NATIONAL ACADEMIES Sciences Engineering Medicine

TRB TRANSPORTATION RESEARCH BOARD

TRB Webinar: Innovation in Geoseismic Foundation Design and Performance

May 23, 2023

3:00 – 4:30 PM

NOVEMBER 2022 UPDATE

PDH Certification Information

1.5 Professional Development Hours (PDH) – see follow-up email

You must attend the entire webinar.

Questions? Contact Andie Pitchford at TRBwebinar@nas.edu

The Transportation Research Board has met the standards and requirements of the Registered Continuing Education Program. Credit earned on completion of this program will be reported to RCEP at RCEP.net. A certificate of completion will be issued to each participant. As such, it does not include content that may be deemed or construed to be an approval or endorsement by the RCEP.

ENGINEERING RCEP REGISTERED CONTINUING EDUCATION PROGRAM

Purpose Statement

This webinar will share the latest findings on the seismic design and performance of foundations.

Learning Objectives

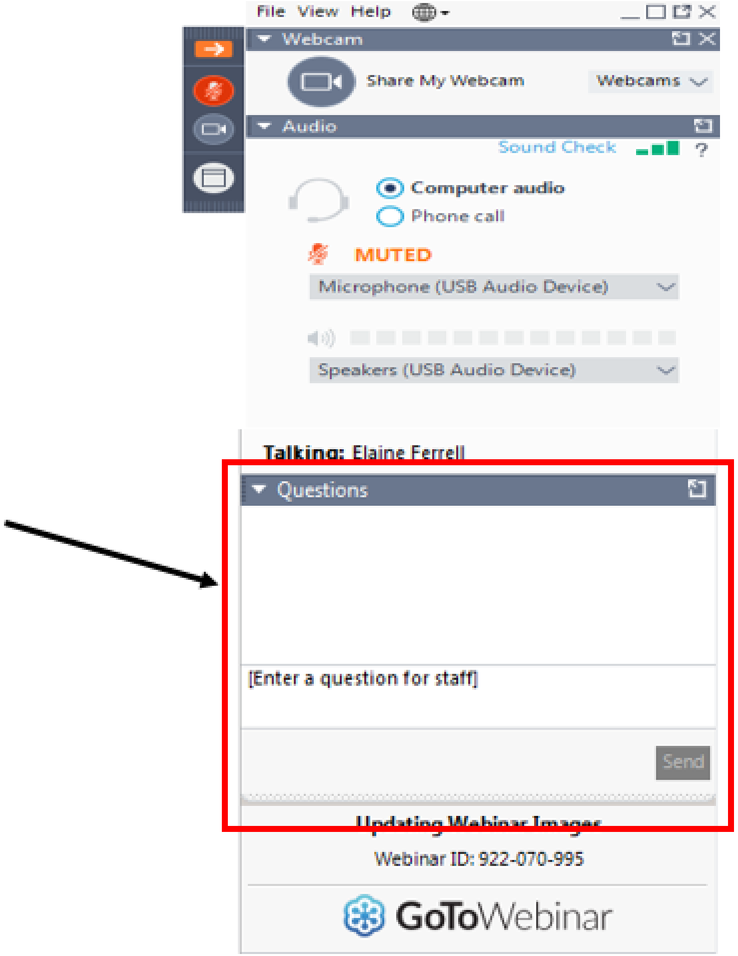
At the end of this webinar, you will be able to:

(1) Assess performance-based liquefaction

(2) Share the latest innovation and findings on geoseismic foundation design and performance

Questions and Answers

- Please type your questions into your webinar control panel
- We will read your questions out loud, and answer as many as time allows



Today's presenters

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Kyle Rollins
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Arash Khosravifar
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SIMPLIFIED PERFORMANCE-BASED LIQUEFACTION HAZARD ANALYSIS

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Dept. of Civil and Construction Engineering

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TRB Webinar

May 23, 2023

Acknowledgments

- Sponsors – Utah, South Carolina, Oregon, Alaska, Montana, Idaho, and Connecticut DOTs, FHWA, and USGS
- Students – Alex Wright, Dr. Kristin Ulmer, Levi Ekstrom, Lucy Astorga Hoch, Brian Peterson, Braden Error, Mikayla Hatch, Tyler Coutu, Alex Arndt, Jenny Blonquist, Heidi Dacayanan, Jingwen He Liang, Clay Fullmer, Sarah Jaen McClellan, Dallin Smith, Lila Lasson, Reed Reimschussell, and Ivy Stout
- Colleagues, Advisers, and Collaborators – Dr. Kyle Rollins, Dr. Steven Kramer, Dr. Les Youd, Dr. Bret Lingwall, Dr. Jorge Meneses, Dr. Scott Olson, David Stevens, Nicholas Harman, Grant Gummow, Darin Sjoblum, Ari Monitoye

Various Approaches for to Represent PGA,M in Liquefaction Hazard Analysis

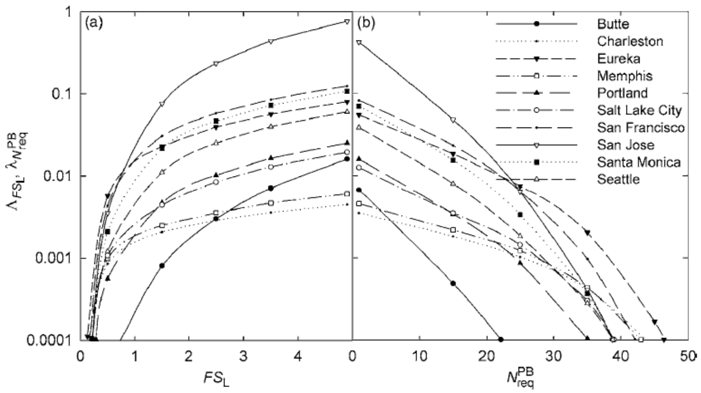
- Deterministic Approach
  - **Considers *an individual seismic source* and corresponding ground motions individually**
  - **Usually assumes mean values for the inputs and models**
- Pseudo-Probabilistic Approach
  - **Considers *probabilistic ground motion* from a single return period**
  - **Usually assumes modal values for the inputs and models**
- Probabilistic (or Performance-Based) Approach
  - **Considers *probabilistic ground motions from ALL return periods***
  - **Accounts for parametric and model uncertainties**
  - **Results depend on desired hazard level or return period**

A Probabilistic Liquefaction Hazard Analysis (PLHA) Approach

- Kramer and Mayfield (2007) introduced a PLHA approach
  - Uses probabilistic ground motions in a probabilistic manner
  - Accounts for uncertainty in seismic loading AND the liquefaction triggering model
  - Produces liquefaction hazard curves for each sublayer in the soil profile

$$\Lambda_{FS_L} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{a_{\max}}} P\left[FS_L < FS_L^* \mid a_{\max_i}, m_j \right] \Delta\lambda_{a_{\max_i}, m_j}$$

$$\lambda_{N_{req}^*} = \sum_{j=1}^{N_M} \sum_{i=1}^{N_{a_{\max}}} P\left[N_{req} < N_{req}^* \mid a_{\max_i}, m_j \right] \Delta\lambda_{a_{\max_i}, m_j}$$



(after Kramer and Mayfield 2007)

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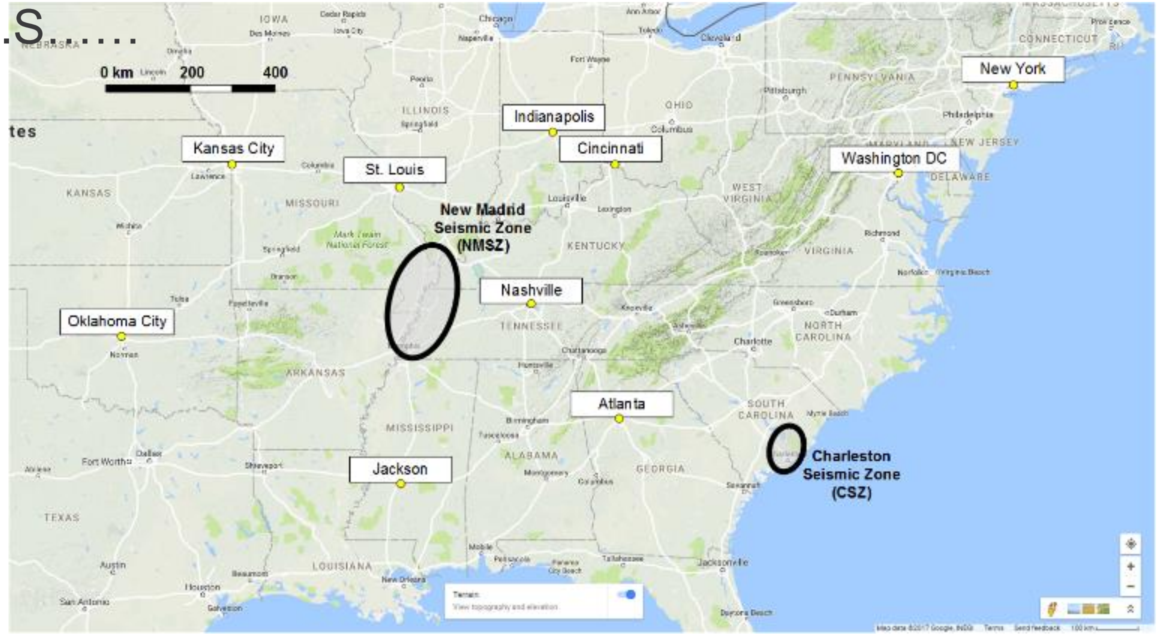
Why This Matters to Regions of Low to Moderate Seismicity

10 cities selected throughout the Central and Eastern U.S.……



(after Franke et al., 2019)

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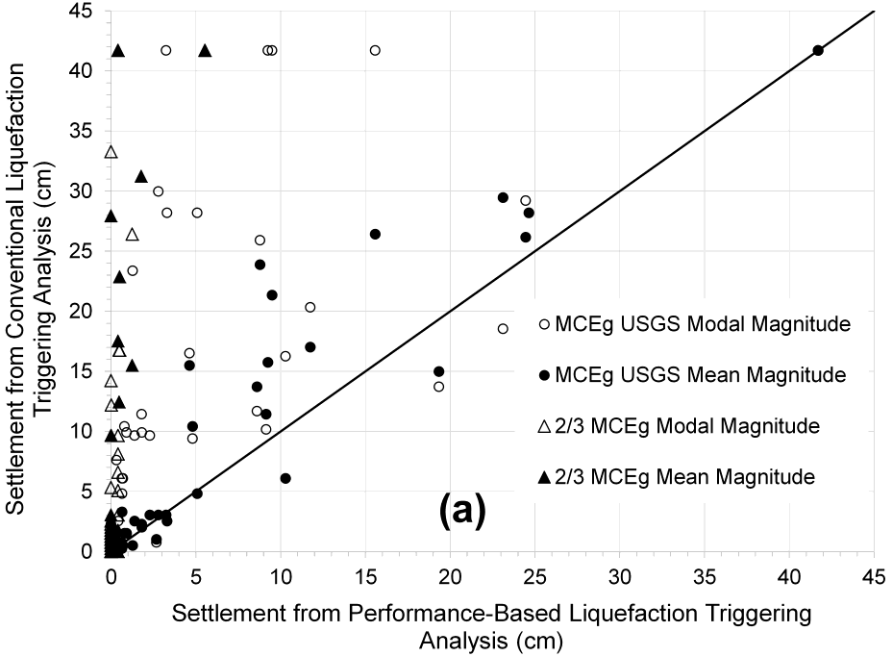
Why This Matters to Regions of Low to Moderate Seismicity

Results if assuming a Site Class D…..



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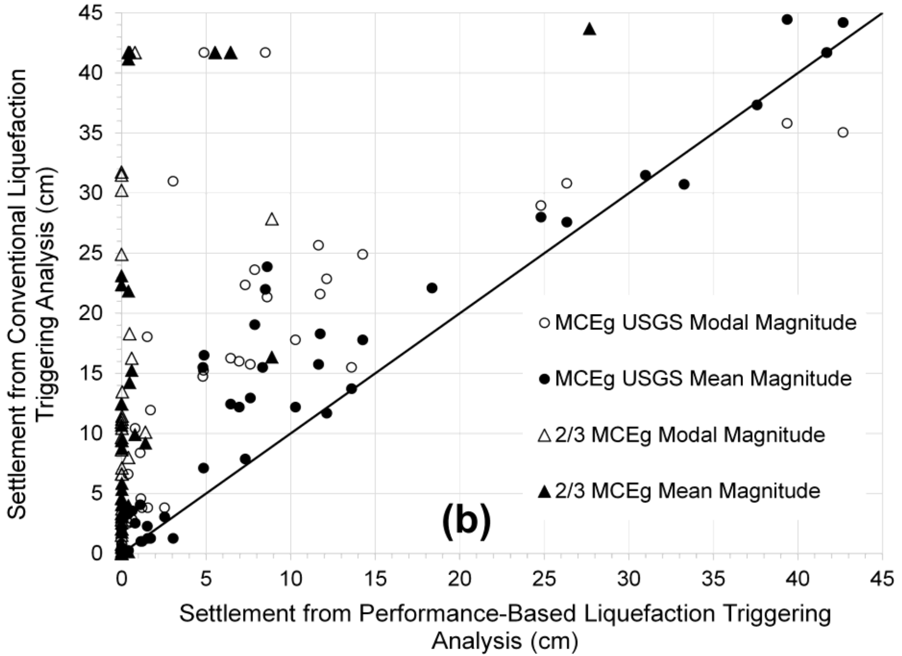
Franke, Lingwall, Youd, Blonquist, and He. (2019). Overestimation of liquefaction hazard in areas of low to moderate seismicity due to improper characterization of probabilistic seismic loading. *Soil Dyn. Earthquake Eng.*, 116, 681-691.

Why This Matters to Regions of Low to Moderate Seismicity

Results if assuming a Site Class E…..



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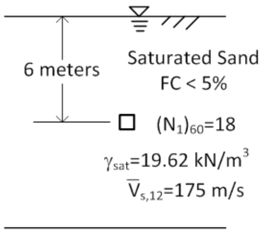
Franke, Lingwall, Youd, Blonquist, and He. (2019). Overestimation of liquefaction hazard in areas of low to moderate seismicity due to improper characterization of probabilistic seismic loading. *Soil Dyn. Earthquake Eng.,* 116, 681-691.

Simplified Probabilistic Liquefaction Triggering Procedure

Mayfield et al. (2010) introduced the concept of a **simplified performance-based liquefaction triggering analysis**

Gridded PB Analysis for Generic Soil Layer → Map Liq Hazard at Targeted Return Periods → Correct for Site-Specific Soil Conditions and Stresses



Liquefaction Reference Parameter Map

DIFFERENT FROM a Liquefaction Hazard Map

Depth Reduction

Soil Stresses

Site Amplification

Simplified Probabilistic Procedures for Other Liquefaction Effects

In 2014, a major multi-state, multi-agency research effort was initiated to develop map-based uniform hazard analysis procedures for various liquefaction effects (settlement, lateral spread, and Newmark seismic slope displacement).

U.S. Department of Transportation Federal Highway Administration

Oregon Department of Transportation

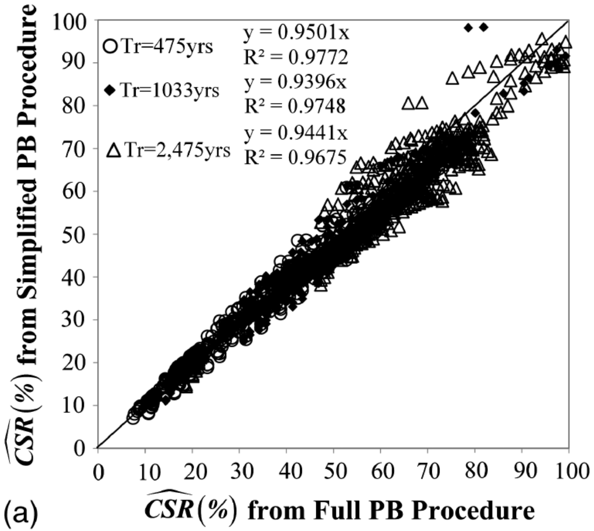
SCDOT South Carolina Department of Transportation

MONTANA MDT DEPARTMENT OF TRANSPORTATION

USGS science for a changing world

Simplified PB Liquefaction Triggering

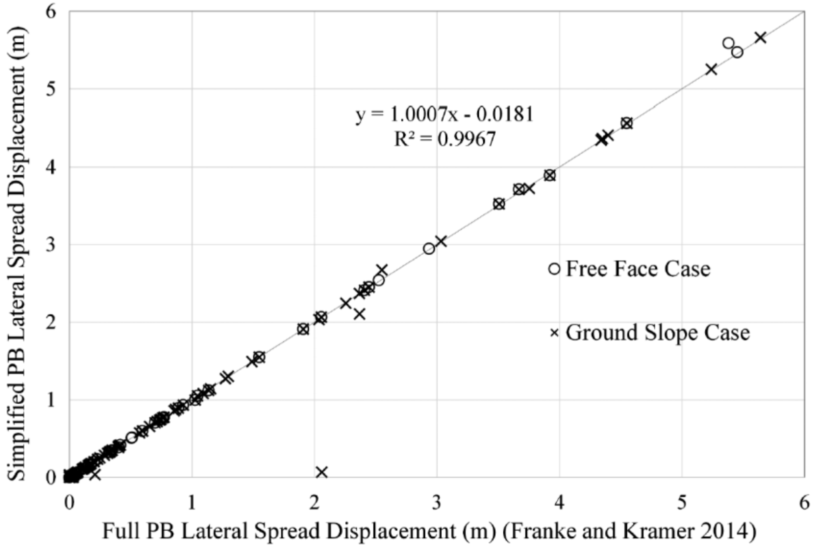
Here are some comparisons from 10 different cities, 5 different soil profiles, and 3 different return periods (Ulmer and Franke 2016, showing Boulanger and Idriss (2012) model results)



(a)

Simplified PB Lateral Spread

A similar simplified performance-based approach was developed using the Youd et al. (2002) lateral spread model. (Ekstrom and Franke 2016)



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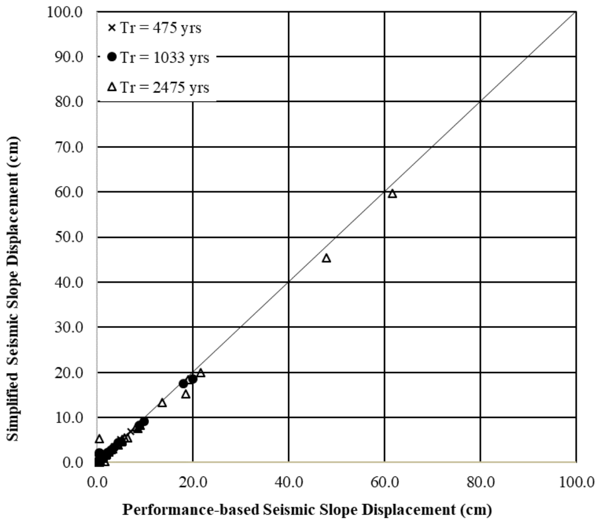
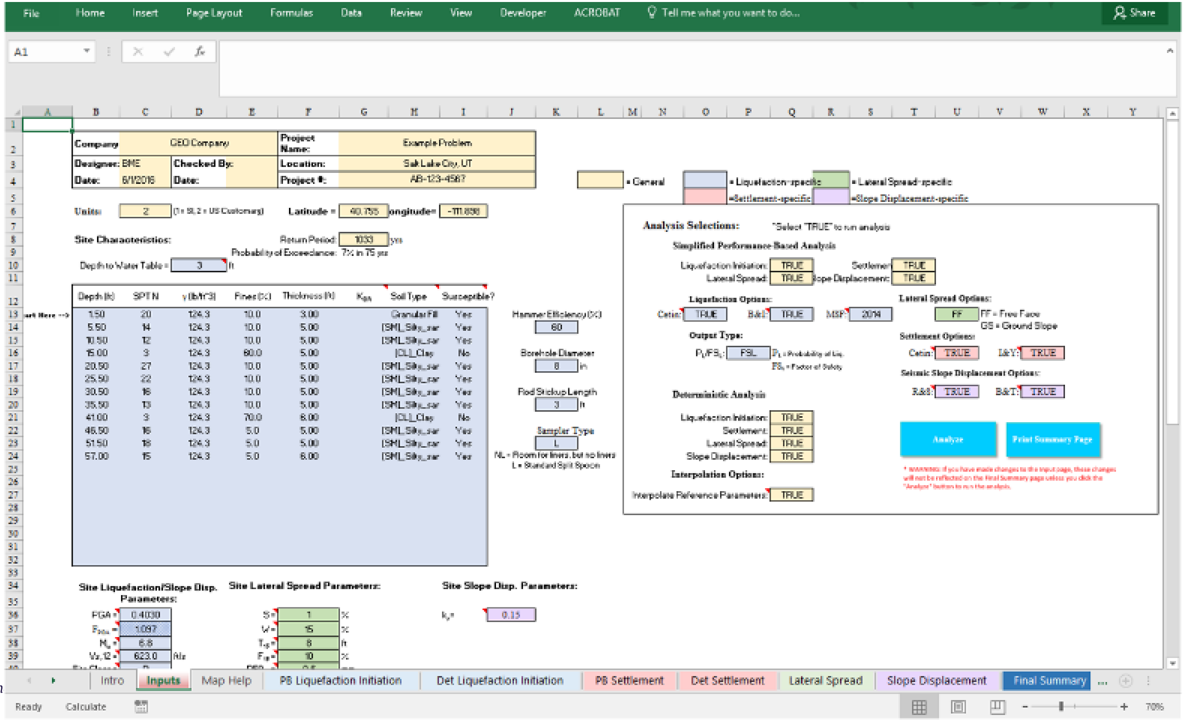
Simplified PB Newmark Seismic Slope Displacement

Lucy Astorga Hoch (GeoEngineers, WA) developed a simplified performance-based approach for Bray and Travasarou (2007) and Rathje and Saygili (2009)



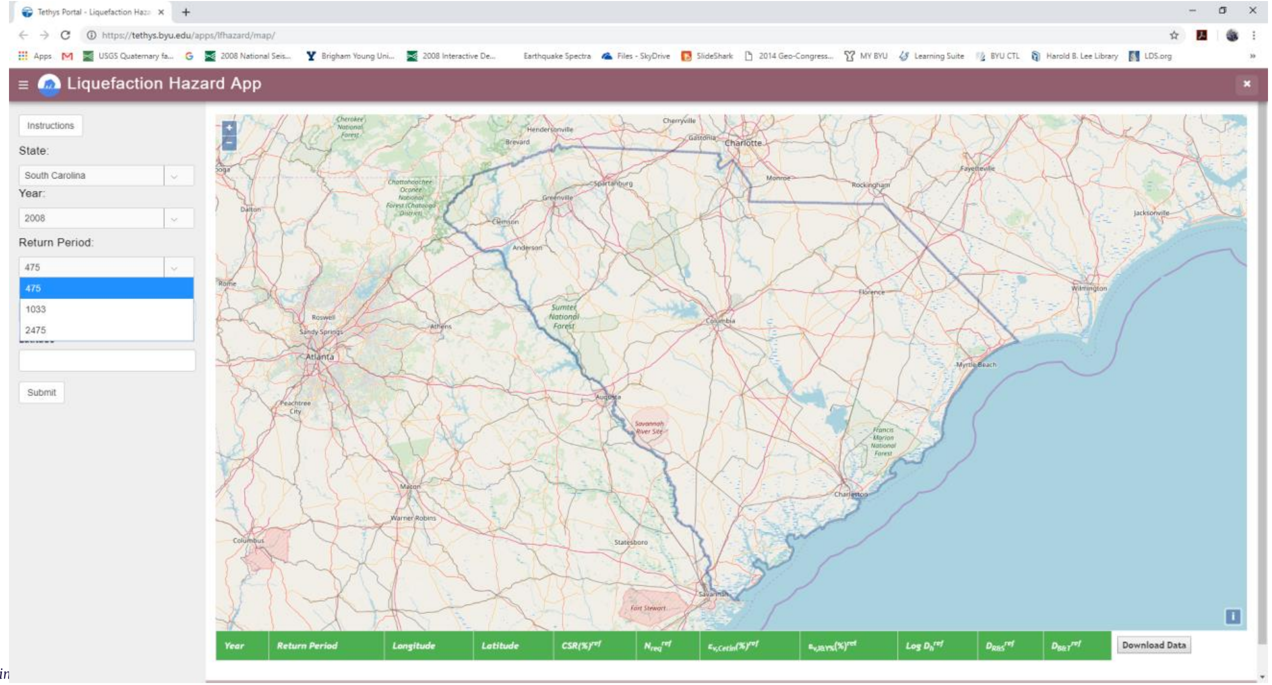
Tools to Perform Simplified PB Liquefaction Hazard Analysis

SPLIQ V1.43, released in July 2022



Tools to Perform Simplified PB Liquefaction Hazard Analysis

Online Liquefaction Reference Parameter Database (currently being updated and populated for all 50 states in the US!)



Overall Recommended Approach

For a given site, perform a (1) simplified performance-based liquefaction assessment, and a (2) deterministic liquefaction assessment. Use the LESSER HAZARD (i.e., higher FS_L, lower settlements or displacements) for design. Abandon use of the pseudo-probabilistic approach!

