PACIFIC EARTHQUAKE ENGINEERING
RESEARCH CENTER

An International Workshop on Large-Scale Shake Table Testing for the Assessment of Soil-Foundation-Structure System Response for Seismic Safety of DOE Nuclear Facilities

A Virtual Workshop – 17-18 May 2021

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Pacific Earthquake Engineering Research Center
Headquarters at the University of California, Berkeley
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1 OBJECTIVES OF THE WORKSHOP

Aging infrastructure within the US Department of Energy (DOE) and the National Nuclear Security Administration (NNSA) nuclear facilities poses a major challenge to their resiliency against natural phenomenon hazards. Examples of mission-critical facilities located in regions of high seismicity can be found at a number of NNSA sites including Lawrence Livermore National Laboratory, Los Alamos National Laboratory, and the Nevada National Security Site. Most of the nation's currently operating nuclear facilities have already reached their operating lifetime, and most currently operating nuclear power plants (NPPs) have already reached the extent of their operating license period. While the domestic demand for electrical energy is expected to grow, if currently operating NPPs do not extend their operations and additional plants are not built quickly enough to replace them, the total fraction of electrical energy generated from carbon-free nuclear power will rapidly decline. The decision to extend operation is ultimately an economic one; however, economics can often be improved through technical advancements (McCarthy et al. 2015[1]) and research and development (R&D) activities.

Similarly, the operating lifetime of the current DOE- and NNSA-owned critical infrastructure can be extended using the Probabilistic Risk Assessment (PRA) framework to systematically identify the risk associated with designing and operating existing facilities and building new ones. Using this framework consists of several steps, including (1) system analysis considering the interaction between components, such as evaluating the soil-foundation-structure system response; and (2) assessment of areas of uncertainty. Both of these steps are essential to assessing and reducing risks to the DOE and NNSA nuclear facilities.

While the risks to the DOE's facilities are primarily due to natural hazard phenomena, data from large-scale tests of the soil-foundation-structural system response to seismic shaking is currently lacking. This workshop aimed to address these key areas by organizing an international workshop focused on advancing the seismic safety of nuclear facilities using large-scale shake table testing.

As a result, this workshop, which was held virtually, brought together a select group of international experts in large-scale shake table testing from the U.S., Japan, and Europe to discuss state-of-the-art experimental techniques and emerging instrumentation technologies that can produce unique experimental data to advance knowledge in natural hazards that impact the safety of the DOE's nuclear facilities. The generated experimental data followed by research and development activities will ultimately result in updates to ASCE 4-16[2], one of the primary design guides for DOE nuclear facilities per DOE-STD-1020-2016[3].

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[1] McCarthy, Kathryn, Hallbert, Bruce, Smith, Curtis, Barnard, Cathy, Leonard, Keith, & Corradini, Michael L. *Light Water Reactor Sustainability Program Integrated Program Plan*. United States. https://doi.org/10.2172/1408393

[2] ASCE/SEI 4-16. *Seismic Analysis of Safety-Related Nuclear Structures*. ASCE Standard, 2017.

[3] DOE-STD-1020-2016. *Natural Phenomena Hazards Analysis and Design Criteria for DOE Facilities*. Issued Dec. 04, 2016. https://www.standards.doe.gov/standards-documents/1000/1020-astd-2016

The ultimate objective of the workshop was to develop a “road map” for the future experimental campaign and innovative instrumentations using the newly constructed DOE-funded large-scale shake table facility at the University of Nevada, Reno (UNR) as well as other large-scale shake table testing facilities. This new facility resulted from a collaborative project engagement between UNR and Lawrence Berkeley National Laboratory. (LBNL) The workshop was planned for two days, May 17-18 2021, and included a virtual tour of the large-scale shake table facility at UNR. On both days, we saw cutting-edge research in experimental research, numerical modeling, and applied engineering through presentations and Q&A discussions.

The workshop was held virtually in Spring 2021 through the Zoom online webinar platform. We opted to hold it virtually consistent with the format of some of the other major conferences in 2021 rather than postponing it to a later date. This report summarizes the proceedings of the workshop and highlights the key outcomes from presentations and discussions.

2 WORKSHOP PROGRAM

2.1 PROGRAM DETAILS

In response to the call for abstracts, we received a total of 27 abstracts from which 26 that met the workshop objectives were selected for presentations during the two-day virtual workshop. The workshop details are presented in Table 2.1 and Table 2.2. The program was broken down into four sessions on both days with three 15-minute breaks in between. The workshop began with opening and welcoming remarks and a brief introduction to the Department of Energy Natural Hazards Mitigation program. Following the summary of the current seismic analysis capabilities that support the DOE mission, researchers from all over the world presented the state-of-art shake table experimental research and provided insights on how best to utilize the large-scale experimental facilities such as the recently completed large-scale laminar soil box and shake table facility at the University of Nevada, Reno to fill critical gaps in the scientific knowledge of soil-foundation-structure-interaction. Additionally, several talks followed the theme of enhancing nuclear safety, which is also undertaken by the DOE in the Nuclear Safety Research and Development (NSR&D) program.

Table 2.1 Program for Monday May 17, 2021

Time	Session	Details
7:30am - 9:00am	1 Moderator: Ramin Motamed (UNR)	**Opening Remarks** (10 min) *Presenter: Ramin Motamed (University of Nevada Reno)* **Welcoming Remarks** (10 min) *Presenter: Todd Lapointe (Department of Energy)* **Briefing on the DOE Natural Hazards Mitigation Program** (15 min) *Presenter: Sharon Jasim-Hanif (Department of Energy)* **Seismic Analysis Capabilities Supporting DOE Mission (30 min)** *Presenter: David McCallen (University of Nevada Reno and Lawrence Berkeley National Laboratory)* **Nonlinear Seismic Fragilities for Use in Seismic Performance Assessments** (25 min) *Presenter: Michael Salmon (Los Alamos National Laboratory)*
Break		

9:15am - 11:00am	2 Moderator: Ian Buckle (UNR)	**Design of a Large-Scale, Biaxial Soil Box and Shake Table for Seismic Soil-Structure Interaction Studies** (25 min) *Presenter: Sherif Elfass (University of Nevada Reno)* **Opportunities for Improving the Seismic Safety of Critical DOE Facilities through Large Scale Shake Table Testing Combined with High-Fidelity Numerical Simulations** (25 min) *Presenter: Payman Tehrani (SC Solutions, Inc)* **Advanced Numerical Modeling of a Large-Scale Soil-box for Experiments in Soil-Structure-Interaction** (25 min) *Presenter: Denis Istrati (University of Nevada Reno)* **Research to Improve Seismic Provisions for Foundation Stiffness and Damping as Applied in Engineering Practice** (25 min) *Presenter: Jonathan Stewart (University of California Los Angeles)*
Break		
11:15am - 12:30pm	3 Moderator: David McCallen (UNR & LBNL)	**Advances in Full Fidelity Soil-Structure-Fluid Interaction Simulation for Nuclear Structures** (25 min) *Presenter: Greg Mertz (Costantino and Associates)* **New Trends in Numerical Modeling of Seismic Soil-Structure-Interaction** (25 min) *Presenter: Youssef Hashash (University of Illinois at Urbana-Champaign)* **Validation Testing for Earthquake Soil Structure Interaction Modeling and Simulation** (25 min) *Presenter: Boris Jeremic (University of California, Davis)*
Break		
12:45pm - 2:25pm	4 Moderator: Khalid Mosalam (UCB & PEER)	**Modeling the Seismic Response of Spent Nuclear Fuel in Dry Storage** (25 min) *Presenter: Nicholas Klymyshyn (Pacific Northwest National Laboratory)* **Design Verification of Large Scale Laminar Soil Box** (25 min) *Presenter: Jenna Wong (San Francisco State University)* **Shake Table Tests for Validation of Numerical FSI Models of Advanced Reactors** (25 min) *Presenter: Faizan Ul Haq Mir (State University of New York at Buffalo)* **E-Defense Shaking Table Test of 3-Story R/C Frame Structure with Pile Foundation** (25 min) *Presenter: Koichi Kusunoki (University of Tokyo)*
2:25pm - 2:30pm	Closing	**Day 1 concluding remarks** (5 min) *Presenter: Ramin Motamed (University of Nevada, Reno)*

Table 2.2 Program for Tuesday May 18, 2021

Time	Session	Details
7:30am - 9:00am	1 Moderator: Ramin Motamed (UNR)	**Briefing on the DOE Nuclear Safety Research and Development Program** (15 min) *Presenter: Patrick Frias (Department of Energy)* **Modeling Vertical Free-Field Motion for SSI Analysis Consistent with Vertical Design Motion Development** (25 min) *Presenter: Farhang Ostadan, (Bechtel Corporation)* **The Different Phenomenology of Dynamic SSI for Buildings, Bridges and Power Plants: Numerical and In-Situ Full-Scale Tests** (25 min) *Presenter: Guido Andreotti (University School for Advanced Studies IUSS Pavia and EUCENTRE)* **Dynamic Distributed Fiber Optic Strain Sensing for Lab Testing** (25 min) *Presenter: Matthew DeJong (University of California Berkeley)*
Break		
9:15am - 11:00am	2 Moderator: Steve McDuffie (DOE)	**Assessing Kinematic Soil-Structure Interaction in Nuclear Power Plant Structures based on Field and Experimental Data** (25 min) *Presenter: Ramin Motamed (University of Nevada Reno)* **Modeling Input Needs from Large-Scale Shake Table Testing – Case Study from Safety Analysis of a DOE Facility** (25 min) *Presenter: Mohamed Talaat (Simpson Gumpertz & Heger Inc)* **Physical Modeling of Soil-Structure Systems in Unsaturated Soils: Challenges and Opportunities** (25 min) *Presenter: Majid Ghayoomi (University of New Hampshire)*
Break		
11:15am - 12:30pm	3 Moderator: Amarnath Kasalanati (PEER)	**Results from Shaking Table Tests on Full-Scale Rail Embankments** (25 min) *Presenter: John McCartney (University of California San Diego)* **LBNL NERSC Base Isolated Floor – Using Science to Protect Science** (25 min) *Presenter: Tim Hart (Lawrence Berkeley National Laboratory)* **Model-in-the-loop Testing of a Base-Isolated Cylindrical Tank** (25 min) *Presenter: Sai Sharath Parsi (State University of New York at Buffalo)*
Break		
12:45pm - 2:25pm	4 Moderator: Mike Salmon (LANL)	**Advanced Instrumentation for Large-Scale Laminar Soil Box Shake Table Testing** (25 min) *Presenter: David McCallen (University of Nevada Reno and Lawrence Berkeley National Laboratory)*

		Distributed Fiber-Optic Sensing for Subsurface Vibration and Deformation Monitoring (25 min) *Presenter: Elnaz Seylabi (University of Nevada Reno)* **3D Effective Stress Analysis on Soil Behavior Around Closely Spaced Pile Group in Centrifugal Shaking Table Tests** (25 min) *Presenter:* Y*uta Nakagama (Tokyo Electric Power Services Co., Ltd.)* **Virtual Tour of Large-Scale Shaking Table Facility at UNR to Study Soil-Structure Interaction Phenomenon(link is external)** (25 min) *Presenter: Ramin Motamed (University of Nevada Reno)*
2:25pm - 2:30pm	Closing	**Day 2 concluding remarks** (5 min) *Presenter:* David McCallen

3 ABSTRACTS

A total of 27 abstracts were received in response to the call for abstracts, most of which were presented at the workshop. This chapter provides details on all the submissions. Full slides for the presentations that were made at the workshop can be found in Appendix A. However, due to the confidential nature of some of the presentations, they are not included in the appendix but the key issues discussed in them relevant to the conclusions of this workshop are included in Chapter 4.

The call for abstracts invited for presentations in the following areas within the common theme of "large-scale shake table testing":

- R&D activities related to the resiliency of nuclear facilities against natural phenomenon hazards
- Probabilistic Risk Assessment (PRA) advancements
- Soil-foundation-structure system analysis considering the interaction
- Advanced simulations and validations
- Assessment of areas of uncertainty and quantification
- State-of-the-art experimental techniques
- Emerging instrumentation technologies
- Large-scale shake table facilities, design basis, and performance objectives, experimental capabilities, example recent projects
- Technical standard developments
- Other areas related to large-scale shake table testing

The submitted abstracts were grouped based on the proposed themes and Figure 2.1 illustrates the distribution. As can be seen, topics related to "R&D Research" and Large-Scale Testing" covered more than half of the presentations. The abstracts are presented hereafter.

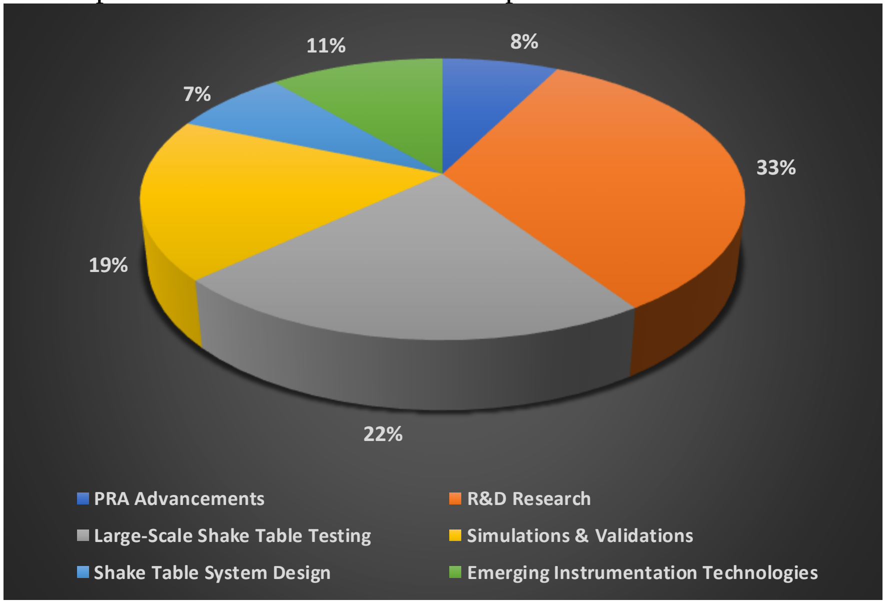

Figure 2.1 Distribution of the submitted abstracts based on the proposed workshop themes.

3.1 DESIGN OF A LARGE-SCALE, BIAXIAL SOIL BOX AND SHAKE TABLE FOR SEISMIC SOIL-STRUCTURE INTERACTION STUDIES

Presenter: Sherif Elfass

Institution/Organization: University of Nevada, Reno

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Abstract: Large-scale experiments in geotechnical earthquake engineering have considerably advanced in recent years. Enhancing experimental capabilities has addressed significant interest in such phenomena as site amplification, dynamic soil behavior, and soil-structure interaction (SSI). One-g shake table tests using a laminar soil box present a viable methodology to investigate these phenomena. This presentation describes the design of a novel, biaxial, 400ton, laminar soil box at the University of Nevada, Reno, for soil–structure interaction studies at a scale not currently possible in the U.S. The box is an octagon in plan. The inner dimension between any two parallel sides is 21.5 ft and the fully assembled height is 15 ft. The box is, however, designed to allow experiments in segments that are 5, 10, and 15 ft high. It consists of nineteen steel frames (laminates) made of hollow structural sections. Each frame is connected to the frame below through a number of elastomeric bearings. The use of elastomeric instead of mechanical bearings adopted elsewhere is intended to improve the robustness of boxes of this size. The number and physical properties of these bearings change at specified levels to accommodate the change in soil stiffness with depth. However, due to the complementary shear developed in the soil during shaking, these bearings are expected to experience significant tension, especially near the bottom of the box walls. Since elastomeric bearings are rarely used to carry high tensile loads, an experimental program was undertaken to confirm their tensile capacity. In addition, base shear is transferred from the table to the soil through a grid of shear connectors at the table/soil interface.

This large-scale laminar soil box is being developed under a collaborative project involving Lawrence Berkeley National Laboratory, UC Davis, and the University of Nevada, Reno. The sponsor is the U.S. Department of Energy.

3.2 ADVANCED NUMERICAL MODELING OF A LARGE-SCALE SOIL-BOX FOR EXPERIMENTS IN SOIL-STRUCTURE-INTERACTION

Presenter: Denis Istrati

Institution/Organization: University of Nevada, Reno

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Abstract: Nuclear power plants are designed to withstand environmental hazards, including earthquakes, to ensure public safety and continuity of function. However, it has been observed that large embedded structures, such as reactor buildings, can affect the response of the soil around them and the nature of the ground shaking they have to withstand, leading to significant soil-structure-interaction (SSI) effects. This presentation describes a series of numerical analyses that were conducted during the design of a large-scale laminar soil box (LLSB) at the University of Nevada, Reno, which is sponsored by the U.S. Department of Energy with the aim to quantify the seismic SSI of nuclear facilities.

The design phase included finite element modeling and nonlinear dynamic analyses of one-, two- and three-dimensional models. These nonlinear analyses were compared with linear and equivalent linear ones for a suite of recorded ground motions with scaled PGAs between 0.25g and 1.0g, revealing the limitations of each modeling approach. Parameters of interest in these analyses included the base shear, overturning moment, pressures below the box, response spectra at the surface, forces and displacements in the walls, as well as, soil accelerations, stresses, and strains.

Different material properties were considered for the walls of the box, and the effect of their lateral, vertical, and rotational stiffness was investigated. The advanced numerical modeling: (i) helped understand the fundamental interaction of the soil with the walls of the box, (ii) provided data for the design of this novel system, and (iii) revealed that an ideal soil box should have walls with low lateral stiffness but high axial and bending stiffness in order to minimize any flexural effects and associated soil distortion close to the walls. The results presented herein provide an insight into the expected performance of the soil box and will help guide future validation efforts.

3.3 RESEARCH TO IMPROVE SEISMIC PROVISIONS FOR FOUNDATION STIFFNESS AND DAMPING AS APPLIED IN ENGINEERING PRACTICE

Presenter: Jonathan Stewart

Institution/Organization: University of California, Los Angeles

Email: jstewart@seas.ucla.edu

Abstract: Current seismic provisions for analysis of foundation stiffness and damping for use in seismic response analyses are well established and validated for stiff foundation elements under linear and nonlinear conditions. However, with the exception of mat foundations, the most commonly used foundation systems consist of inter-connected discrete elements that comprise a foundation system. The stiffness and energy dissipation properties of such systems are relatively poorly understood, particularly for rotational responses. For tall structures, the rotational response is dominant, so this uncertainty substantially impacts our ability to predict inertial soil-structure interaction effects. A combined program of laboratory testing and simulations, considering different foundation configurations and excitation levels, would produce significant insights that could have a substantial practical impact. We encourage the design of a research program, organized in consultation with engineers familiar with practical design procedures, that would address these needs.

3.4 ADVANCES IN FULL FIDELITY SOIL-STRUCTURE-FLUID INTERACTION SIMULATION FOR NUCLEAR STRUCTURES

Presenter: Greg Mertz

Institution/Organization: Costantino and Associates

Email: greg@gemertz.com

Abstract: Proper seismic modeling of next-generation nuclear structures in accordance with ASCE 4 now requires the triad coupling relationship of complex structural systems, encased fluids, and surrounding soil foundations. This presentation demonstrates a new method to completely capture the benefits and validation of the SASSI soil-structure interaction approach within a modern commercial finite element package such as ANSYS, which can couple fluids and structures. This is accomplished by using SASSI to develop pre-calculated soil impedance substructures and associated seismic load vectors and to store them in the Soil Library.

This method overcomes the limited structural element types in SASSI and allows fluid interactions in the structure and fluid to be fully coupled to the soil response in an equivalent linear manner in frequency and time domains. Greater efficiency can be obtained by working within a single once the soil library is developed without the inefficiencies of data interchange between two software platforms and analysis teams.

An example is presented that demonstrates the method’s functional equivalence to SASSI for comparative models and capabilities beyond SASSI for soil-structure-fluid interaction models. Experimental studies are suggested for the UNR soil column test apparatus to learn more about the triad coupling approach.

3.5 NEW TRENDS IN NUMERICAL MODELING OF SEISMIC SOIL-STRUCTURE-INTERACTION

Presenter: Youssef Hashash

Institution/Organization: University of Illinois at Urbana-Champaign

Email: hashash@illinois.edu

Abstract: Rapid urbanization and infrastructure development in areas with seismic hazard throughout the world require a detailed understanding of their anticipated performance during anticipated shaking events. These evaluations are essential to the assessment of the resiliency and post-earthquake performance of individual infrastructure elements such as buildings, tunnels, and bridges as well as overall post-earthquake system performance. While designing for life safety remains a primary objective of seismic design, performance-based design, and functional recovery are increasingly important considerations. Conventional and simplified approaches whereby separate evaluations of the seismic response of a structure and the supporting soil are performed are often insufficient for performance-based and functional recovery design. Advances in analysis software, computing hardware, and big data management tools have made it possible to analyze complex nonlinear seismic soil-structure-fluid interactions in three dimensions (3D) in research and more importantly engineering practice. Large-scale laboratory experiments can provide us with insights into such complex interactions. They complement centrifuge experiments and are invaluable for the calibration of advanced numerical simulations. This presentation will describe recent developments in 3D seismic soil-structure interaction (SSSI) modeling of a range of infrastructure systems. Available state-of-the-art and practice numerical analysis tools will be first described with a focus on computational efficiency, high-fidelity modeling of both the soil and the structure systems, and scalability of analysis from desktop computers to high-performance computing systems and management of large data volumes. Developments made by our research group will be presented covering several problems including tall buildings, tunnels, etc. The analyses provide insights into the system response that would not be possible using simpler decoupled analysis approaches. The presented examples demonstrate that the age of advanced 3D SSSI is upon us providing the tools to tackle what otherwise would have been considered intractable problems and will benefit greatly from large-scale experiments.

3.6 VALIDATION TESTING FOR EARTHQUAKE SOIL STRUCTURE INTERACTION MODELING AND SIMULATION

Presenter: Boris Jeremic

Institution/Organization: University of California, Davis

Email: jeremic@ucdavis.edu

Abstract: Presented will be numerical modeling validation concept and field and laboratory testing suggestions for problems involving earthquakes, soil, structures, and their interaction (ESSI). The focus will be on modeling requirements for ESSI problems and on the determination of the accuracy of available models to represent the real world. A number of suggested field and laboratory testing setups will be illustrated in view of their use for ESSI modeling and simulation. Modeling and simulation of wave propagation in solids and structures, inelastic behavior of solids, structures and interfaces, and interaction of solids/structures and fluids, both internal and external, main modeling problems for successful ESSI analysis, will be used to motivate a suggested set of tests. The need for quantification of variability and uncertainty in test data will be analyzed and illustrated as well.

3.7 MODELING THE SEISMIC RESPONSE OF SPENT NUCLEAR FUEL IN DRY STORAGE

Presenter: Nicholas Klymyshyn

Institution/Organization: Pacific Northwest National Laboratory

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Abstract: The US Department of Energy's Spent Fuel and Waste Science and Technology (SFWST) program is investigating the mechanical loading of spent nuclear fuel (SNF) to close knowledge gaps and inform the range of relevant loads for mechanical testing of irradiated SNF. The SFWST program has performed full-scale dynamic testing of multimodal transportation and cask drop scenarios related to normal conditions of transportation and is now preparing for a full-scale SNF dry storage system shake table test to measure the dynamic response of SNF to hypothetical ground motion. The SFWST test team includes several US organizations and potential international collaborators from Spain, South Korea, and Germany. The test plan is still being developed and discussions with international collaborators are ongoing, but this presentation will include an overview of the test program and the project team.

Researchers at Pacific Northwest National Laboratory (PNNL) are developing explicit dynamic finite element models of SNF and SNF cask systems to support the shake test program. Vertical and horizontal SNF canister storage systems are being modeled with a range of features to explore the structural dynamics of the canister systems and to ultimately make pre-test predictions of their response to shake table excitation. This presentation will describe PNNL's current progress in finite element modeling the SNF cask systems to inform the test plan. It will also cover the remaining modeling steps needed to support the test campaign and briefly describe the next modeling steps after the test data is collected.

3.8 DESIGN VERIFICATION OF LARGE SCALE LAMINAR SOIL BOX

Presenter: Jenna Wong

Institution/Organization: San Francisco State University

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Abstract: As part of a collaborative United States Department of Energy (US DOE) project, a large-scale laminar soil box is currently under construction at the University of Nevada, Reno (UNR). This box will expand the experimental capability for soil-structure-interaction (SSI) research providing new data, especially for SSI numerical model validations. This project brings together the engineering expertise of faculty from Lawrence Berkeley National Laboratory, the University of California, Davis, UNR, and San Francisco State University (SFSU). At SFSU, extensive work has focused on design verification of the soil box which once completed will be a 4.6m tall octagonal-shaped system with an inner radius of 6.6m. Comprised of steel and elastomeric bearings, the structural system of the box is quite complex and unique harnessing the natural flexibility of bearings to provide containment without restraining natural soil movement. As this is a first-of-its-kind experimental system, the design verification is quite complex. Throughout this project, computational simulations have been conducted to explore not only the box's structural response but more importantly the nonlinear soil behavior with the goal of looking ahead to future experimental studies. To achieve this, various models have been used including single brick elements, soil columns, cores, and full-scale finite element models on OpenSees. This also has expanded work into examining soil constitutive models for calibration and sensitivity. Through a systematic approach, design verification for this study progressively worked from static to dynamic analyses to fully characterize the dynamic properties of the box and explore potential responses of experimental test specimens. This presentation covers the verification studies used to develop large-scale computational models and build the foundation for future SSI validation efforts.

3.9 SHAKE TABLE TESTS FOR VALIDATION OF NUMERICAL FSI MODELS OF ADVANCED REACTORS

Presenter: Faizan Ul Haq Mir

Institution/Organization: State University of New York at Buffalo

Email: faizanul@buffalo.edu

Abstract: Some safety-critical components in advanced nuclear reactors contain fluids (e.g. coolant) or are submerged in fluids. Dynamic analysis of such components for seismic design should consider the interaction effects of the contained or surrounding fluid, referred to as fluid-structure interaction (FSI) effects. Analytical solutions for evaluating such effects are available, but their application is restricted to simple structural and fluid domain geometries, simple boundary conditions, small amplitude seismic inputs, and linear fluid responses. Full-scale testing of advanced reactor vessels and their internal components for informing their seismic design is not feasible given their size and cost. The only plausible alternative for performing seismic design and risk assessment calculations with consideration of FSI effects is the use of verified and validated numerical models.

Physical test data that could be used for validating numerical FSI models over a wide range of earthquake shaking do not exist. An experimental program involving a scaled model of a reactor vessel and simplified representations of submerged internals was executed on a six-degrees-of-freedom earthquake simulator at the University at Buffalo to generate such data. Hydrodynamic responses (pressure, wave height, base reactions) and submerged component responses (acceleration, strain, reduction in frequency due to submergence) to a wide range of multi-component shaking were generated. The data from the experiments are being used to validate a previously verified FSI solver: Arbitrary Lagrangian and Eulerian (ALE) in LS-DYNA. The presentation will involve details of the experimental program and ongoing validation studies.

3.10 E-DEFENSE SHAKING TABLE TEST OF 3-STORY R/C FRAME STRUCTURE WITH PILE FOUNDATION

Presenter: Koichi Kusunoki

Institution/Organization: Earthquake Research Institute, the University of Tokyo

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Abstract: Since the E-Defense was built in 2005, several large-scale shaking table tests were conducted. When the superstructures were shaken, the bottom of the specimen was fixed to the table. On the contrary, when the test's objective is Soil-Structure-Interaction (referred to as SSI, hereafter), soil layers and pile foundations were constructed in a large-scale laminar shear box. The superstructure was simplified down to the elastic single-degree-of-freedom system. However, the SSI is affected by the non-linear behavior of not only the soil layer and foundation but also the superstructures.

Kumamoto Earthquake occurred in 2016 in the far west of Japan. In a complex, three low-rise R/C apartment buildings supported by pile foundations, designed and constructed in the same year, tilted a lot during the earthquake. The superstructures, however, suffered minor damage. The pile foundations were investigated by digging out the surrounding soil, and it was observed that they failed in shear.

We planned to conduct a shaking table test with a three-story R/C specimen supported by a pile foundation. The test was the first shaking table test at E-Defense with a realistic superstructure and pile foundation. One of the main results was that the piles were severely damaged without any crack in the soil surface or tilting of the superstructure. There were two more apartment buildings constructed nearby the tilted three buildings in almost the same year. No damage could be observed from the outside. According to the shaking table test result, the piles of the buildings were investigated, too, and it was found that some of them were severely damaged as the shaking table test. It is required to develop a damage detection system for piles, such as a structural health monitoring system.

3.11 MODELING VERTICAL FREE-FIELD MOTION FOR SSI ANALYSIS CONSISTENT WITH VERTICAL DESIGN MOTION DEVELOPMENT

Presenter: Farhang Ostadan

Institution/Organization: Bechtel Corporation

Email: fostadan@bechtel.com

Abstract: The current approach to developing horizontal design motion for structural design is based on ground motion models that provide horizontal motion and incorporates site amplification to develop horizontal design motion. For vertical design motion, the soil column analysis is no longer performed; instead, the applicable frequency-dependent empirical spectral vertical-to-horizontal (V/H) ratios are applied to obtain the vertical design motion. The steps used by the industry today are described in ASCE 43-05, ASCE 4-16, and NUREG/CR 6728. Once the free-field horizontal and vertical design motions are determined, the motions are specified in the soil-structure interaction (SSI) model at the foundation level of the structure in the free field and SSI analysis is performed to develop seismic structural responses for the design of the structure and equipment.

In the computer program SASSI SSI formulation for vertical SSI analysis, free-field soil column analysis is performed first to obtain the free-field ground motion at the site layers within the embedment depth of the structure using P-wave formulation. Consequently, the vertical motion in the free field amplifies from the foundation level to the ground surface with the resulting V/H ratio exceeding the pre-determined value determined for the site. Thus, the structure is inherently subjected to higher vertical motion resulting in higher vertical seismic demand for the structure and equipment design.

Increasing the vertical SSI input motion not only increases the vertical structural responses but also challenges the seismic stability of the structure in the vertical direction when both buoyancy forces and seismic loads are considered.

In this presentation, a new formulation for vertical motion for SSI analysis is described. The applicable site V/H ratio used for the development of the vertical design motion is also used as a basis for the development of the free-field motion for SSI analysis.

3.12 THE DIFFERENT PHENOMENOLOGY OF DYNAMIC SSI FOR BUILDINGS, BRIDGES AND POWER PLANTS: NUMERICAL AND IN-SITU FULL-SCALE TESTS

Presenter: Guido Andreotti

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Abstract: The study of Soil-Structure-Interaction is traditionally decomposed into “kinematic” and “inertial” effects, which are respectively due to: (i) the stiffness of structural elements in contact with the ground that modifies the free-field input motion and (ii) the transmission of the structural inertial forces back to the soil. For buildings, the dynamic SSI problem is typically solved using the so-called substructure method, which is based on decomposing the superstructure-foundation-soil system into two subsystems whose response is determined independently. The response of the overall system is obtained from the application of the superposition theorem, in which the basic assumption is represented by the linearity of the superstructure-foundation-soil system. However, when both structural and soil nonlinearities are significant, this method is not reliable. We demonstrated that the dynamic SSI problems of bridges with Cast-In- Drilled-Hole (CIDH) pile columns and the underground structures of power plants (e.g. hydraulic tunnels) are governed by the relative variation of stiffness and strength of soil and structural elements, as a consequence of the different evolution of nonlinearity. Moreover, providing evidence that the mechanical properties of soil influence the capacity of the structure (i.e. length and position of plastic hinges) and vice-versa, we showed that “kinematic” and “inertial” effects cannot be treated separately in this kind of structure. Aiming to study the SSI problem experimentally at EUCENTRE has been developed the “MobiLab”, which is a mobile laboratory to perform in-situ dynamic tests of full-scale structures and geotechnical systems. The advantage of this approach is to perform SSI experimental tests directly in situ on full-scale systems, considering the real soil conditions and imposing real ground motions.

3.13 ASSESSING KINEMATIC SOIL-STRUCTURE INTERACTION IN NUCLEAR POWER PLANT STRUCTURES BASED ON FIELD AND EXPERIMENTAL DATA

Presenter: Ramin Motamed

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Abstract: Recorded earthquake motions at free-field ground accelerometers and at the foundation level of adjacent instrumented structures can provide insight into the extent of the kinematic Soil-Structure Interaction (SSI) phenomenon. Kinematic SSI effects are divided into two main categories namely (1) base slab averaging, and (2) embedment effects. Although considering kinematic interaction effects reduces foundation-level motions relative to the free-field motions, the current design guidelines are specific to regular buildings and irrelevant to nuclear structures with much larger foundation dimensions and depths. The objective of this research is to develop a simplified procedure for evaluating the extent of kinematic SSI in nuclear facilities. In this study, we utilized data from five well-instrumented nuclear facilities in Japan where a free-field downhole array was located adjacent to each instrumented structure with sensors at the foundation level enabling the empirical assessment of kinematic SSI. This presentation features two example instrumented sites namely the Kashiwazaki-Kariwa Nuclear Power Plant and the Hamaoka Nuclear Power Plant in Japan.

The extent of kinematic SSI was quantified at these sites using the ratio of response spectra (RRS) between foundation motions and the corresponding free-field motions and compared with the recommended RRS based on ASCE/SEI 4117. This presentation illustrates that ASCE/SEI 41-17 recommended RRS can overpredict the foundation motion in nuclear structures by 50%. Data from the five selected nuclear facilities in Japan will be supplemented with results from a series of moderate-scale shake table tests at the University of Nevada Reno to fill the data gaps. Overall, this presentation highlights the need to develop a technical basis for simplified formulations for assessing kinematic SSI in nuclear structures for inclusion in future design guides such as ASCE 4 "Seismic Analysis of Safety-Related Nuclear Structures" which is one of the primary design guides for DOE nuclear facilities per DOE-STD-1020-2016.

3.14 PHYSICAL MODELING OF SOIL-STRUCTURE SYSTEMS IN UNSATURATED SOILS: CHALLENGES AND OPPORTUNITIES

Presenter: Majid Ghayoomi

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Abstract: Understanding the fundamental mechanisms and performance of soil-foundation-structure systems requires assessing the collective response of the superstructure, foundation, and surrounding soil under monotonic or seismic loads. Accounting for soil-foundation-structure interaction (SFSI) effects, regardless of the approach, has one major component; i.e., the surrounding soil. Under earthquakes, for example, soil characteristics impact the motion transferred to the foundation through the soil-foundation interface, change the flexibility and natural frequency of the system related to soil-foundation stiffness and damping, and alter seismically induced settlements. Shallow soils that support the surface structures are often unsaturated, with complex dynamic behavior that differs from dry and saturated soil deposits because of inter-particle suction stresses. Despite the proven significant effects of the degree of saturation on dynamic soil properties, the current practice relies on procedures that do not directly include the effects of the degree of saturation. Physical modeling plays a key role in expounding these effects and calibrating and validating coupled numerical models. However, in the past, difficulties in modeling unsaturated soils and controlling and measuring the degree of saturation have slowed down these efforts. The use of geotechnical centrifuge, as much as it has been attractive to the geotechnical engineering community due to its capacity to capture soil's stress-dependent response, often poses procedural and analytical challenges when dealing with unsaturated soils. Large-scale physical models, however, can address these challenges and better simulate both the state of suction stress and hydromechanical conditions in unsaturated soils. This presentation discusses the existing challenges and potential opportunities in modeling soil-structure systems in unsaturated ground, how to control and measure the state of saturation, and the impacts of the degree of saturation on the overall performance of different systems such as kinematic and inertial soil-structure interaction, rocking foundations, pile lateral response, and seismic compression.

3.15 RESULTS FROM SHAKING TABLE TESTS ON FULL-SCALE RAIL EMBANKMENTS

Presenter: John McCartney

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Abstract: A series of earthquake motions of increasing amplitude was applied to a full-scale rail embankment using the Powell Laboratory shaking table at UCSD. The goal of this study was to evaluate the differential settlement of the rails during an earthquake and to identify the primary causes of settlement (lateral slope movement or seismic compression of the soil layers underlying the rails). This presentation will present the process used in designing the shaking table tests, some of the key results from the experiments, and a comparison with preliminary simulations performed using FLAC2D. Although greater settlements were observed for the rail closest to the slope face, the magnitude of the measured tilt angle between the rails was within acceptable limits.

3.16 ADVANCED INSTRUMENTATION FOR LARGE-SCALE LAMINAR SOIL BOX SHAKE TABLE TESTING

Presenter: David McCallen

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Abstract: Recent developments in advanced sensors and agile communications are providing transformational new methods for monitoring the earthquake response of engineered systems in real-time. Over the past four years, a laser-based optical sensor system for directly measuring earthquake-induced building drift has been under progressive development with support from the DOE Office of Nuclear Safety's Nuclear Safety Research and Development Program. The underlying technology base has undergone three generations of improvement and has matured to the point of readiness for initial field deployments. This technology has been extensively tested in the laboratory under simulated earthquake conditions, and careful value engineering has resulted in a system that can measure building dynamic inter-story drifts to within ~2mm, as well as permanent drifts due to nonlinear system response. The individual optical sensors – termed Discrete Diode Position Sensors (DDPS) – have been fitted with on-board wireless communication nodes so that the sensors can form a self-configuring network for agile sensor-to-sensor data transmission. This technology has been deployed on a building situated just adjacent to the Hayward Fault in the San Francisco Bay Area at the DOE's Lawrence Berkeley National Laboratory and has been operating reliably under real-world field conditions for the past 19 months.

This presentation will provide an overview of the unique features of the optical sensor technology, including the ability to measure very broad-band dynamic response and summarize extensive experimental evaluations of sensor performance and accuracy. The most recent work in developing an advanced generation four, biaxial version of the sensor will be summarized. The presentation will also describe how a DDPS network could be deployed on the UNR large-scale laminar soil box experimental system to facilitate agile, wireless shake table system data acquisition on this very large experimental system.

3.17 DISTRIBUTED FIBER-OPTIC SENSING FOR SUBSURFACE VIBRATION AND DEFORMATION MONITORING

Presenter: Elnaz Seylabi

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Abstract: Distributed fiber-optic sensing technologies convert fiber-optic cables into massive sensing arrays by sending a laser pulse down the cable and measuring changes in the profile of light backscattered from every point along a continuous fiber caused by perturbations in the environment surrounding the cable. In this way, fiber-optic techniques can be used to spatially resolve measurands such as strain rate, strain, and temperature at resolutions of a few meters along several kilometers, providing a sensing coverage equivalent to tens of thousands of point sensors. Classically, most studies have been focused on measuring absolute temperature (Distributed Temperature Sensing, DTS) and (pseudo-) static strain (Distributed Strain Sensing, DSS). In recent years, research on the development of Distributed Acoustic Sensing (DAS) for measuring dynamic strain caused by vibrations affecting the cable has accelerated. DAS has proven successful in various applications such as exploration geophysics, earthquake monitoring, and geotechnical characterization. Therefore, it is developing growing demand in the oil and gas industry, geothermal energy, seismology, and structural health monitoring applications.

This presentation discusses the potential of using these novel distributed fiber-optic sensing technologies for health monitoring and performance assessment of subsurface infrastructures such as deep foundations, which are often inaccessible for inspections after extreme events and under long-term service loads. In particular, we will focus on exploring the use of DAS for accurately measuring the dynamic (and permanent) strain inflicted on a buried structure and surrounding soil during and after significant seismic events at unprecedented spatial and temporal resolution. The presentation will also demonstrate how the large-scale soil box experimental facility at the University of Nevada Reno provides a unique opportunity to assess the performance, capabilities, and limitations of this sensing technology in broadband measurement of subsurface deformations and vibrations compared to the conventionally used point sensors such as strain gauges and accelerometers.

3.18 3D EFFECTIVE STRESS ANALYSIS ON SOIL BEHAVIOR AROUND CLOSELY SPACED PILE GROUP IN CENTRIFUGAL SHAKING TABLE TESTS

Presenter: Yuta Nakagama

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Abstract: Behaviors of soil around closely spaced pile groups with a normalized pile spacing of 2.5 were investigated based on a series of dynamic effective stress analyses of the three-dimensional (3D) finite element method (FEM). Soil-pile structure system in both dry sand and saturated sand model in the centrifuge shaking table tests with 30g in the previous study (Suzuki et al., 2006) was simulated by using FLIP ROSE 3D ver.1.6 (FLIP consortium, 2018), considering three-dimensional multiple shear mechanisms for soil (Iai, 1993). The major findings as obtained from the present study are shown as follows:

1) The pile group effect was successfully simulated by analysis in the dry sand model. While the group pile effect in the saturated sand model during liquefaction was not remarkable as compared with dry sand. These results agreed with the previous findings.

2) The computed result shows that the mechanism of subgrade reaction in a pile group is different between dry and saturated ground; the horizontal compression normal stress of soil at the front side of the pile mainly contributes to the subgrade reaction in dry sand, while pore water pressure reduction at the back side of a pile is the main cause in saturated sand.

3) The calculated hysteresis of pore water pressure and relative displacement between soil and pile was qualitatively consistent with the observed hysteresis in the large-scale shaking table tests (Tokimatsu and Suzuki 2004). That was, the pore water pressure tended to decrease as the relative displacement increased at the back side of the pile in the trailing row, and the trend was more significant in the pile outside of the pile group than the pile inside.

We confirmed that a series of computed results supported the presumed mechanisms revealed in the previous shaking table tests.

3.19 EQUIPMENT ISOLATION IN ADVANCED NUCLEAR POWER PLANTS

Presenter: Kaivalya Lal

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Abstract: Seismic isolation can substantially reduce earthquake loadings on structures, systems, components (SSCs), and equipment, and is being considered for application to advanced nuclear reactors. The conventional implementation isolates the reactor building at its base and was the focus of several completed research projects funded by the U.S. Department of Energy (DOE) and the U.S. Nuclear Regulatory Commission (NRC). An alternate implementation of seismic isolation involves the protection of SSCs and equipment inside a reactor building, which is the focus of an ongoing MEITNER project funded by the DOE Advanced Research Projects Agency–Energy (ARPA-E).

The MEITNER project includes an experimental program that is investigating the application of seismic isolation to equipment. Four models are being evaluated, with three isolated at the base, and one isolated at its mid-height. This study examines a non-traditional solution for seismic isolation of safety-related equipment, wherein a cylindrical vessel was isolated at its mid-height and near its center of gravity. The test specimen was 240 inches tall and had an outer diameter of 60 inches. The vessel was supported on a stiff steel frame by three equally spaced mounts. Friction Pendulum bearings were installed between each mount and the steel frame. The test specimen was subjected to three component ground motions using a 6-degrees-of-freedom earthquake simulator at the University at Buffalo. The vessel was filled with water and sample internals were attached to the vessel head. The total weight of the vessel including the water, head, and internals was approximately 40 kips. Three configurations were tested, non-isolated, isolated using single Friction Pendulum bearings, and isolated using triple Friction Pendulum bearings. Data from the experiments show that mid-height isolation enables significant reductions in seismic demands in both the vessel and its internals.

3.20 DYNAMIC DISTRIBUTED FIBER OPTIC STRAIN SENSING FOR LAB TESTING

Presenter: Matthew DeJong

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Abstract: Distributed fiber optic sensing (DFOS) is emerging as a viable sensing option for both field and laboratory applications. In particular, Brillion Optical Frequency Domain Reflectometry (BOFDR) is a DFOS technique that is particularly suited to laboratory testing and dynamic loading. This presentation will highlight the benefits of BOFDR for laboratory testing and then summarize recent laboratory tests that demonstrate these benefits.

BOFDR allows strain measurement at approximately every millimeter along a fiber optic cable up to 50 meters long at a sampling rate of up to 250 Hz, depending on the sensing cable length. Fiber optic cables can be embedded in concrete, epoxied to the surface of steel beams or reinforcing bars, or embedded in the soil. Our research has recently quantified the performance of 6 different types of sensing cable, providing different sensing options for different applications. Thus, for measuring SSI under earthquake loading, BOFDR could enable high-resolution measurement of dynamic distributed strain in the soil, within the structure, or on the surface of the structure, while minimizing cabling.

To demonstrate these benefits, three laboratory testing campaigns will be summarized. First, a testing campaign involving tensile testing of reinforced concrete members with cables embedded in both the concrete and rebar will be used to demonstrate the ability of BOFDR to measure strain inside reinforced concrete elements and to detect the onset of concrete cracking very early. Second, test results from cyclic loading of a beam-column joint will be used to demonstrate the ability to detect distributed damage (e.g. plastic hinge length) up to large strain levels. Third, results from dynamic tests of a scaled wind turbine tower model with bolted sections and surface epoxied cables will be used to demonstrate the ability to detect damage in both steel connections and material.

3.21 NONLINEAR SEISMIC FRAGILITIES FOR USE IN SEISMIC PERFORMANCE ASSESSMENTS

Presenter: Michael Salmon

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Abstract: This presentation presents a novel approach for the computation of seismic fragilities which incorporates the nonlinear response of soil and structure elements. The presentation summarizes the key features of the approach which includes capturing nonlinear response at multiple hazard levels and the consideration of realistic ground motions.

DOE O-420.1c mandates that the national laboratories examine changes in seismic hazard on a 10-year basis. If the hazard has changed significantly, then re-analysis of structures, systems, and components important to safety must be performed. The challenge for existing facilities is to compute the performance for a hazard that is frequently above the design basis. This has been typically done using probabilistic risk approaches wherein the seismic fragility of a structure system or component is derived based on a response analysis of the structure to a given design-level or review-level ground motion. The risk assessment approach then inherently assumes a linear response of the structure for other ground motion levels. This may be unnecessarily conservative for robust structures and/or when beneficial response nonlinearities exist, wherein functional performance is maintained for much larger ground motions than was analyzed.

The novel seismic performance approach being undertaken at Los Alamos for a key critical nuclear facility addresses unintended conservatism by explicitly computing the mean probability of failure of the structure to performance at multiple ground motion hazard levels. It is suggested that this new paradigm is appropriate and promising for critical facilities to obtain a more accurate calculation of seismic performance. The framework for the project is presented in detail so that other projects wanting to undertake new seismic risk assessments may learn from lessons on this project. Early lessons learned from the application of the novel approach are presented.

3.22 MODELING INPUT NEEDS FROM LARGE-SCALE SHAKE TABLE TESTING – CASE STUDY FROM SAFETY ANALYSIS OF A DOE FACILITY

Presenter: Mohamed Talaat

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Abstract: This presentation summarizes a state-of-the-art seismic risk assessment of a DOE structure and focuses on the opportunities that large-scale shake table testing capabilities provide for increasing confidence in simulated behavior under extreme conditions. The facility is located in a deep soil site with soil layer inversions and significant soil nonlinear response expected at high shaking levels. The seismic response characterization uses explicit nonlinear dynamic analysis of the structure and surrounding soil domain. The subject of this presentation is motivated by the characterization of seismic demands on the structure components using the soil-structure interaction (SSI) simulation and the strong effect of nonlinear soil response on these demands.

Preliminary risk assessment indicated that annual frequencies of some structural failure modes of interest are influenced by the effect of soil nonlinearities on structure demands. The nonlinear soil response at risk-significant ground shaking levels exhibits soil deformations that exceed what is typically involved in physical testing performed for site characterization. The fidelity of the simulation model to generate estimates of the structure demand, and therefore risk estimates, can therefore be strongly influenced by judgment involved in defining soil material properties at large strains, e.g., hysteretic damping.

This fidelity is also strongly influenced by the idealizations involved in computationally propagating a constitutive soil material model based on in-situ test location or a small-sized sample response to the dynamic response of a large soil domain, whose adequacy is not presently benchmarked against test data at high deformation levels. This calls into focus whether and how numerical models should adapt soil properties and modeling parameters extracted from individual samples to represent nonlinear behavior that engages a relatively large soil domain.

We will summarize the preliminary results of the case-study project where insights indicate a strong influence of such soil behavior, and underscore model improvement opportunities from shake-table testing.

3.23 OPPORTUNITIES FOR IMPROVING THE SEISMIC SAFETY OF CRITICAL DOE FACILITIES THROUGH LARGE SCALE SHAKE TABLE TESTING COMBINED WITH HIGH-FIDELITY NUMERICAL SIMULATIONS

Presenter: Payman Tehrani

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Abstract: The soil-foundation-structure system response of a critical DOE facility was evaluated as part of its seismic safety assessment. The assessment used an integrated 3D nonlinear seismic Soil-Structure Interaction (SSI) analysis methodology in which the structure, site, and their interaction are modeled explicitly. The semi-embedded reinforced concrete building is supported on a site with varying geomaterial properties over depth including multiple shear stiffness reversals where a relatively soft geomaterial is sandwiched between significantly more competent geomaterial layers.

