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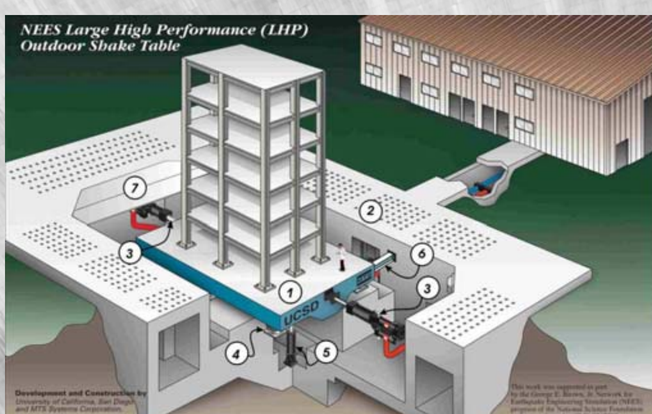
Report DSO-11-02

Soil-Structure Interaction Planning Phase 2 LHPOST Site Visit

Dam Safety Technology Development Program



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14. ABSTRACT
This ongoing research will help answer the questions that continue to arise as Reclamation is faced with detailed analysis and modifications of spillways and other earth retaining structures subjected to significant seismic loading. The objective of this research is a better analytical tool to predict the seismic lateral earth pressures for configurations that include groundwater, cohesion and compaction/in-place density effects under various ground accelerations. This report represents the completion of the first year of the second phase (Planning Phase 2) of this research project. The ultimate project purpose completing a full scale shake table test of a concrete cantilever retaining wall.

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Dam Safety Technology Development Program

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Mission Statements

The mission of the Department of the Interior is to protect and provide access to our Nation's natural and cultural heritage and honor our trust responsibilities to Indian Tribes and our commitments to island communities.

The mission of the Bureau of Reclamation is to manage, develop, and protect water and related resources in an environmentally and economically sound manner in the interest of the American public.

BUREAU OF RECLAMATION
Dam Safety Technology Development Program
Waterways and Concrete Dams Group, 86-68130

DSO-11-02

Soil-Structure Interaction Planning Phase 2 LHPOST Site Visit

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Introduction

A site visit was conducted to the University of California San Diego (UCSD), Camp Elliott Structural Engineering Test Facility to evaluate the capabilities of the Large High Performance Outdoor Shake Table (LHPOST). This site visit report was prepared to document activities and discussions of the site visit. The LHPOST shake table was identified during the Scoping Phase (FY2009) of this research effort as the most likely laboratory facility for completing a full-scale testing program for further investigation of soil-structure interaction (SSI) of concrete retaining walls under large magnitude seismic loading [1]. This site visit report further documents completion of the first year of the second phase (Planning Phase 2) of a proposed five-phase research project funded under the Dam Safety Technology Development Program.

Additional phases of the project are planned in subsequent fiscal years with annual progression toward a full-scale physical model test some time during FY2013. Subsequent project phases are briefly outlined below:

- Planning Phase 2 Year 2 – FY2011
    - Contracting the professional services of a practicing PhD research consultant
    - Completing a centrifuge model test using a cohesive soil with the help of the procured research consultant
    - Identification of funding sources for final design engineering of the full-scale shake table test
    - Identification of funding sources for construction of the full-scale shake table test

- Feasibility Phase 3 – FY2012
    - Validation finite element analyses of the cohesive soil centrifuge test
    - Feasibility level design development of the full-scale shake table test
    - Detailed costs estimates including reduced scope testing alternatives
    - Draft testing procedure including static and dynamic load requirements as well as soil types and properties
    - Detailed instrumentation and data acquisition requirements
    - Pre-test finite element analysis studies

- Final Design Phase 4 – FY2013
    - Final design level plans and specifications for the full-scale shake table test
    - Final testing procedure including static and dynamic load requirements as well as soil types and properties

  - Final instrumentation and data acquisition requirements
  - Final design level bid schedule and cost estimate
  - Final contract documents and bid package

- Laboratory Testing Phase 5 – FY2014
  - Contractor evaluation and procurement
  - Construction
  - Full-scale shake table tests and data collection
  - Data post-processing
  - Post-test finite element analysis studies
  - New design guidelines for design and evaluation of concrete cantilever retaining walls subjected to strong seismic ground motions
  - Final report and closeout

Problem and Background

Reclamation has numerous spillways and other earth retaining structures founded on soil and backfilled with various embankment materials, some in high seismic areas. Queries of Reclamation's spillway database indicate that there are approximately 35 gated spillway structures in Reclamation's inventory located in high seismic regions. Recent finite element analyses of spillway walls completed by Reclamation indicate dynamic loads on the walls due to seismic lateral earth pressures may be greater than or less than values predicted by traditional analytical methods, such as Mononobe-Okabe [5, 6] or Woods [8], depending on site-specific parameters such as foundation conditions and embankment alignment and depending on assumed model parameters such as numerical energy source and boundary conditions. However, verifying the computed seismic earth pressures from various finite element computer programs, such as FLAC [2] and LS-DYNA [3], has historically been problematic and unvalidated with physical model testing. Recent small-scale centrifuge model tests completed by others and supported by numerical analyses suggest that dynamic earth pressures are substantially less than those predicted by the traditional analytical methods [9]. Further, seismic performance of retaining walls, with a few exceptions, has generally been satisfactory, even for wall systems originally designed with no consideration of seismic loading.

Additional complications exist due to the limitations of traditional analytical methods. Specifically, inherit assumptions of the Mononobe-Okabe or Woods methods often result in difficulties analyzing and designing soil-structure systems that are prevalent in, not only Reclamation's spillway inventory, but the infrastructure inventory of many other federal agencies, such as the United States Army Corps of Engineers, the Natural Resources Conservation Service, the Federal Highway Administration, and other Department of Interior Agencies. Specifically, the Mononobe-Okabe method is limited to small to medium ground accelerations requiring that the seismic inertial angle (ψ) be less than or equal to the backfill's angle of internal friction (φ). Woods method is not limited by the magnitude of ground accelerations; however, Woods method assumes a non-yielding wall system with no groundwater within the backfill and no soil cohesion. The existing field conditions for many of Reclamation's spillway walls are inconsistent with these fundamental assumptions.

Efforts to either validate historical analytical methods or to develop new guidelines for dynamic soil-structure interaction (SSI) have increased substantially over the last several decades. Studies including physical model testing in conjunction with numerical analyses are prevalent throughout the technical literature. Physical model testing has traditionally consisted of small-scale model centrifuge testing with cohesionless sand backfill. The primary reason for focused efforts on scaled model testing is financial. Specifically, as presented herein, costs for developing and executing a full-scale shake table test of a cantilever concrete retaining wall are non-trivial and boundary effects associated with full-scale testing are problematic. Conversely, small-scale model centrifuge testing is far less expensive; however, small-grained dry sand backfill is exclusively used for such tests because of scaling effects associated with soil cohesion. Numerical analyses performed to verify physical model test results are often never completed independent of the model testing. Convenient dismissal of non-conforming physical model test data or adjusting of numerical model boundary conditions at the soil-wall interface to validate research results is common practice. In fact, recent Reclamation efforts, completed as part of this research effort [10], to validate centrifuge model test results have proven difficult with mixed results in terms of being able to reproduce physical model test results. As a result, no industry standard guidelines or methodologies have been developed to supplant the traditional Mononobe-Okabe and Woods methods.

The importance of the dynamic SSI issue for Reclamation is significant. Bradbury Dam is a good example of the potential far-reaching impacts regarding quantification of seismic lateral earth pressures. Bradbury Dam is a Reclamation-owned dam located on the Santa Ynez River approximately 25 miles northwest of Santa Barbara, California. The dam is a zoned earthfill structure, 279 feet high, with a crest length of 3,350 feet, and crest width of 40 feet at elevation 766.0. Between 1994 and 1995, dam safety modifications were constructed including downstream modifications of the embankment to eliminate the potential for failure of the dam due to earthquake-induced liquefaction of the foundation alluvium. In addition, the spillway crest structure was demolished and replaced with a new crest structure to accommodate the embankment modifications and to substantially increase seismic load carrying capability above those used for the original design in the early 1950s. The new crest structure was designed in the early 1990s using a deterministic approach consistent with standard engineering design practices at the time. As such, an effective pseudo-dynamic horizontal acceleration of 0.7g and corresponding dynamic soil loads suggested by a consultant review board were used for the new crest structure design.

Since construction of the new crest structure was completed in 1995, earthquake engineering has evolved significantly and transitioned from a deterministic approach to a probabilistic approach. Identification, evaluation and documentation of potential seismic sources have expanded exponentially with the progression of the internet and the personal computer. As a result, the current probabilistic seismic hazard for Bradbury Dam is significantly greater than the hazard used for design and construction of the 1995 modifications. Specifically, the current seismic hazard data points include:

- 3,100 year event (3.2×10^{-4}) = 0.7g peak ground acceleration (PGA)
- 10,500 year event (9.5×10^{-5}) = >1.0g PGA
- 50,000 year event (2.0×10^{-5}) = 1.6g PGA

Risk analyses recently completed using the current seismic hazard indicate that risks are above Reclamation's public protection guidelines [11] and the possibility of a second rehabilitation of the spillway crest structure fifteen years following construction of the first rehabilitation is possible.

There are an estimated 35 gated spillway structures in Reclamation's inventory located in high seismic zones as part of embankment dam projects that could potentially require future remediation if seismic hazards continue to increase as they have over the past several decades. Several recent design modifications have incorporated extremely conservative, limit state approaches to dynamic SSI. Specifically, rehabilitation projects located in high seismic areas, such as Deer Creek Dam and Echo Dam, have utilized lateral earth loads based on static, limit state passive lateral earth pressure coefficients to avoid the limitations of traditional dynamic methods and conflicting results from dynamic numerical modeling. For a typical fifteen-foot high cantilever retaining wall with cohesionless backfill and a pseudo-dynamic horizontal acceleration of 0.6g, the ratio of total computed shear at the base of the wall stem using passive limit state pressures to that computed using the Mononobe-Okabe (M-O) method (static active pressure plus dynamic pressure) is 1.8.

One approach to mitigate this problem is to determine if seismic lateral earth pressure loads resulting from increased seismic hazards are less than those determined using traditional methods or, in certain instances, less than those predicted by high-end numerical analyses. Completion of a full-scale concrete retaining wall shake table test could provide valuable insight in this regard and could potentially save millions of dollars in rehabilitation construction costs for Reclamation spillway walls.

This SSI research project, at the completion of all five phases, will help answer the questions that continue to arise as Reclamation is faced with detailed analysis and modifications of spillways and other earth retaining structures in areas of increased or high seismicity. A better analytical tool to predict the seismic lateral earth pressures for configurations that include groundwater effects, cohesion effects and compaction/in-place density effects under various ground accelerations is the objective of this proposed research.

The proposed research project will culminate with a full-scale model testing program to verify analytical results and either establish new evaluation and design standards for SSI or confirm criteria established by historical methods.

FY2009 Scoping Phase 1 Summary and Conclusions

The Scoping Phase 1 of this research project was completed in FY2009 and consisted of a literature review and compilation of technical references regarding seismic SSI and focused specifically on concrete retaining structures subjected to dynamic lateral earth pressures. The literature review included widely-accepted historical methods as well as recently completed studies focused on numerical modeling and physical testing. One of the primary objectives of the literature review was to determine if a full-scale concrete retaining wall shake table test has previously been performed and documented within the United States research community or overseas, and to identify sources of existing test data on the measured response of concrete retaining walls under seismic loads.

The FY2009 Scoping Phase 1 also consisted of evaluating laboratory capabilities at Reclamation and other research facilities to determine the feasibility of dynamic testing of a full-scale cantilever concrete retaining wall. Once an appropriate facility was identified, an appraisal-level design configuration for testing a full-scale model was developed.

The findings of the FY2009 Scoping Phase 1 study can be summarized as follows:

- Due to increased seismic hazards in high seismic areas, Reclamation spillway walls will continue to be a source of high risk and potentially costly mitigation efforts without addressing the considerable knowledge gap that currently exists regarding quantification of seismic lateral earth pressures.

- Historical analytical methods and more recent physical test model studies completed in conjunction with numerical analyses fail to independently and adequately validate research results in the field of dynamic SSI. Research study results and conclusions have historically been inconsistent. This is the primary reason why no industry-accepted and consistent guidelines have been established to either disprove or supplant the Mononobe-Okabe or Woods methods in the field of dynamic SSI.

- The LHPOST shake table at the Network for Earthquake Engineering Simulations (NEES) Camp Elliot facility (Figure 1) operated by the UCSD and under funding by the NEES organization via the National Science Foundation (NSF) has the exclusive capabilities in the United States to perform a full-scale dynamic test of a cantilever concrete retaining wall for the purpose of developing a standardized method of determining seismic lateral earth pressures for design and evaluation of earth retaining structures.

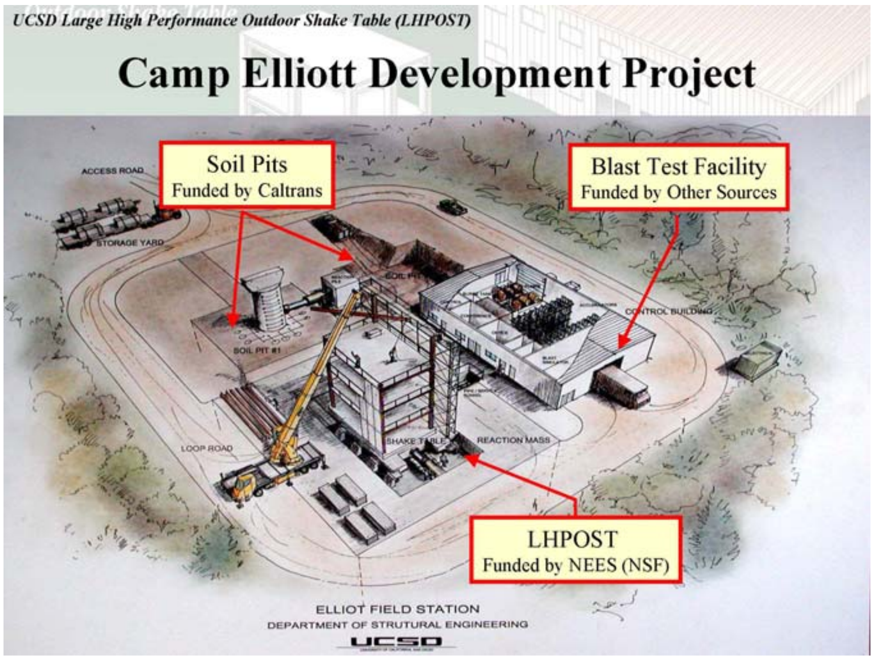

Figure 1 – Project layout of Camp Elliott home of the LHPOST.

- A conceptual design configuration consisting of opposing cantilever and gravity retaining walls is proposed for physical testing as shown on Figures 2 and 3. Each retaining wall is 15 feet tall and bounded in the non-shaking direction by removable precast concrete panels that provide easy access for placement and removal of backfill materials.

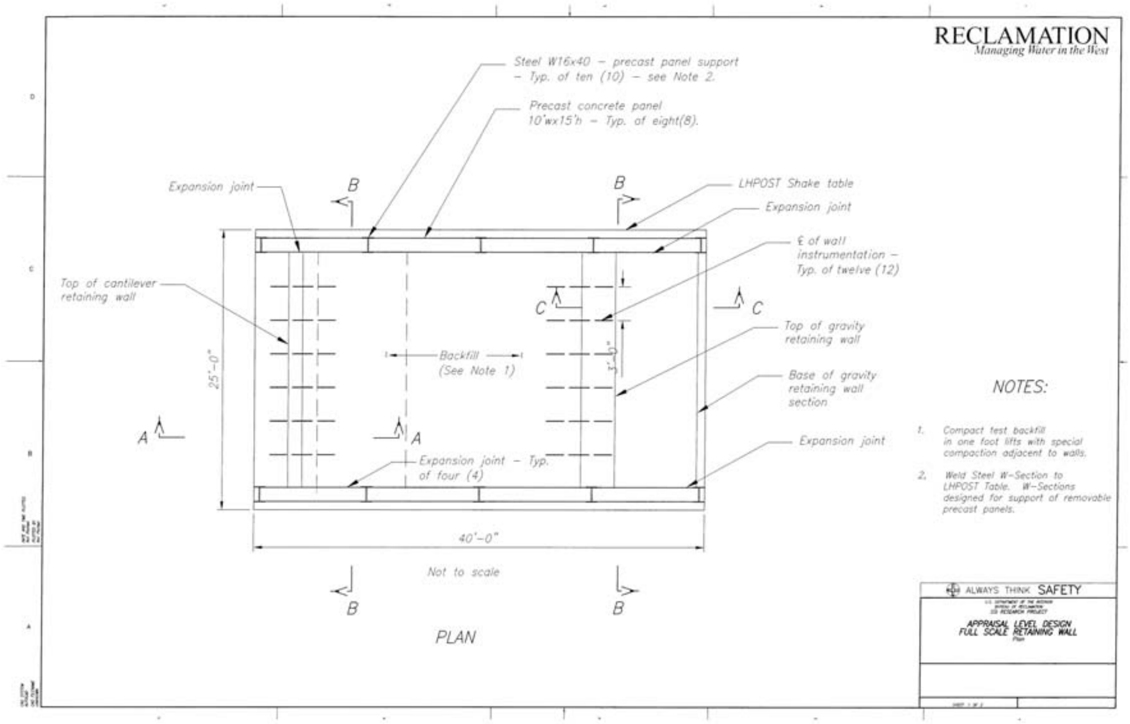

Figure 2 – Plan view of proposed full-scale model configuration.

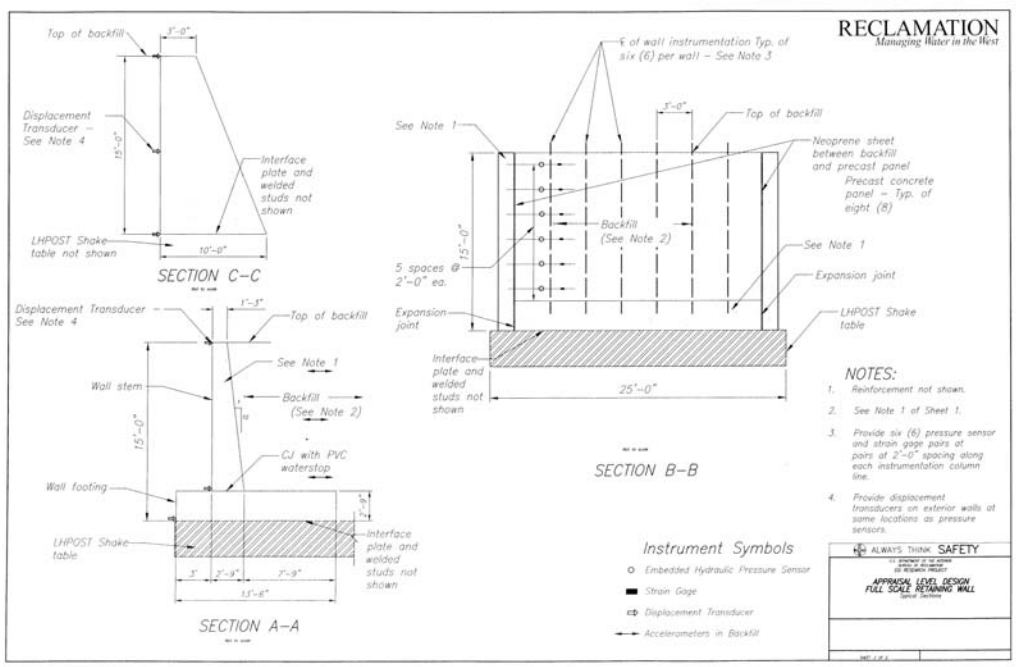

Figure 3 – Typical sections of proposed full-scale model configuration.

- Proposed testing procedures for the full-scale shake table test would include key parameters associated with Reclamation spillway wall field conditions including:

- Various backfill material types with varying degrees of cohesion
- Various phreatic surface levels within the backfill
- Various magnitudes and frequencies of the shaking inputs including time history runs

- The estimated cost in the 2009 dollars to construct and execute the proposed full-scale test program is $2.8 million, which price includes:
  - Construction costs
  - Instrumentation procurement and installation costs
  - Observation and oversight of instrumentation and data acquisition by a full-time Reclamation engineer
  - Execution costs, including data collection costs
  - Removal and replacement of backfill for three cycles of material type testing
  - Demolition and removal on constructed test features

FY2010 Planning Phase 2 Site Visit Activities and Observations

A site visit was conducted to the University of California San Diego (UCSD), Camp Elliott Structural Engineering Test Facility to evaluate the capabilities of the Large High Performance Outdoor Shake Table (LHPOST). The site visit was completed by Reclamation Team Leader, Steve Dominic and Senior Structural Technical Specialist, Larry Nuss on August 18, 2010. The site visit was coordinated with and lead by Professor Ahmed Elgamal of the USCD, Department of Structural Engineering and Editor-in-Chief of the *Journal of Soil Dynamics and Earthquake Engineering*. Photos documenting the LHPOST site visit are included in appendix A. A description of the site visit activities is presented below.

- Completed introductions with the UCSD structural engineering Professor Ahmed Elgamal who served as tour guide and host for Reclamation visitors Dominic and Nuss.
- Conducted a comprehensive tour of the Camp Elliott facilities as documented in the photographs of appendix A. Observed construction progress of a full-scale structural bridge pier experiment on the LHPOST shake table (Photos 3 through 6) and
- Observed the sub-surface configuration of the table's mechanical and structural systems (Photos 16 through 24).
- Observed previously tested structural specimens and test equipment and fixtures (Photos 10 through 14 and Photos 29 through 31).
- Observed data acquisition equipment (Photos 26 through 28).

- Observed a ballistics (high-impact) test of composite material consisting of balsa wood layered between two composite laminates (Photos 8 through 9).
- Reclamation presented two presentations to Professor Elgamal for the purpose of providing background information on Reclamation's dam safety and risk based decision making process and updating Professor Elgamal on the progress of Reclamation's soil-structure interaction research efforts. The two presentations included:
    - Safety Evaluation of Existing Dams – Risk Analysis and Risk Assessment for Dam Safety (appendix B)
    - Soil-Structure Interaction Status Update (appendix C)
- Former UCSD Ph.D. student, Patrick Wilson, joined the discussions for the purpose of presenting the results of his soil-structure interaction experiment completed in 2006 using the LHPOST shake table under the direction of Professor Elgamal (appendix D). Mr. Wilson is currently employed with Earth Mechanics, Inc. and indicated his potential future availability to further discuss his research with Reclamation at the TSC.
- Mr. Wilson's experiment was designed to simulate the performance of a full-scale bridge abutment subjected to passive and seismic lateral earth pressures. The tests were performed using a laminar soil box restrained to perform as a rigid container as shown in Figure 4. The structural concrete backwall of the container was suspended from a beam supported on rollers along the long edges of the soil container. Four load cell jacks were mounted on the back side of the backwall along with string potentiometers to capture the force displacement relationship during shaking as shown on Figure 5. A series of tests were performed to measure lateral earth pressures under different loading conditions including two passive tests and twenty-six dynamic tests up to earthquake records with peak horizontal accelerations of 1.2g.

Figure 4 – Soil container on LHPOST shake table utilized for 2006 bridge abutment experiment [6].

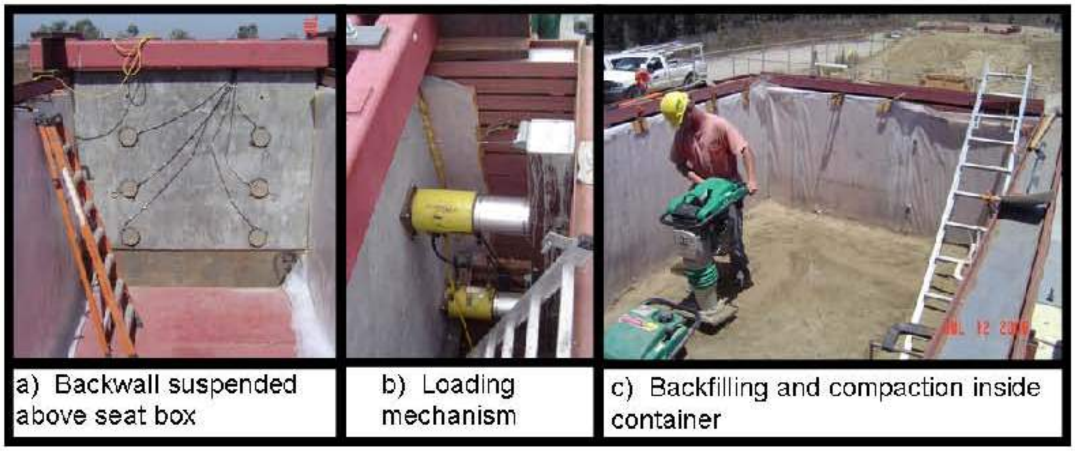

Figure 5 – Test setup of soil container on LHPOST shake table utilized for 2006 bridge abutment experiment showing suspended backwall, loading cells, and backfilling operations [6].