Conclusions

- The conventional pseudo-probabilistic approach can overpredict liquefaction hazard in areas of low to moderate seismicity
  - Especially where the modal $M_w \geq 7.5$ and is located more than 100 km away from the site
- Performance-based approaches can solve the problem, but require special tools
- Simplified performance-based methods can give you the benefits of the PB approach with the convenience of the conventional approach
- Free tools have been developed and are already being used (Utah, South Carolina)
- Online database of liquefaction reference parameter values for **SPT** *and* **CPT** is currently being populated for the entire US

Related References

- Boulanger, R. W., and Idriss, I. M. (2012). "Probabilistic standard penetration test-based liquefaction-triggering procedure." *J. Geotech. Geoenviron. Eng.*, 10.1061/(ASCE)GT.1943-5606.0000700, 1185–1195.
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- Franke, K.W. and Kramer, S.L. (2014). Procedure for the empirical evaluation of lateral spread displacement hazard curves. *J. Geotech. Geoenvir. Eng.*, 140(1), 110-120.
- Franke, K.W., Ulmer, K.J., Ekstrom, L.T., and Meneses, J. (2016). Clarifying the differences between traditional liquefaction hazard maps and performance-based liquefaction reference parameter maps. Soil Dynamics and Earthquake Engineering, Elsevier, 90(2016), 240-249.
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- Franke, K.W., Youd, T.L., Ekstrom, L.T., and He, J. (2017). Probabilistic Lateral Spread Evaluation for Long, Linear Infrastructure using Performance-Based Reference Parameter Maps. *Proc., Geo-Risk 2017,* ASCE, Paper No. 6, Denver, CO, June 2017
- Franke, Lingwall, Youd, Blonquist, and He. (2019). Overestimation of liquefaction hazard in areas of low to moderate seismicity due to improper characterization of probabilistic seismic loading. *Soil Dyn. Earthquake Eng.,* 116, 681-691.
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- Youd, T.L., Hansen, C.M., and Bartlett, S.F. (2002). Revised multilinear regression equations for prediction of lateral spread displacements*. J. Geotech. Geoenvir. Eng.,* 128(12), 1007-1017.

SIMPLIFIED PERFORMANCE-BASED LIQUEFACTION HAZARD ANALYSIS

Kevin W. Franke, PhD, PE, DGE, M.ASCE

Dept. of Civil and Construction Engineering

Brigham Young University

TRB Webinar

May 23, 2023

Liquefaction-Induced Pile Downdrag from Full-scale Blast Tests

Kyle Rollins
Civil & Construction Engineering
Brigham Young University

TRB Webinar: Innovation in Geoseismic Foundation Design and Performance – May 24, 2023

Research Sponsors

US National Science Foundation NSF Rapid Grant CMMI-1408892

Caltrans/PEER

Tonkin & Taylor

Istituto Nazionale di Geofisica e Vulcanologia

Utah DoT udot.utah.gov

U.S. Department of Transportation Federal Highway Administration

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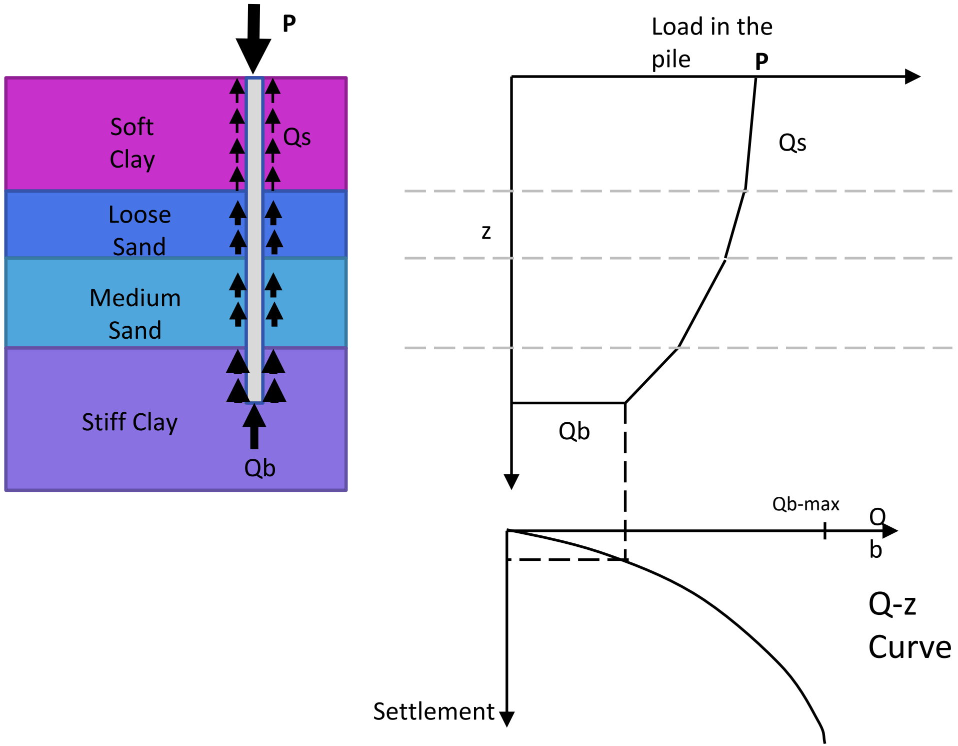
Geofondazioni



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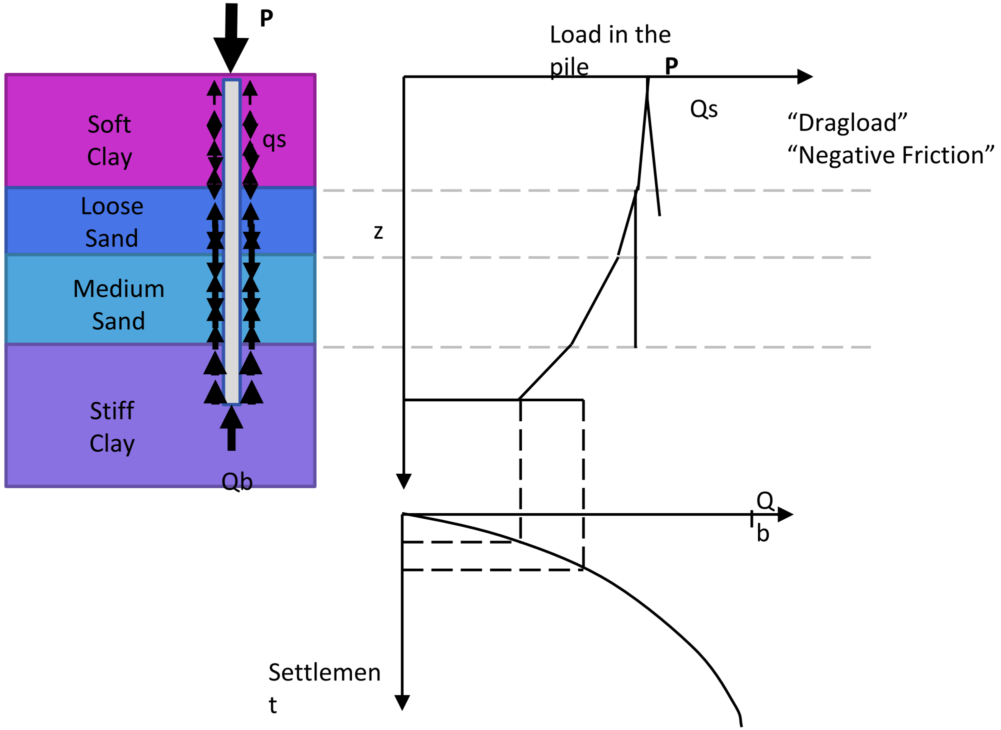
Settlement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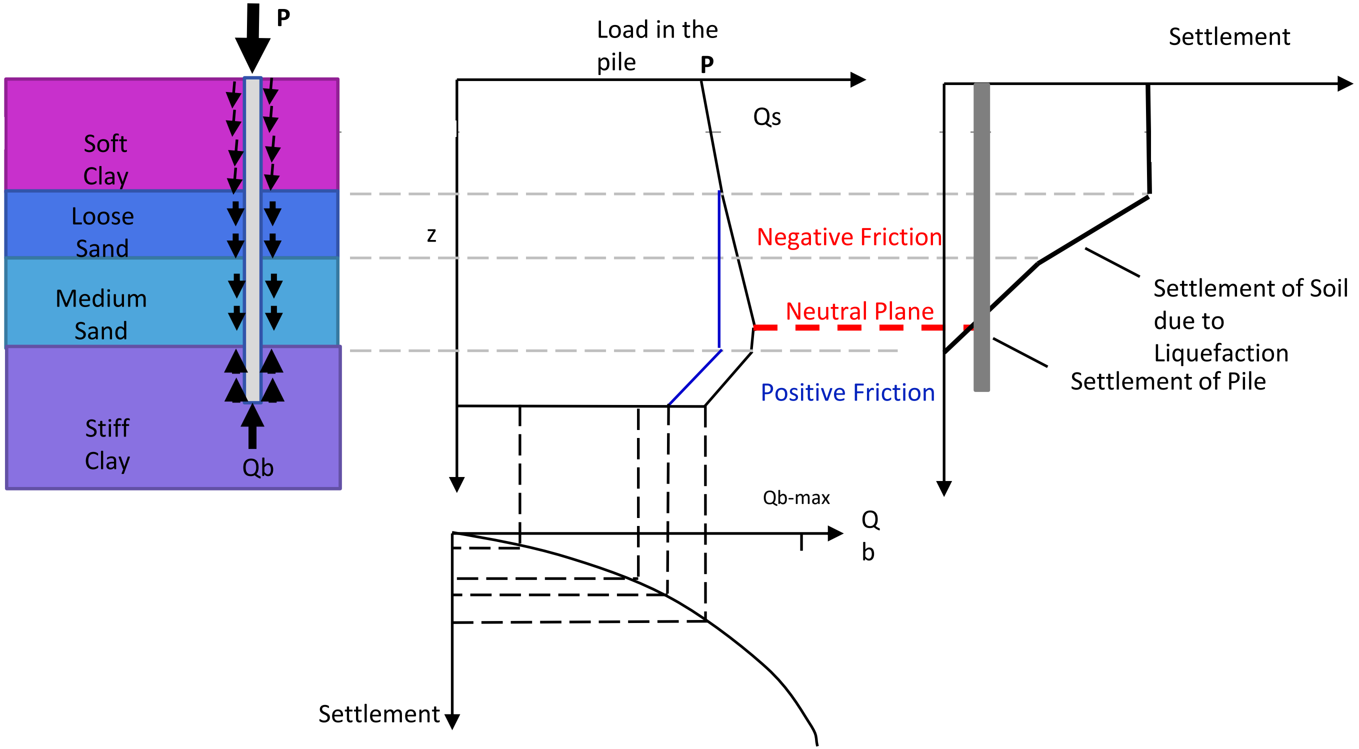
Settlement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Qb-max
Q
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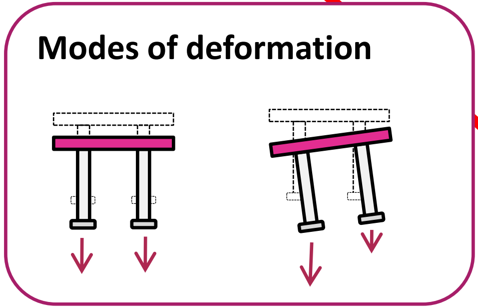
Settlement
Settlement
Settlement of Soil
due to
Liquefaction
Settlement of Pile

Juan Pablo II Bridge-Concepcion Chile

Liquefaction-induced pier settlements along bridge span



Resurrection River Bridge, Seward, AK

Pile Downdrag During M_w9.2 Alaska Earthquake

Piles Pulled out of Pile caps

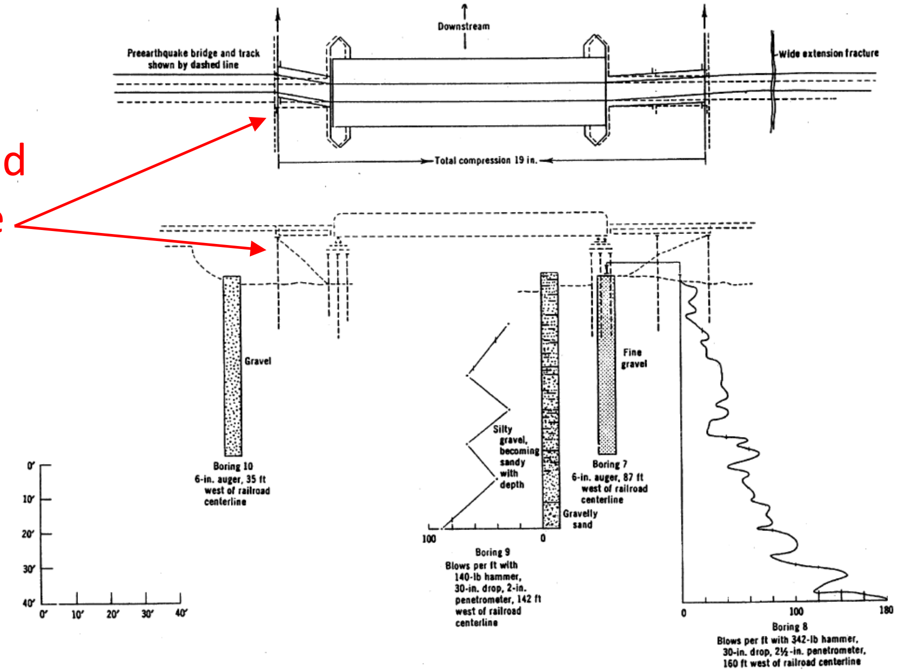


FIGURE 1 Construction, damage, and subsurface information, Bridge 3.3, Seward, Alaska (7) (1 lb = 4.45N, 1 in. = 0.0254m, 1 ft = 0.3048m).

Research Objectives

- Measure development of negative skin friction during liquefaction and reconsolidation
  - Determine skin friction in liquefied sand
  - Determine skin friction in non-liquefied soil
- Evaluate neutral plane concept to account for pile settlement

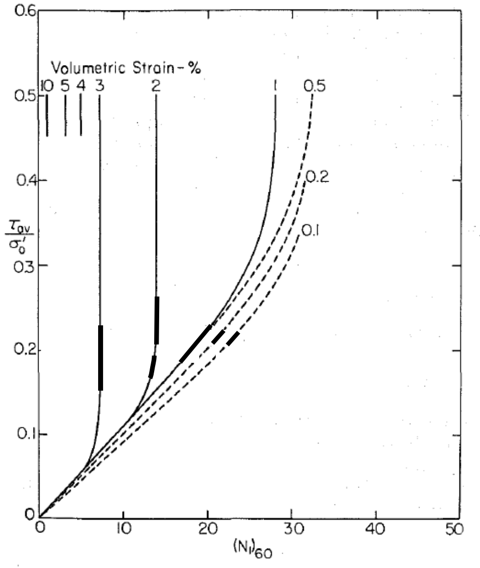
Full-Scale Blast Liquefaction Downdrag Tests

- Vancouver, British Columbia, Canada - 2005
  - 32 cm Driven Steel Pipe Pile
- Christchurch, New Zealand - 2013
  - 60 cm Augercast Piles
- Mirabello, Italy - 2016 (with INGV)
  - 25 cm Micropile
- Turrell, Arkansas, USA – 2017 (with Univ. of Arkansas)
  - 1.2 m Drilled Shafts and 45 cm Driven Piles
- Mirabello, Italy - 2021 (with INGV)
  - 51.5 cm Tapered Single and Group Piles

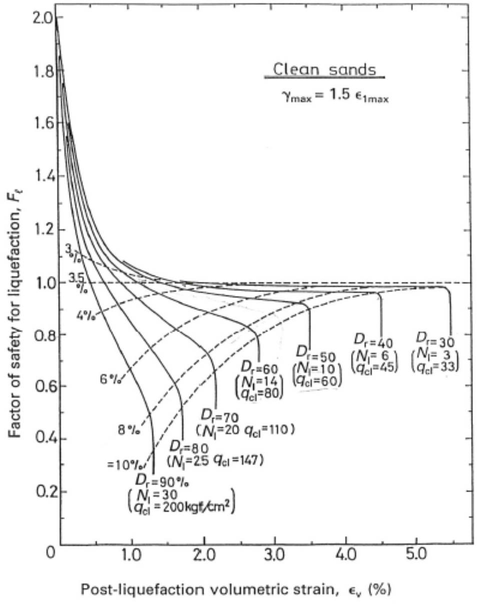
17 Test Piles at 5 Sites!

Design Procedure for Pile Downdrag

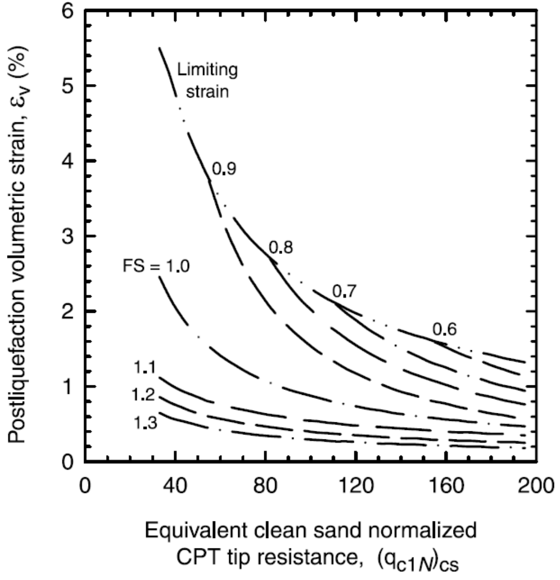
1. Determine settlement vs. depth profile and assume neutral plane location



Tokimatsu and Seed (1987)

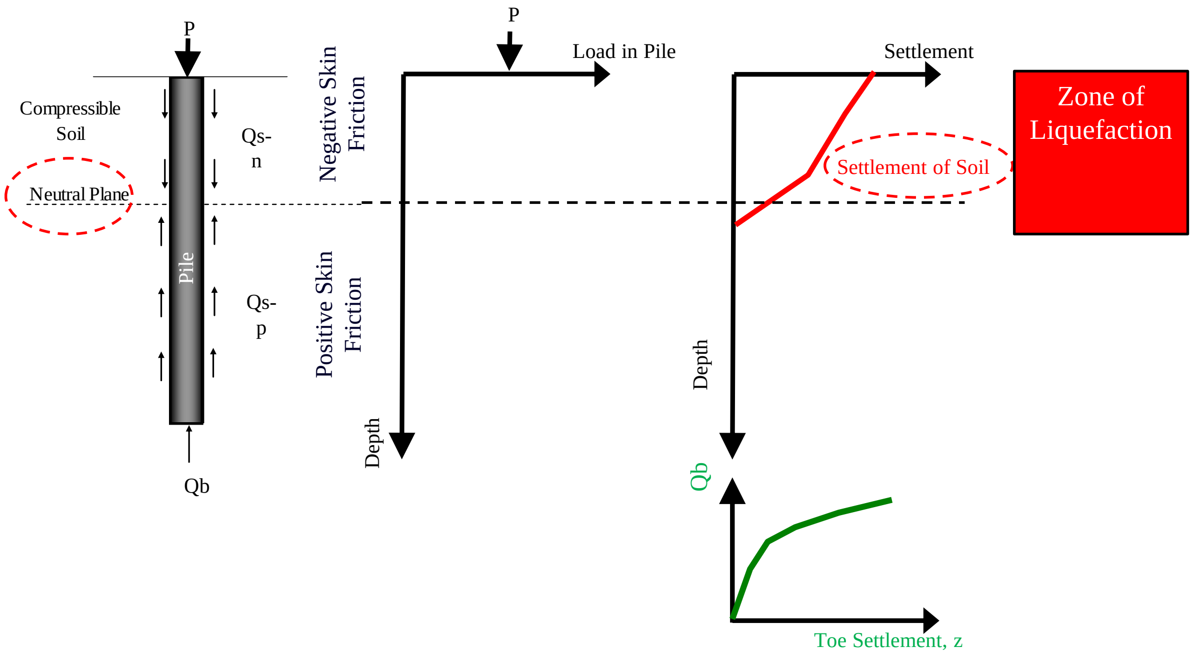


Yoshimine & Ishihara (1992)



Zhang et al. (2004)

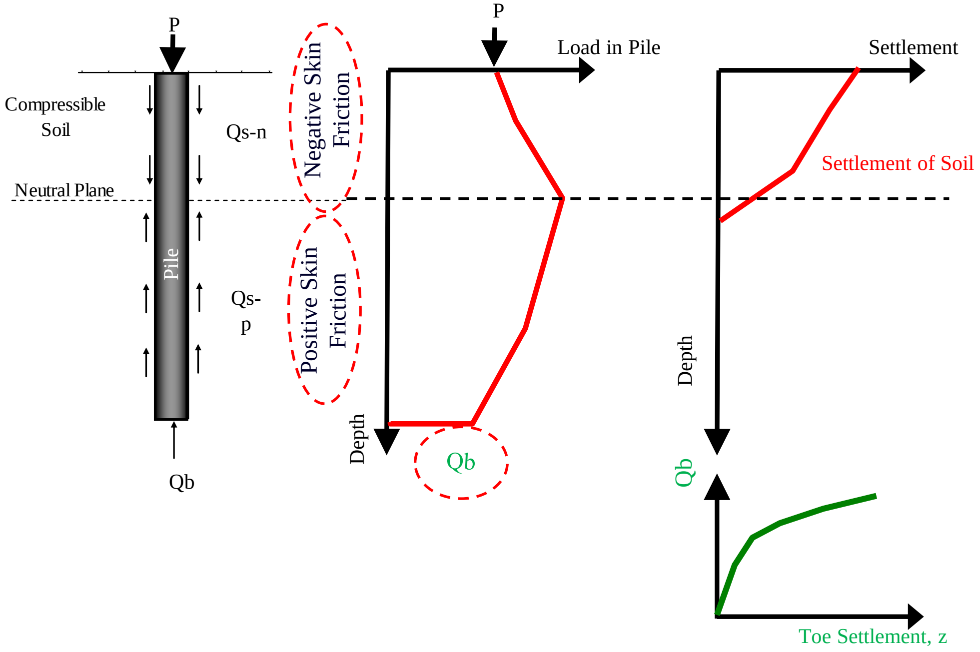
Load in Pile vs Depth with Downdrag



Design Procedure for Pile Downdrag

1. Determine settlement vs. depth profile and assume neutral plane location
2. Compute load distribution in pile
   - Negative friction above neutral plane
   - Positive friction below neutral plane
     - Use 50% of skin friction in liquefied layers
   - Find required toe resistance, Q_b

Load in Pile vs Depth with Downdrag



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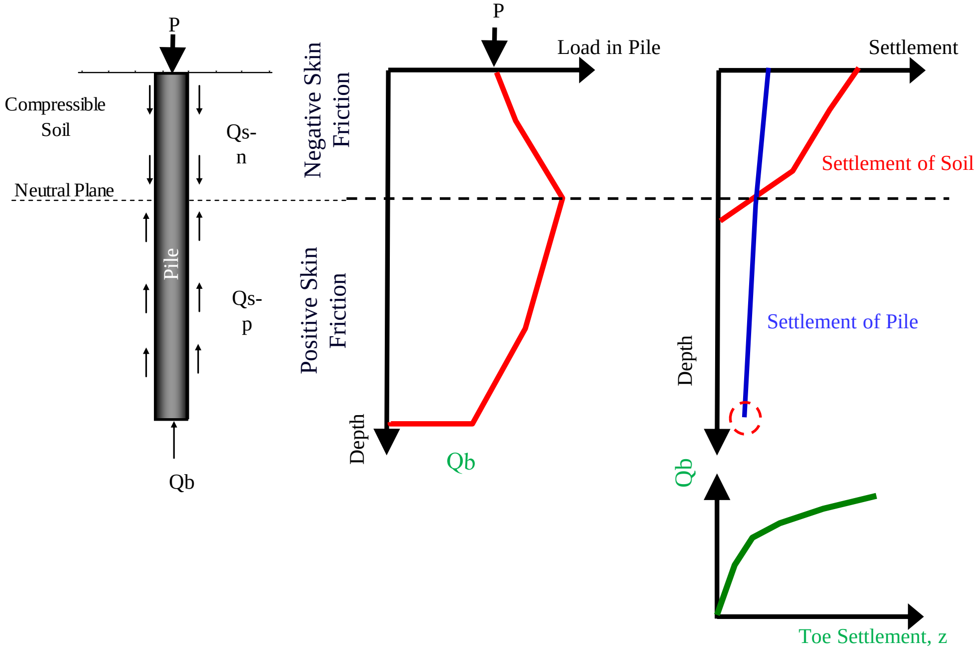
   - Find required toe resistance, Q_b
3. Determine settlement at toe of pile

Load in Pile vs Depth with Downdrag



Design Procedure for Pile Downdrag

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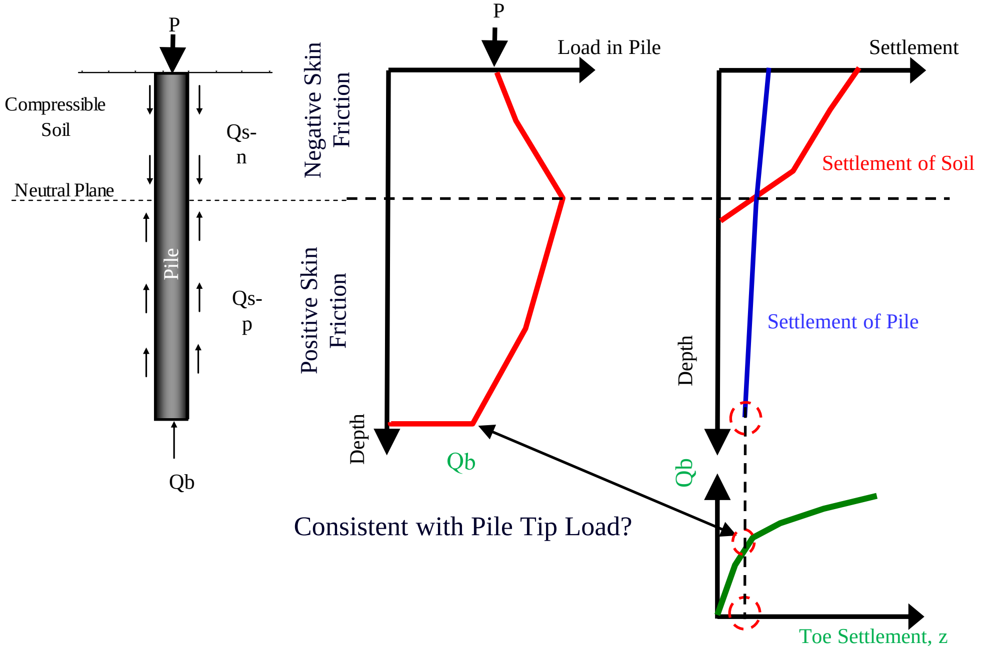
   - Find required toe resistance, Q_b
3. Determine settlement at toe of pile
4. Use Q-z curve to determine if mobilized Q_b for toe settlement is equal to required, Q_b