The adopted integrated seismic assessment approach would accommodate the incorporation of spatial variability of site stratigraphy and dynamic geomaterial properties, e.g. density, shear stiffness and strength, and hysteretic damping through continuum finite element discretization for 3D site response characterization. However, it is common practice to idealize the site as horizontally layered with uniform "average" properties assigned to each layer considering the limited footprint of the structure and in many cases the limited scope of geotechnical investigations and site characterization programs. Additionally, the benefit of investment in a more rigorous representation of the 3D site effects cannot be reliably quantified due to the scarcity of large-scale experimental data documenting the impact of such considerations on the site response and ultimately structural demands. Large-scale shake table testing combined with high-fidelity SSI simulations provides a unique opportunity to quantify the sensitivity of the structural response to common site response analysis idealizations.

The seismic SSI analysis results of the DOE facility under study will be used in this presentation to identify the key inputs into 3D site response analysis whose impact on the structural response should be quantified via large-scale shake table testing and simulations of various resolutions.

3.24 MODEL-IN-THE-LOOP TESTING OF A BASE-ISOLATED CYLINDRICAL TANK

Presenter: Sai Sharath Parsi

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Abstract: Model-in-the-loop (MIL) testing, also referred to as real-time hybrid testing, could be used as an alternate strategy for seismic qualification of structures, systems, or equipment because only part/parts of an equipment or a system are physically constructed and tested, referred to herein as a physical subsystem (PS), and the effects of the remaining components on the test specimen are simulated virtually using numerical models, referred to as a virtual subsystem (VS). The Advanced Research Program Agency–Energy (ARPA-E) is currently funding a MEITNER project to investigate opportunities for the application of MIL for the qualification of large safety-class nuclear equipment. As a part of this project, an experimental program is currently underway at the University at Buffalo wherein MIL is conceptualized, implemented, and demonstrated for a base-isolated cylindrical tank (4 feet high and 4 feet in diameter). The tank and the contained fluid are part of the PS, and a system of seismic isolators at its base constitute the VS. In the MIL experiments, the tank is mounted atop a uniaxial hydraulic shake table, 4 feet x 3 feet in plan, which is used to mimic the isolation system motion at the base of the tank. MIL controls are designed to drive the shake table actuator, the shear force at the interface is measured using load cells and it is provided as feedback to the VS model, so the PS and VS interact in real-time throughout the test. The goal of the control design is to drive the shake table platform such that the acceleration at the base of the vessel is equivalent to that which the isolation system represented in the VS would have imposed. The test system modeling, design, verification, and validation of controls, and test results for this MIL demonstration will be discussed in this presentation.

3.25 LBNL NERSC BASE ISOLATED FLOOR – USING SCIENCE TO PROTECT SCIENCE

Presenter: Tim Hart

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Abstract: The National Energy Research Scientific Computing Center (NERSC), located at the Lawrence Berkeley National Laboratory, is the high-performance supercomputer center for the Department of Energy Office of Science. This 149,000-square-foot facility, which opened in 2015, houses the supercomputers supporting research projects of more than 7,000 active users from national laboratories, universities, and private industry.

The NERSC facility is directly adjacent to the Hayward Fault, which according to USGS estimates has a 72 percent chance of generating a large magnitude earthquake within the next thirty years. Given this high probability and the importance of the supercomputer systems to DOE research, the supercomputers are located on a base isolated floor system that was constructed with the building. This floor was the first of its kind installed at a DOE national laboratory.

The floor was designed to accommodate 18 inches of lateral building movement and utilizes a system of steel-framed carts on wheels. The carts are isolated from the building floor by large springs that are tuned to isolate the floor, allow for maximum flexibility in the layout of the computing systems, and allow for the computers to be installed and replaced as needed for research.

To validate the design, full-scale sections of the isolated floor were tested at the UC Berkeley Richmond Field Station shake table. The sections were tested for 7 different ground motion time histories and for a variety of layouts to simulate the incremental installations of the computers. The tests showed that the system isolated approximately 50 percent of the ground motions imparted from the floor.

The proposed presentation will review the procedures used to design the isolated floor, the shake table testing done to verify the design, the challenges that were encountered during the design and construction process, and how those challenges were overcome.

3.26 INVESTIGATING THE INTEGRITY OF BURIED STRUCTURES AND FACILITIES DURING EARTHQUAKES: EXPERIMENTAL CONCEPTS FOR THE SOIL-STRUCTURE-INTERACTION EXPERIMENTAL FACILITY AT UNR IN RENO

Presenter: Yves Guglielmi

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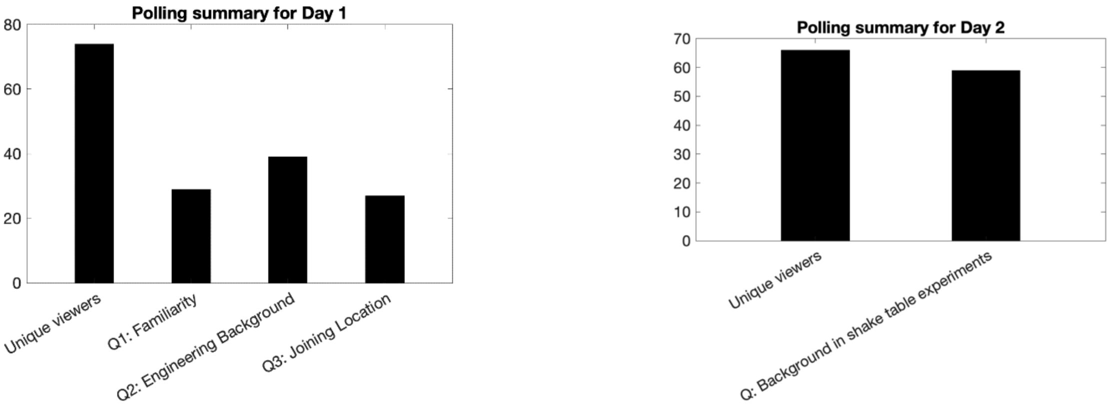
Abstract: The integrity of buried structures important for subsurface energy applications such as hydrocarbon and geothermal wells, pipelines, and underground storage facilities can be severely impacted during earthquakes. LBNL and the Nuclear Waste Management Organization of Japan have collaborated since 2016 on understanding the modification of fault hydrogeology during and following earthquakes. This phenomenon has great importance for the geologic disposal of radioactive waste, where the long-term performance depends on the capability of the natural system to contain radionuclides for hundreds of thousands of years. Within the frame of a California Energy Commission-funded project, LBNL researchers are modeling how the complex vertical-to-horizontal strain associated to the passage of the seismic waves can potentially damage hydrocarbon wells and pipelines. LBNL is developing new broadband sensors for measuring the co-seismic three-dimensional displacement in boreholes and is using these and other high-resolution instrumentation in dedicated field experiments (San Juan Bautista, CA) that are analyzed with full-physics numerical modeling to explore these complex dynamic hydro-mechanical processes. Adding the 21.5 ft x 15 ft soil box facility at UNR in Reno to this research portfolio offers a unique opportunity to study the response of the complex soil-buried structure system to controlled dynamic wave loadings at the relevant multi-meter scale, and thus to bridge the gap between field-scale experiments and numerical models. One experimental concept comprises installing a vertical well assembly replica fully instrumented with advanced displacement sensors in the soil box which would be filled with layered soils. This would allow studying the consequences of dynamically triggered shear at the soil layer interfaces and assessing the vertical ground motion amplification due to variations in seismic wave velocities on borehole integrity. A following step would be to saturate the soil and explore the effects of dynamic shaking on the characteristics of the soil layers and their interfaces.

4 CONCLUSIONS

4.1 SUMMARY

The objective of the workshop was to take stock of the current state-of-art in large-scale shake table testing and identify the scope of application for the newly constructed DOE-funded laminar soil box shaking facility at the University of Nevada, Reno (UNR) to fill scientific and engineering knowledge gaps.

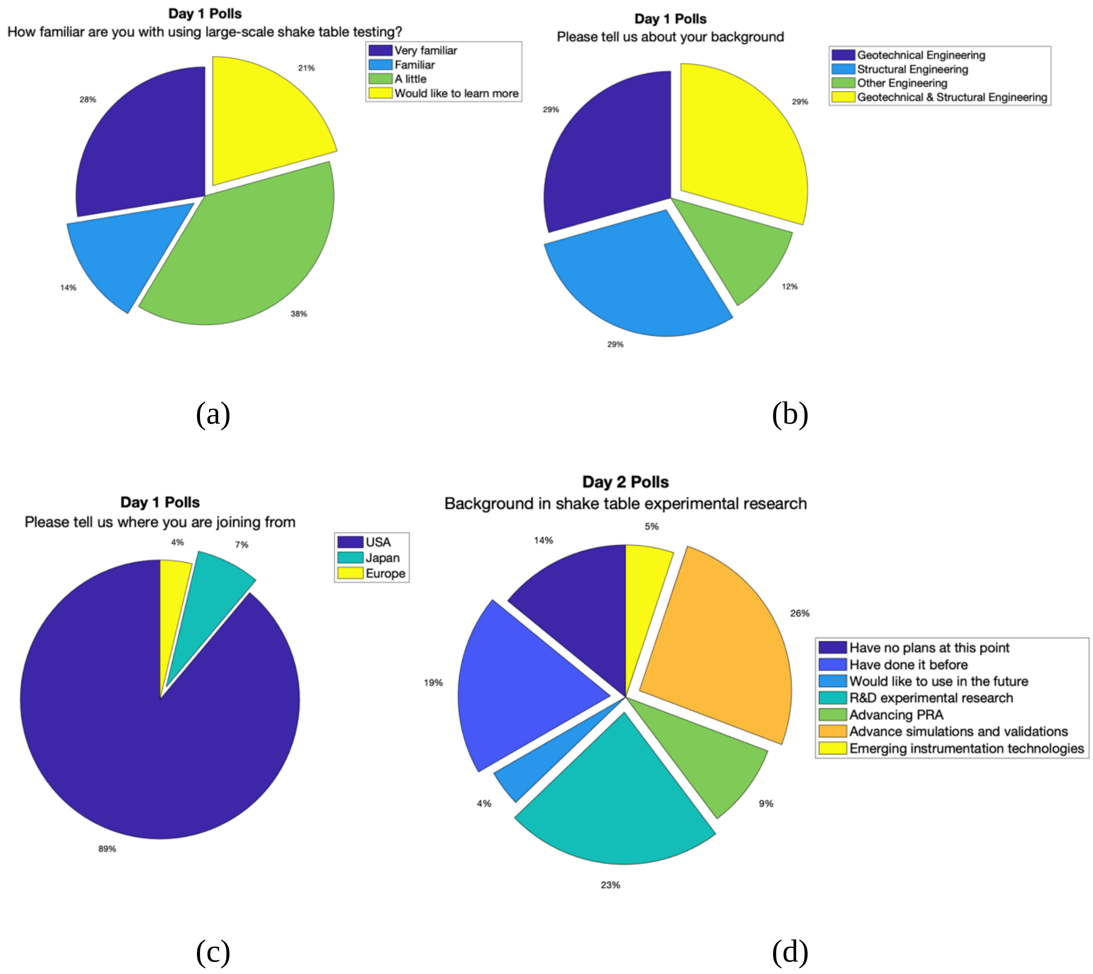
The workshop was well attended on both days with a broad scientific representation. The discussions between the presentations were live and the viewers participated enthusiastically in the polls. Figures 4.1a and 4.1b show the maximum number of unique viewers at any given time on Day 1 and Day 2 respectively. These numbers may be assumed to be the maximum number of potential responders to the polling questions, of which more than 50% responded on Day 1 and almost 100% participated on Day 2.



(a) (b)

Figure 4.1 Viewer participation in the polls carried out to understand the background and needs of the audience on (a) Day 1, and (b) Day 2 of the workshop.

Figure 4.2 summarizes the responses to individual polling questions which highlights the interest of the scientific community in large-scale shake table testing despite having little to no familiarity with the domain (Figure 4.2a). The background of participants was evenly distributed between geotechnical and structural engineering (Figure 4.2b), though most of them were joining from the United States (Figure 4.2c). Most interesting was the background of the participants in the shake table experimental research presented in Figure 4.2d. It shows that the majority of experimentalists wanted to use large-scale shake tables to advance research and development-type research as well as simulations and validations, a theme observed in tandem with the presentations made during the workshop.



(a) (b)

(c) (d)

Figure 4.2 Individual polling responses to (a) Day 1: question – Familiarity with large-scale shake table testing, (b) Day 1: question – Engineering background, (c) Day 1: question – Joining location, and (d) Day 2: question – Background in shake table experiments.

According to the presentation, discussions and post-workshop assessment of abstracts, the following key areas of the workshop were identified:

1. Practical design and numerical modeling of the large-scale laminar soil box
2. Knowledge gaps in soil-structure interaction
3. Seismic monitoring systems and laboratory instrumentation
4. Importance of verification and validation (V&V) exercises
5. The need for experimental data to refine PRA methodologies

4.1.1 Practical design and numerical modeling of the laminar soil box

Detailed presentations on the engineering design considerations explained the novel approach of using elastomeric bearings to fulfill two competing demands of using a rubber compound that can resist high tensile forces and also have a high shear modulus while keeping in mind that high shear modulus translates to a stiffer shear response, which is undesirable. This complex design optimization was achieved after several iterations and extensive numerical testing. Eigenvalue analysis with elemental mass showed excellent comparison with hand calculations demonstrating that the box is “invisible” to the soil. The increasing complexity of 1-dimensional (1D), 2-dimensional (2D), and 3-dimensional (3D) numerical models of the soil box were also discussed and emphasized the need for 2D or 3D models to model the complex nature of soil-structure interaction, quantifying its capabilities, and parameterize the final design.

4.1.2 Knowledge gaps in soil-structure interaction (SSI)

The presentations in this workshop were not limited to soil-structure interaction but rather expanded to fluid-soil interaction, fluid-structure interaction, and soil-pile-structure interaction. The data gathered from some of these studies will be made available on DesignSafe, a popular collaborative platform for scientists and engineers, while some of them will be used to make recommendations to existing code provisions. For example, one study identified smaller damping in the convective mode under fluid-structure interactions than prescribed in ASCE 4, whereas another study highlighted the limitations of ASCE 4 provisions for kinematic SSI because they are based on simple structures with regular foundation footprint and embedment depths which makes them inapplicable to nuclear power plant structures. A centrifuge shaking test at 30 g to study soil-pile-structure interaction concluded that the pile group effect is more significant in dry sand than saturated sand and that this effect causes much greater shear forces in the piles in the leading row compared to the piles in the trailing rows. Another study recognized two types of soil nonlinearities that are coupled– primary soil nonlinearity due to material, and secondary soil nonlinearity due to induced in nearby structures due to SSI. It identified stiffness contrast ratio to be one of the main variables of SSI and proposed to test this coupled effect in the soil box. From the DOE/NNSA perspective, this soil box system can be utilized to answer fundamental questions related to equivalent linear versus fully nonlinear analysis methods for representation of soil-structure interaction in the seismic analysis of mission-critical facilities.

4.1.3 Seismic monitoring systems and laboratory instrumentations

Fiber optic strain sensing is an emerging field in seismic monitoring systems. These sensors measure cracks opening in concrete more accurately than DIC (digital image correlation). Fiber optic cables respond to changes in the environment around the fiber (temperature, strain, etc) which alters the light backscattering profile, meaning that the cable itself is the sensor. A single continuous fiber-optic cable laid around a soil column or SSI system may be able to replace a dense instrumentation consisting of potentiometers, accelerometers, strain gauges, and bender elements while capturing the same quantities of interest. It was recommended in a presentation to establish a seismic monitoring system to detect the damage in pile systems because the failure of pile systems does not always show surface manifestation or damage to the upper structure.

Another developing technology is to use light-sensitive diodes as a direct, physical alternative to record inter-story drift, as opposed to the conventional method of calculating it indirectly from double integration of measured acceleration, which is rife with complications. Diffused lasers were recommended as part of system diagnostics of the laminar soil box at UNR.

4.1.4 Importance of verification and validation (V&V) exercises

The goal of simulations is to predict and inform engineers so they can assess safety and improve the economy. Verification and validation procedures are used to build confidence and credibility in modeling and computational simulations. The best way to carry out validation experiments was proposed to be a joint design and execution by experimentalists and computationalists to capture the physics within the confines of experimental uncertainty. Third-party analysts were advocated to validate more advanced simulations and follow a parameter selection protocol. The importance of inter-code comparisons for the purpose of V&V was re-iterated in several presentations. Shake table experiments were suggested to be complemented with numerical models with carefully constrained constitutive relationships.

4.2 SUGGESTED EXPERIMENTS/STUDIES

As the seismic demands increase everywhere and we move from conservative to best estimate performance analysis, the value of nonlinear analysis and characterization of soil at large strains was universally acknowledged. Before leveraging the unique capabilities of the large-scale laminar soil box to answer some of the toughest questions in SSI, all agreed to fully characterize the soil box without any structure in it as the first order of business. An advisory panel was recommended consisting of researchers and practitioners. The hope is that the proposed experiments/studies will fill the gaps in knowledge and generate validation data for SSI:

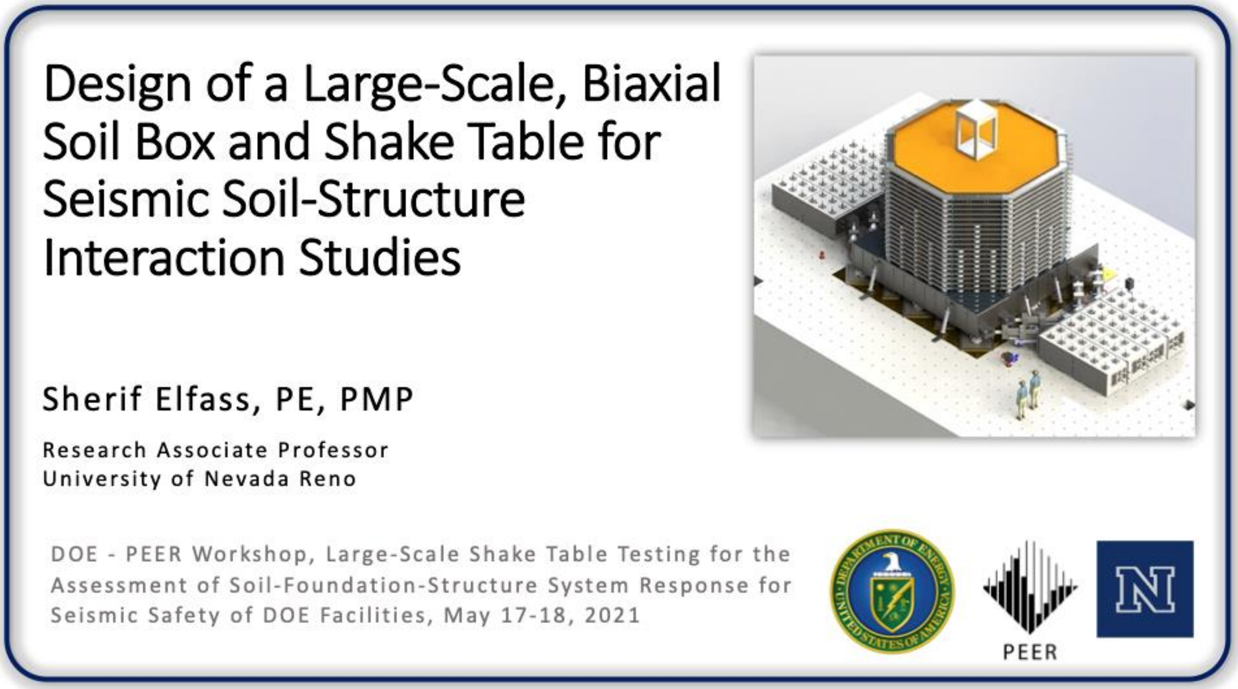
- Understand the importance of rotational coupling of systems that are not rigidly connected at the foundation level and use instrumentation to evaluate resulting load distributions.
- A parametric study was proposed for partially embedded structures with different levels of emptiness containing different volumes of fluid and free surface with heavy instrumentation to monitor divergence from analytical solution at different shaking levels to identify an acceptable range of equivalent linear analysis.
- Centrifuge tests can be utilized as a complement to large-scale 1g testing to study key mechanisms of soil-foundation-structure system response.
- Reproduce existing centrifuge experiments to further enhance the fidelity of simulations.
- Base slab averaging versus inelastic interface and inelastic behavior experiment – Validate or investigate the effects of incoherent motions on SSI by applying Love waves and SV waves with variable soil profiles and variable surface soil

layers. The above experiment is also vital to analyze deeply embedded structures such as small modular reactors (SMR).

- Create an opportunity to test multi-degree-of-freedom excitations.
- Understand the failure mechanism of the SSI systems.
- Investigate kinematic and inertial interaction.
- The effect of spatial variability on response variability may diminish at higher nonlinear responses. Hence, it is important to constrain soil layer variability at those large strains, especially applicable to structures with a large footprint like nuclear power plant structures.
- Model soil layers with different water retention regimes. Distribution of soil moisture in depth will help build suction versus degree of saturation profiles which should be included in the design of SSI systems.
- Investigate SSI effects on equipment isolation systems.
- A comparative study to evaluate equivalent linear versus fully nonlinear models of structure/soil systems

APPENDIX A :
WORKSHOP PRESENTATIONS

DESIGN OF A LARGE-SCALE, BIAXIAL SOIL BOX AND SHAKE TABLE FOR SEISMIC SOIL-STRUCTURE INTERACTION STUDIES

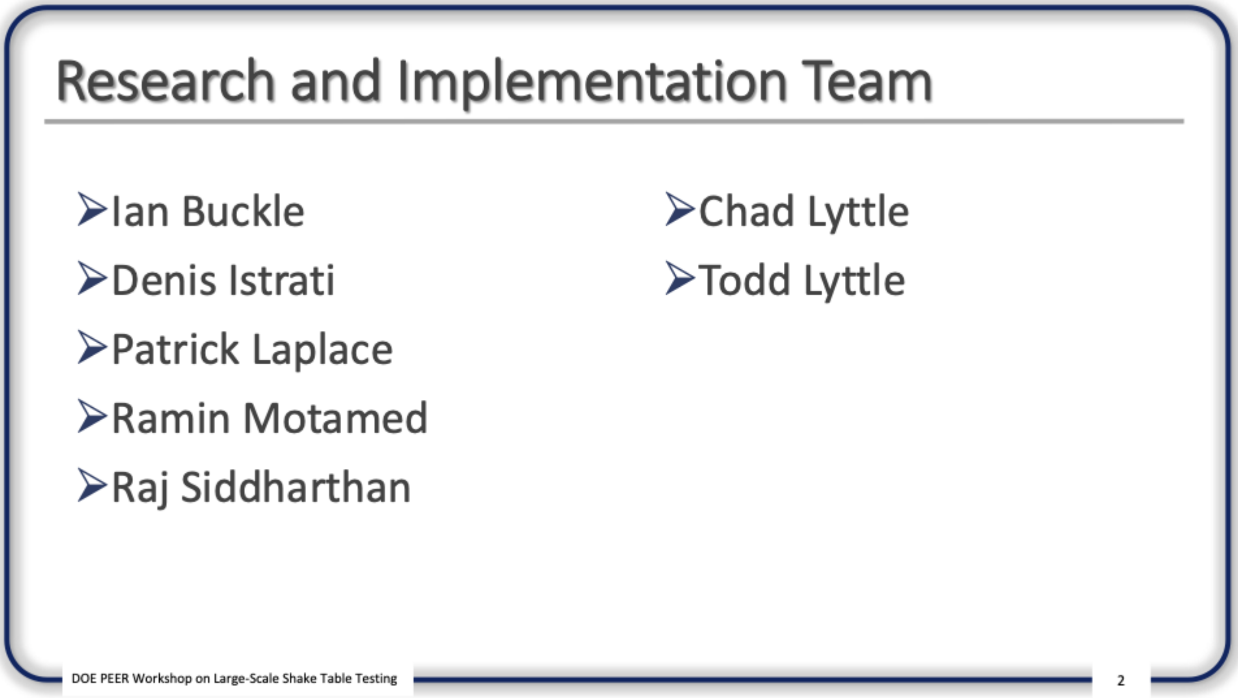


Design of a Large-Scale, Biaxial Soil Box and Shake Table for Seismic Soil-Structure Interaction Studies

Sherif Elfass, PE, PMP

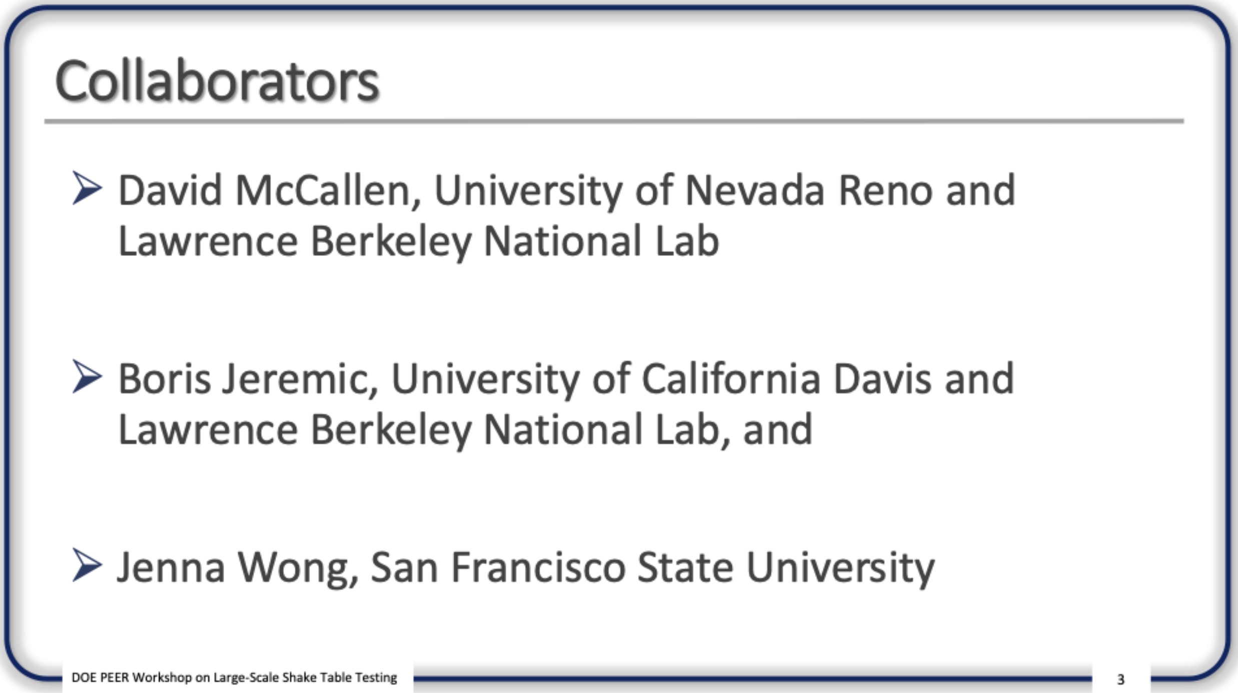
Research Associate Professor
University of Nevada Reno

DOE - PEER Workshop, Large-Scale Shake Table Testing for the Assessment of Soil-Foundation-Structure System Response for Seismic Safety of DOE Facilities, May 17-18, 2021



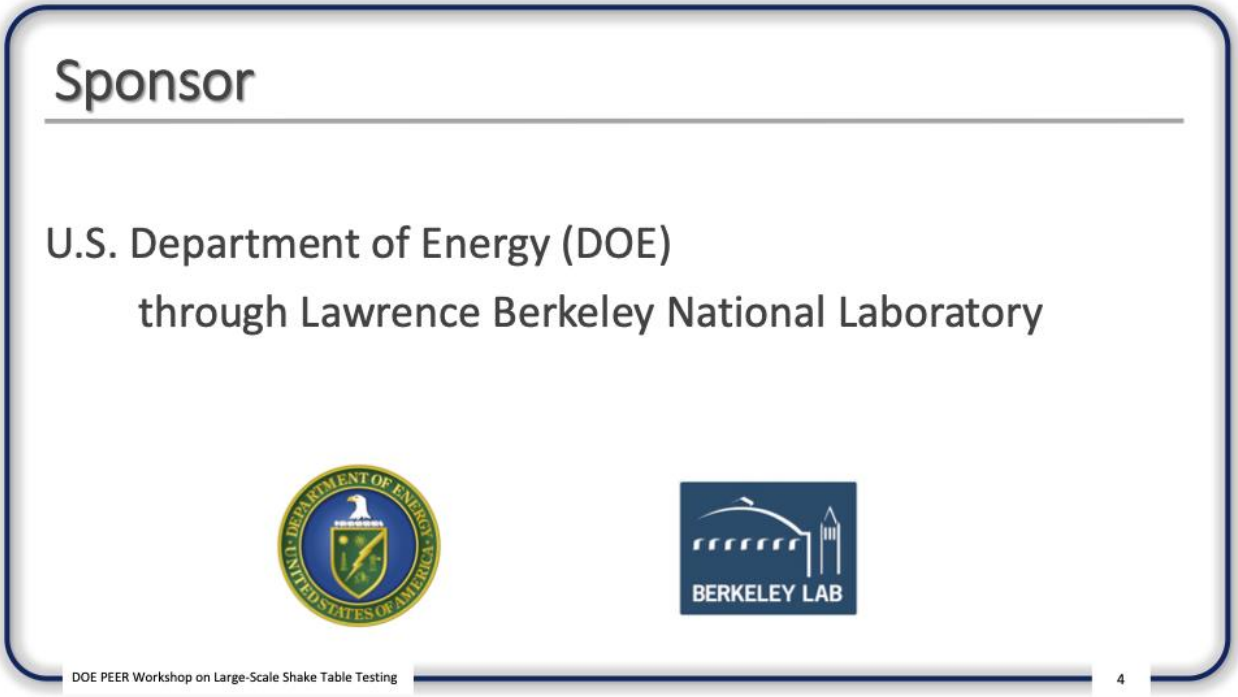
Research and Implementation Team

- Ian Buckle
- Denis Istrati
- Patrick Laplace
- Ramin Motamed
- Raj Siddharthan
- Chad Lyttle
- Todd Lyttle



Collaborators

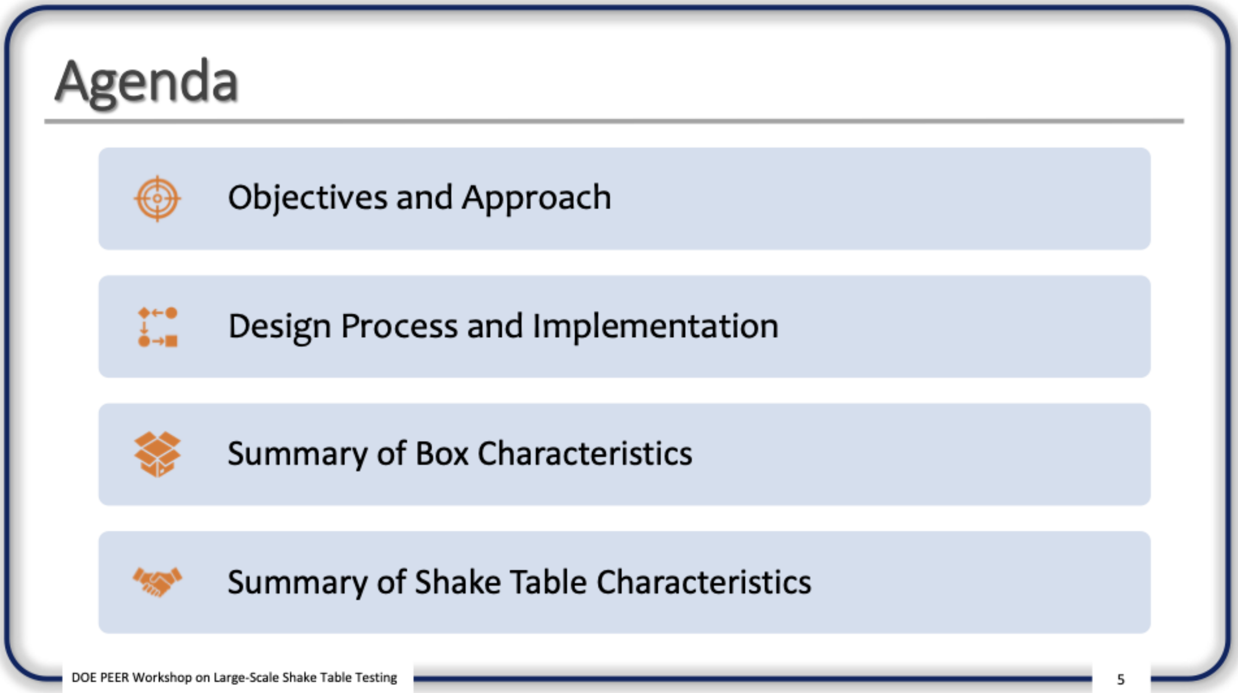
- David McCallen, University of Nevada Reno and Lawrence Berkeley National Lab
- Boris Jeremic, University of California Davis and Lawrence Berkeley National Lab, and
- Jenna Wong, San Francisco State University



Sponsor

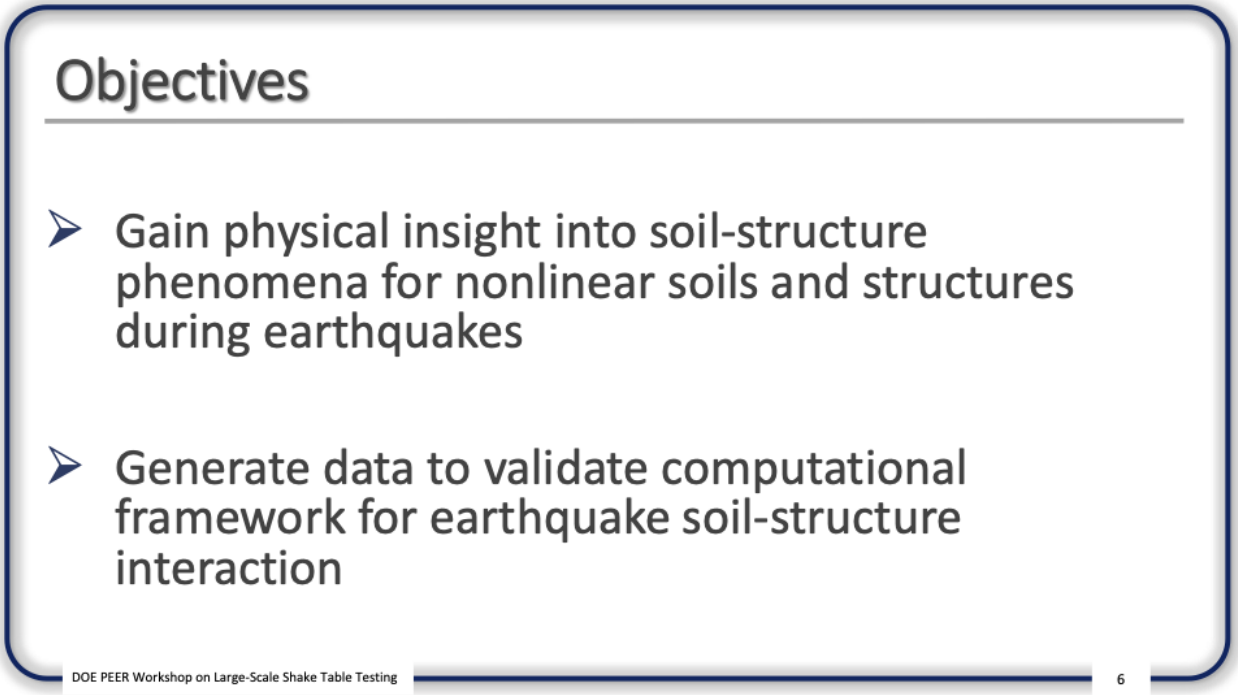
U.S. Department of Energy (DOE)

through Lawrence Berkeley National Laboratory



Agenda

- Objectives and Approach
- Design Process and Implementation
- Summary of Box Characteristics
- Summary of Shake Table Characteristics

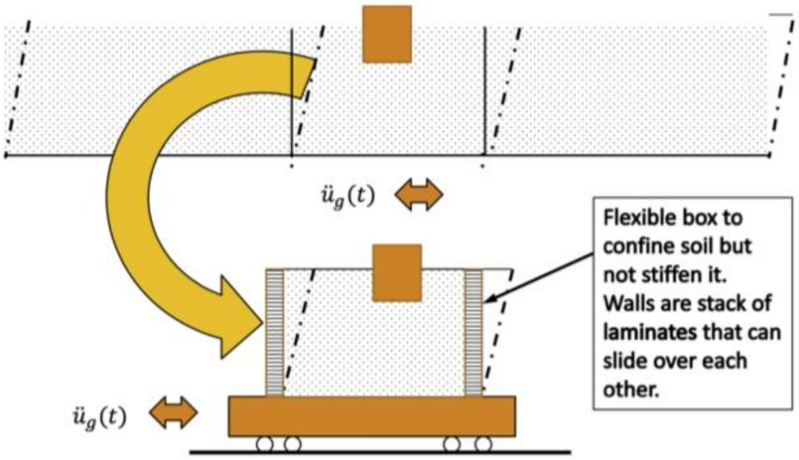


Objectives

- Gain physical insight into soil-structure phenomena for nonlinear soils and structures during earthquakes
- Generate data to validate computational framework for earthquake soil-structure interaction

Approach

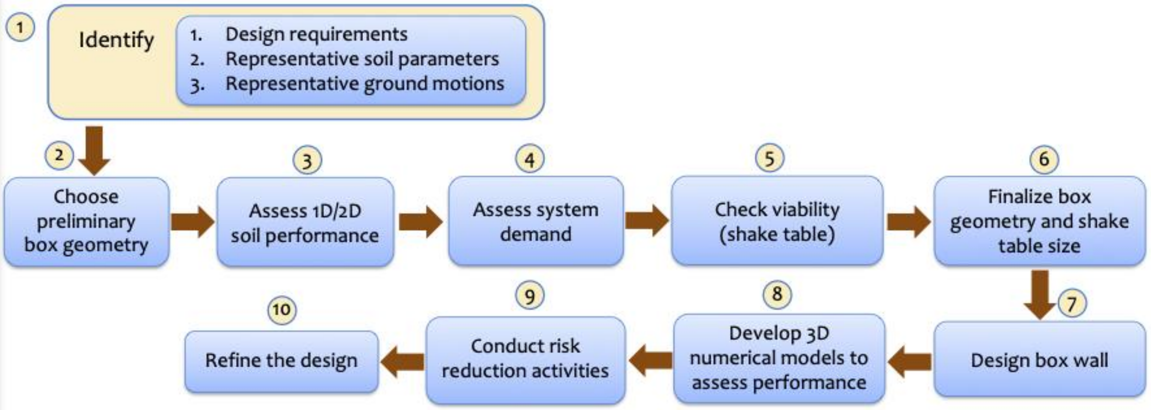
Build a customized shake table with largest practical size of soil box



Flexible box to confine soil but not stiffen it. Walls are stack of laminates that can slide over each other.

Design Process

Design process was iterative and required extensive interaction among team members



① Identify
1. Design requirements
2. Representative soil parameters
3. Representative ground motions

② Choose preliminary box geometry

③ Assess 1D/2D soil performance

④ Assess system demand

⑤ Check viability (shake table)

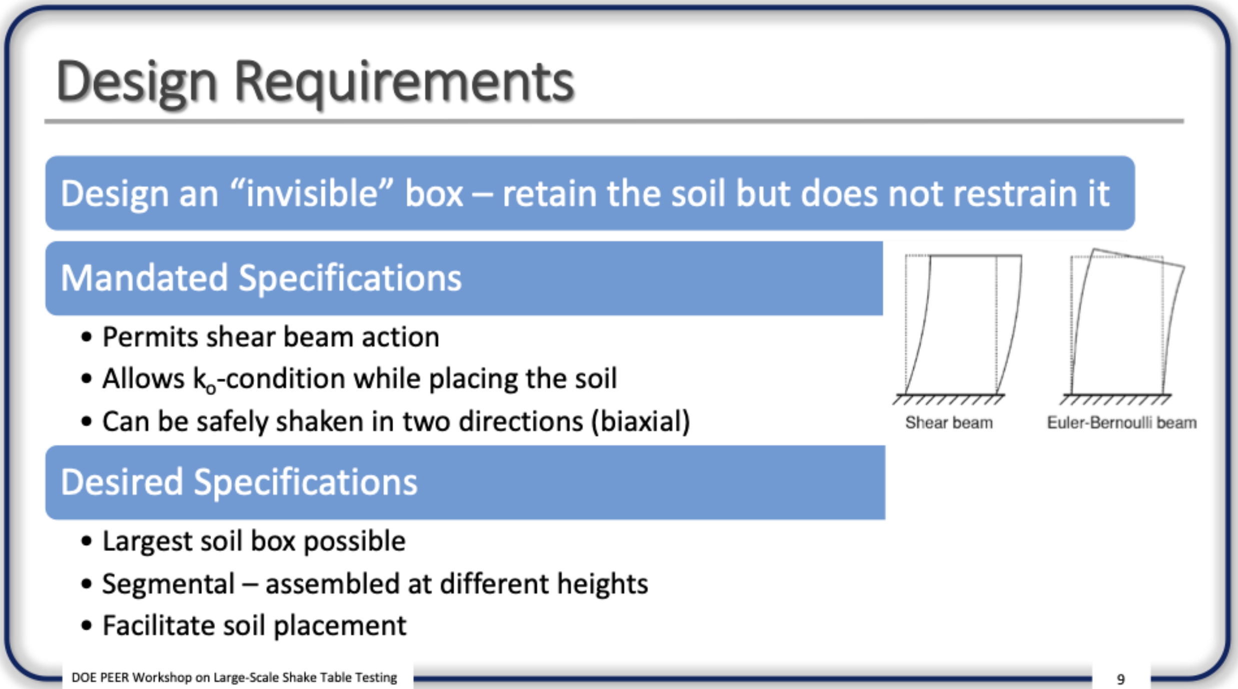
⑥ Finalize box geometry and shake table size

⑦ Design box wall

⑧ Develop 3D numerical models to assess performance

⑨ Conduct risk reduction activities

⑩ Refine the design



Design Requirements

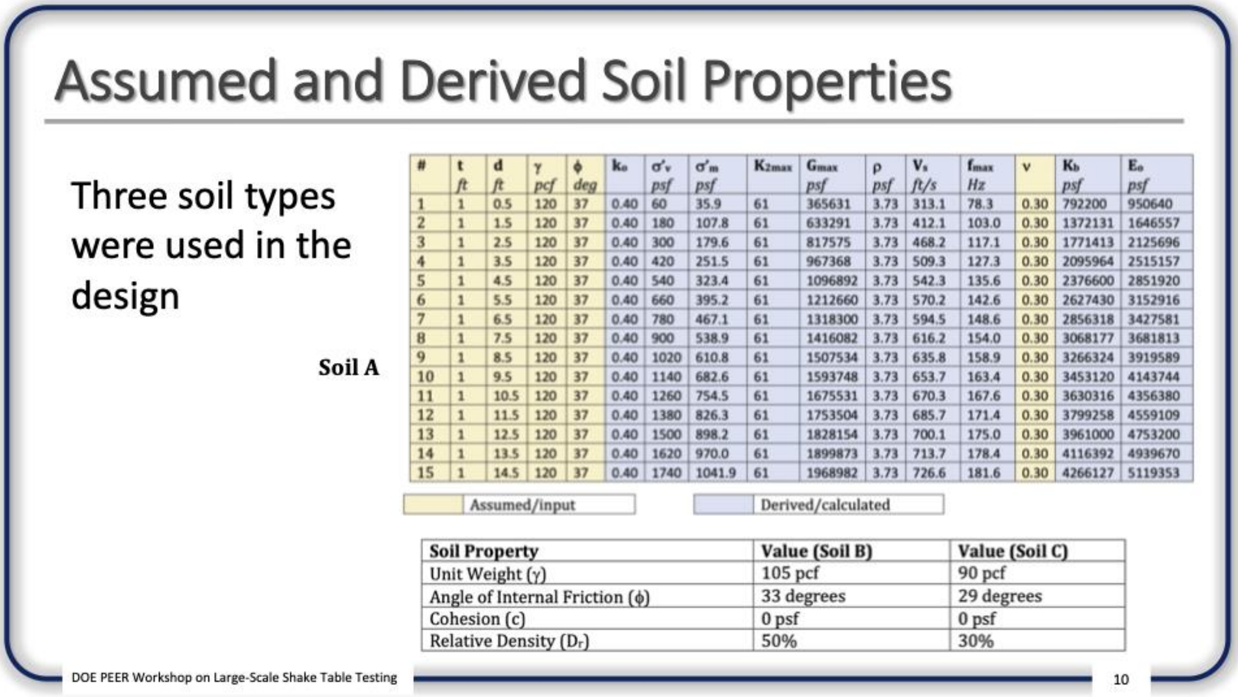
Design an "invisible" box – retain the soil but does not restrain it

Mandated Specifications

- Permits shear beam action
- Allows k_o-condition while placing the soil
- Can be safely shaken in two directions (biaxial)

Desired Specifications

- Largest soil box possible
- Segmental – assembled at different heights
- Facilitate soil placement



Assumed and Derived Soil Properties

Three soil types were used in the design

Soil A

#	t	d	γ	ϕ	k_o	σ'_v	σ'_m	K_{2max}	G_{max}	ρ	V_s	f_{max}	ν	K_b	E_o
	ft	*ft*	*pcf*	*deg*		*psf*	*psf*		*psf*	*psf*	*ft/s*	*Hz*		*psf*	*psf*
1	1	0.5	120	37	0.40	60	35.9	61	365631	3.73	313.1	78.3	0.30	792200	950640
2	1	1.5	120	37	0.40	180	107.8	61	633291	3.73	412.1	103.0	0.30	1372131	1646557
3	1	2.5	120	37	0.40	300	179.6	61	817575	3.73	468.2	117.1	0.30	1771413	2125696
4	1	3.5	120	37	0.40	420	251.5	61	967368	3.73	509.3	127.3	0.30	2095964	2515157
5	1	4.5	120	37	0.40	540	323.4	61	1096892	3.73	542.3	135.6	0.30	2376600	2851920
6	1	5.5	120	37	0.40	660	395.2	61	1212660	3.73	570.2	142.6	0.30	2627430	3152916
7	1	6.5	120	37	0.40	780	467.1	61	1318300	3.73	594.5	148.6	0.30	2856318	3427581
8	1	7.5	120	37	0.40	900	538.9	61	1416082	3.73	616.2	154.0	0.30	3068177	3681813
9	1	8.5	120	37	0.40	1020	610.8	61	1507534	3.73	635.8	158.9	0.30	3266324	3919589
10	1	9.5	120	37	0.40	1140	682.6	61	1593748	3.73	653.7	163.4	0.30	3453120	4143744
11	1	10.5	120	37	0.40	1260	754.5	61	1675531	3.73	670.3	167.6	0.30	3630316	4356380
12	1	11.5	120	37	0.40	1380	826.3	61	1753504	3.73	685.7	171.4	0.30	3799258	4559109
13	1	12.5	120	37	0.40	1500	898.2	61	1828154	3.73	700.1	175.0	0.30	3961000	4753200
14	1	13.5	120	37	0.40	1620	970.0	61	1899873	3.73	713.7	178.4	0.30	4116392	4939670
15	1	14.5	120	37	0.40	1740	1041.9	61	1968982	3.73	726.6	181.6	0.30	4266127	5119353

▨	Assumed/input
▨	Derived/calculated

Soil Property	Value (Soil B)	Value (Soil C)
Unit Weight (γ)	105 pcf	90 pcf
Angle of Internal Friction (ϕ)	33 degrees	29 degrees
Cohesion (c)	0 psf	0 psf
Relative Density (D_r)	50%	30%

