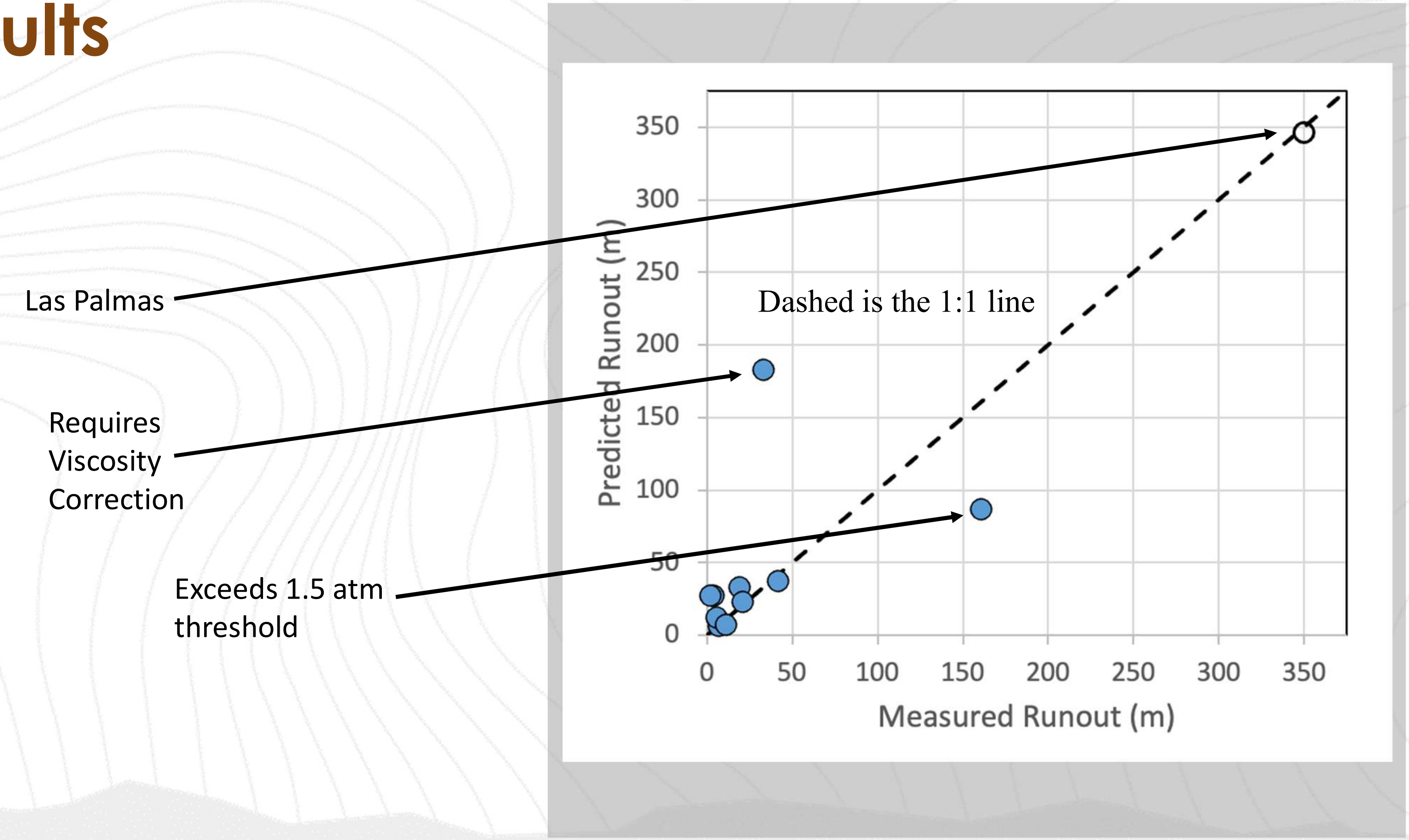
Flow Failure Case Histories after [1]Weber et al. (2022) and [2]Moss et al. (2019).