Load in Pile vs Depth with Downdrag



Design Procedure for Pile Downdrag

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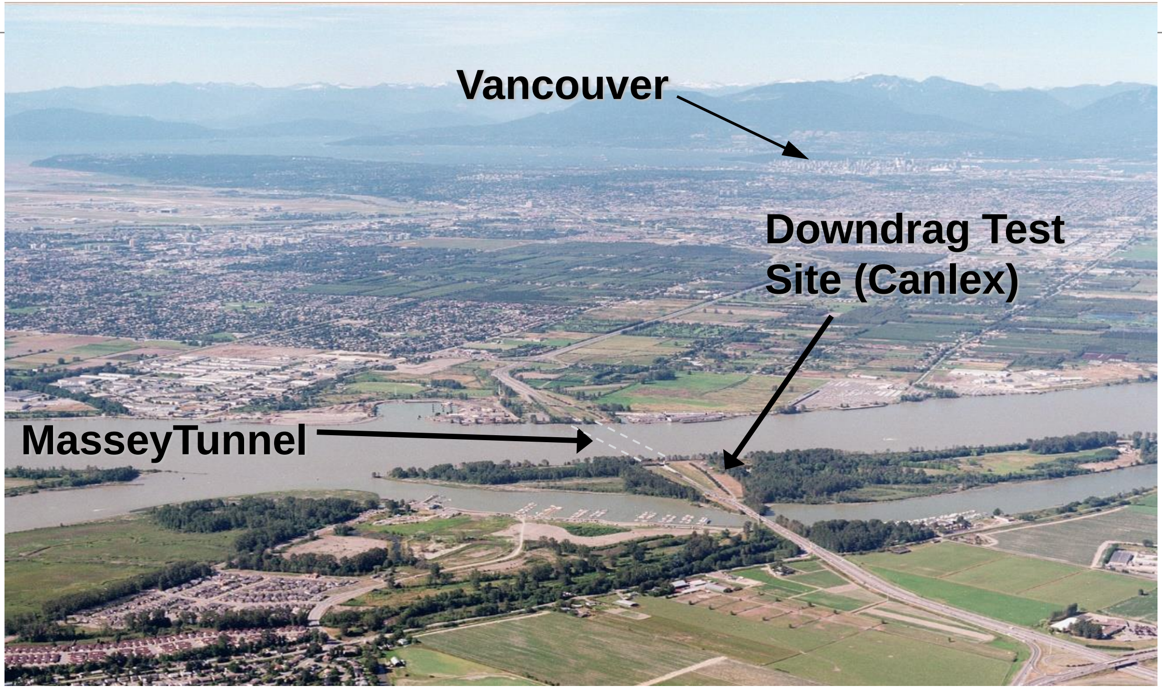
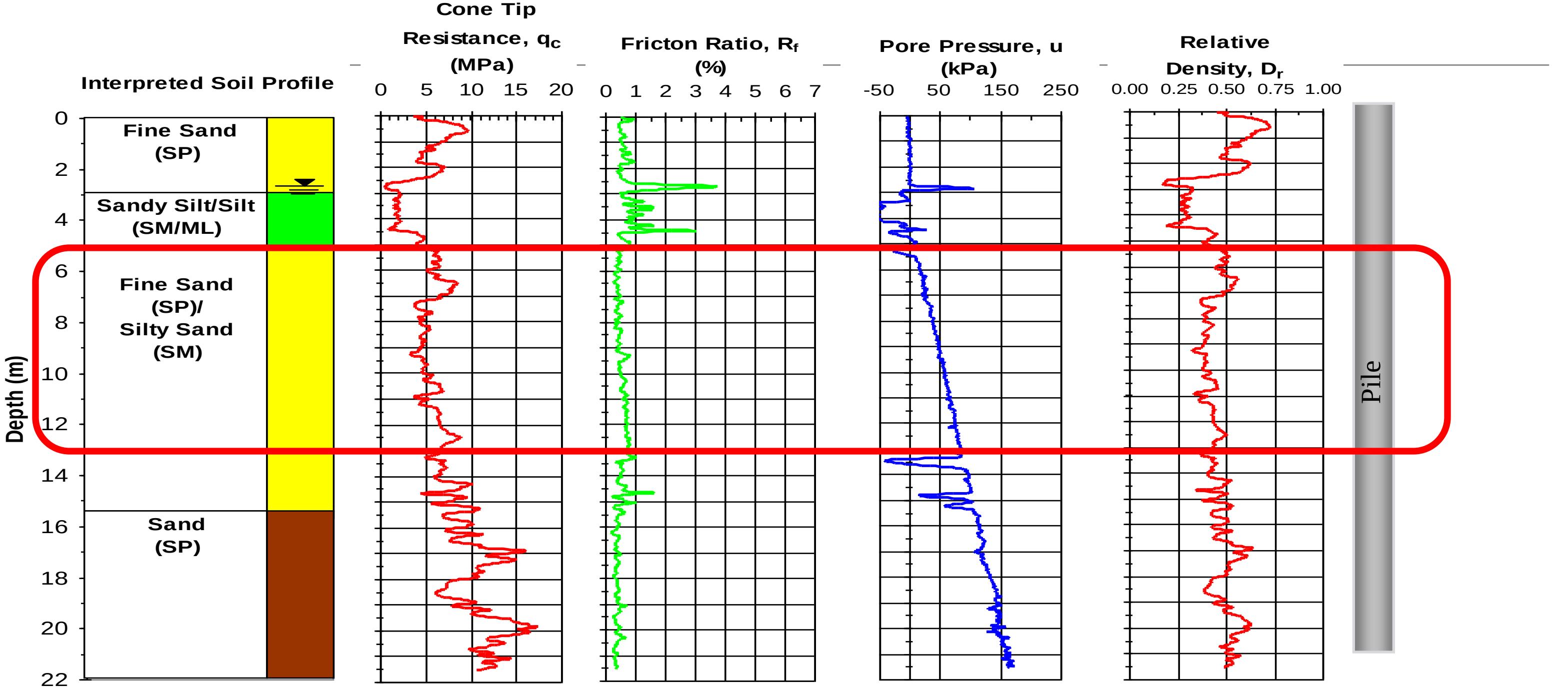
   - Find required toe resistance, Q_b
3. Determine settlement at toe of pile
4. Use Q-z curve to determine if mobilized Q_b for toe settlement is equal to required, Q_b
5. Revise location of neutral plane and repeat the process until convergence

Vancouver Canada Test Site (Rollins & Spencer 2006)

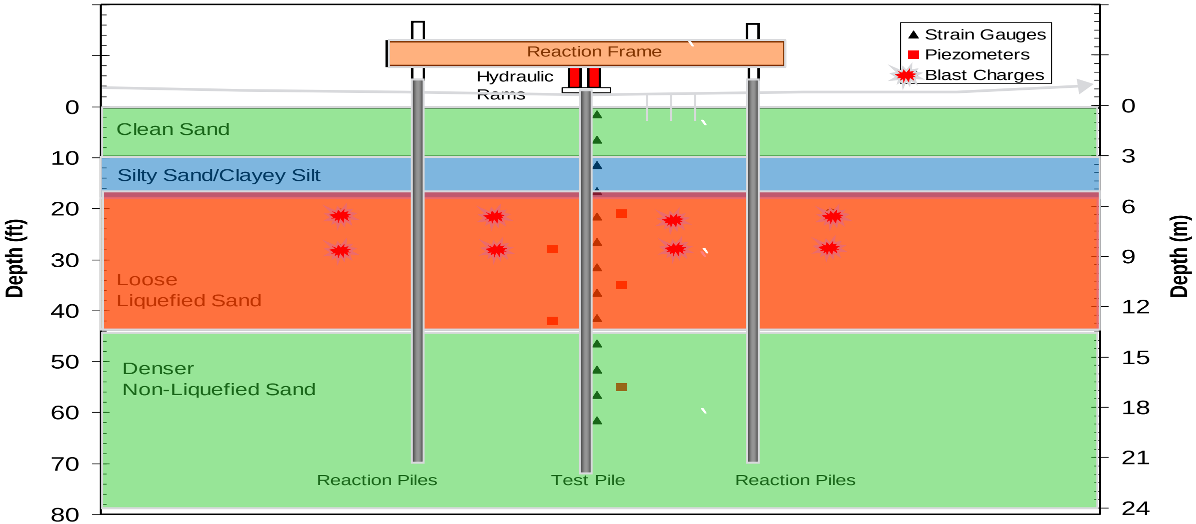


Geotechnical Soil Profile



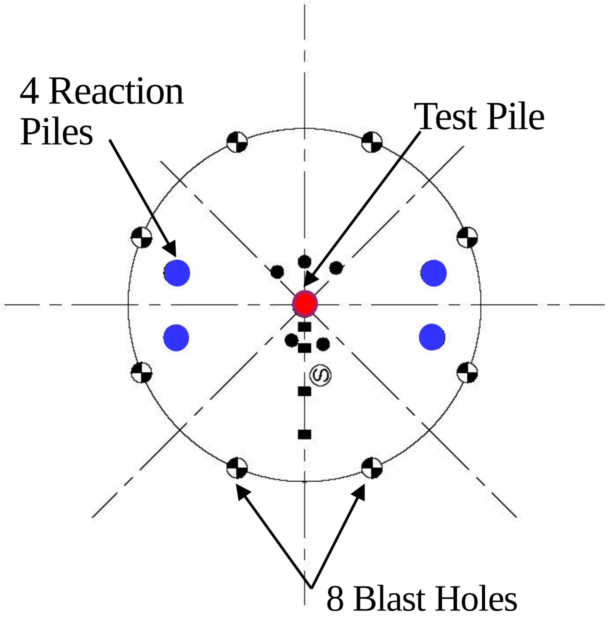
Equivalent SPT $(N_1)_{60}$ is 10 in target zone and 17 near pile tip

Vancouver Downdrag Test Set-up



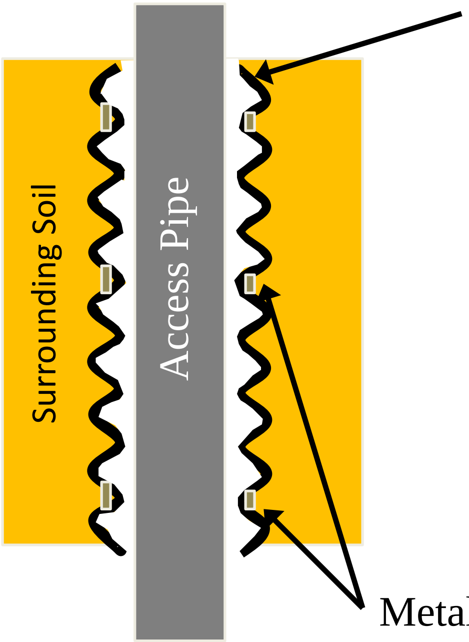
32 mm Driven Steel Pipe Pile

Elevation View

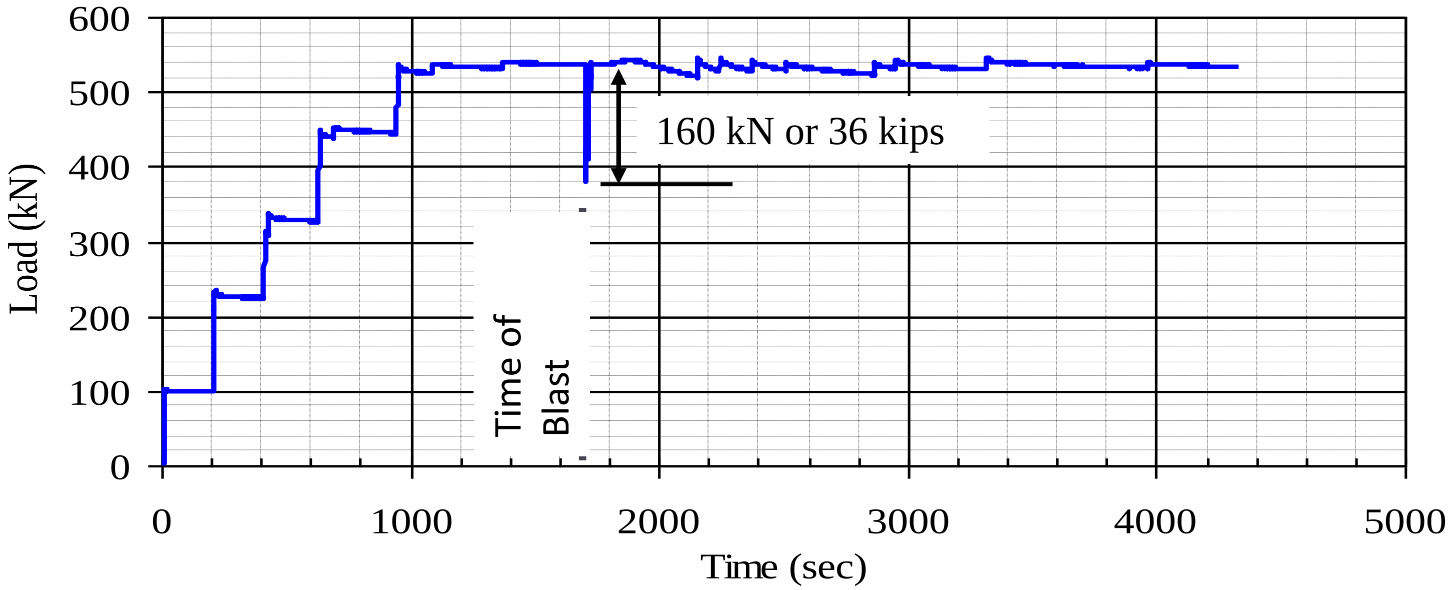


Plan View

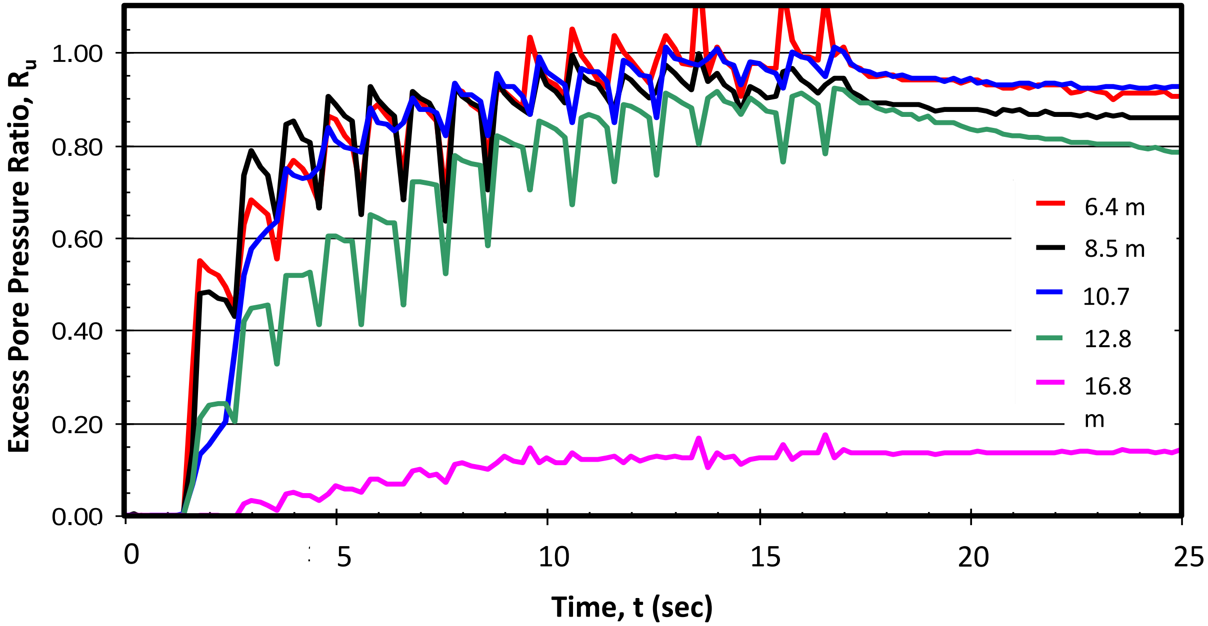
‘Sondex’ Settlement Monitoring vs. Depth



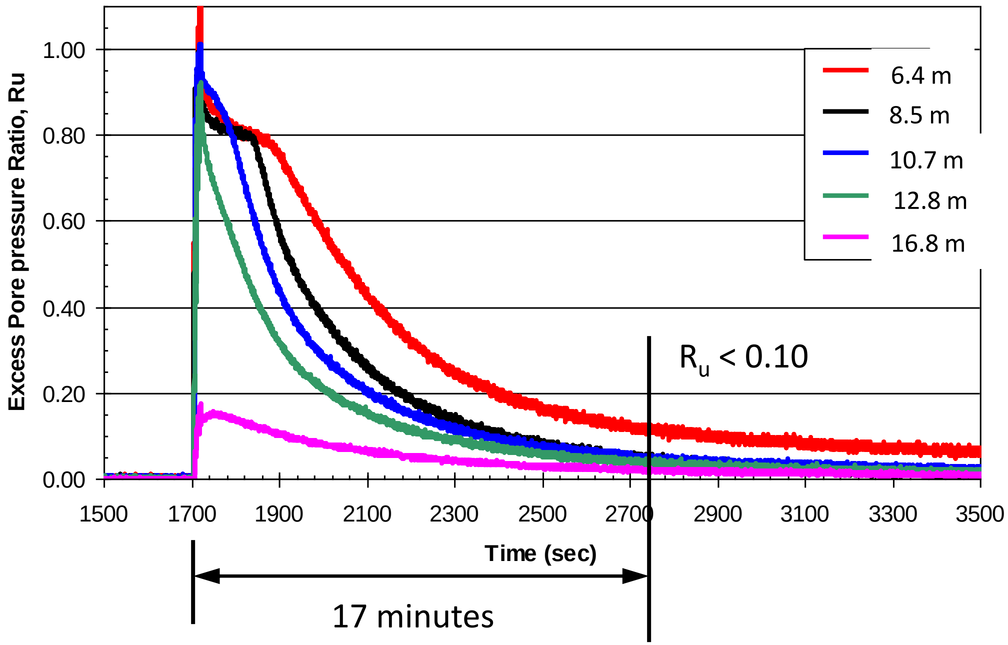
Re-loading Due to Pile Settlement



Pore Pressure Generation During Blasting



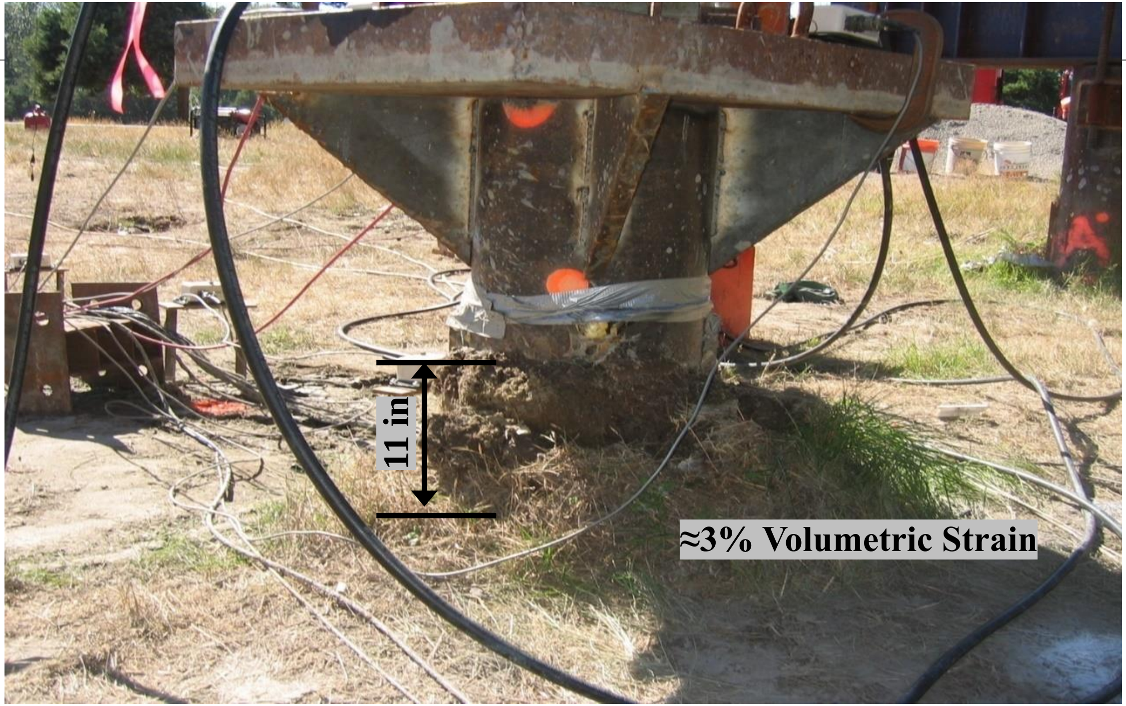
Pore Pressure Dissipation



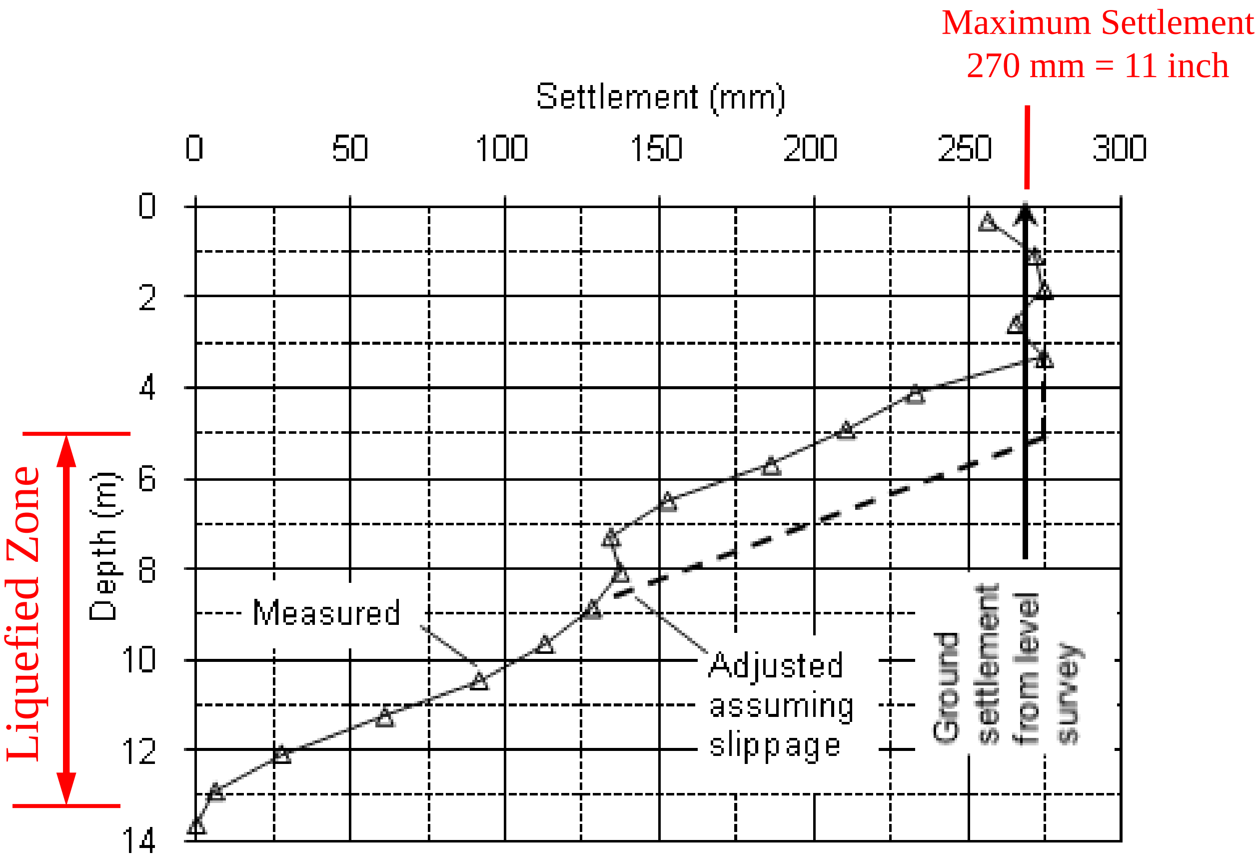
Ground Settlement Trough Around Test Pile