- Findings of Mr. Wilson's full-scale bridge abutment experiment can be summarized as follows:
  - The passive test results indicated triangular-shaped passive wedges with slip-plane angles generally consistent with Mohr-Coulomb theory. Results of the actual measured slip angle were best predicted by an average of direct shear and triaxial shear laboratory test results.
  - The mobilized wall-soil friction was estimated to be less than 1-2 degrees for the passive tests due to the ability of the wall to rigidly move upward with the passive wedge. A test not allowing the wall to move upward was not possible because the curvilinear failure wedge would have been longer than the soil box.
  - The first passive test was completed at optimum moisture content of 95 percent and the second passive test was complete 20 days later at a reduced moisture content of approximately 85 percent. While the force-displacement curve for both tests indicated approximately the same residual strength, the peak strengths were offset and the reduced water content soil exhibited brittle behavior compared to the optimal water content soil.
  - Dynamic test results indicated that for lower magnitude events (<0.6g) the inertia of the wall was the primary contributing component to the dynamic load. Further, dynamic pressure distributions were shown to reduce static pressures near the top of the wall resulting in essentially no net increase in base shear for lower magnitude events.
  - Dynamic test results indicated that for higher magnitude events (>0.6g) the soil contribution to the net dynamic force on the wall was noticeable and fairly representative of values predicted by M-O. For high accelerations that exceeded the friction angle of the soil ($\psi > \phi$) reduction factors for K_{ae} were used based on NCHRP

Report 611 [13] assuming a small amount of backfill cohesion to predict the dynamic force on the wall.

- At higher frequency shaking the backfill was observed to be out-of-phase with the movement of the wall resulting in very little to no net increase in loading on the wall and with lower frequency shaking the backfill was observed to be in-phase with the wall resulting in a net increase in loading on the wall.
- Numerical modeling completed with the OpenSees finite element software [12] resulted in good comparisons with the results of Mr. Wilson's experiment.
- A paper further presenting the dynamic results of Mr. Wilson's experiment will be presented in the November 2010 issue of the ASCE journal.
- Mr. Wilson has concluded from his research that friction angles of soils are typically greater than used in design. Actual shear strength angles are typically representative of the average of triaxial test results and direct shear test results.

- Discussed that the waiting period for the LHPOST shake table is currently two years.
- Professor Elgamal presented options available for Reclamation regarding completion of a retaining wall test in regards to dynamic soil-structure interaction.
  - Submit a proposal through the NEES organization protocol.
  - Eliminate the NEES proposal and seek direct buy-in from the National Science Foundation (NSF) to pay for the use of LHPOST resources.
  - Perform a smaller scale test using a new soil container that was recently purchased for the Camp Elliott facilities for approximately $500,000 and would be readily available for use to avoid the schedule delays and budget issues associated with an LHPOST experiment.
- Discussed available teaming partners primarily focused on Caltrans and the United States Army Corps of Engineers (USACE). Professor Elgamal and Mr. Wilson's 2006 experiment was sponsored by Caltran and, as a result, their knowledge of financial options available through Caltran could prove valuable to Reclamation's experiment efforts. In August 2010, Reclamation submitted a research proposal for funding with FEMA through John Plisich in the Atlanta, Georgia office (john.plisich@dhs.gov).
- Following the tour of the Camp Elliott facilities, Professor Elgamal included a tour of the structural testing facilities located on the main UCSD campus located in La Jolla, California. Observations from the main campus tour are presented in Photos 32 through 42 of appendix A.
- As a sidelight, UCSD has developed a very capable non-destructive testing program that might be applicable to the East Canyon Dam cracking issue.

Summary of LHPOST Site Visit Findings

The findings of the LHPOST Site Visit can be summarized as follows:

- The LHPOST shake table at the NEES Camp Elliot facility operated by the UCSD has been used to perform more than ten full-scale dynamic structural experiments and has unique capabilities that would be required to perform a full-scale dynamic test of a cantilever concrete retaining wall for the purpose of developing a standardized method of determining seismic lateral earth pressures for design and evaluation of earth retaining structures.
- The LHPOST shake table is currently scheduled for experiments for the next two years suggesting that the earliest that a Reclamation test could be performed would be the fall of 2012. This schedule would still work with the proposed phased approach outlined in the Introduction section of this report.
- Two options are available for procuring the LHPOST shake table for a full-scale retaining wall experiment
  - Submit a proposal through the NEES organization protocol.
  - Eliminate the NEES proposal and seek direct buy-in from the NSF to pay for the use of LHPOST resources.
- Professor Elgamal and the UCSD structural engineering department have substantial experience working with Caltrans and, as a result, Caltrans could prove to be the most likely candidate for a Reclamation teaming partner in terms of funding and full-scale retaining wall shake table test using the LHPOST.

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Appendix A – Site Photographs

Photo 1. Soil pit experiments funded by Caltrans including testing for bridge piers surrounded by riprap (foreground) and bridge abutment foundation tests (background).

Photo 2. Close up view of soil pit with Caltran funded bridge pier surrounded by riprap.

Photo 3. Construction progress of bridge on LHPOST shake table.

Photo 4. Close-up view of bridge pier footing connected to LHPOST shake table.

Photo 5. Reinforcement cage extending from construction joint of bridge pier being construction on LHPOST shake table.

Photo 6. Scaffolding for construction of bridge pier on LHPOST shake table.

Photo 7. Steel reaction frame for future LHPOST shake table experiment.

Photo 8. Composite section consisting of balsa wood between two laminar

composites being prepared for impact testing.

Photo 9. Composite section consisting of balsa wood between two laminar composites following impact test showing shear failure at end supports with delamination of exterior laminates. This test simulated a truck impacting a bridge pier at a scaled velocity of 20 m/sec.

Photo 10. Damaged reinforced concrete column from previous structural testing.

Photo 11. Composite wrapped concrete columns showing improved structural performance compared to unwrapped members shown in Photo 10.

Photo 12. Performance comparison between conventional reinforced concrete column and composite wrapped column.

Photo 13. Performance comparison between conventional reinforced concrete column and composite wrapped column.

Photo 14. Stockpile area with equipment and test fixtures from previous site experiments.

Photo 15. Office trailers for Camp Elliott.

Photo 16. Oil tank for LHPOST hydraulic system.

Photo 17. Hydraulic pressurization cylinders for LHPOST shake table (5000 psi oil pressure).

Photo 18. Actuator arm for LHPOST shake table.

Photo 19. Hydraulic bearing supports of LHPOST shake table.

Photo 20. Threaded post-tensioned anchors for pier foundation being constructed on top of LHPOST table—see Photo 4.

Photo 21. Structural bolted connection of LHPOST shake table platen connection—typical of two connections.

Photo 22. Close-up of threaded post-tensioned anchor for pier foundation being constructed on top of LHPOST table—see Photo 4.

Photo 23. Hydraulic actuator pin connection to LHPOST shake table.

Photo 24. Primary hydraulic fluid delivery pipes.

Photo 25. Coolant tank for LHPOST hydraulic system.

Photo 26. Primary data acquisition servers.

Photo 27. Data cables from LHPOST instrumentation to collection servers—see Photo 26.

Photo 28. Client workstation for primary data acquisition servers.

Photo 29. Railroad track for non-destructive testing experiment.

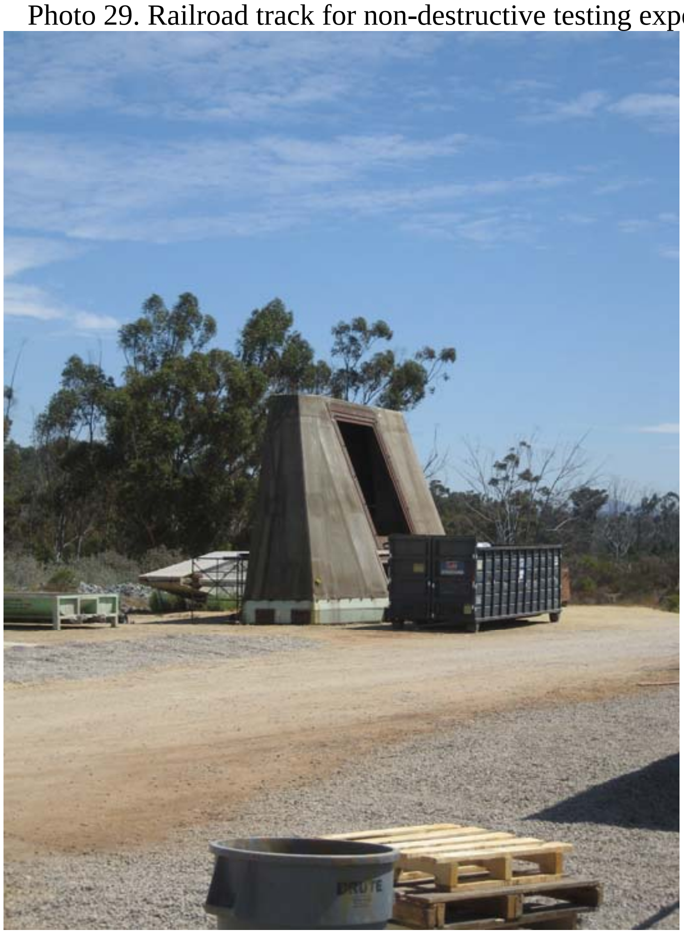

Photo 30. Navy test specimen from previous LHPOST studies.

Photo 31. Laminar soil box test fixture for Patrick Wilson's large scale bridge abutment experiment.

Photo 32. Wind turbine steel tower being prepared for testing at the UCSD La Jolla Campus facilities.

Photo 33. Reaction walls for structural testing at the UCSD La Jolla Campus facilities.

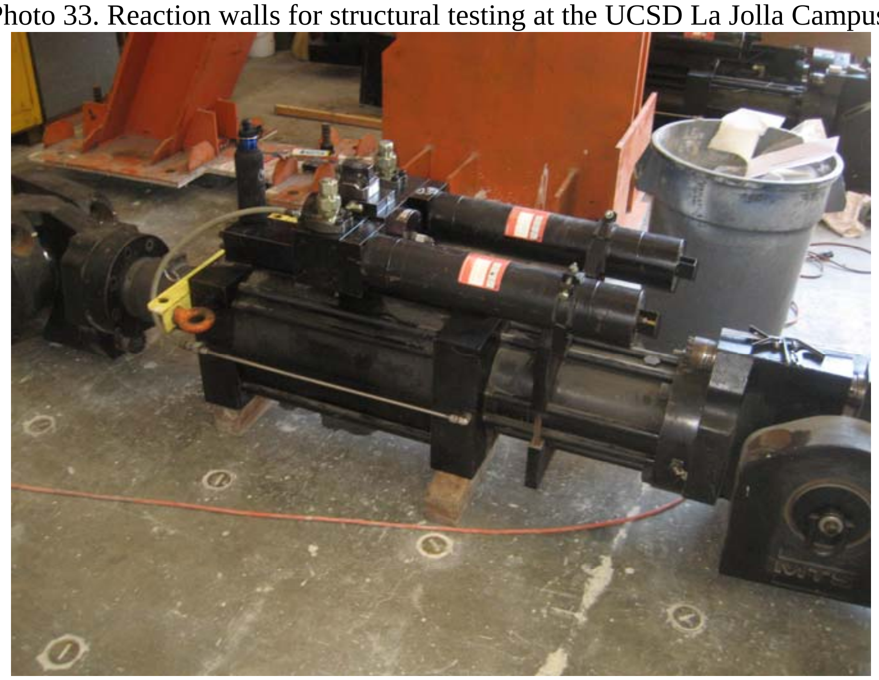

Photo 34. Typical static actuator at La Jolla Campus facilities.

Photo 35. Railroad track at La Jolla Campus facility for non-destructive testing.

Photo 36. Small scale shake table at La Jolla Campus facility.

Photo 37. Structural steel highway traffic sign support column being prepared for testing at the La Jolla Campus facilities.

Photo 38. Composite wings of Predator remote controlled surveillance aircraft previously tested at the La Jolla Campus facilities.

Photo 39. Composite material used for manufacturing test members to be tested at the La Jolla Campus facilities.

Photo 40. Structural seismic isolation system previously tested at the La Jolla Campus facilities.

Photo 41. Structural test fixture for seismic isolation systems at the La Jolla campus facilities.

Photo 42. Structural seismic isolation system previously tested at the La Jolla Campus facilities.

Appendix B – Safety Evaluation of Existing Dams – Risk Analysis and Risk Assessment for Dam Safety Presentation

RECLAMATION

Managing Water in the West

Safety Evaluation of Existing Dams

Risk Analysis and Risk Assessment for Dam Safety

LHPOST Site Visit - August 18, 2010

U.S. Department of the Interior
Bureau of Reclamation

What Is Risk Analysis?

- Represented by a simple equation:

$$Risk = (P_{event})(P_{response})(consequences)$$

- Risk can be evaluated by answering…
  - What undesired event could occur?
  - How likely is it?
  - What would happen if it did?

RECLAMATION

For Dam Safety…

1976 Failure of Teton Dam

1978 Reclamation Safety of Dams Act

1979 Federal Guidelines for Dam Safety

1997 and 2003 Guidelines for Achieving Public Protection in Dam Safety Decision Making

RECLAMATION

Background on Dam Safety

- Reclamation has 378 **high** and **significant** hazard dams
- Over 50 % of these dams are more than 50 years old (oldest 100 years old)
- Potential loading conditions (floods and earthquakes) have increased for many of the dams
- Populations growing downstream of dams

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Risk Analysis used in Dam Safety for…

- Gaining a better understanding of what can cause the dam to fail
- Quantifying the engineering judgments (need to build the case to support the estimates)
- Identifying need for additional studies
- Setting priorities. Should corrective action take place immediately, next year, in 6 years, etc?

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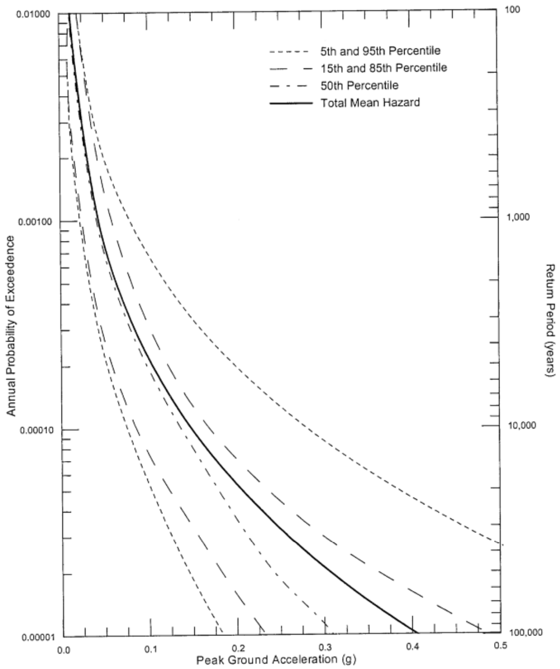
The typical steps of a dam safety risk analysis…

- Identify failure modes
- Determine frequency of loads of concern
  - static (normal)
  - hydrologic
  - seismic
- Estimate likelihood of failure
- Estimate potential life loss
- Compute risk and identify uncertainties
- Examine the conclusions
- Build the case and make recommendations

RECLAMATION

Event Tree



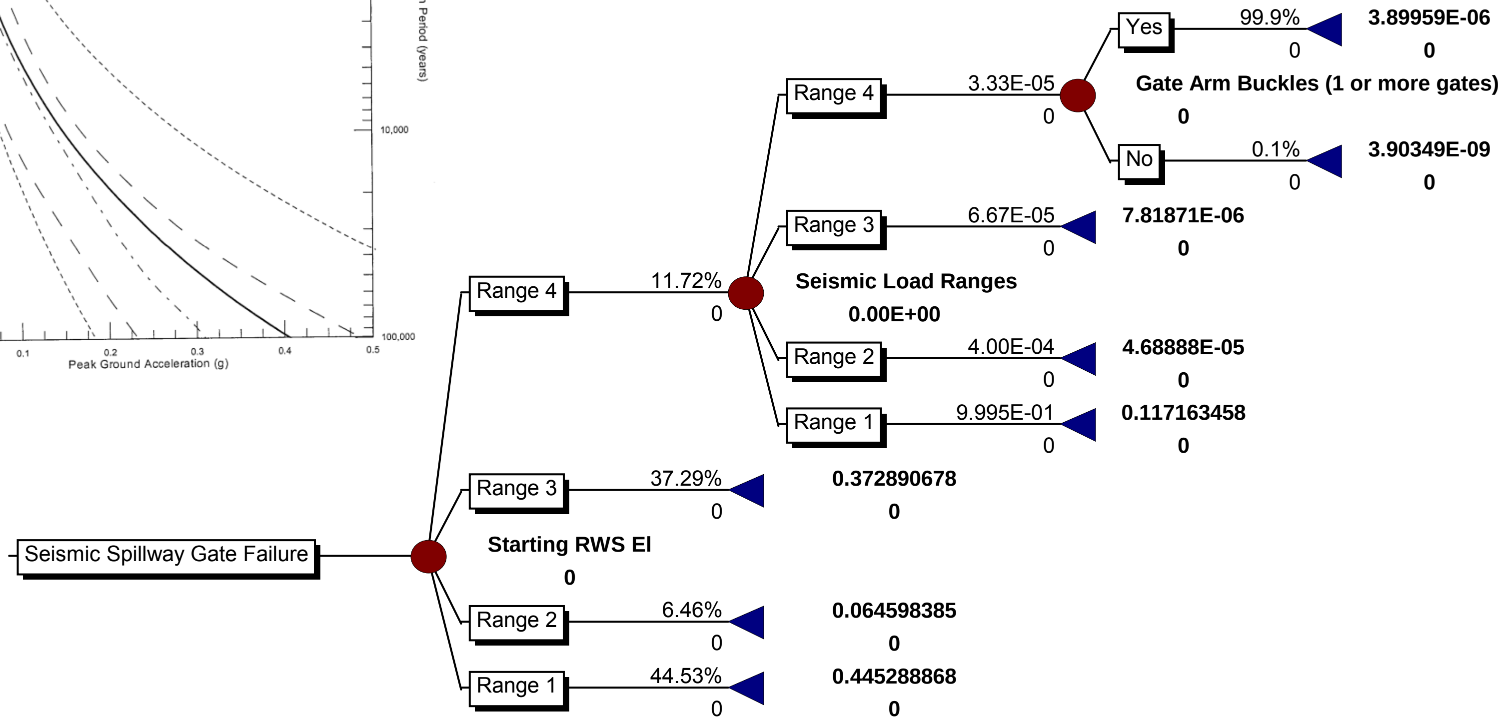
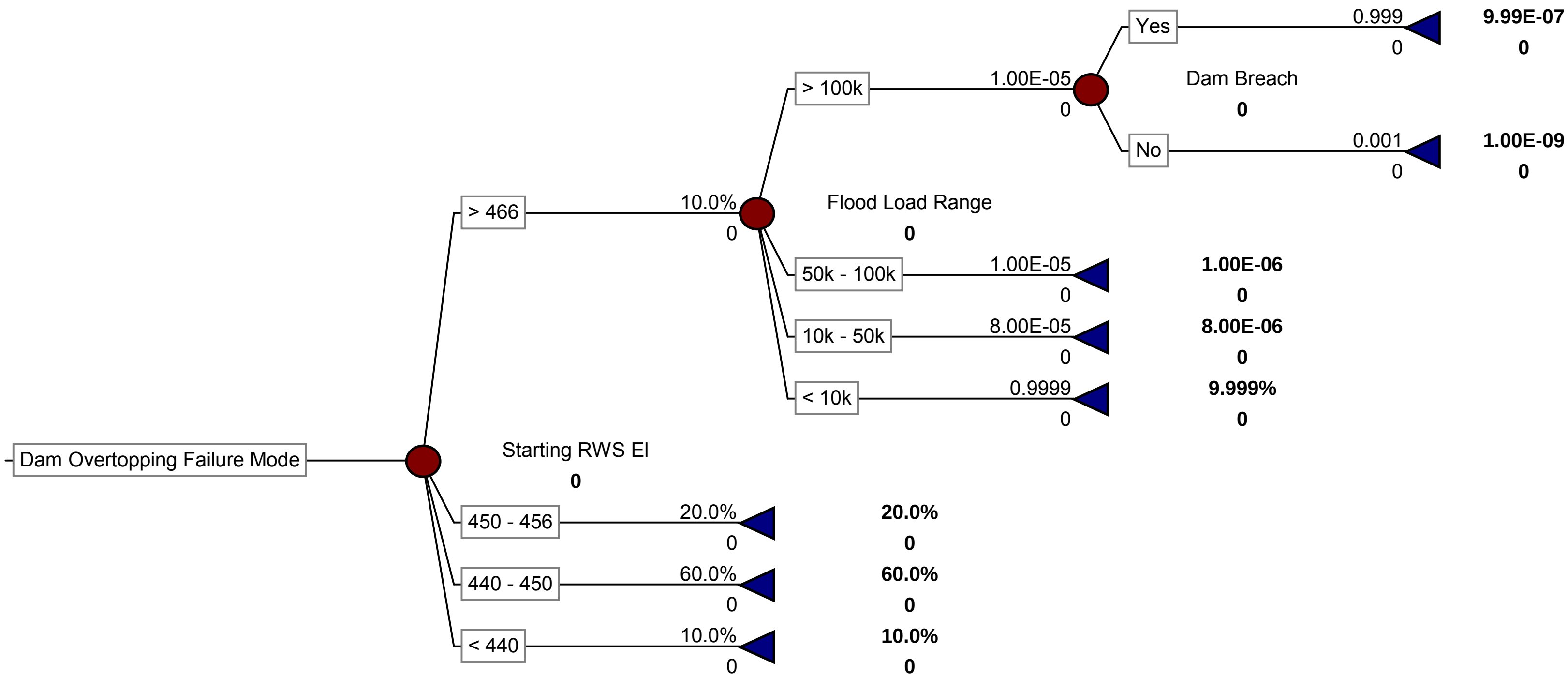


Figure 15-2 - Example Event Tree

Embankment Dam Overtopping Event Tree



Risk Analysis Estimates

- Estimates are often limited by lack of information or analyses and studies – creates uncertainty
- Sensitivity studies can be performed to evaluate the impact of variability in key nodes

Risk Analysis Estimates

- Summarize What is Known and Not Known
- More Likely and Less Likely Factors are Identified
- A range of estimates is made for a given node

RECLAMATION

Risk Estimates

- Virtually Certain – 0.999
- Very Likely – 0.99
- Likely – 0.9
- Neutral – 0.5
- Unlikely – 0.1
- Very Unlikely – 0.01
- Virtually Impossible – 0.001

RECLAMATION

Estimate Consequences

- Potential loss of life
  - Population at Risk can increase over time, which will likely increase loss of life estimates
  - Based primarily on affected downstream population, available warning time, and estimated severity of the flood wave
  - Better methods are needed for large populations with limited warning
  - Estimates are based on predicting human behavior

RECLAMATION

Estimating Loss of Life

Recommended Fatality Rates for Estimating Loss of Life Resulting from Dam Failure

Flood Severity	Warning Time (minutes)	Flood Severity Understanding	Fatality Rate (Fraction of people at risk expected to die)	
			Suggested	Suggested Range
HIGH	no warning	not applicable	0.75	0.30 to 1.00
	15 to 60	vague	Use the values shown above and apply to the number of people who remain in the dam failure floodplain after warnings are issued. No guidance is provided on how many people will remain in the floodplain.	
		precise		
	more than 60	vague		
		precise		
MEDIUM	no warning	not applicable	0.15	0.03 to 0.35
	15 to 60	vague	0.04	0.01 to 0.08
		precise	0.02	0.005 to 0.04
	more than 60	vague	0.03	0.005 to 0.06
		precise	0.01	0.002 to 0.02
LOW	no warning	not applicable	0.01	0.0 to 0.02
	15 to 60	vague	0.007	0.0 to 0.015
		precise	0.002	0.0 to 0.004
	more than 60	vague	0.0003	0.0 to 0.0006
		precise	0.0002	0.0 to 0.0004

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Four Basic Types of Risk Analysis

Screening Level Risk Analysis (a basic screening evaluation of full inventory)

Comprehensive Facility Review (done every 6 years, identifies specific issues of concern)

Issue Evaluation Risk Analysis (a detailed look by a qualified team at the specific issues)

Risk Reduction Analysis (identifies alternatives that reduce risk, both structural and non-structural)