Ground Motions

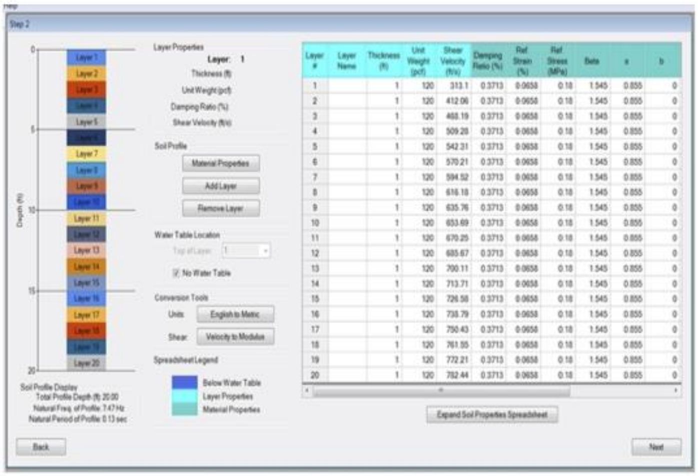
Ground Motions	N. PGA	PGV	PGD
	(g)	(cm/s)	(cm)
Nishi-Akashi 090	0.26	21.67	6.84
Nishi-Akashi 000	0.26	25.48	4.57
Landers 345	0.26	9.37	8.514
Landers 260	0.26	48.38	41.31
Hector 090	0.26	35.88	8.56
Hector 000	0.26	25.77	19.57
Gilroy 090	0.26	17.61	8.49
Gilroy 000	0.26	21.3	5.08
Erzincan ns	0.26	72.87	21.76
Erzincan ew	0.26	41.42	14.86
El Centro 270	0.26	39.09	30.15
El Centro 180	0.26	28.98	8.11
Denali 360	0.26	34.18	26.7
Denali 090	0.26	19.15	11
Cerro 237	0.26	32.28	13.14
Cerro 147	0.26	18.06	8.2

All records were normalized to a peak of 0.26g

Linear Scaling:

0.5x, 1x, 2x, 3x, and 4x

1D Site Response Analyses - DEEPSOIL



Layer: 1

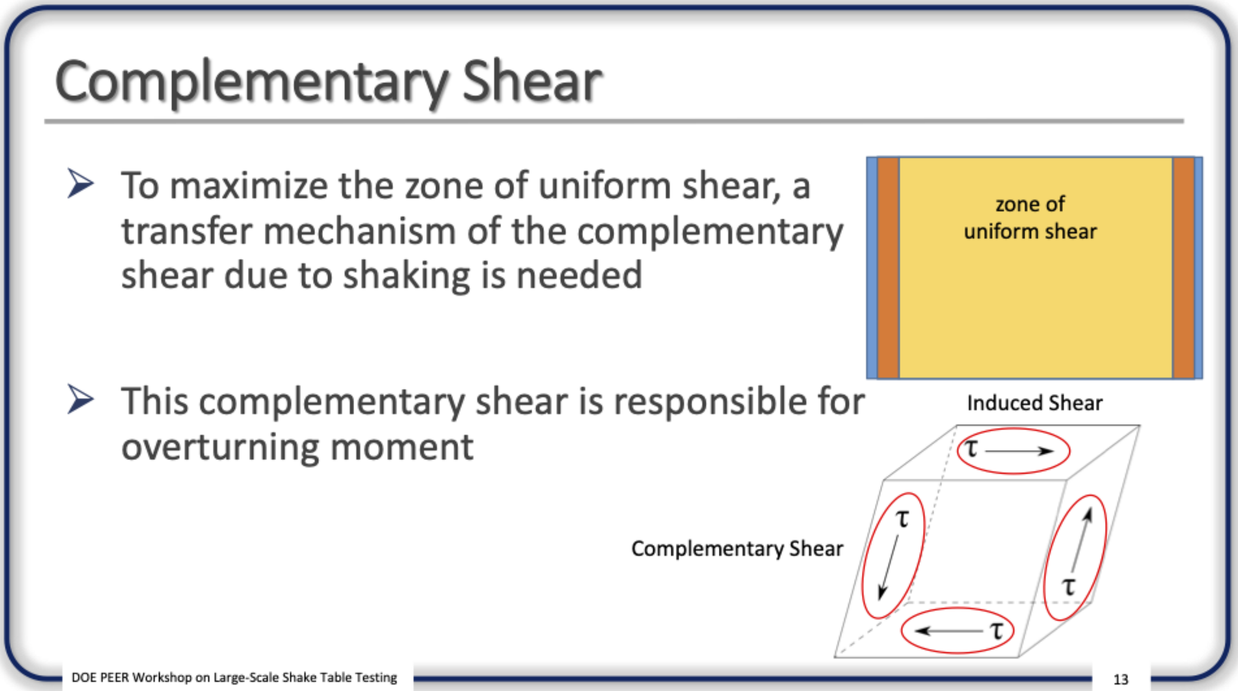
Top of Layer: 1

☑ No Water Table

Soil Profile Display
Total Profile Depth (ft): 20.00
Natural Freq. of Profile: 7.47 Hz
Natural Period of Profile: 0.13 sec

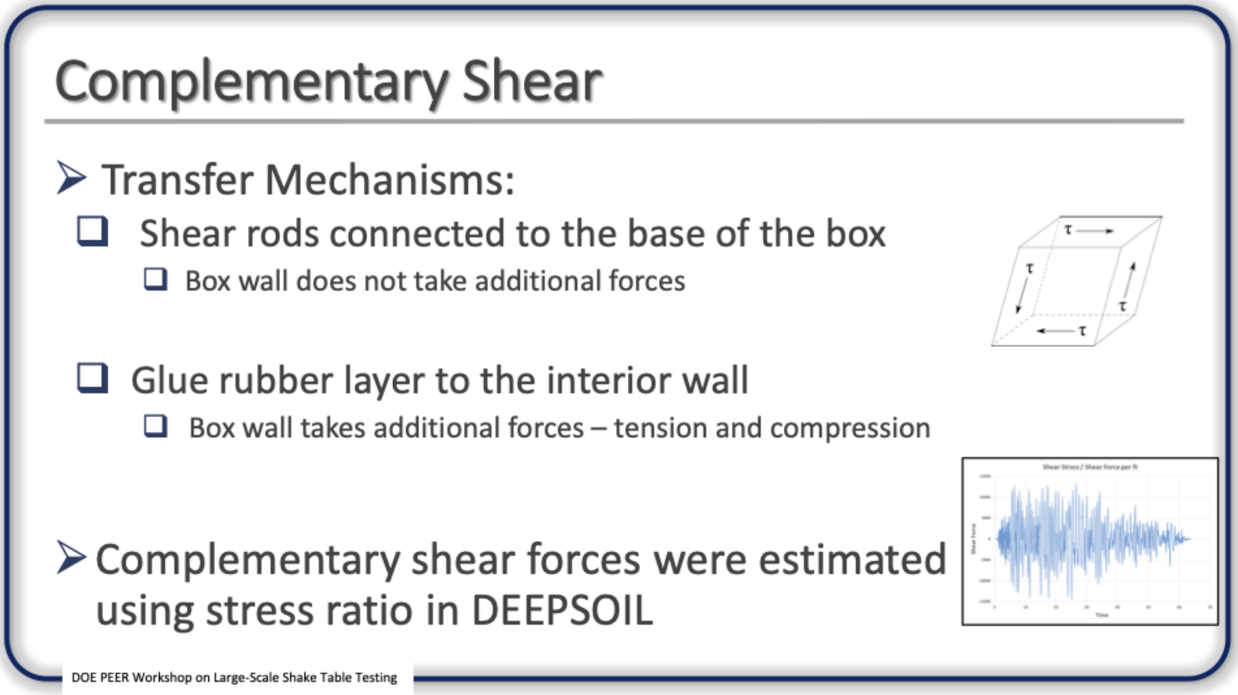
Layer #	Layer Name	Thickness (ft)	Unit Weight (pcf)	Shear Velocity (ft/s)	Damping Ratio (%)	Ref Strain (%)	Ref Stress (MPa)	Beta	s	b
1		1	120	313.1	0.3713	0.0658	0.18	1.545	0.855	0
2		1	120	412.06	0.3713	0.0658	0.18	1.545	0.855	0
3		1	120	468.19	0.3713	0.0658	0.18	1.545	0.855	0
4		1	120	509.28	0.3713	0.0658	0.18	1.545	0.855	0
5		1	120	542.31	0.3713	0.0658	0.18	1.545	0.855	0
6		1	120	570.21	0.3713	0.0658	0.18	1.545	0.855	0
7		1	120	594.52	0.3713	0.0658	0.18	1.545	0.855	0
8		1	120	616.18	0.3713	0.0658	0.18	1.545	0.855	0
9		1	120	635.76	0.3713	0.0658	0.18	1.545	0.855	0
10		1	120	653.69	0.3713	0.0658	0.18	1.545	0.855	0
11		1	120	670.25	0.3713	0.0658	0.18	1.545	0.855	0
12		1	120	685.67	0.3713	0.0658	0.18	1.545	0.855	0
13		1	120	700.11	0.3713	0.0658	0.18	1.545	0.855	0
14		1	120	713.71	0.3713	0.0658	0.18	1.545	0.855	0
15		1	120	726.58	0.3713	0.0658	0.18	1.545	0.855	0
16		1	120	738.79	0.3713	0.0658	0.18	1.545	0.855	0
17		1	120	750.43	0.3713	0.0658	0.18	1.545	0.855	0
18		1	120	761.55	0.3713	0.0658	0.18	1.545	0.855	0
19		1	120	772.21	0.3713	0.0658	0.18	1.545	0.855	0
20		1	120	782.44	0.3713	0.0658	0.18	1.545	0.855	0

- Depth= 20ft & 15ft
- Discretization: 20 & 15 layers of 1-ft each
- Three types of soil
- Seed & Idriss vs Darendeli, Masing Rules



Complementary Shear

- To maximize the zone of uniform shear, a transfer mechanism of the complementary shear due to shaking is needed
- This complementary shear is responsible for overturning moment

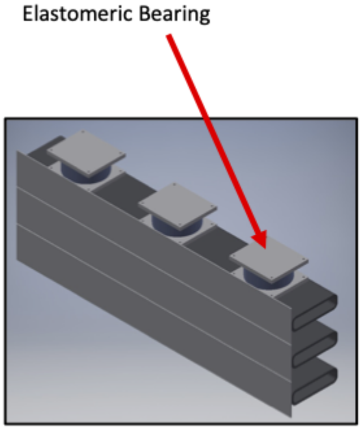


Complementary Shear

- Transfer Mechanisms:
  - ☐ Shear rods connected to the base of the box
    - ☐ Box wall does not take additional forces
  - ☐ Glue rubber layer to the interior wall
    - ☐ Box wall takes additional forces – tension and compression
- Complementary shear forces were estimated using stress ratio in DEEPSOIL

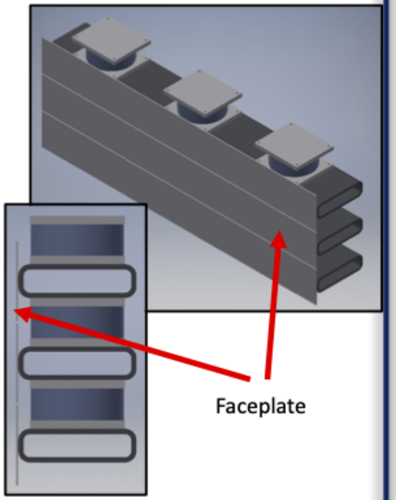
Box Wall Design

- Alternating layers of
  - Hollow steel sections (mass)
  - Elastomeric bearings, EB (stiffness)
- This type of bearing is inherently more robust and better able to withstand large, earthquake-induced, deformations than mechanical devices
- It is believed to be the first application of an elastomeric bearing for this purpose in the US and perhaps worldwide



Box Wall Design (cont'd)

- Box wall transfers complementary shear, through friction, to shake table platen
  - EB under alternating cycles of compression and tension
- Faceplates to retain the soil
- Gap between face plates is sealed from soil spill



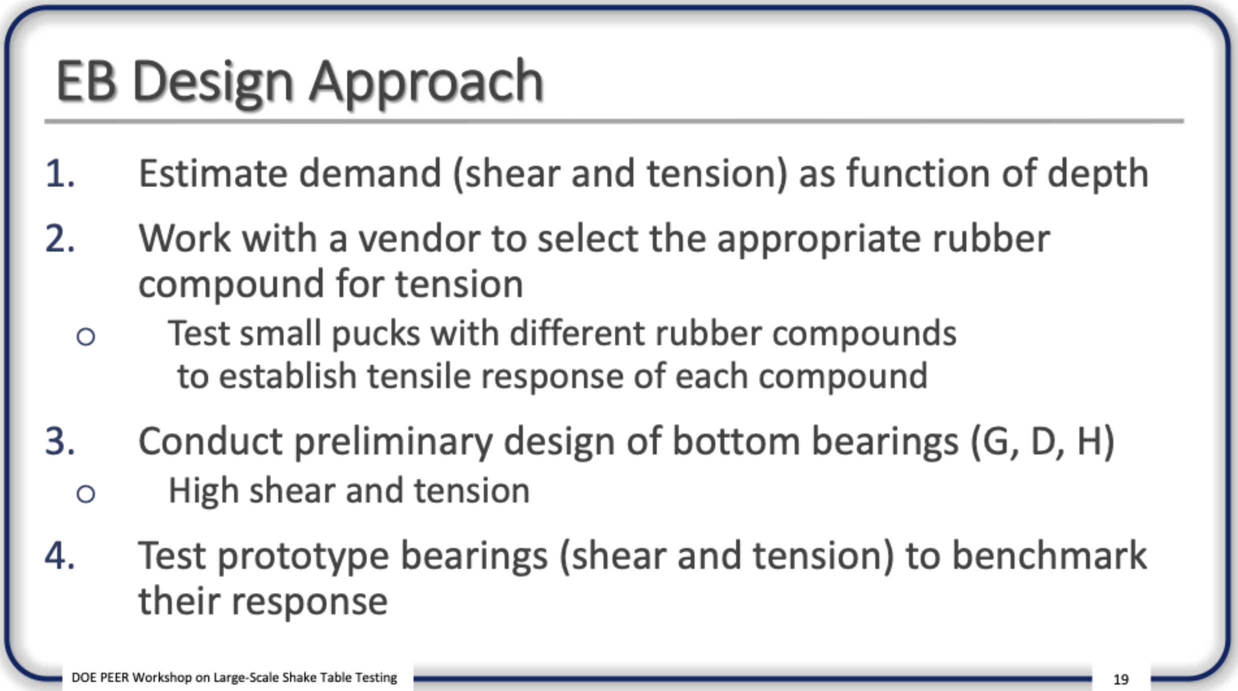
Frame Arrangement of the Soil Box

1	HSS14x4x1/4
2	
3	
4	
5	
6	
7	
8	
9	
10	
11	
12	
13	
14	
15	HSS14x4x5/8
16	
17	
18	
19	
	2" Base Frame

Challenge with Elastomeric Bearing Design

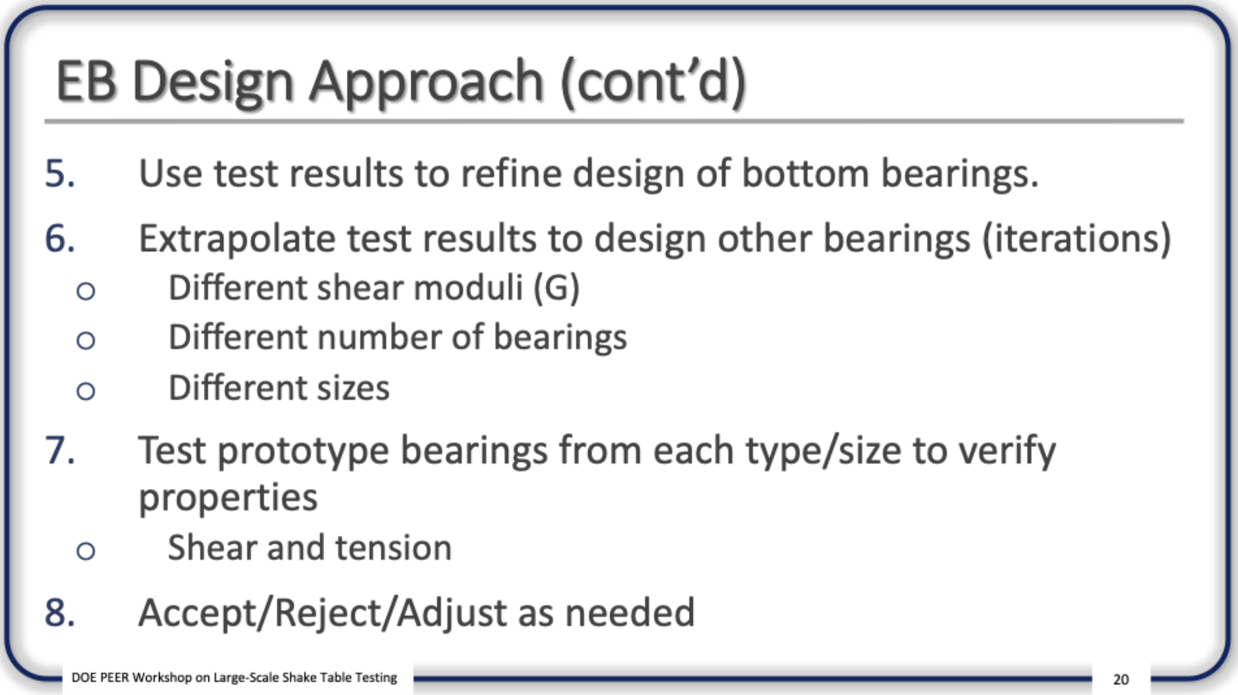
- Two competing requirements
  - Resisting high tensile forces requires rubber compounds with high shear modulus
  - High shear modulus translates to stiffer shear response, which is not desired
    - Shear stiffness of RB should be lower than shear stiffness of soil
- The challenge is to find the "sweet spot"
- Soil shear stiffness and complementary shear vary with depth

Optimized design required iterations and extensive testing



EB Design Approach

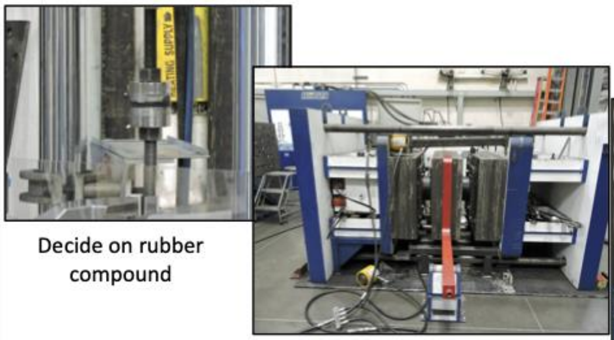
1. Estimate demand (shear and tension) as function of depth
2. Work with a vendor to select the appropriate rubber compound for tension
   - ○ Test small pucks with different rubber compounds to establish tensile response of each compound
3. Conduct preliminary design of bottom bearings (G, D, H)
   - ○ High shear and tension
4. Test prototype bearings (shear and tension) to benchmark their response



EB Design Approach (cont'd)

5. Use test results to refine design of bottom bearings.
6. Extrapolate test results to design other bearings (iterations)
   - ○ Different shear moduli (G)
   - ○ Different number of bearings
   - ○ Different sizes
7. Test prototype bearings from each type/size to verify properties
   - ○ Shear and tension
8. Accept/Reject/Adjust as needed

Elastomeric Bearing Test Program



Decide on rubber compound

Confirming Design Parameters

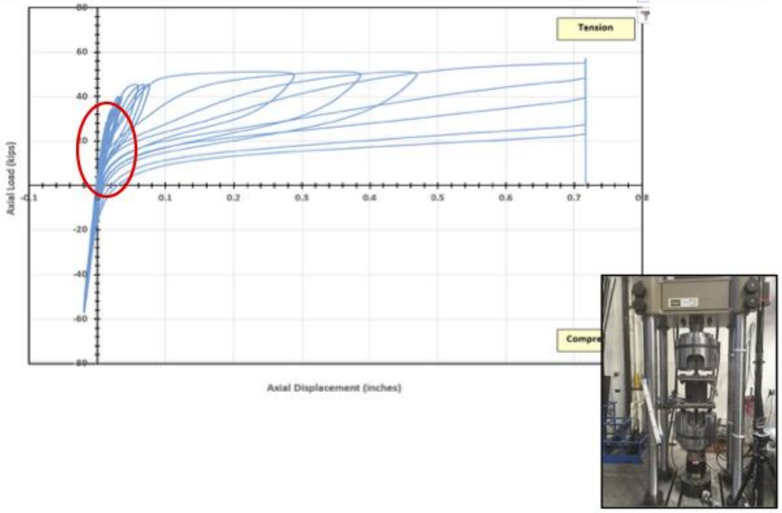
Assess Tension Response

Assess Shear Stiffness

Typical EB Response under Tension

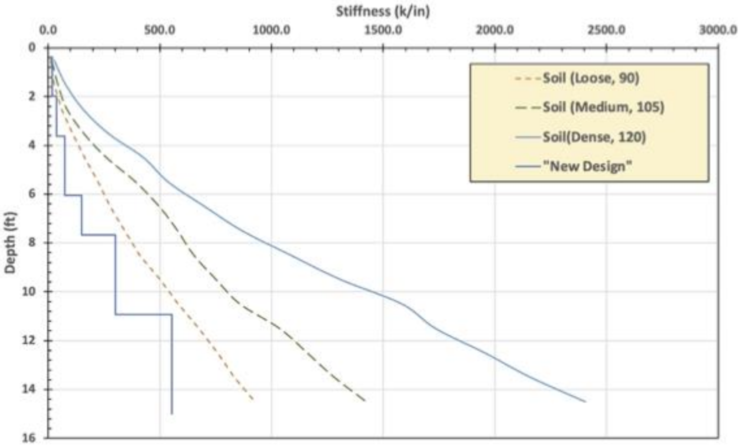
- G = 150 psi
- D = 10 inches
- F_t = 22.8 kips

- Demand = F_t/FOS



EB Design for Shear

- Soil stiffness calculated from DEEPSOIL
- Box stiffness (EB)
  - Shear moduli of EB
  - Number of EB (8, 16, 32)
  - Size of EB



EB Distribution along Box Height

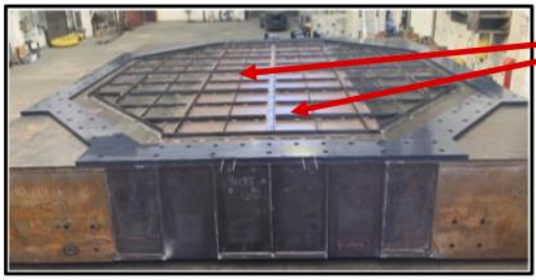
Depth	Number Bearing	Type of Bearing	Type	Size of Bearing
0	8	RB1	A	8"
	8	RB1	A	8"
	8	RB1	A	8"
	8	RB2	B	11"
	8	RB2	B	11"
	16	RB2	B	11"
	16	RB2	B	11"
5	16	RB2	B	11"
	16	RB3	C	11"
	16	RB3	C	11"
	32	RB3	C	11"
	32	RB3	C	11"
	32	RB3	C	11"
10	32	RB3	C	11"
	32	RB4	D	11"
	32	RB4	D	11"
	32	RB4	D	11"
	32	RB4	D	11"
	32	RB5	E	11"
15	408			

As-Built EB Properties

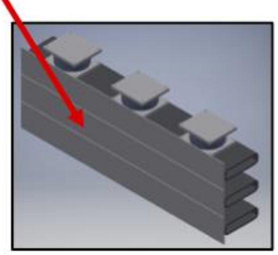
			Tested					Calculated	
Name	Type	Bearing Outer Dameter	Effective Shear Stiffness k/in			Compression Stiffness	Tension Stiffness	k_{tor}	k_{theta}
			100%	25%	7%	k/in	k/in	k-in/rad	k-in/rad
RB1	A	8"	1.13	1.21	1.42	224	170	2.32	252
RB2	B	11"	2.76	3.36	3.70	890	757	4.99	2,485
RB3	C	11"	4.73	5.62	6.77	1,541	1,154	9.59	4,777
RB4	D	11"	8.35	10.11	12.88	2,020	1,679	16.90	8,414
RB5	E	11"	10.81	13.08	16.65	3,525	2,931	21.89	10,899

Soil-Box Interface

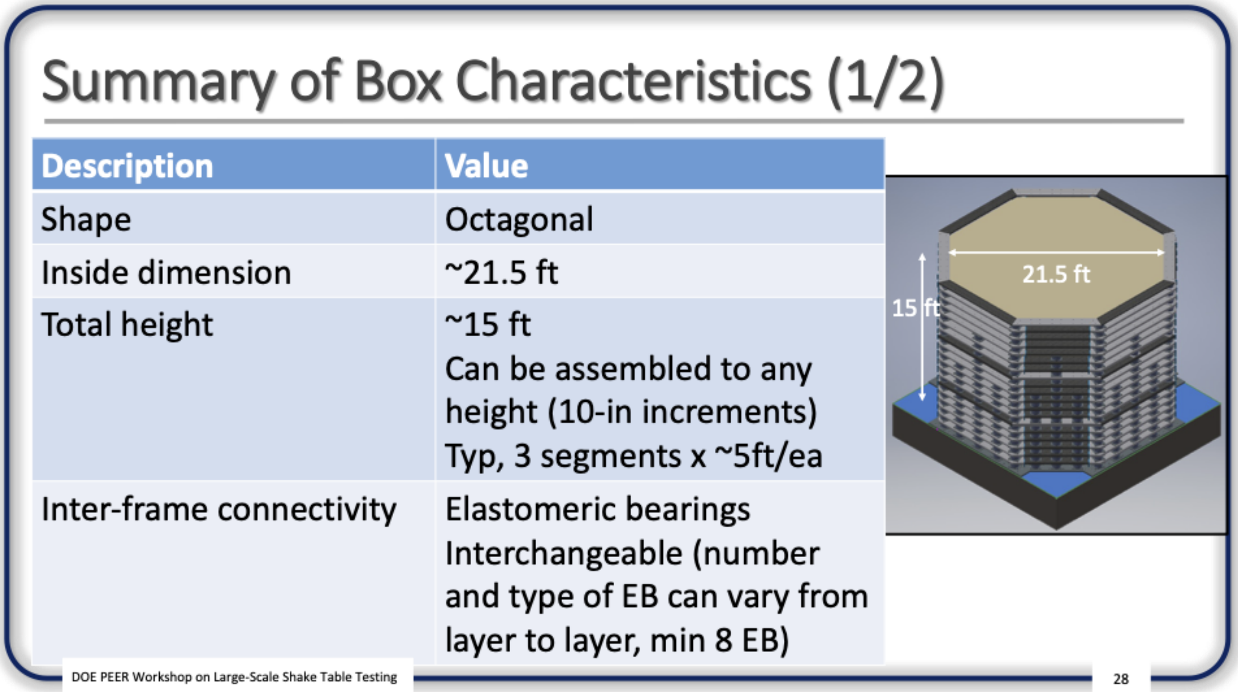
- Base Interface
  - 2D grid of 2x2x1/4 and 3x3x1/4 angles to increase shear transfer



- Sidewall Interface
  - 1/4 inch rubber layer glued onto the faceplates.



Large-Scale Laminar Soil Box

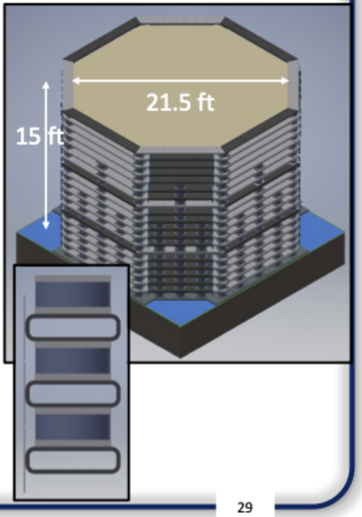


Summary of Box Characteristics (1/2)

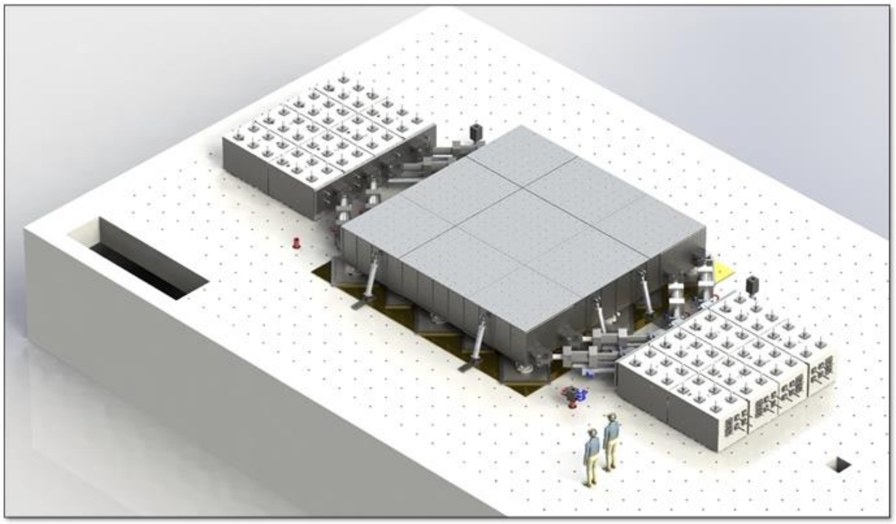
Description	Value
Shape	Octagonal
Inside dimension	~21.5 ft
Total height	~15 ft Can be assembled to any height (10-in increments) Typ, 3 segments x ~5ft/ea
Inter-frame connectivity	Elastomeric bearings Interchangeable (number and type of EB can vary from layer to layer, min 8 EB)

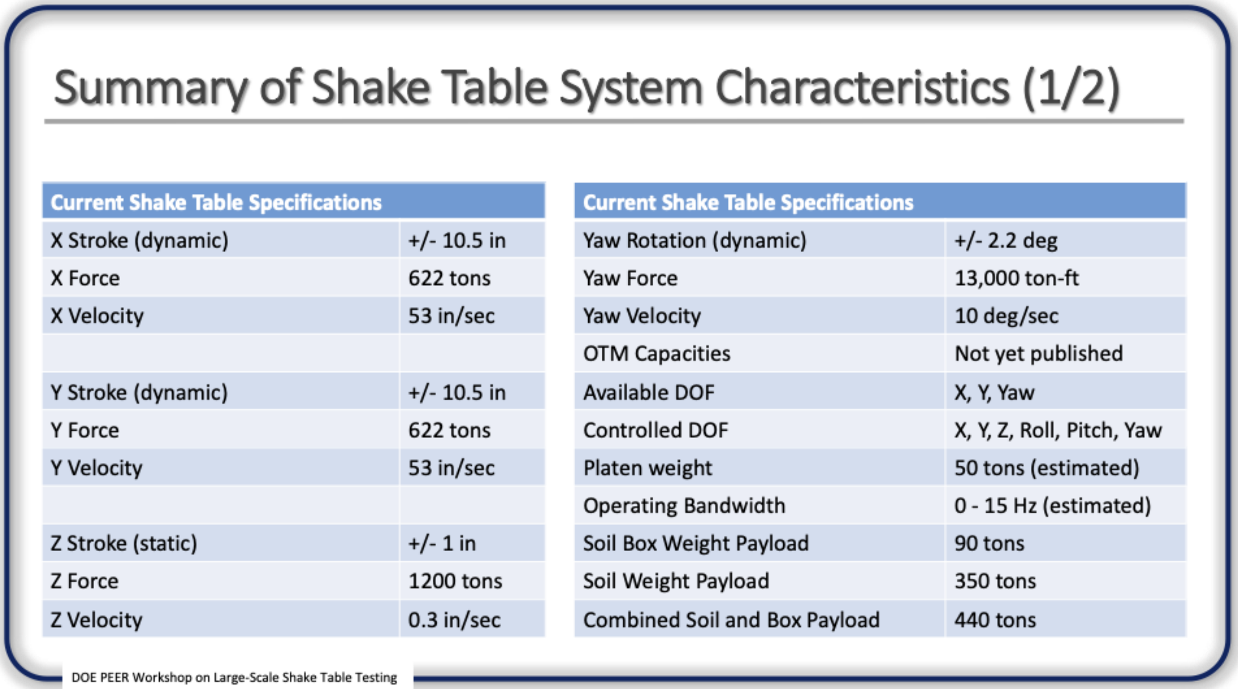
Summary of Box Characteristics (2/2)

Description	Value
Excitation	Biaxial
Fundamental frequency (empty box)	~1.5hz
Width of foundation	~5ft
Max soil strain	15% (physical constraint)
Box weight	68 tons 58 tons (above first EB layer)
Soil weight (γ=120 pcf)	352 tons
Box/Soil	16%



Shake Table System

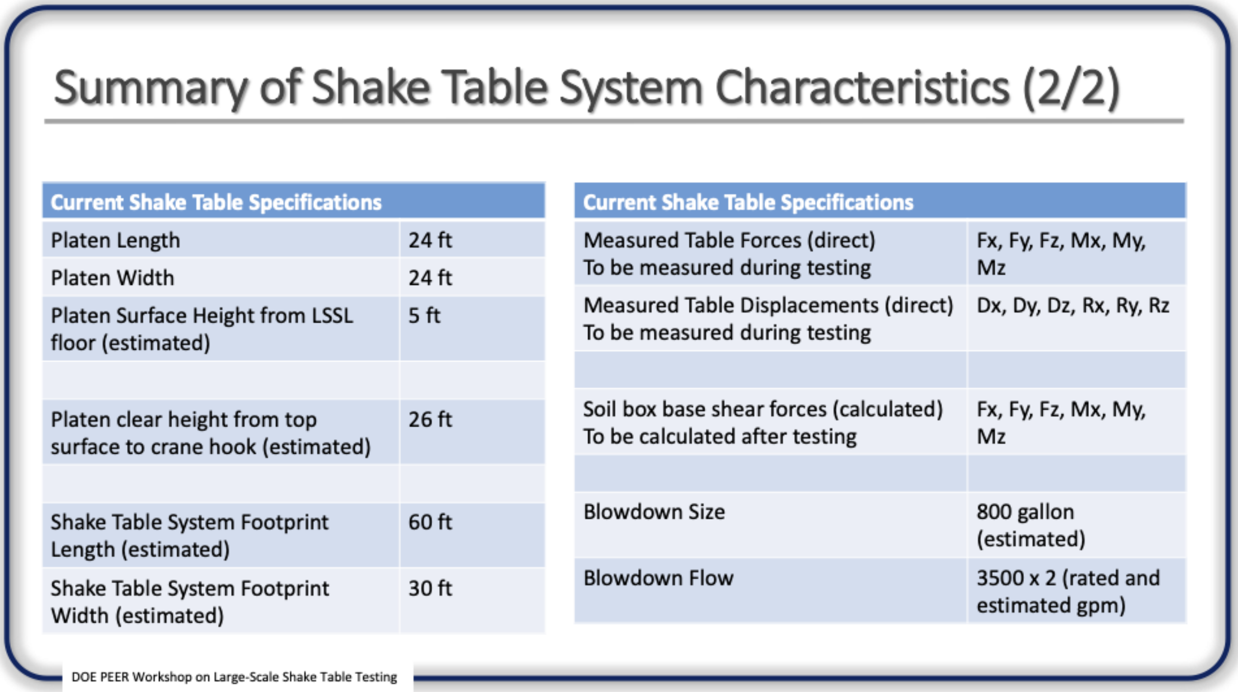




Summary of Shake Table System Characteristics (1/2)

Current Shake Table Specifications	
X Stroke (dynamic)	+/- 10.5 in
X Force	622 tons
X Velocity	53 in/sec
Y Stroke (dynamic)	+/- 10.5 in
Y Force	622 tons
Y Velocity	53 in/sec
Z Stroke (static)	+/- 1 in
Z Force	1200 tons
Z Velocity	0.3 in/sec

Current Shake Table Specifications	
Yaw Rotation (dynamic)	+/- 2.2 deg
Yaw Force	13,000 ton-ft
Yaw Velocity	10 deg/sec
OTM Capacities	Not yet published
Available DOF	X, Y, Yaw
Controlled DOF	X, Y, Z, Roll, Pitch, Yaw
Platen weight	50 tons (estimated)
Operating Bandwidth	0 - 15 Hz (estimated)
Soil Box Weight Payload	90 tons
Soil Weight Payload	350 tons
Combined Soil and Box Payload	440 tons



Summary of Shake Table System Characteristics (2/2)

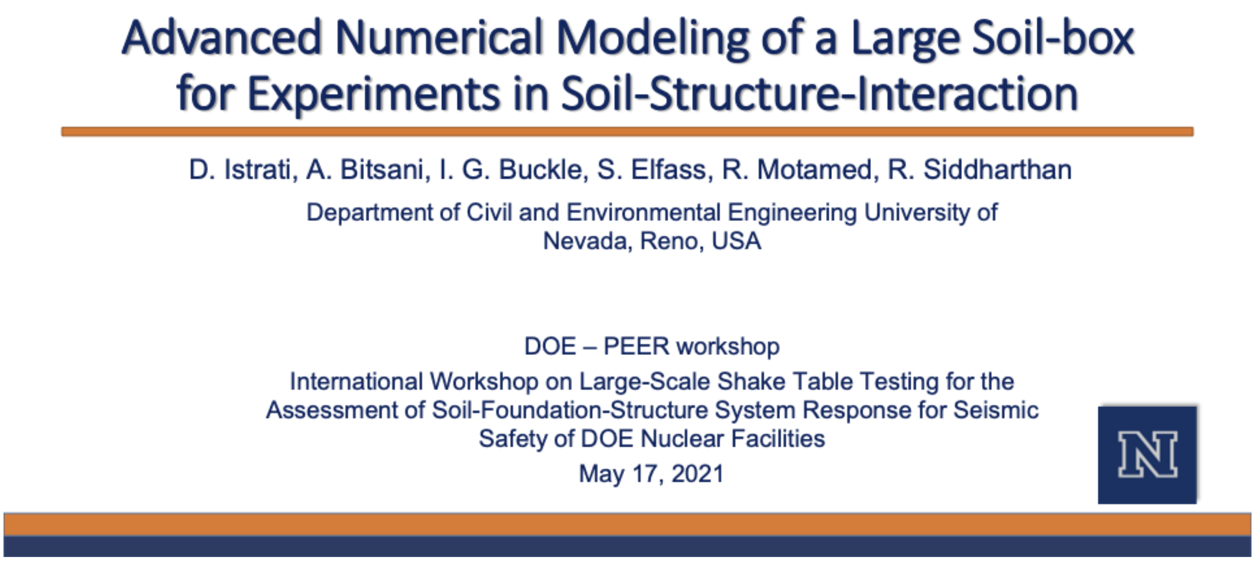
Current Shake Table Specifications	
Platen Length	24 ft
Platen Width	24 ft
Platen Surface Height from LSSL floor (estimated)	5 ft
Platen clear height from top surface to crane hook (estimated)	26 ft
Shake Table System Footprint Length (estimated)	60 ft
Shake Table System Footprint Width (estimated)	30 ft

Current Shake Table Specifications	
Measured Table Forces (direct) To be measured during testing	Fx, Fy, Fz, Mx, My, Mz
Measured Table Displacements (direct) To be measured during testing	Dx, Dy, Dz, Rx, Ry, Rz
Soil box base shear forces (calculated) To be calculated after testing	Fx, Fy, Fz, Mx, My, Mz
Blowdown Size	800 gallon (estimated)
Blowdown Flow	3500 x 2 (rated and estimated gpm)

Thank you!

Questions?

ADVANCED NUMERICAL MODELING OF A LARGE-SCALE SOIL-BOX FOR EXPERIMENTS IN SOIL-STRUCTURE-INTERACTION

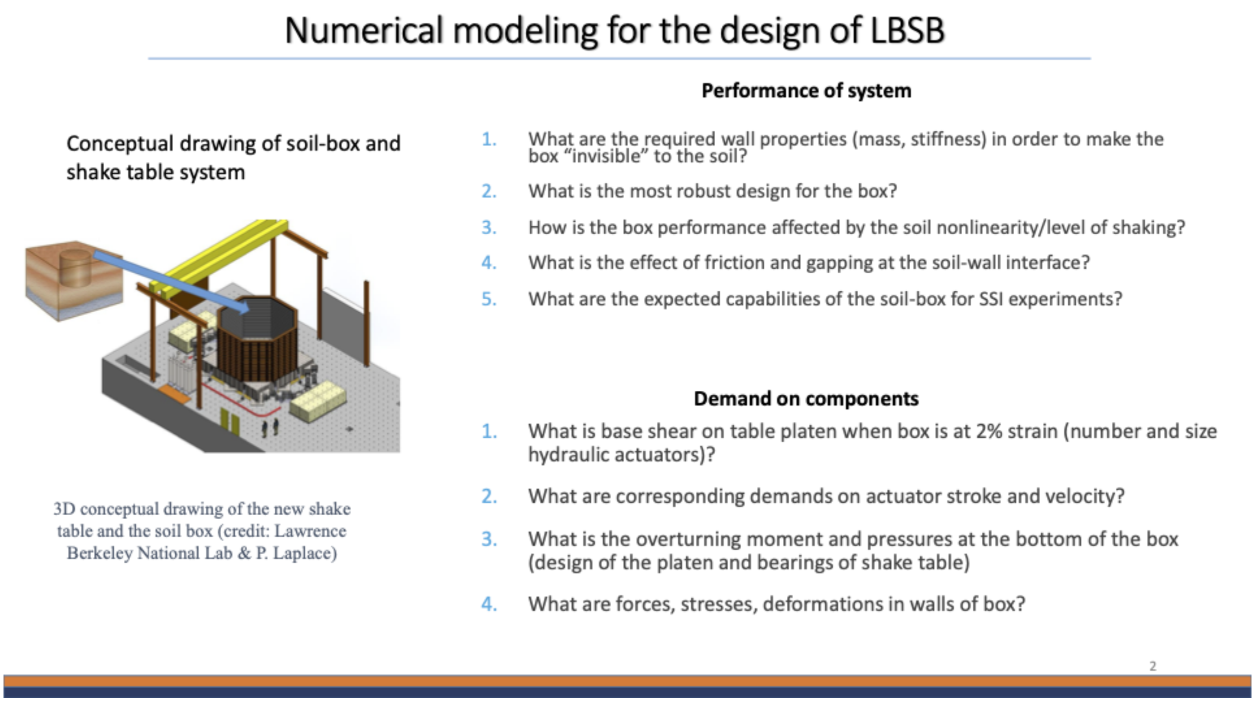


Advanced Numerical Modeling of a Large Soil-box for Experiments in Soil-Structure-Interaction

D. Istrati, A. Bitsani, I. G. Buckle, S. Elfass, R. Motamed, R. Siddharthan

Department of Civil and Environmental Engineering University of
Nevada, Reno, USA

DOE – PEER workshop
International Workshop on Large-Scale Shake Table Testing for the
Assessment of Soil-Foundation-Structure System Response for Seismic
Safety of DOE Nuclear Facilities
May 17, 2021



Numerical modeling for the design of LBSB

Conceptual drawing of soil-box and shake table system

3D conceptual drawing of the new shake table and the soil box (credit: Lawrence Berkeley National Lab & P. Laplace)

Performance of system

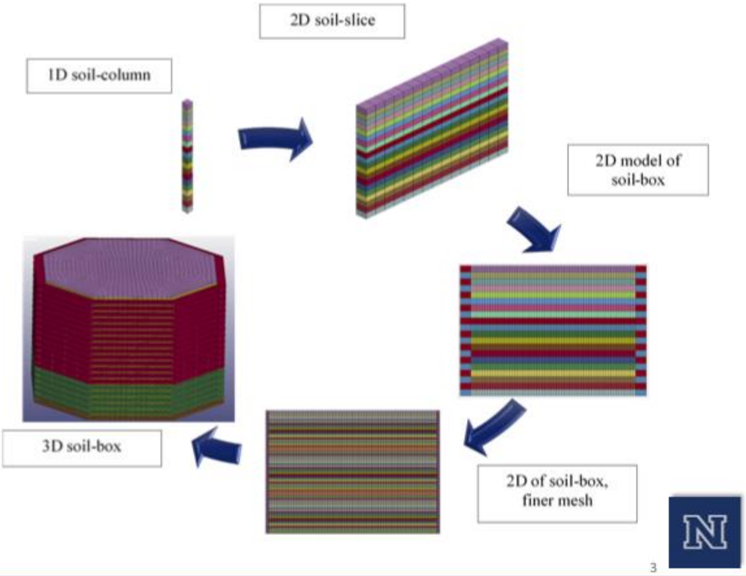
1. What are the required wall properties (mass, stiffness) in order to make the box "invisible" to the soil?
2. What is the most robust design for the box?
3. How is the box performance affected by the soil nonlinearity/level of shaking?
4. What is the effect of friction and gapping at the soil-wall interface?
5. What are the expected capabilities of the soil-box for SSI experiments?

Demand on components

1. What is base shear on table platen when box is at 2% strain (number and size hydraulic actuators)?
2. What are corresponding demands on actuator stroke and velocity?
3. What is the overturning moment and pressures at the bottom of the box (design of the platen and bearings of shake table)
4. What are forces, stresses, deformations in walls of box?