Case History	H_o (m)	x_o (m)	Slope (degree)	γ (kN/m^3)	σ (atm)	Runout (m)	$s_{u,r}$ (kPa)
Waschusett Dam[1]	26.5	215	26.6	19.3	1.6	42	14.1
Fort Peck Dam[1]	60	480	14.0	19.2	2.7	161	34.8
Uetsu Railway Embankment[1]	9.5	28	21.8	18.5	0.8	33	1.8
Lower San Fernando Dam[1]	43	215	18.4	19.3	1.8	19	24.7
Hachiro-Gata Roadway Embankment[1]	4	20	21.8	19.2	1.2	4	3.3
Lake Ackerman Highway Embankment[1]	8	39	26.6	19.3	0.7	7	5.1
Chonan Middle School[1]	6	30	33.7	18.9	2.1	2	6.9
Soviet Tajik May 1 Slope[1]	30	148	16.7	18.5	2.7	21	16.4
Shibecha-Cho Embankment[1]	10	51	28.1	14.9	1.6	5	10.3
Route 272 Roadway Embankment[1]	8	40	30.8	17.0	1.6	11	6.6
Las Palmas Tailings Dam[2]	25	150		15		350	8.3

Residual Strength Back Analysis using the Incremental Momentum Method (Weber et al., 2015)



Results

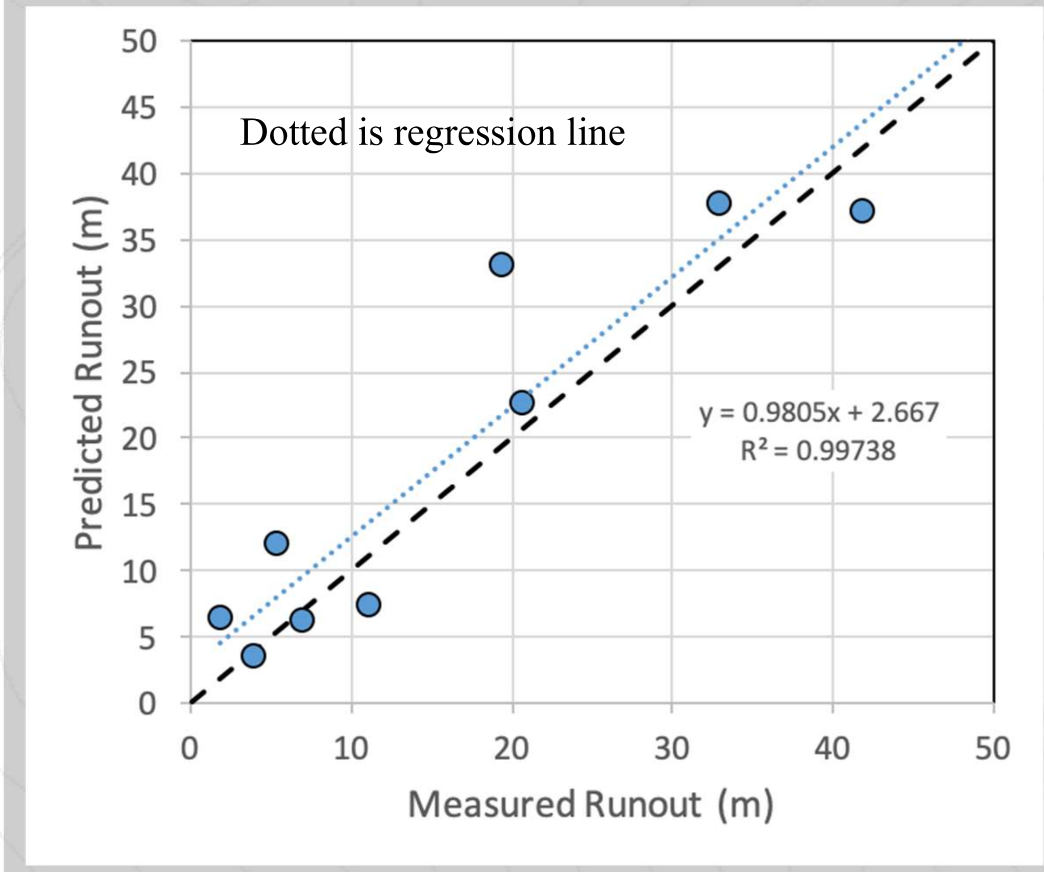


Las Palmas
Requires Viscosity Correction
Exceeds 1.5 atm threshold
Dashed is the 1:1 line
Predicted Runout (m)
Measured Runout (m)
0
50
100
150
200
250
300
350