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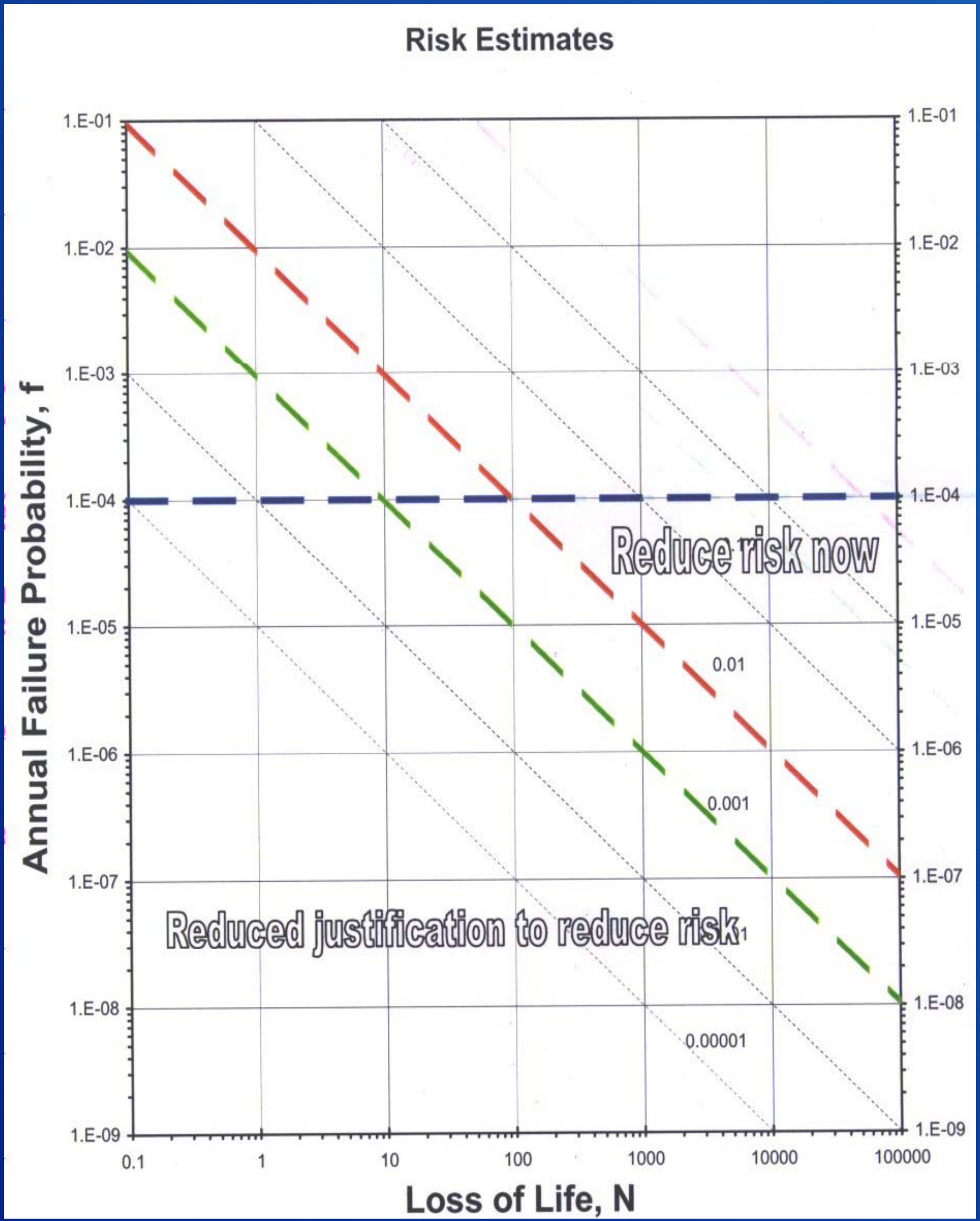
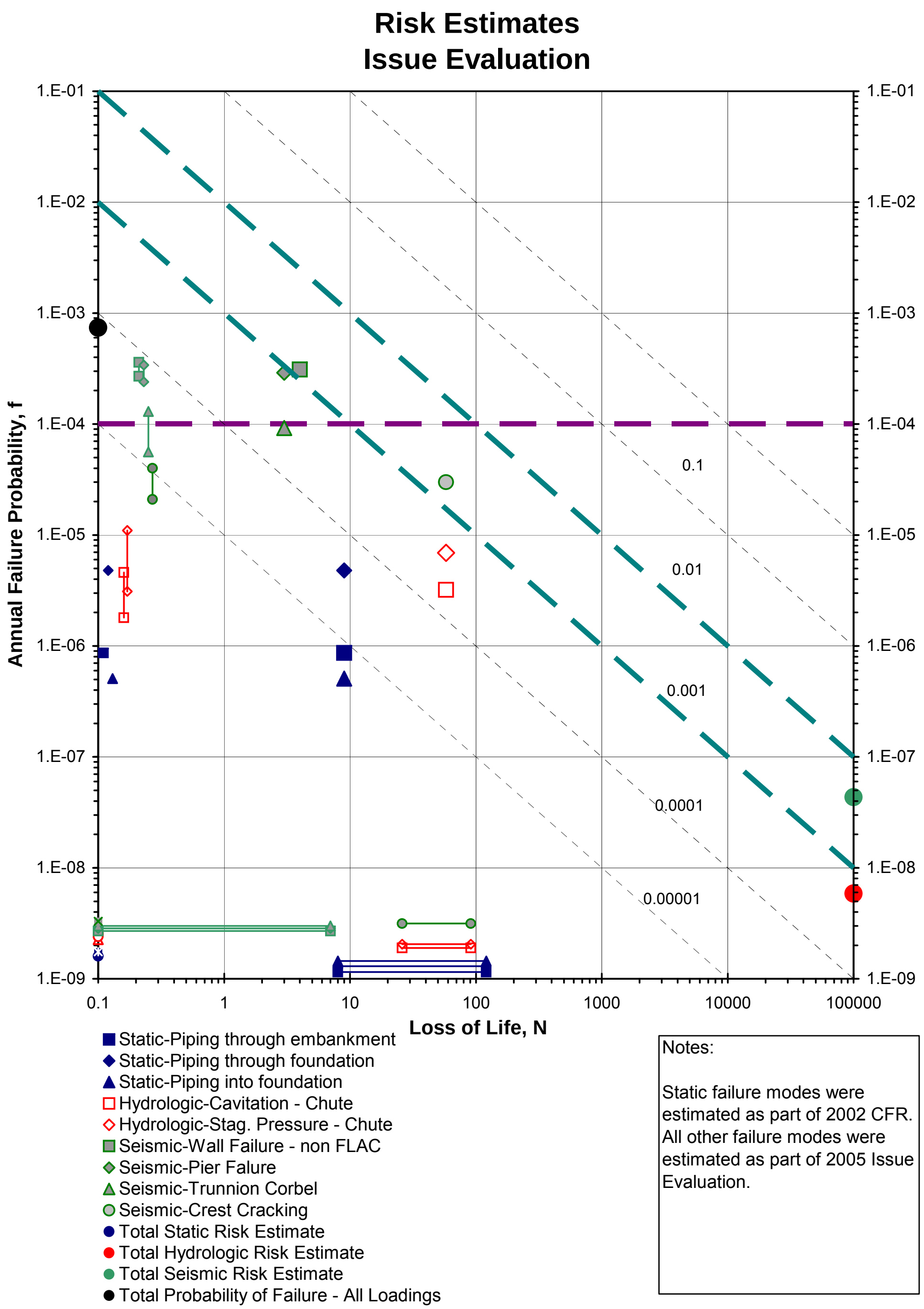
Reclamation Public Protection Guidelines



RECLAMATION



Risk Estimates
Issue Evaluation
Annual Failure Probability, f
Loss of Life, N
1.E-01
1.E-02
1.E-03
1.E-04
1.E-05
1.E-06
1.E-07
1.E-08
1.E-09
0.1
1
10
100
1000
10000
100000
0.1
0.01
0.001
0.0001
0.00001
Static-Piping through embankment
Static-Piping through foundation
Static-Piping into foundation
Hydrologic-Cavitation - Chute
Hydrologic-Stag. Pressure - Chute
Seismic-Wall Failure - non FLAC
Seismic-Pier Falure
Seismic-Trunnion Corbel
Seismic-Crest Cracking
Total Static Risk Estimate
Total Hydrologic Risk Estimate
Total Seismic Risk Estimate
Total Probability of Failure - All Loadings
Notes:
Static failure modes were estimated as part of 2002 CFR. All other failure modes were estimated as part of 2005 Issue Evaluation.

f-N Chart

Appendix C – Soil-Structure Interaction Status Update Presentation

RECLAMATION

Managing Water in the West

Soil-Structure Interaction

Status Update

LHPOST Site Visit - August 18, 2010

U.S. Department of the Interior
Bureau of Reclamation

SSINT - Problem and Background

- Spillway retaining walls in high seismic areas
  - Founded on soil vs. rock
  - Adjacent to abutments
  - Backfill geometry
  - Counterfort/non-counterfort
  - Special compaction
  - Cohesion
  - Gated with water – potential failure mode

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SSINT - Problem and Background

- Issues
  - Limitations of traditional methods
    - Cohesionless backfill (M-O)
    - $\phi \geq \psi$ (M-O)
    - Non-Yielding wall (Woods)
    - Conservative approach (Passive)
  - Increasing seismic hazards
  - Increasing consequences
  - Numerical modeling complicated and expensive
  - Validation testing limited/non-existent

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SSINT - Problem and Background

- Potentially Impacted Reclamation Dams
  - 35 gated spillway structures in high seismic areas
  - Currently quantifying potential impact in rehabilitation construction costs.
  - Projects have been delayed or re-scheduled.

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SSINT - Problem and Background

- Bradbury Dam
  - Deterministic design late 1980s/early 1990s – 0.7g PGA
  - Current 2009 probabilistic seismic hazard
    - ✓3,100 year (3.2×10^{-4}) = 0.7g
    - ✓10,500 year (1.0×10^{-4}) = >1.0g
    - ✓50,000 year = 1.6g
  - Unacceptable risks
  - Potential second rehabilitation approximately 20 years later

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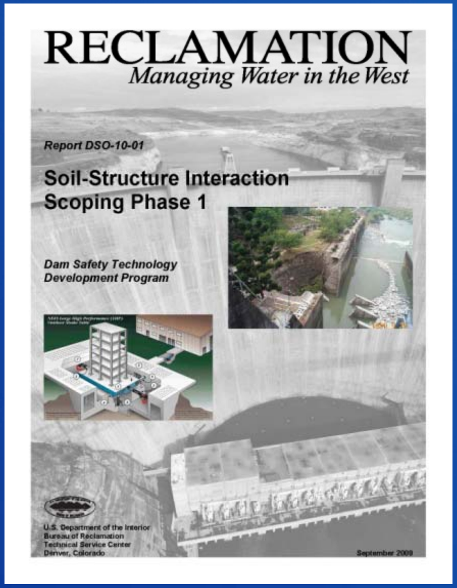
SSINT - Dam Safety Technology Development Proposal

- Five Year Plan – Full Scale Test
  - Phase 1 – FY09 – Scoping Phase
  - Phase 2 – FY10 – Planning Phase
  - Phase 3 – FY11 – Feasibility Design
  - Phase 4 – FY12 – Final Design
  - Phase 5 – FY13 – Laboratory Testing

RECLAMATION

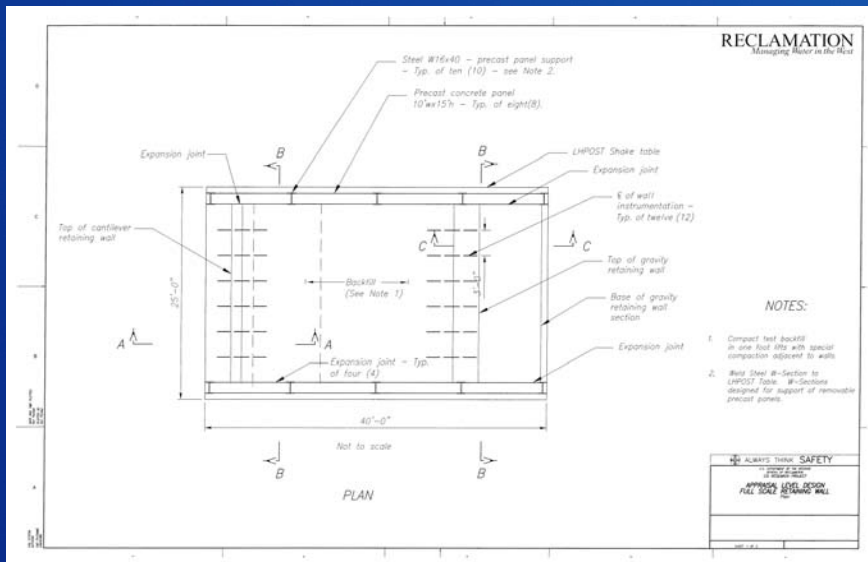
SSINT - Dam Safety Technology Development Proposal

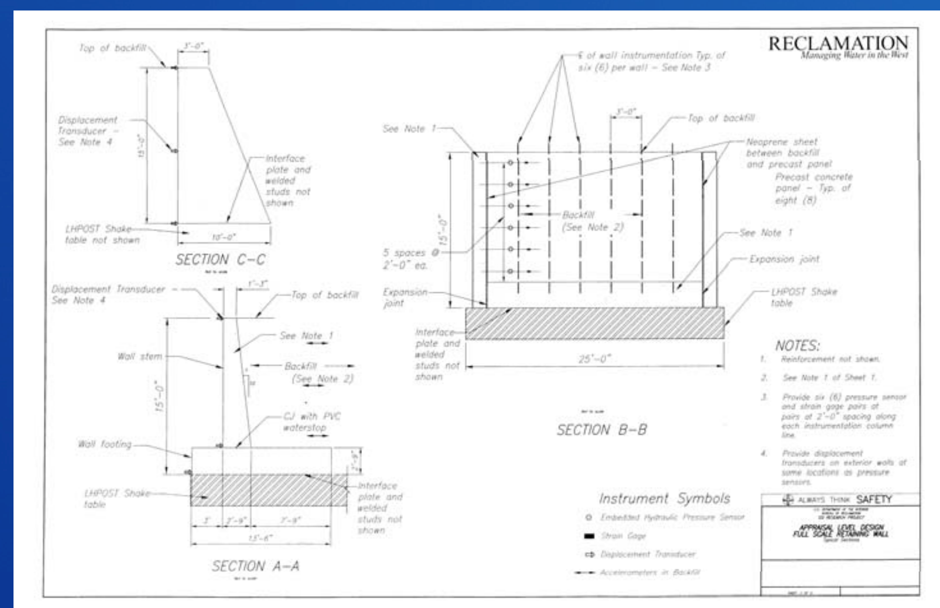
- Phase 1 – FY09 – Scoping Phase
  - Literature review
  - Identify laboratory resources and capabilities
  - Develop appraisal level full-scale model concept and cost estimate
    - ✓Model layout
    - ✓Instrumentation options



RECLAMATION

Full Scale Model Concept





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Cost Estimate

- Quantities
  - 350 cy of material per test – 1 foot lifts
  - 4400 sf of special compaction adjacent to walls
  - 130 cy of concrete
  - 15,000 lbs of reinforcement
- Estimated Costs
  - **$2.8 Million**

RECLAMATION

SSINT - Dam Safety Technology Development Proposal

- Phase 2 – FY10 – Planning Phase
  - Sitar visit to Reclamation
  - Finite element validation studies
  - Traditional method cost comparisons
  - DHS Proposal

RECLAMATION

Sitar Visit – Nov. 2009

- Centrifuge testing at UC Davis
  - Wall inertial moments significant contributor to dynamic moment - in phase
  - Earth pressures out of phase < M-O method
- Generally good seismic performance
  - No known large scale failure of well designed retaining walls
  - Walls not designed for earthquake can withstand PHA = 0.3g
- Reclamation spillways different problem

RECLAMATION

Finite Element Studies

- Have modeled several Reclamation spillways - very different answers based on foundation conditions
- Have achieved fairly good numerical comparisons with FLAC & LS-DYNA models
- Difficulty re-producing centrifuge model results - have had some success re-producing hysteretic pattern, but not force magnitude

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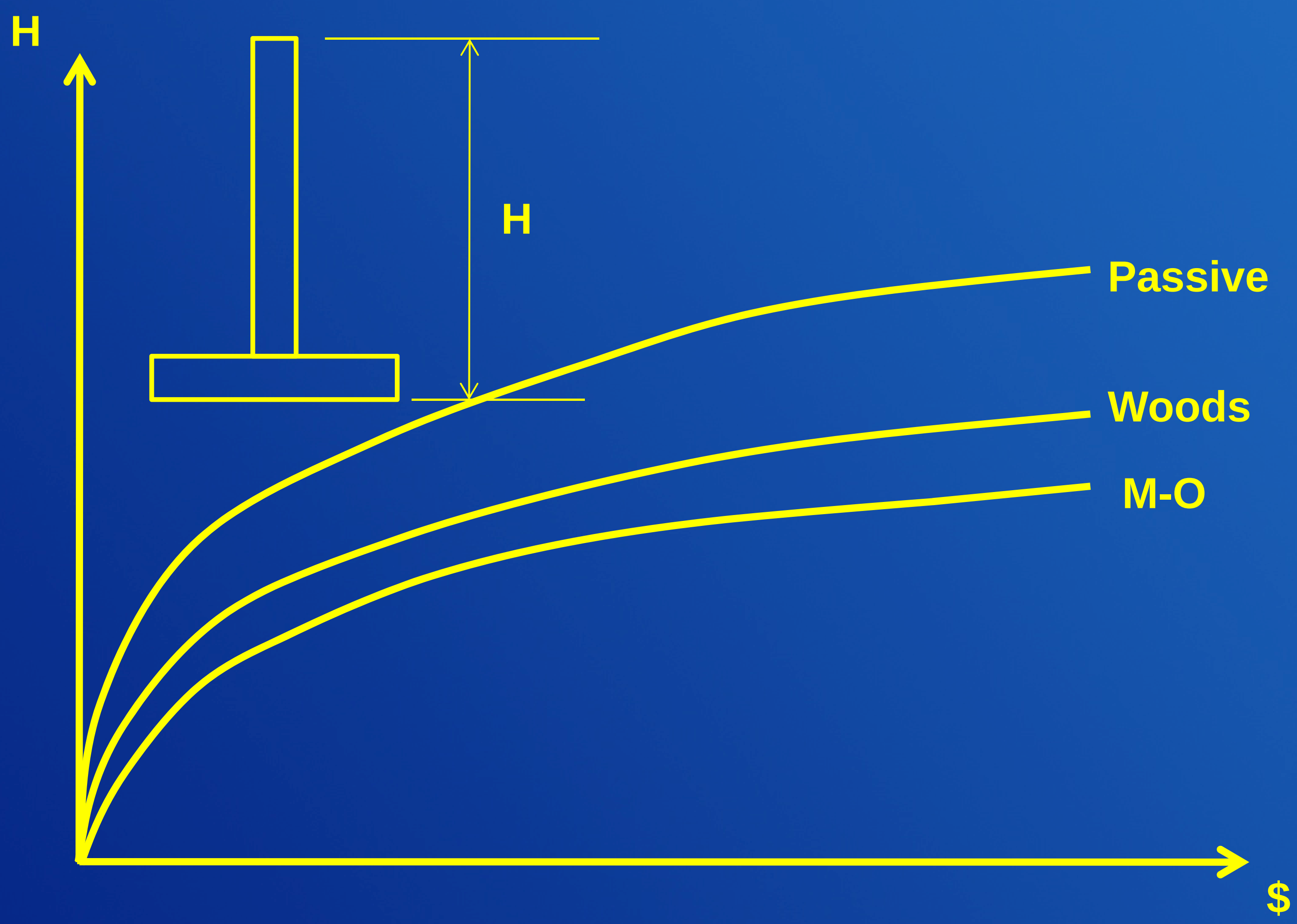
Cost Comparisons of Traditional Methods



Hurdles Ahead

- More questions than answers
- FE model required for every dynamic earth retaining problem?
- Pending projects in CAS and Final Design
- Full scale testing very expensive – Reclamation will be unable to fund test without partners
- Will we get out of full-scale test what we are hoping for?

RECLAMATION

Appendix D – Lateral Earth Pressure in Earthquake Engineering: An Overview

Lateral Earth Pressure in Earthquake Engineering: An Overview

Ahmed Elgamal and Patrick Wilson

University of California, San Diego

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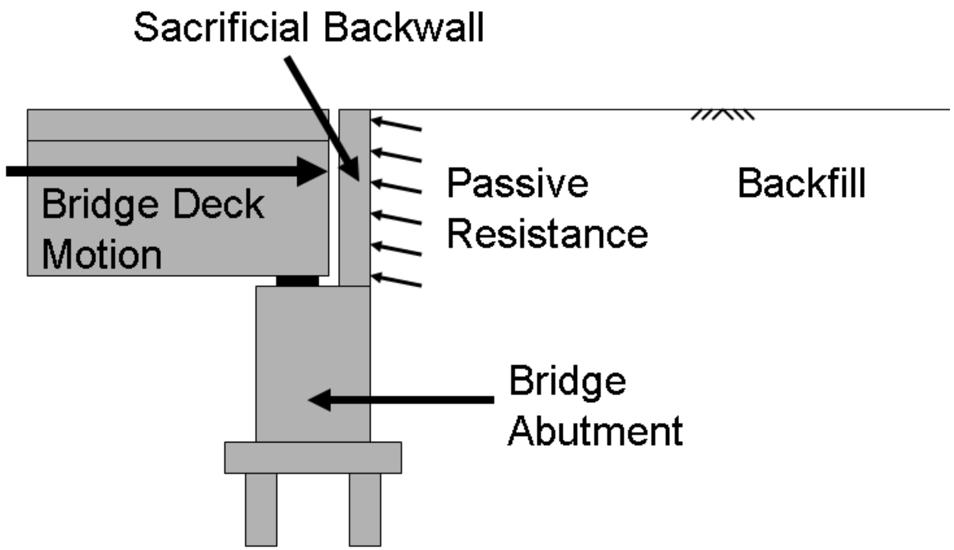
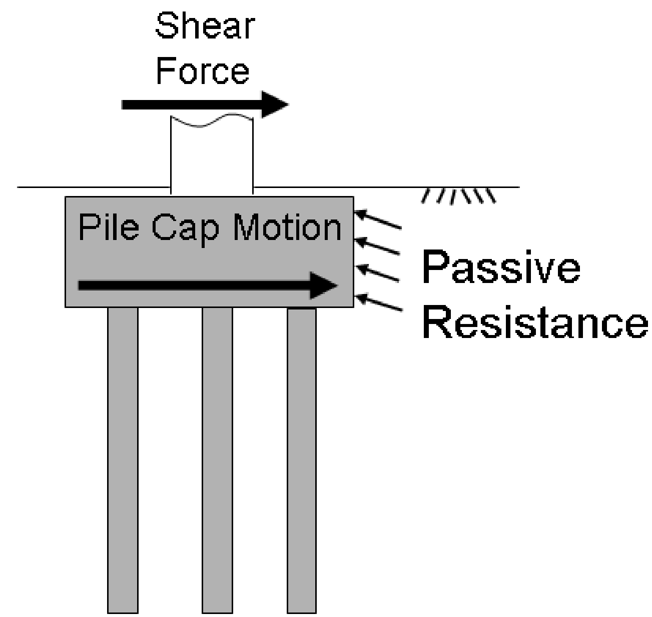
Passive Earth Pressure Mobilization with Displacement





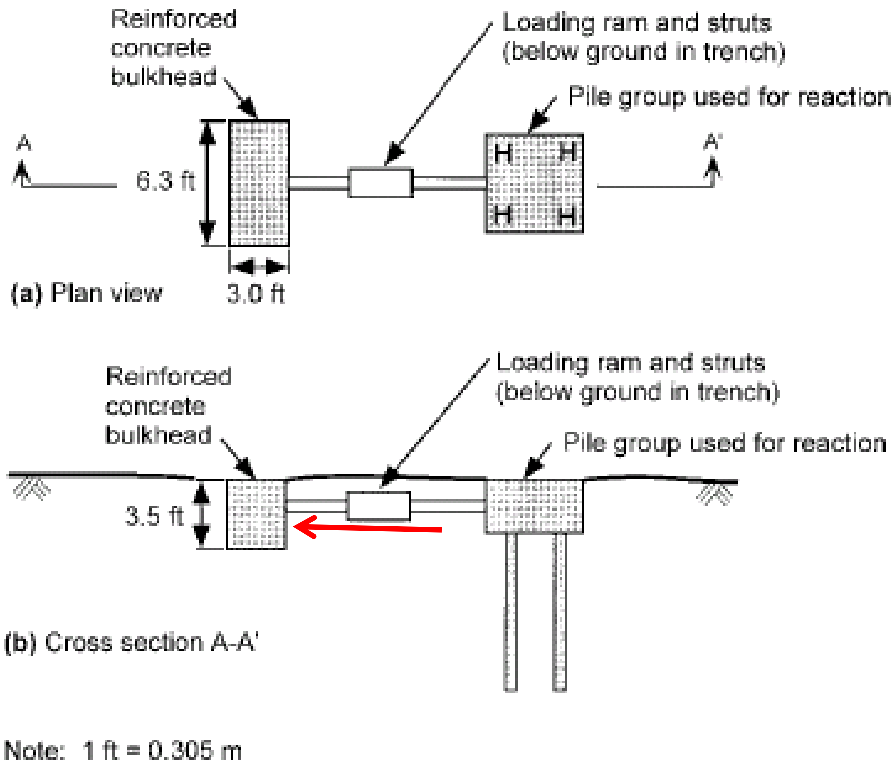
- Passive pressure provides resistance to earthquake-induced foundation displacement
- Required: Estimates of the peak resistance and **the force-displacement relationship**
- Theoretical predictions (e.g. Rankine, Coulomb, Log Spiral) do not provide information on the mobilization of passive resistance with displacement
- In **some** cases, passive resistance may be **detrimental** (e.g., Thermal expansion of integral abutment bridges)