Before Liquefaction

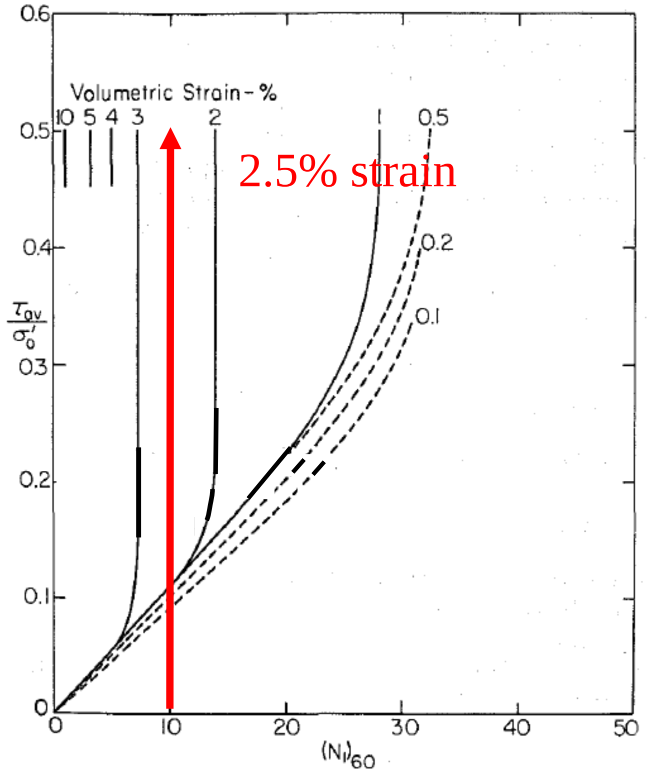
After Liquefaction



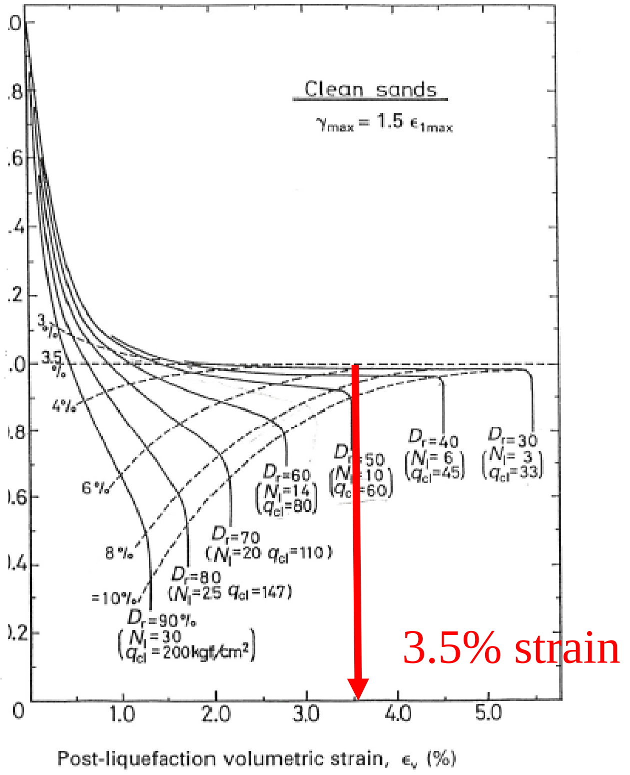
Settlement versus Depth



Expected Liquefaction Settlement

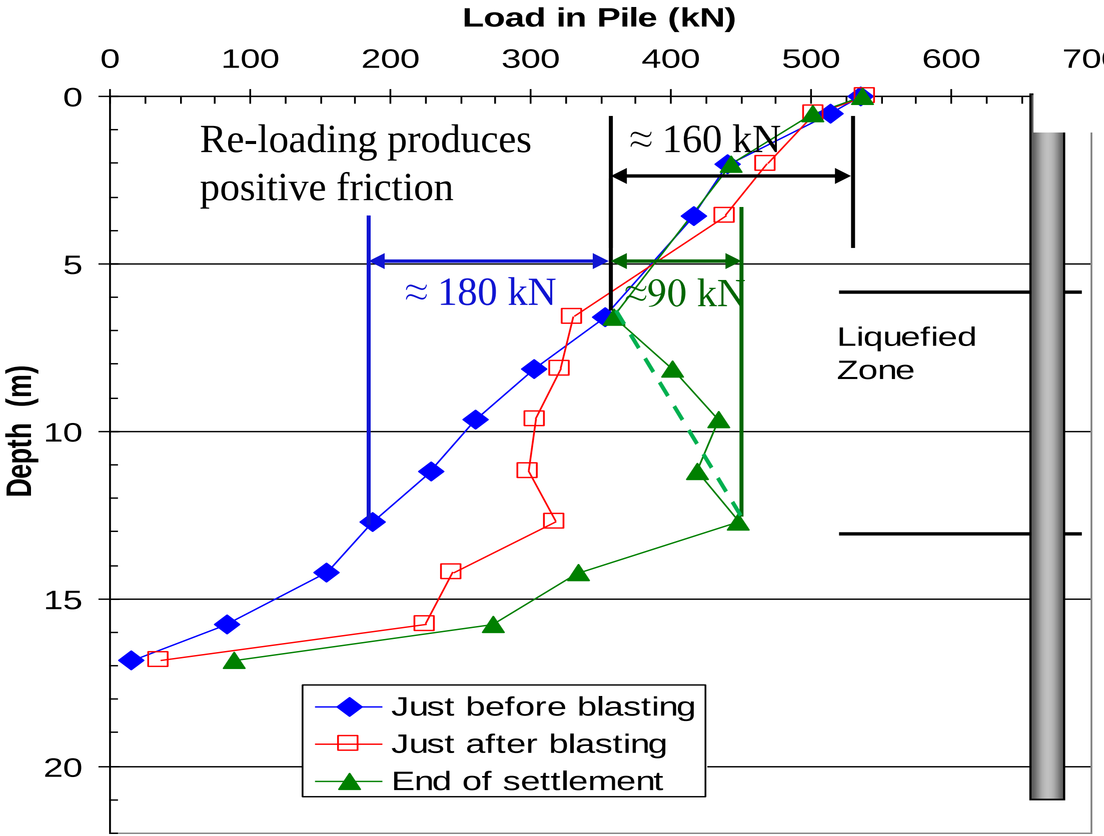


Tokimatsu and Seed (1987)

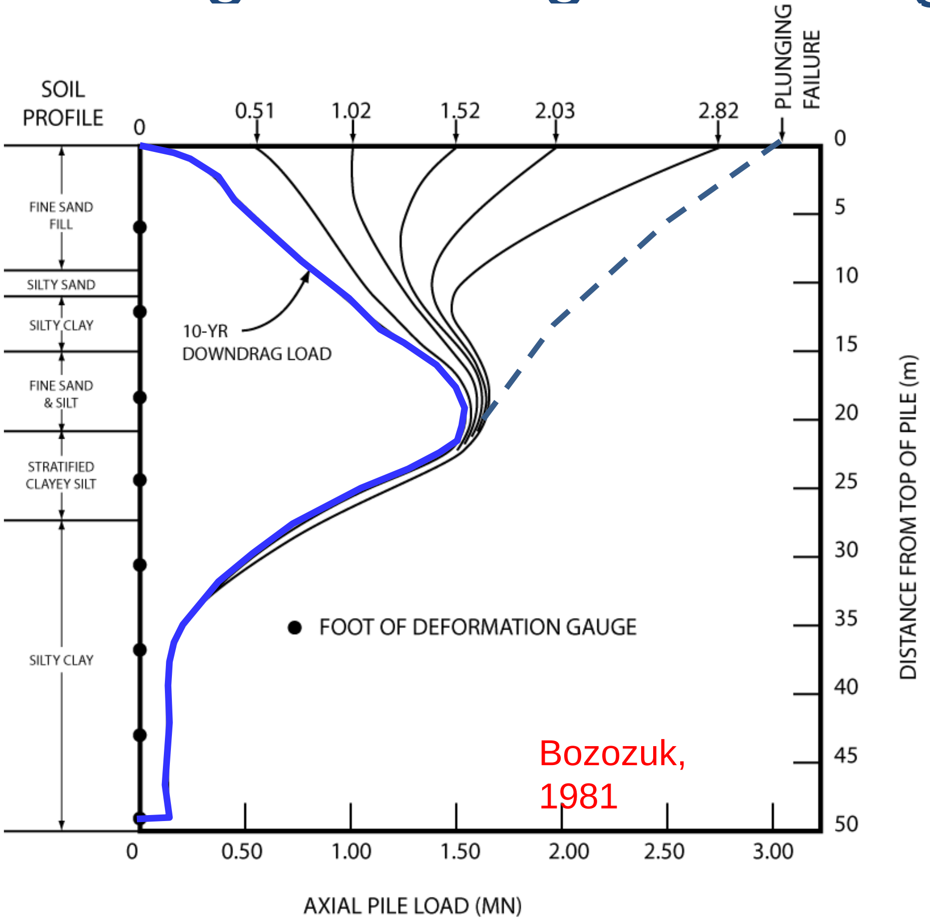


Yoshimine & Ishihara (1992)

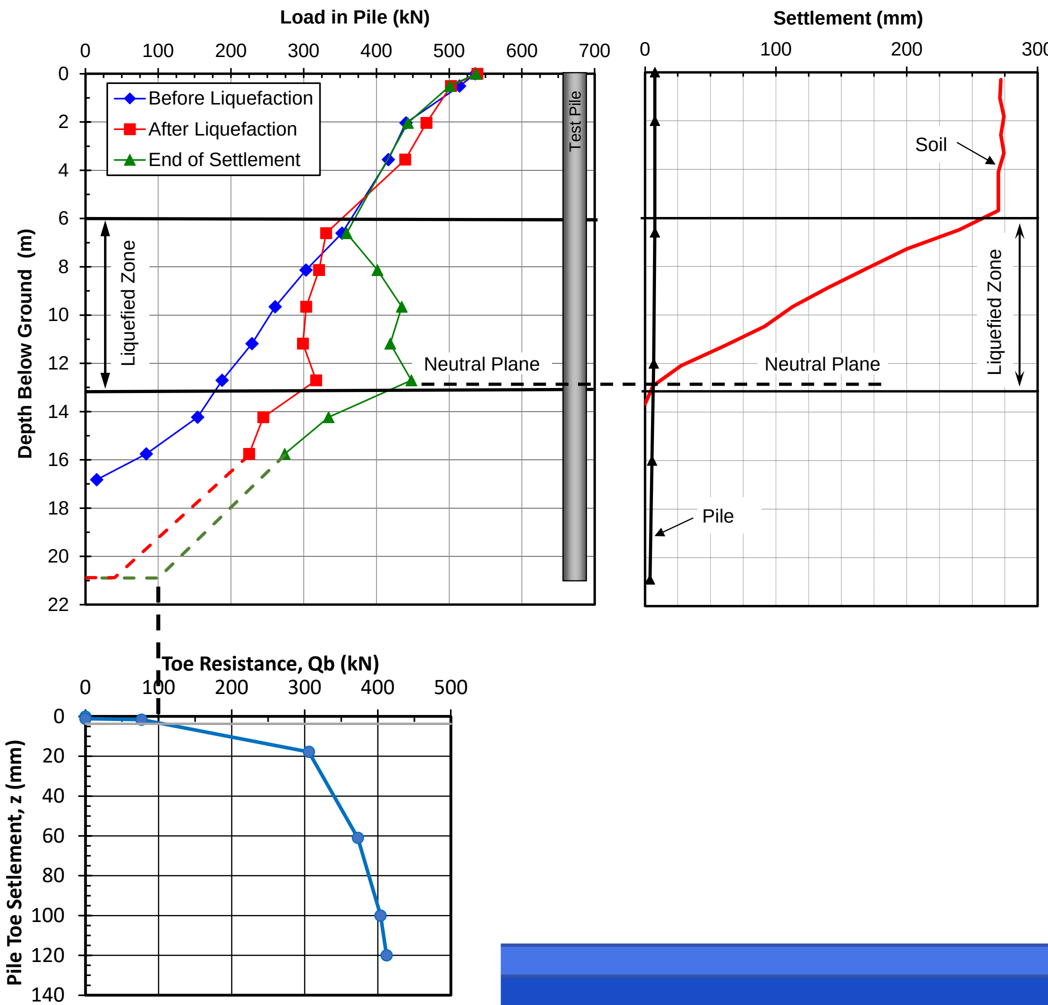
Side Shear Transfer Before and After Liquefaction



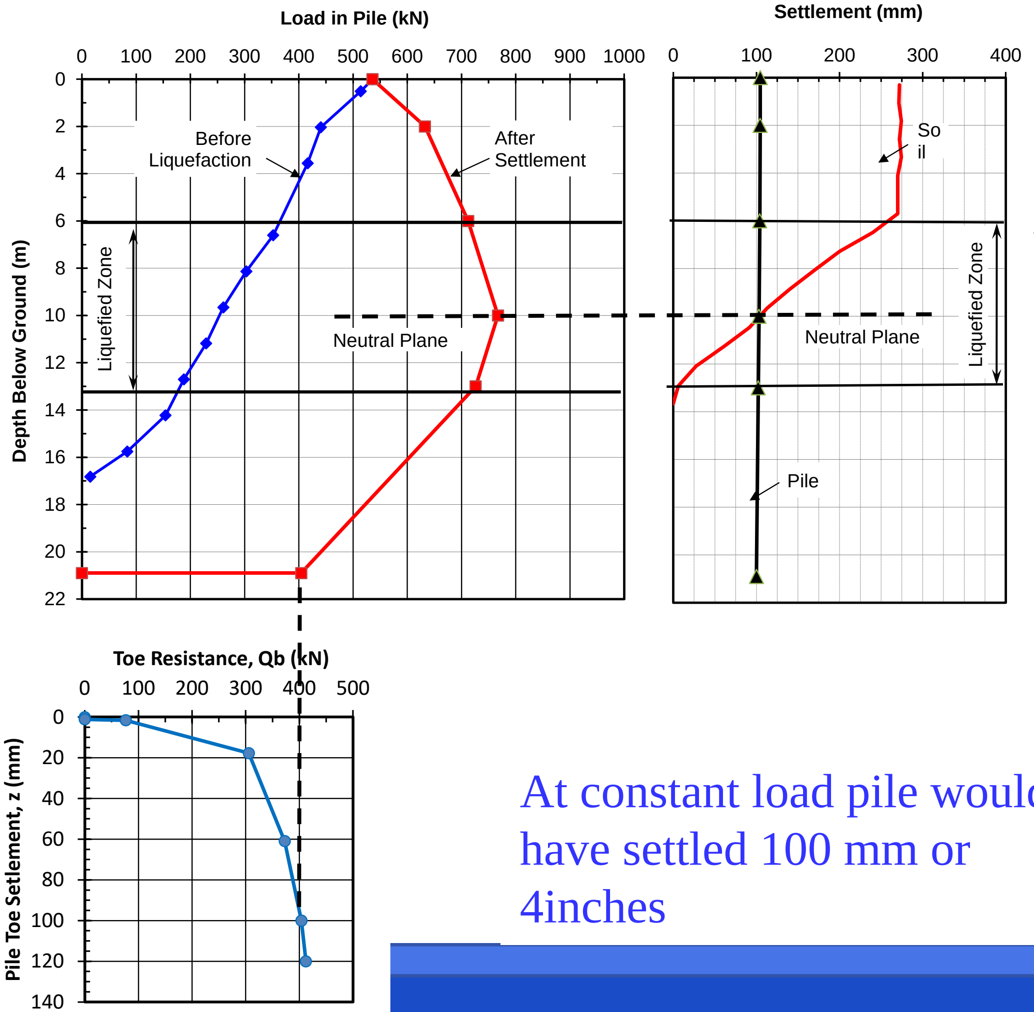
Reloading Following Downdrag



Overall Behavior from Field Load Test



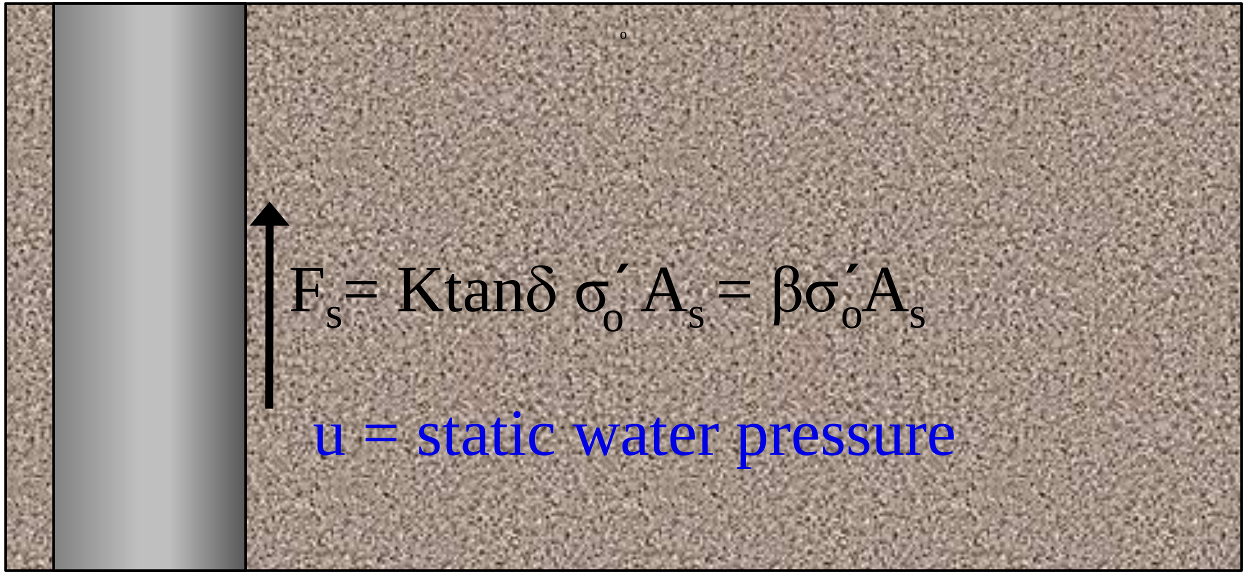
Overall Behavior with Constant Applied Load



At constant load pile would have settled 100 mm or 4inches

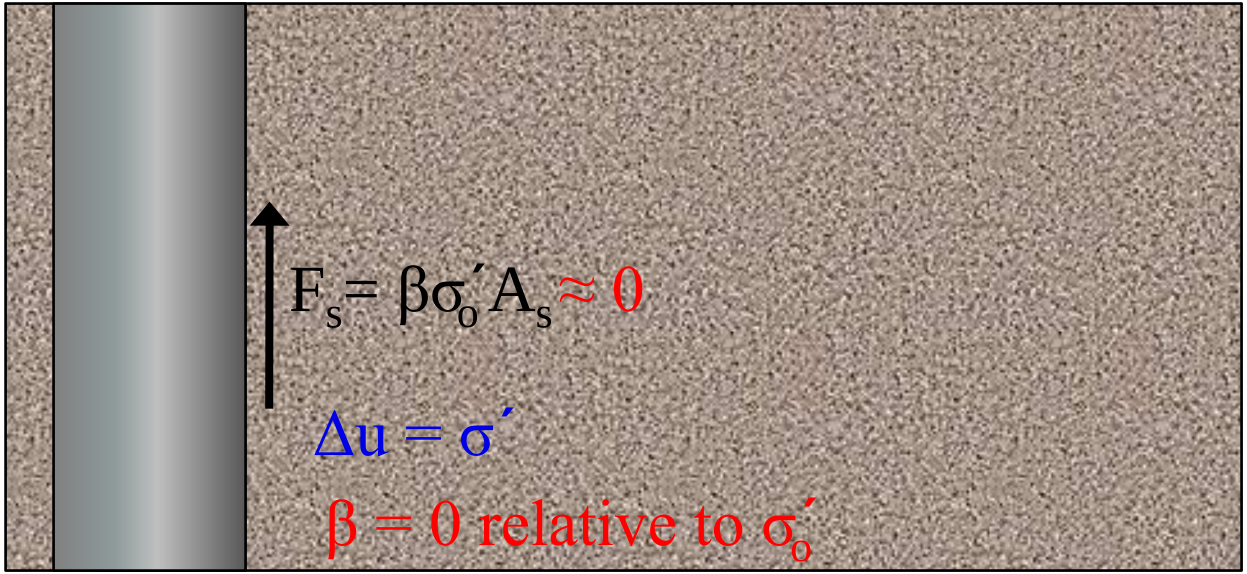
Conceptual View of Behavior

Static Loading Before Liquefaction



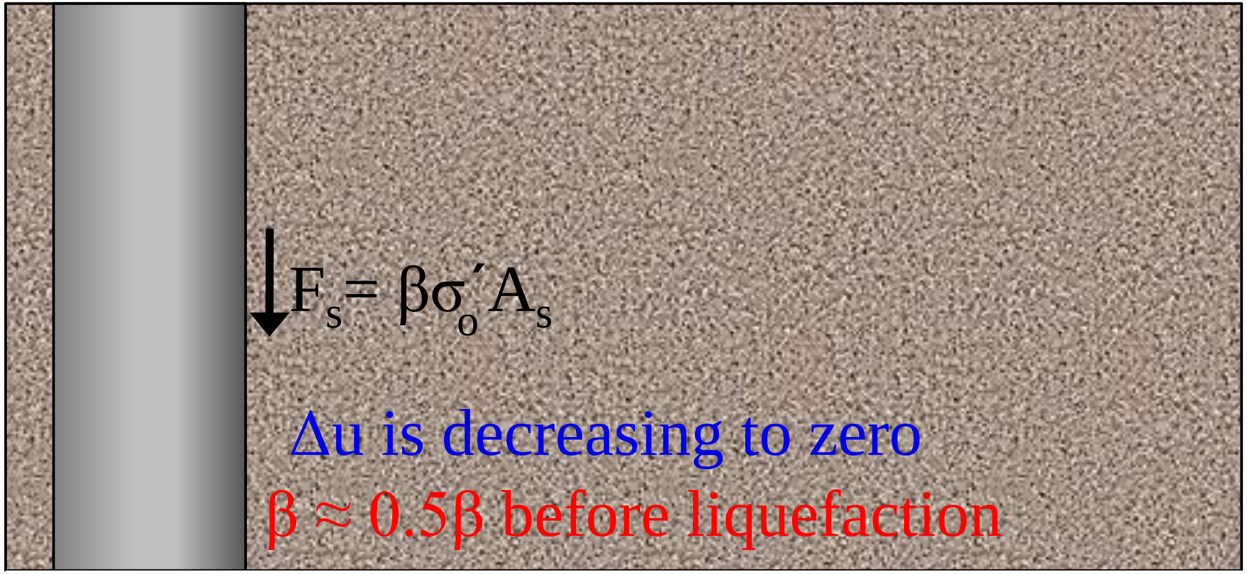
Conceptual View of Behavior

Immediately After Liquefaction

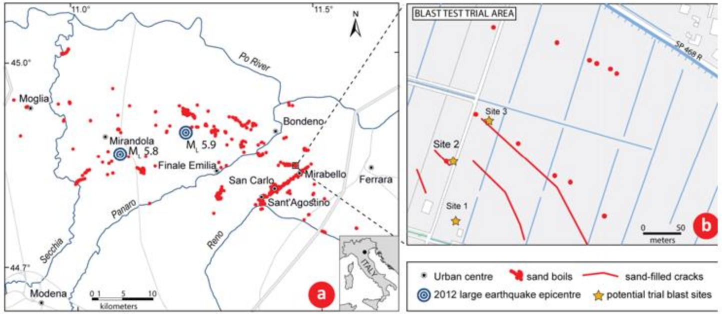


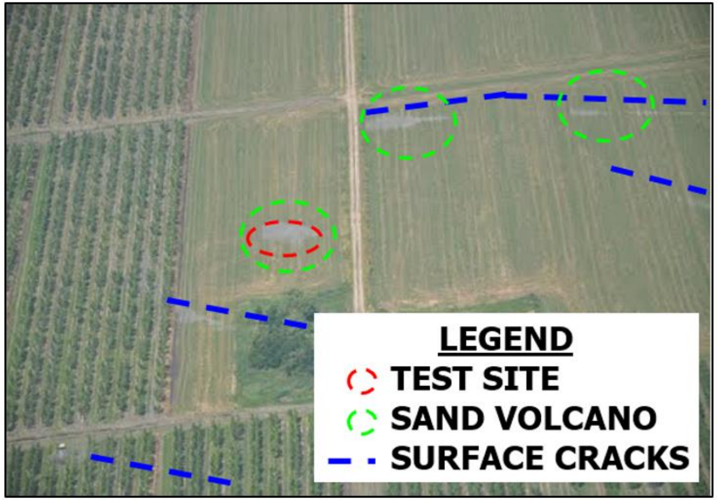
Conceptual View of Behavior

During Reconsolidation

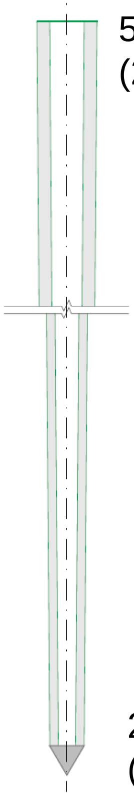


Test Site – Mirabello, Italy





Tapered Piles for Mirabello Site

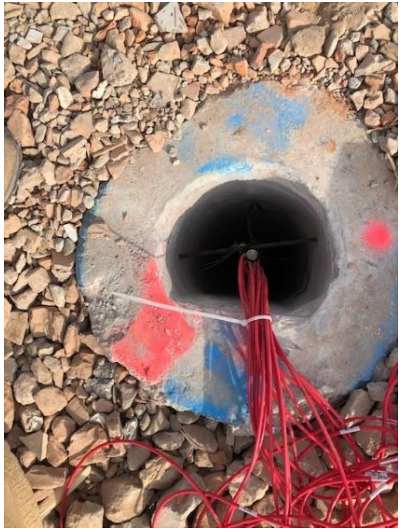


51.5 cm head
(20 inch)

17 m (56 ft)Tapered Piles
(1.5 cm/meter)