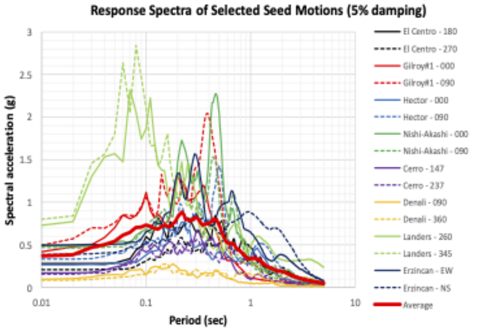
Overview of Numerical Models in Design Phase

- Conduct extensive numerical analyses and generate information that can be used in order to answer the key questions.
- Several models with increasing complexity were developed including:
  A. 1D soil column
  B. 2D soil-slice
  C. 2D slice of box + soil
  D. 3D model of box + soil



Input motions

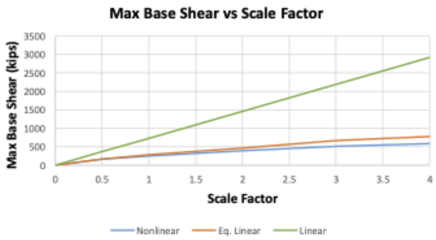
No.	Earthquake	Station	M	Site Vs30 (m/s)	Site Class
1	1940 Imperial Valley-02	El Centro Array #9	6.95	213	D
2	1989 Loma Prieta	Gilroy Array #1	6.9	1428	B
3	1995 Kobe	Nishi-Akashi	6.9	609	C
4	1999 Hector Mine	Hector	7.1	726	C
5	1979 Imperial Valley	Cerro Prieto	6.5	472	C
6	2002 Denali, Alaska	Carlo (temp)	7.9	399	C
7	1992 Landers	Lucerne	7.3	1369	B
8	1992 Erzincan	Erzincan	6.7	352	D

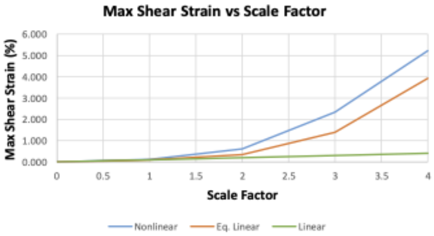

Response Spectra of Selected Seed Motions (5% damping)

✓ Suite of 8, 2-component ground motions taken from PEER database, for sites with **similar seismogenic and geotechnic features** as found at sites of nuclear facilities, and scaled for PGA as follows:

Scale Factor	**1.0**	**2.0**	**3.0**	**4.0**
PGA	0.26g	0.52 g	0.78 g	1.04 g

1D models: Linear vs. Eq. linear vs. Nonlinear dynamic analyses

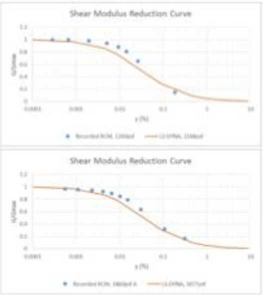


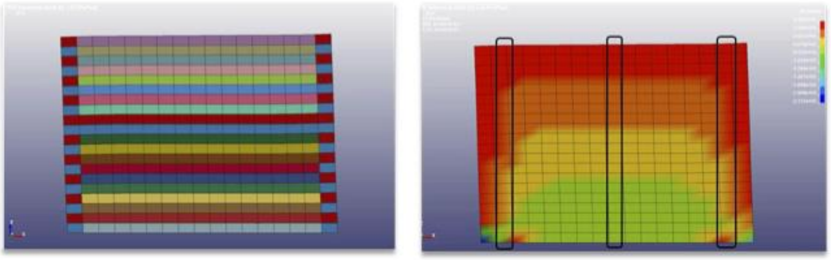


Dense Soil – El Centro 180 – Box 18ftx18ftx20ft						
	Nonlinear		Equivalent Linear		Linear	
Scale Factor	Max Shear Strain (%)	Max Base Shear (kips)	Max Shear Strain (%)	Max Base Shear (kips)	Max Shear Strain (%)	Max Base Shear (kips)
0	0.000	0	0.000	0	0.000	0
0.5	0.047	158	0.039	169	0.051	365
1	0.123	248	0.093	282	0.101	730
2	0.601	395	0.333	468	0.203	1460
3	2.331	513	1.401	671	0.304	2190
4	5.219	585	3.927	776	0.406	2920

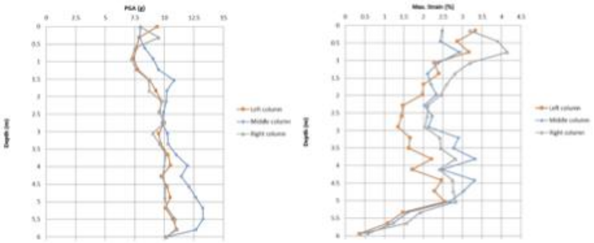
- Linear analyses provide an upper bound for forces and a lower bound for shear strains as expected. The opposite is true for nonlinear analyses.
- Equivalent linear analyses give similar base shears up to SF=2, but still they cannot accurately predict the soil strains. This means that **at large soil strains nonlinear analyses are required**

2D models: Combination of steel and rubber materials



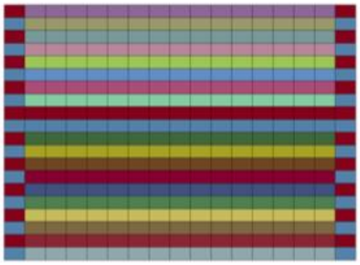


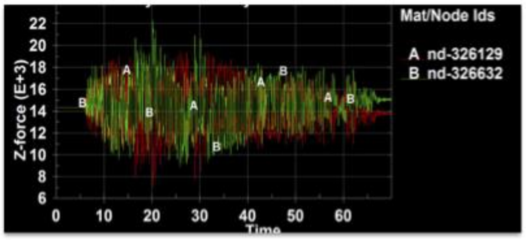
Snapshot of the deformations (left) and the shear stresses (right) of the 2D soil-box model at t=22.4sec

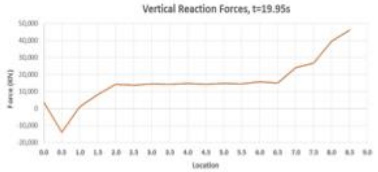


- Soil-columns close to the walls witness different accelerations and strains than the soil-column at the center
- Distorted soil regions close the walls indicates a significant boundary effect

2D models: Combination of steel and rubber materials







- Vertical forces in walls and in soil-columns are out of phase during shaking indicating the generation of overturning moment at the bottom of the box
- Significant complementary shear stresses introduce tension in the walls. Walls need to be designed for that.
- This type of walls **not recommended** because pure shear behavior is limited only to the center half width of the box

2D models: Alternative wall configurations

- ✓ Options to increase the axial and bending stiffness of the walls. Investigate the effect of these stiffnesses.
- ✓ Develop detailed model with nodes at the middle of the walls, where the balls/plugs/bearings will be located

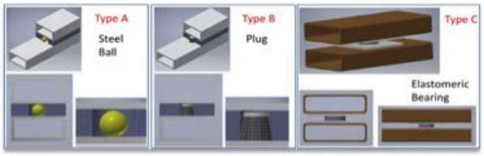
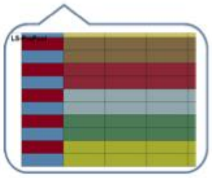
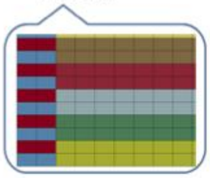


Figure: Three different design alternatives for the walls of the box (credit: S. Elfass)

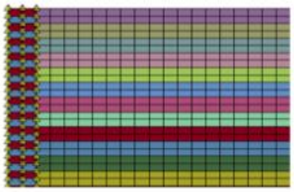
Model 7



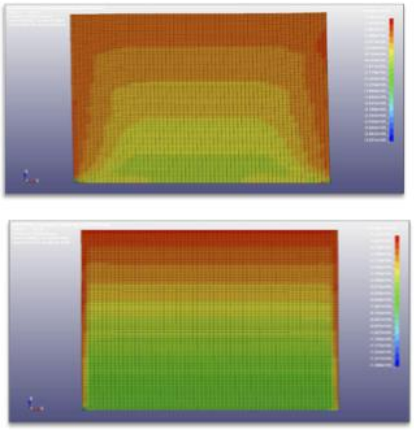
Model 8

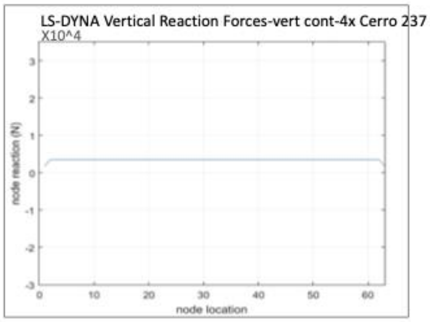


- ✓ Apply vertical constraints or very stiff springs to increase the axial and flexural stiffness



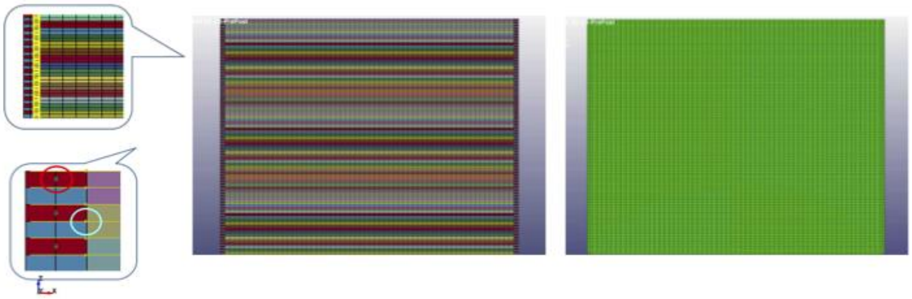
2D models: Comparison of different wall options

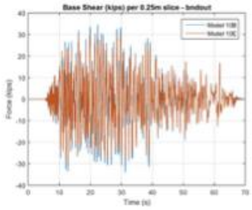


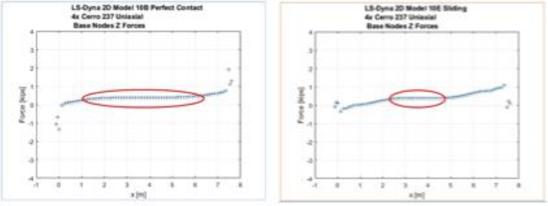


- ✓ Forcing the walls to behave in shear (by zeroing the vertical displacements) has a beneficial effect because it reduces the boundary effect and disturbed soil regions close to the walls leading to more uniform shear stresses along the same soil layer
- ✓ When the axial and flexural stiffness of the walls is very high/infinite, the walls attract/handle the overturning moment, increasing significantly the axial forces (both tension and compression) for which the walls have to be designed.

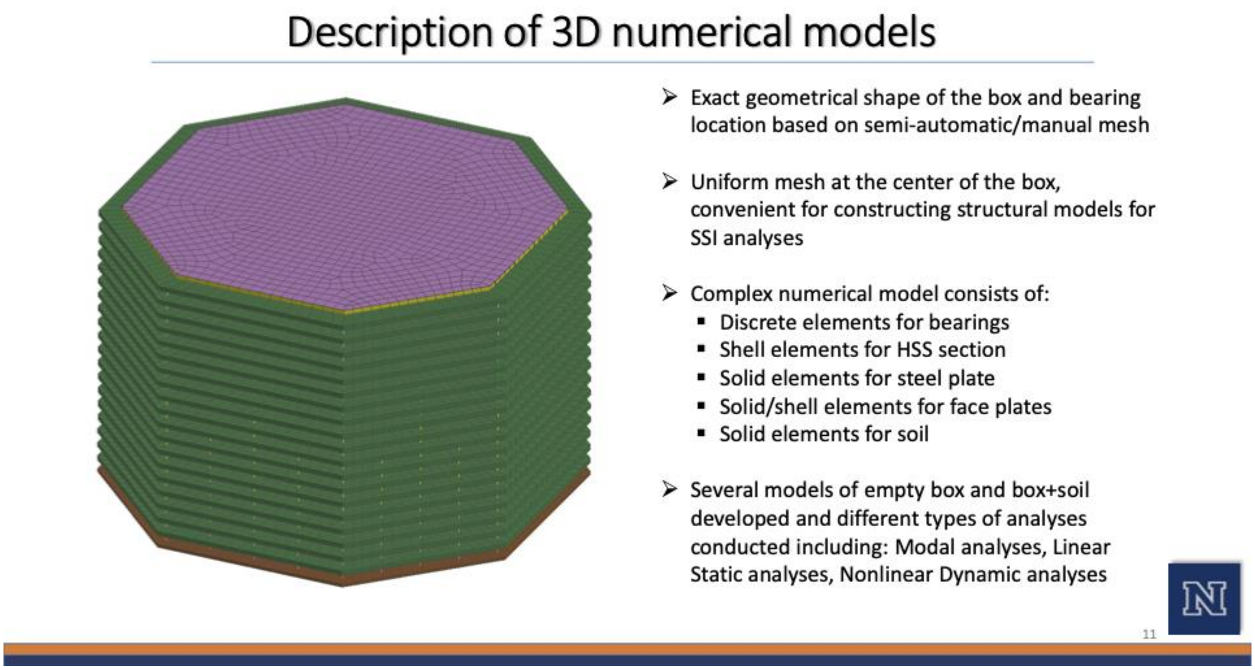
2D models: Role of friction at the soil-wall interface





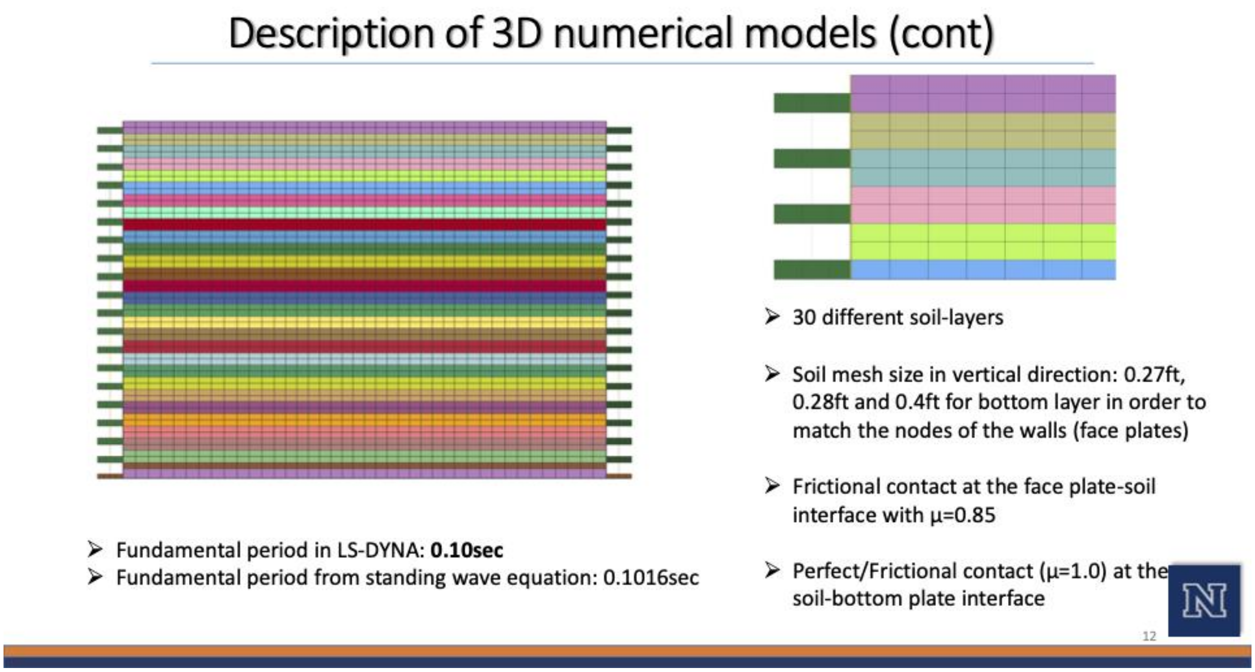


- ✓ When the friction at the soil-wall interface is zero then significant sliding and uplift of the soil is observed close to the soil-wall interface
- ✓ Uplift of soil results in shifting of the center of mass during the shaking and significant boundary effects with distorted soil regions. Soil behavior deviates from pure shear condition.



Description of 3D numerical models

- Exact geometrical shape of the box and bearing location based on semi-automatic/manual mesh
- Uniform mesh at the center of the box, convenient for constructing structural models for SSI analyses
- Complex numerical model consists of:
  - Discrete elements for bearings
  - Shell elements for HSS section
  - Solid elements for steel plate
  - Solid/shell elements for face plates
  - Solid elements for soil
- Several models of empty box and box+soil developed and different types of analyses conducted including: Modal analyses, Linear Static analyses, Nonlinear Dynamic analyses



Description of 3D numerical models (cont)

- 30 different soil-layers
- Soil mesh size in vertical direction: 0.27ft, 0.28ft and 0.4ft for bottom layer in order to match the nodes of the walls (face plates)
- Frictional contact at the face plate-soil interface with μ=0.85
- Perfect/Frictional contact (μ=1.0) at the soil-bottom plate interface

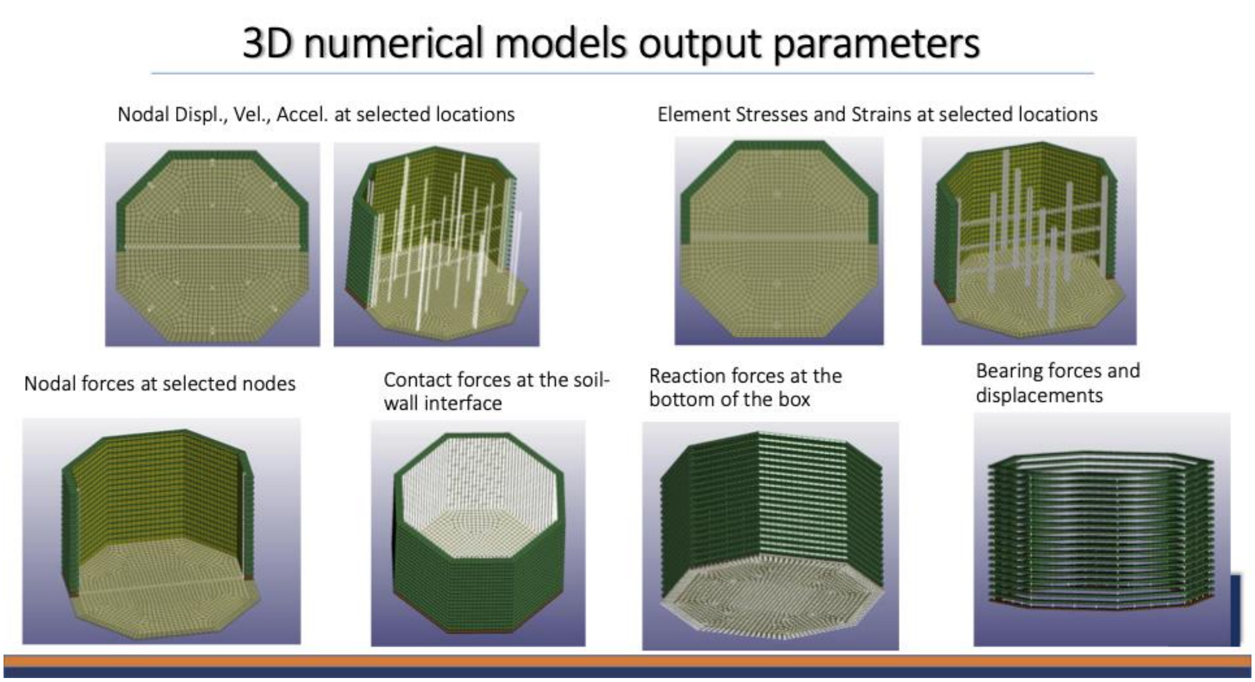
- Fundamental period in LS-DYNA: **0.10sec**
- Fundamental period from standing wave equation: 0.1016sec

3D numerical models of the soil-box

(A) Increase confidence in parameters/values obtained from simplified 1D, 2D and 3D models that were developed during the preliminary design phase

(B) Calculate design parameters that could not be quantified based on previously developed simpler models

(C) Understand the behavior of the box and the expected ground motion at the soil surface to assist the design of SSI experiments



3D numerical models output parameters

Nodal Displ., Vel., Accel. at selected locations

Element Stresses and Strains at selected locations

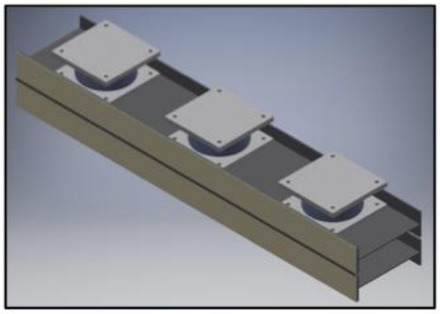
Nodal forces at selected nodes

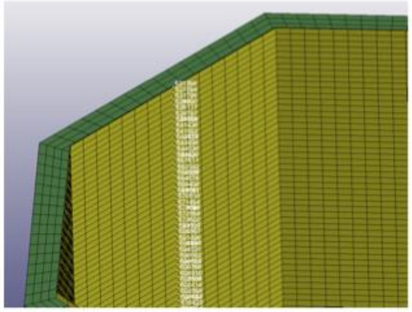
Contact forces at the soil-wall interface

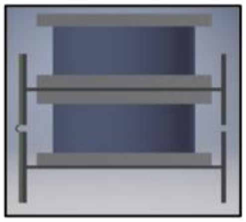
Reaction forces at the bottom of the box

Bearing forces and displacements

Vertical Gap of Face Plates

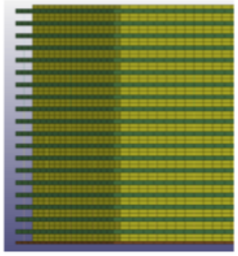


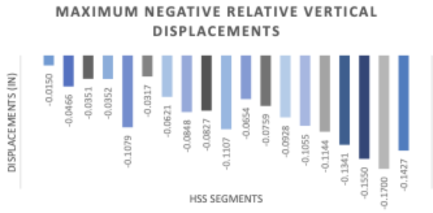


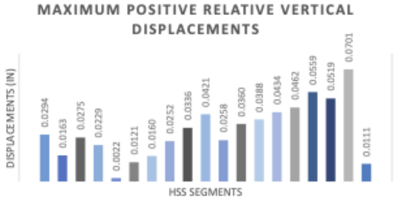


- Given the absolute z-displacements of coinciding nodes and the identification of the nodes it is possible to calculate the opening/closing of the gap. Δz=ztop,i-zbottom,i
- If Δz>0, opening of gap occurs (opposite is true for closing of gap)

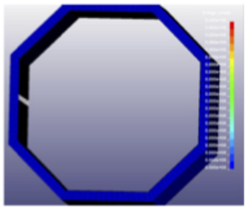
Vertical Gap of Face Plates & Stresses in Walls

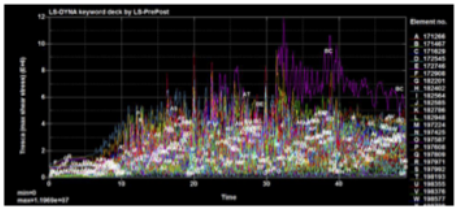






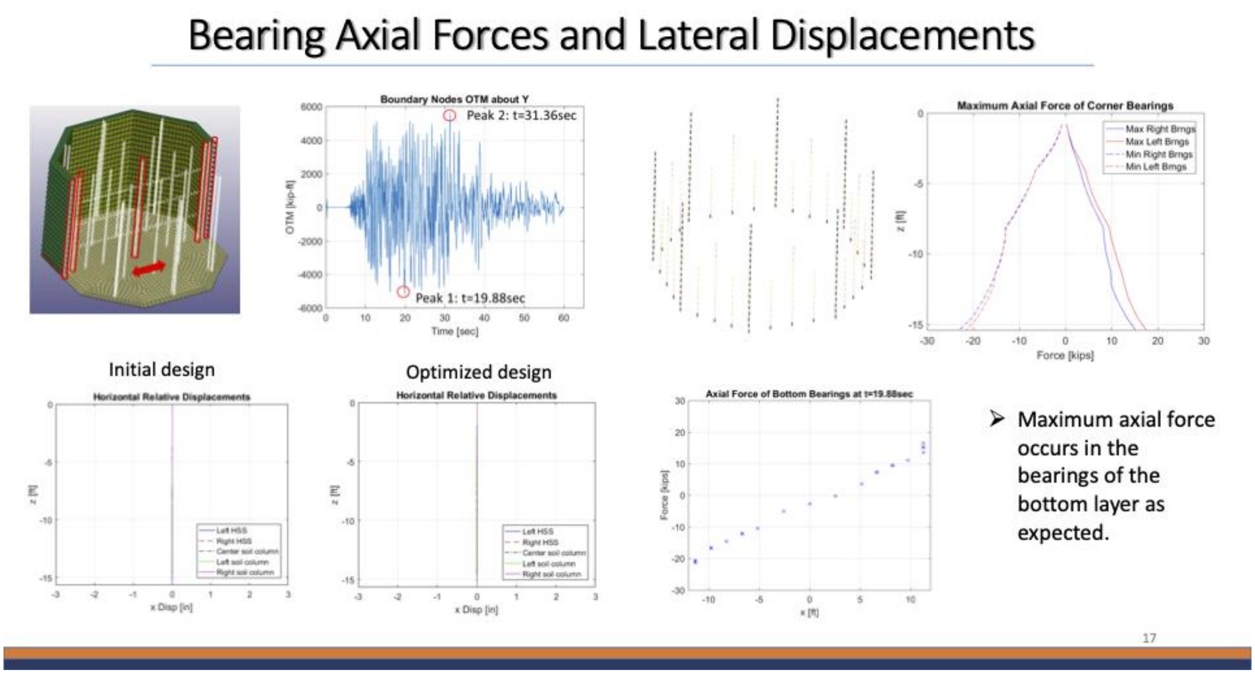
Nonlinear Dynamic
Analyses: Cerro SF4 - biaxial





Face plates: A36 ,
fy=36ksi (tensile)
fxy=fy/√3=20.8ksi (pure shear)

- Max Shear Stress =11969kPa =1.74ksi < fxy=20.8ksi
- The demand on the face plates is approximately 10% of the estimated capacity

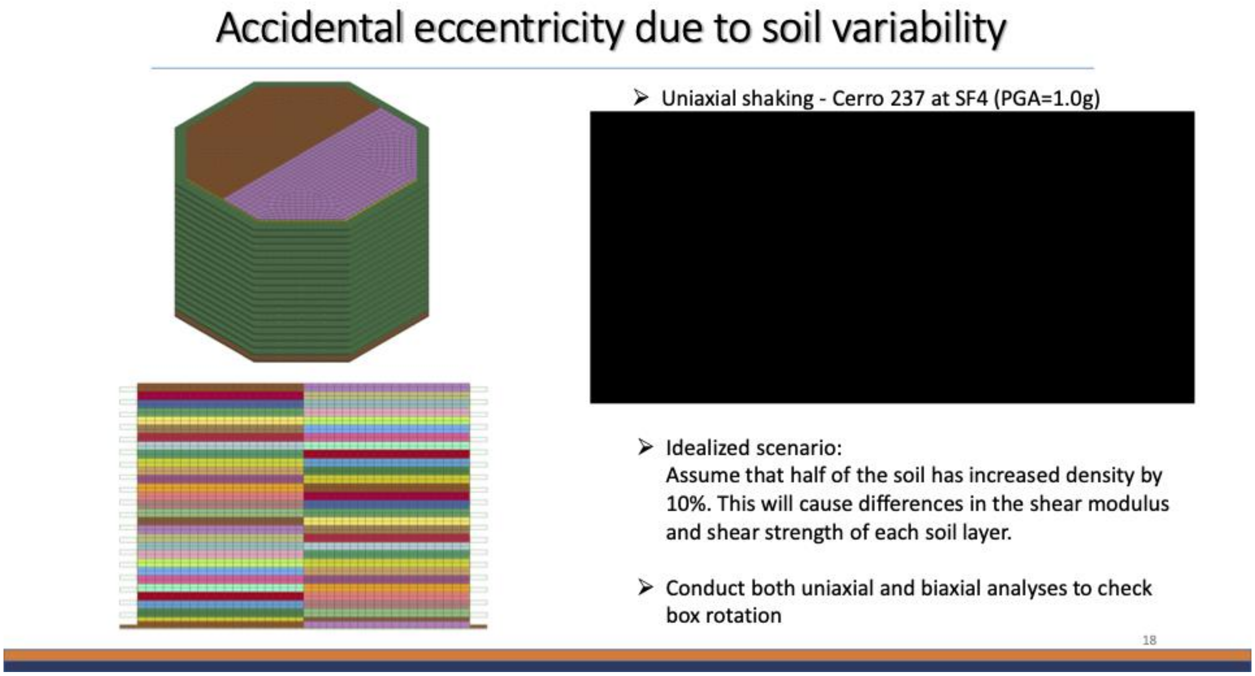


Bearing Axial Forces and Lateral Displacements

Initial design

Optimized design

- ➢ Maximum axial force occurs in the bearings of the bottom layer as expected.

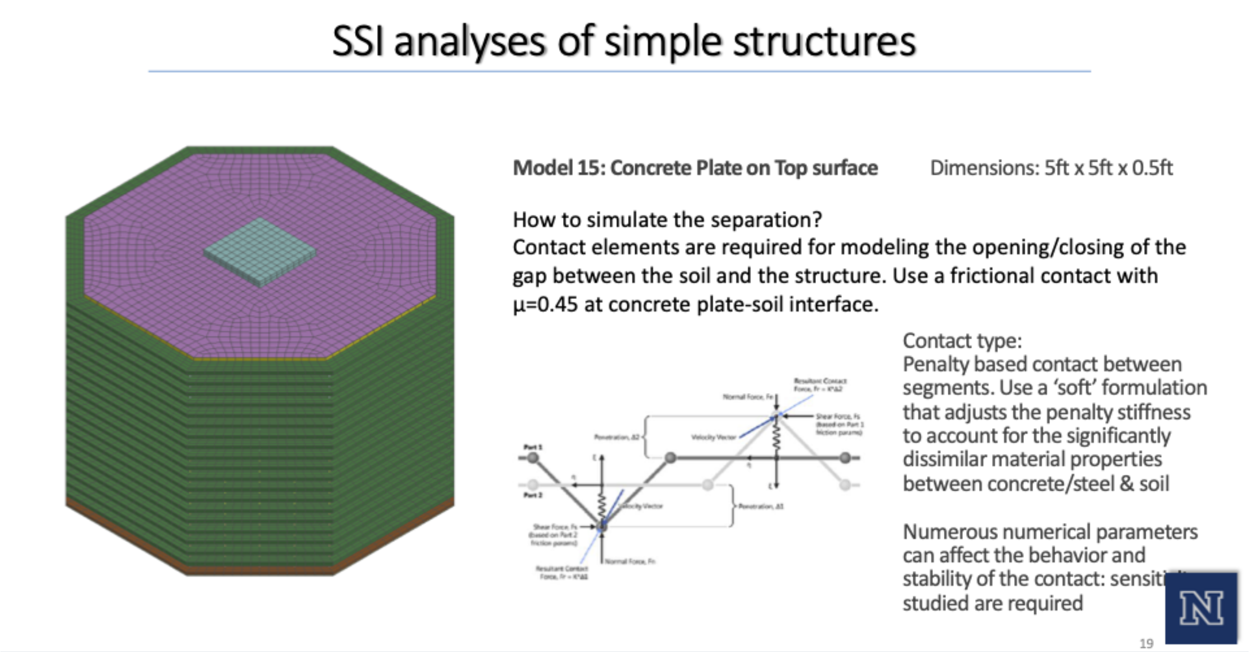


Accidental eccentricity due to soil variability

- ➢ Uniaxial shaking - Cerro 237 at SF4 (PGA=1.0g)

- ➢ Idealized scenario:
  Assume that half of the soil has increased density by 10%. This will cause differences in the shear modulus and shear strength of each soil layer.

- ➢ Conduct both uniaxial and biaxial analyses to check box rotation



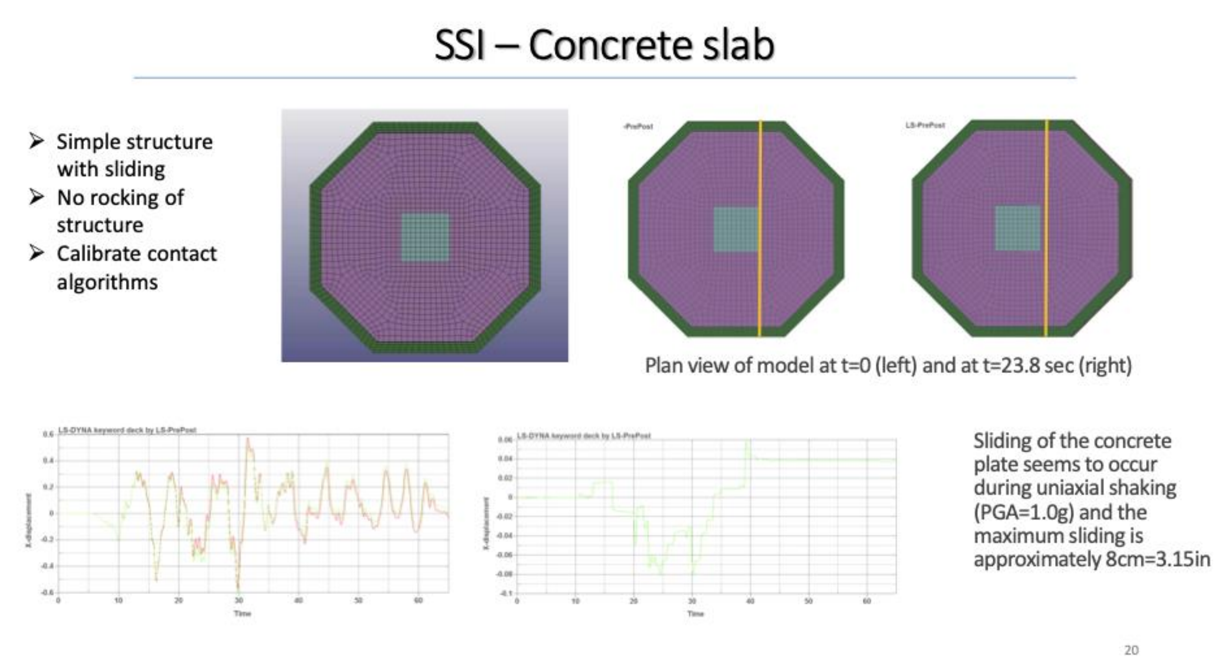
SSI analyses of simple structures

Model 15: Concrete Plate on Top surface Dimensions: 5ft x 5ft x 0.5ft

How to simulate the separation?
Contact elements are required for modeling the opening/closing of the gap between the soil and the structure. Use a frictional contact with μ=0.45 at concrete plate-soil interface.

Contact type:
Penalty based contact between segments. Use a 'soft' formulation that adjusts the penalty stiffness to account for the significantly dissimilar material properties between concrete/steel & soil

Numerous numerical parameters can affect the behavior and stability of the contact: sensiti
studied are required



SSI – Concrete slab

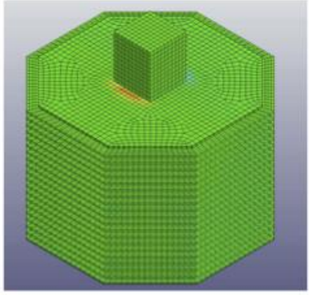
- Simple structure with sliding
- No rocking of structure
- Calibrate contact algorithms

Plan view of model at t=0 (left) and at t=23.8 sec (right)

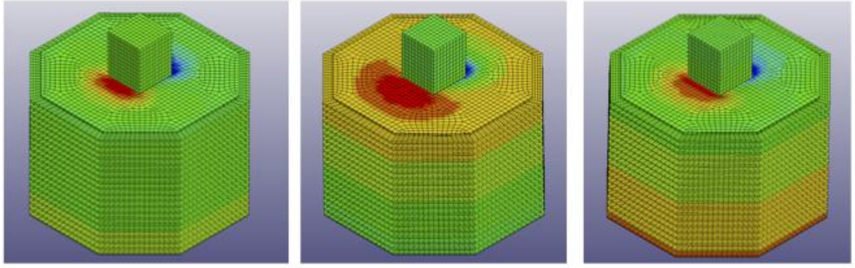
Sliding of the concrete plate seems to occur during uniaxial shaking (PGA=1.0g) and the maximum sliding is approximately 8cm=3.15in

SSI – Concrete block

No embedment in soil



No embedment in soil (at three instants during shaking)



- ➢ Simple structure (hollow box) with rocking
- ➢ Concentrated nonlinear soil behavior around the structure
- ➢ Physics more complicated due (a) soil localized non-linearity, and (b) gap opening between the soil and the structure

Summary: 2D and 3D Models

Investigated in **2D** models:

- Effect of sliding
- Effect of friction and gapping
- Sensitivity of results to the contact type
- Effect of bottom plate
- Sensitivity of results to the numerical approach of transferring shear between the bottom plate and the soil
- Effect of friction between the soil & bottom plate
- Sensitivity of results to the ground motion
- Sensitivity of results to the element formulation

Investigated in **3D** models:

- Effect of friction and gapping – uniaxial and biaxial motion
- Effect of bottom plate – uniaxial motion
- Effect of soil accidental eccentricity
- Soil-structure interaction capabilities
- Sensitivity of results to the in-plane mesh
- Sensitivity of results to the magnitude of ground motion
- Sensitivity of results to the ground motion

Some conclusions

1. Laminar walls that are flexible in every direction are witnessing vertical soil displacements in regions close to the walls, indicating that the soil is not in pure shear and demonstrating the existence of **a significant boundary effect** caused by the walls.

2. Large overturning moment is generated at the bottom of the soil-box during extreme ground shaking. **OTM can introduce significant uplift** in the walls via the complementary shears. Walls should be designed for both shear & tension.

3. To ensure that the soil-box will behave as realistically as possible, it is necessary to have walls **with small lateral stiffness but very high axial and bending stiffness,** together with **a nearly perfect contact (high-coefficient of friction) at the soil-wall interface**, which will transfer the complementary shear of the soils to the walls and minimize the boundary effect.

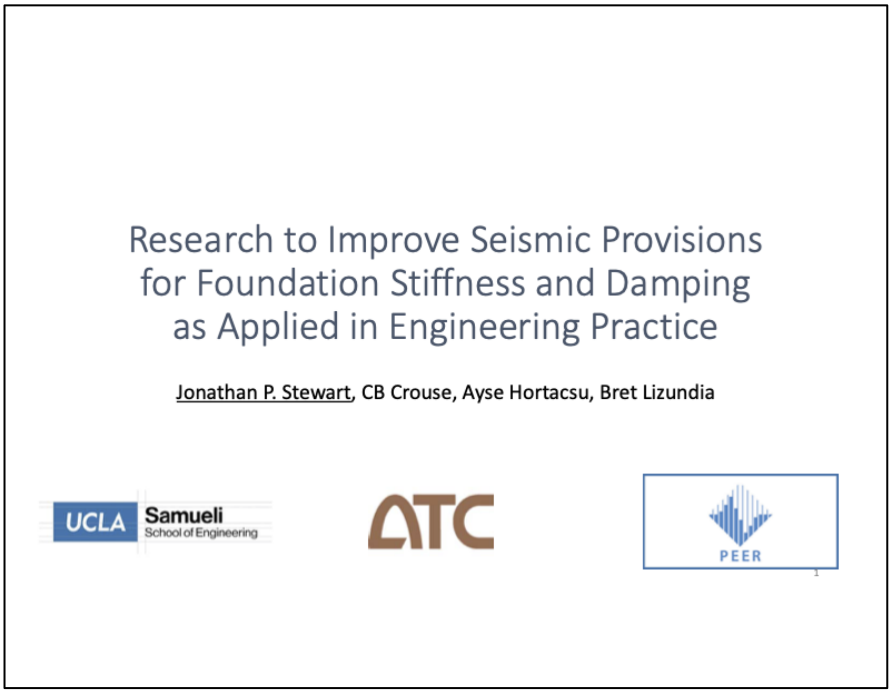
4. 1D numerical models are efficient and insightful during the preliminary design phase of a soil-box. However, more **advanced 2D and 3D models** are (i) understanding the soil-wall interaction, (ii) providing all the parameters for the final design, (iii) quantifying the soil-structure interaction capabilities.

Acknowledgments

This work has been sponsored by the Department of Energy, as part of a multi-institutional research project with a subcontract number 7236255, led by LBNL. The support of David McCallen and Boris Jeremic is gratefully acknowledged.

Thank you!

RESEARCH TO IMPROVE SEISMIC PROVISIONS FOR FOUNDATION STIFFNESS AND DAMPING AS APPLIED IN ENGINEERING PRACTICE



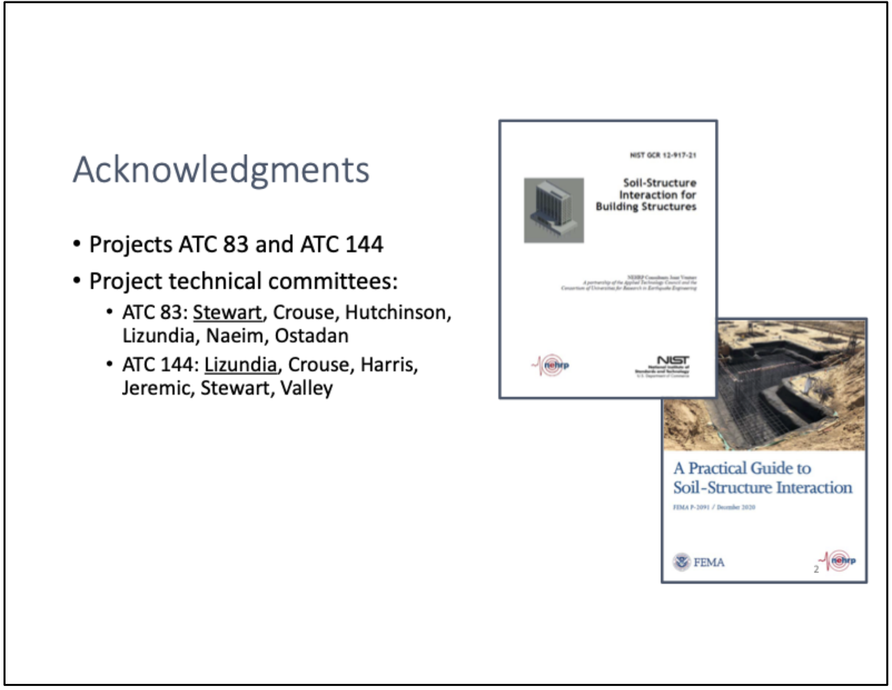
Research to Improve Seismic Provisions for Foundation Stiffness and Damping as Applied in Engineering Practice

Jonathan P. Stewart, CB Crouse, Ayse Hortacsu, Bret Lizundia

UCLA Samueli School of Engineering

ATC

PEER



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- Projects ATC 83 and ATC 144
- Project technical committees:
  - ATC 83: Stewart, Crouse, Hutchinson, Lizundia, Naeim, Ostadan
  - ATC 144: Lizundia, Crouse, Harris, Jeremic, Stewart, Valley

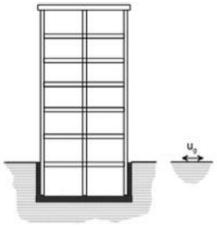
Outline

- Substructure and direct approaches for SSI analysis
- Procedures in codes / guidelines documents for buildings
- Importance of testing

Substructure and Direct Approaches

Substructure Approach

- Foundation input motions

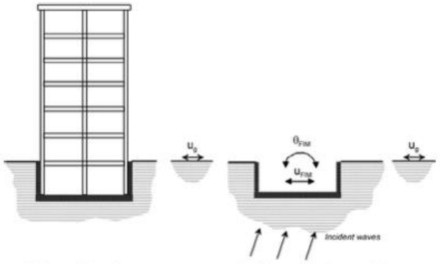


(a) Complete System

NIST, 2012

Substructure Approach

- Foundation input motions

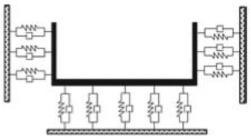


(a) Complete System (b) Kinematic Interaction

NIST, 2012

Substructure Approach

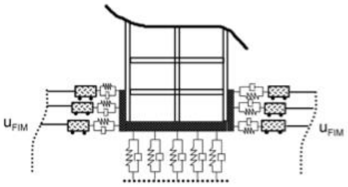
- Foundation input motions
- Foundation-soil interaction elements



NIST, 2012

Substructure Approach

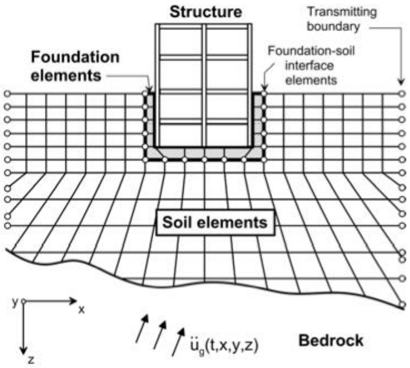
- Foundation input motions
- Foundation-soil interaction elements
- Analyze response



NIST, 2012

Direct Approach

- Avoids limiting assumptions from substructure approach
- Application of incoherent input is challenging



NIST, 2012

Procedures in Codes / Guidelines Documents for Buildings

Applications for Buildings

- Force-Based Methods (NEHRP, ASCE-7)
- Displacement-Based Methods (FEMA 440, ASCE-41)
- Response History Analyses (NIST 2012, TBI 2017)

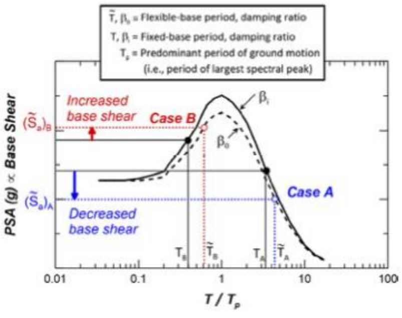
Applications for Buildings

- Force-Based Methods (NEHRP, ASCE-7)
- Displacement-Based Methods (FEMA 440, ASCE-41)
- Response History Analyses (NIST 2012, TBI 2017)

Force-Based Methods

Base shear adjusted. Based on

- Period lengthening, $\tilde{T}/T$
- Change in system damping, $\beta_i \rightarrow \beta_0$

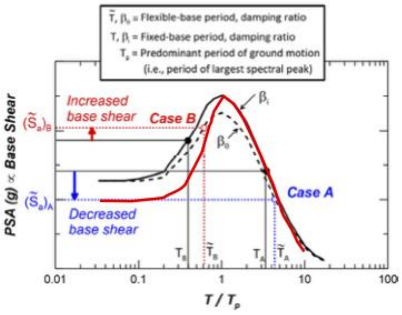


NIST, 2012

Force-Based Methods

Base shear adjusted

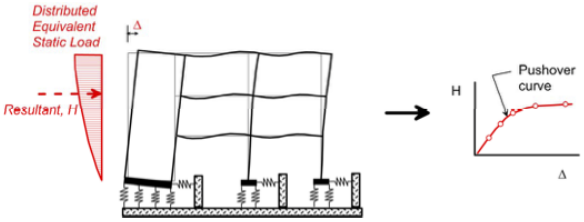
Kinematic adjustment to spectra could, in principle, be applied



NIST, 2012

Displacement-Based Methods

Pushover analysis to evaluate capacity diagram

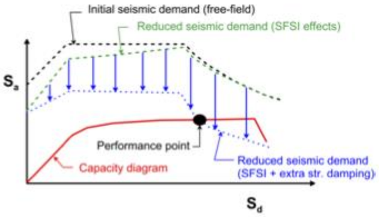


NIST, 2012

Displacement-Based Methods

Pushover analysis to evaluate capacity diagram

Combine with demand to evaluate displacement

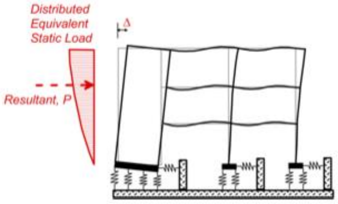


NIST, 2012

Displacement-Based Methods

SSI impact through:

- Springs in pushover analysis

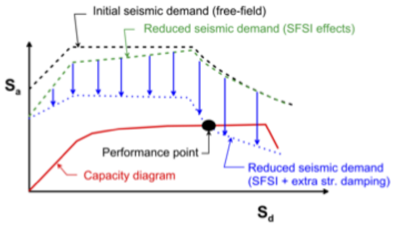


NIST, 2012

Displacement-Based Methods

SSI impact through:

- Springs in pushover analysis
- Kinematic reduction of demand

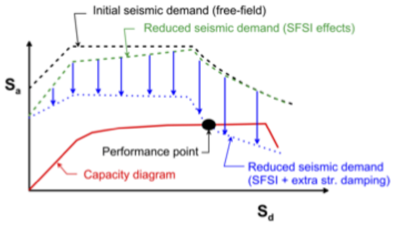


NIST, 2012

Displacement-Based Methods

SSI impact through:

- Springs in pushover analysis
- Kinematic reduction of demand
- Additional system damping



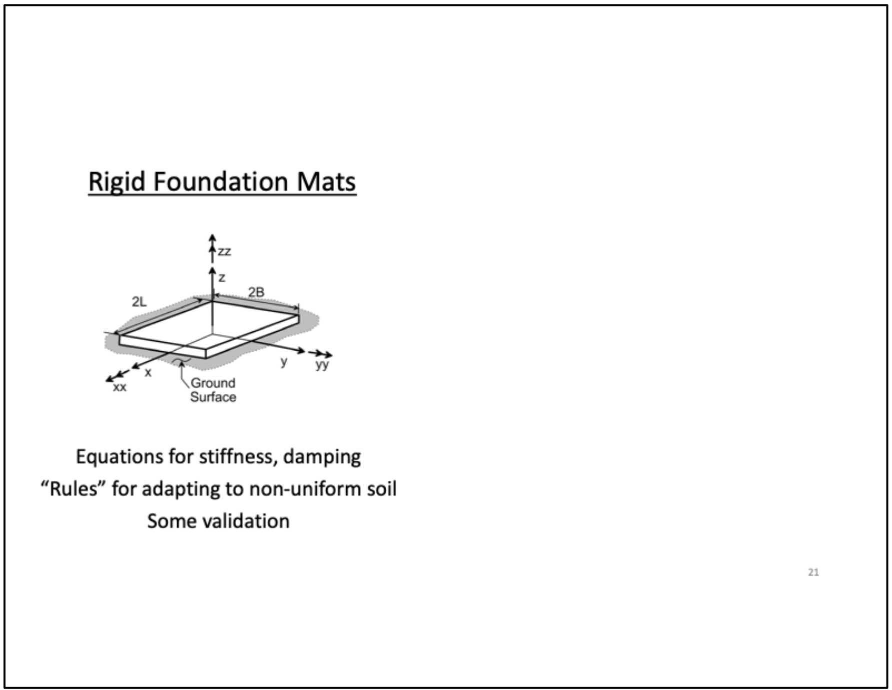
NIST, 2012

SSI Modeling Requirements

Force and displacement methods both require:

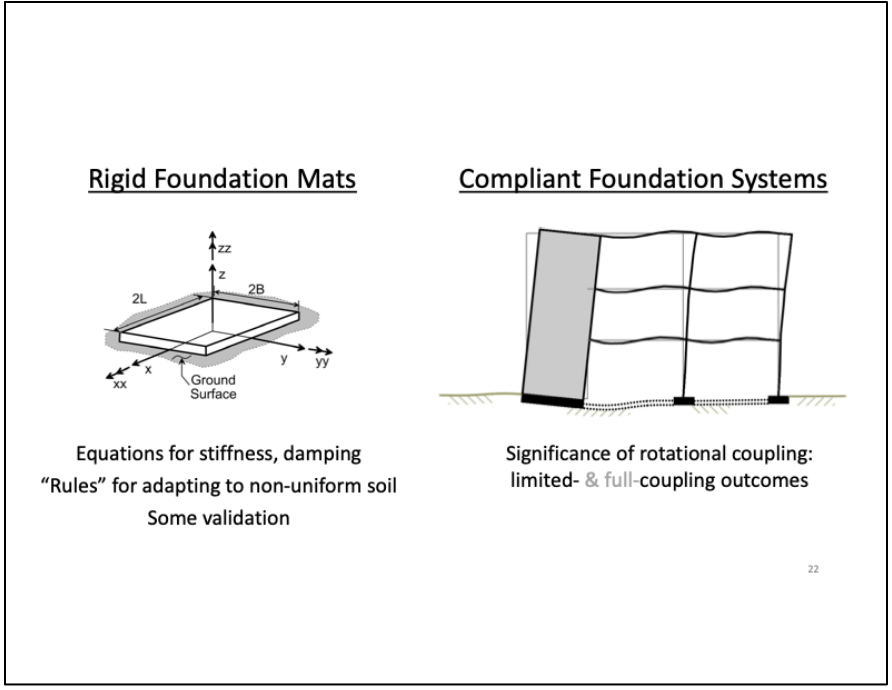
- Lateral and rotational stiffness of foundation
- Foundation damping, β_f:
  - $\beta_0 = \beta_f + \frac{\beta_i}{(\tilde{T}/T)^2}$
  - β_f = f[Radiation damping (β_x, β_{yy}) and material damping, D]

Foundation translation ***Foundation rotation***



Rigid Foundation Mats

Equations for stiffness, damping
"Rules" for adapting to non-uniform soil
Some validation



Rigid Foundation Mats

Equations for stiffness, damping
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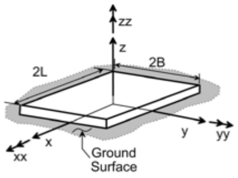
Compliant Foundation Systems

Significance of rotational coupling:
limited- & full-coupling outcomes

Rigid Foundation Mats



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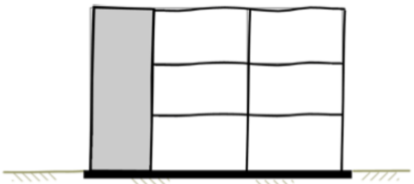
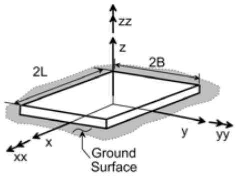
Compliant Foundation Systems



Significance of rotational coupling:
limited- & full-coupling outcomes

Rigid Foundation Mats



Equations for stiffness, damping
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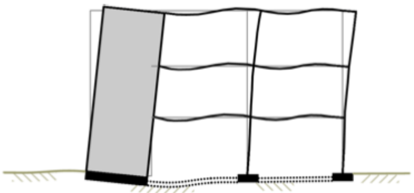
Compliant Foundation Systems



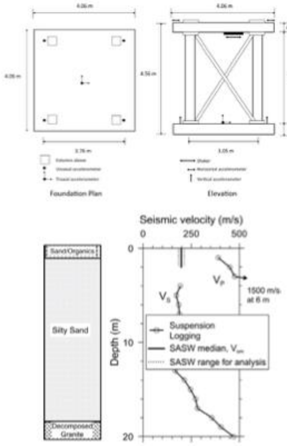
Significance of rotational coupling:
limited- & full-coupling outcomes
What are the effective foundation
dimensions for stiffness, damping,
and base slab averaging?