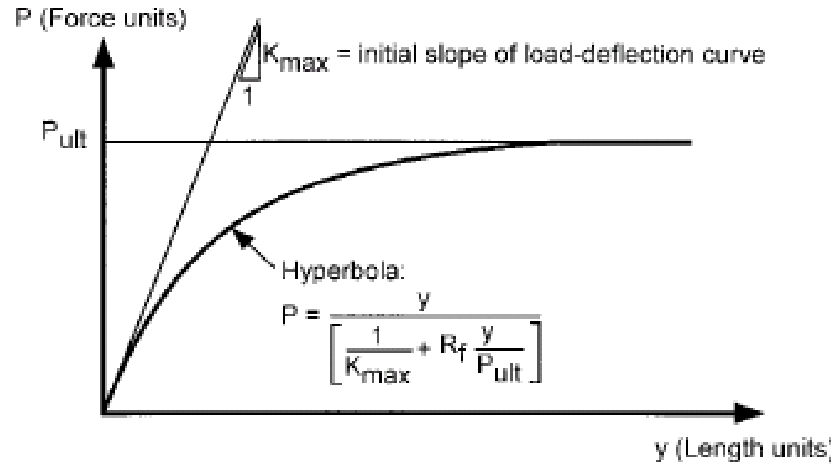
Elgamal/Wilson (UCSD)

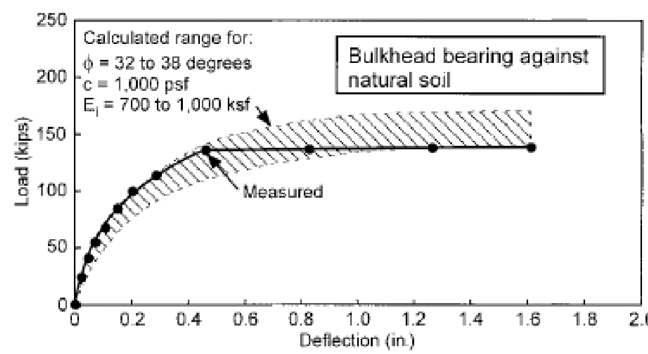
Outline of the Presentation

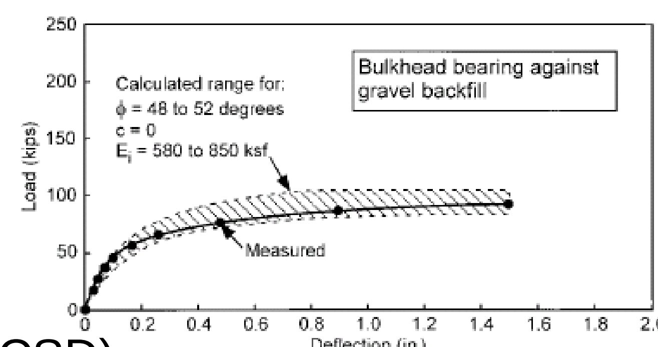
- UCSD large scale soil container experiments using outdoor shake table
- Passive earth pressure force-displacement relationships
- Dynamic earth pressure and retaining wall earthquake response
- Conclusions

Duncan and Mokwa (2001)









- Conducted passive pressure load tests on an anchor block
- Log Spiral provided good estimate of peak passive pressure 
- Hyperbolic model to represent the load-displacement behavior

Fang, Ho, Chen (2002)

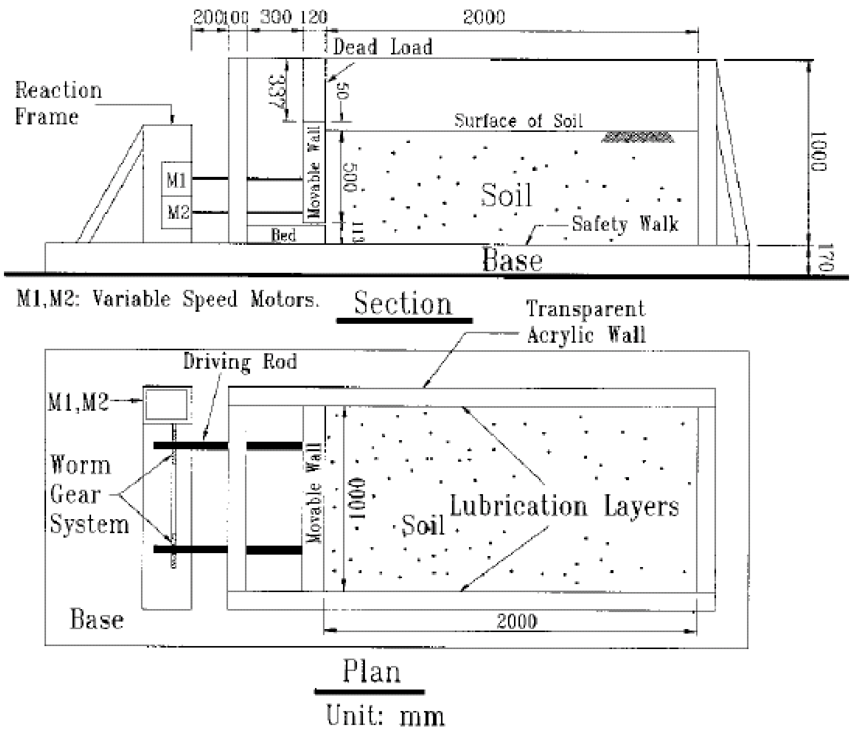


Fig. 3. National Chiao Tung Univ. retaining-wall facility

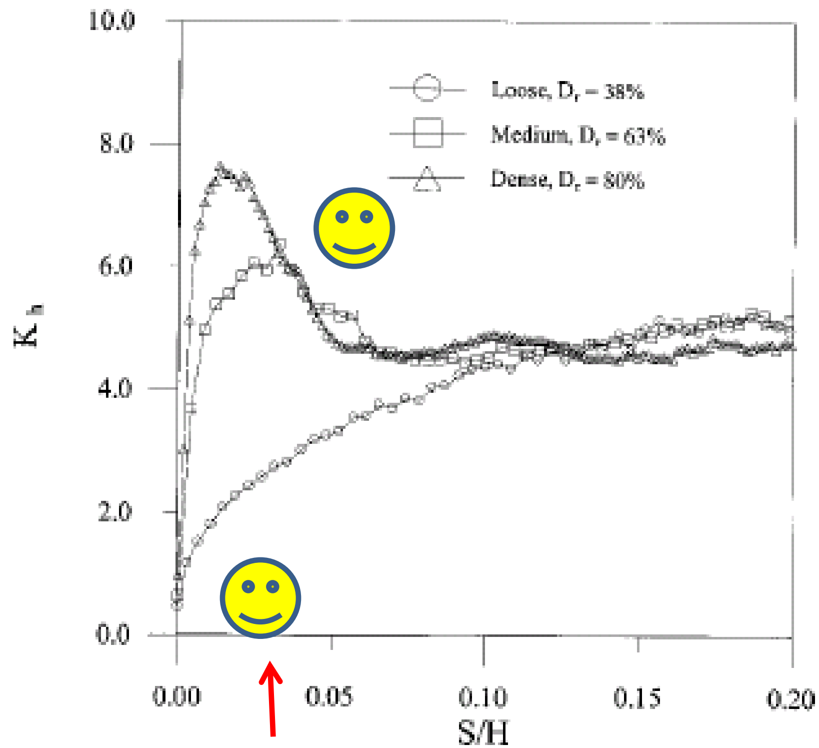
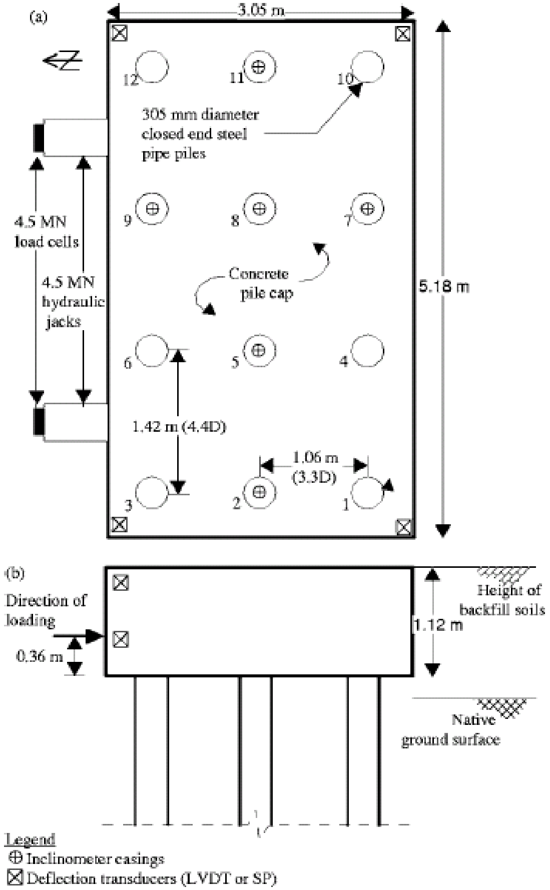
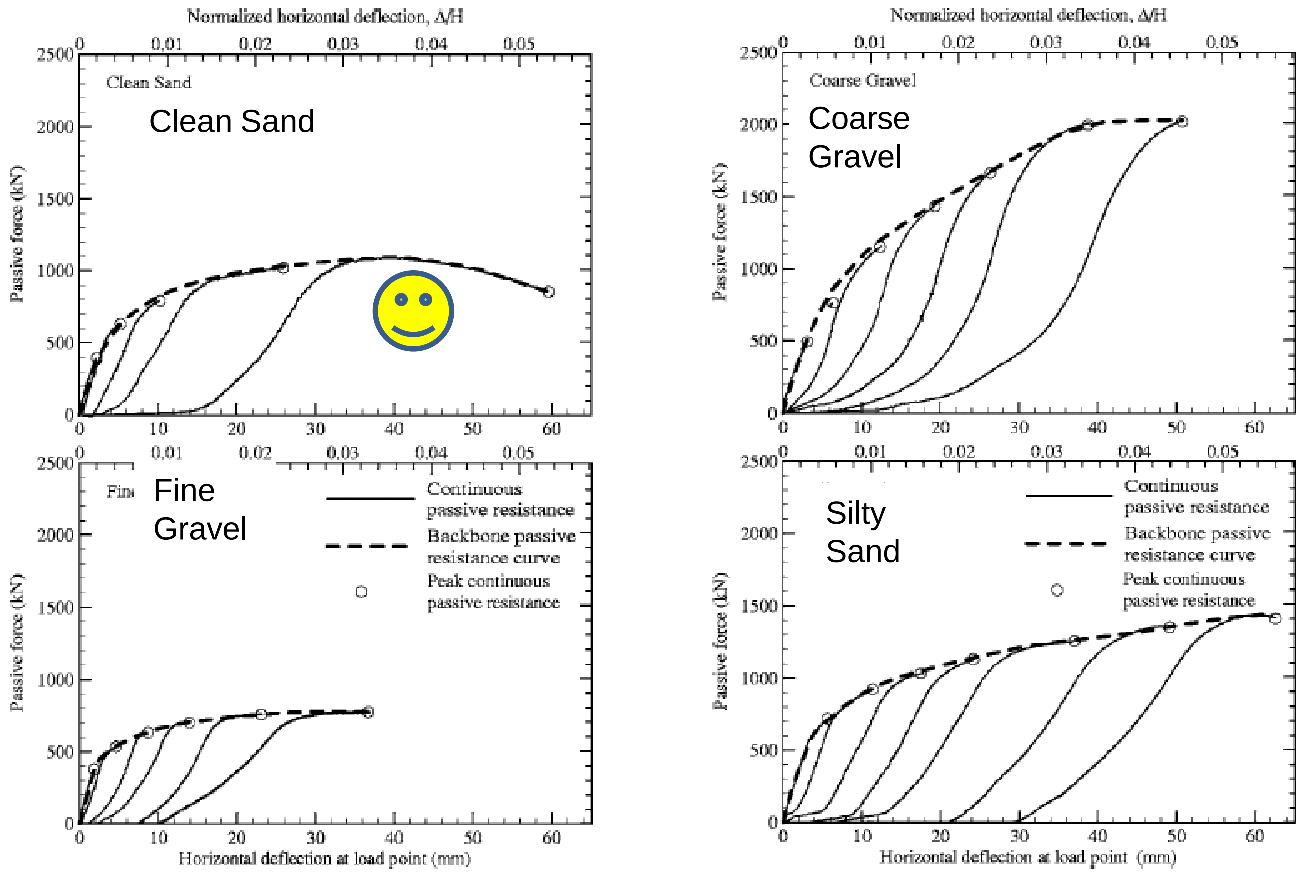


Fig. 14. Variation of K_h with wall movement for loose, medium dense, and dense backfill

- Performed passive pressure tests with loose, medium dense and dense sand backfill
- As expected, post-peak softening occurred with the medium dense and dense sand
- Estimated the residual resistance using the residual shear strength

Cole and Rollins (2006), Rollins and Cole (2006)





- Performed cyclic load tests on a pile cap, with and without backfill (4 different soils)
- Log Spiral provided good estimate of peak passive resistance
- Duncan and Mokwa (2001) hyperbolic model provided best agreement for monotonic loading
- Developed new model for cyclic loading conditions

Shamsabadi, Rollins and Kapuskar (2007)

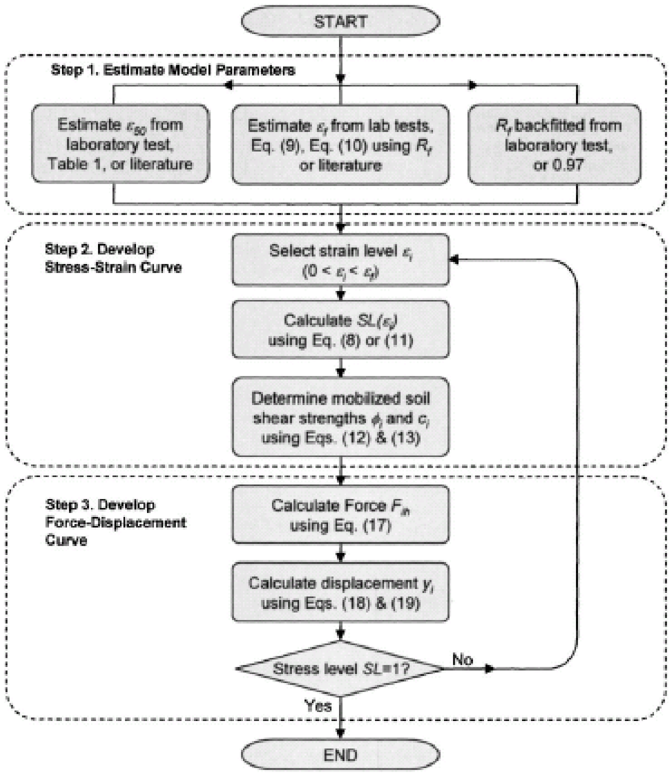


Fig. 6. Flowchart of LSH procedure

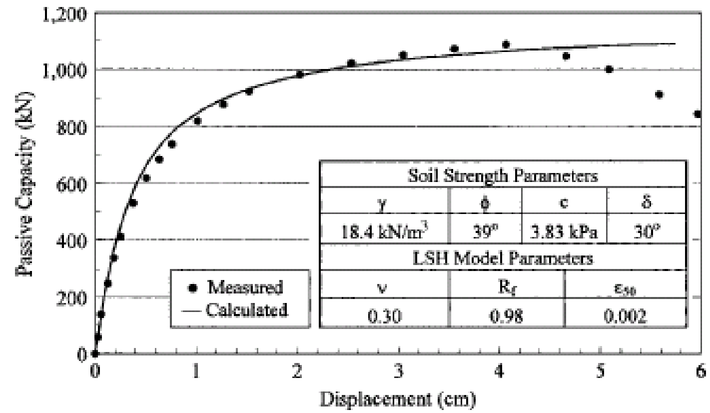


Fig. 9. Passive capacity of BYU's pile cap in clean sand

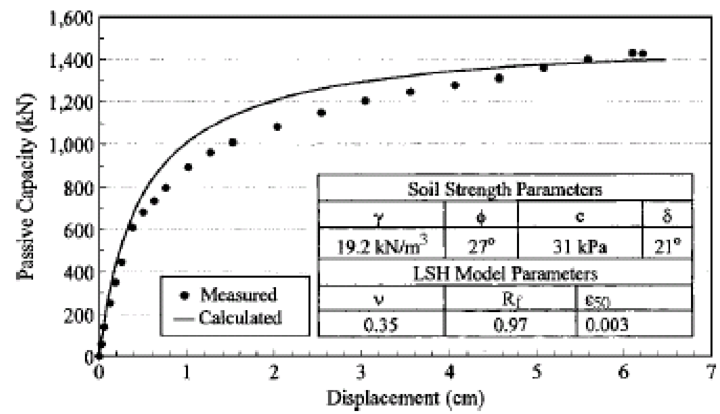
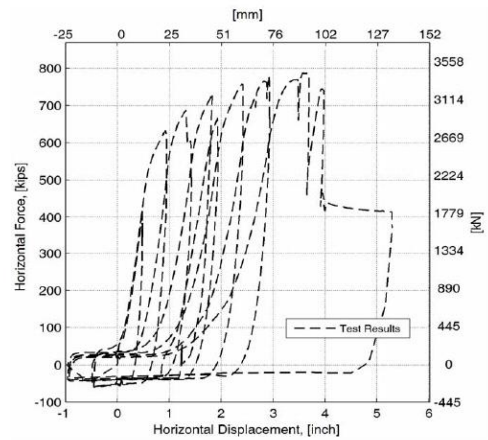


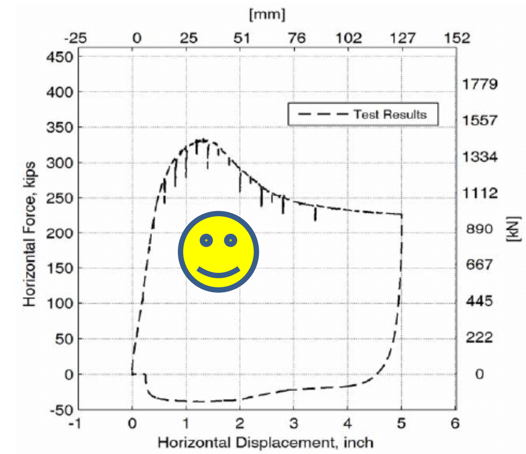
Fig. 10. Passive capacity of BYU's pile cap in silty sand

$$F(y_i) = \frac{F_{\text{ult}}(2Ky_{\max} - F_{\text{ult}})y_i}{F_{\text{ult}}y_{\max} + 2(Ky_{\max} - F_{\text{ult}})y_i}$$

- Proposed model considering Log Spiral failure surfaces and hyperbolic soil stress-strain behavior (LSH model)
- Found good agreement with experimental results
- Simple Hyperbolic Force-Displacement (HFD) equation above also provides the same relationship using only the backfill soil stiffness and ultimate capacity

Bozorgzadeh and Ashford Tests at UCSD (2007)



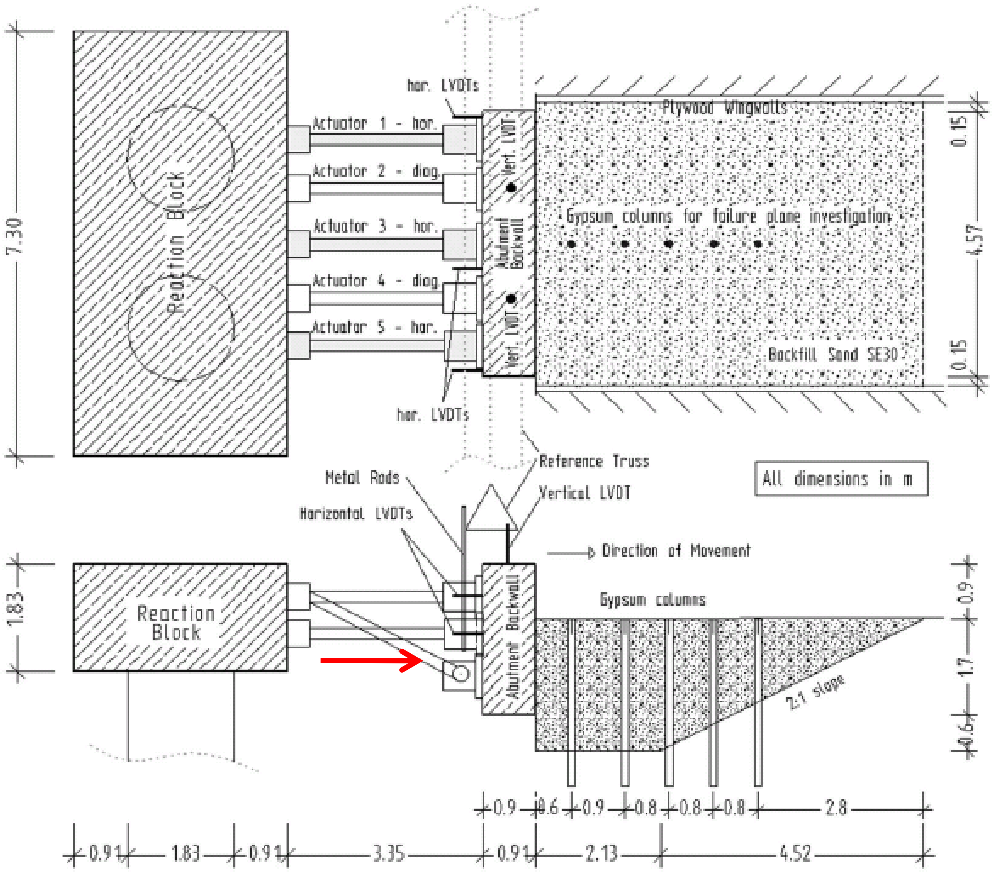


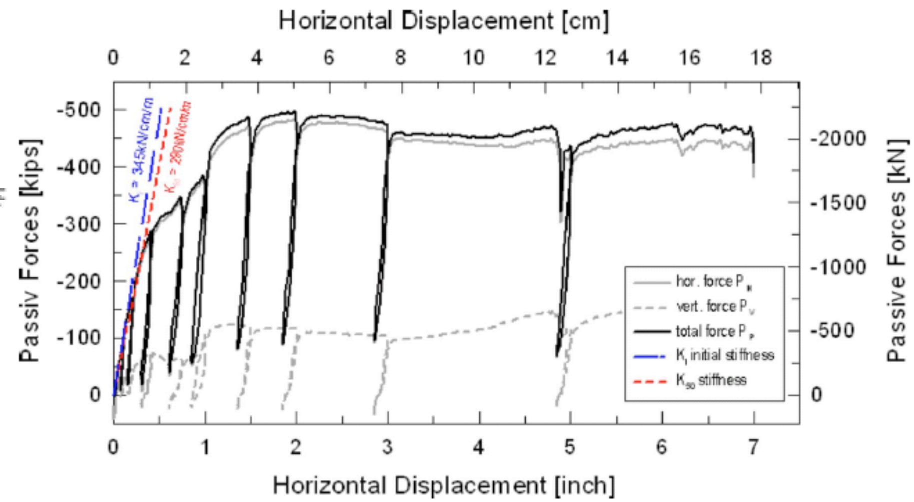
Test 1 Test 2

- Performed 5 tests on 1.5, 1.7 and 2.3 meter tall walls
- Silty sand and clayey sand backfills
- Log Spiral provided good estimate of the peak resistance
- Significant post-peak strain softening was observed in tests where the wall moved up

Elgamal/Wilson (UCSD)

Lemnitzer, Ahlberg, Nigbor, Shamsabadi, Wallace, Stewart (2009)





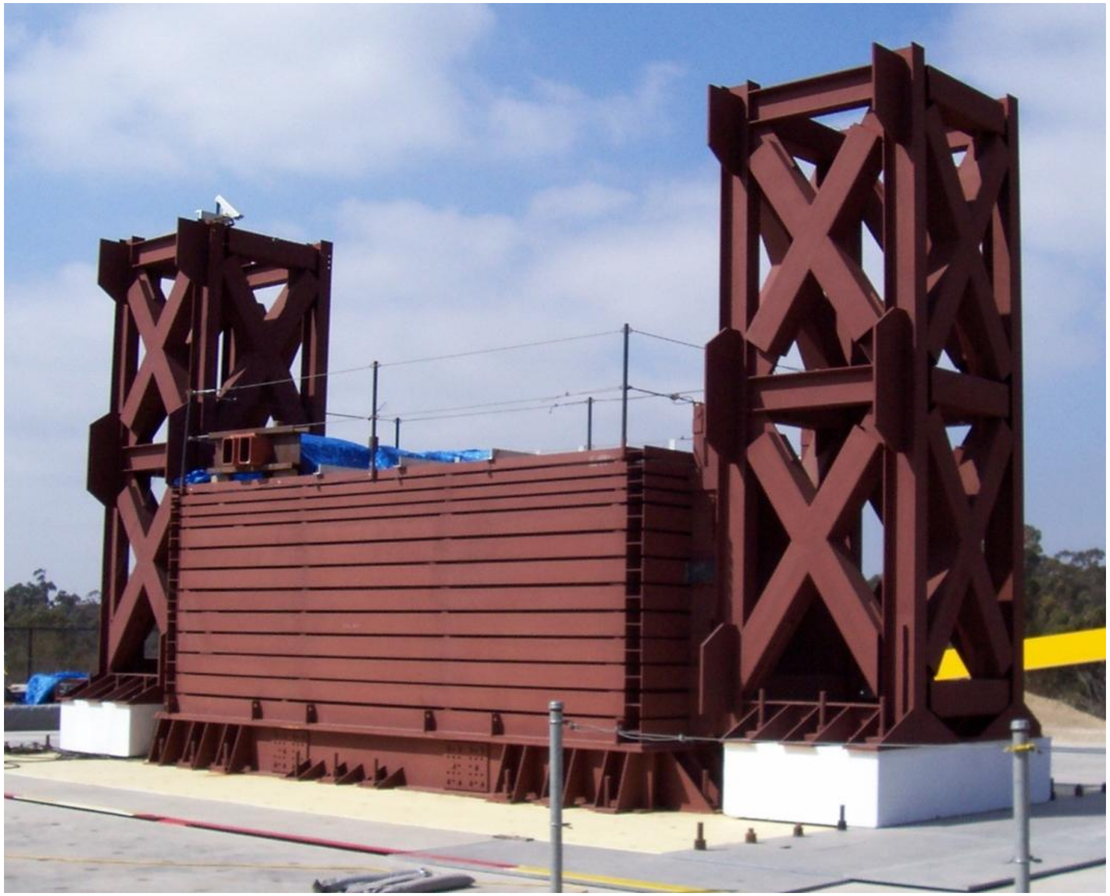
•Performed test on 1.7 meter tall wall with well-graded silty sand backfill