Results

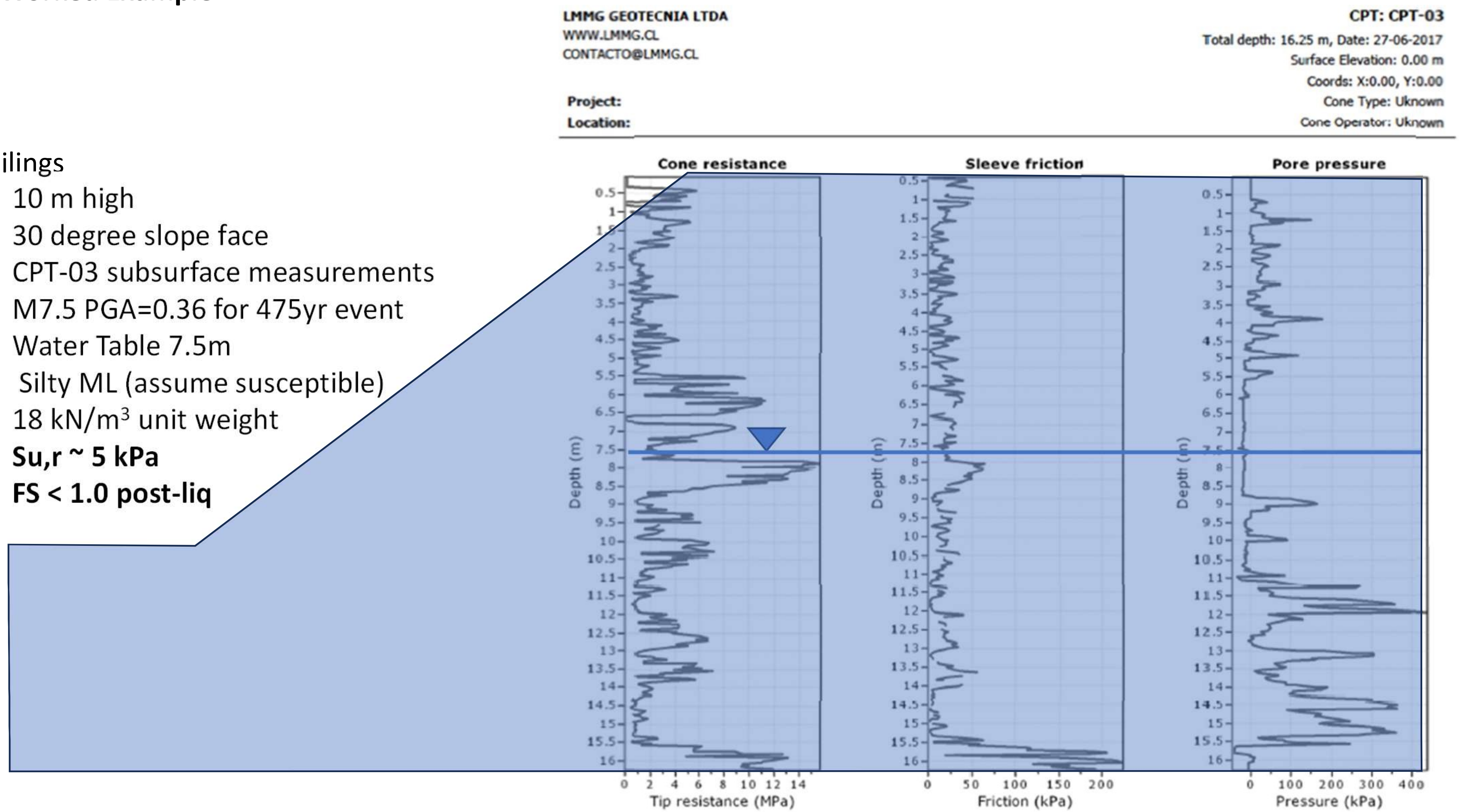
The results show that there is promise for this simple method to give reasonable runout estimates. Although we only have eleven well documented flow failures to make this assessment, future failures and tests will be able to contribute to this assessment. Given that the current modeling capacity to capture flow failure runout accurately is quite limited, this provides a calibrated means of assessing runout distances for engineering design and analysis.

Note that sloping ground was not analyzed as a variable within this study, and should be considered in future studies. It is recommended that users perform detailed subsurface investigations to carefully assess the steady state strength using existing relationships (e.g., Weber et al., 2022) and limit the application of this solution to conditions where the overburden stress is less than 1.5 atm.