Importance of Testing

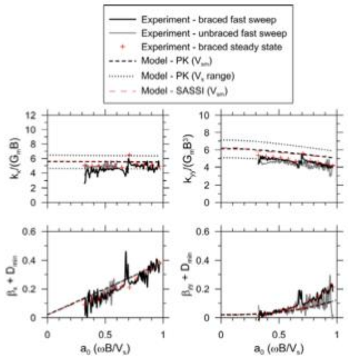
Testing

Example insights

Research opportunities using UNR-DOE Large-Scale Shake Table



NEES/UCSB Garner Valley Test Structure



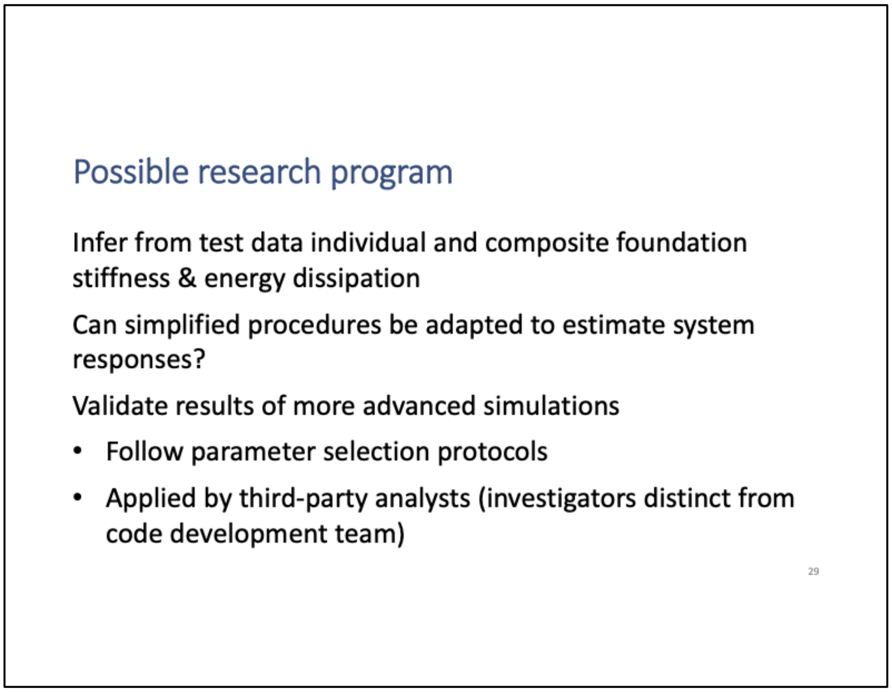
Tileylioglu et al. (2011)

Possible research program

Form advisory panel

Construct foundation systems with varying degrees of coupling (consistent superstructure)

Instrument structure/foundations to evaluate load distributions and foundation responses



Possible research program

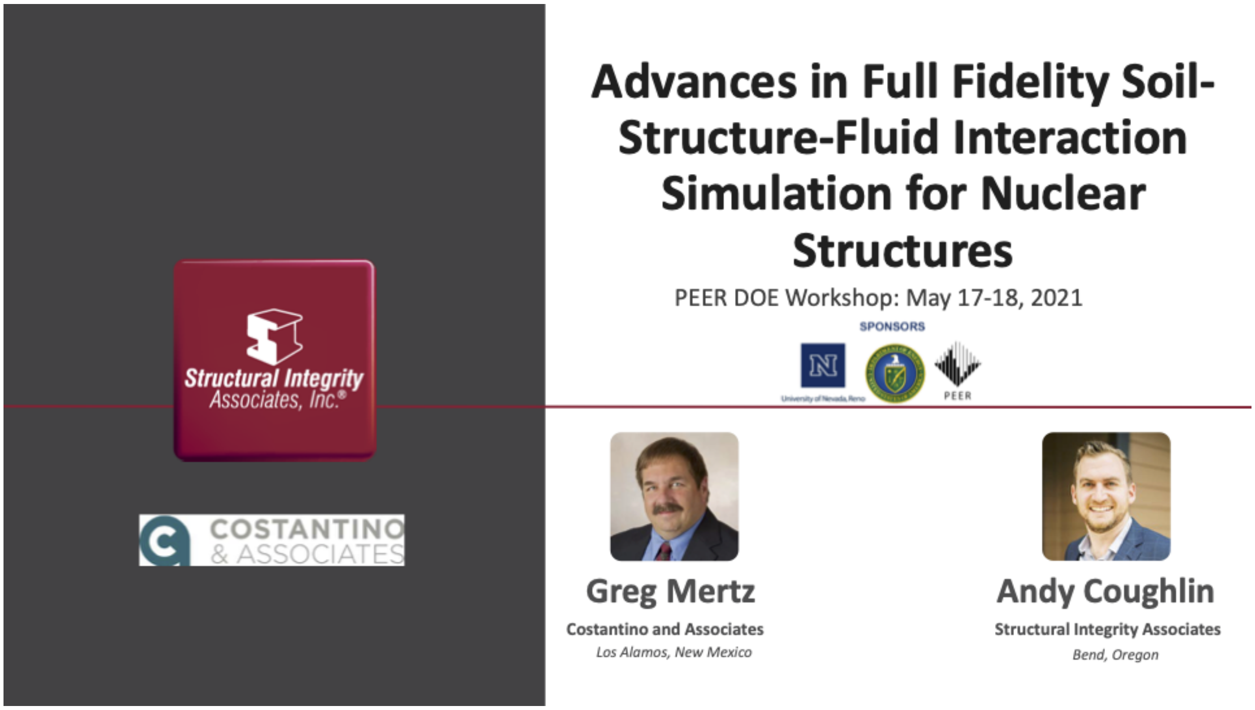
Infer from test data individual and composite foundation stiffness & energy dissipation

Can simplified procedures be adapted to estimate system responses?

Validate results of more advanced simulations

- Follow parameter selection protocols
- Applied by third-party analysts (investigators distinct from code development team)

ADVANCES IN FULL FIDELITY SOIL-STRUCTURE-FLUID INTERACTION SIMULATION FOR NUCLEAR STRUCTURES



Advances in Full Fidelity Soil-Structure-Fluid Interaction Simulation for Nuclear Structures

PEER DOE Workshop: May 17-18, 2021

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Andy Coughlin

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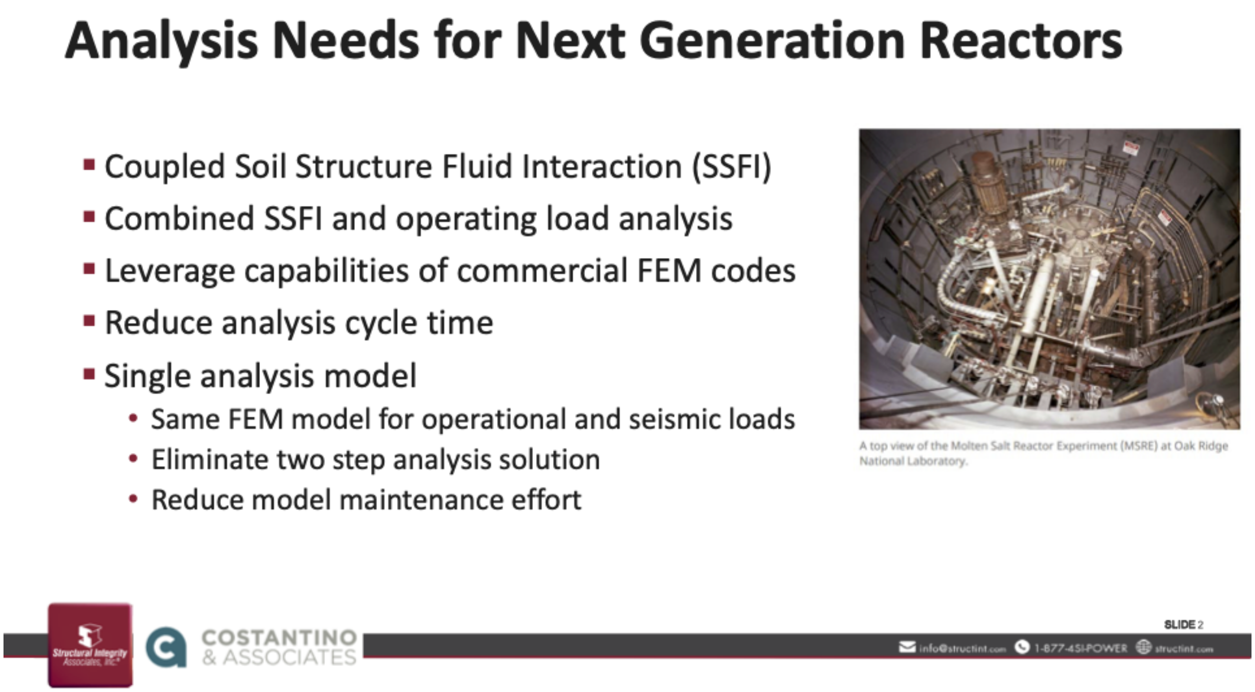

Analysis Needs for Next Generation Reactors

- Coupled Soil Structure Fluid Interaction (SSFI)
- Combined SSFI and operating load analysis
- Leverage capabilities of commercial FEM codes
- Reduce analysis cycle time
- Single analysis model
  - Same FEM model for operational and seismic loads
  - Eliminate two step analysis solution
  - Reduce model maintenance effort

A top view of the Molten Salt Reactor Experiment (MSRE) at Oak Ridge National Laboratory.

SLIDE 2

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Analysis Needs for Next Generation Reactors

- Equivalent Linear vs Nonlinear Analysis
  - Should we design for significant inelastic deformation during a structures life?
  - Operating basis should remain elastic
  - Safe shutdown may have limited inelastic deformation
    - ASCE 43 Limit State C
    - Most designs are targeting elastic response
      - Regulatory precedent
  - Nonlinear analysis is valuable for beyond (original) design basis events
    - Seismic demand 25 years after construction

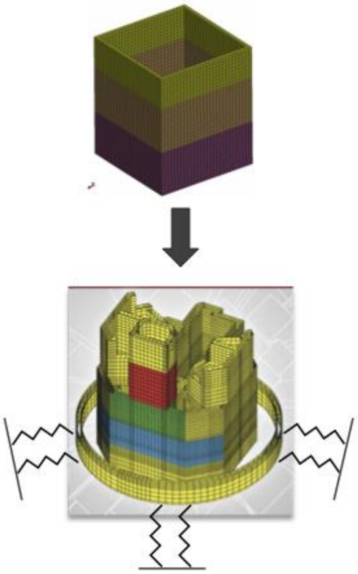
Existing Analysis Tools

- SASSI
  - Very good for developing frequency domain soil impedance and load vectors
  - Lacks many state-of-the-art analysis features
  - Narrow userbase
- Commercial FEM
  - Excellent element libraries
    - Ability to perform fluid-structure interaction with acoustic elements
  - Excellent constraints, etc. for model development
  - Graphical pre and post processing
  - Wide userbase

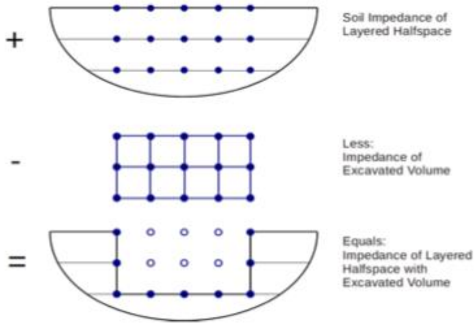
Buildings designed using SASSI and commercial FEM

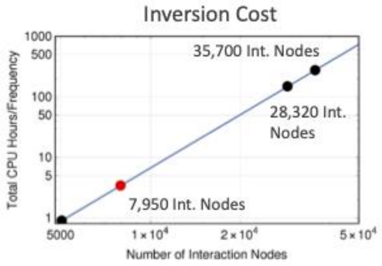
Proposed Solution

- Combine the best of SASSI and Commercial FEM
  - Use SASSI to develop soil impedances and load vectors for a given excavation and site profile
    - Store in a soil library
  - Use a commercial FEM code to
    - Develop building model
    - Generate operational demands
    - Generate seismic demands in the frequency domain
      - Post processing to convert frequency domain to time domain
    - Combine operational and seismic demands
- Functionally equivalent to a SASSI solution



SASSI Soil Impedance

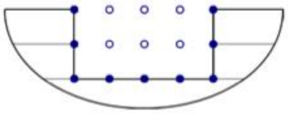




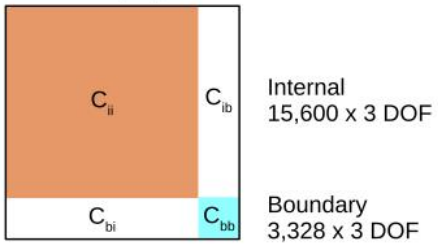
*Use SASSI House bricks with mixed lumped and consistent mass

Impedance Matrix Storage

- Full Storage can be large
- Condense out internal DOF to reduce storage



Case 2, 18,928 Interaction Nodes



	Case 1		Case 2		Case 3	
	Full	Reduced	Full	Reduced	Full	Reduced
Nodes	5,168	1,360	18,928	3,328	35,280	5,085
File Size GiB	1.7	0.12	24	0.72	84	1.7
Size Ratio	14		32		48	

Impedance Matrix Storage

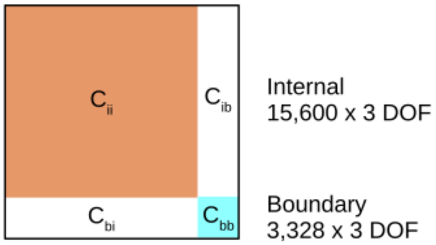
- Condensed Impedance

$$[C_{Red}] = [C_{bb}] \text{-} [C_{bi}]\,[C_{ii}]^{-1}\,[C_{ib}]$$

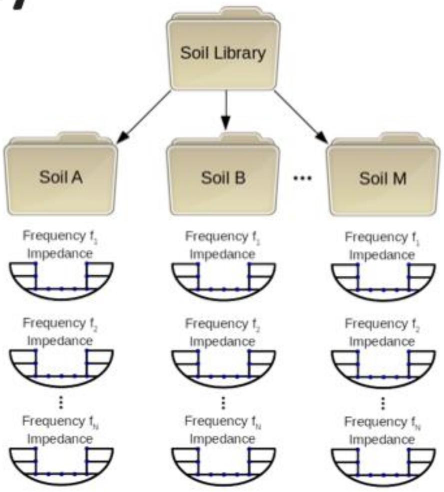
  - Let $[X]$ be the solution of $[C_{ii}][X] = [C_{ib}]$, then

$$[C_{Red}] = [C_{bb}] \text{-} [C_{bi}][X]$$

Case 2, 18,928 Interaction Nodes



Store Impedances and Load Vectors as Soil Library



Soil Library

Soil A

Soil B

…

Soil M

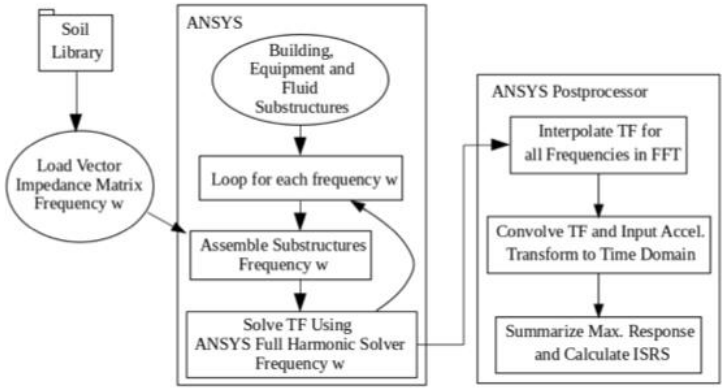
Frequency f_1 Impedance

Frequency f_2 Impedance

⋮

Frequency f_N Impedance

Commercial FEM Solution (ANSYS)



Soil Library

Load Vector Impedance Matrix Frequency w

ANSYS

Building, Equipment and Fluid Substructures

Loop for each frequency w

Assemble Substructures Frequency w

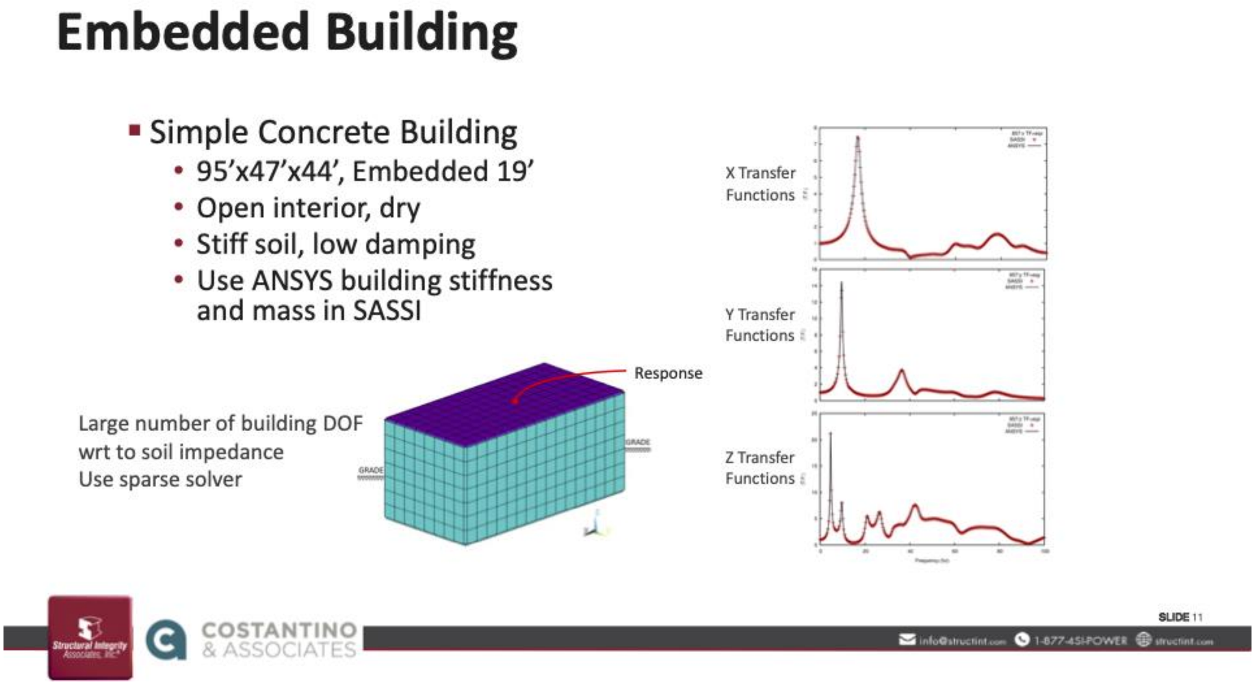
Solve TF Using ANSYS Full Harmonic Solver Frequency w

ANSYS Postprocessor

Interpolate TF for all Frequencies in FFT

Convolve TF and Input Accel. Transform to Time Domain

Summarize Max. Response and Calculate ISRS



Embedded Building

- Simple Concrete Building
  - 95'x47'x44', Embedded 19'
  - Open interior, dry
  - Stiff soil, low damping
  - Use ANSYS building stiffness and mass in SASSI

Large number of building DOF wrt to soil impedance
Use sparse solver

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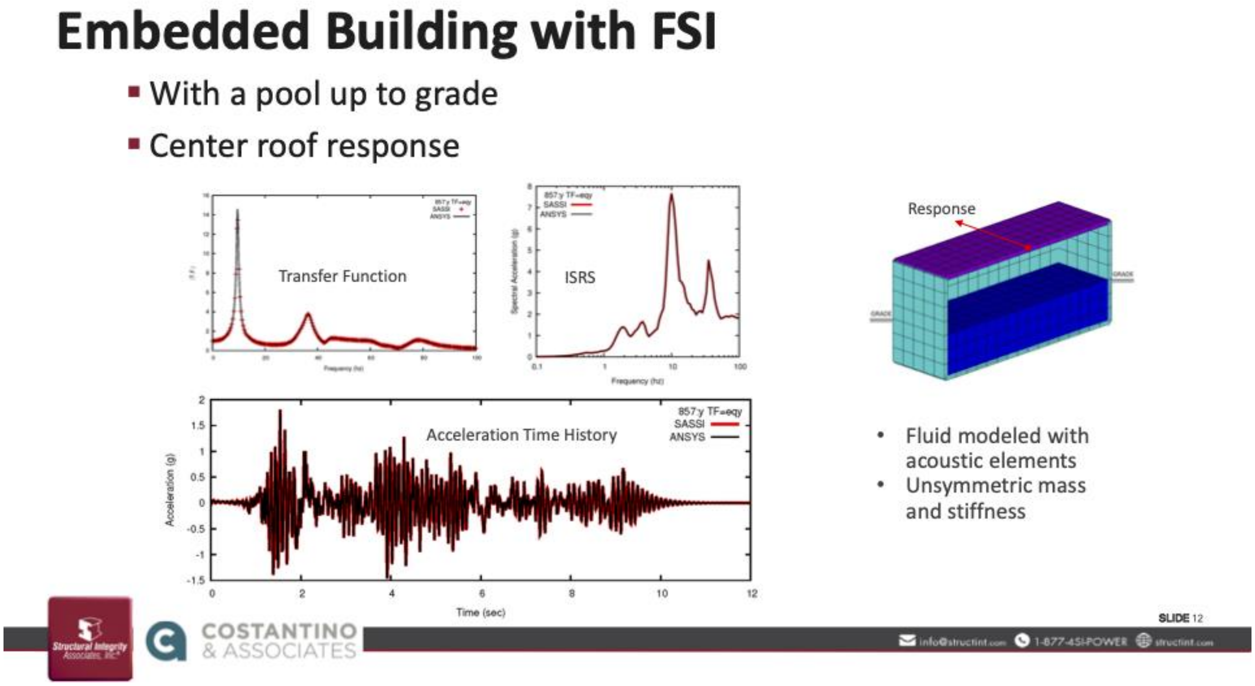
SLIDE 11



Embedded Building with FSI

- With a pool up to grade
- Center roof response

- Fluid modeled with acoustic elements
- Unsymmetric mass and stiffness

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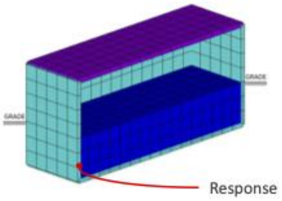
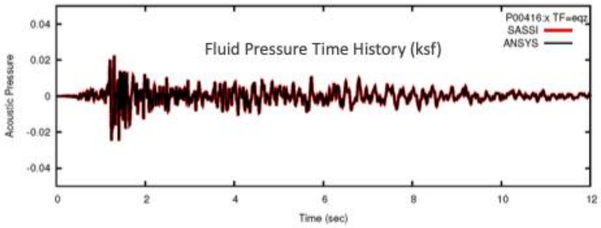
SLIDE 12

Embedded Building with FSI (cont)

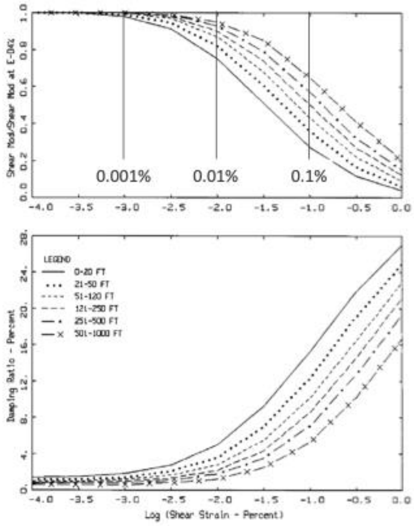
- With a pool up to grade
- Fluid response to vertical motion



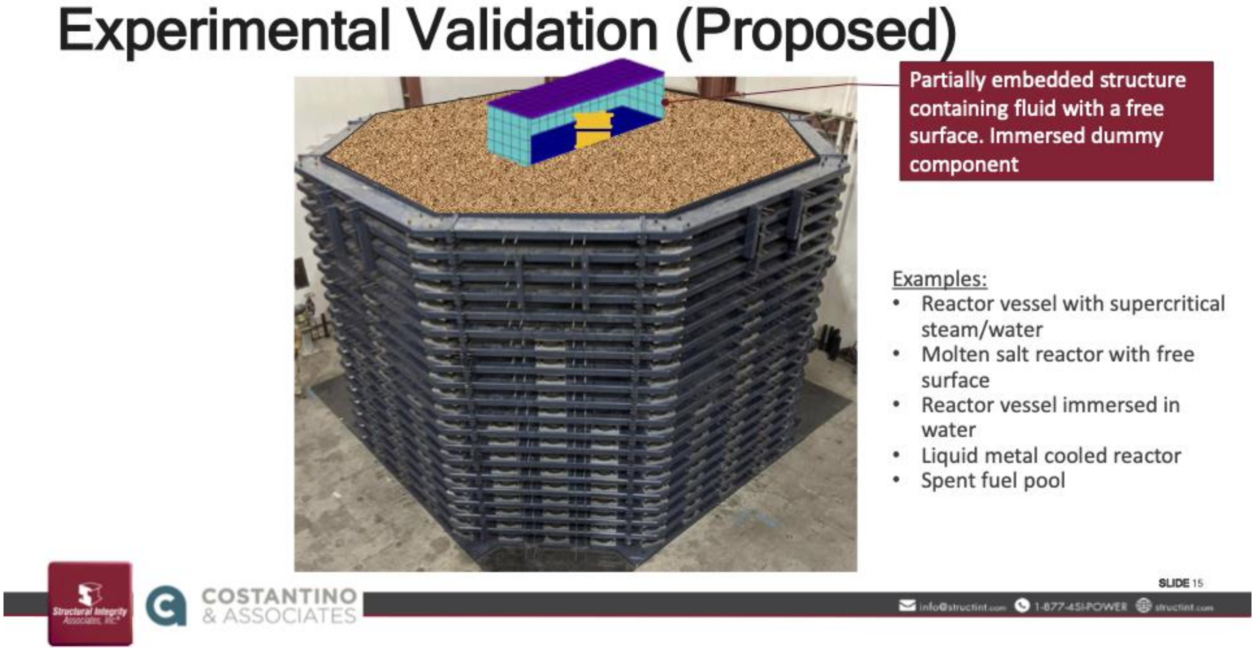


Experimental Needs

- Validation of Analysis Methods
  - Soil box seismic environment w/o building
    - Not a halfspace
    - Need to fully characterize the response of a soil box
  - Range of soil strain levels
    - Operating basis – low strain (boring, but important test)
    - Design basis – moderate strain
    - Beyond design basis – high strain
  - Soil box with test structure
    - Focus on SSI interaction
    - Wall tractions with waterproofing
    - Soil pressure on wall
    - Foundation rocking



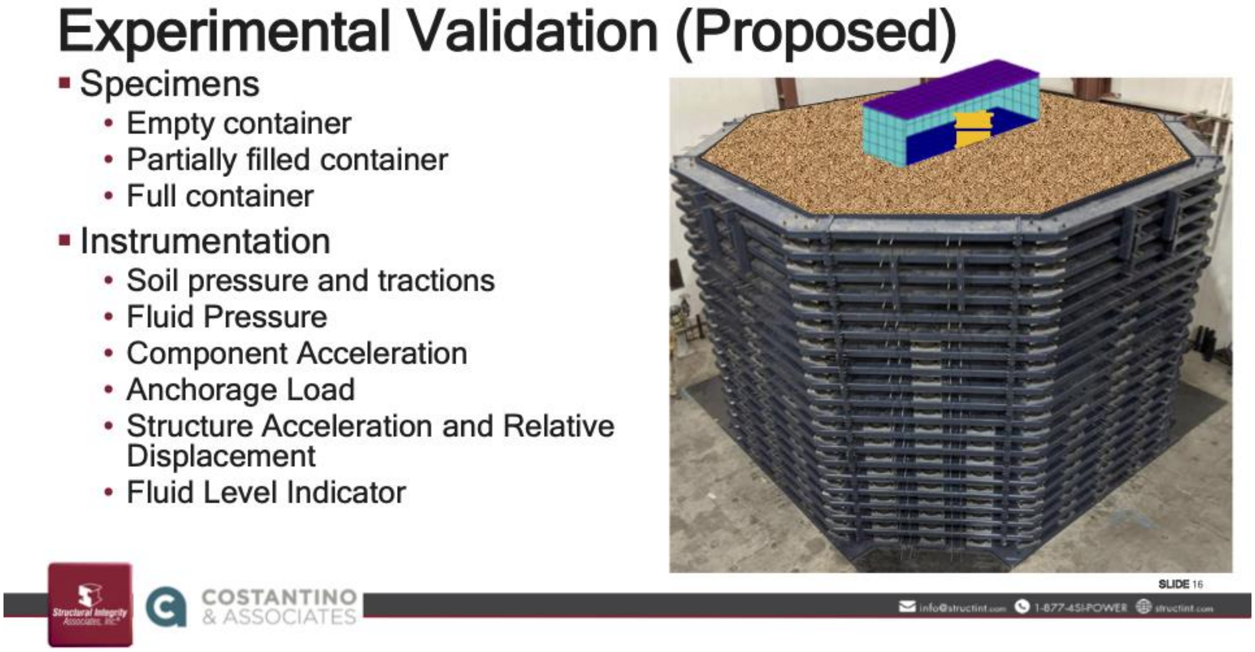
MODULUS REDUCTION AND DAMPING CURVES
EPRI SOIL



Experimental Validation (Proposed)

Partially embedded structure containing fluid with a free surface. Immersed dummy component

Examples:
- Reactor vessel with supercritical steam/water
- Molten salt reactor with free surface
- Reactor vessel immersed in water
- Liquid metal cooled reactor
- Spent fuel pool



Experimental Validation (Proposed)

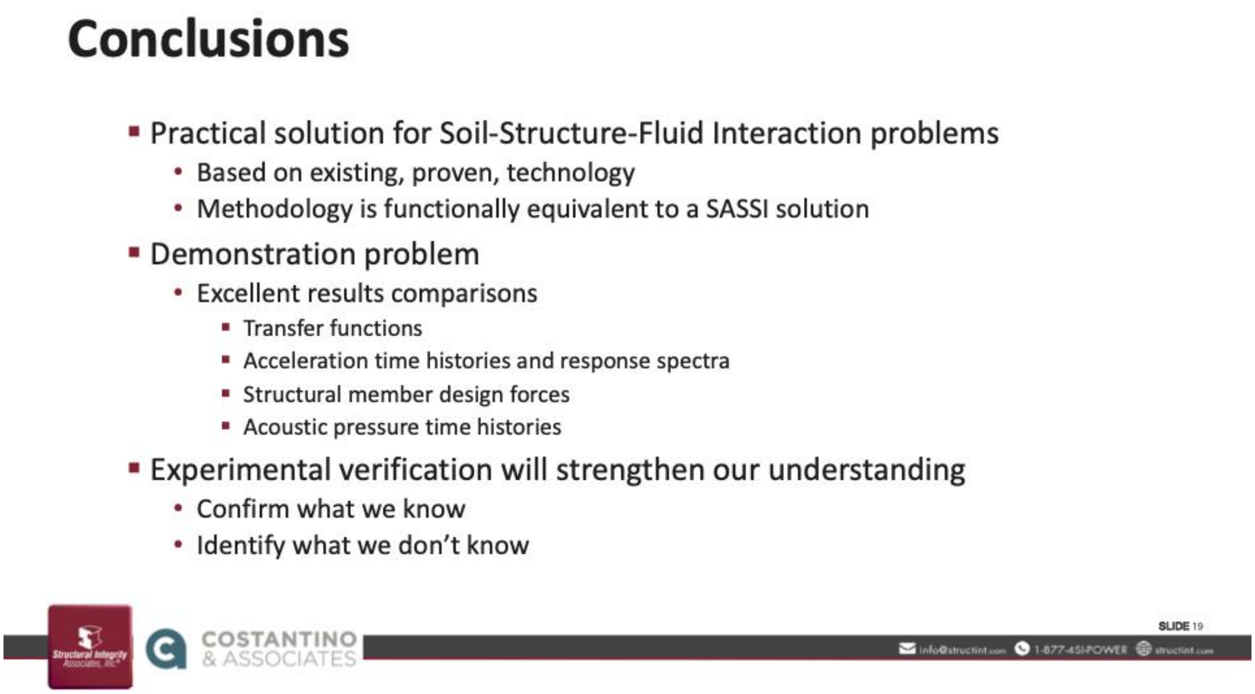
- Specimens
  - Empty container
  - Partially filled container
  - Full container
- Instrumentation
  - Soil pressure and tractions
  - Fluid Pressure
  - Component Acceleration
  - Anchorage Load
  - Structure Acceleration and Relative Displacement
  - Fluid Level Indicator

Experimental Validation (Proposed)

- Analytical preparation
  - Create soil impedance functions
  - Frequency Domain Solution
  - Time Domain Solution
  - Target range of soil strain
    - 0.1g, 0.5g, 1.0g, 2.0g PGA
- Experimental Runs
  - 0.1g, 0.5g, 1.0g, 2.0g PGA
  - Monitor divergence from analytical runs

Experimental Validation (Proposed)

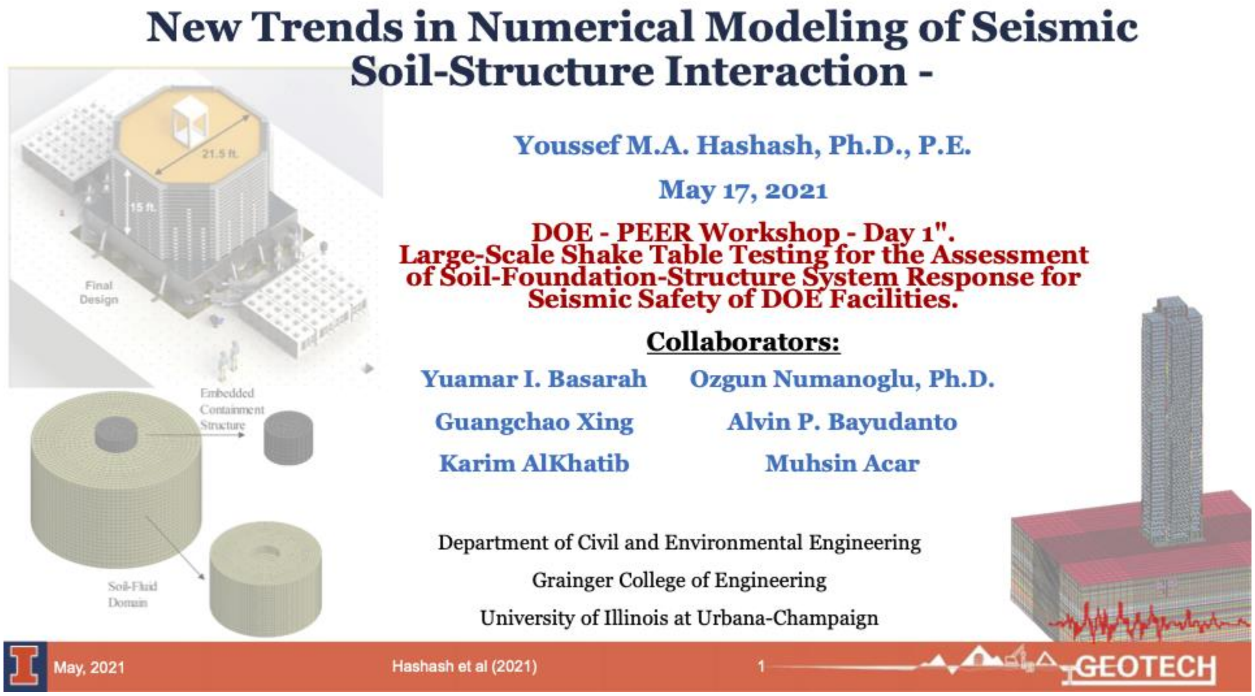
- Results
  - Identify acceptable range for equivalent linear analyses
  - Validate coupled time domain solution
  - Establish method to qualify Gen IV reactors with Soil-Fluid-Structure interaction
- Possible expansions
  - Isolation and damping
  - Pebble bed fuel
  - Cask tipping



Conclusions

- Practical solution for Soil-Structure-Fluid Interaction problems
  - Based on existing, proven, technology
  - Methodology is functionally equivalent to a SASSI solution
- Demonstration problem
  - Excellent results comparisons
    - Transfer functions
    - Acceleration time histories and response spectra
    - Structural member design forces
    - Acoustic pressure time histories
- Experimental verification will strengthen our understanding
  - Confirm what we know
  - Identify what we don't know

NEW TRENDS IN NUMERICAL MODELING OF SEISMIC SOIL-STRUCTURE-INTERACTION



New Trends in Numerical Modeling of Seismic Soil-Structure Interaction -

Youssef M.A. Hashash, Ph.D., P.E.

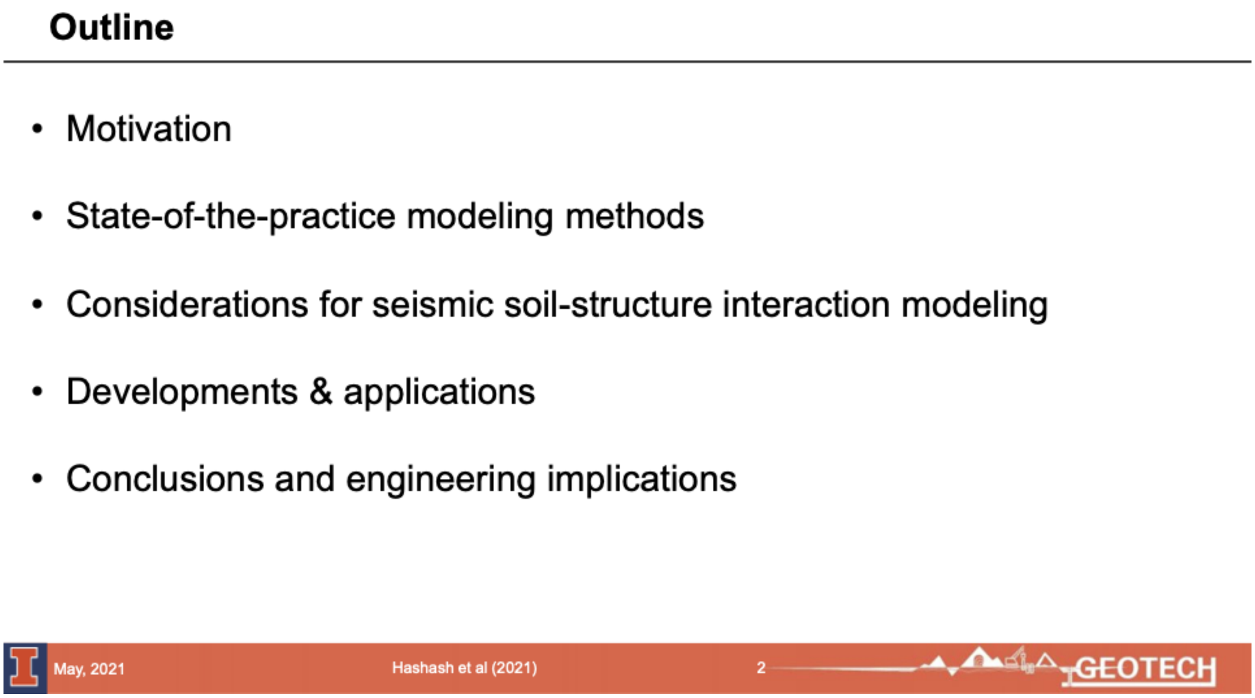
May 17, 2021

DOE - PEER Workshop - Day 1".
Large-Scale Shake Table Testing for the Assessment of Soil-Foundation-Structure System Response for Seismic Safety of DOE Facilities.

Collaborators:

Yuamar I. Basarah **Ozgun Numanoglu, Ph.D.**
Guangchao Xing **Alvin P. Bayudanto**
Karim AlKhatib **Muhsin Acar**

Department of Civil and Environmental Engineering
Grainger College of Engineering
University of Illinois at Urbana-Champaign



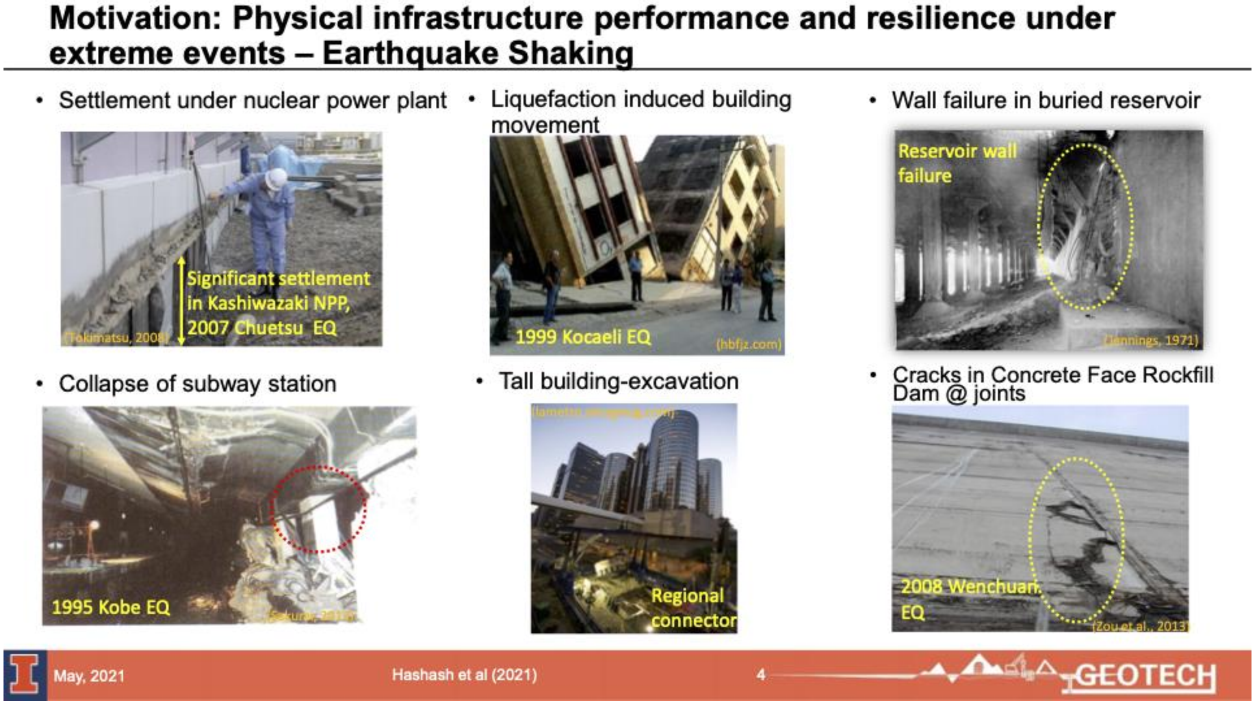
Outline

- Motivation
- State-of-the-practice modeling methods
- Considerations for seismic soil-structure interaction modeling
- Developments & applications
- Conclusions and engineering implications

Motivation: Physical infrastructure performance and resilience under extreme events – Earthquake Shaking

Urban Areas

(Seismic) Soil-Structure Interaction Is Everywhere!