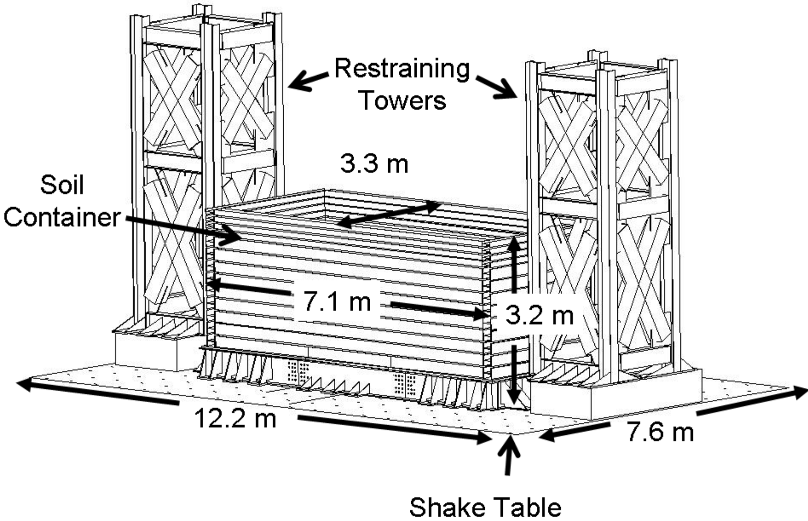
•Peak resistance was well estimated by the Log Spiral method

•Shape of the Duncan and Mokwa (2001) and Shamsabadi et al. (2007) hyperbolic curves provided a good match with the load-displacement behavior

•Wall-soil friction δ_{mob} was approximately **14 degrees** as the peak resistance occurred

UC San Diego Earth Pressure Experiments

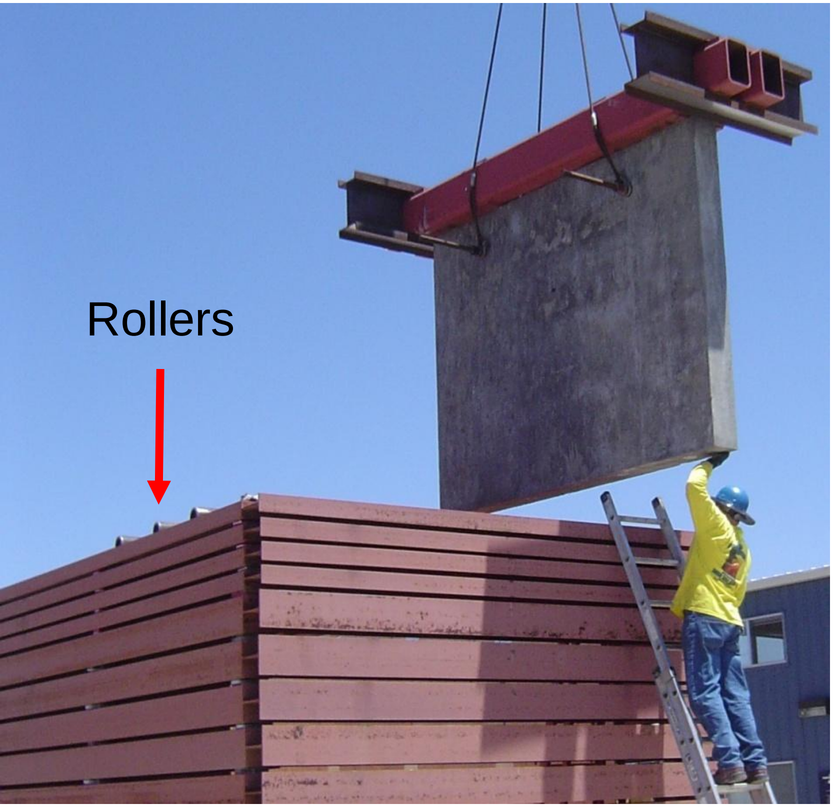


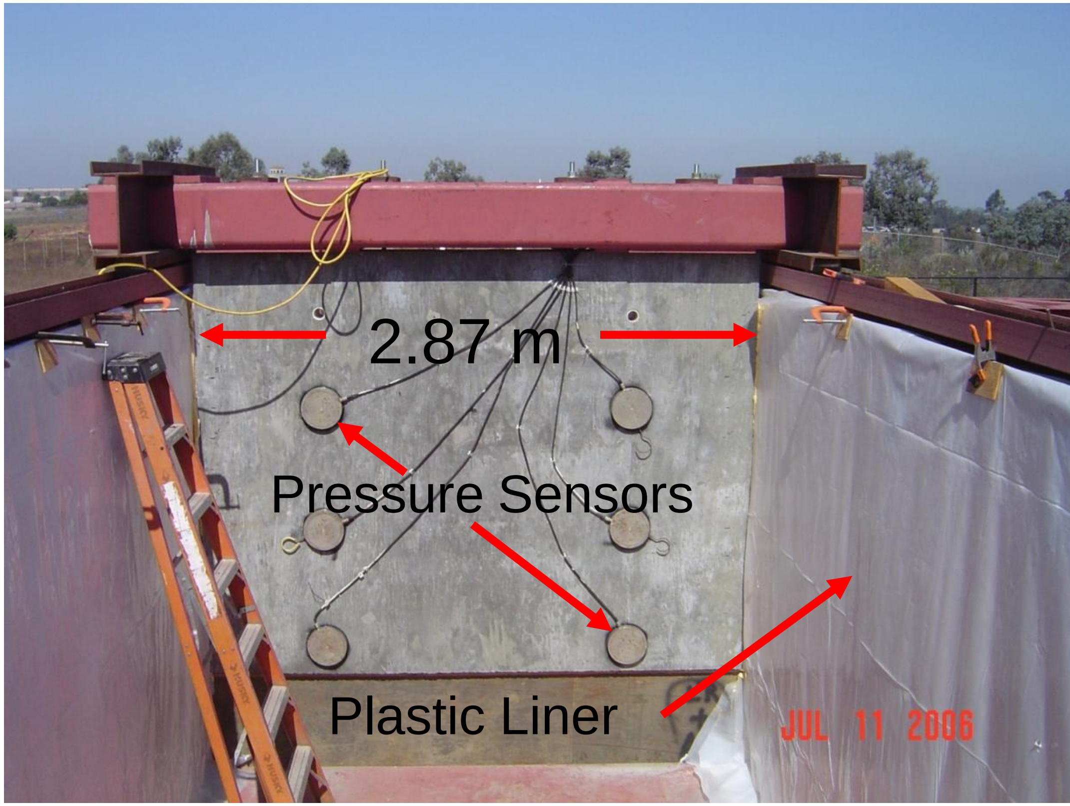


- Lateral earth pressure test series conducted on the outdoor shake table at the Englekirk Structural Engineering Center
- 2 passive earth pressure load-displacement tests
- 26 shake table excitation dynamic earth pressure tests
- Passive and dynamic earth pressure FE simulations

Elgamal/Wilson (UCSD)

Test Configuration





- Vertical test wall suspended from a supporting beam resting on rollers
- 2.87 m wide plane strain section
- Soil container inside walls lined with 3 layers of smooth plastic
- Pairs of pressure sensors mounted at 3 depths

Backfilling, Compaction and Verification

- Well graded sand with 7% silt and up to 7% fine gravel (SW-SM) was used in all tests
- 95% relative compaction
- Verified by nuclear gauge

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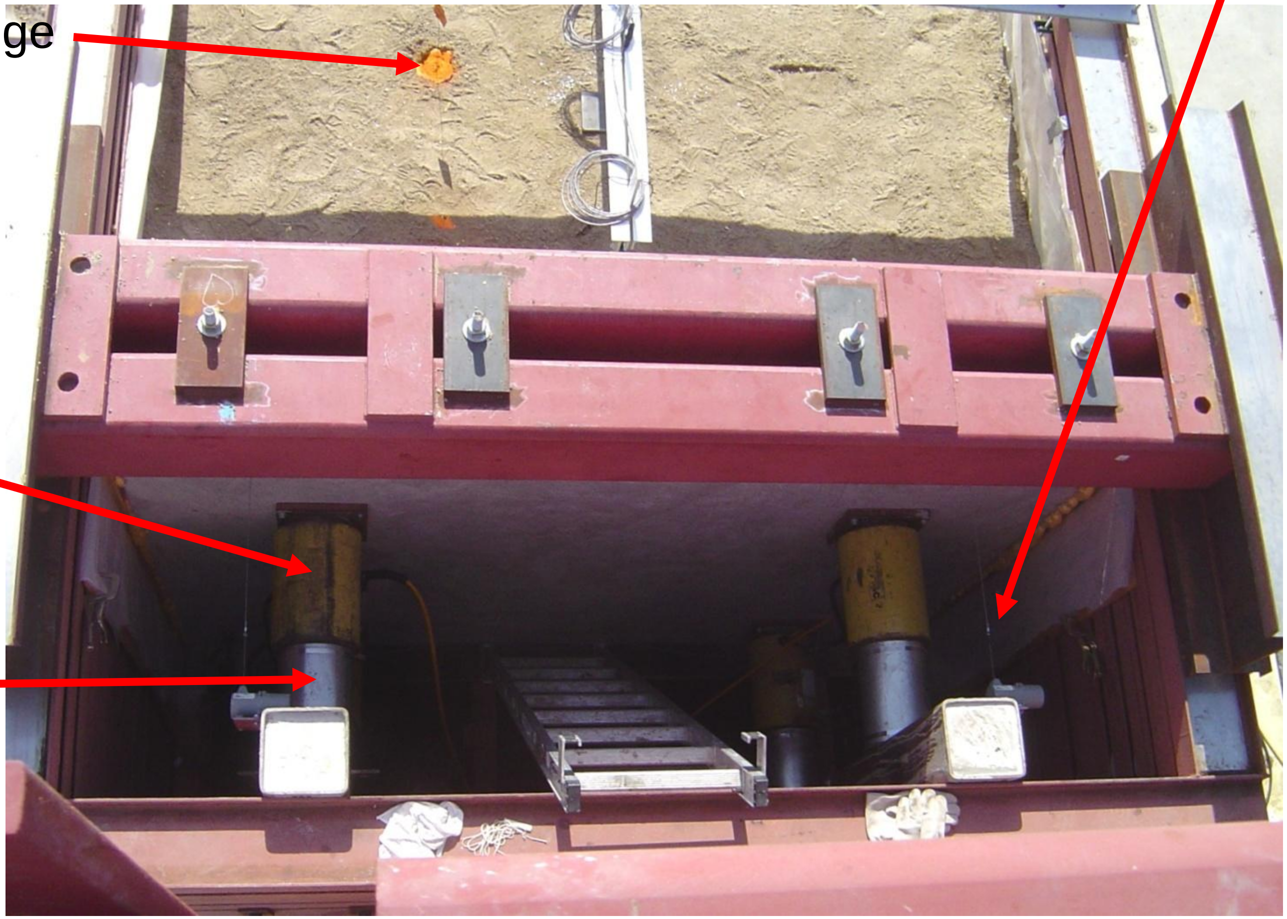
Test Configuration

Foam Cores to identify passive failure wedge (Test 1)

String Potentiometers

Hydraulic Jacks

Load Cells

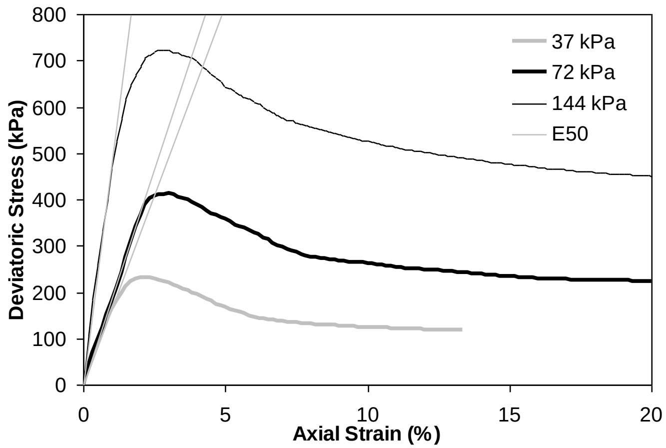


- Test wall could be pushed into the backfill using 4 hydraulic jacks
- Jacks were connected through a manifold system to allow independent control
- Reaction was measured by 4 load cells mounted behind the jacks

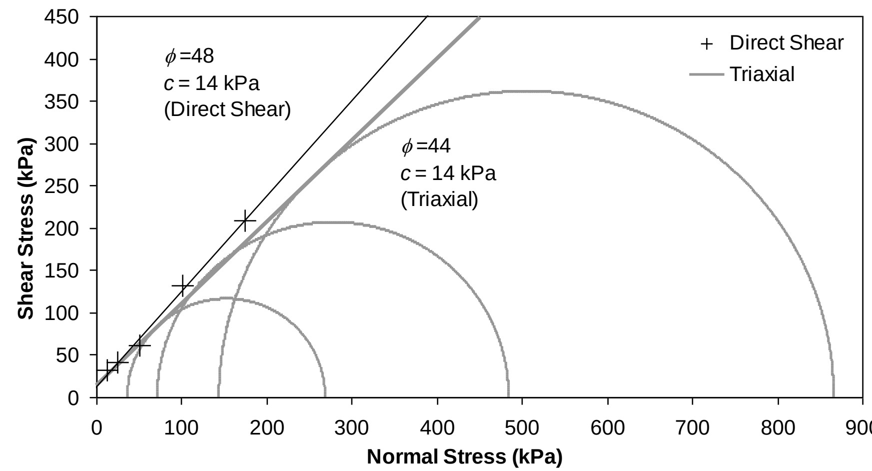
Backfill Soil Shear Strength

Laboratory Test	Φ (deg)	c (kPa)	Φ_r (deg)	c_r (kPa)
Triaxial	44	14	36	6
Direct Shear	48	14	35	8

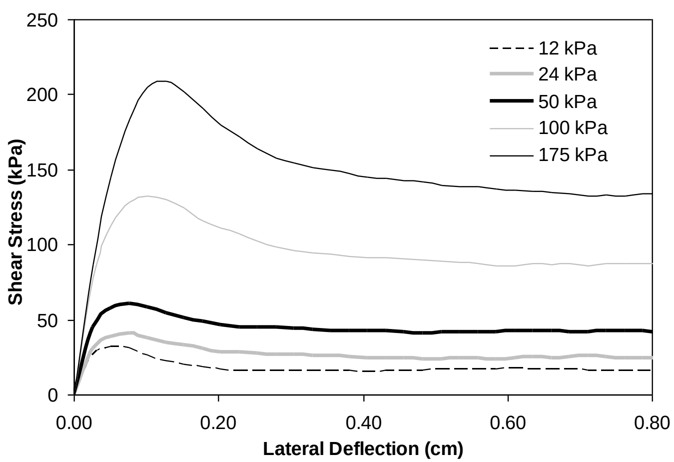
Stress-strain from triaxial



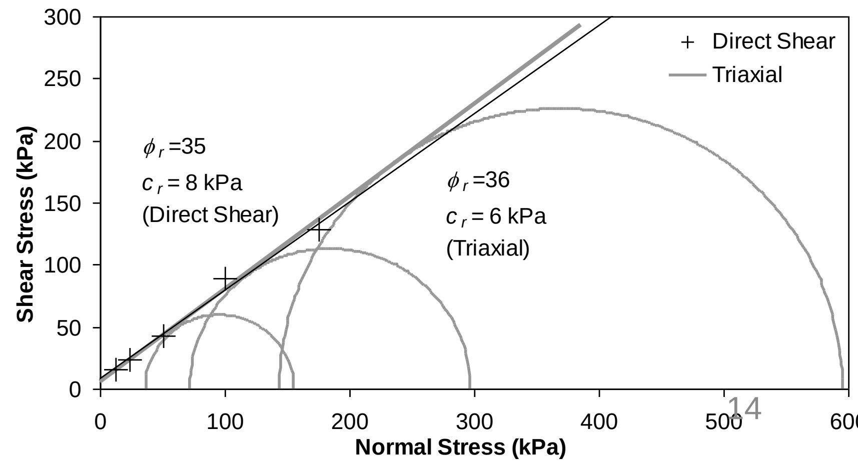
Peak



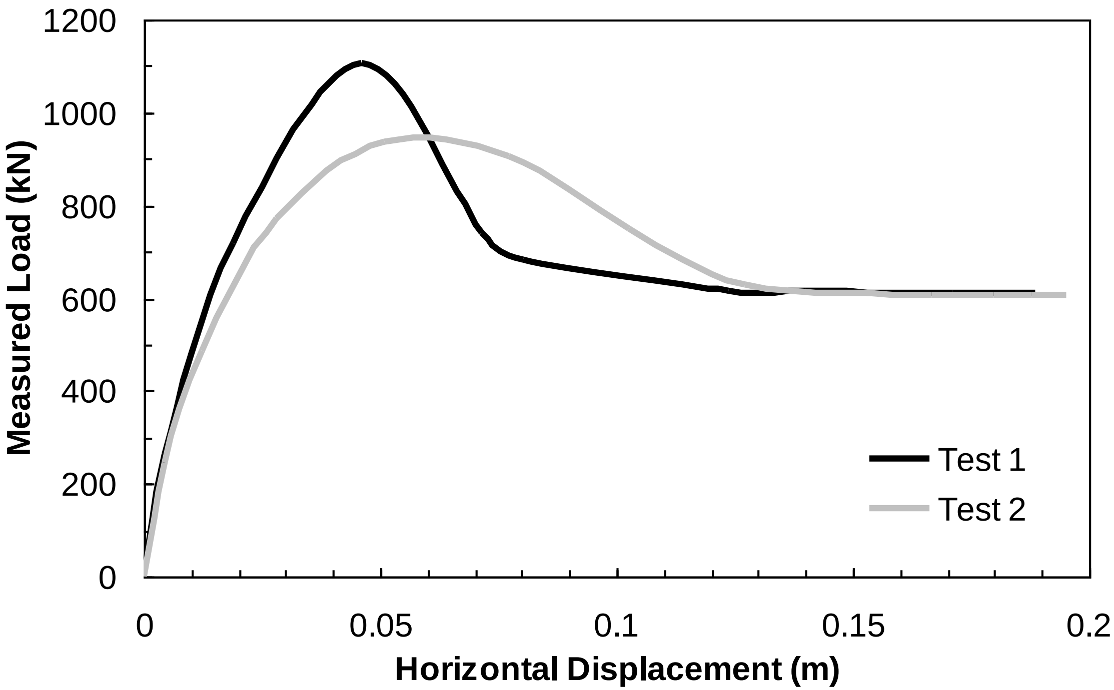
Stress-deflection from direct shear



Residual



Passive Earth Pressure Load-Displacement Tests



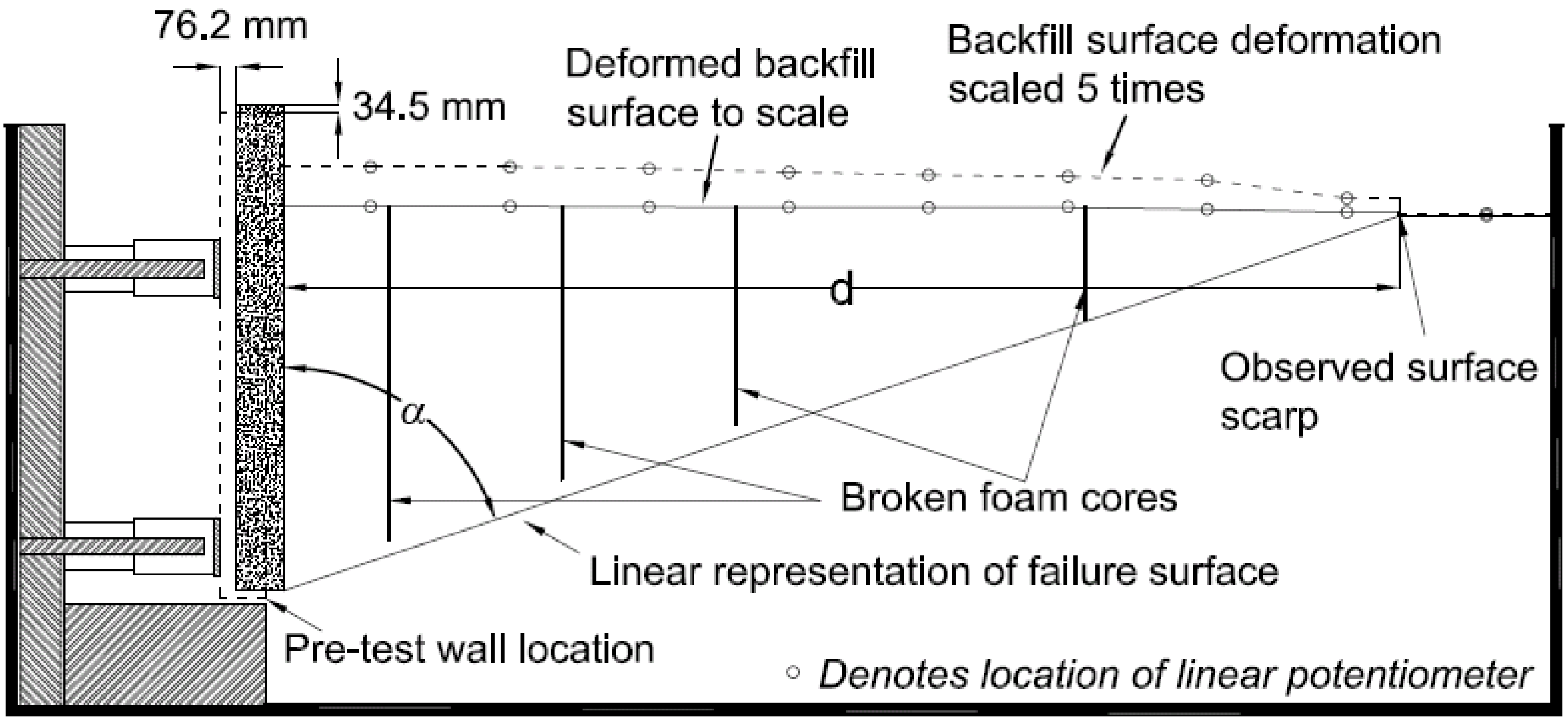
Installed breakable foam cores

Excavating foam cores

Test 1 passive wedge shape

- Test 1: 23 days between backfill construction and testing (drier condition)
- Test 2: 3 days between backfill construction and testing (closest to the lab shear test moisture content)
- Test wall moved up with adjacent backfill, low δ_{mob}
- Triangular passive failure wedge shape

Elgamal/Wilson (UCSD)



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Passive Earth Pressure Load-Displacement Tests

Test 1(drier condition) Source of shear strength parameters	Φ (deg)	c (kPa)	Log Spiral/Coulomb Value (kN/m)	Log Spiral/Coulomb % of measured
Estimated from Failure Wedge[a]	51[b]	11.5[b]	378	98
Triaxial Test	44	14	295	77
Direct Shear	48	14	350	91

[a]Field moisture content at time of test lower than the lab condition

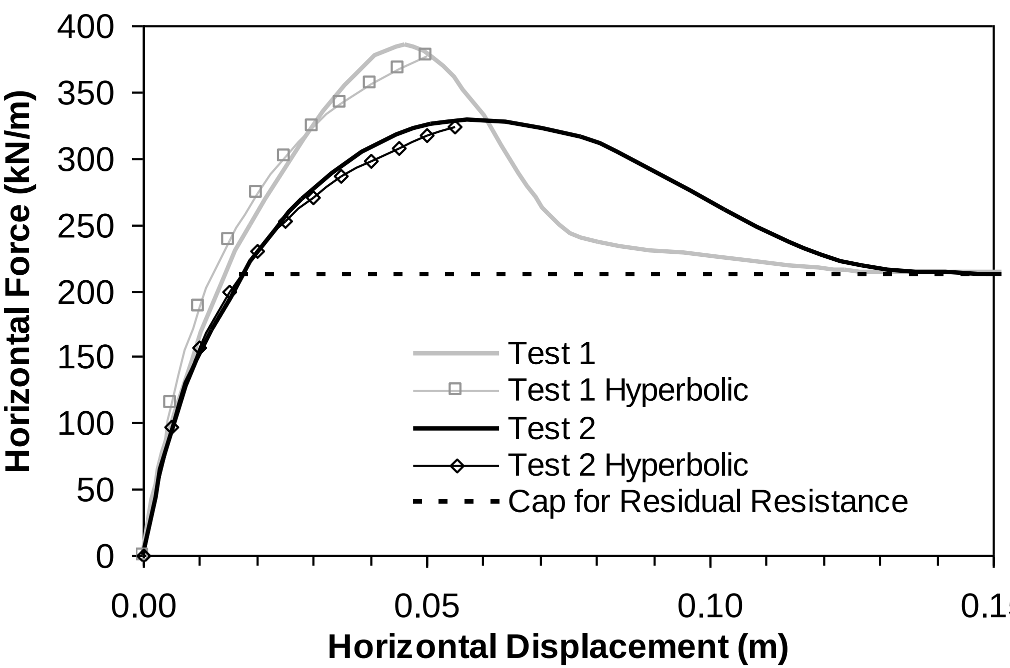
[b]Average for a range of ϕ = 50 - 52 and c = 10 - 13