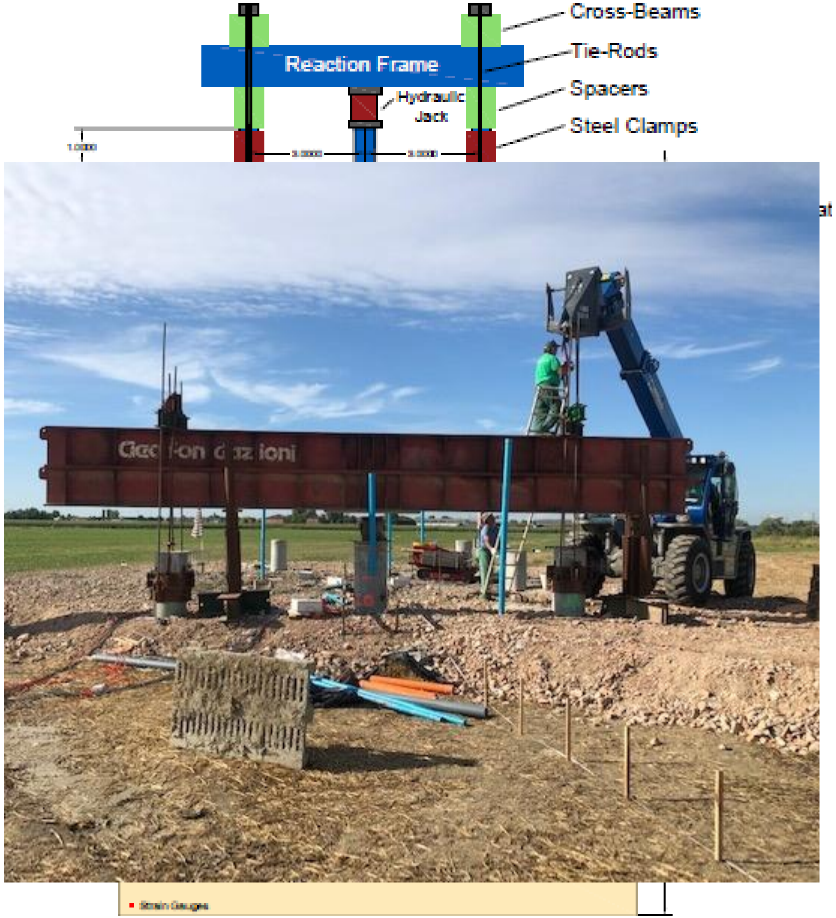
26 cm tip
(10 inch)