Motivation: Physical infrastructure performance and resilience under extreme events – Earthquake Shaking

- Settlement under nuclear power plant

Significant settlement in Kashiwazaki NPP, 2007 Chuetsu EQ

(Tokimatsu, 2008)

- Liquefaction induced building movement

1999 Kocaeli EQ

(hbfjz.com)

- Wall failure in buried reservoir

Reservoir wall failure

(Jennings, 1971)

- Collapse of subway station

1995 Kobe EQ

(Sakurai, [illegible])

- Tall building-excavation

Regional connector

- Cracks in Concrete Face Rockfill Dam @ joints

2008 Wenchuan EQ

(Zou et al., 2013)

Outline

- Motivation
- State-of-the-practice modeling methods
- Considerations for seismic soil-structure interaction modeling
- Developments & applications
- Conclusions and engineering implications

Simplified methods and Code-based Procedures for Seismic Soil-structure Interaction

ASCE 7-16 Chapter 19:

- SSI may be used with equivalent lateral force, linear dynamic, or nonlinear dynamic analysis.
- Site class C, D, E, or F
- Modified response spectrum to consider SSI

Needs to represent the soil and structure explicitly to understand the seismic behavior of SSI system

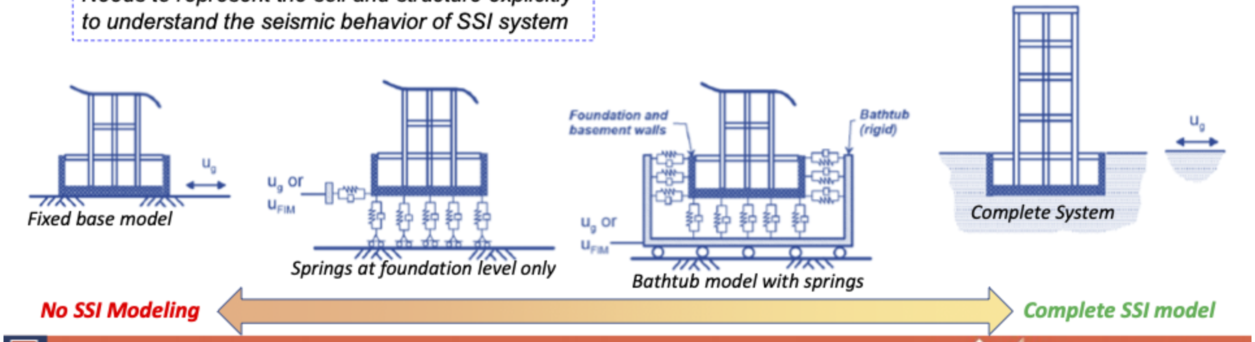
Tall Building Initiative (TBI, 2017):

Fixed base model:

Input motion applied directly at the base of the building (No SSI)

Bathtub model using springs to represent soil for flexible base

Complete SSI model where all components are modeled explicitly



Outline

- Motivation
- State-of-the-practice modeling methods
- Considerations for seismic soil-structure interaction modeling
  - Input: Site investigations and ground motions
  - Elements of seismic soil-structure interaction analysis
  - Structural modeling and soil-structure interface conditions
- Developments & applications
- Conclusions and engineering implications

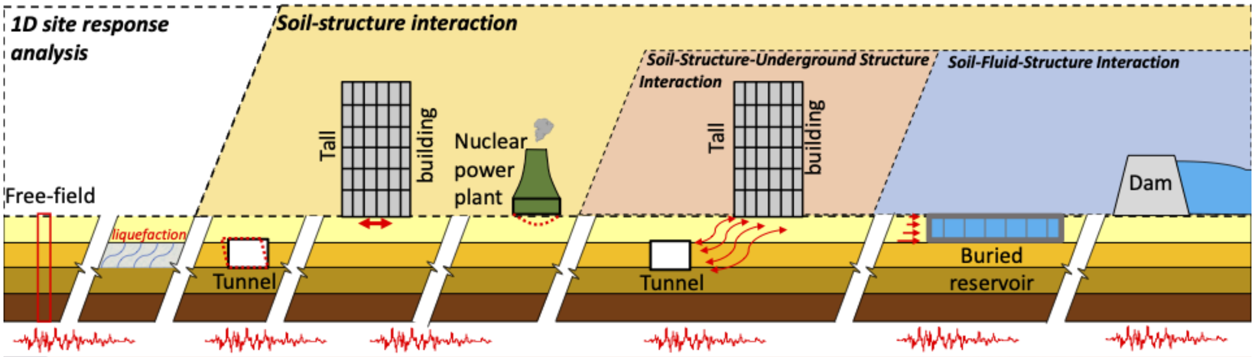
Elements of Seismic Soil-structure Interaction Analysis

Free-field condition

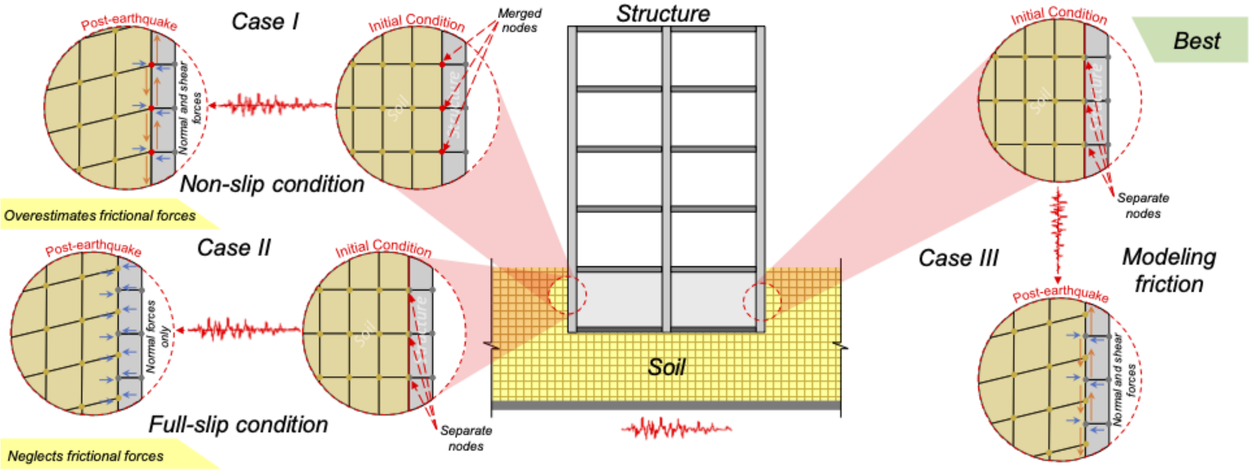
- 1D site response analysis
  - DEEPSOIL, SHAKE, Strata Shear beam model
- Soil liquefaction assessment

Soil-structure interaction

- Ex.: Tunnel, Tall building, Nuclear power plant on sand ...
- **Soil-structure-underground structure interaction:**
  - Tall building-tunnel system in urban area
- **Soil-fluid-structure interaction:**
  - Dam & buried reservoir



Soil-Structure Interface Conditions



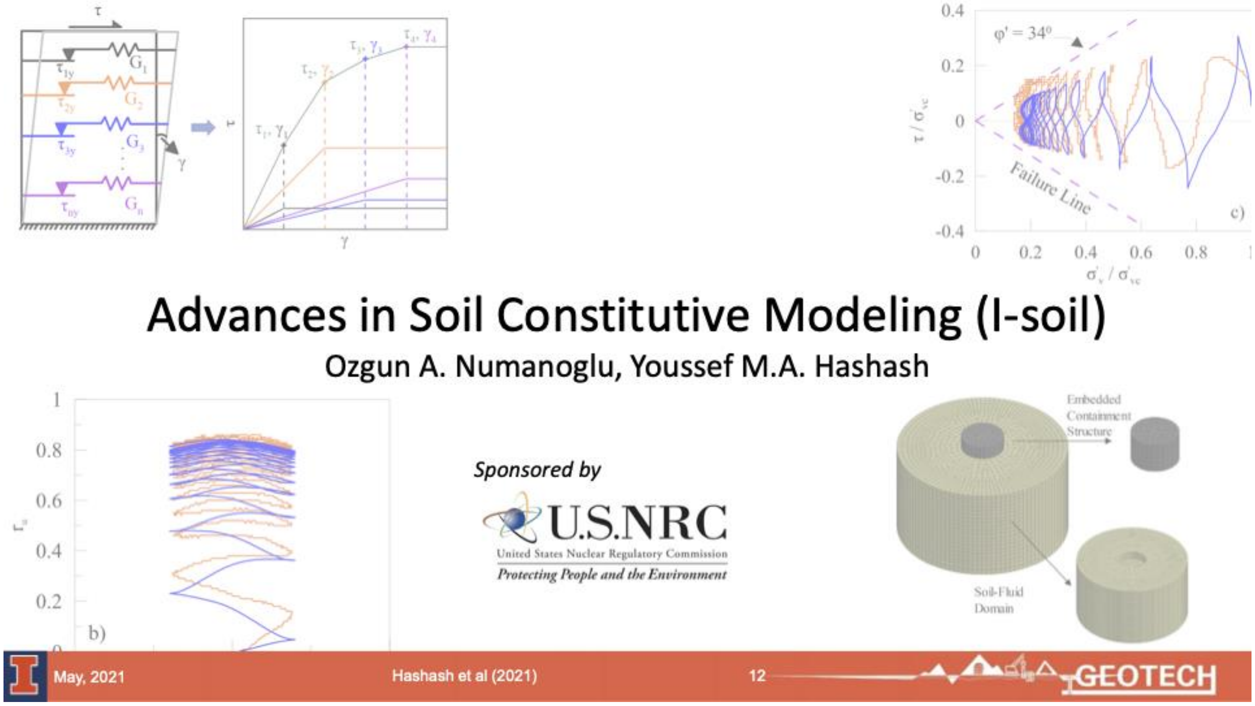
- The interface between soil and structure can be modeled as full-slip, non-slip, or something in between by modeling friction (reflecting real world conditions).

Gaps/Needs in Numerical and Material Constitutive Modeling

- Higher fidelity representation of both the structure and soil as well as interface interaction - friction, sliding, gapping.
- 3-D geometries and multi-directional base excitation.
- Simulation run times and need for representation of uncertainty – computational cost.

Outline

- Motivation
- State-of-the-practice modeling methods
- Considerations for seismic soil-structure interaction modeling
- Developments & applications
    - Advances in soil constitutive modeling (I-soil)
    - Seismic settlement of heavy structures on dense sands
    - Other problems
    - Computational considerations
- Conclusions and engineering implications



Advances in Soil Constitutive Modeling (I-soil)

Ozgun A. Numanoglu, Youssef M.A. Hashash

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U.S.NRC
United States Nuclear Regulatory Commission
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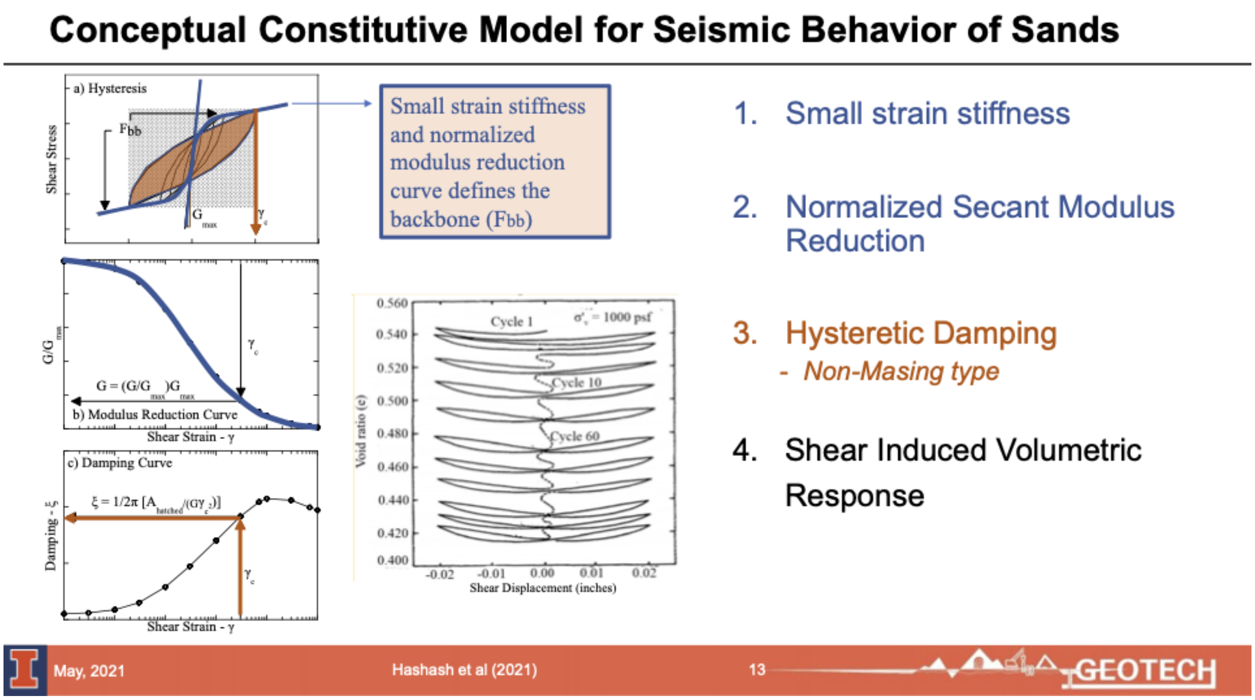


Conceptual Constitutive Model for Seismic Behavior of Sands

Small strain stiffness and normalized modulus reduction curve defines the backbone (Fbb)

1. Small strain stiffness
2. Normalized Secant Modulus Reduction
3. Hysteretic Damping
   - *Non-Masing type*
4. Shear Induced Volumetric Response

Development of I-soil – A New Practical Soil Model

- Extension of MAT79 in LS-DYNA - A 3D, distributed element plasticity model introduced by Chiang and Beck (1994) based on Iwan (1969) distribution of elasto-plastic nested components.

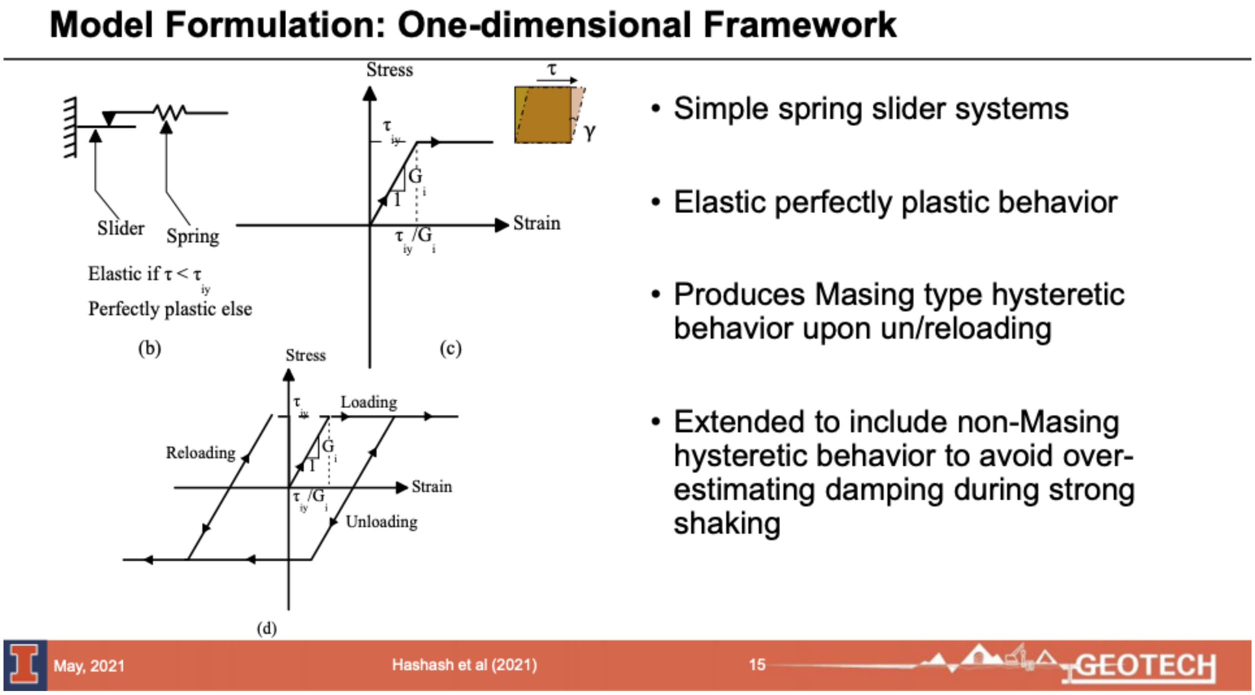
- Model formulation
  - Piecewise-linearized, hysteretic, non-Masing type nonlinear model to represent modulus reduction and damping curves.
  - Models shear induced volumetric strains and excess porewater pressures.
  - Represents medium dense to very dense sand behavior. Promising for loose to medium sands. Easy to calibrate and use.

- Analysis platform:
  - Implemented in LS-DYNA (with solid-fluid coupled framework)

- Not included
  - Plasticity due to hydrostatic loading
  - Anisotropy
  - Critical state behavior

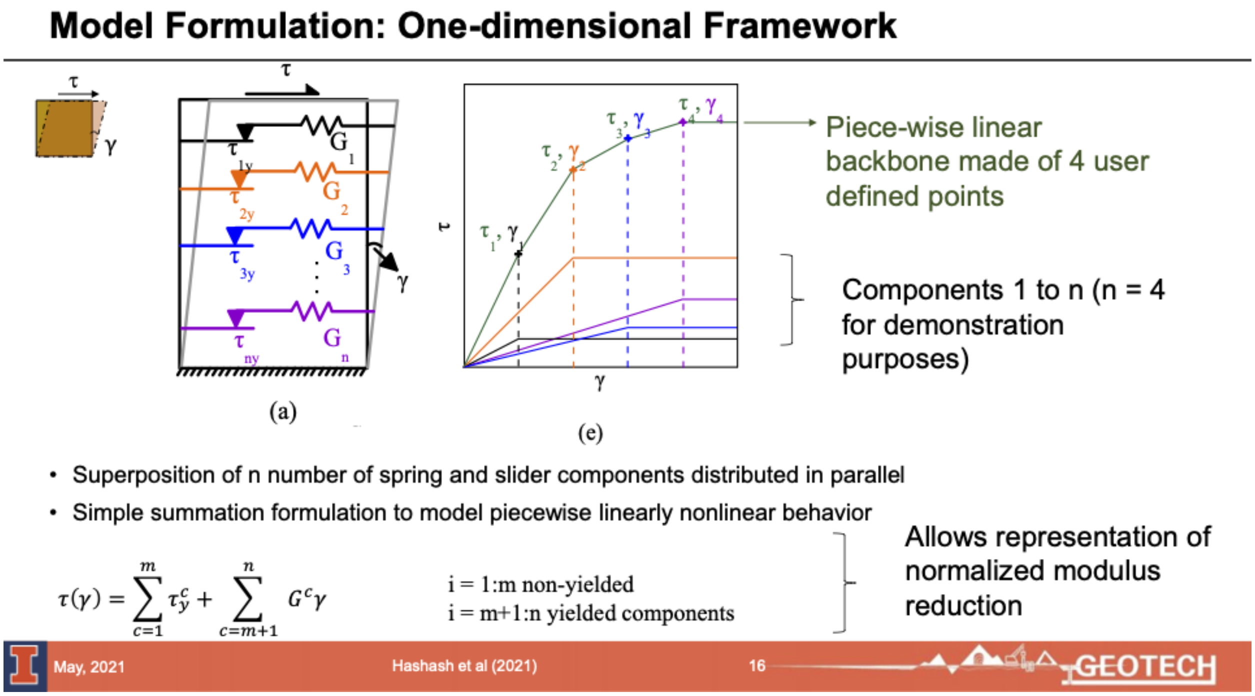
Why LS-DYNA?

- Designed to solve dynamic problems
- Detailed representation of structural components
- Fluid modeling capability
- Ease of use (e.g., GUI, pre- and post-processor)
- Computationally efficient
- Parallel computing capabilities
- Easy to automate/queue analyses



Model Formulation: One-dimensional Framework

- Simple spring slider systems
- Elastic perfectly plastic behavior
- Produces Masing type hysteretic behavior upon un/reloading
- Extended to include non-Masing hysteretic behavior to avoid over-estimating damping during strong shaking



Model Formulation: One-dimensional Framework

Piece-wise linear backbone made of 4 user defined points

Components 1 to n (n = 4 for demonstration purposes)

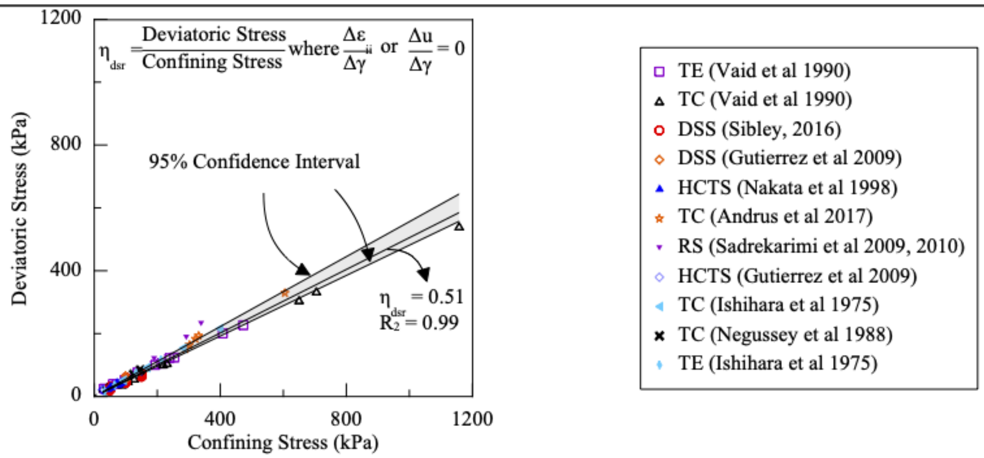
- Superposition of n number of spring and slider components distributed in parallel
- Simple summation formulation to model piecewise linearly nonlinear behavior

$$\tau(\gamma) = \sum_{c=1}^{m} \tau_y^c + \sum_{c=m+1}^{n} G^c \gamma$$

i = 1:m non-yielded
i = m+1:n yielded components

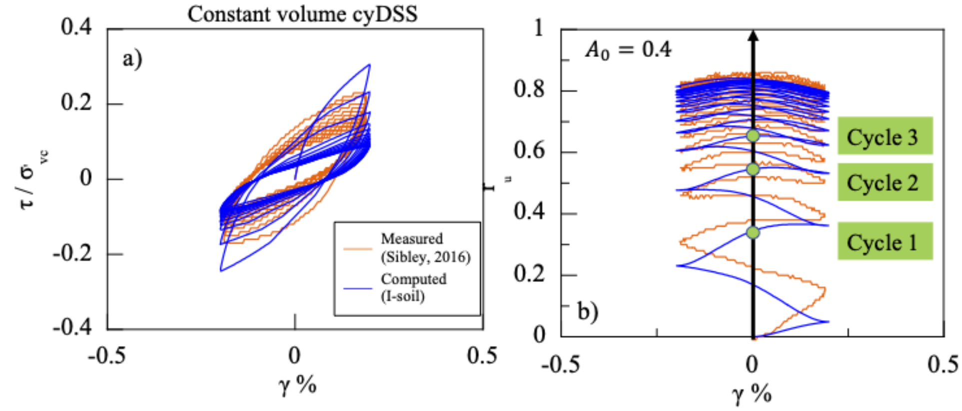
Allows representation of normalized modulus reduction

Volumetric Response Calibration: η_{dsr}



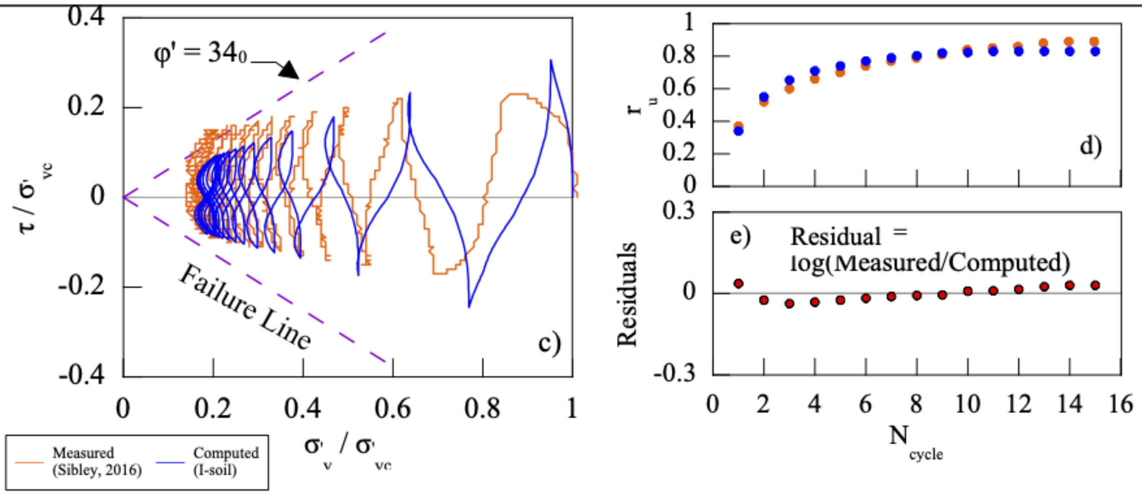
- Values between 0.4 – 0.8 with mean value of 0.51 for 10 – 1200 kPa, 40 – 95% D_R
- η_{dsr} = 0.51 => $\phi'_{dsr} = 31^0 \leq \phi'_{critical\ state\ (or\ constant\ volume)} = 30^0 - 32^0$
- Negussey et al. 1988, Vaid et al. 1990, Sadrekarimi and Olson, 2011

Volumetric Response Calibration : A_0



- A trial-and-error procedure to obtain good estimation of hysteretic and volumetric response
- e.g., A_0 = 0.4 captures both behavior well for a given sand specimen

Volumetric Response Calibration : A_0



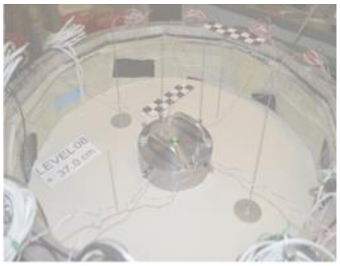
- For a given trial, the aim is to keep the residuals near zero value throughout shearing.

Input Parameters

Source	Parameters/ Symbol	Physical Contribution/Meaning	Dense Sand
	σ'_{ref}	Reference effective mean stress at which the parameters are defined	kPa
Normalized modulus reduction curve (MR) and shear wave velocity (Vs)	n number of τ -γ pairs	Curve fitted n point discretized backbone curve at σ'ref that matches Vs at very small strains and MR at different shear strain levels	MR: Darendeli (2001) Vs: Field Measurement (Alternatively Menq (2003) correlation
MR curves at different confining pressures	b	Power law coefficient defining the effective mean stress dependency characteristics of stiffness	0.5 (Arulmori et al. 1992)

In simple terms, I-soil input:

- Modulus reduction and damping curves commonly used for 1D nonlinear site response analysis
- Two volumetric response parameters (A_0 and η_{dsr}) calibrated using cyDSS or empirical correlations.



Seismic settlement of heavy structures on dense sands

Ozgun A. Numanoglu, Youssef M.A. Hashash, Scott M. Olson, Alfonso Cerna Diaz,
Cassandra J. Rutherford, Thomas Weaver, Lopamudra Bhaumik

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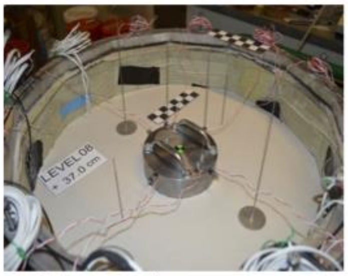
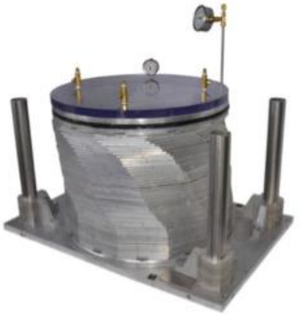
National Science Foundation
WHERE DISCOVERIES BEGIN

Seismic Settlement of Dense Sand

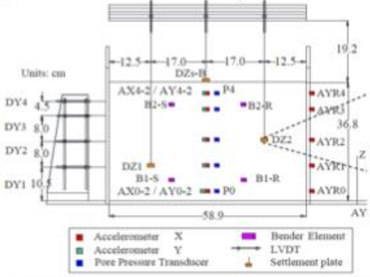
Free-field test specimen

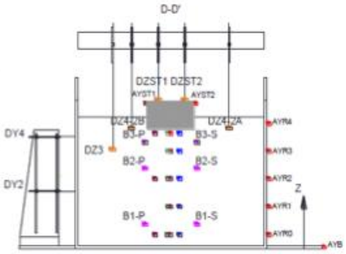


SSI test specimen



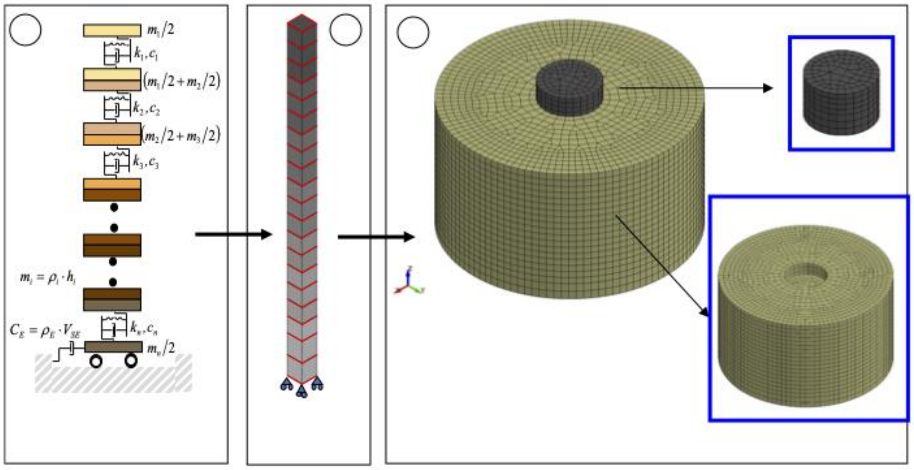
2D Laminar Container





- Uni-bi-directional broadband base excitations
- 95% relative density Ottawa 40/70 sand material

1-D to 3-D stepwise modeling approach



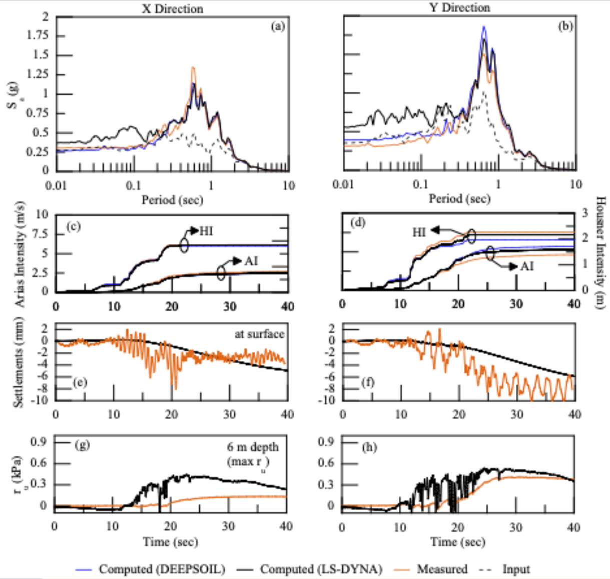
Rigid Steel Box with
YM: 200 GPa
Poisson's ratio : 0.3

- Profile from shear beam simulations
- Pressure dependency of constitutive model handles the effect of increase in confining pressure

- 1D Nonlinear site response => Shear beam => Three dimensional SSI

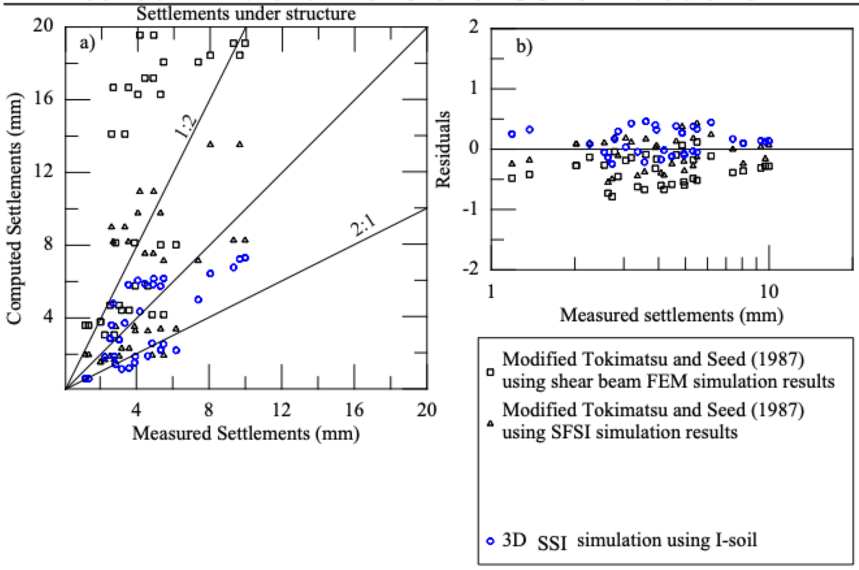
DEEPSOIL LS-DYNA LS-DYNA

Representative model- experiment comparisons
Seismic Settlement of Dense Sand

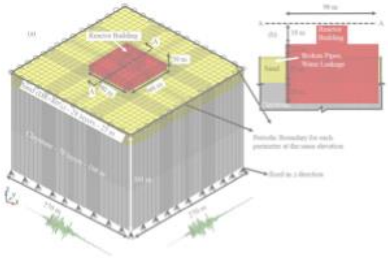


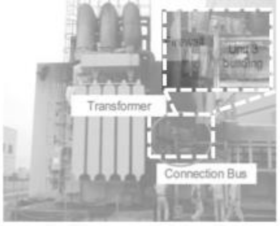
- Spectral response
- Housner and Arias intensities
- Seismic settlements
- Normalized excess porewater pressures
- Representative results for Landers event

Structure settlements: Modified Tokimatsu and Seed (1987) method + multi-dimensional SSI simulations



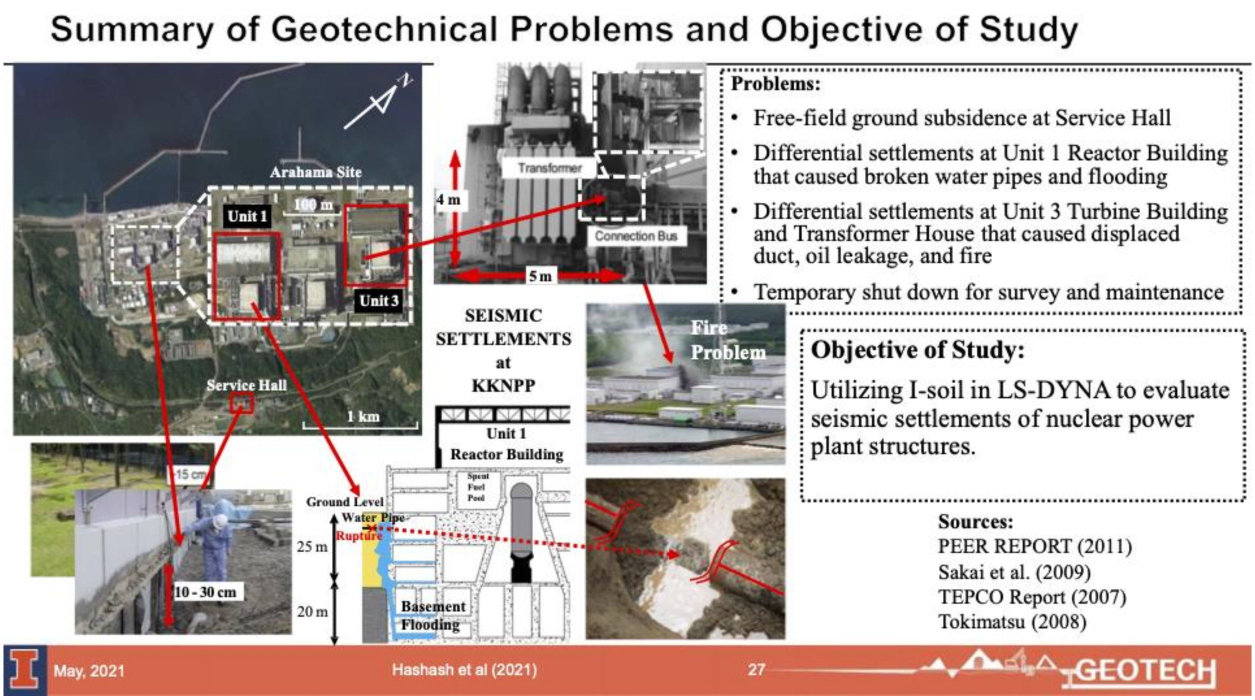
- With the help of FEM and I-soil on estimation of the effects of structure on soil behavior (increased confinement), much better empirical estimations were achieved.
- Strains were extracted from central array under the structure





Numerical Simulations of Kashiwazaki-Kariwa Nuclear Power Plant, Japan

Alvin Bayudanto, Ozgun A. Numanoglu, Youssef M.A. Hashash



Summary of Geotechnical Problems and Objective of Study

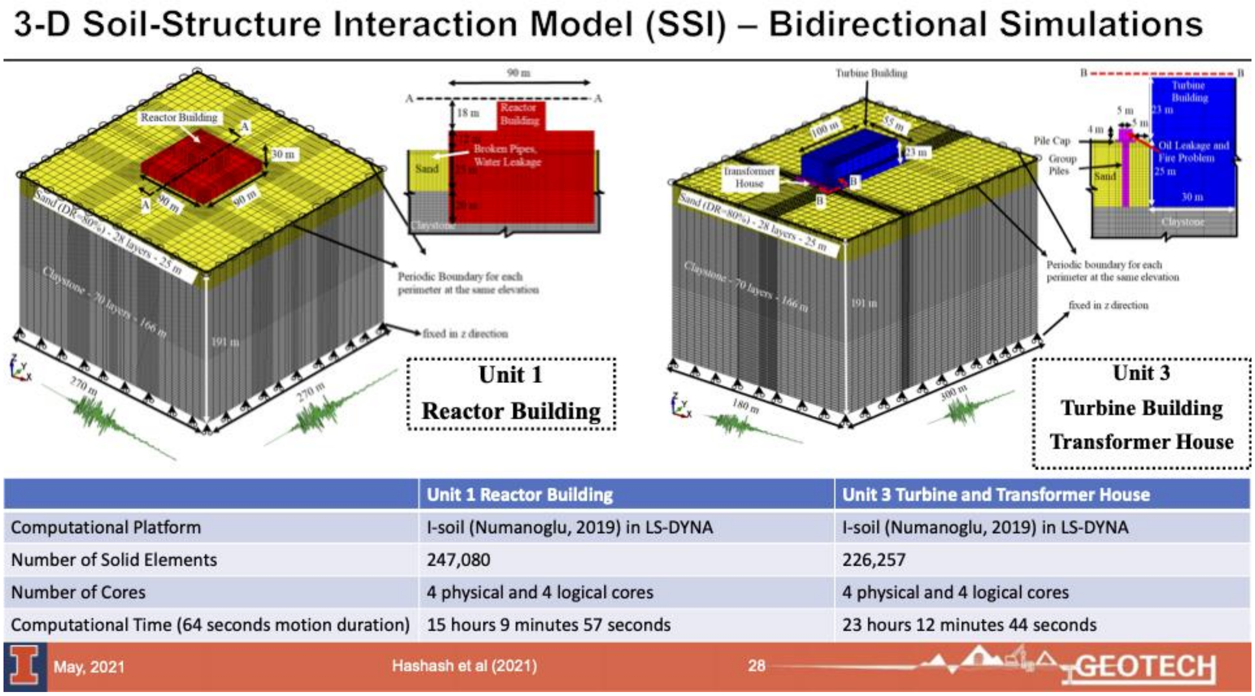
Problems:

- Free-field ground subsidence at Service Hall
- Differential settlements at Unit 1 Reactor Building that caused broken water pipes and flooding
- Differential settlements at Unit 3 Turbine Building and Transformer House that caused displaced duct, oil leakage, and fire
- Temporary shut down for survey and maintenance

Objective of Study:

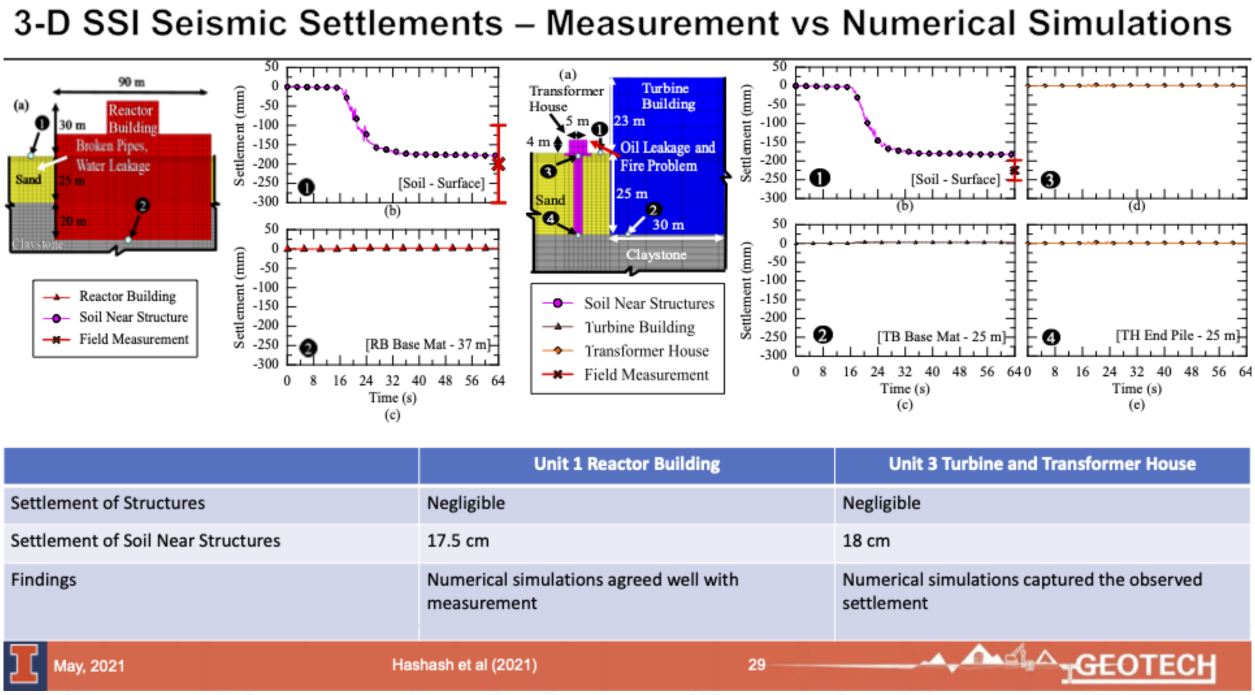
Utilizing I-soil in LS-DYNA to evaluate seismic settlements of nuclear power plant structures.

Sources:
PEER REPORT (2011)
Sakai et al. (2009)
TEPCO Report (2007)
Tokimatsu (2008)



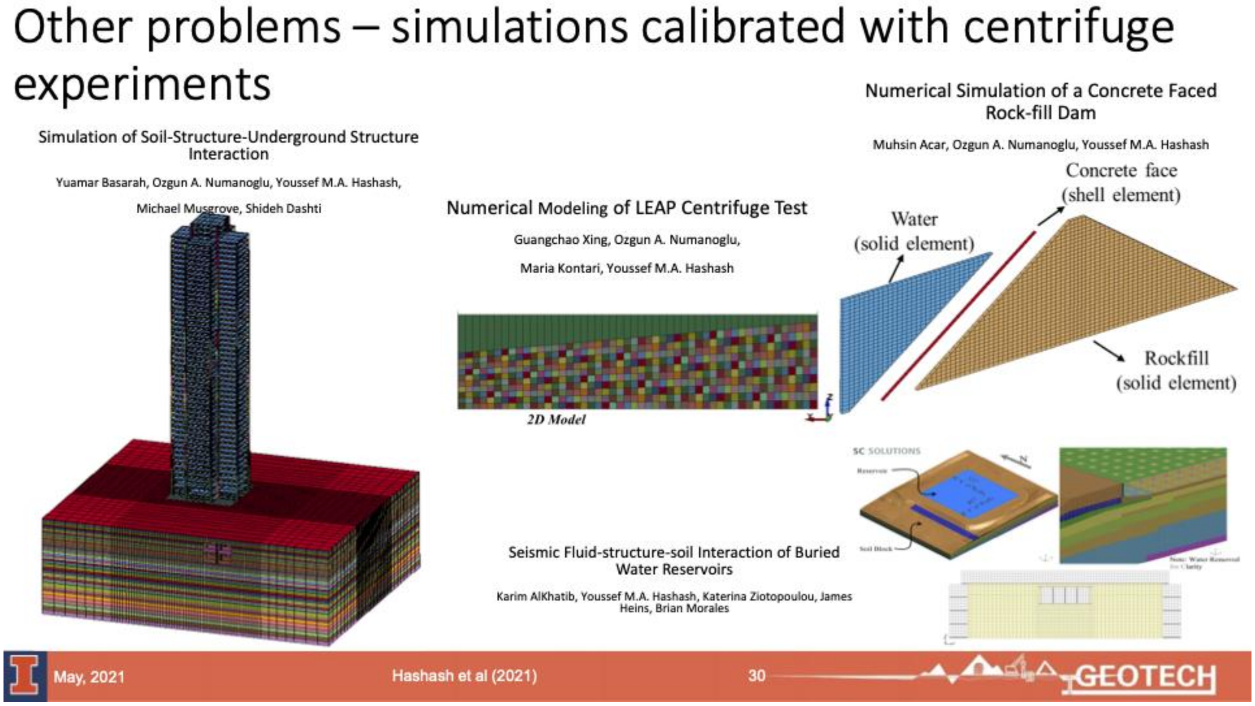
3-D Soil-Structure Interaction Model (SSI) – Bidirectional Simulations

	Unit 1 Reactor Building	Unit 3 Turbine and Transformer House
Computational Platform	I-soil (Numanoglu, 2019) in LS-DYNA	I-soil (Numanoglu, 2019) in LS-DYNA
Number of Solid Elements	247,080	226,257
Number of Cores	4 physical and 4 logical cores	4 physical and 4 logical cores
Computational Time (64 seconds motion duration)	15 hours 9 minutes 57 seconds	23 hours 12 minutes 44 seconds



3-D SSI Seismic Settlements – Measurement vs Numerical Simulations

	Unit 1 Reactor Building	Unit 3 Turbine and Transformer House
Settlement of Structures	Negligible	Negligible
Settlement of Soil Near Structures	17.5 cm	18 cm
Findings	Numerical simulations agreed well with measurement	Numerical simulations captured the observed settlement



Other problems – simulations calibrated with centrifuge experiments

Simulation of Soil-Structure-Underground Structure Interaction

Yuamar Basarah, Ozgun A. Numanoglu, Youssef M.A. Hashash, Michael Musgrove, Shideh Dashti

Numerical Modeling of LEAP Centrifuge Test

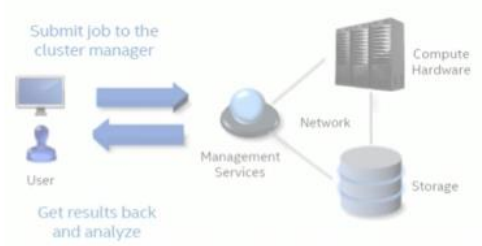
Guangchao Xing, Ozgun A. Numanoglu, Maria Kontari, Youssef M.A. Hashash

Numerical Simulation of a Concrete Faced Rock-fill Dam

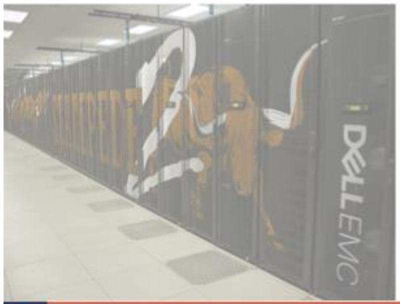
Muhsin Acar, Ozgun A. Numanoglu, Youssef M.A. Hashash

Seismic Fluid-structure-soil Interaction of Buried Water Reservoirs

Karim AlKhatib, Youssef M.A. Hashash, Katerina Ziotopoulou, James Heins, Brian Morales



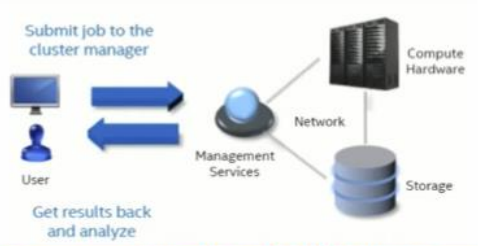
High Performance Computing (HPC)



Computational Platform for Large-scale Simulations

- Running large scale of simulations to evaluate the uncertainties
- Needs more powerful computer resources
- Using high performance computing (HPC) or supercomputer
    - Contains thousands of compute nodes (servers) that work together to complete tasks faster (parallel processing)
    - Submit a single job up to 6,144 cores with 128 nodes at one time
    - Max durations: 48 hrs/job
    - Max jobs in queue : 25 jobs

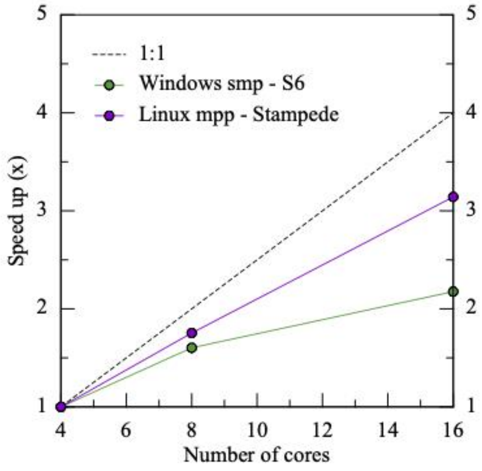
HPC Stampede in Texas Advanced Computing Center

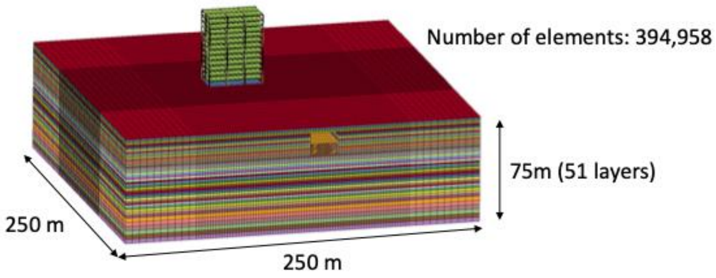


Source: Introduction to HPC https://www.youtube.com/watch?v=bkLVuNfiCVs

Example of runtime comparison (Tall building-tunnel)

Stampede vs Single server



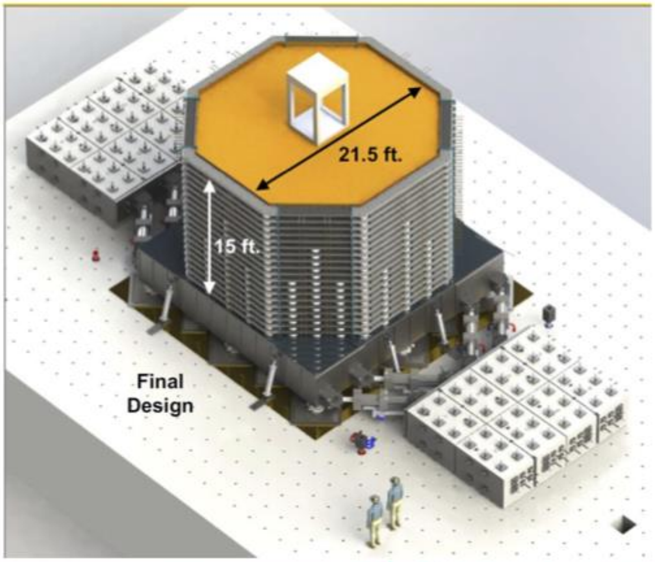


For 32.7 s of simulation

Machine	No of cores	Runtime
Single server	12	25 hr
HPC on Stampede	12	11 hr

Role of large scale testing facilities

- All the considered simulations based on centrifuge experiments or field performance studies.
- Large scale testing facilities will provide importance additional capabilities to further enhance fidelity of simulations.
- Experimental cost and time considerations.
- In-place soil characterization
- Reproduce selected existing centrifuge experiments?
- Interface behavior?