Test 2 (closest to the lab shear test condition) Source of shear strength parameters	Φ (deg)	c (kPa)	Log Spiral/Coulomb Value (kN/m)	Log Spiral/Coulomb % of measured
Estimated from Failure Wedge	45[a]	14.5[a]	315	97
Triaxial Test	44	14	299	92
Direct Shear	48	14	355	109

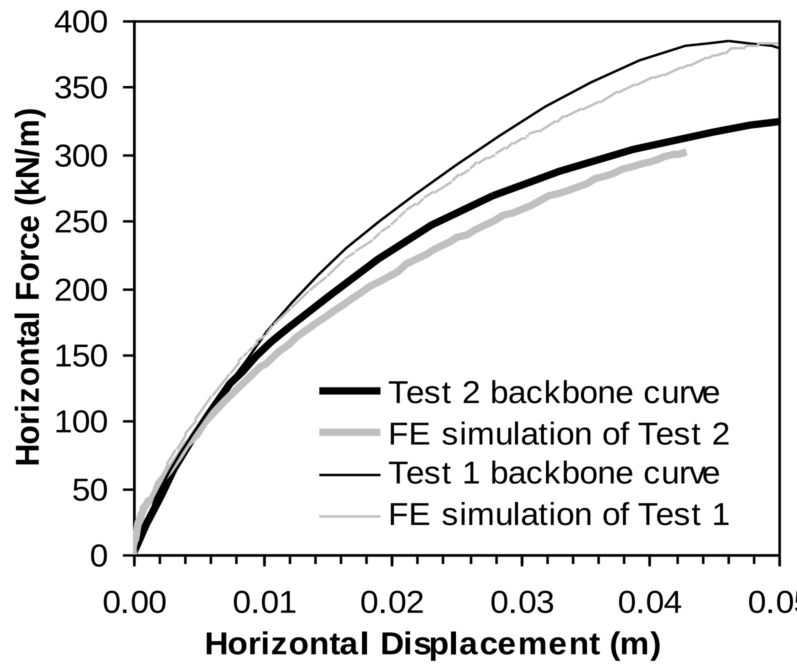
[a]Average for a range of ϕ = 44 - 46 and c = 13 - 16

Passive Earth Pressure Load-Displacement Tests

Hyperbolic model comparison



Plaxis (2D) FE model comparison



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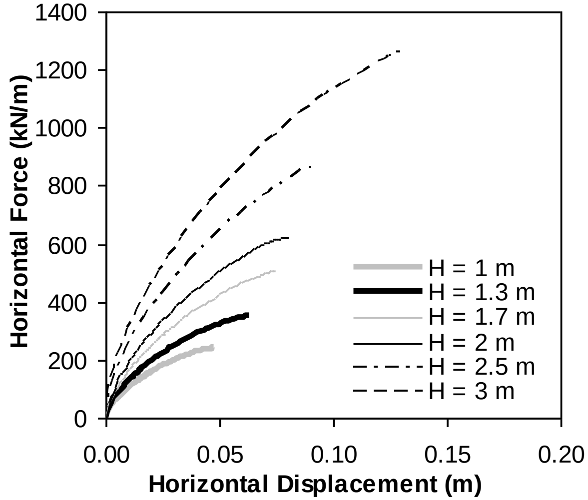
Passive Pressure Load-Displacement Simulations for Additional Soils and Wall Heights

Test Soil:

ϕ = 46 deg

c = 14 kPa

E_{50}^{ref} = 40 Mpa

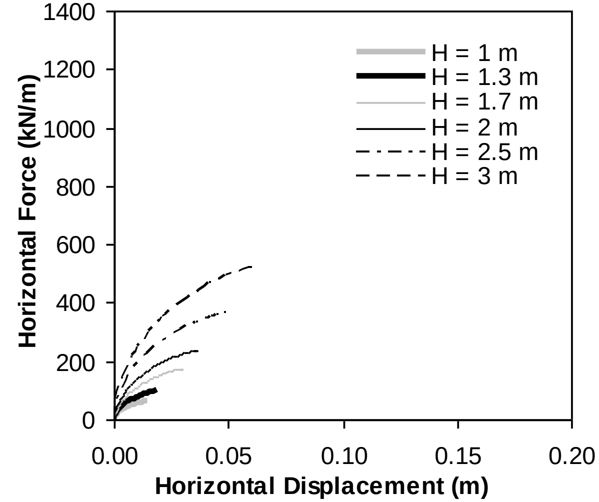


Dense Clean Sand:

ϕ = 38 deg

c = 0 kPa

E_{50}^{ref} = 35 Mpa

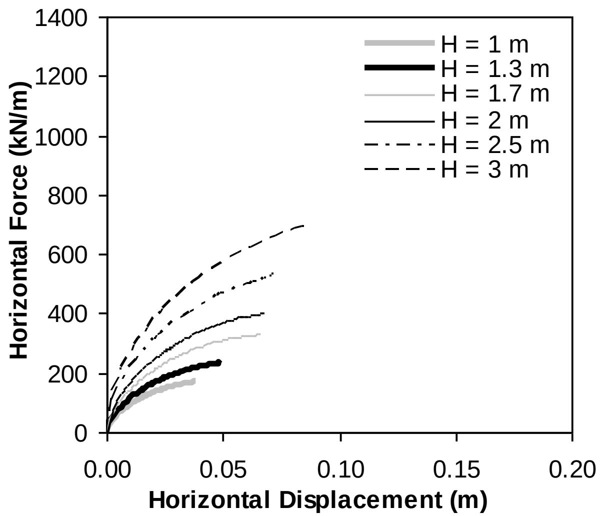


Medium-Dense Silty Sand:

ϕ = 33 deg

c = 24 kPa

E_{50}^{ref} = 30 Mpa

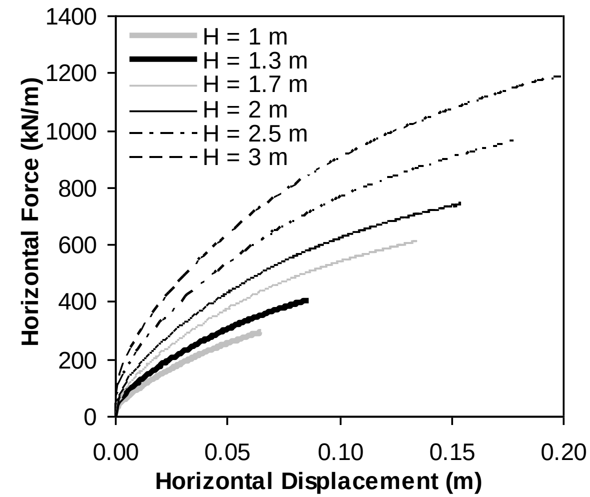


Medium-Dense Clayey Sand:

ϕ = 23 deg

c = 95 kPa

E_{50}^{ref} = 30 Mpa



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OpenSees Spring Model

- "HyperbolicGapMaterial"
- Matthew Dryden and Greg Fenves (UC Berkeley)
- Hyperbolic model parameters also calibrated for all curves on the previous slide

Hyperbolic Gap Material

Contact Authors: Matthew Dryden: dryden@berkeley.edu
Patrick Wilson: prwilson@ucsd.edu

This command is used to construct a hyperbolic gap material object.

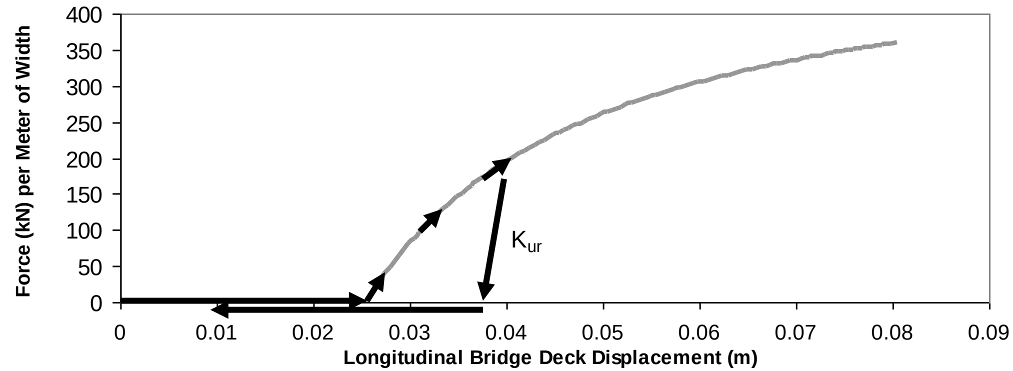
uniaxialMaterial HyperbolicGapMaterial $matTag $Kmax $Kur $Rf $Fult $gap

$matTag	unique material object integer tag
$Kmax	initial stiffness
$Kur	unloading/reloading stiffness
$Rf	failure ratio
$Fult	ultimate (maximum) passive resistance*
$gap	initial gap*

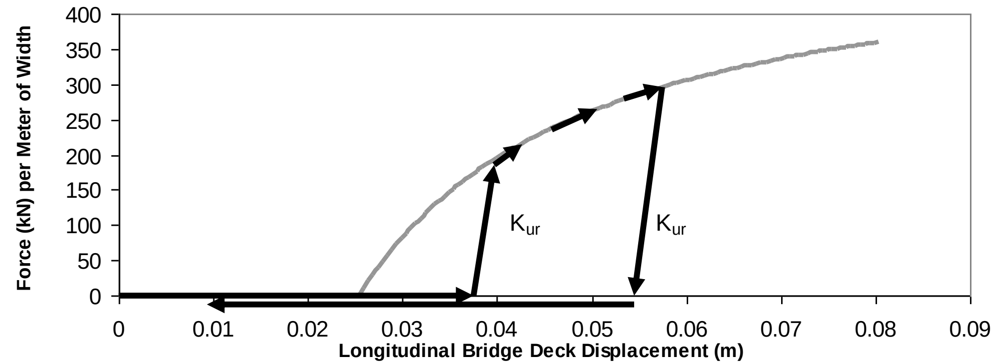
$$F(x) = \frac{x}{\dfrac{1}{K_{max}} + R_f \dfrac{x}{F_{ult}}}$$

Duncan and Mokwa (2001)

Representative initial loading cycle



Representative subsequent loading cycle



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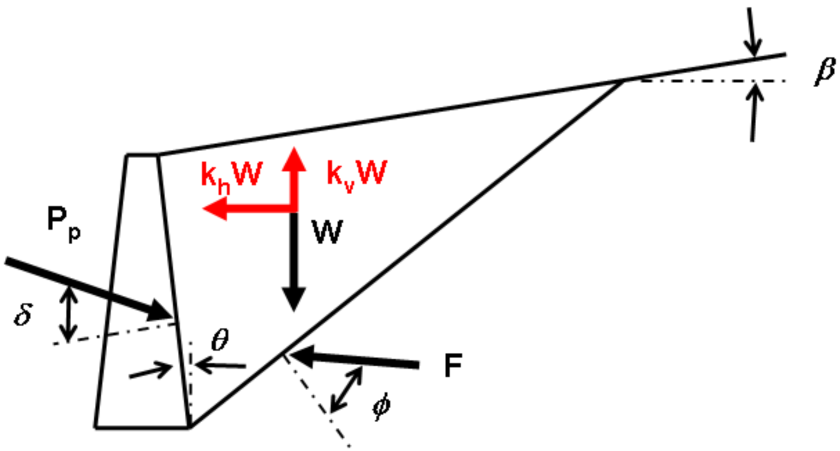
Dynamic Earth Pressure and Retaining Walls (*Behavior During Earthquakes)*

- Some structures have failed or been damaged
  - Often due to a weak (for instance liquefiable) underlying layer
  - Some cases where damage occurred with dry backfill include:
    - U-shaped channel floodway structures in 1971 San Fernando (Clough and Gragaszy 1977)
    - Masonry and reinforced concrete walls in 1995 Kobe (Gazetas et al. 2004)
    - Gravity walls in 1999 Chi-Chi (Fang et al. 2003)
- Others have performed well, even in cases where earthquake loading was not considered in the design
  - Examples include:
    - Anchored walls in 1994 Northridge (Lew et al. 1995)
    - Reinforced concrete walls in 1995 Kobe (Gazetas et al. 2004)
    - Walls in 2008 Great Sichuan Eq. (Sitar and Al Atik 2009)

Mononobe-Okabe (1926, 1929)

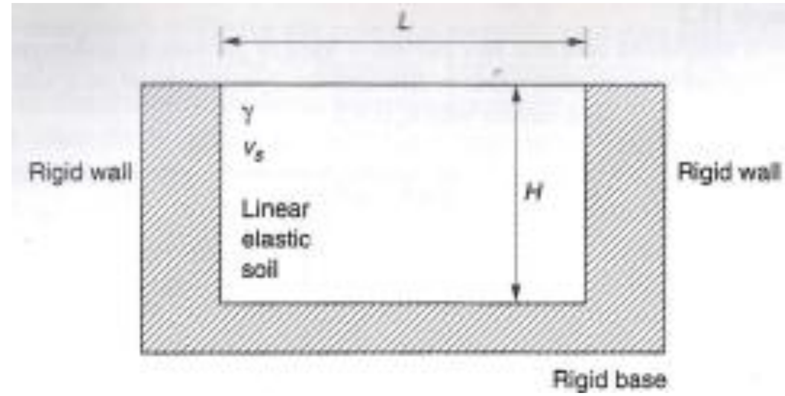


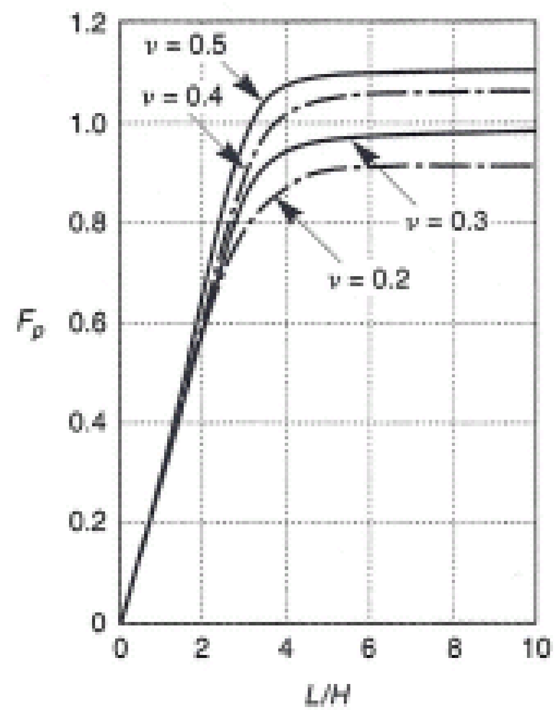
$$P_{AE} = \frac{1}{2} K_{AE} \gamma H^2 (1 - k_v) \qquad P_{PE} = \frac{1}{2} K_{PE} \gamma H^2 (1 - k_v)$$

$$K_{AE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi)\sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi)\cos(\beta - \theta)}}\right]^2}$$

$$K_{PE} = \frac{\cos^2(\phi - \theta - \psi)}{\cos\psi \cos^2\theta \cos(\delta + \theta + \psi)\left[1 + \sqrt{\dfrac{\sin(\delta + \phi)\sin(\phi - \beta - \psi)}{\cos(\delta + \theta + \psi)\cos(\beta - \theta)}}\right]^2}$$

Wood (1973)

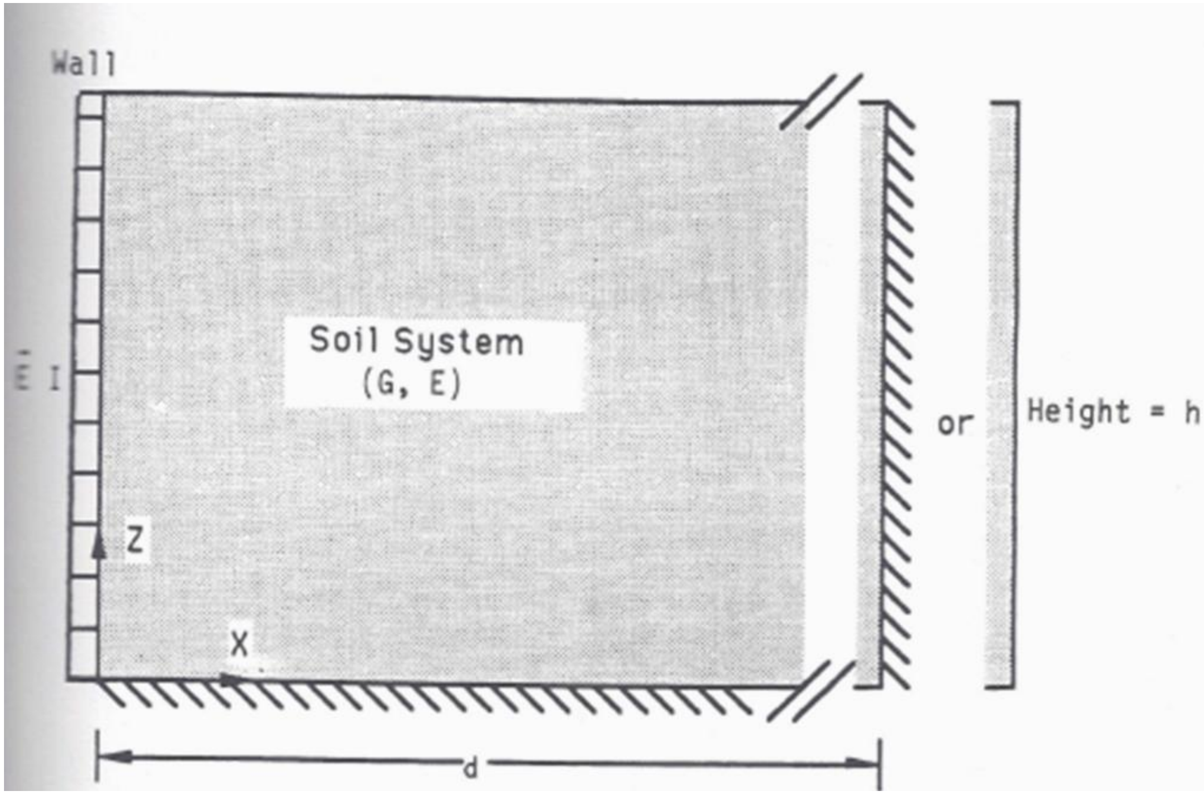




Figures from Kramer (1996)

$$\Delta P_{eq} = \gamma H^2 \frac{a_h}{g} F_p$$

Alampalli and Elgamal (1990)



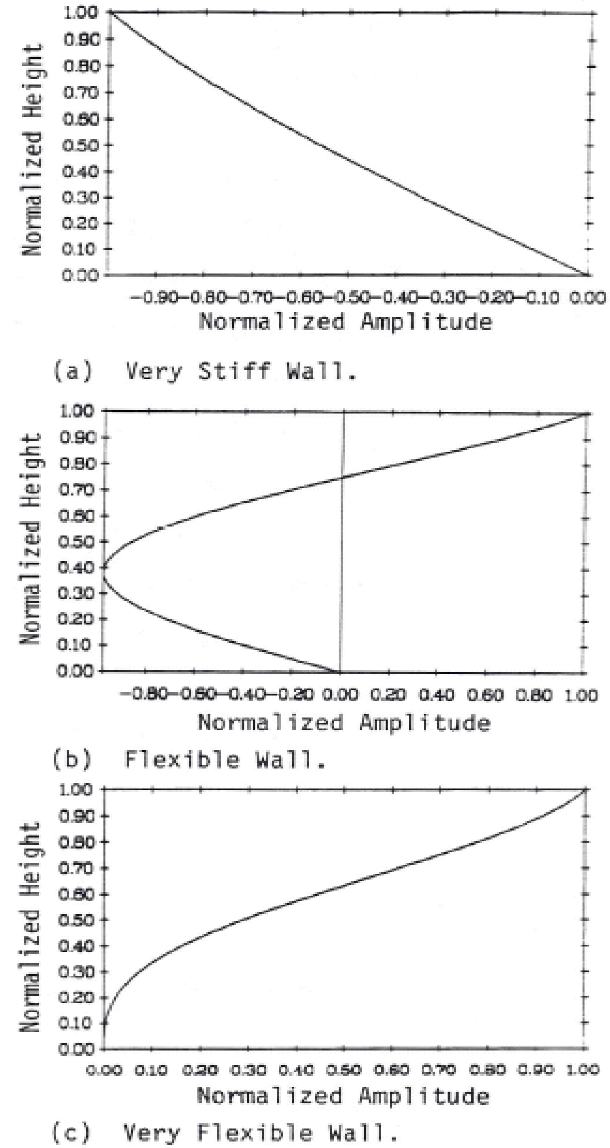


Figure 5.7. Dynamic Pressure Distributions At Wall-Soil Interface.