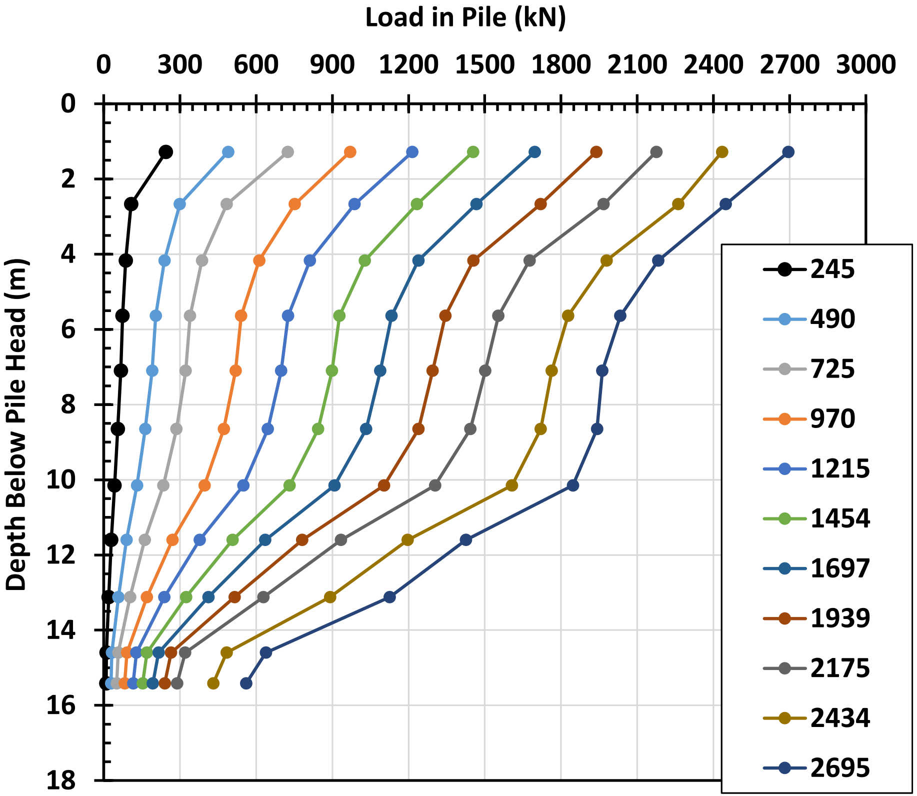
16 Hollow Tapered Piles



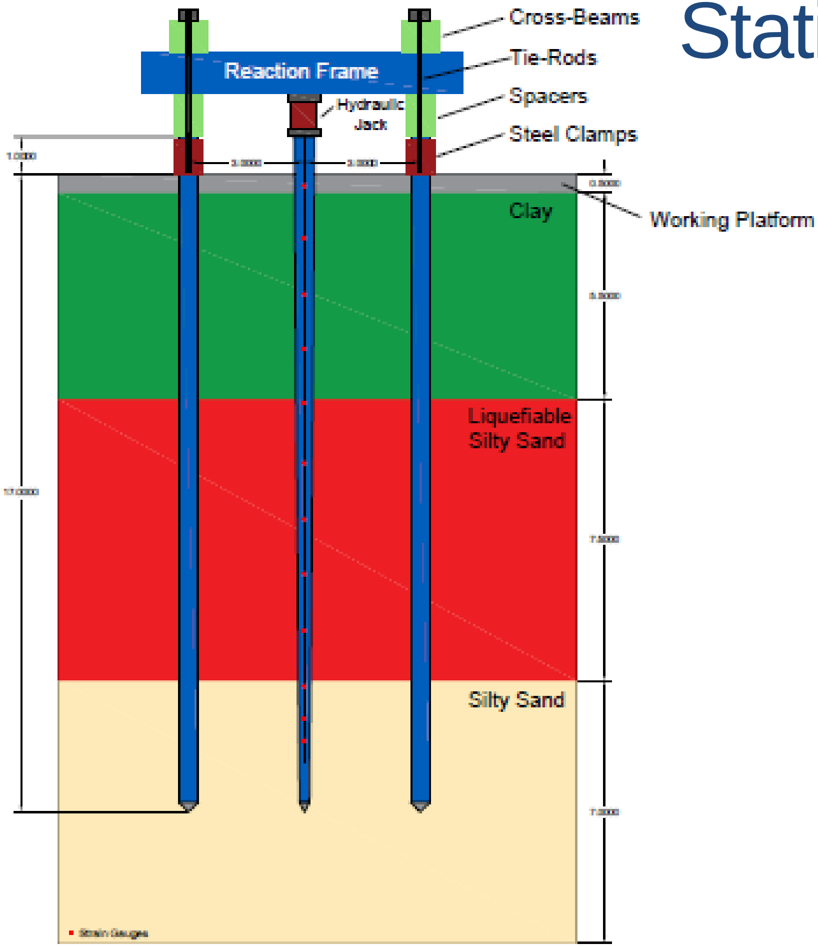
7 Strain gauged Piles

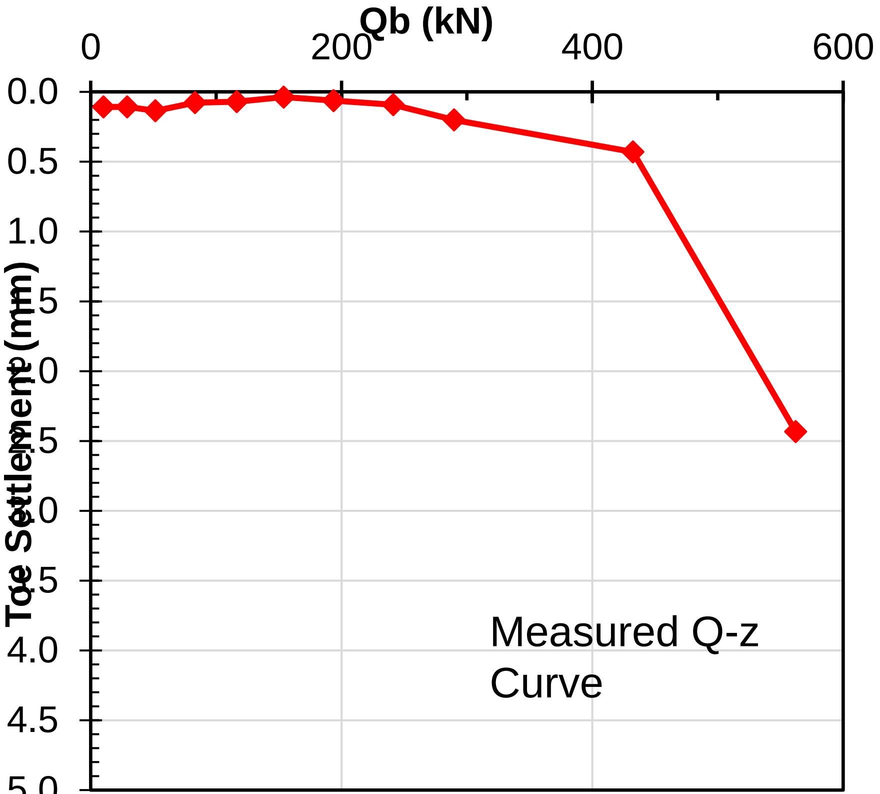
Load Test Results for Pile 20



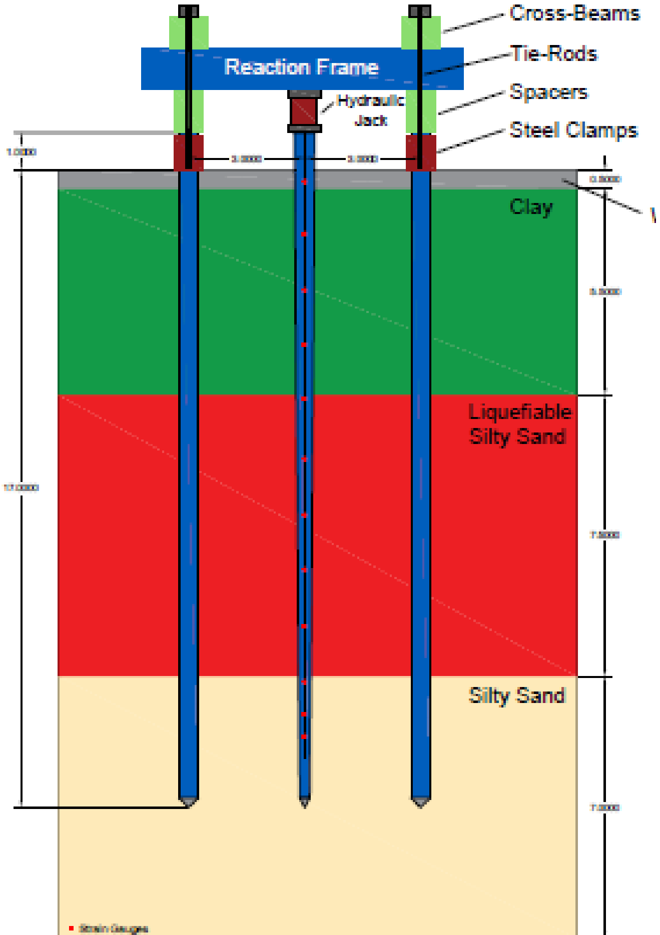


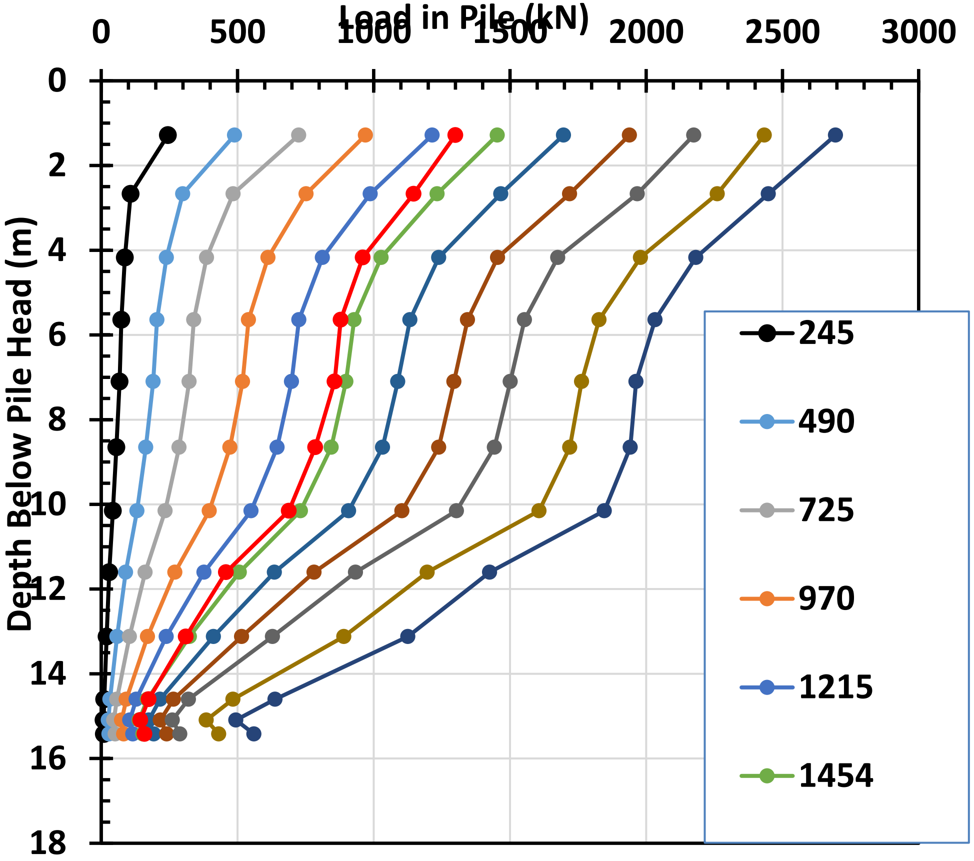
Static Load Test Results for Pile 20



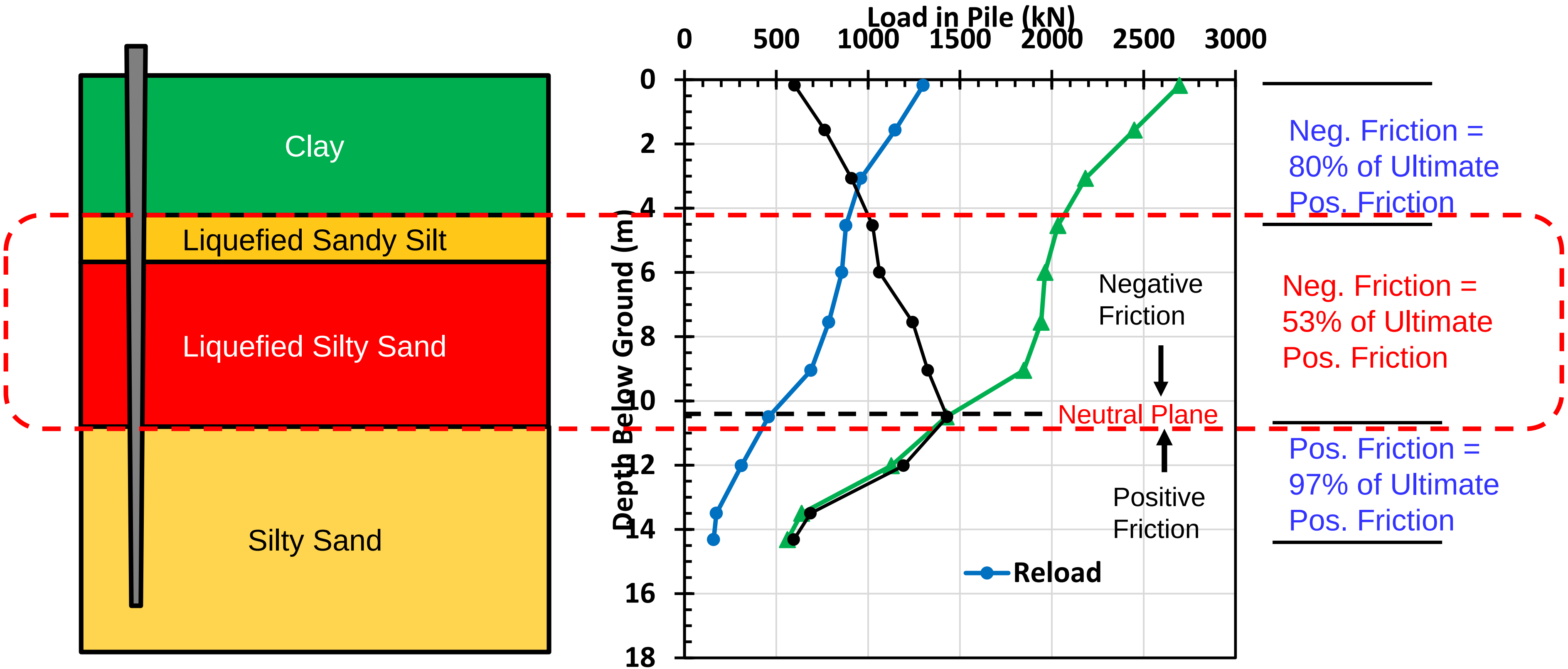


Load Test Results for Pile 20

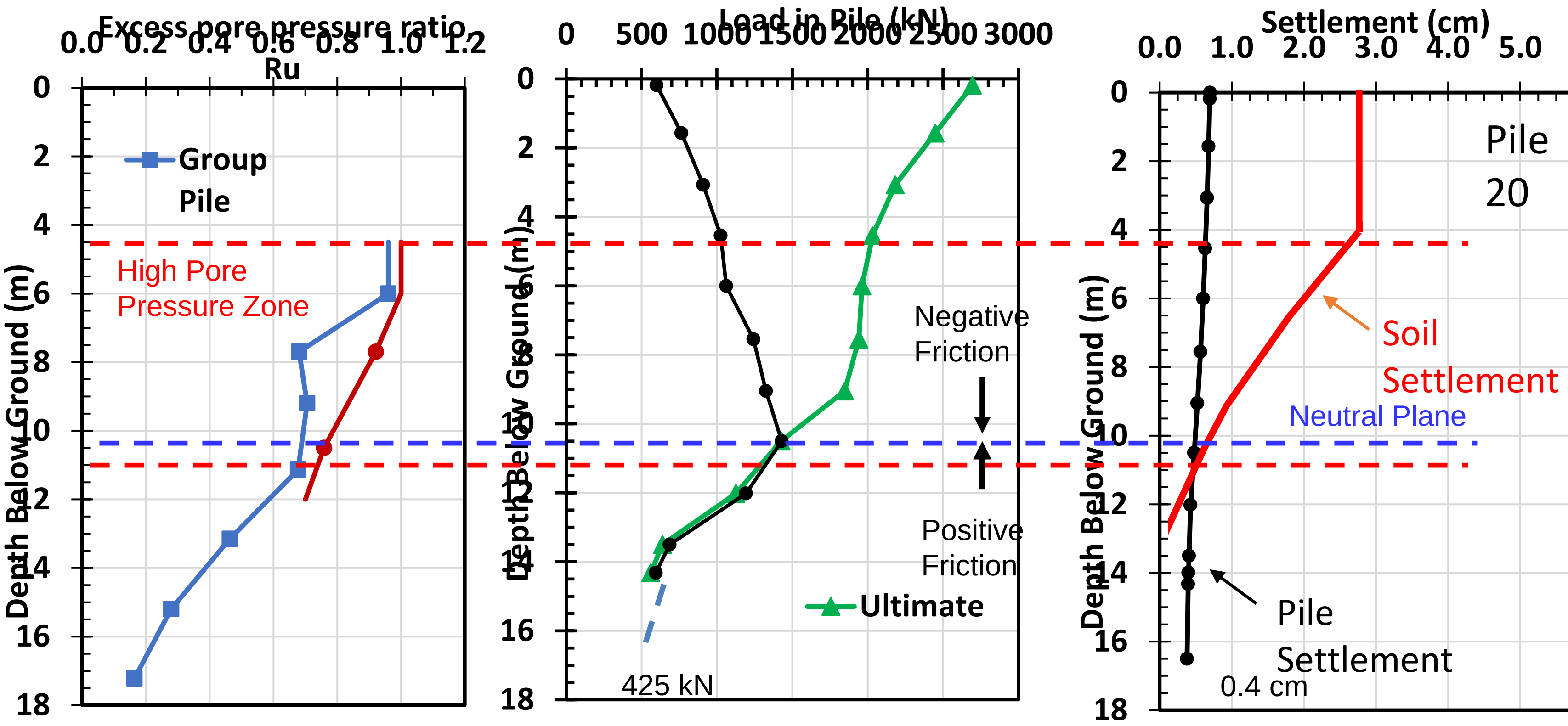




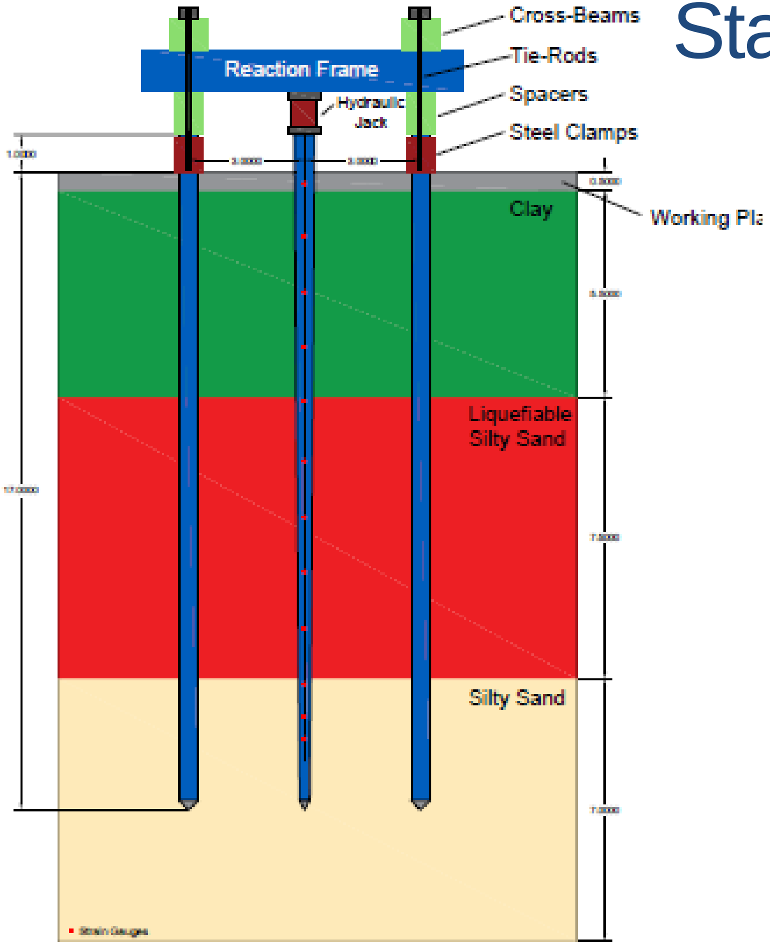
Blast Test Results for Pile 20

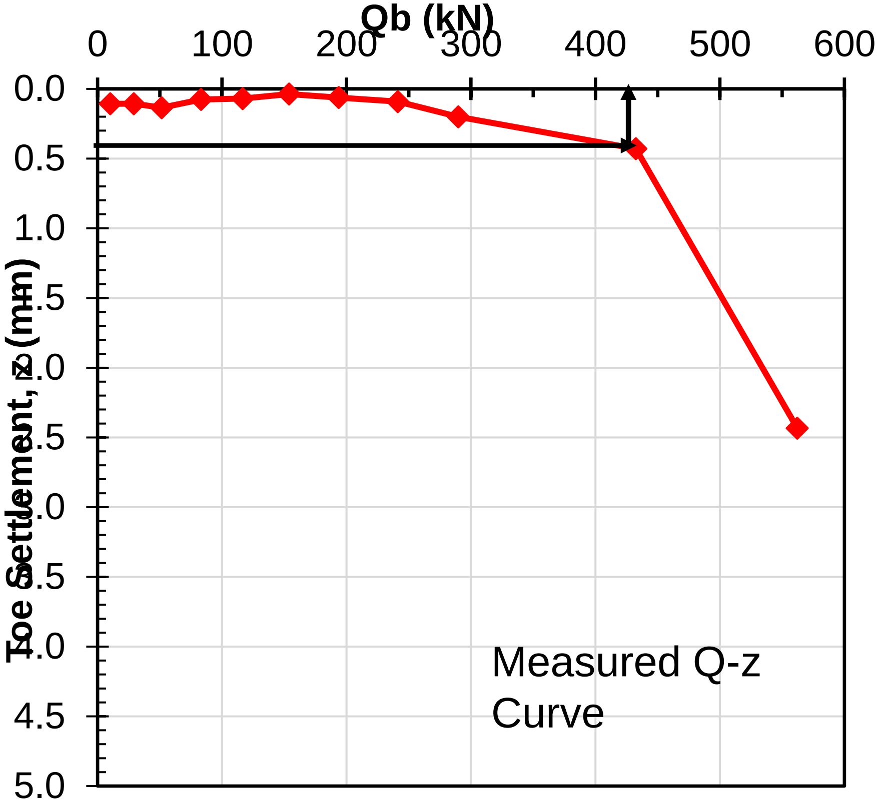


Blast Test Results for Pile 20

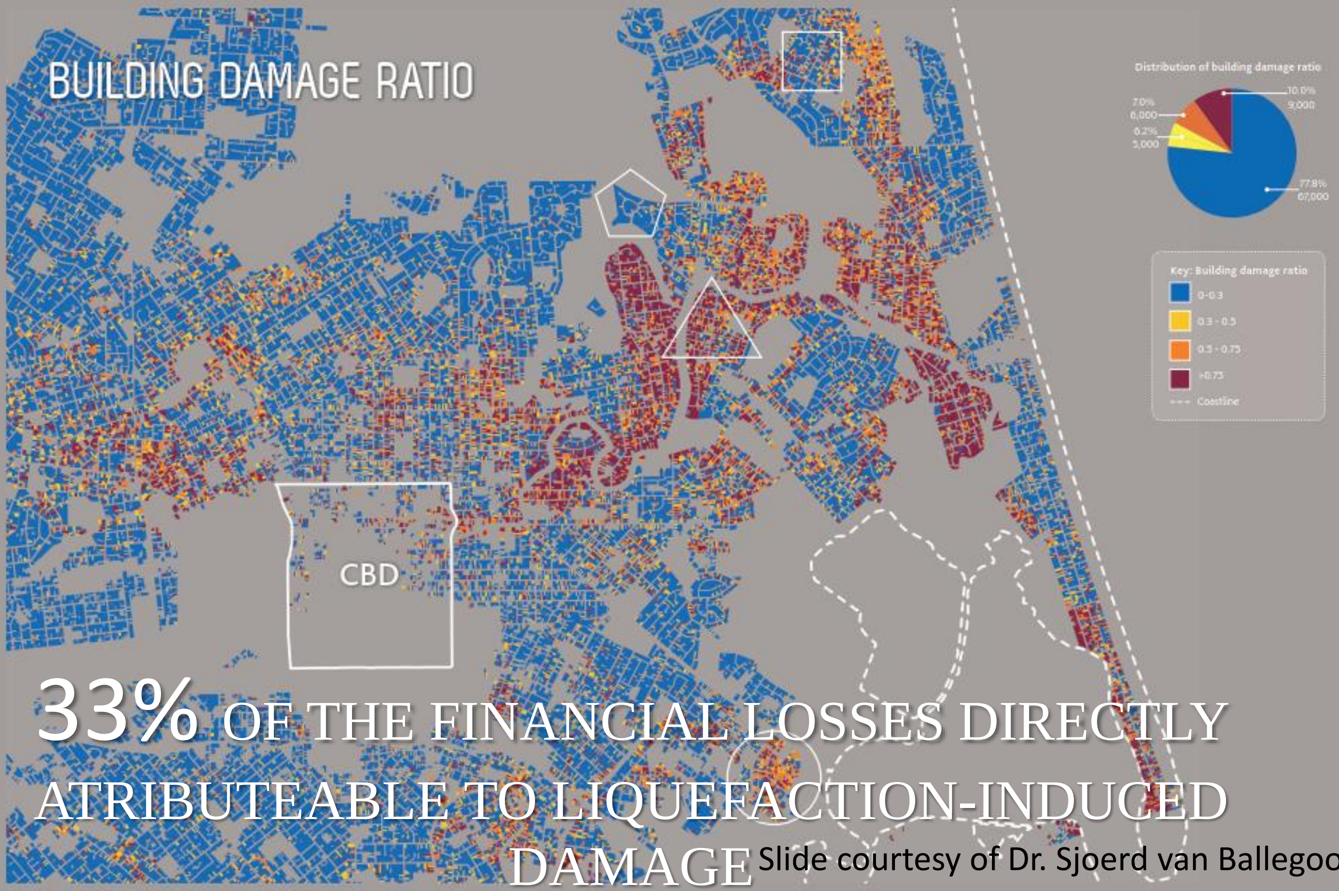


Static Load Test Results for Pile 20





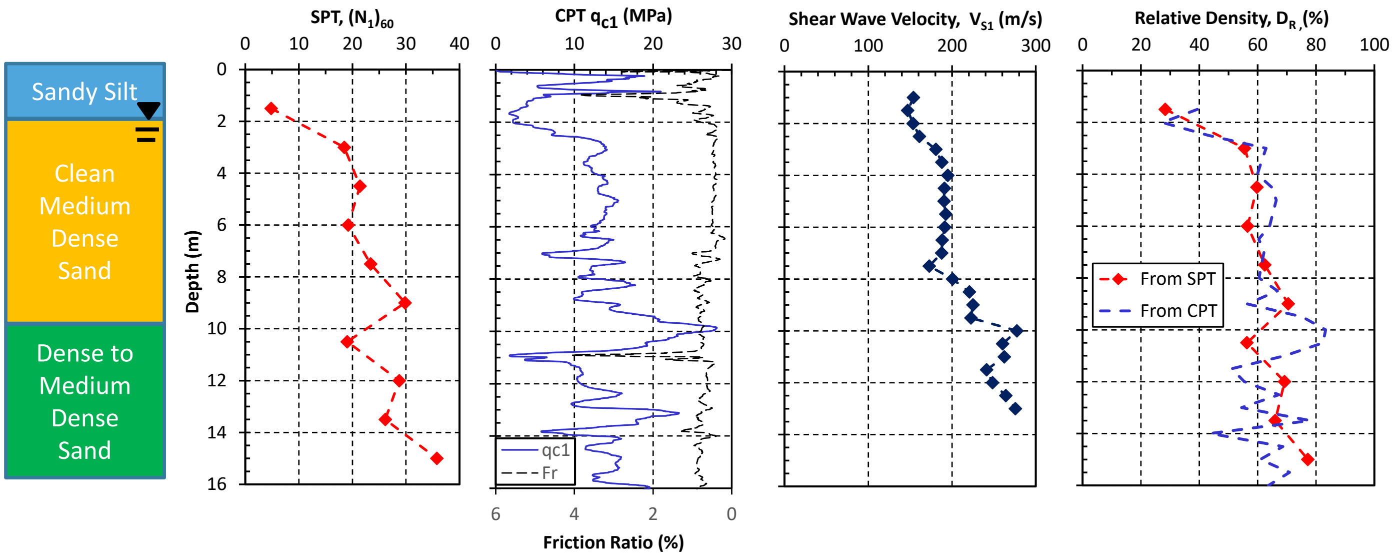
NZ$40B ECONOMIC IMPACT



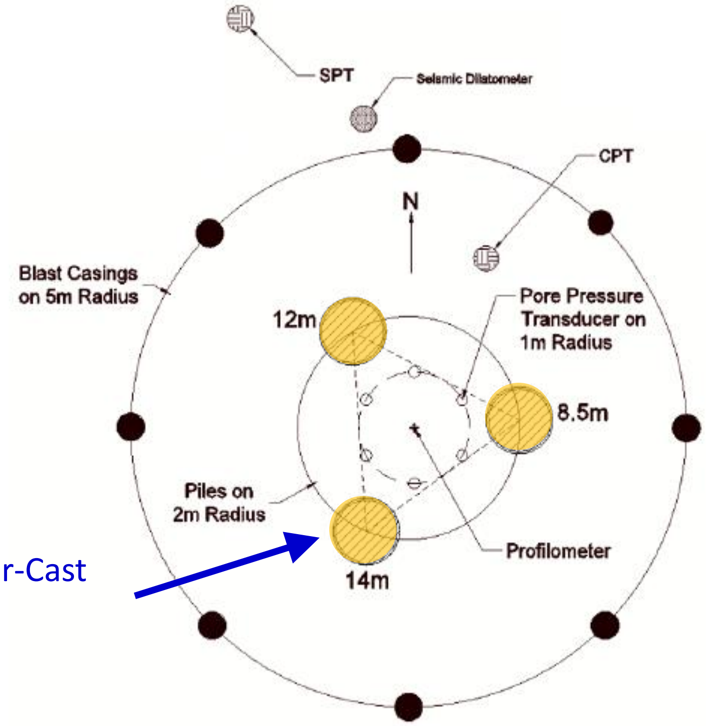
33% OF THE FINANCIAL LOSSES DIRECTLY ATRIBUTEABLE TO LIQUEFACTION-INDUCED DAMAGE

Slide courtesy of Dr. Sjoerd van Ballegooy

Geotechnical Profiles at Site 4 Christchurch

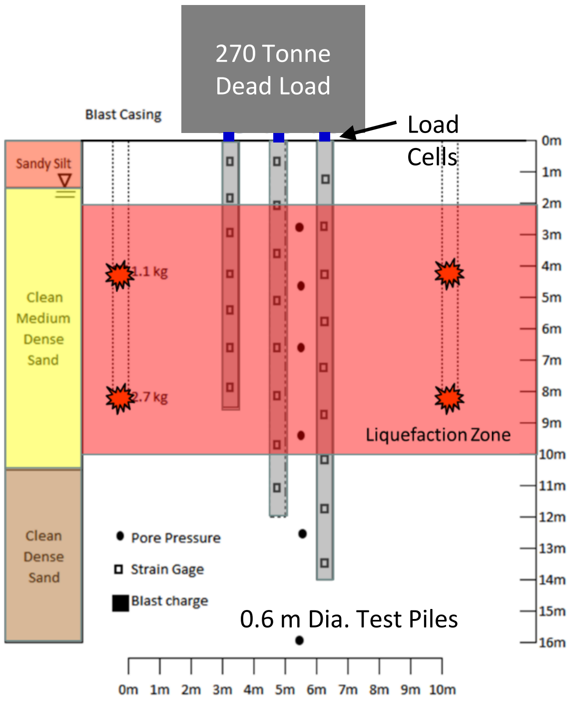


Christchurch Test Pile Layout



0.6 m Auger-Cast Piles

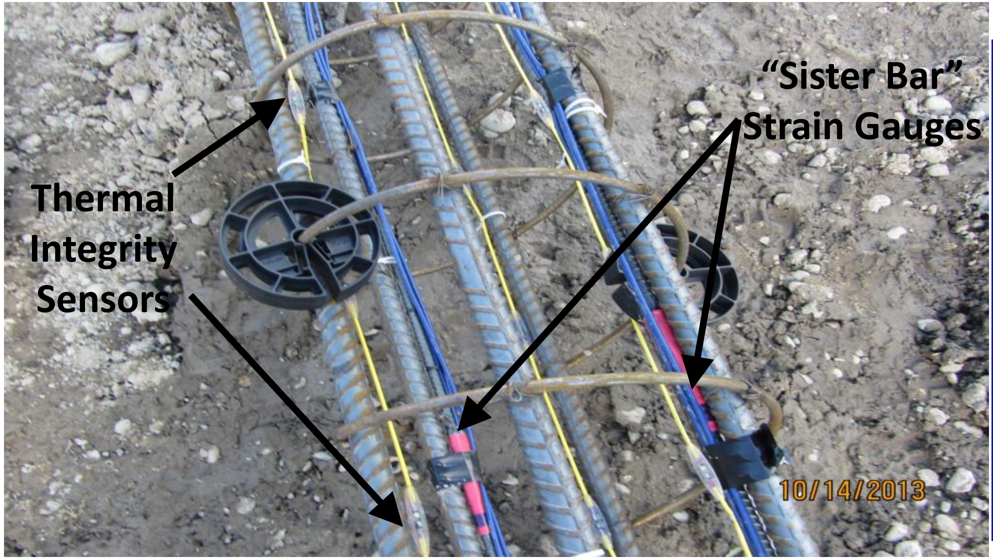
NZ Blast Liquefaction Induced Downdrag Tests

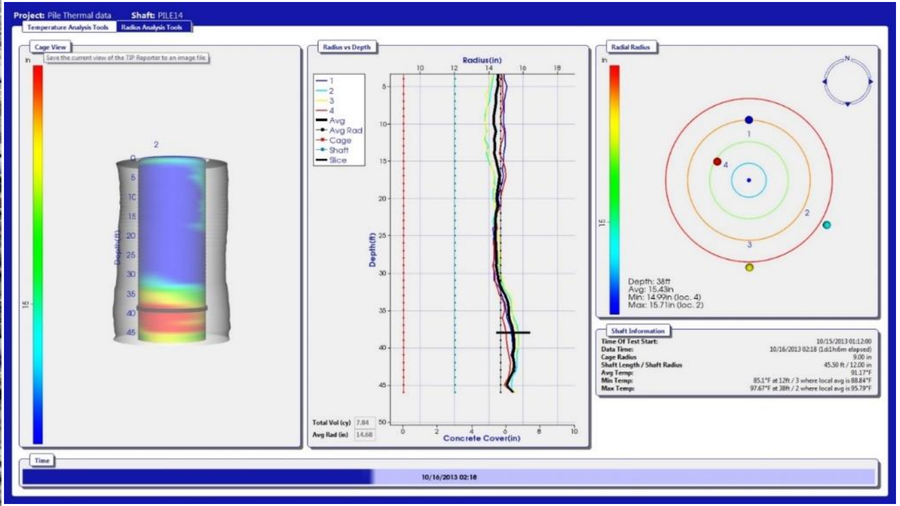


270 Tonne Load on Piles

Load on Test Piles in Christchurch – Dec 2013

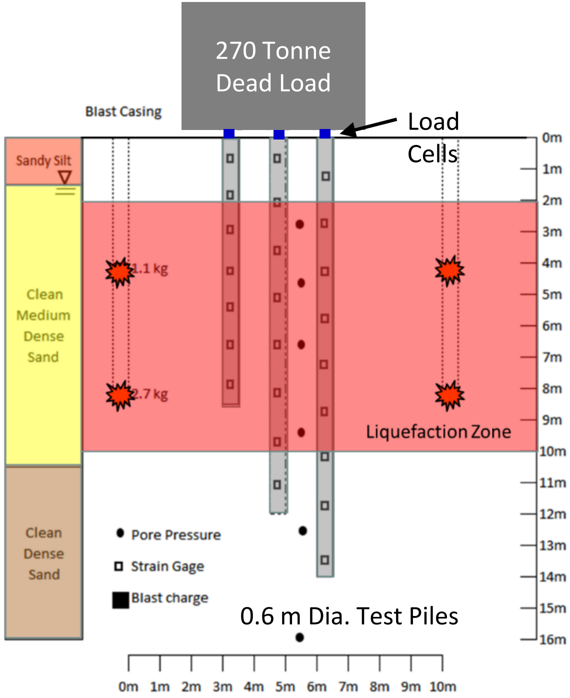
Auger Cast Pile Instrumentation





In reality, piles were 0.7 to 0.75 m in diameter not 0.6 m as designed

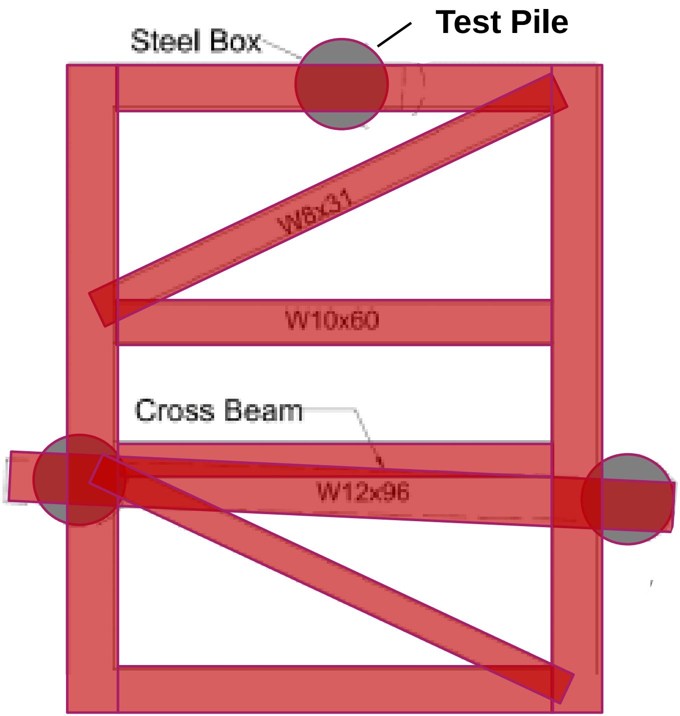
NZ Blast Liquefaction Induced Downdrag Tests

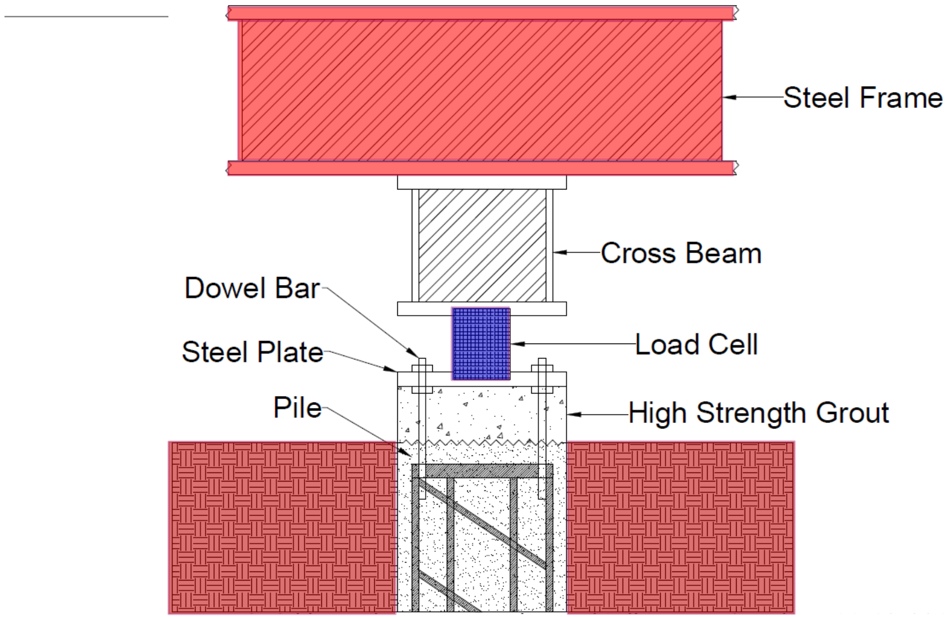


270 Tonne Load on Piles

Load on Test Piles in Christchurch – Dec 2013

Support Frame and Load Cell for each Pile





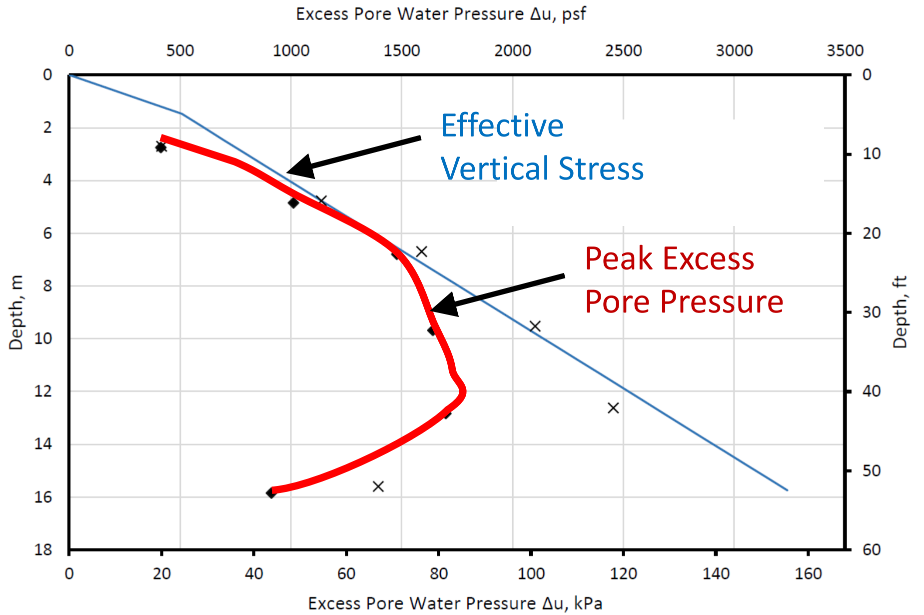
Plan View – Support Frame

Elevation View – Load Cell for each pile

Sand Boils Around Pile Group

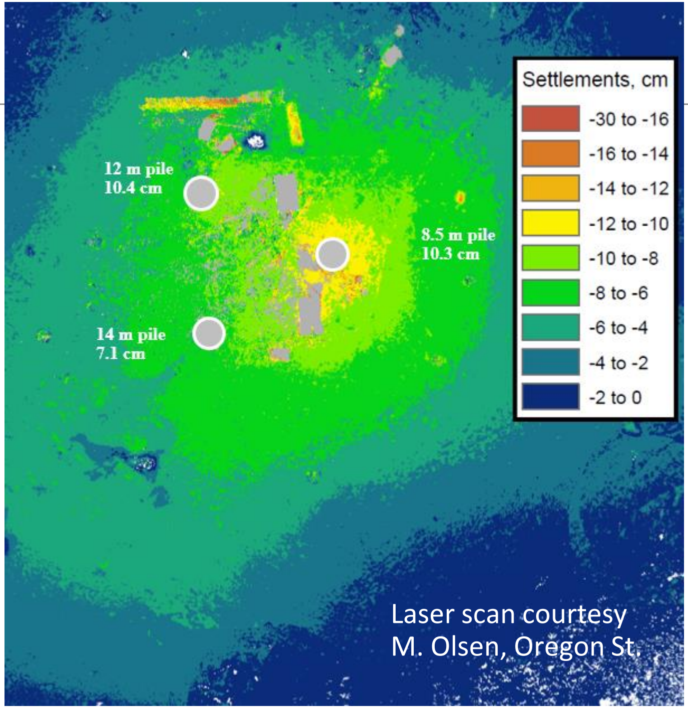
Test with Load on Piles

Excess Pore Pressure Ratio

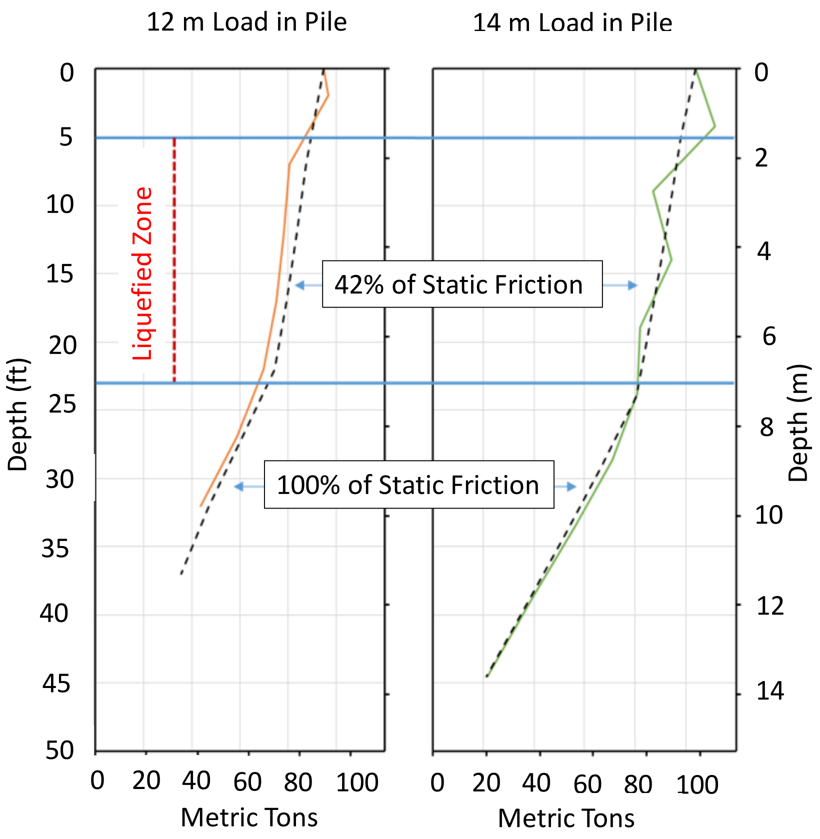


Test with Load on Piles

- Piles Settled 3 to 5 inches
- More Than the Surrounding Soil



Load in Pile vs. Depth - After Settlement



Conclusions Based on Single Pile Testing

- In non-liquefied soils, negative friction was equal to positive friction
- In liquefied soils, negative & positive skin friction after liquefaction & reconsolidation was 43 to 55% of ultimate skin friction before liquefaction
- These results are consistent for available tests (17 piles) and suggest that this may be a typical result
- Depth to neutral plane increased (and pile settlement decreased) as pile length increased
- Pile head settlement generally consistent with neutral plane concept.
- Reloading redevelops positive friction from top down


Come visit us in Utah!
Natural wonders that engineers can appreciate

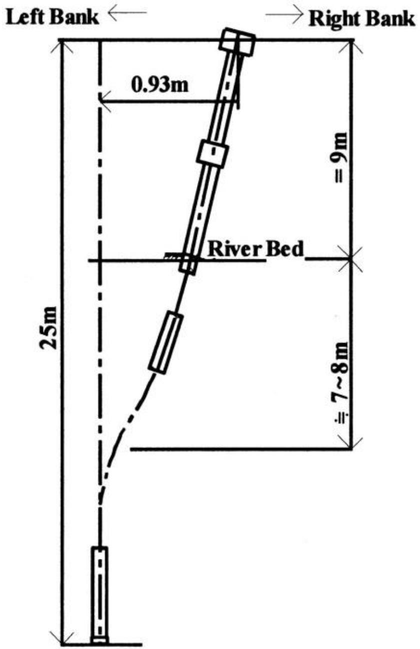
Design of pile foundations subjected to superstructure inertial loads and liquefaction-induced lateral ground deformations

Arash Khosravifar, PhD, PE (CA),
Associate Professor, Portland State University

TRB Webinar: Innovation in Geoseismic Foundation Design and Performance

May 23, 2023

Showa Bridge: 1964 Niigata earthquake



Damage to steel piles of Pier 4 of Showa Bridge (PWRI; from Yasuda & Berrill 2001)