Conclusions and Engineering Implications

- Advances in analysis software, computing hardware, and big data management tools has enabled a new era in the analysis of complex nonlinear seismic soil-structure interactions (SSSI) in three dimensions (3D). Multiple available platforms.

- Three-Dimensional SSSI modeling, with relatively simplified soil models, was successfully employed in analysis of complex engineering problems including: (a) settlement of heavy structures on dense sand, (b) tall building-tunnel interaction, (c) soil liquefaction, (c) concrete-face rock fill dams, (e) buried water supply reservoirs.

- Large scale testing facilities provide important data for further validation of SSI simulations.

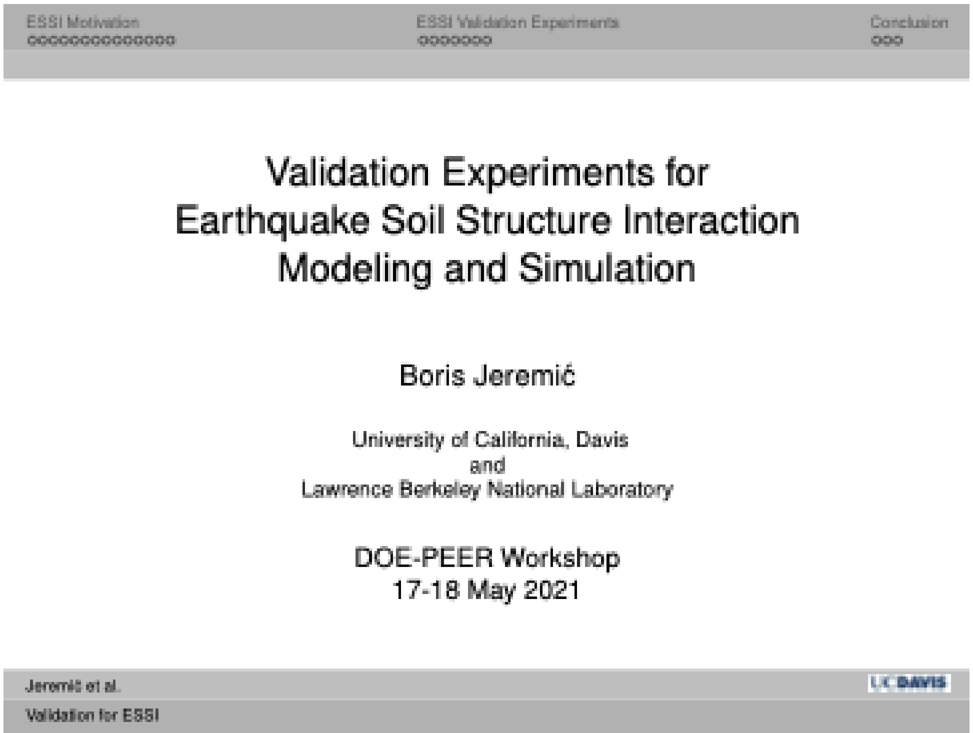
- *If interested:* The presented new constitutive model (I-soil) that is calibrated, tested, and implemented in the numerical analysis platform LS-DYNA is computationally efficient and easy to use *and available upon request – Contact: hashash@illinois.edu* .

- Expanded presentation (KGS 2021 Lecture):
  - https://www.youtube.com/watch?v=sQZHOxe_p-Q

Thank you

Questions?

VALIDATION TESTING FOR EARTHQUAKE SOIL STRUCTURE INTERACTION MODELING AND SIMULATION



ESSI Motivation ESSI Validation Experiments Conclusion

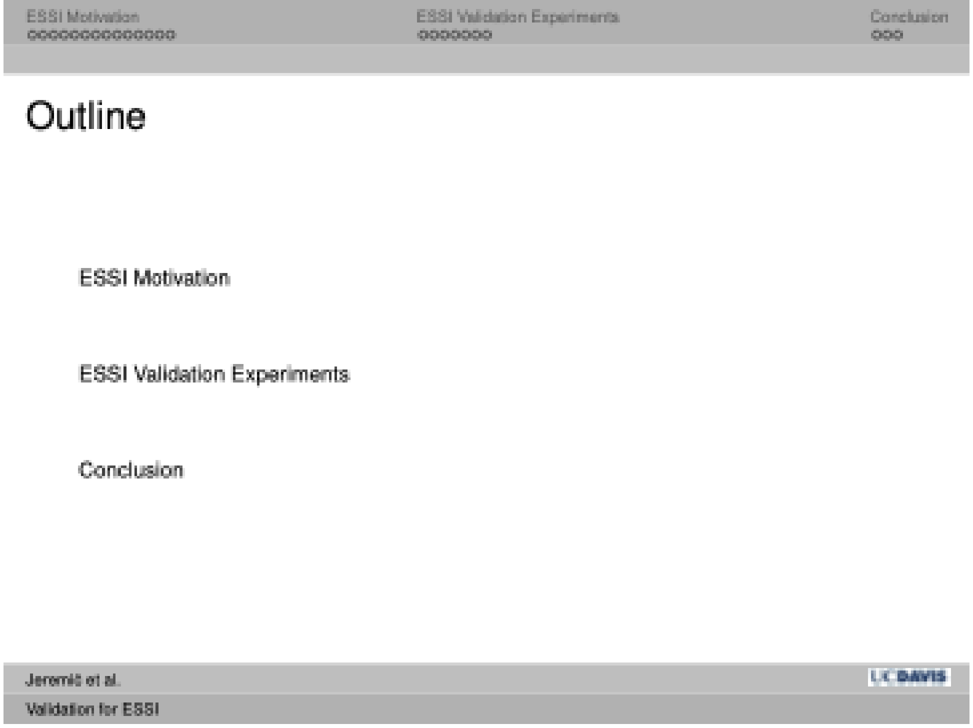
Validation Experiments for Earthquake Soil Structure Interaction Modeling and Simulation

Boris Jeremić

University of California, Davis
and
Lawrence Berkeley National Laboratory

DOE-PEER Workshop
17-18 May 2021

Jeremić et al.
Validation for ESSI



ESSI Motivation ESSI Validation Experiments Conclusion

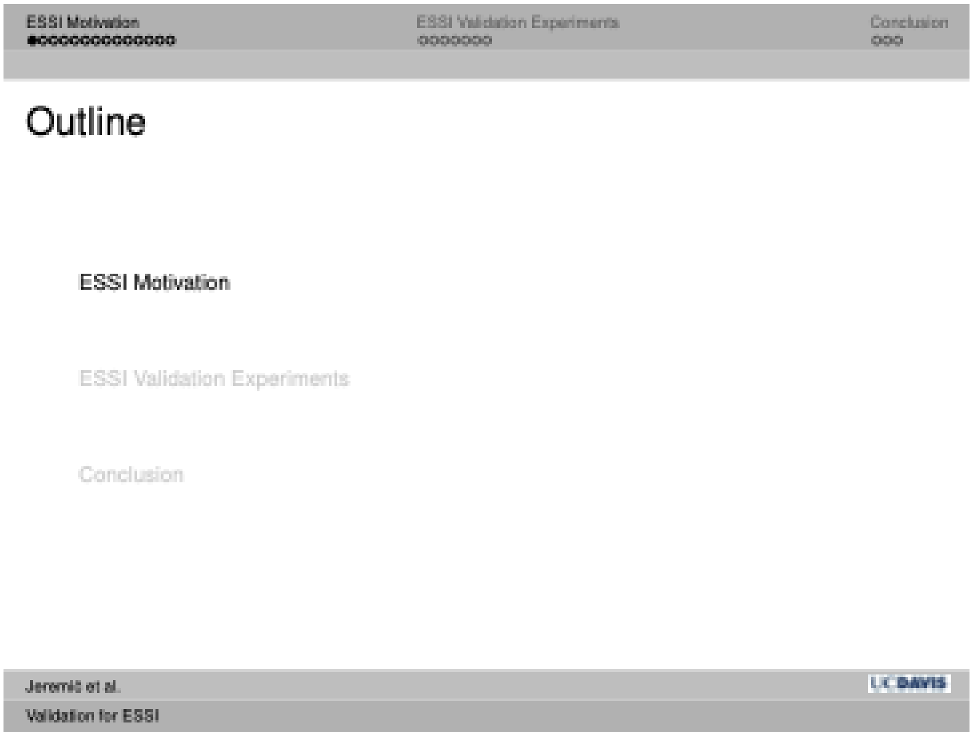
Outline

ESSI Motivation

ESSI Validation Experiments

Conclusion

Jeremić et al.
Validation for ESSI



Outline

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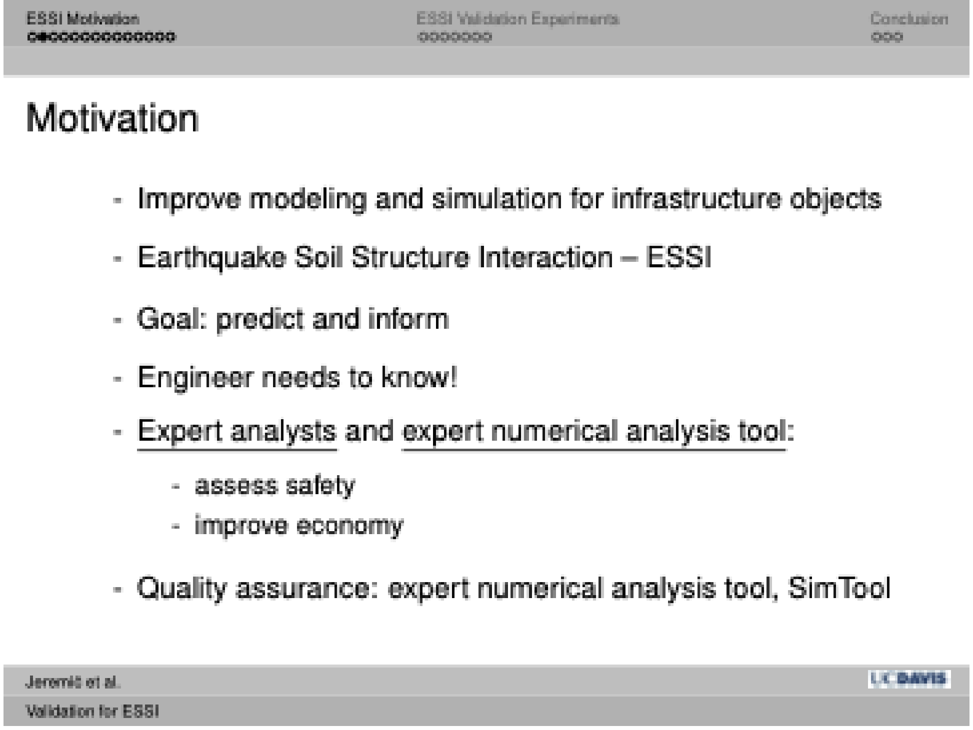
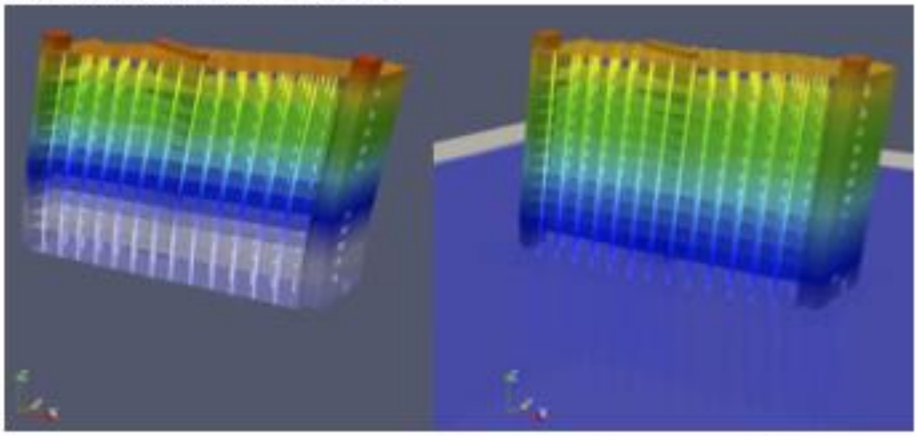


Motivation

- Improve modeling and simulation for infrastructure objects
- Earthquake Soil Structure Interaction – ESSI
- Goal: predict and inform
- Engineer needs to know!
- Expert analysts and expert numerical analysis tool:
  - assess safety
  - improve economy
- Quality assurance: expert numerical analysis tool, SimTool

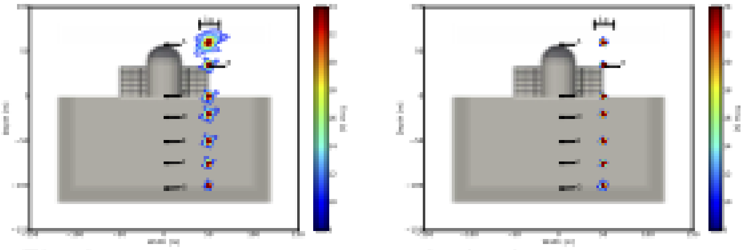
ESSI Analysis is a Must



(MP4) nonESSI ↑ Ventura Hotel, Northridge eq. ↑ ESSI

Inelastic ESSI, Benefits and Detriments

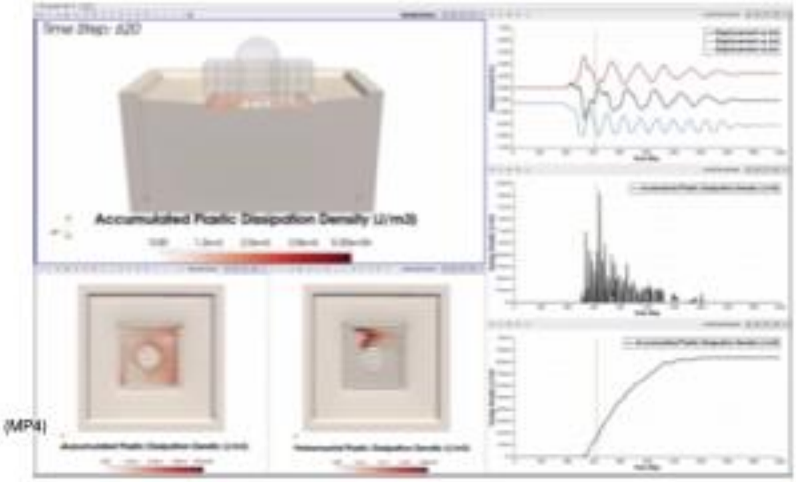
R. Morita, K. Saito, and A. Yuyama. Development and analysis of seismic experience database of structures, systems and components in nuclear power plants based on investigation reports and maintenance records. Nuc.Eng.&Des., 375:111078, 2021.



Elastic

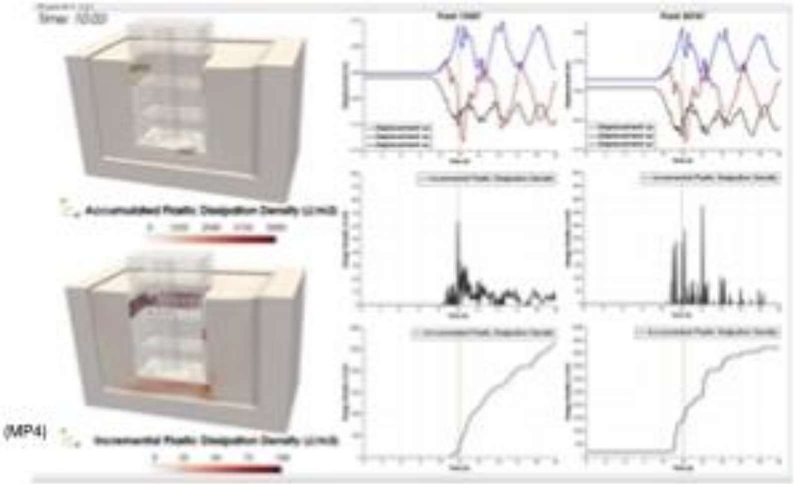
Inelastic

NPP ESSI and Energy Dissipation



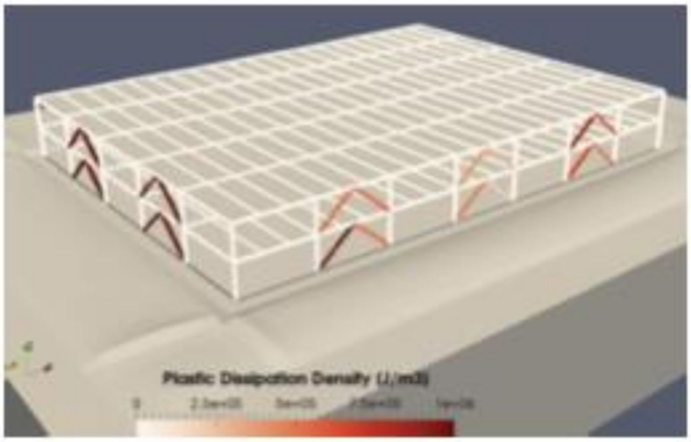
(MP4)

SMR ESSI and Energy Dissipation



(MP4)

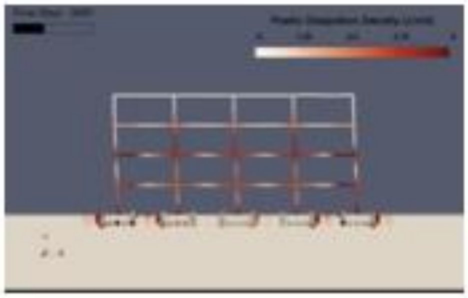
Base Slab Averaging vs Inelasticity



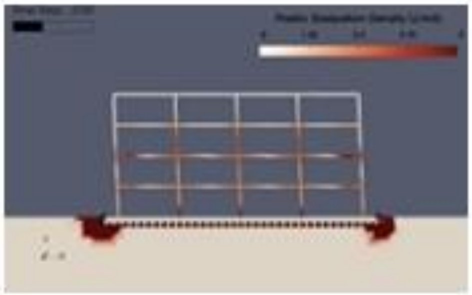
(MP4)

FEMA/ASCE-7

ESSI and Energy Dissipation for Design

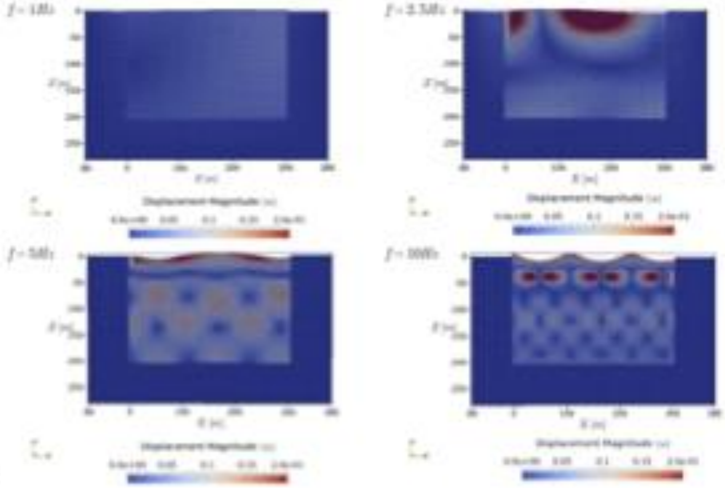


(MP4)



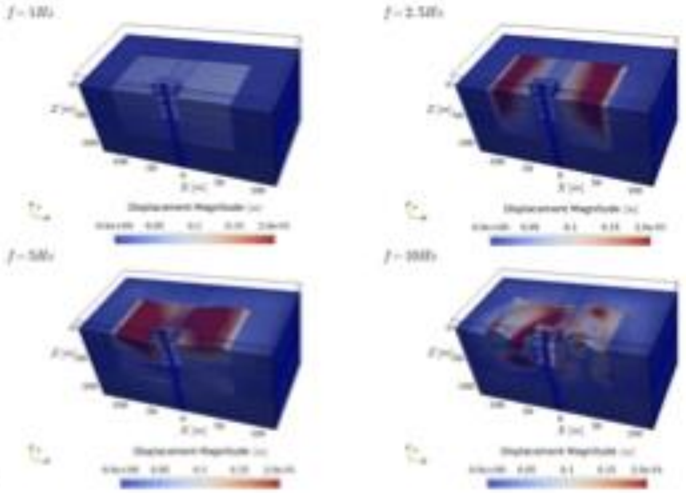
(MP4)

Seismic Motions, Horizontal vs Vertical



(MP4)

SMR ESSI



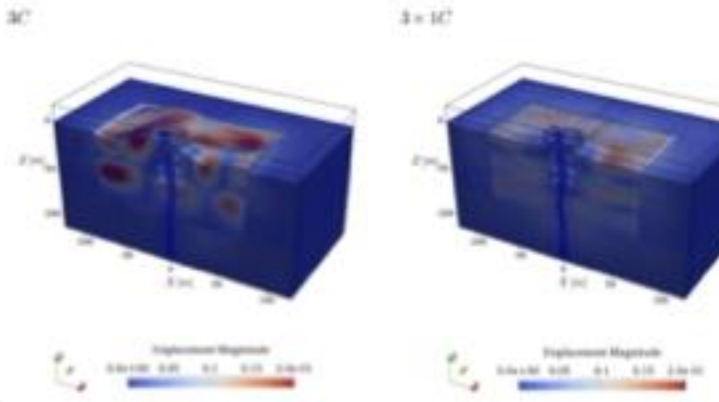
(MP4)

ESSI Motivation
○○○○○○○○○○○●○○○

ESSI Validation Experiments
○○○○○○○

Conclusion
○○○

SMR ESSI, 3C vs 3×1C



(OGV)

Jeremić et al.

Validation for ESSI

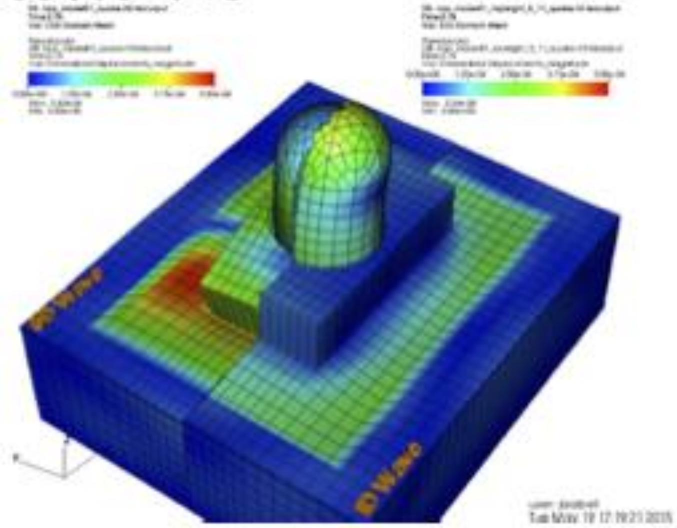
UCDAVIS

ESSI Motivation
○○○○○○○○○○○○●○○

ESSI Validation Experiments
○○○○○○○

Conclusion
○○○

NPP ESSI, 6C vs 1C



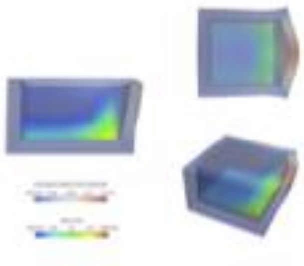
(MP4)

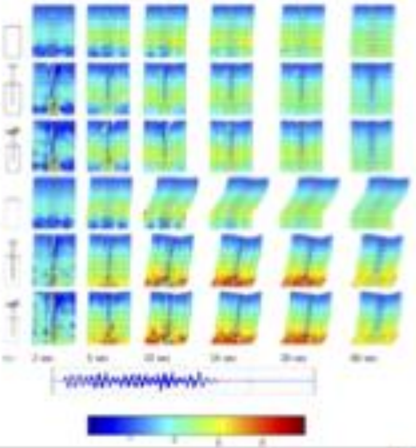
Jeremić et al.

Validation for ESSI

UCDAVIS

ESSI with External/Pool and Internal/Pore Fluid





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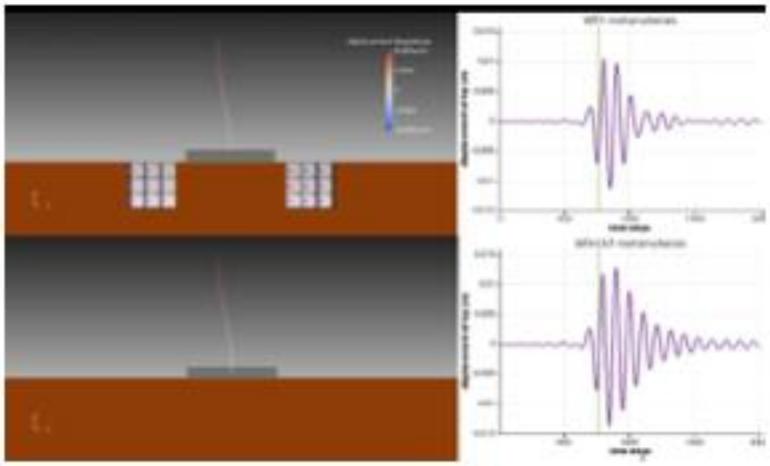
Seismic Shielding, Meta-Materials, ETHZ-UCD



Outline

ESSI Motivation

ESSI Validation Experiments

Conclusion

Verification and Validation

- US-DOE/Sandia NL work, over last 40+ years
- Verification: provides evidence that the model is solved correctly. Mathematics issue.
- Validation: provides evidence that the correct model is solved. Physics issue.
- V&V procedures are the primary means of assessing accuracy in modeling and computational simulations
- V&V procedures used to build confidence and credibility in modeling and computational simulations
- Prediction under Uncertainty: use of computational model to predict the state of SSI system under conditions for which the computational model has not been validated.

V&V: Important References

- W. L. OBERKAMPF, T. G. TRUCANO, AND C. HIRSCH. Verification, Validation and Predictive Capability in Computational Engineering and Physics. In Proc. of the Foundations for V&V in the 21st Century Workshop, 2002, Johns Hopkins Univ.
- W. L. OBERKAMPF, Short Course on V&V in Computational Mechanics, US-DOE – SNL, 2003, Albuquerque, NM.
- I. Babuška and J.T. Oden. Verification and Validation in Computational Engineering and Science: Basic Concepts. Comp. Meth. in App. Mech. and Eng., 193(36-38):4057-4066, 2004.
- W. L. Oberkampf and C. J. Roy. Verification and Validation in Scientific Computing. Cambridge Univ. Press, 2010.

Physical Experiments

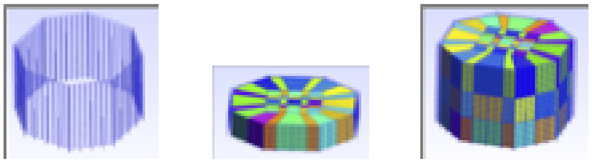
- Traditional Experiments
  - Improve the fundamental understanding of physics involved
  - Improve the mathematical models for physical phenomena
  - Assess component performance
- Validation Experiments
  - Model validation experiments
  - Designed and executed to quantitatively estimate model's ability to simulate well defined physical behavior
  - The simulation tool, SimTool, is the customer!

Validation Experiments

- A validation experiment is jointly designed and executed by experimentalist and computationalist
- A validation experiment is designed to capture the relevant physics
- Validation experiments on unit level problems and progressing up the hierarchy of increasing computational difficulty
- Experimental uncertainty analysis should be developed

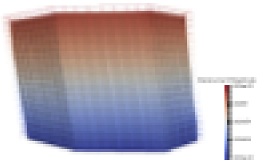
Great Need for Validation Experiments

- Inelastic response, energy dissipation of interfaces
- Base slab averaging vs inelastic interface/contact response
- Deeply embedded SMRs, near surface interface, deep soil
- Energy balance, input and dissipation
- Seismic shielding, meta-materials, seismic trenches
- 6C seismic motions, NPP and SMR response
- ESSI with internal/pore and external/pool fluids



Initial Validation Experiments

- Base slab averaging vs inelastic interface/contact response: use unique box capabilities, apply Love waves and a combination of Love and SV waves, with variable soil profiles and variable surface soil layers to validate/investigate effects of incoherent motions on SSI
- Deeply embedded structure, SMR: validate/investigate inelastic behavior of near surface interface, and inelastic behavior of deep soil



Outline

Acknowledgment

- Collaborators: Feng, Yang, Behbehani, Sinha, Wang, Karapiperis, Wang, Lacoure, Pisanó, Abell, Tafazzoli, Jie, Preisig, Tasiopoulou, Watanabe, Cheng, Yang.

- Funding from and collaboration with the US-DOE, US-FEMA/ATC, US-NRC, US-NSF, CNSC-CCSN, UN-IAEA, ENSI-CH-B&H, Shimizu Corp and UC is greatly appreciated,

Summary

Quality assurance, V&V, for ESSI analysis

Great need for validation experiments for ESSI

Numerical modeling to predict and inform

Engineer needs to know!

`http://real-essi.us/`

MODELING THE SEISMIC RESPONSE OF SPENT NUCLEAR FUEL IN DRY STORAGE

U.S. DEPARTMENT OF ENERGY | Office of NUCLEAR ENERGY

Spent Fuel and Waste Science and Technology (SFWST)

Clean. Reliable. Nuclear.

PNNL-SA-162410

Modeling the Seismic Response of Spent Nuclear Fuel in Dry Storage

Nicholas A. Klymyshyn
Pacific Northwest National Laboratory
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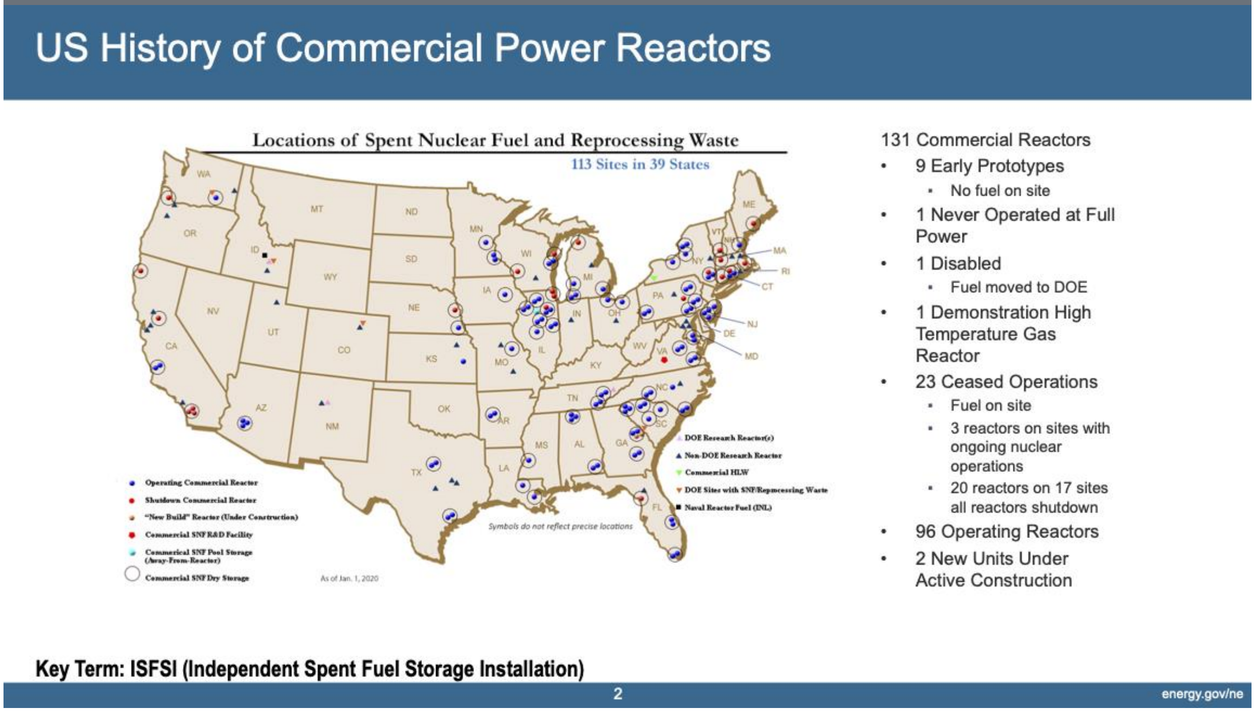
Julio Garcia, Payman K. Tehrani
SC Solutions

May 17, 2021
PEER workshop
(Zoom virtual meeting)

This work was sponsored by the US Dept. of Energy, Office of Nuclear Energy.



US History of Commercial Power Reactors

Locations of Spent Nuclear Fuel and Reprocessing Waste

113 Sites in 39 States

131 Commercial Reactors

- 9 Early Prototypes
  - No fuel on site
- 1 Never Operated at Full Power
- 1 Disabled
  - Fuel moved to DOE
- 1 Demonstration High Temperature Gas Reactor
- 23 Ceased Operations
  - Fuel on site
  - 3 reactors on sites with ongoing nuclear operations
  - 20 reactors on 17 sites all reactors shutdown
- 96 Operating Reactors
- 2 New Units Under Active Construction

Key Term: ISFSI (Independent Spent Fuel Storage Installation)

Multimodal Transportation Test: Measuring Realistic Mechanical Loads on SNF

DOE Spent Fuel & Waste Science and Technology Program (SFWST)

YouTube Video of MMTT

(Note: Use link for video on Youtube - https://www.youtube.com/watch?v=wGKtgrozrGM&feature=youtu.be)

30 cm Cask Drop (1/3 Scale)

- Cask drop testing at BAM in Germany.
- Cask dynamics data used to inform a full scale drop of a SNF assembly at Sandia National Laboratories.
- PNNL modelers are using the data to validate fuel assembly models and perform a parametric study on the potential SNF loads in the general cask drop scenario.

Full Scale SNF Cask Shake Test

- Test Goal: Record the mechanical loading on SNF in storage cask systems during simulated hypothetical earthquakes in the US.
  - Consider earthquakes up to the design basis of SNF dry storage sites in the US.
  - Consider earthquakes up to 300 years of dry storage.
  - Cask system integrity is assured by the regulations – not a concern of this test.
- DOE SFWST Program Goal: Close the Stress Profiles Knowledge Gap
  - Materials testing of SNF needs realistic range of loading.
  - Finite element models need validation data.
  - Test data and analysis will close the knowledge gap.
- International Collaboration and Test Team
  - US, Spain, South Korea, Germany

UC San Diego Large High-Performance Outdoor Shake Table (LHPOST)

Van Den Einde L, Conte JP, Restrepo JI, Bustamante R, Halvorson M, Hutchinson TC, Lai C-T, Lotfizadeh K, Luco JE, Morrison ML, Mosqueda G, Nemeth M, Ozcelik O, Restrepo S, Rodriguez A, Shing PB, Thoen B and Tsampras G (2021) NHERI@UC San Diego 6-DOF Large High-Performance Outdoor Shake Table Facility. Front. Built Environ. 6:580333.doi: 10.3389/fbuil.2020.580333

DOE SFWST Shake Test Team

Key Organizations and Staff

US National Laboratories

Sandia National Laboratories
Elena Kalinina (Lead)
Doug Ammerman

Pacific Northwest National Laboratory
Nick Klymyshyn
Steve Ross

Industry and Contractors

SC Solutions
Norm Abrahamson
Derrick Watkins
Julio Garcia
Payman Tehrani

Gordon Bjorkman

Academia

UCSD
Joel Conte
Jose Restrepo
Koorosh Lotfizadeh

Spain

ENSA
ENRESA

South Korea

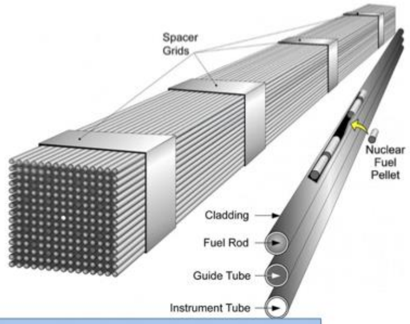
KEPCO NF
KAERI

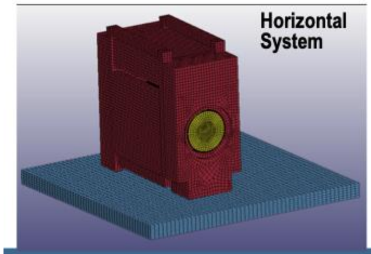
Germany (Potential)

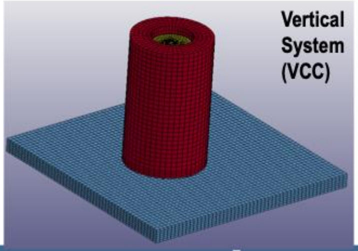
GNS

Test Plan Overview

- Test 50 to 100 ground motions
  - Covers the US (lower 48) up to ISFSI design basis
- Two full scale cask systems (Instrumented assemblies)
  - Horizontal System
  - Vertical Concrete Cask (VCC) system (Fabricated Mockup)
- Potential reduced scale system (contains dummy assemblies)
  - 1/3 scale dual purpose metal cask (ENSA ENUN 32P)
  - 1/3 scale vertical canister system (Fabricated Mockup)



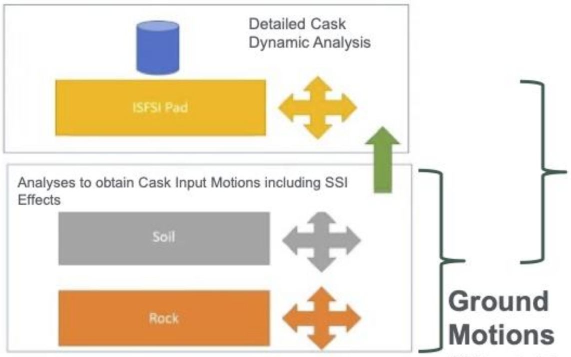




1/3 Scale Cask Model TBD

Input Motions to Shake Table

Modeling Strategy



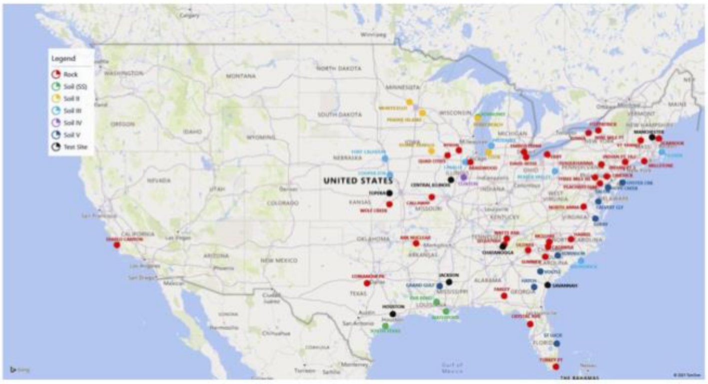
Soil Structure Interaction (SSI) Effects
(Soil + ISFSI Pad
+ Neighbor Casks)

Ground Motions
(Site + Rock + Soil)

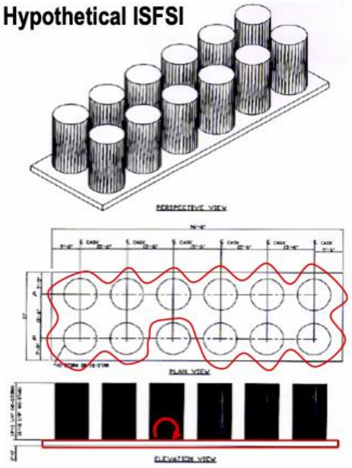
Ground Motions + **SSI Effects** = **Input Motion to Shake Table**

Ground Motions for United States

- Geographic Coverage with Representative Sites: seven sites in CEUS; four sites in WUS
- Wide Range of Site Conditions: Hard Rock, Soft Rock and Soil
- Generic Controlling Earthquake Scenarios (Magnitude and Distance pairs)
- Intensity Amplitudes Covering Hazard from 1E-3 to 1E-5 Annual Frequencies of Exceedance



Soil-Structure Interaction (SSI) Effects

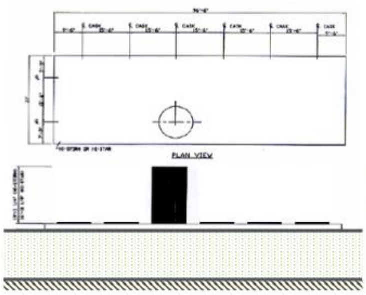


Numerical Simulations to be Combined with Free Field Motion to Account for:

- Underlaying Radiation Soil Damping
- Underlaying ISFSI Pad Flexibility
- Effects of nearby Casks
- Potential Rotational Motions (Shake table can reproduce up to 2 deg of rotational input)

Soil-Structure Interaction (SSI) Effects

- Test set-up will simulate SSI effects through Input Motions to Shake Table
- Supporting Test Plan: Verification and Replication of SSI Effects on a potentially Rigid Shake Table Set-up prior to Experimental Tests



PNNL Modeling Overview

- 2021
  - Supporting Test Plan
  - Shake Table Model Development
    - Report on modeling 2021
- 2022
  - Pre-Test Predictions (Shake Table Scenario)
    - Supporting Test Safety!
    - Report on pretest predictions modeling 2022
  - Test in July
  - Data Collection and Distribution
- 2023
  - Model Validation and Refinement Using Test Data
    - Report on validation and refinement 2023
    - Shake table configuration
  - SNL analysis of data and report
  - Potential NEUP (Nuclear Energy University Program)
- 2024
  - Model Application to Realistic Systems
    - How would real, complete systems respond to test conditions?
    - ISFSI configuration
    - Final Report in 2024

Assessment:

- Do we have a complete technical story?
- Do we need soil box testing to close the knowledge gap?

Accurate model predictions require understanding the key physics.

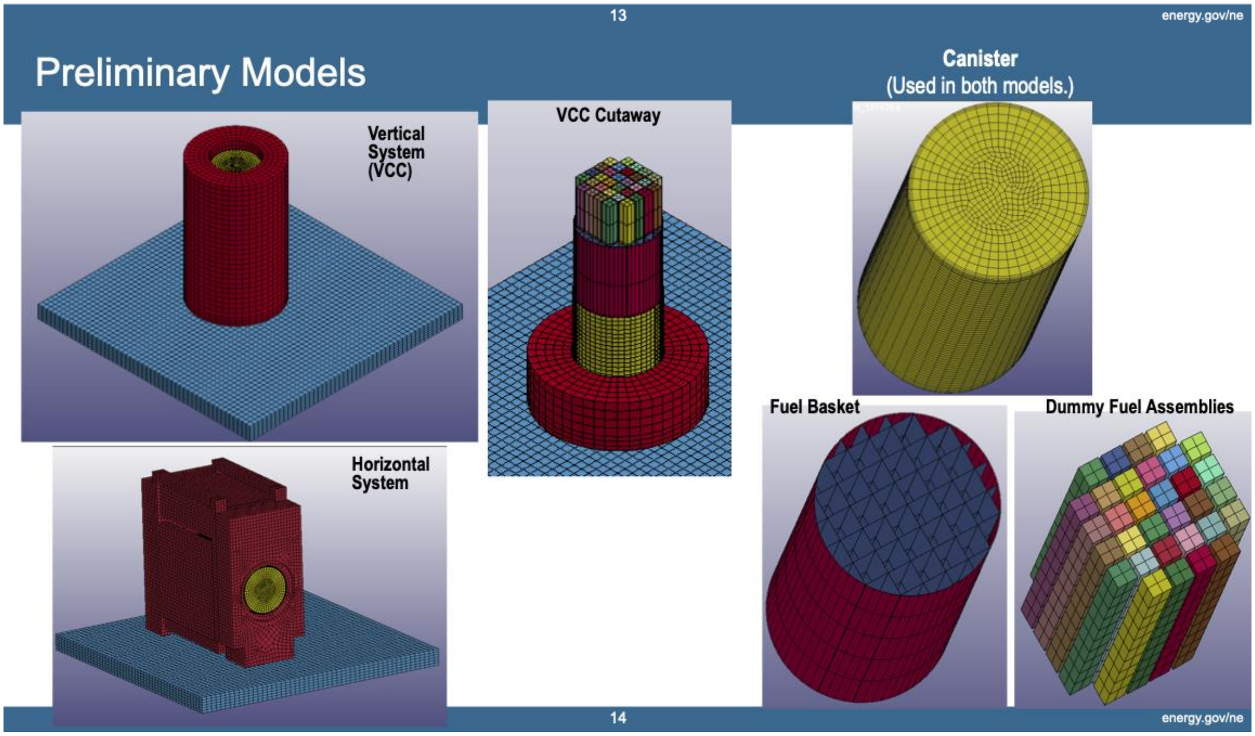
Key Questions:

- What are the mechanical loads on the SNF? (Quantify them.)
- Will a cask tip over?
- Will a cask impact another cask on the pad?
- Will a cask walk (slide/roll) off the edge of a pad and tip over?

We Expect the Answers Depend On:

- Pad Motion
- Friction
- Damping
- Contact
- Gross Pad Deflection
- Local Pad Deflection
- Soil Structure Interaction

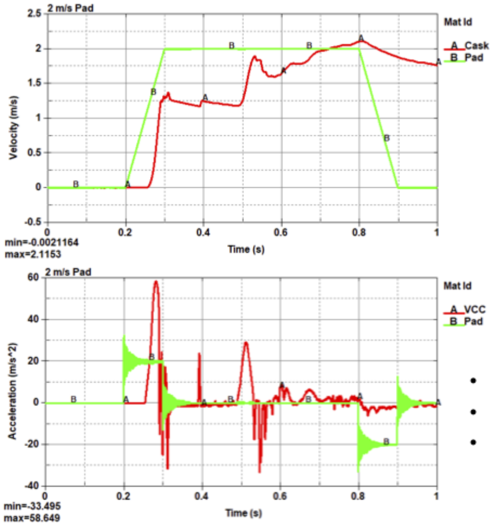
The test data will inform us about most of these phenomena. Which ones are most important?