In-Class Worked Example

Tailings
- 10 m high
- 30 degree slope face
- CPT-03 subsurface measurements
- M7.5 PGA=0.36 for 475yr event
- Water Table 7.5m
- Silty ML (assume susceptible)
- 18 kN/m^3 unit weight
- **Su,r ~ 5 kPa**
- **FS < 1.0 post-liq**



Hungr 1995	ShortCourse	
Ho (m)	10	
xo (m)	20	set at twice the height
c (kPa)	5	
gamma (kn/m3)	18	
xf (initial)	22.68016541	set at twice the width
solver set to zero	0	initiate solver
xf-xo	2.680165411	order of magnitude estimate

Compare to Shibecha-Cho Embankment Case History

- ✓ 0.38 g in the Kirshiro Oki Earthquake
- ✓ 33.7 ft high (~10.2 m)
- ✓ 28 degree slope
- ✓ correct SPT 8.1 bpf (~2 MPa corrected CPT)
- ✓ back-analyzed Su,r = 215 psf (~10 kPa)
- ✓ < 1 atm effective overburden
- ✓ post-liq FS ~ 0.79
- ✓ **max runout 17.9 ft (5.4 m)**

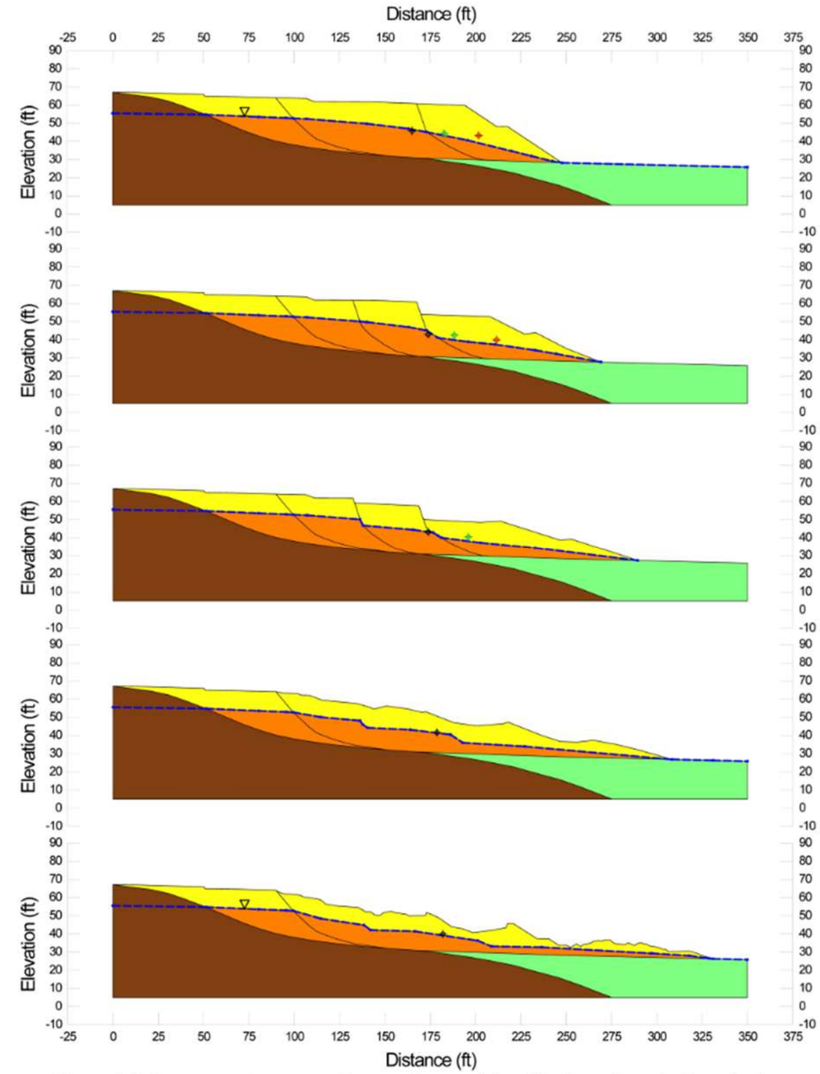
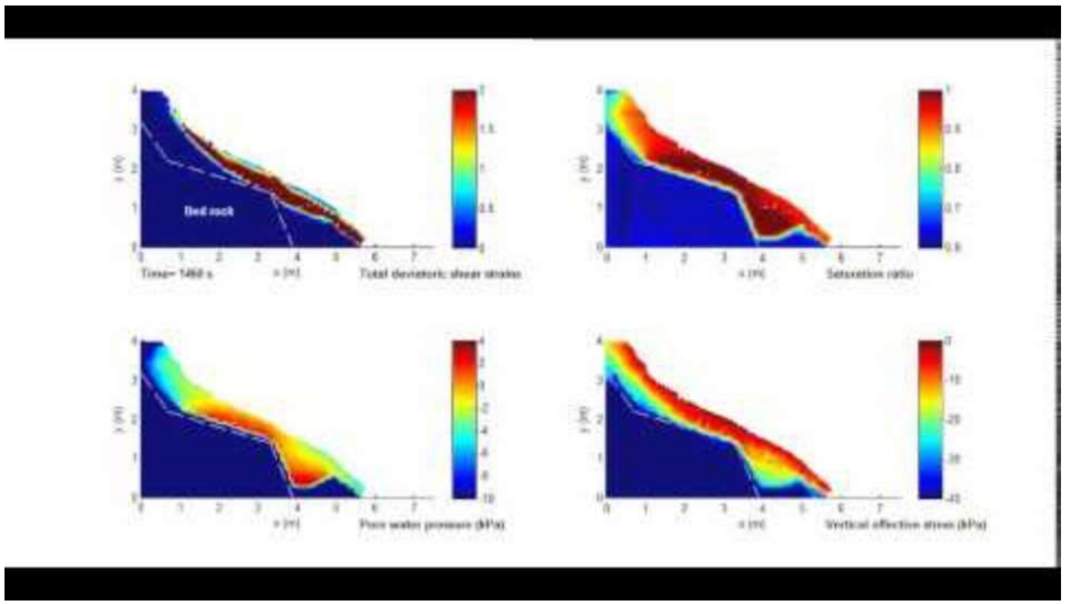