- •Developed model considering a flexible wall and supporting soil backfill
- •Based on compatibility of wall and backfill mode shapes
- •Found that the dynamic pressure distribution depends on the wall flexibility

Veletsos and Younan (1997)

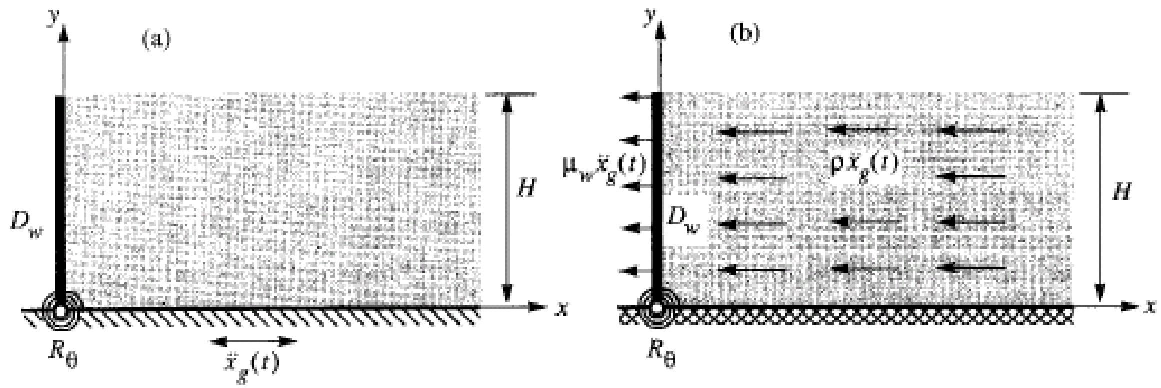


FIG. 1. Soil-Wall System Investigated: (a) Base-Excited System; (b) Force-Excited System

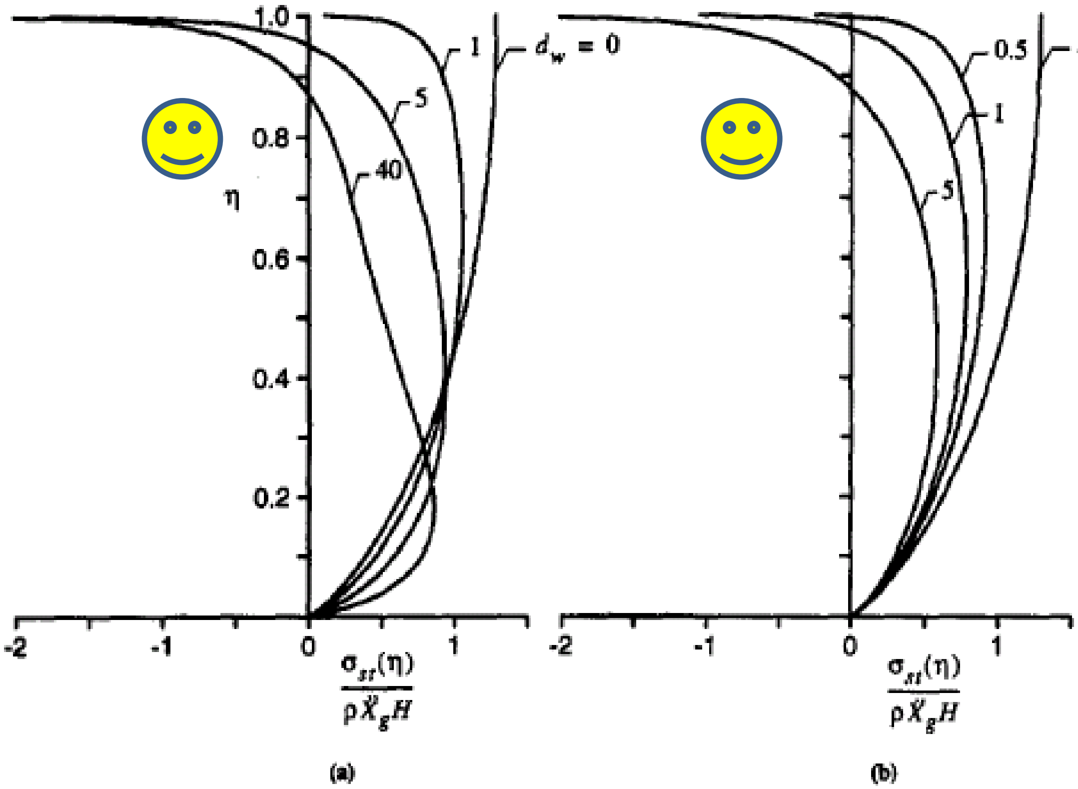


FIG. 2. Distributions of Wall Pressure for Statically Excited Systems with Different Wall and Base Flexibilities ($\nu = 1/3$, $\mu_w = 0$): (a) for $d_\theta = 0$; (b) for $d_w = 0$

- Presented a model considering rotating and bending walls
- Found that the magnitude and distribution of dynamic pressure depends on the wall movement

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Richards, Huang, Fishman (1999)

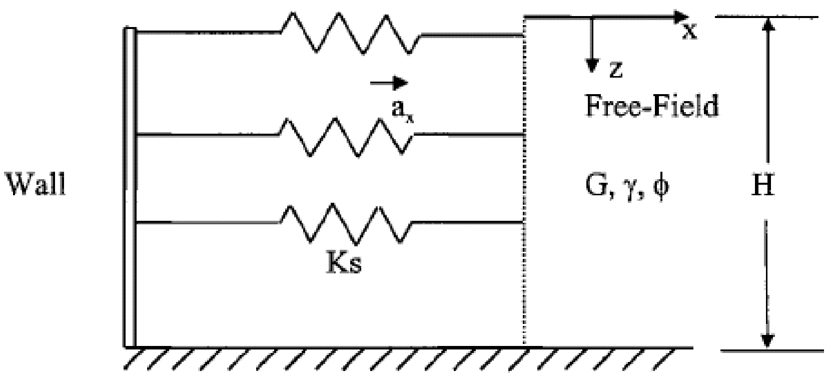
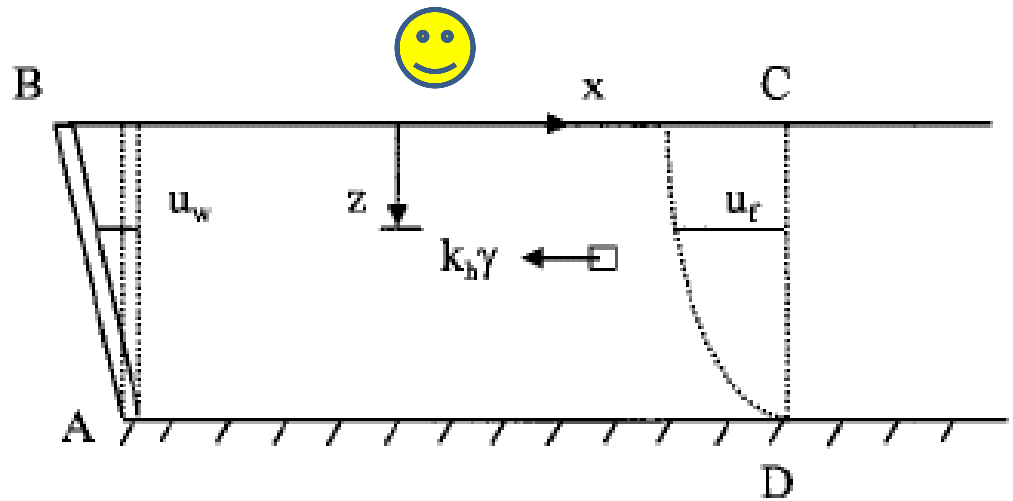


FIG. 1. Model for Dynamic Pressure Increment



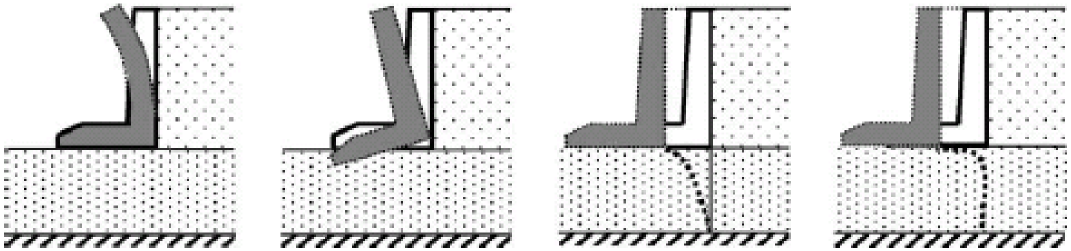
•Developed a kinematic method

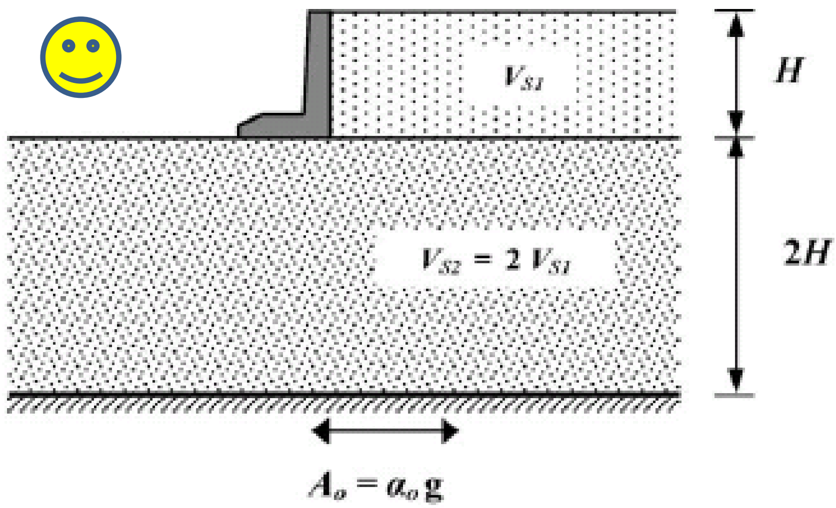
•Used springs to represent the soil

•Dynamic earth pressure on wall depends on free-field stress deformation compared to movement of the wall

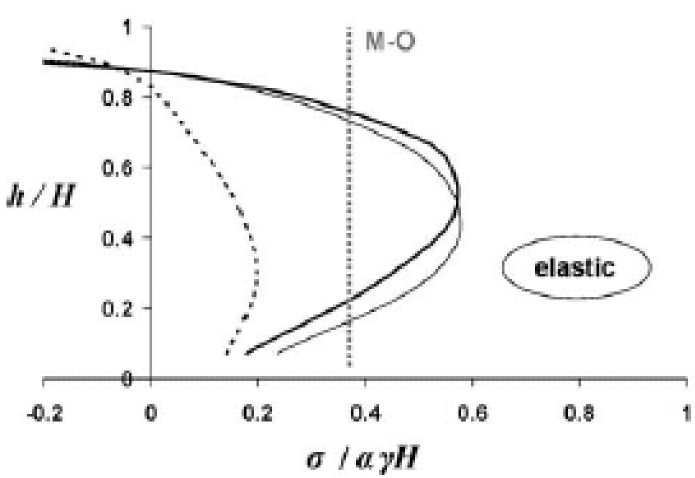
•Included the plastic nonlinear free field response

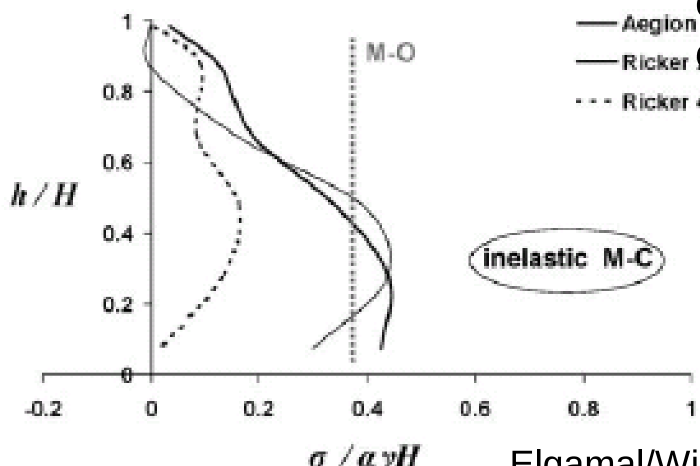
Gazetas, Psarropoulos, Anastasopoulos, Gerolymos (2004)



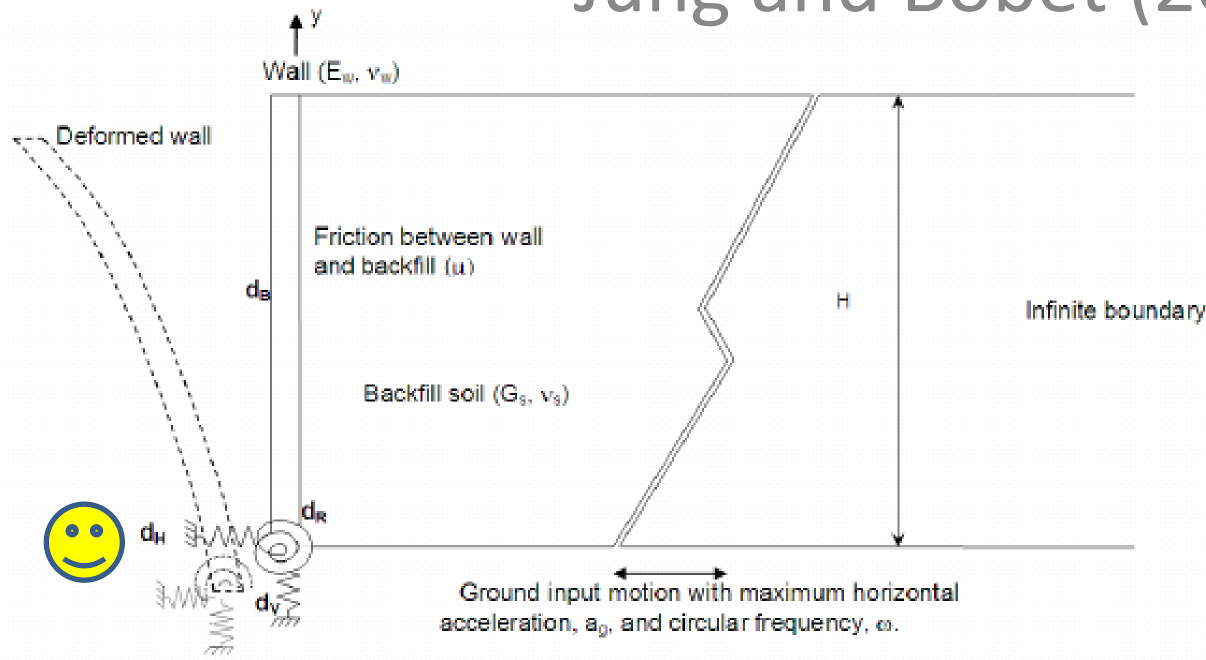


- Used FE models to consider 3 types of retaining walls
  - L-shaped walls
  - Prestressed-anchored pile walls
  - Reinforced soil walls
- Linear & nonlinear soil models
- Found that including realistic effects tends to reduce the damaging effects of dynamic excitation on the walls
  - Wall flexibility
  - Foundation soil flexibility
  - Soil yielding
  - Soil-wall separation

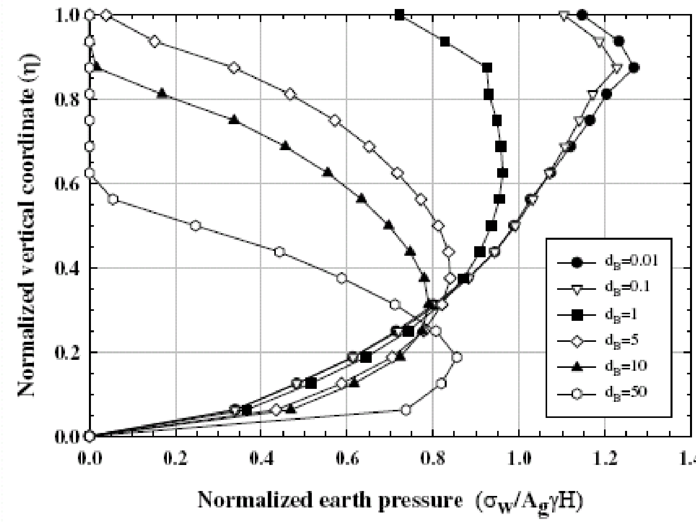




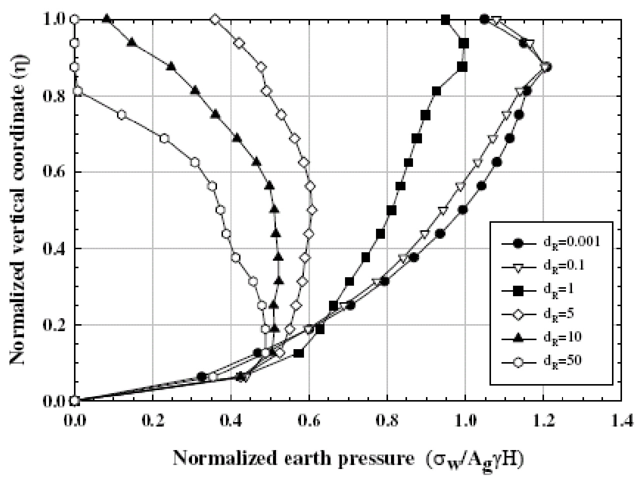
Jung and Bobet (2008)



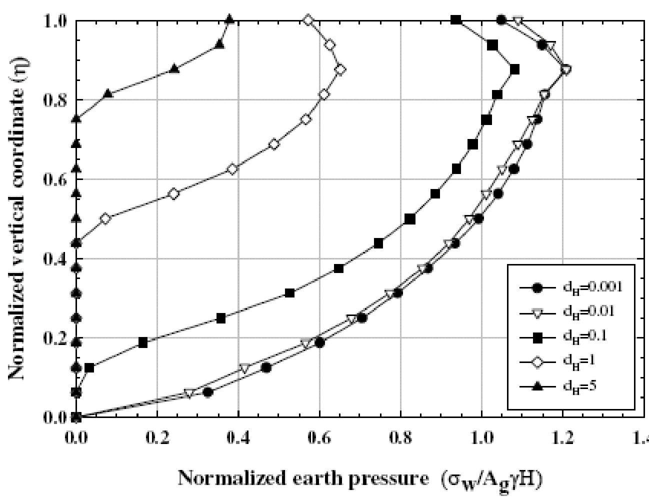
Bending



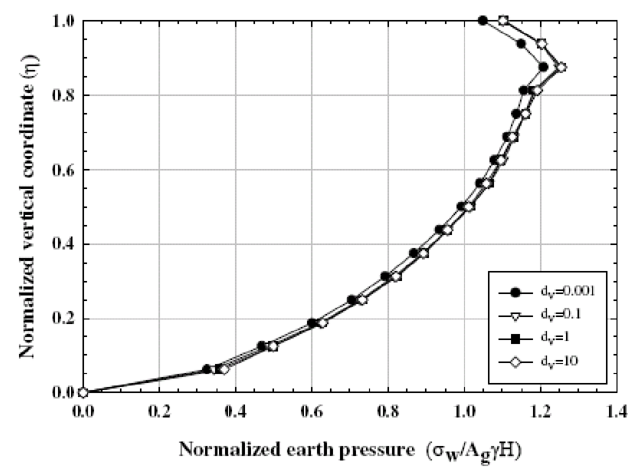
Rotation



Horizontal Translation

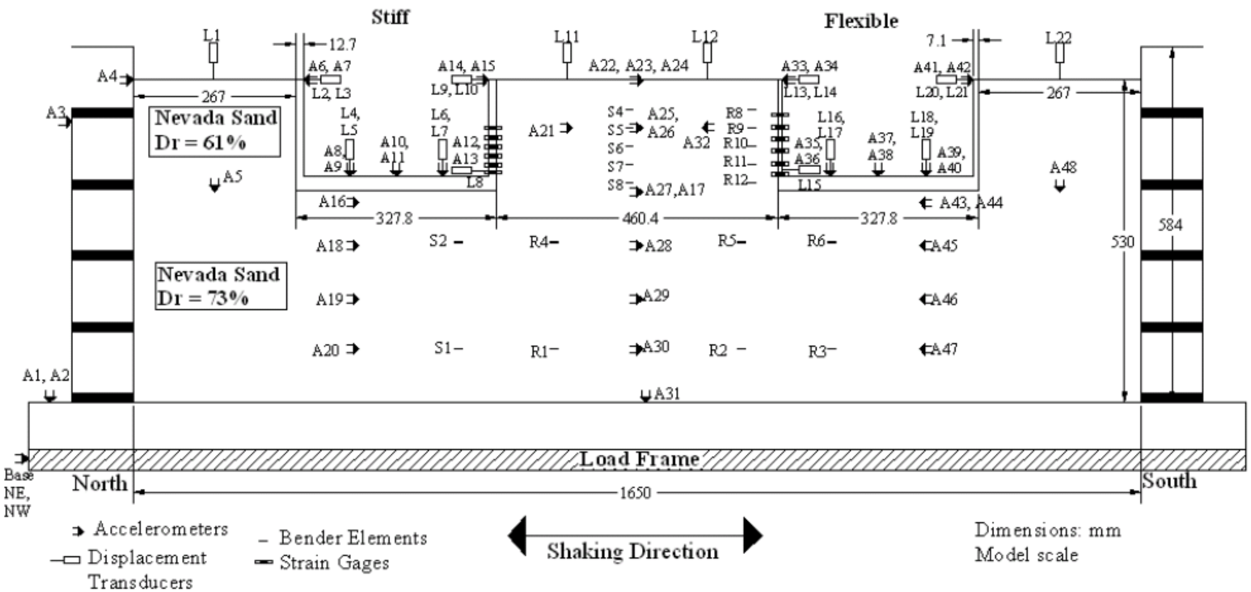


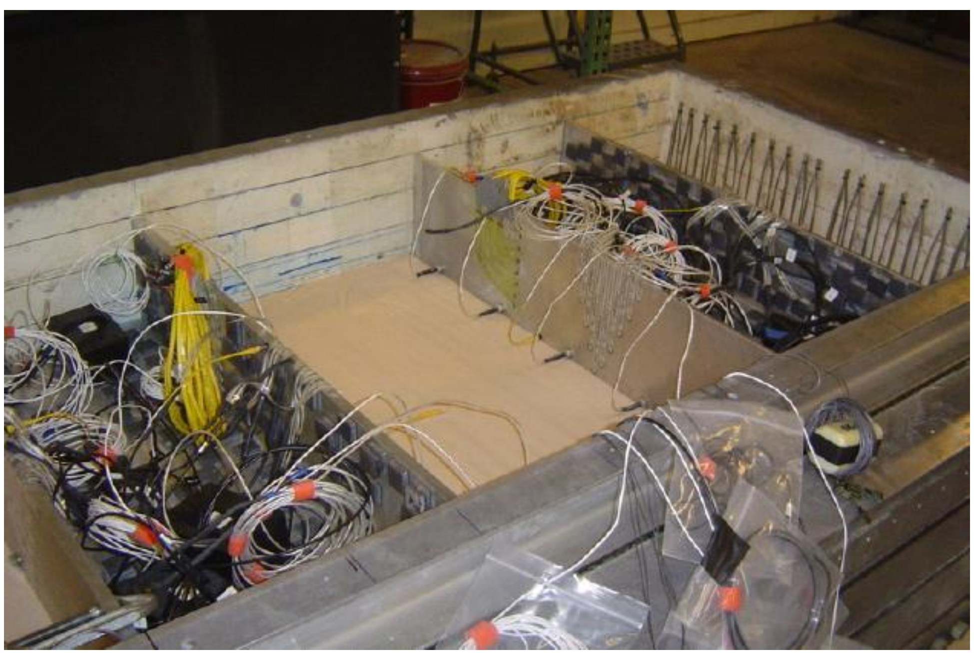
Vertical Translation



Elgamal/Wilson (UCSD)

Al Atik and Sitar (2008)





- •Univ. Calif. Davis centrifuge facility
- •36 g centrifugal acceleration
- •Flexible and stiff U shaped walls
- •6 meter prototype height
- •Found that the point of application of the dynamic **earth pressure resultant was 0.3 of the wall height from the base**
- •Wall inertia contributed significantly to maximum moment
- •**Suggested that designing for maximum dynamic earth pressure is overly conservative**

Elgamal/Wilson (UCSD)

Mononobe-Okabe with Cohesion (NCHRP 2008)

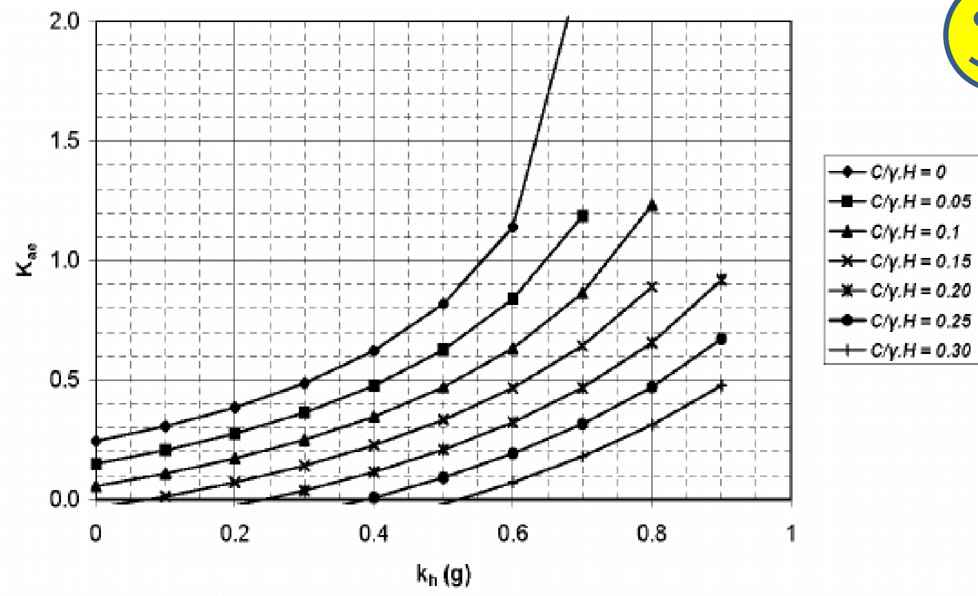


FIGURE 7-11. Seismic Coefficient Charts for c-ϕ Soils for ϕ = 35°

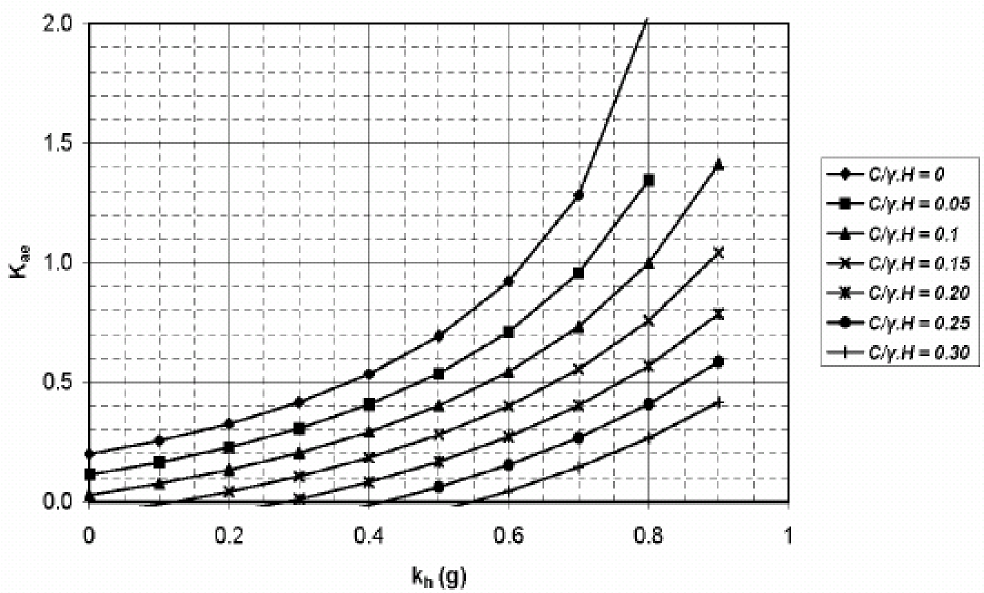
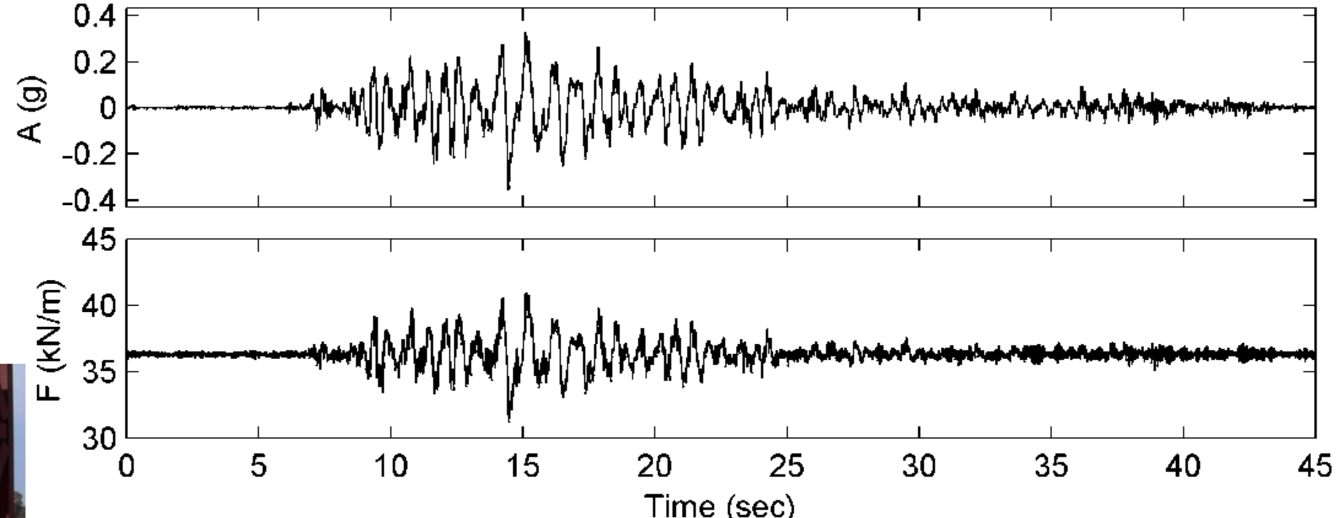