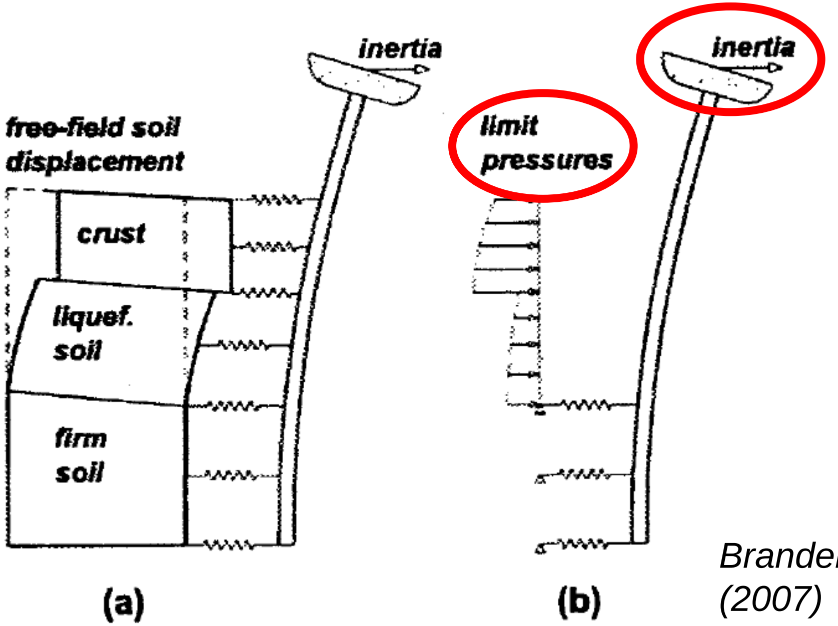
rtland State UNIVERSITY

- 1983 Nihon-Kai-Chubu earthquake: approach embankment failed due to lateral spreading but piles survived without damage (Finn 1999)

Portland State University

Problem Statement

- How to combine inertia (I) and lateral spreading kinematic (K) loads in design
- <u>No consensus</u> in design codes (next slide)
- Effects of ground motion duration and soil profile on I&K factors



Brandenberg et al. (2007)

Design Guidelines

- <u>No consensus</u> in design codes on how to combine inertial and kinematic loads on piles
- Load combination are site- and project-specific

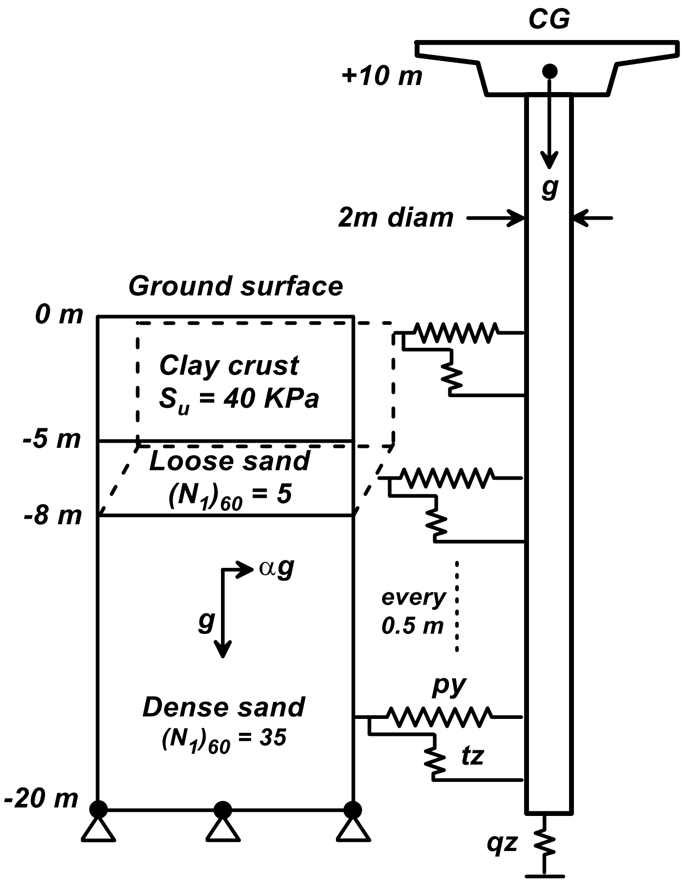
Design Code	Applicability
MCEER/ATC (2003) (ST)	Peak inertia is likely to occur early in the ground motion. Design piles for **<u>independent effects of inertia and lateral spreading</u>**. For large magnitude and long-duration earthquakes the two loads may interact.
AASHTO (2020) (ST)	Simultaneous inertial and lateral spreading loads **<u>only for large magnitude earthquakes (M>8)</u>**
PEER (2011) (ST)	**<u>100% kinematic + (65% to 85%) inertial</u>** (multiplied by 0.35 to 1.4 to account for the effects of liquefaction on peak inertial load)
Caltrans (2012), ODOT (2014), Supplement to Canadian Highway Bridge Design Code S6-14 (ST)	**<u>100% kinematic + 50% inertial</u>**
WSDOT (2021) (ST)	**<u>100% kinematic + 25% inertial (if M>7.5 contrib. >20% of hazard)</u>**
ASCE 61-14 (2014) Section C4.7 and Port of Long Beach Wharf Design Criteria (POLB 2015) (MT)	Locations of max moment from inertial and lateral ground deformation are spaced far enough apart that **<u>the two loads do not need to be superimposed</u>**. Max moments occur at different times. The two loads should be treated uncoupled for marginal wharves.
Port of Anchorage Modernization	Combine peak inertial with **100% peak kinematic** demands from

MT: Maritime Transportation ST: Surface Transportation

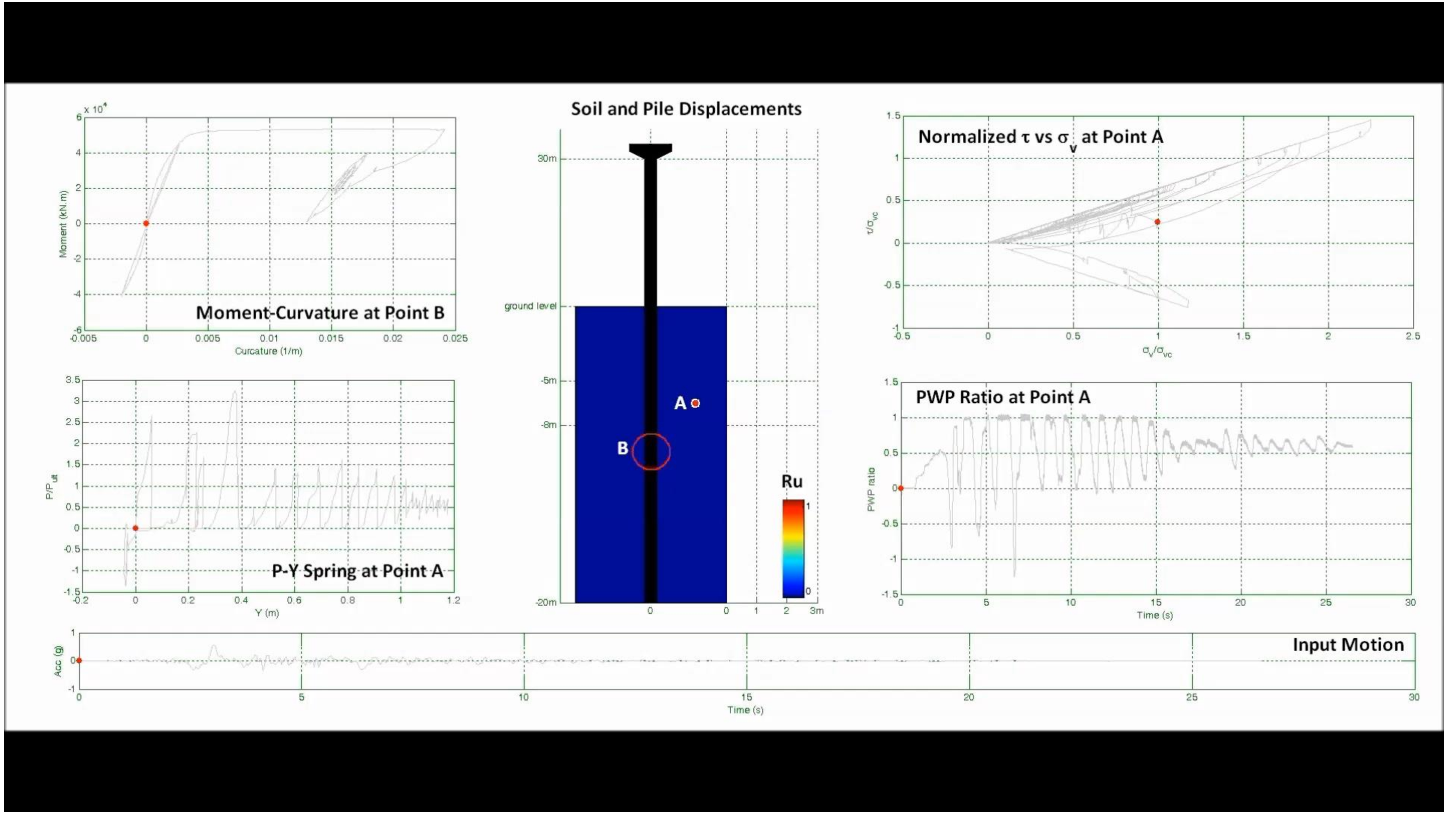
Nonlinear Dynamic Analysis (NDA)

FE model

- OpenSees FE framework
- 2D effective-stress analysis
- Soil elements (PDMY and PIMY)
- Pile elements (nonlinear)
- Interface elements (PYSimple and PYLiq)

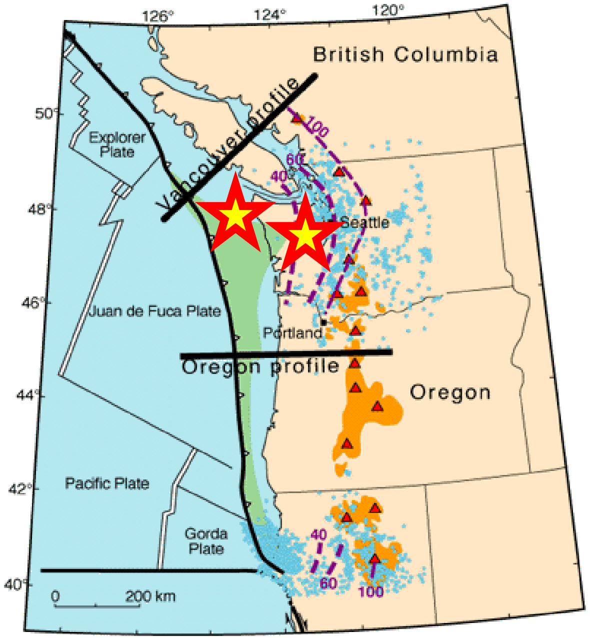


NDA Animation

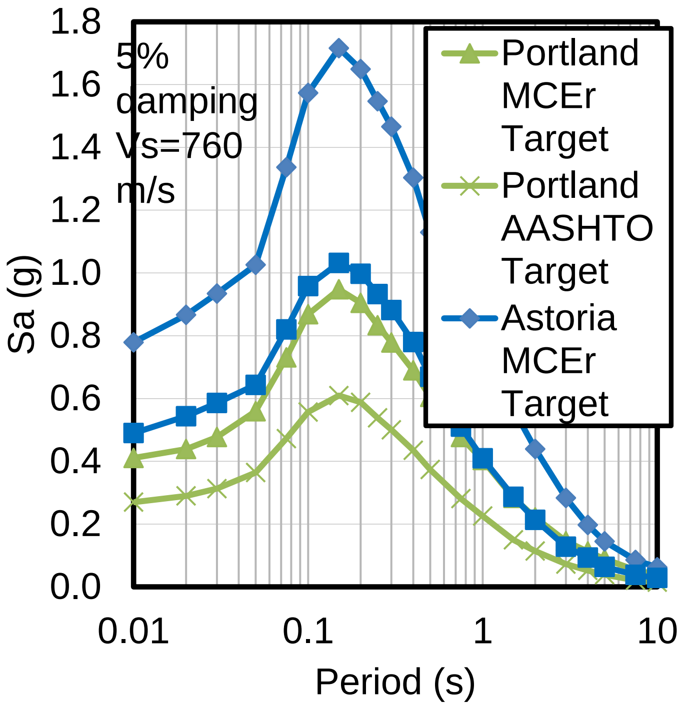


Site-Specific Ground Motions

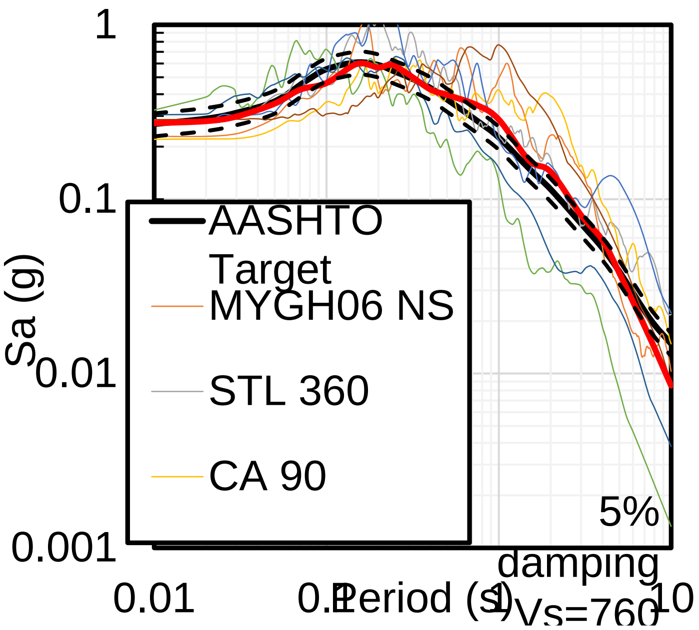
- Ground motions developed for two sites in Oregon
- Contributions from shallow crustal faults and Cascadia Subduction Zone

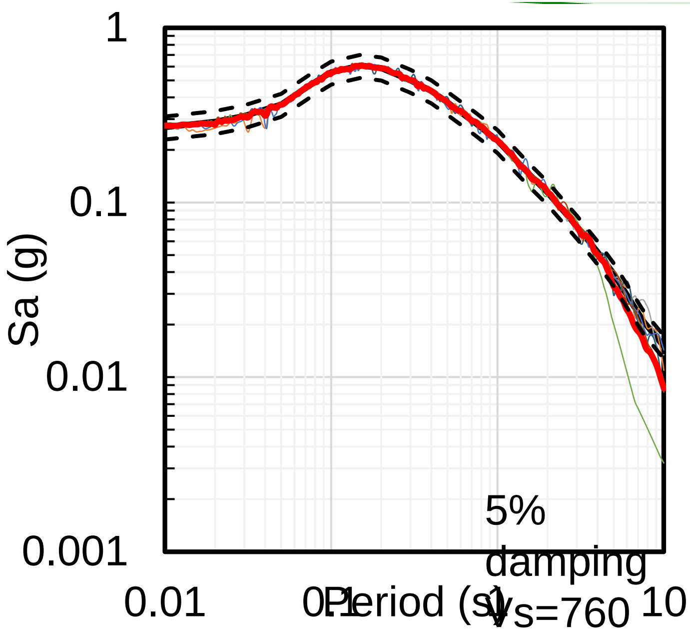


http://earthquake.usgs.gov/data/crust/cascadia.php



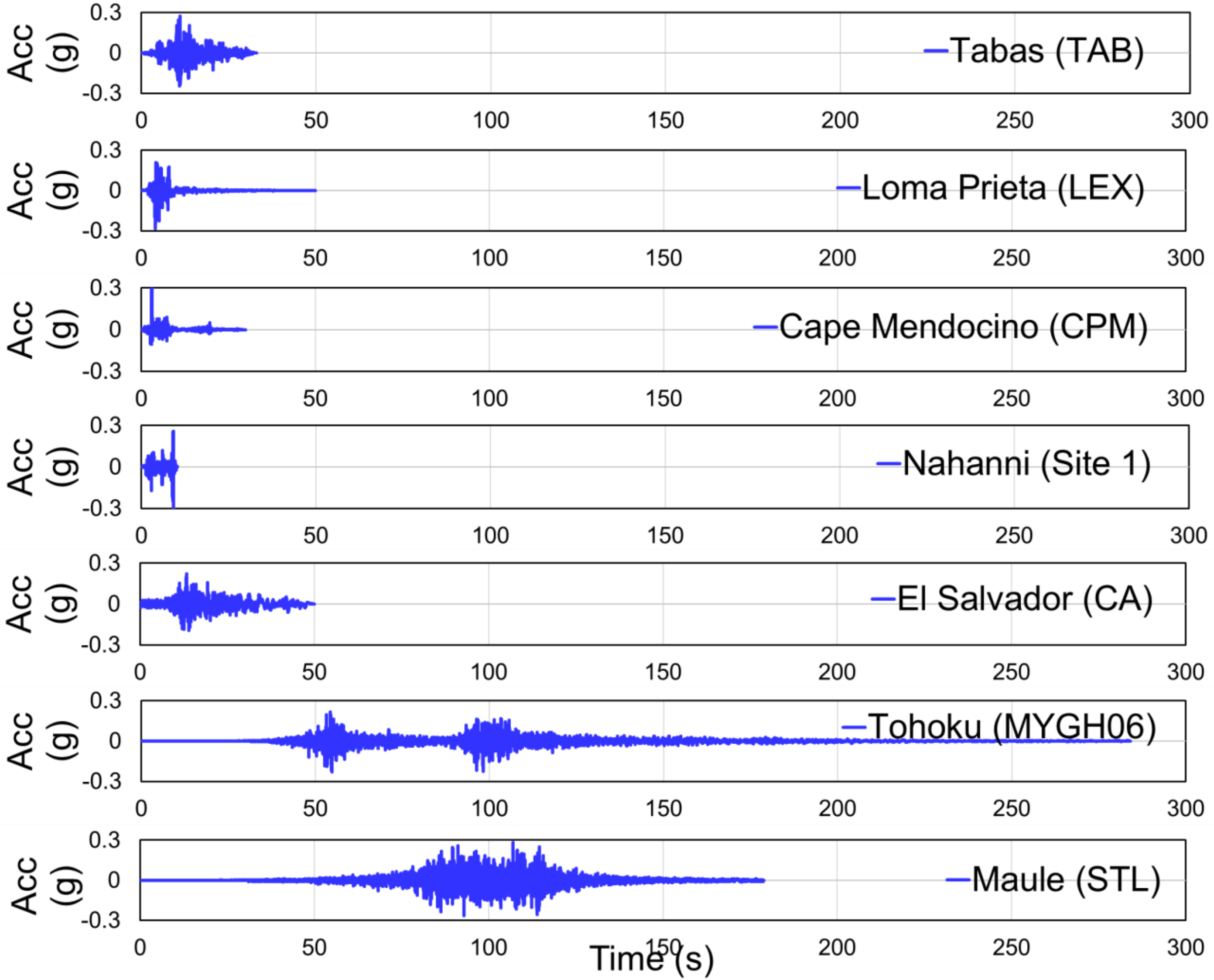
Ground Motion Selection, Scaling and Matching (Portland)





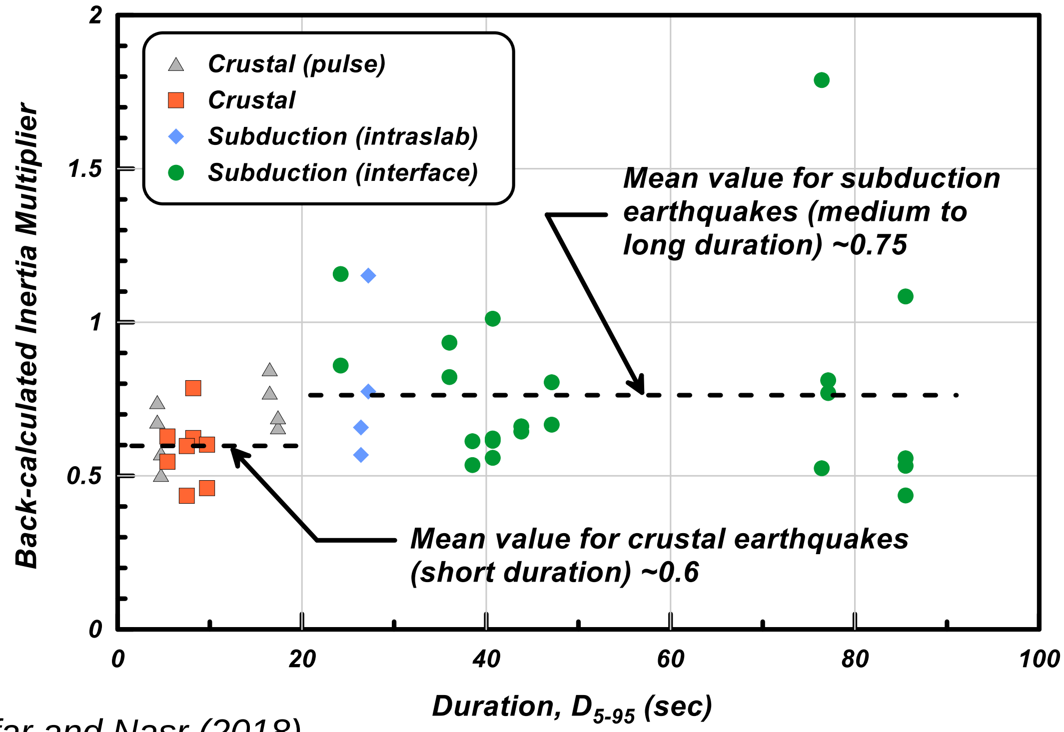
Event	Mag.	Dist. (km)	Vs30 (m/s)	Duration $D_{5\text{-}95}$ (sec)	Seed PGA (g)
2011 Tohoku (MYGH06)	9	63.8	593	85.5	0.27
2010 Maule (STL)	8.8	64.9	1411	40.7	0.24
2001 El Salv. (CA)	7.7	151.8**	Rock	27.2	0.1
1978 Tabas (T1)	7.35	2.05	767	16.5	0.87
1985 Nahanni (Site 1)	6.76	9.6	605	7.5	1.25
1992 Cape Men. (CPM)	7.01	6.96	568	9.7	1.51
1989 Loma Prieta (LEX)	6.93	5.02	1070	4.3	0.41

Ground Motion Selection & Scaling (Portland)



Effect of Long-duration motions on I+K

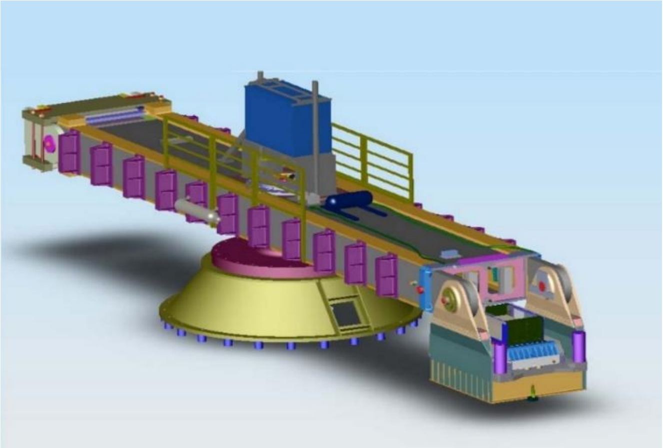
- 100% Kinematic + **60%** Inertial (crustal earthquakes with short duration)
- 100% Kinematic + **75%** Inertial (subduction earthquakes with medium to long duration)

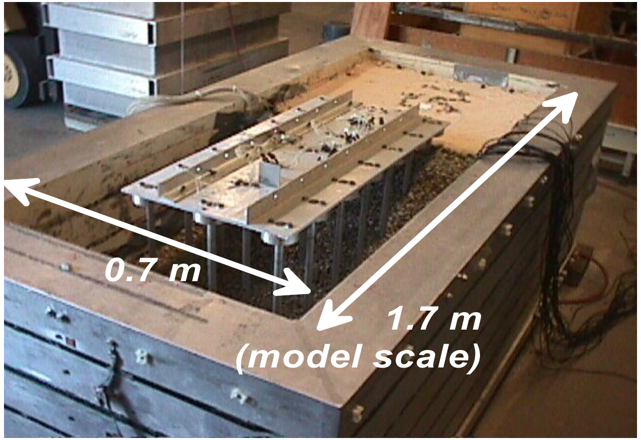


Khosravifar and Nasr (2018)

Lessons learned from pile-supported wharves

- Five (5) centrifuge tests by Dickenson, McCullough, and Schlechter from OSU (tests conducted at UC Davis)