Figure 4.4: Incremental cross-sections used to model and back-analyze the liquefaction-induced failure of the Shibecha-Cho Embankment.

Need a more precise answer? Then calibrated numerical modeling..

<u>FE/FD</u>, DEM, MPM



Samila Bandara (http://uk.linkedin.com/pub/samila-ban...)
http://onlinelibrary.wiley.com/doi/10... EPFL (Swiss Federal Institute of Technology Lausanne)

Thank you!

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Background

Tailings dams and other metastable soil conditions can exhibit flow failure, either due to static or seismic loading.

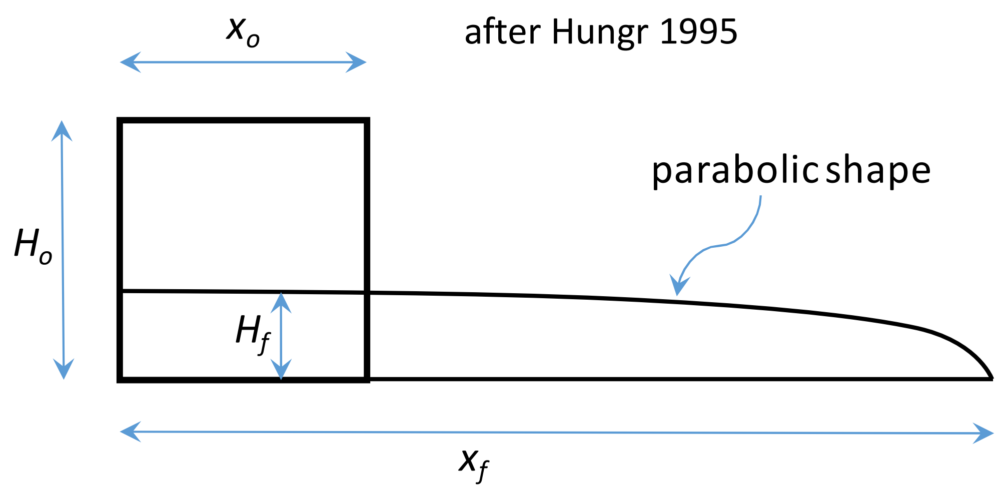
Flow failure, where the soil liquefies and exhibits steady state strength, can result in large deformations on the order of 10's to 100's of meters or more.

In this paper the "dam break" solution is examined with respect to flow failure laboratory experiments conducted by other researchers, and with respect to flow failure field measurements conducted by the author and other researchers.

It is found that after accounting for the strain rate effects on viscosity of the fluidized soil that the "dam break" solution provides reasonable estimates of runout distance, sufficient for engineering design purposes.

Las Palmas 2010

"Dam Break" Estimate for Deformations.



$$\frac{c}{4}x_f - \left(\frac{c}{4}x_o^2 + \gamma H_o^2 \frac{x_o}{2}\right) x_f + \frac{9}{16}\gamma H_o^2 x_o^2 = 0$$

Adjusting for viscosity effects (1 case history) and limiting cases to 1.5 atm (1 case history) the "Dam Break" solution provides a reasonable estimate for the Weber et al. (2015) database where runout was measured.