FIGURE 7-12. Seismic Coefficient Charts for c-ϕ Soils for ϕ = 40°

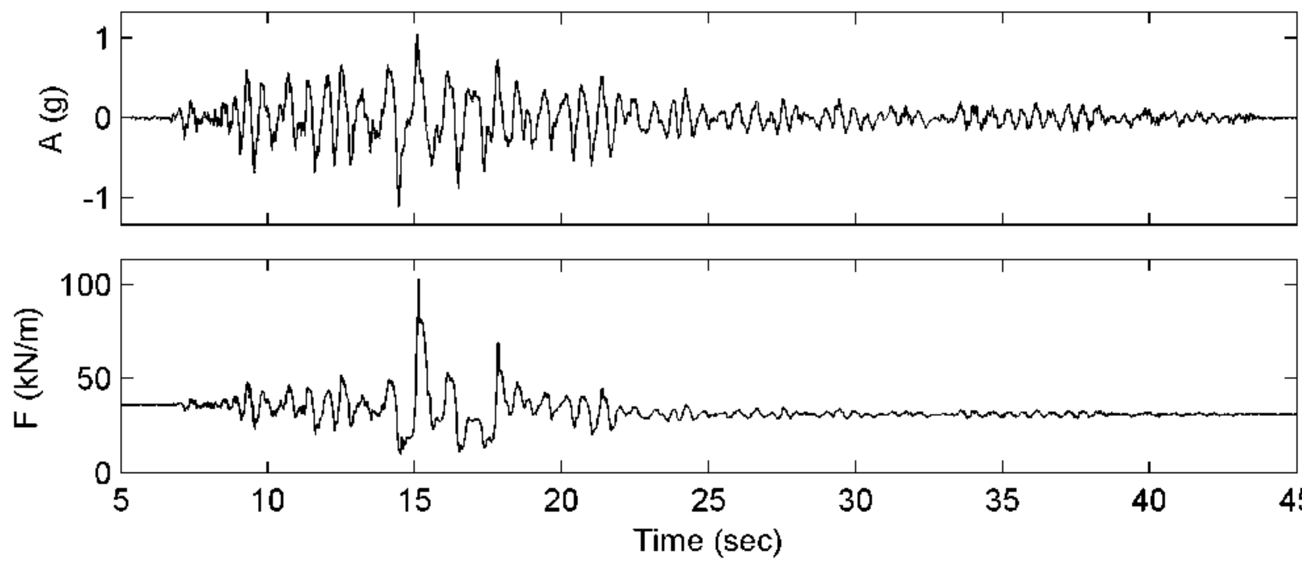
- Many backfill soils exhibit cohesion which is not accounted for in classical Mononobe-Okabe analysis
- By including cohesion, the predicted dynamic active earth pressure can be zero (or negative) up to a significant level of input acceleration
- Several others have also applied the effect of cohesion to the Mononobe-Okabe method including:
  - Saran and Prakash 1968
  - Richards and Shi 1994
  - Shukla et al. 2009

UC San Diego Dynamic Excitation Tests

Base acceleration (A) and total lateral force measured by load cells (F) per meter of wall width from a moderate and a very strong test



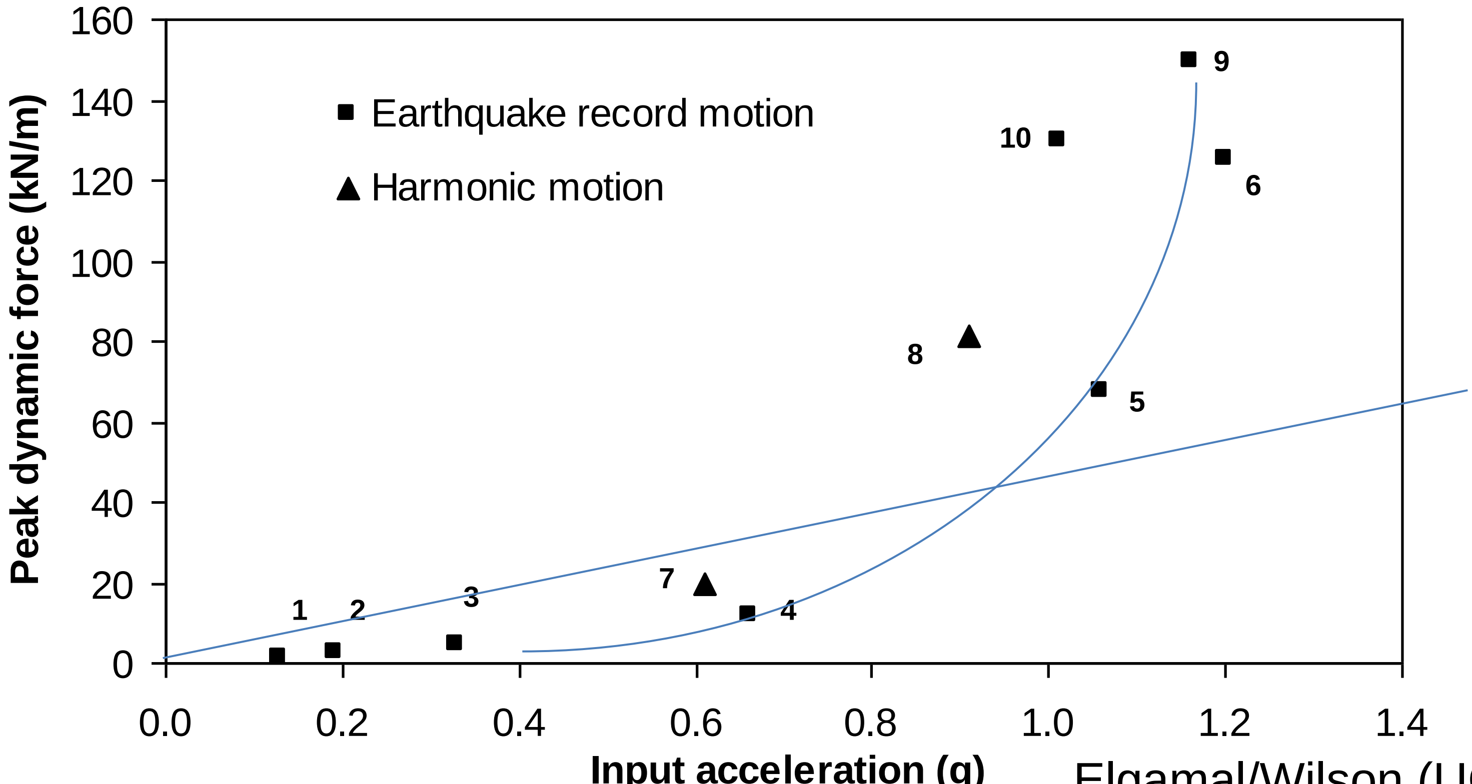
Elgamal/Wilson (UCSD)



•Modified Century City Station 1994 Northridge earthquake motion (EM) & Harmonic motions (HM)

UC San Diego Dynamic Excitation Tests

Test	Input	Peak Input Value A (positive) m/s/s	g	V m/s	D m	Total Load Cell Force (includes the wall inertia) Initial kN/m	Final	Max	Min	Peak Change (from Initial) Increase kN/m	Decrease
1	1/3 x EM	1.27	0.13	0.27	0.06	36.0	36.0	37.5	34.9	1.5	1.1
2	2/3 x EM	1.91	0.19	0.33	0.08	36.1	36.1	39.0	33.4	2.9	2.7
3	1 x EM	3.21	0.33	0.41	0.08	36.4	36.4	41.0	31.2	4.6	5.2
4	2 x EM	6.47	0.66	0.80	0.14	38.2	36.2	50.3	22.3	12.1	15.9
5	3 x EM	10.36	1.06	1.16	0.22	35.9	31.1	103.2	9.7	67.3	26.2
6	3.3 x EM	11.74	1.20	1.28	0.24	30.9	22.4	155.9	7.8	125.0	23.1
7	1 x HM	6.01	0.61	0.78	0.14	20.1	19.9	40.0	7.3	19.9	12.8
8	1.5 x HM	8.89	0.91	1.16	0.21	19.7	14.4	101.2	1.6	81.5	18.1
9	3.3 x EM (repeat)	11.41	1.16	1.28	0.24	12.4	8.7	162.0	0.6	149.6	11.8
10	3 x EM (repeat)	9.88	1.01	1.16	0.21	9.0	8.8	138.8	2.6	129.8	6.4

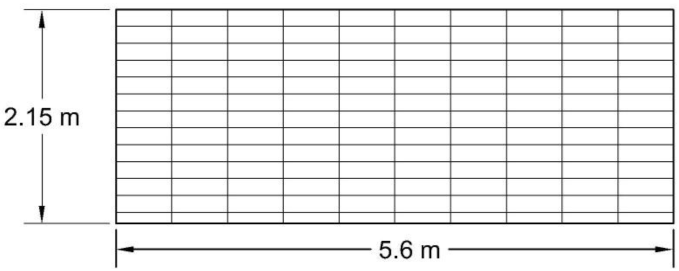


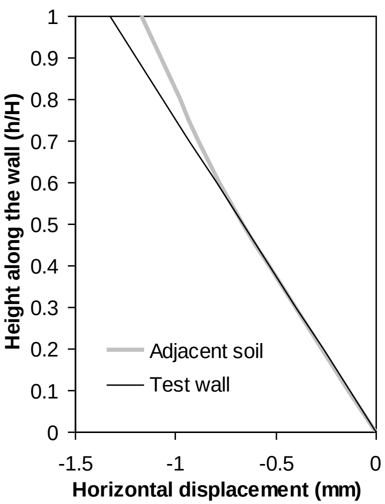
Elgamal/Wilson (UCSD)

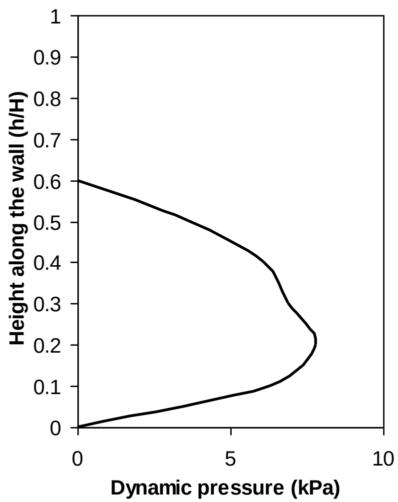
FE Simulations: Preliminary Results

•Test wall rotated slightly during experiments

•Modeled similar to previous numerical studies



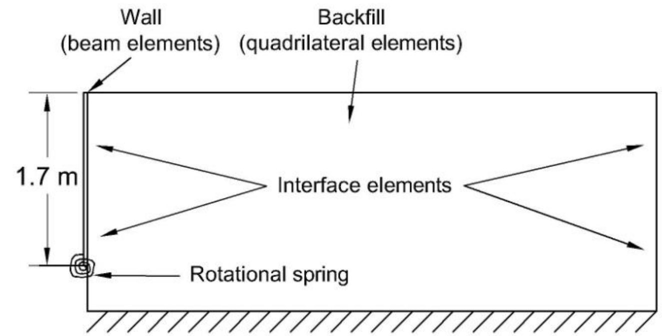


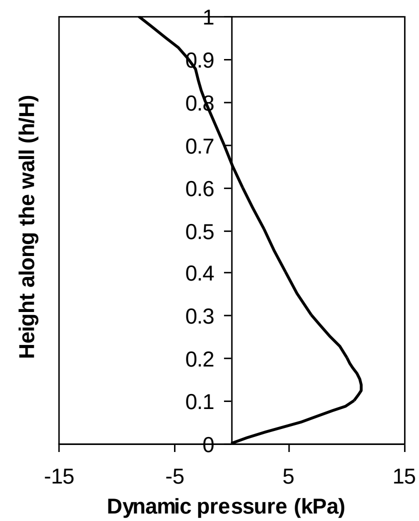


•Soil separates from wall at the top

•Dynamic pressure is zero near the top

Elgamal/Wilson (UCSD)





•Wall is pushed slightly first to achieve higher initial stress conditions

•Pressure decreases near the top and increases below as in the lower level experiments

Low Dynamic Earth Pressure up to about 0.60 g!

- Stiff backfill did not displace as much as the wall near the backfill surface . As wall rotated slightly away (up to about 2.5 mm of displacement at the top), pressure decreased near the top while increasing near the middle

---

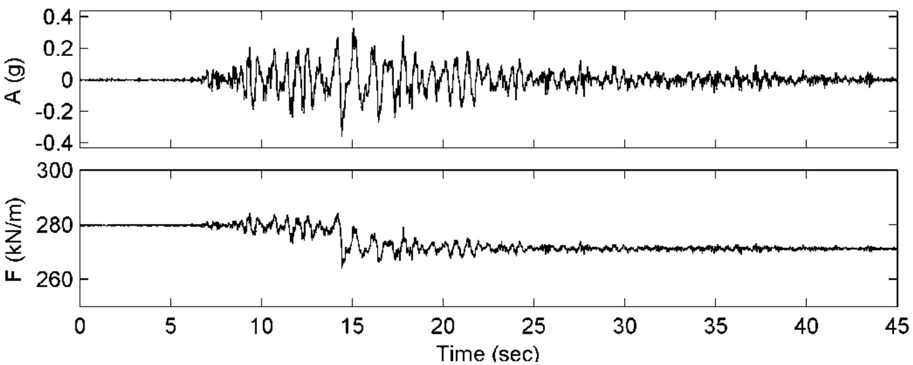
- High friction angle due to plane strain, low confinement, and strong compaction
- Cohesion in the soil added significantly to the shear strength, particularly for the tested 1.7 meter wall height

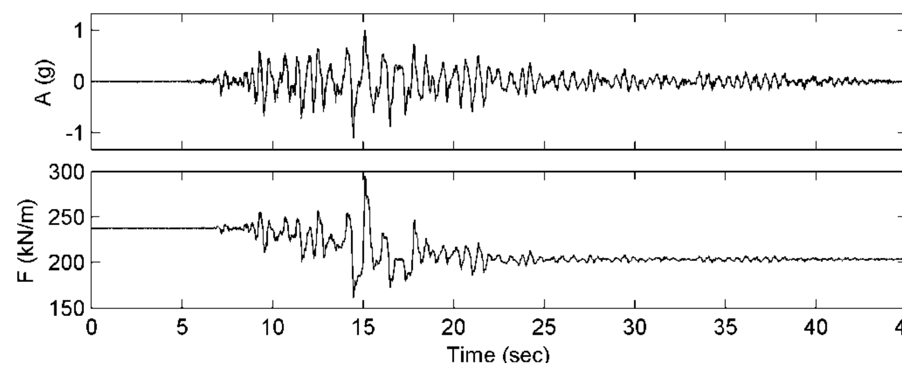
Some Reasons for possible lower dynamic earth pressure

- Using the triaxial “low” friction angle (in practice, many retaining walls are primarily in plane strain conditions)
- Ignoring (down-playing) the cohesion intercept of shear strength
- Not including the effect of soil aging after construction (e.g., gain is strength due to reduction in water content after construction at OMC
- Peak earth pressure pulses are short in duration
- Out of phase inertial forces may evolve for taller walls

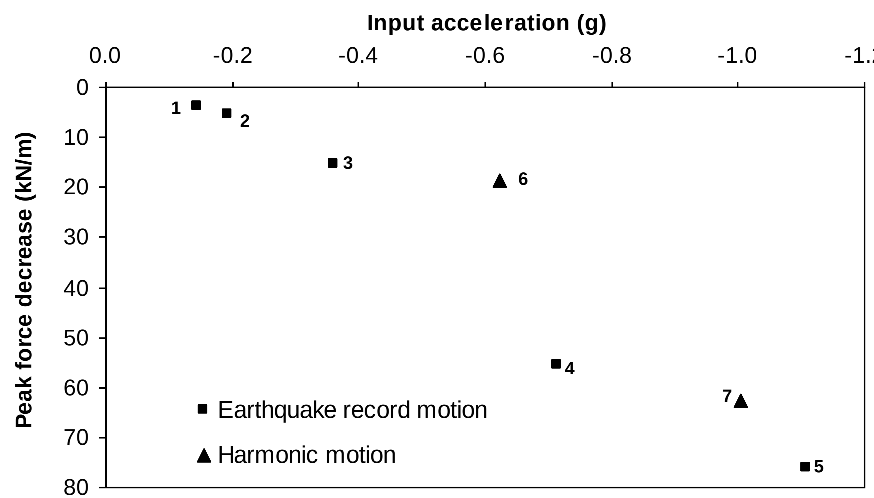
Dynamic Tests: With Mobilized Passive Pressure behind the Wall

Base acceleration (A) and total lateral force measured by load cells (F) per meter of wall width from a moderate and a very strong test





Test	Input	Peak Input Value A (positive) m/s/s	g	A (negative) m/s/s	g	V m/s	D m	Total Load Cell Force (includes the wall inertia) Initial kN/m	Final	Max	Min	Peak Change (from Initial) Increase kN/m	Decrease
1	1/3 x EM	1.13	0.12	-1.42	-0.14	0.12	0.03	283.3	281.6	285.2	279.5	1.9	3.8
2	2/3 x EM	2.02	0.21	-1.89	-0.19	0.25	0.04	282.0	280.0	285.5	276.6	3.5	5.4
3	1 x EM	3.24	0.33	-3.54	-0.36	0.41	0.07	279.6	271.3	284.3	264.2	4.7	15.4
4	2 x EM	6.51	0.66	-7.00	-0.71	0.79	0.14	271.0	238.1	279.5	215.3	8.5	55.7
5	3 x EM	9.85	1.00	-10.9	-1.11	1.16	0.21	237.7	203.3	295.3	161.4	57.6	76.3
6	1 x HM	5.84	0.6	-6.11	-0.62	0.78	0.14	203.6	202.8	225.0	185.0	21.4	18.6
7	1.5 x HM	9.20	0.94	-9.86	-1.01	1.15	0.21	203.1	181.2	261.4	140.3	58.3	62.8



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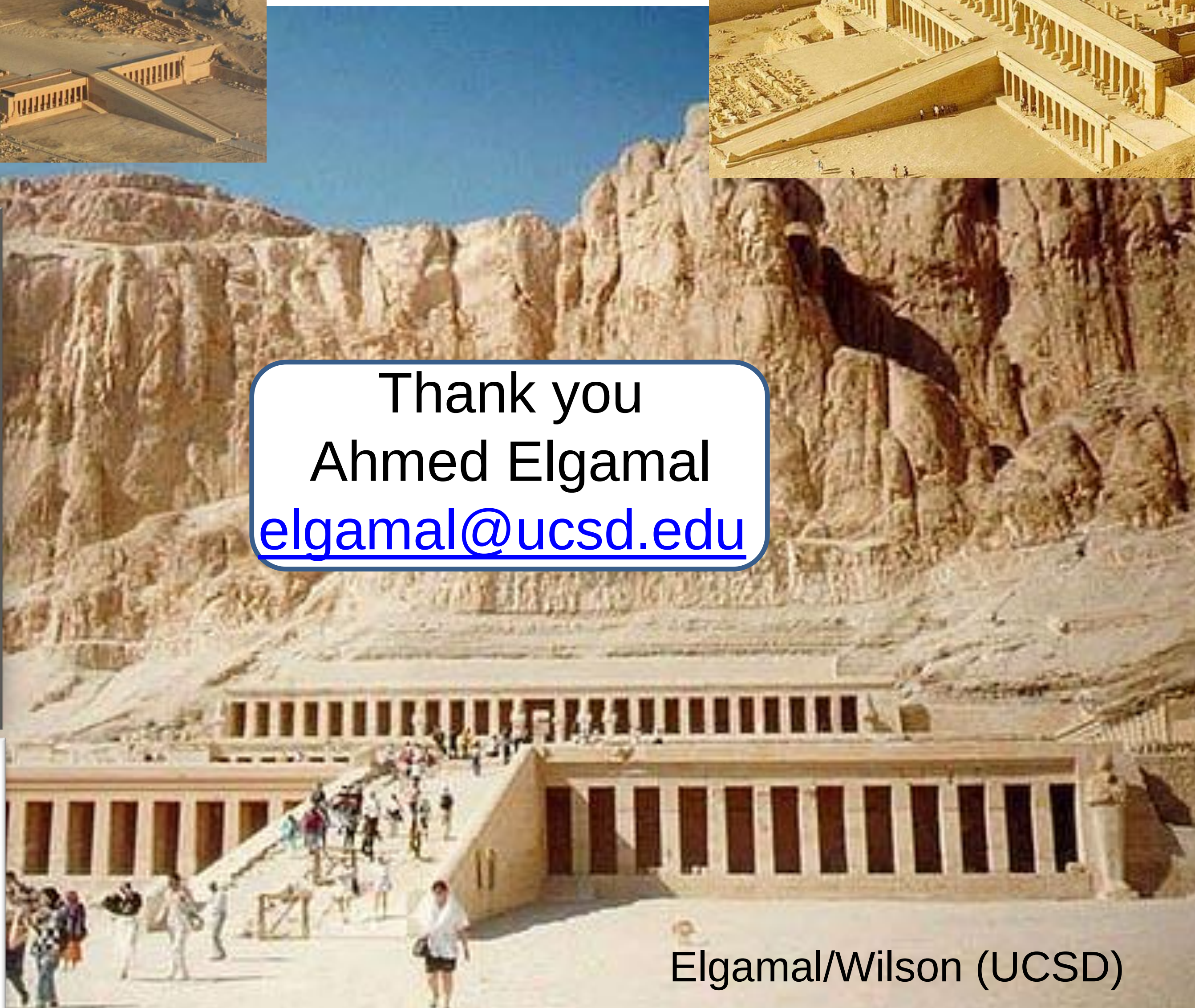

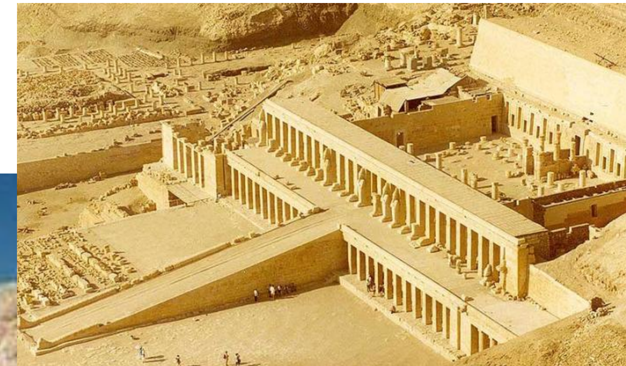

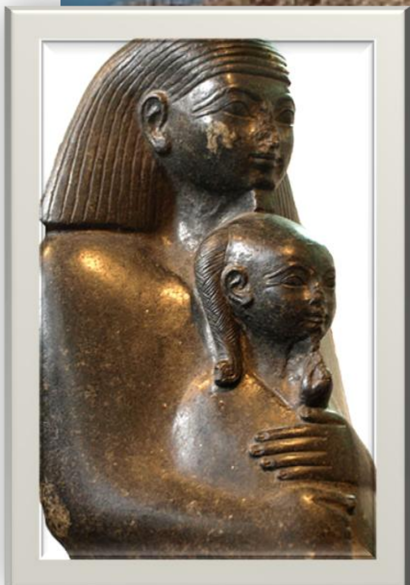

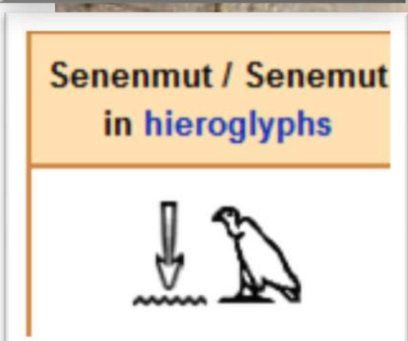

Thank you
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Elgamal/Wilson (UCSD)

Architect Engineer of **Hatshepsut Temple** (about 2000 years BC)