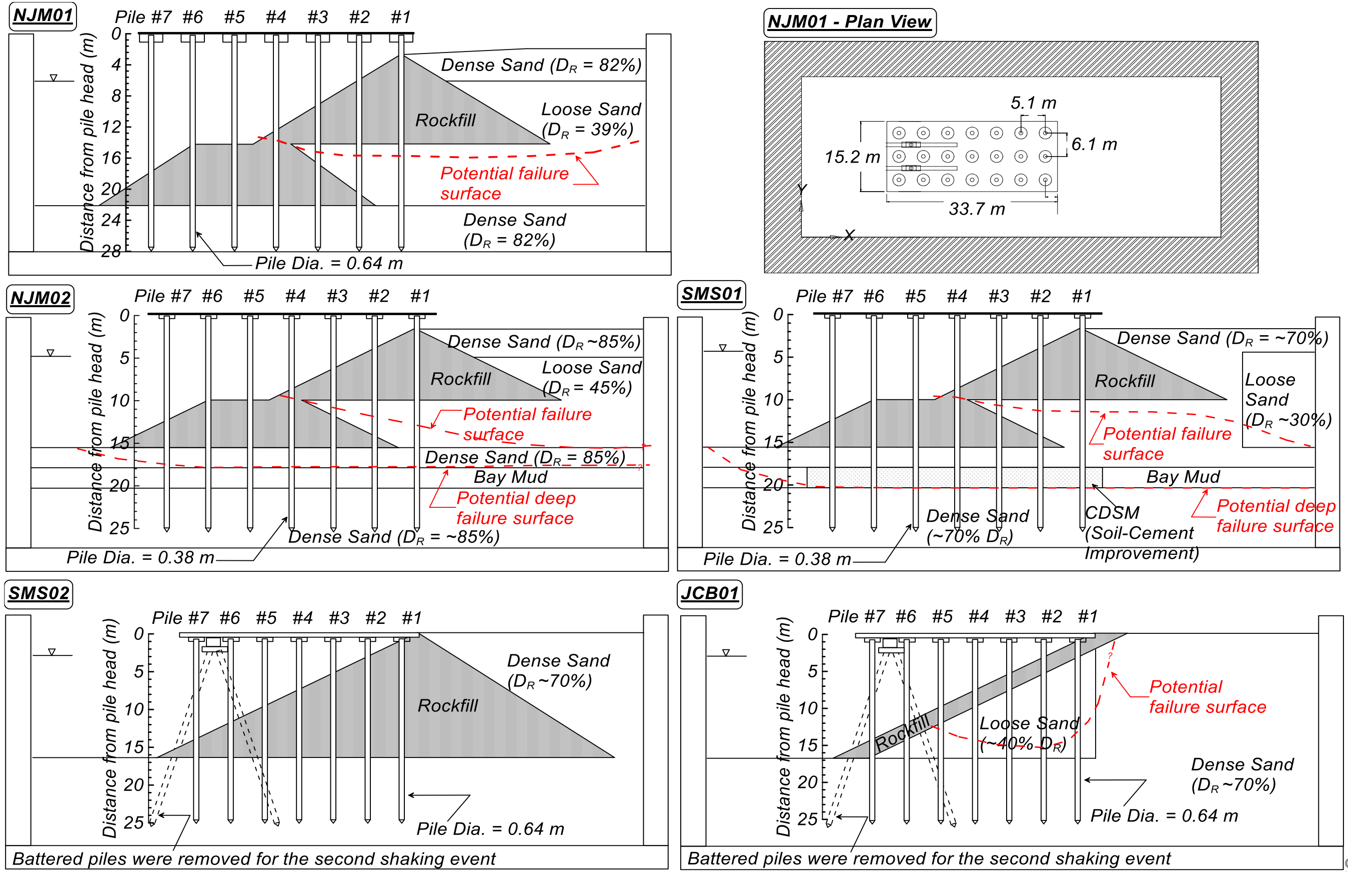




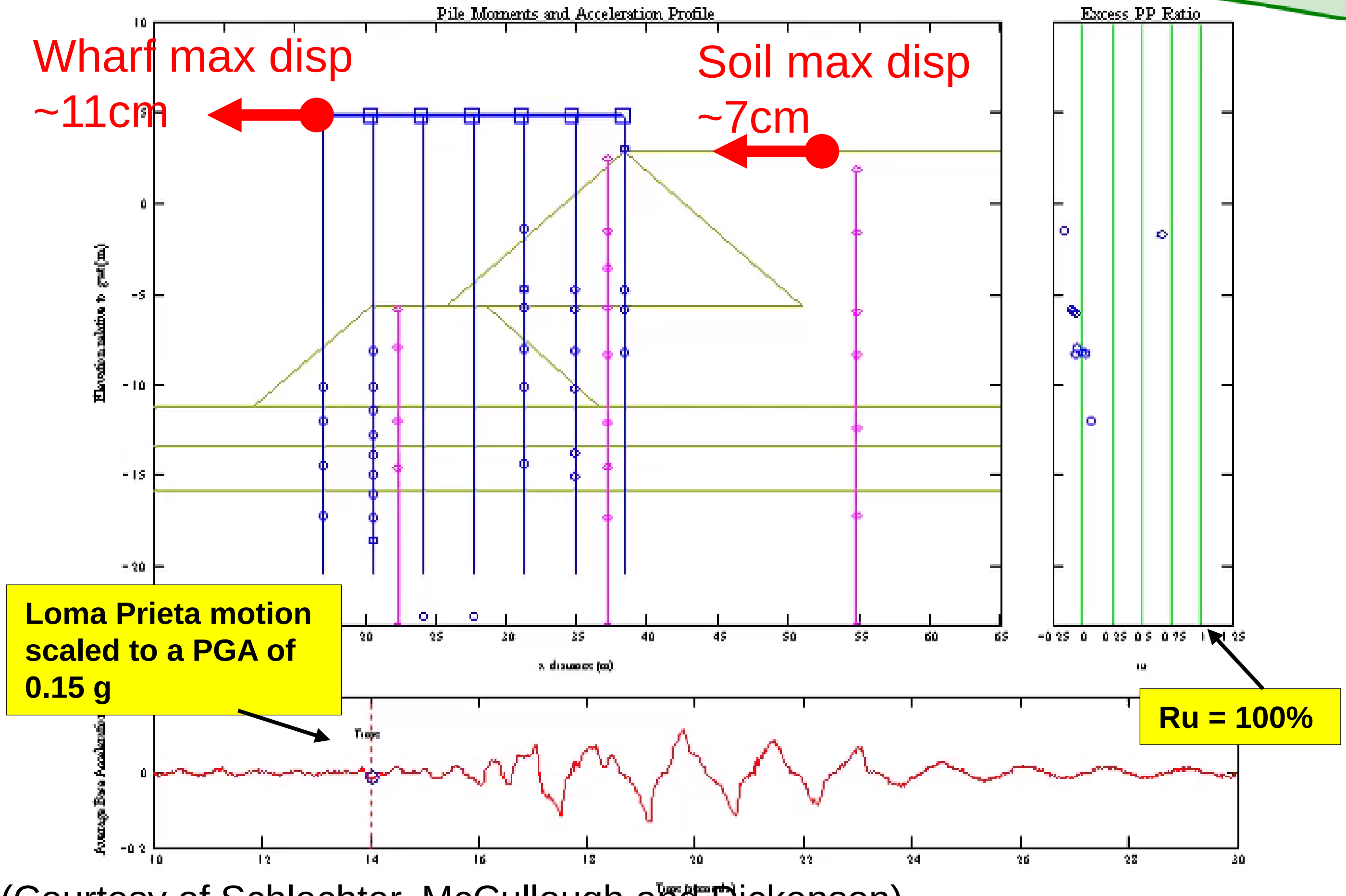
Source: Center for Geotechnical Modeling at UC Davis

Centrifuge Test Layouts

- 3-by-7 pile group
- 0.4 to 0.6 meter piles representative of concrete piles in prototype scale



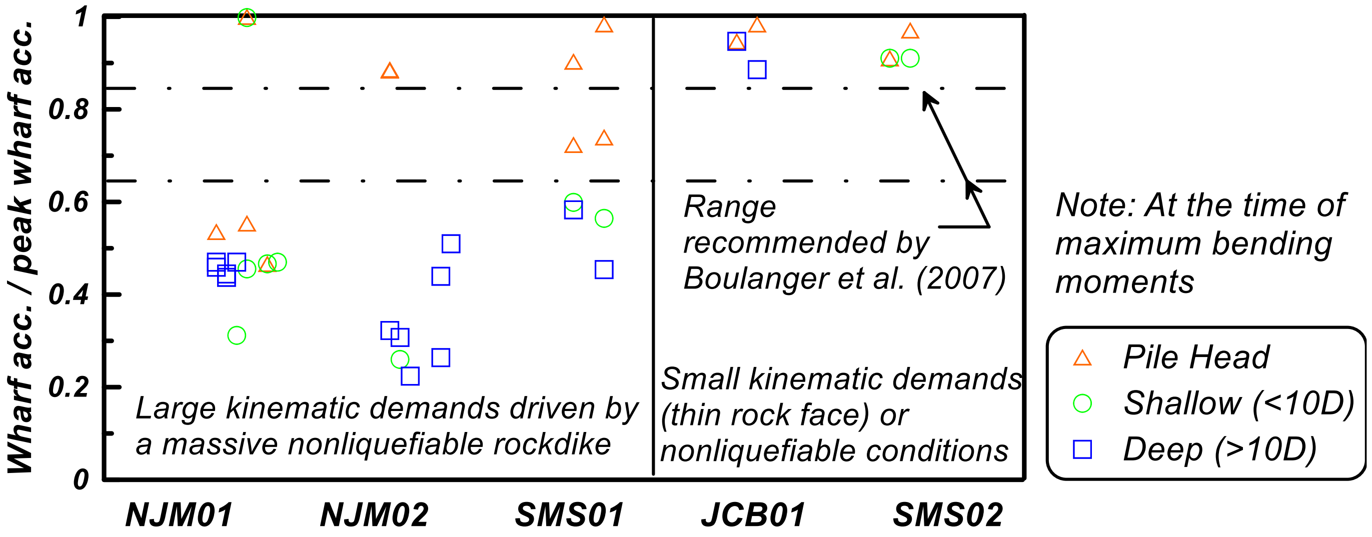
Representative Results (Animation)



(Courtesy of Schlechter, McCullough and Dickenson)

Inertial and Kinematic Interaction Factors

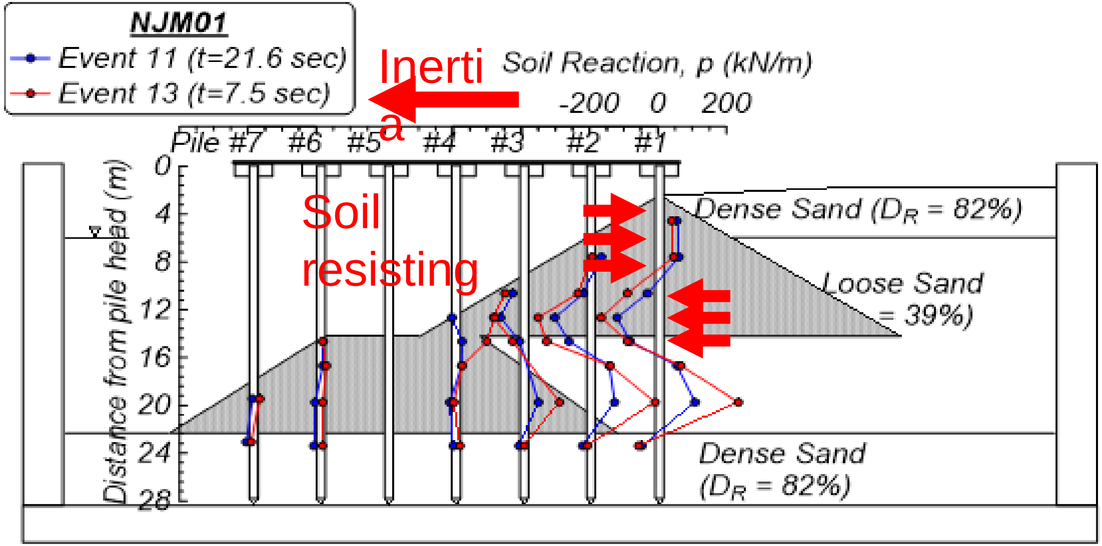
- 100% Kinematic + **X%** Inertia
- Dependent on soil profile
- Dependent on depth (larger when M is max at pile head and smaller for in-ground M)



Souri et al. (2022a,b)

Estimating Kinematic Demands

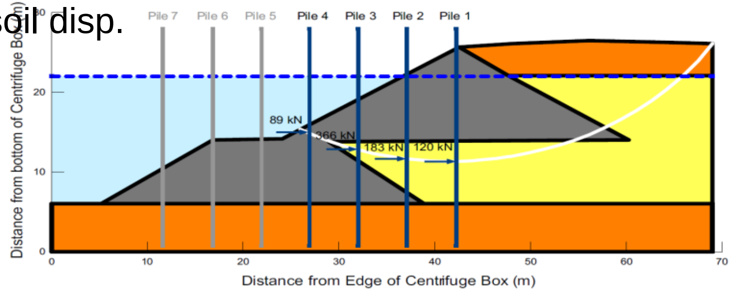
- Uniformly downslope ultimate soil pressure does not capture soil reactions for relatively flexible piles (0.64-m dia).
- Disp-base approach is recommended (i.e. apply pile-restrained soil disp. to end nodes of p-y springs)

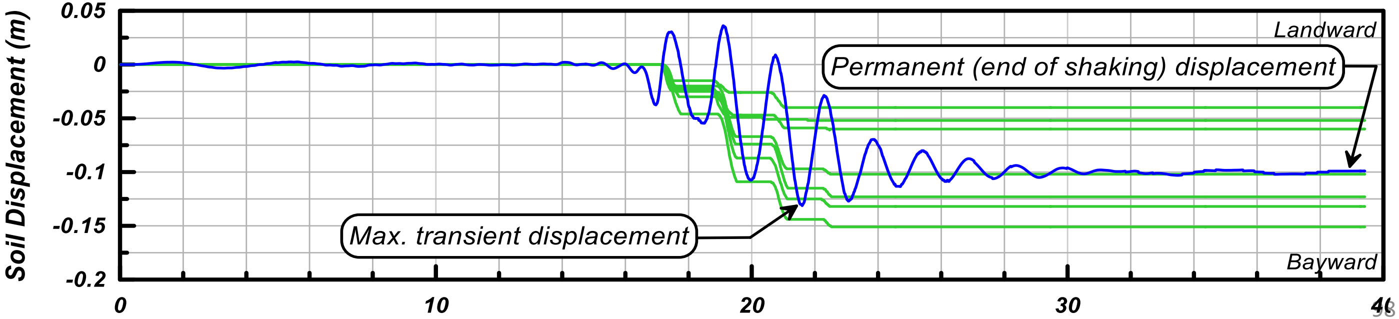


Estimating Kinematic Demands

- Newmark sliding block analysis to get pile-restrained soil disp. (Caltrans 2012)
- Lateral pile analysis (LPILE) and Limit Equilibrium analysis (SlopeW)
- Sr compatible with weighted approach by Kramer (2008)
- Median Newmark displacements
  - ✓ reasonably estimated permanent soil disp,
  - ✓ underestimated max transient soil disp.

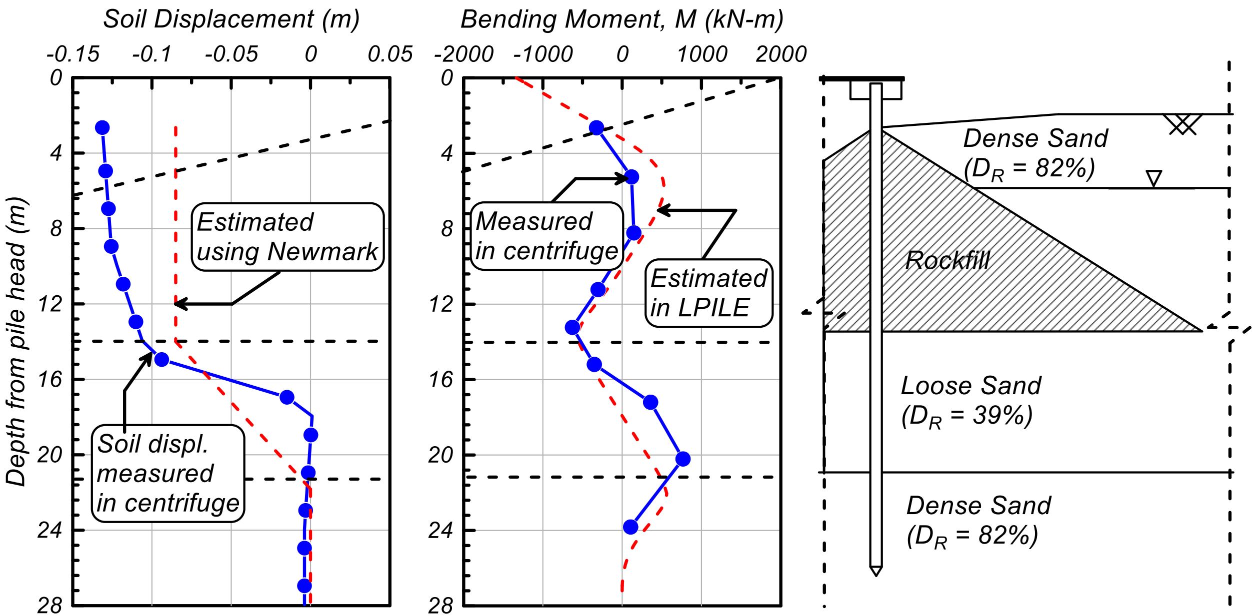
Torkelson et al. (2022)





Estimating Soil Displacements with Depth

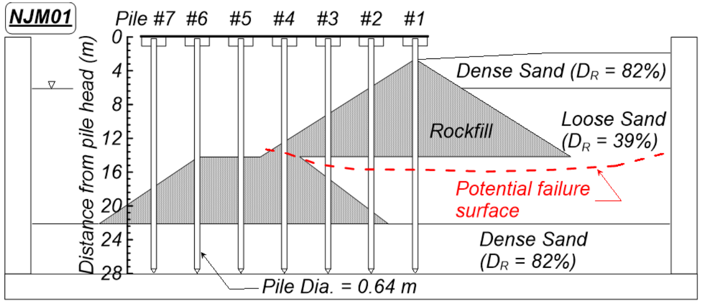
- Develop idealized soil disp. profile (e.g., Armstrong et al. 2014)
- P-y springs tapered over 1D at layer boundaries per 2011 PEER report
- Median Newmark disp. underestimated max transient disp.
- Idealized soil disp. overestimated pile curvature at layer boundaries
- Design recommendations: Use median Newmark values + idealized disp. profile



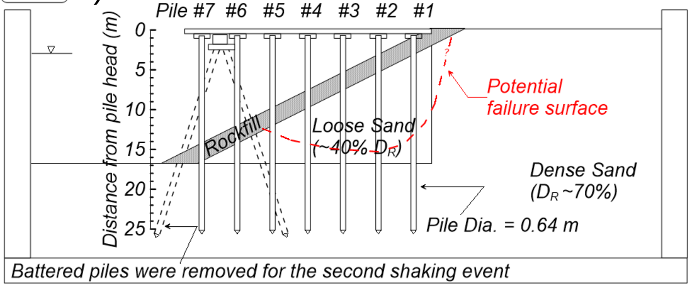
Design Recommendations

(Case) Load combination	Portion of permanent soil displacements applied at end nodes of p-y springs[1]	Portion of peak deck inertial force applied at deck[2]	Applicability
(A) Inertia only	NA	100%	Adequate to estimate bending moments at pile head.
(B1) Combined kinematic and inertial demands-Profile B1[3]	100%	0.3 to 0.6[5]	Suitable to estimate bending moments below grade down to depth of 10D.
(B2) Combined kinematic and inertial demands-Profile B2[4]	100%	0.9 to 1.0[5]	Suitable to estimate bending moments below grade down to depth of 10D.
(C) Kinematic only	100%	NA	Adequate to estimate pile bending moments deeper than 10D.

Profile B1: **deep-seated liquefaction underlying significant nonliquefiable crust** (i.e. rockfill).



Profile B2: **generally smaller kinematic loads** associated with either nonliquefiable profile or weak/softened soils closer to the ground surface, and thin nonliquefiable crust (i.e. sliver rockfill).



Conclusions

- Interaction of I+K is soil profile dependent
  - Bending moments at shallow locations (<10D) can be reasonably estimated by combining kinematic demands with a portion of peak deck inertial load ranging from **0.3 to 0.6** for deep-seated liquefaction profiles and thick nonliquefiable crusts (Profile B1) and **0.9 to 1.0** for weak/softened soils close to ground surface and thin nonliquefiable layer (Profile B2)
  - Bending moments adjacent to the pile head can be reasonably estimated by applying 100% inertia only, while bending moments at deep locations (>10D) can be reasonably estimated by applying the kinematic demands only

- Interaction of I+K increases with ground motion duration
  - **15% increase** for subduction earthquakes compared to shallow crustal earthquakes
  - Approach similar to WSDOT (2021) based on hazard contributions may be reasonable

Conclusions

- The wide range of inertial multipliers observed in this research highlights the benefit of performing coupled nonlinear dynamic analysis that captures complex soil-pile-structure interaction for varying soil profiles.
  - For example, POA (2017) peer-reviewed 2D nonlinear numerical analysis

- The load combination factors proposed here are appropriate for pseudo-static analysis using the p-y spring approach and are not necessarily appropriate for use with the simplified equivalent fluid pressure for lateral spreading load.

- It is recommended that the median displacements computed using Newmark-type analysis be applied in combination with an idealized soil displacement profile with distinct transitions.

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Thank you

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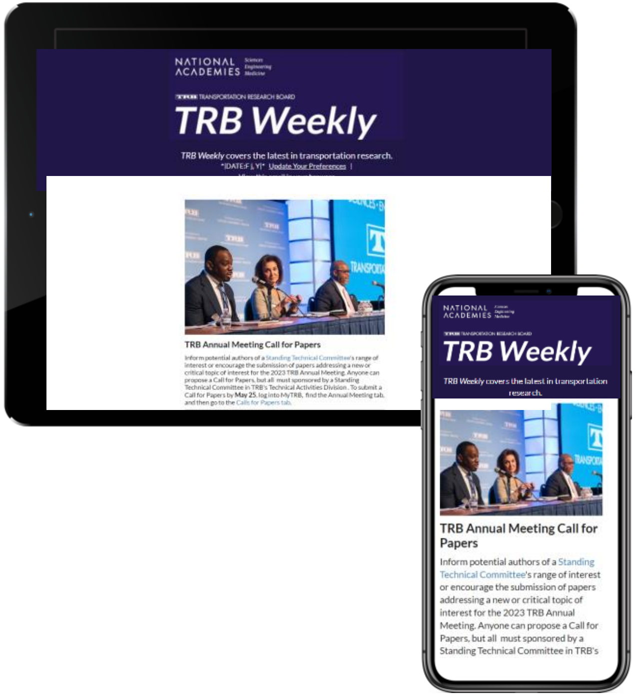

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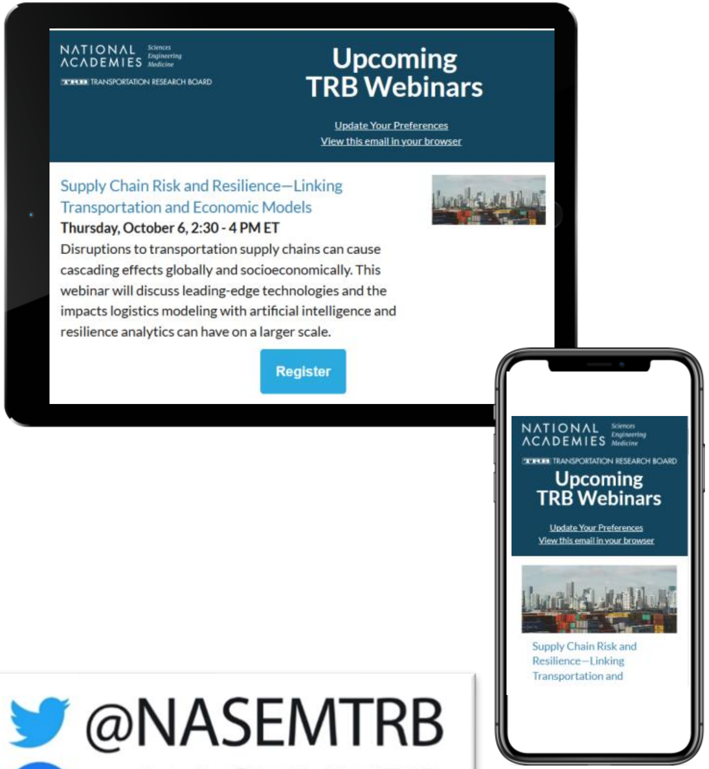

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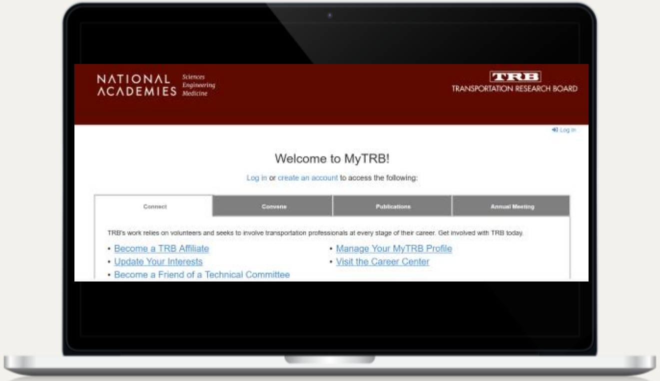

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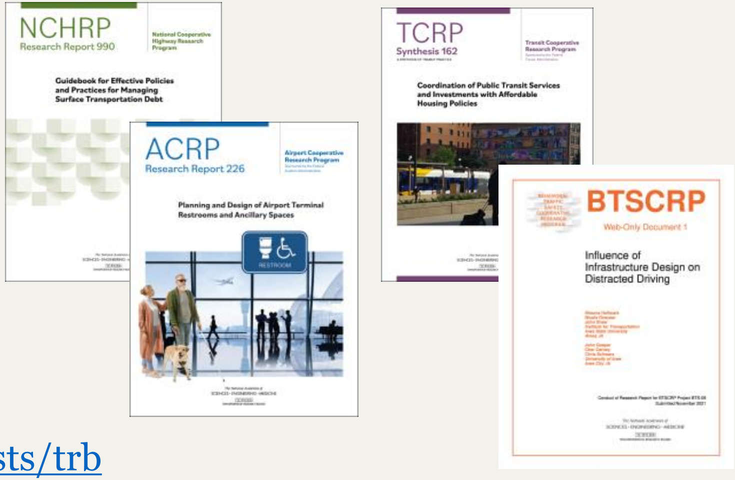

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