Preliminary Models

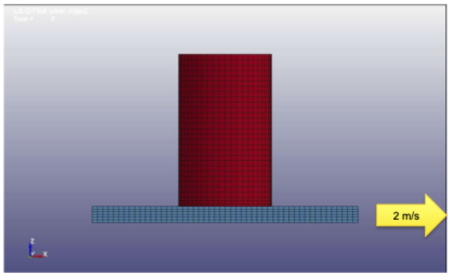
Preliminary VCC Model: 1D Horizontal Motion

Green = Applied Pad Velocity
Red = Resulting Cask Velocity



Model: Full Scale Cask, 1D Horizontal Sliding

- No Tip Over
- Rocking/Rolling
- Chaotic/Unstable Response
- NUREG/CR-6865



- 2 m/s Applied Velocity
- ~2 g Step Acceleration
- Friction Coefficient: ~0.01

- 268 mm Max Lift-up
- 3500 mm VCC Diameter
- 4.3° Lift-up Angle

NUREG/CR-6865 – Cask Rocking and Rolling

As the cask rocks back and forth, energy is absorbed every time the cask impacts the pad. This can be a significant energy dissipation mechanism, and the type of soil underlying the pad can have a noticeable effect on the amount of energy dissipated. This mechanism is believed to be the most important soil-structure interaction effect after the cask begins to tip. It is important to note that the cylindrical cask can assume either a rocking motion or a rolling motion. Significant energy is dissipated if the cask is rocking back and forth, but very little energy is dissipated in the rolling motion.

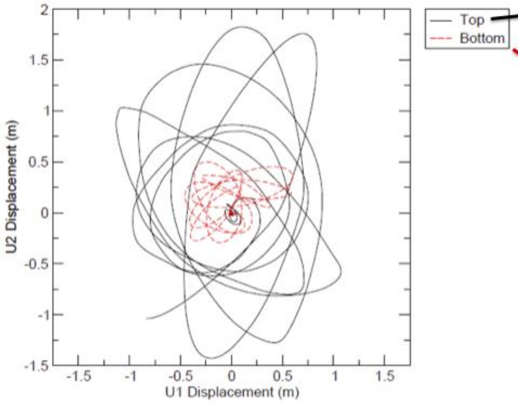


Figure 5.9: Lateral Displacement Trajectories for Cylindrical Cask Top and Bottom, Iran Tabas Earthquake, NUREG/CR-0098 Spectral Shape, PGA=1.0 g, Stiff Soil Profile, Cask/Pad μ=0.55

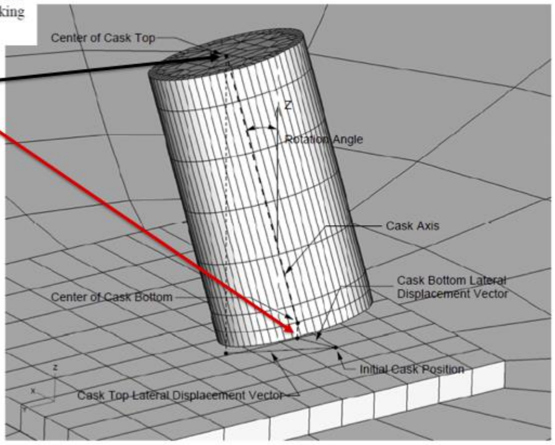
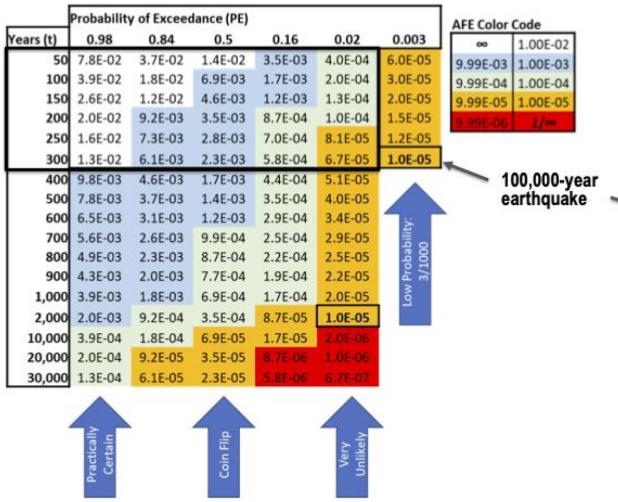


Figure 5.8: Explanation of Key Response Quantities

Note: NUREG/CR model predicts cask tip over in cases as low as 0.6g PGA.

Seismic Hazard Range of Interest for SNF Cask Shake Test

Annual Frequency of Exceedance
(AFOE or AFE)



Years (t)	Probability of Exceedance (PE) 0.98	0.84	0.5	0.16	0.02	0.003
50	7.8E-02	3.7E-02	1.4E-02	3.5E-03	4.0E-04	6.0E-05
100	3.9E-02	1.8E-02	6.9E-03	1.7E-03	2.0E-04	3.0E-05
150	2.6E-02	1.2E-02	4.6E-03	1.2E-03	1.3E-04	2.0E-05
200	2.0E-02	9.2E-03	3.5E-03	8.7E-04	1.0E-04	1.5E-05
250	1.6E-02	7.3E-03	2.8E-03	7.0E-04	8.1E-05	1.2E-05
300	1.3E-02	6.1E-03	2.3E-03	5.8E-04	6.7E-05	**1.0E-05**
400	9.8E-03	4.6E-03	1.7E-03	4.4E-04	5.1E-05	
500	7.8E-03	3.7E-03	1.4E-03	3.5E-04	4.0E-05	
600	6.5E-03	3.1E-03	1.2E-03	2.9E-04	3.4E-05	
700	5.6E-03	2.6E-03	9.9E-04	2.5E-04	2.9E-05	
800	4.9E-03	2.3E-03	8.7E-04	2.2E-04	2.5E-05	
900	4.3E-03	2.0E-03	7.7E-04	1.9E-04	2.2E-05	
1,000	3.9E-03	1.8E-03	6.9E-04	1.7E-04	2.0E-05	
2,000	2.0E-03	9.2E-04	3.5E-04	8.7E-05	**1.0E-05**	
10,000	3.9E-04	1.8E-04	6.9E-05	1.7E-05	2.0E-06	
20,000	2.0E-04	9.2E-05	3.5E-05	8.7E-06	1.0E-06	
30,000	1.3E-04	6.1E-05	2.3E-05	5.8E-06	6.7E-07	

AFE Color Code

∞	1.00E-02
9.99E-03	1.00E-03
9.99E-04	1.00E-04
9.99E-05	1.00E-05
9.99E-06	1/∞

100,000-year earthquake

Columbia Generating Station, SNF Shake Test Range

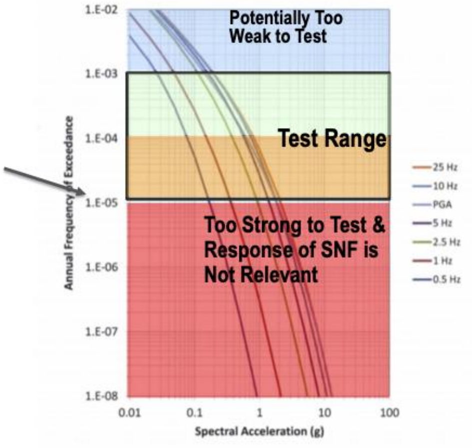


Figure 2.2.2-1: Mean Base Rock Hazard Curves for Oscillator Frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at Columbia Generating Station (PNNL, 2014) at 5% Spectral Damping

Model Development Case: Columbia Generating Station

RESPONSE TO NRC REQUEST FOR INFORMATION PURSUANT TO TITLE 10 OF THE CODE OF FEDERAL REGULATIONS 50.54(F) REGARDING RECOMMENDATION 2.1 OF THE NEAR-TERM TASK FORCE RECOMMENDATION 2.1: SEISMIC FOR SEISMIC HAZARD REEVALUATION AND SCREENING FOR RISK EVALUATION
Enclosure

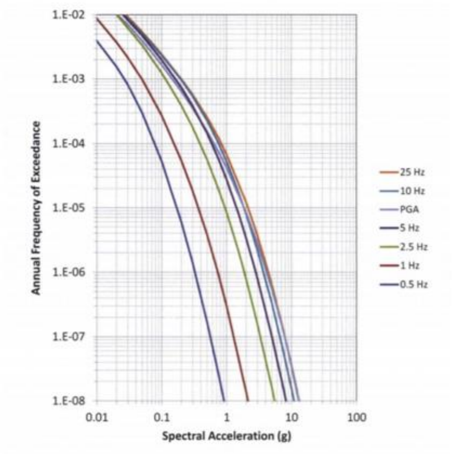


Figure 2.2.2-1: Mean Base Rock Hazard Curves for Oscillator Frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at Columbia Generating Station (PNNL, 2014) at 5% Spectral Damping

Hazard Curves define the earthquake spectra for a location over a broad range of probability.

Annual Frequency of Exceedance (AFE, AFOE) relates spectra to a yearly probability of occurrence.

Spectra define the characteristics of earthquakes: amplitude, frequency content, etc.

Ground Motion Time Histories are created (or selected) to match the spectrum at a particular AFE.

SNF Shake Test Range

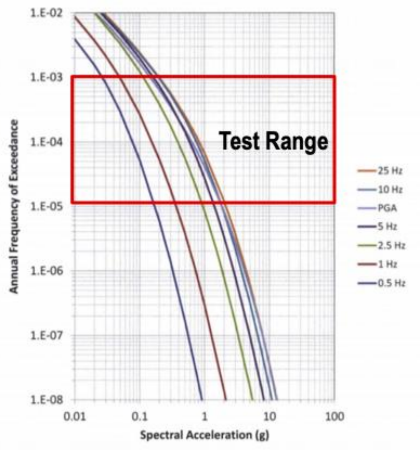
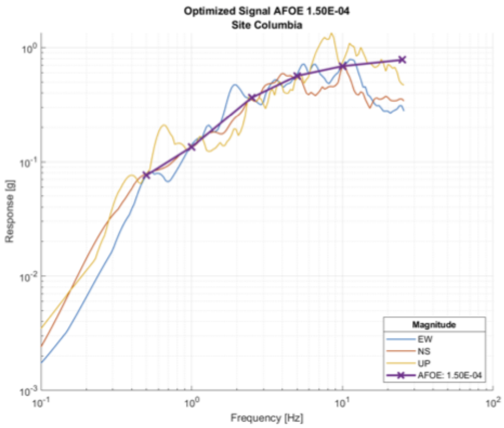
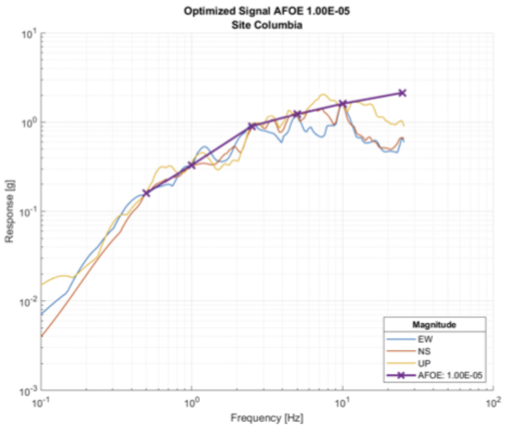


Figure 2.2.2-1: Mean Base Rock Hazard Curves for Oscillator Frequencies of 0.5, 1, 2.5, 5, 10, 25 and 100 Hz at Columbia Generating Station (PNNL, 2014) at 5% Spectral Damping

Model Development Ground Motions: Modify Historical Earthquakes to Match Target Spectra

Methodology: Select an AFOE value. Construct the AFOE target Spectra from site hazard information. Search a short database of earthquake data to find a starting time history. Adjust the time history (signal) Fourier components by hand to optimize agreement with target spectra. Matching 25 Hz is low priority because base data is sampled at 50 Hz.

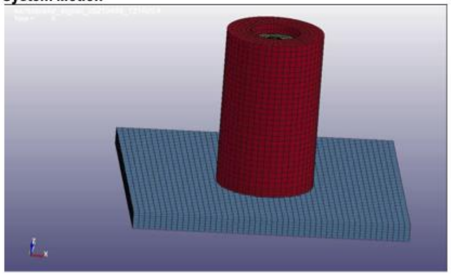




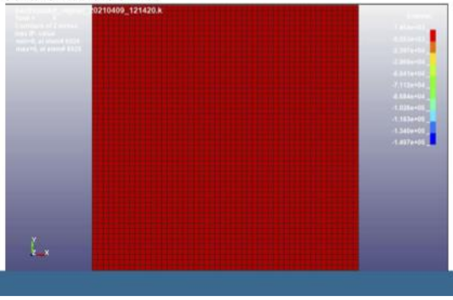
Model Development Case: Columbia Generating Station, AFE 1.5E-4

- AFE 1.5E-4
  - 6,700 Year Return Period
  - ~2% chance of exceedance in 150 years
- Base Rock Motion Applied to Pad
  - No Soil/Structure Interaction
- Low Friction (~0.01)
- 0.29 g (peak horizontal)
- 0.18 m/s (peak horizontal)
- ~27 mm Cask Relative Sliding
- Shifting Weight Observed in Contact Stress

System Motion



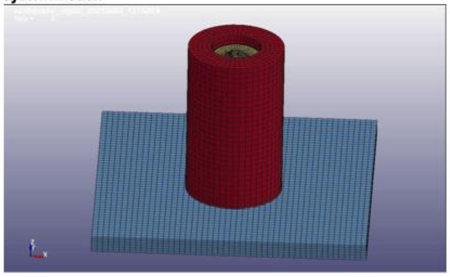
Contact Pressure on Pad



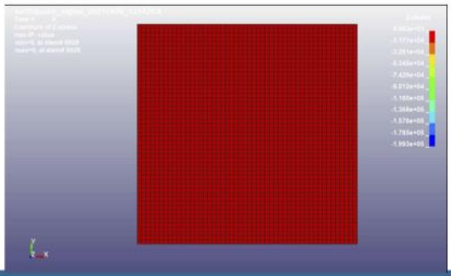
Model Development Case:
Columbia Generating Station, AFE 1E-5

- AFE 1E-5
  - 100,000 Year Return Period
  - 0.3% chance of exceedance in 300 years
- Base Rock Motion Applied to Pad
  - No Soil/Structure Interaction
- Low Friction (~0.01)
- 0.38 g (peak horizontal)
- 0.32 m/s (peak horizontal)
- ~120 mm Cask Relative Sliding
- Shifting Weight Observed in Contact Stress
- ~5 mm Max Lift-up (<0.1°)

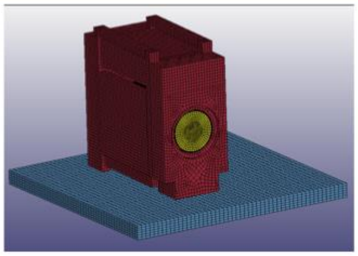
System Motion

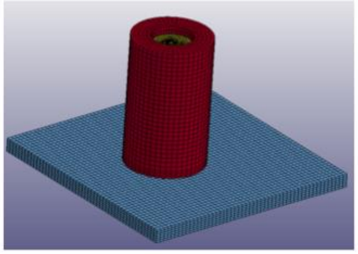


Contact Pressure on Pad



PNNL Model Development Next Steps





1/3 Scale Cask Model TBD

Upgrade the plain concrete pad model.

Detailed Shake Table Model for As-Tested Configuration:
- Pre-Test Predictions
- Model Validation

Detailed ISFSI & Soil Column Model for Realistic Dry Storage Analysis:
- Connect/Reconcile with Shake Table Motion
- Closing the Knowledge Gap

Conclusions

- DOE SFWST program is preparing a full-scale shake table test of SNF casks.
  - The goal is to determine SNF mechanical loads in a realistic range of earthquakes.
  - Not interested in canister safety or integrity, which is already assured by the regulations.
- Shake table inputs being developed by SC Solutions.
  - Broad range of ground motion that represents US sites. (1E-3 to 1E-5 AFE)
  - Soil-Structure interaction will be considered for a full ISFSI pad on soil.
- PNNL explicit finite models focus on the pad, cask, and SNF response.
  - To be validated with test data.
- Next modeling steps:
  - Pretest predictions
  - Model validation with test data
  - Model application to irradiated, ISFSI storage configuration

Questions?

Clean. **Reliable. Nuclear.**

DESIGN VERIFICATION OF LARGE SCALE LAMINAR SOIL BOX

Design Verification of Large Scale Laminar Soil Box

Jenna Wong, PhD, PE
Assistant Professor - San Francisco State Univ.
Faculty Affiliate - LBNL

DOE PEER Workshop

May 17, 2021

San Francisco State University

Fast Facts

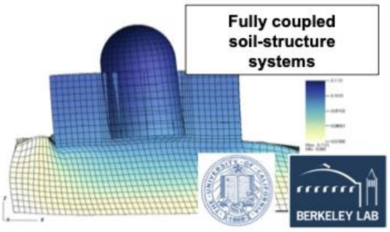
- Hispanic Serving Institution (HSI)
- Primarily Undergraduate Institution (PUI)

Graduate Student Researchers

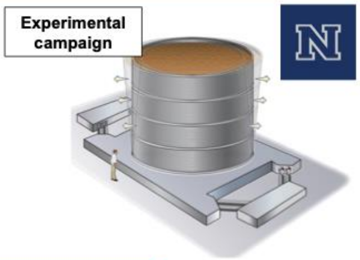
Vanessa Duran

Sepehr Shakeri

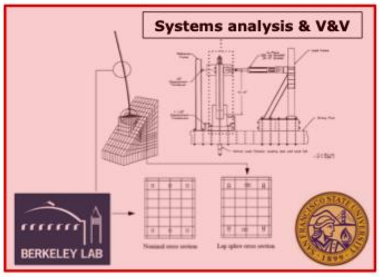
Multi-institution, Multidisciplinary Team



Fully coupled soil-structure systems

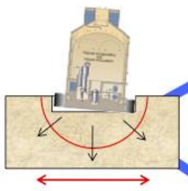


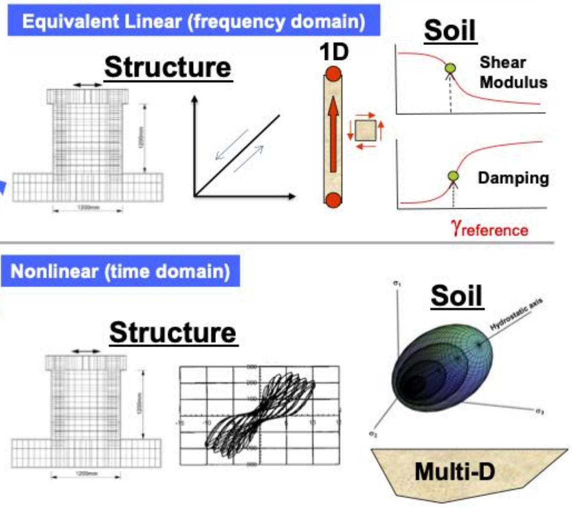
Experimental campaign



Systems analysis & V&V

Our project is assisting in the pursuit a fully nonlinear framework for performance-based design





- Enhanced understanding for beyond design basis events
- Full realization of performance-based design
- More realism for truly nonlinear systems

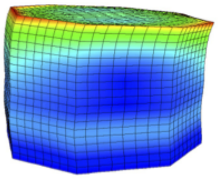
Study Breakdown

Study Goals: For this large scale laminar soil box, we wanted:

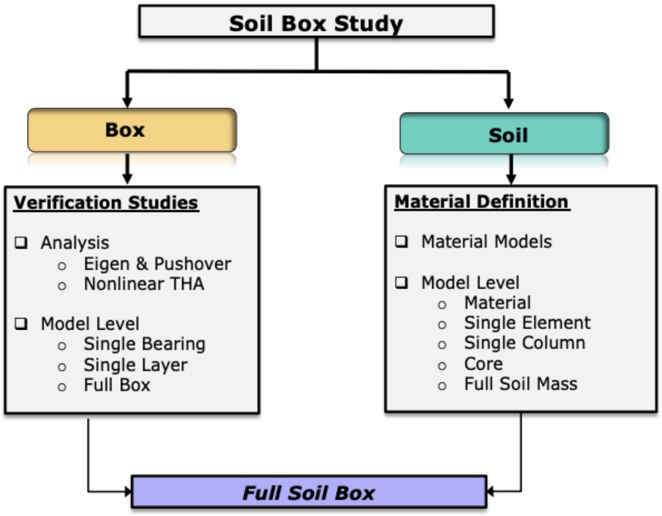
- ❑ To ***provide*** an independent design verification
- ❑ To ***explore*** effectiveness of reduced order models for parametric studies
- ❑ To ***support*** commissioning activities and experimental design with a simplified reduced-order model as a complement to the large 3D models of the soil box

↓

Method: Systematic approach characterizing the soil box's dynamics and conducting inter-code comparisons for linear and nonlinear analysis



Study Breakdown



Soil Box Study

Box

Verification Studies

- ❑ Analysis
  - Eigen & Pushover
  - Nonlinear THA
- ❑ Model Level
  - Single Bearing
  - Single Layer
  - Full Box

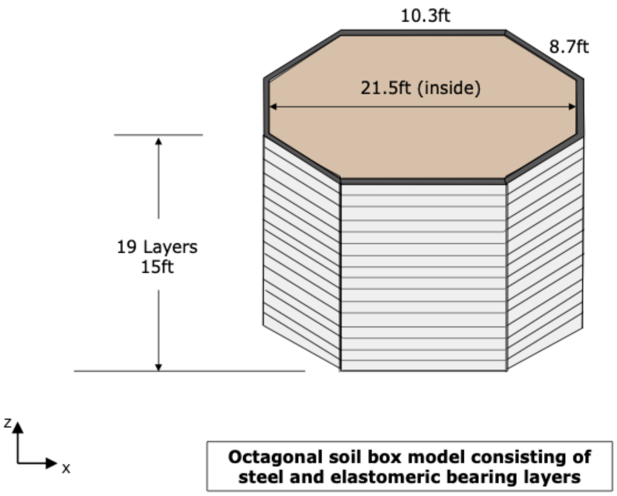
Soil

Material Definition

- ❑ Material Models
- ❑ Model Level
  - Material
  - Single Element
  - Single Column
  - Core
  - Full Soil Mass

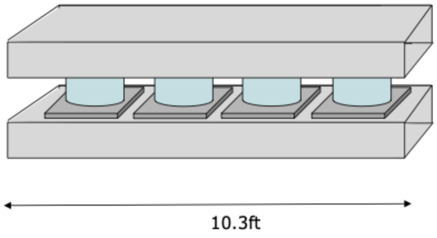
Full Soil Box

Soil Box Geometry



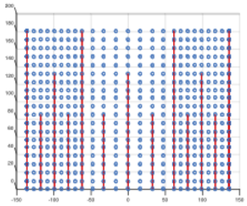
Octagonal soil box model consisting of steel and elastomeric bearing layers

Soil Box Geometry



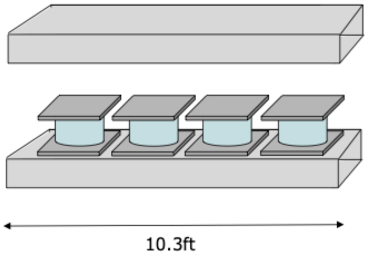
Elastomeric Bearings per layer will distributed throughout structure as follows:

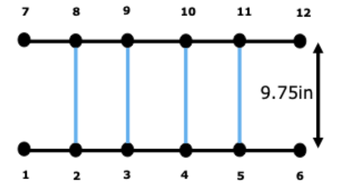
32 bearings in Layers 1-9
16 bearings in Layers 10-14
8 bearings in Layers 15-19



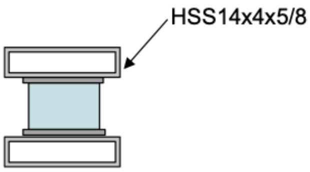
Name	Type	Bearing Outer Dameter	Effective Shear Stiffness k/in			Compression Stiffness	Tension Stiffness	k_{tor}	k_{theta}
			100%	25%	7%	k/in	k/in	k-in/rad	k-in/rad
RB1	A	8"	1.13	1.21	1.42	224	170	2.32	252
RB2	B	11"	2.76	3.36	3.70	890	757	4.99	2,485
RB3	C	11"	4.73	5.62	6.77	1,541	1,154	9.59	4,777
RB4	D	11"	8.35	10.11	12.88	2,020	1,679	16.90	8,414
RB5	E	11"	10.81	13.08	16.65	3,525	2,931	21.89	10,899

Soil Box – Model Section





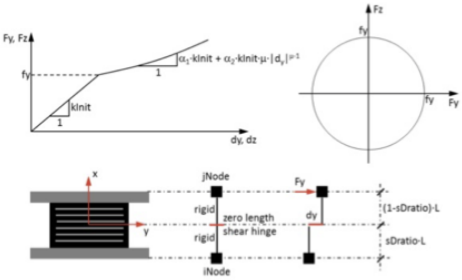
Model representation using elastic beam elements for HSS tubing and Connecting Plates (black) and elastomeric elements for the Bearings (blue)



Steel Uniaxial Material

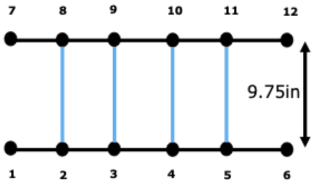
- Representative A992

Soil Box – Model Section



Constitutive model for the Elastomeric Bearing (Plasticity) element in OpenSees

[Element Developed by: Andreas Schellenberg, University of California, Berkeley]



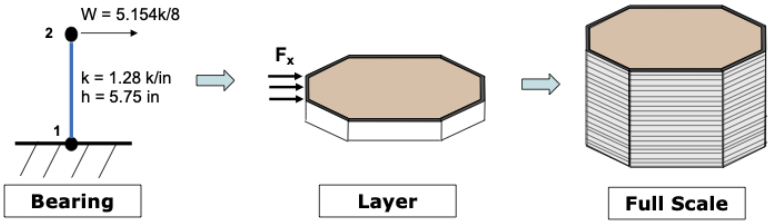
Model representation using elastic beam elements for HSS tubing and Connecting Plates (black) and elastomeric elements for the Bearings (blue)

FEM element

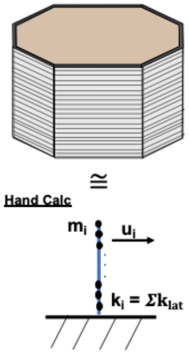
Elastomeric Bearing (Plasticity)

- Properties defined to enforce linear-elastic behavior
- Captures P-Delta Effects
- Element does not contribute to Rayleigh damping

Box Analysis – Dynamic Characterization



Box Analysis – Dynamic Characterization



Eigen Analysis w/ elemental mass

	Hand Calc *Fixed Base* *No Rot DOFs*	**ESSI** *Fixed Base* *Rot DOFs*	**UNR (SAP)** *Fixed Base* *Rot DOFs*	**OpenSees** *Fixed Base* *Rot DOFs*
T_1 (s)	0.8071	0.7854	0.7286	0.7301

Mode	**Hand Calc** *Fixed Base* *No Rot DOFs*	**ESSI** *Fixed Base* *Rot DOFs*	**UNR SAP2000** *Fixed Base* *Rot DOFs*	**OpenSees** *Fixed Base* *Rot DOFs*
1	0.8071	0.7854	0.7286	0.7301
2	0.3870	0.7854	0.7286	0.7301
3	0.2447	0.7476	0.6974	0.7010
4	0.1769	0.3805	0.3288	0.3833
5	0.1400	0.3805	0.3288	0.3833
6	0.1184	0.3620	0.3131	0.3201
7	0.1071	0.2411	0.2044	0.2443
8	0.0924	0.2411	0.2044	0.2443
9	0.0805	0.2307	0.1959	0.2307
10	0.0712	0.1869	0.1502	0.1721
11	0.0604	0.1775	0.1502	0.1721
12	0.0564	0.1731	0.1435	0.1392

Soil Analysis – Dynamic Characterization

#	**t** *ft*	**d** *ft*	**γ** *pcf*	**ϕ** *deg*	**k_o**	**σ'$_v$** *psf*	**σ'$_m$** *psf*	**K_{2max}**	**G_{max}** *psf*	**ρ** *psf*	**V_s** *ft/s*	**f_{max}** *Hz*	**ν**	**K_b** *psf*	**E_o** *psf*
1	1	0.5	120	37	0.40	60	35.9	61	365631	3.73	313.1	78.3	0.30	792200	950640
2	1	1.5	120	37	0.40	180	107.8	61	633291	3.73	412.1	103.0	0.30	1372131	1646557
3	1	2.5	120	37	0.40	300	179.6	61	817575	3.73	468.2	117.1	0.30	1771413	2125696
4	1	3.5	120	37	0.40	420	251.5	61	967368	3.73	509.3	127.3	0.30	2095964	2515157
5	1	4.5	120	37	0.40	540	323.4	61	1096892	3.73	542.3	135.6	0.30	2376600	2851920
6	1	5.5	120	37	0.40	660	395.2	61	1212660	3.73	570.2	142.6	0.30	2627430	3152916
7	1	6.5	120	37	0.40	780	467.1	61	1318300	3.73	594.5	148.6	0.30	2856318	3427581
8	1	7.5	120	37	0.40	900	538.9	61	1416082	3.73	616.2	154.0	0.30	3068177	3681813
9	1	8.5	120	37	0.40	1020	610.8	61	1507534	3.73	635.8	158.9	0.30	3266324	3919589
10	1	9.5	120	37	0.40	1140	682.6	61	1593748	3.73	653.7	163.4	0.30	3453120	4143744
11	1	10.5	120	37	0.40	1260	754.5	61	1675531	3.73	670.3	167.6	0.30	3630316	4356380
12	1	11.5	120	37	0.40	1380	826.3	61	1753504	3.73	685.7	171.4	0.30	3799258	4559109
13	1	12.5	120	37	0.40	1500	898.2	61	1828154	3.73	700.1	175.0	0.30	3961000	4753200
14	1	13.5	120	37	0.40	1620	970.0	61	1899873	3.73	713.7	178.4	0.30	4116392	4939670
15	1	14.5	120	37	0.40	1740	1041.9	61	1968982	3.73	726.6	181.6	0.30	4266127	5119353

🟨 Assumed/input

🟦 Derived/calculated

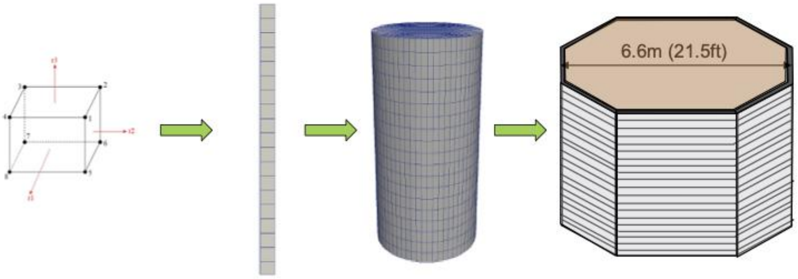
# = Layer number
t = Layer thickness
d = Depth to mid layer

γ = Soil unit weight
ϕ = Angle of internal friction of soil
ν = Poisson's ratio

k_o = Coefficient of lateral earth pressure at rest = $1 - \sin\varphi$
σ'$_v$ = Vertical effective stress = d * γ
σ'$_m$ = Mean effective stress = σ'$_v$ (1+2 K_o)/3 = $\sigma_v' \frac{(1+2\,K_o)}{3}$
K_{2max} = Shear modulus number (Seed and Idriss, 1970)
G_{max} = Maximum (small strain) shear modulus
= 1000 K_{2max} (σ'$_m$)^0.5 = $1000\,K_{2max}\sqrt{\sigma_m'}$

ρ = Soil mass density = γ / g, where g is the acceleration of gravity
V_s = Shear wave velocity = (G_{max}/ρ)^0.5 = $\sqrt{\frac{G_{max}}{\rho}}$
f_{max} = Fundamental frequency of the layer = Vs/(4t) = $\frac{v_s}{4\,t}$
K_b = Bulk modulus = (2 G (1 + ν)) / (3 (1 – 2 ν)) = $\frac{2\,G\ \ (1+\upsilon)}{3\,(1-2\upsilon)}$
E_o = Initial (max) Young's modulus = 2 (1+ν) G_{max}

Soil Analysis – Dynamic Characterization



Single Brick

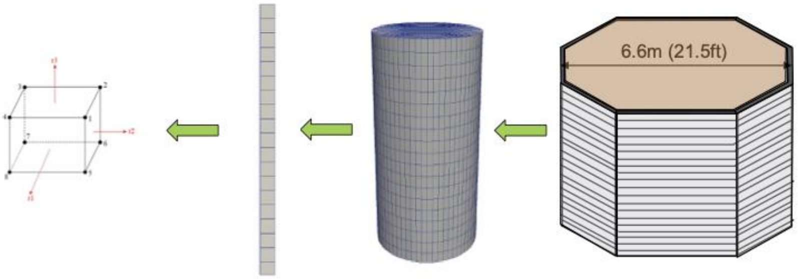
Column

Core

Full System

Linear Elastic AND Nonlinear Soil Materials

Soil Analysis – Dynamic Characterization



Single Brick

8 nodes & 1 element
Run time: 1 min

Column

64 nodes & 15 elements
Run time: 15 min

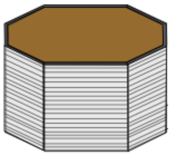
Core

2,900 nodes & 2,508 elements
Run time: 6 hr to 24hr

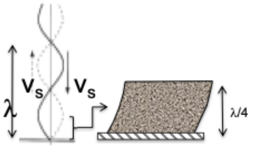
Full System

8,500 nodes & 7,920 elements
Run time: days to weeks

Soil Analysis – Dynamic Characterization



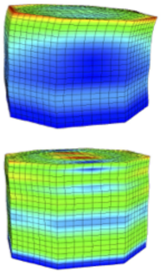
Linear Elastic Material for Soil



Eigen Analysis w/ elemental mass

Mode	UNR LS DYNA *Fixed Base Rot DOFs*	OS *Fixed Base Rot DOFs*	ESSI *Fixed Base Rot DOFs*
1	0.101	0.1129	0.1127
2	-	0.1128	0.1127
3	-	0.1102	0.1102
4	-	0.0634	0.0633
5	-	0.0633	0.0632
6	-	0.0485	0.0485
7	-	0.0479	0.0478
8	-	0.0478	0.0478
9	-	0.0437	0.0437
10	-	0.0437	0.0437
11	-	0.0435	0.0435
12	-	0.0405	0.0405

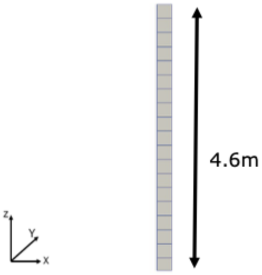
Excellent result as it shows the box is "invisible" to the soil



Check against standing wave equation

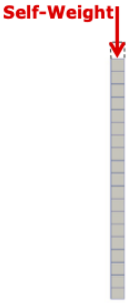
	Shear Modulus G_{max} (ksf)	Density ρ (lb-s²/ft⁴)	Fundamental Freq. f_1	Fundamental Period T_1	OS Fundamental Period T_1
Depth = 7ft	1.3e6	3.728	9.842 Hz	0.1016 s	0.1129

Soil Analysis – Reduced Order Analysis

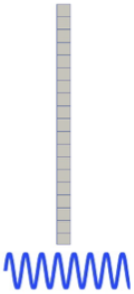


Two Stage Analysis

1st Stage – Gravity Initialization



2nd Stage - Nonlinear THA

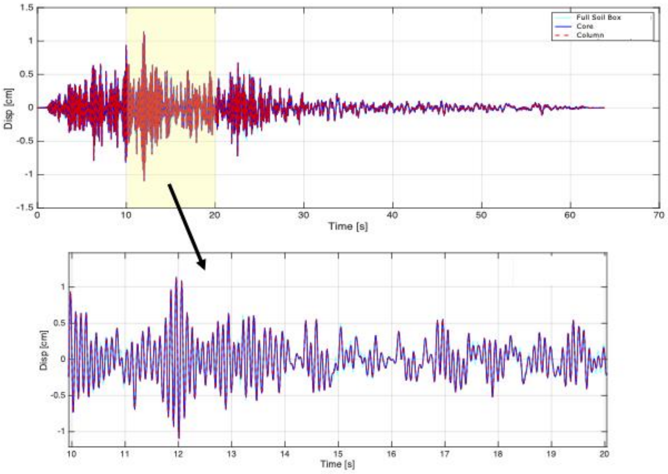


Model Constraints
- **Equaldof in x, y, and z dir. For EACH layer**
- **Base nodes fixed**

Damping
2% - Soil
Rayleigh Damping
Anchored at 1st and 3rd Modes

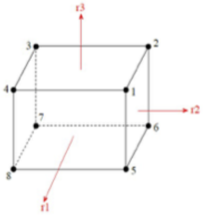
Soil Analysis – Reduced Order Analysis

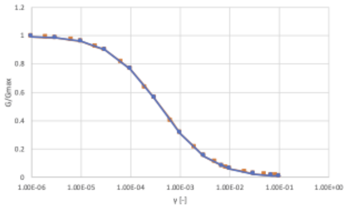
Comparison of Full Scale and Reduced Order Models
Linear Elastic Soil Material



Soil Analysis – Nonlinear Soil Material

PRESSURE INDEPENDENT MULTIYIELD MATERIAL





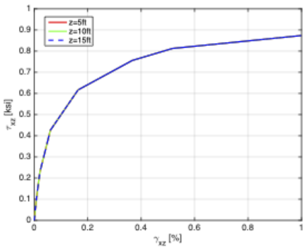
FEM element

StdBrick

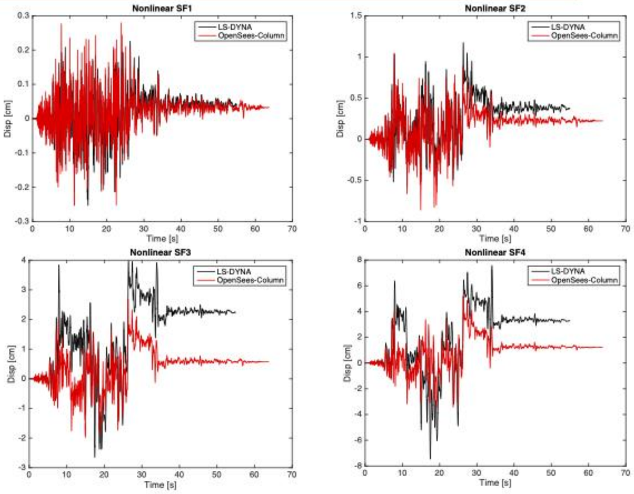
Element Outputs:

- *6 components of total strain*
- *6 components of plastic strain*
- *6 components of stress*

for all (8) Gauss Points



Soil Analysis – Nonlinear Soil Analysis

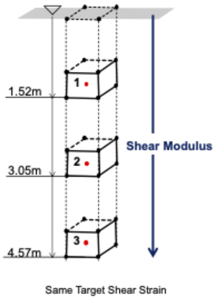


Damped Scenario
2% - Soil
Rayleigh Damping
Anchored at 1st and 3rd Modes

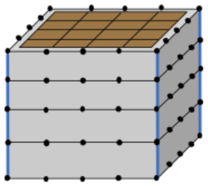
Gravity Initiated

Validation against experimental results will be crucial in better understanding the variances in numerical results.

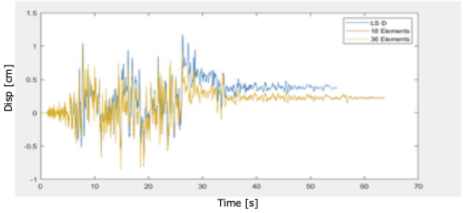
Additional Analyses



Soil Material Models
- Von Mises
- Drucker Prager
- Multi-Yield

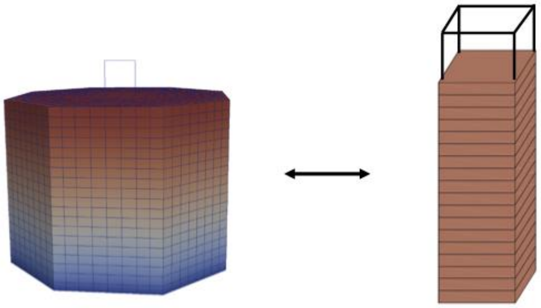


Contact Surfaces



Sensitivity Analyses
- Reduced order model definition
- Soil material parameters
- Numerical modeling approaches

Looking Ahead…



Utilizing reduced order analyses to explore commissioning structures

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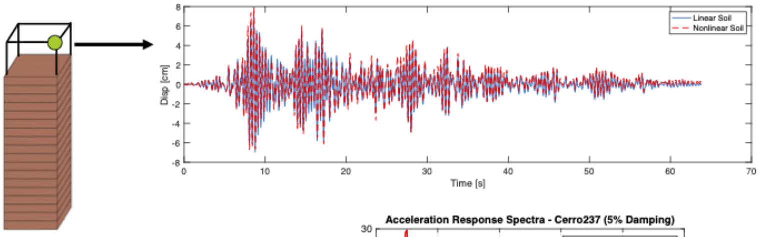
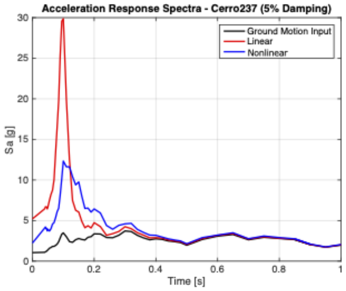
Looking Ahead…

10% difference between linear and nonlinear max displacements



Objectives:
- Evaluate the structural variations for linear and nonlinear soil materials
- Identify ideal systems for commissioning efforts



Conclusions

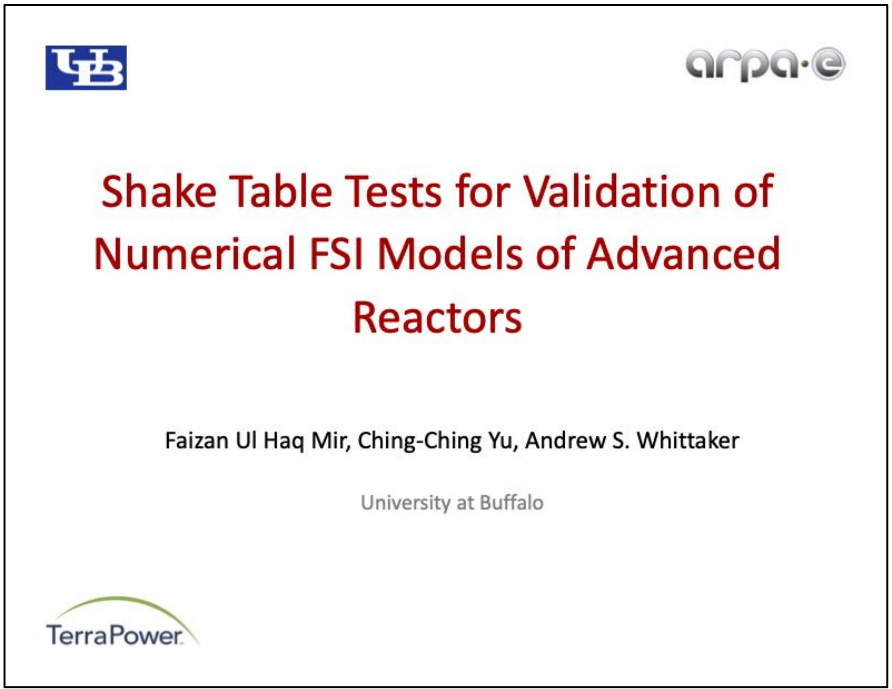
- Design verification of a large laminar soil box is a complicated process
- Data for SSI numerical model validation is still limited emphasizing need for this testbed
- Efforts to explore soil materials, box dynamics, and structural response predictions can be conducted at various scales of the soil box system
- Future research and development offers a great opportunity for collaboration across various engineering fields

Acknowledgements

- Sponsors
  - Department of Energy
  - Lawrence Berkeley National Laboratory
- Project Team
  - Dr. David McCallen, Dr. Ian Buckle, Dr. Denis Israti, Dr. Sherif Elfass, Dr. Boris Jeremic, & Dr. Frank McKenna

Thank you!

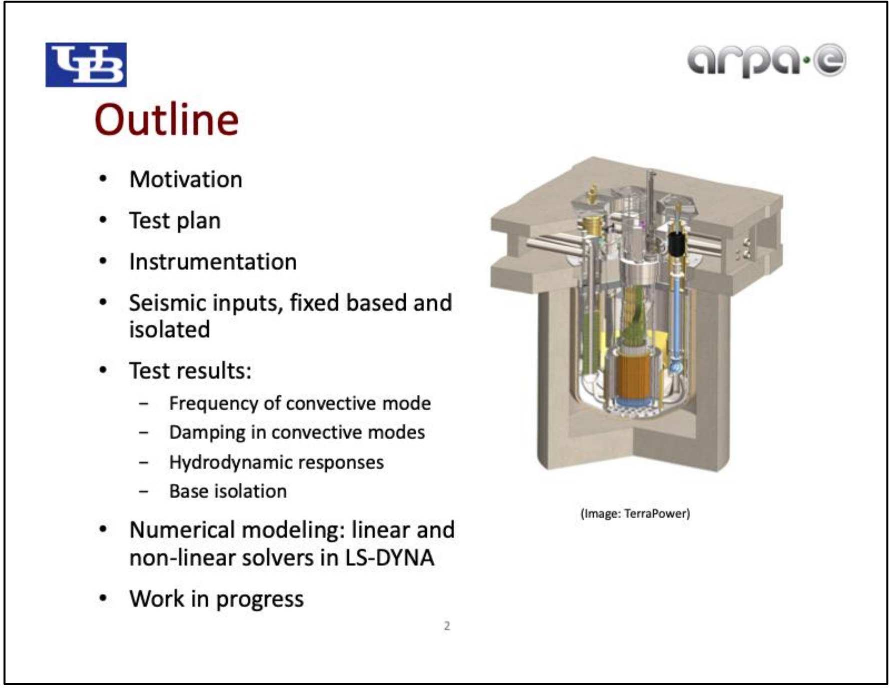
SHAKE TABLE TESTS FOR VALIDATION OF NUMERICAL FSI MODELS OF ADVANCED REACTORS



Shake Table Tests for Validation of Numerical FSI Models of Advanced Reactors

Faizan Ul Haq Mir, Ching-Ching Yu, Andrew S. Whittaker

University at Buffalo



Outline

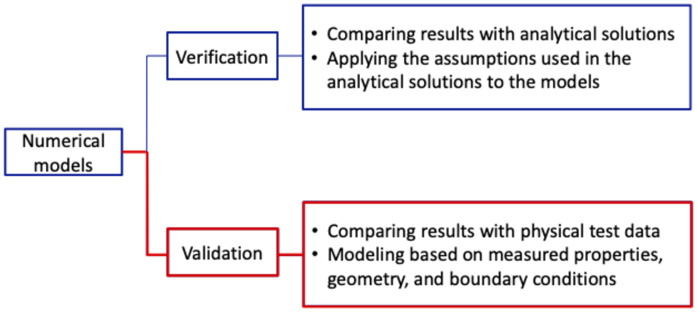
- Motivation
- Test plan
- Instrumentation
- Seismic inputs, fixed based and isolated
- Test results:
    - Frequency of convective mode
    - Damping in convective modes
    - Hydrodynamic responses
    - Base isolation
- Numerical modeling: linear and non-linear solvers in LS-DYNA
- Work in progress

(Image: TerraPower)

Motivation

- Advanced reactors
  - Liquid metals for heat evacuation
  - Vessels may be thinner than in PWRs and BWRs
    - Reduced seismic capacity
- Fluid-structure interaction (FSI)
  - Building structure, reactor vessels, vessel internals
- Legacy FSI procedures - added mass and damping
  - Idealized geometries only (e.g. Chen et al. 1976, Dong 1978)
- FSI calculations for seismic design, qualification and risk assessment
  - Physical testing of equipment not feasible in most cases
  - Verified and validated FSI models needed
    - Conventionally supported
    - Isolated

Verification and validation



Numerical models

Verification
- Comparing results with analytical solutions
- Applying the assumptions used in the analytical solutions to the models

Validation
- Comparing results with physical test data
- Modeling based on measured properties, geometry, and boundary conditions

Building a dataset

- Base-supported vessel
- Phase-I:
  - No vessel head, RVIs, or HMOs
  - Results used for validation of numerical models in LS-DYNA: ALE and ICFD
- Phase-II:
  - Simplified representations of central and off-center RVIs
  - Testing completed; numerical modeling continues

Conclusions

- Experimental and analytical predictions of sloshing frequency
- Smaller damping in convective mode than ASCE 4
- Analytical solutions for lateral, rocking, and vertical excitations: results may be superimposed
- Base isolation leads to reduction in responses, except wave height
- Lagrangian approaches in LS-DYNA do not predict convective responses well
- ALE model in LS-DYNA validated; tracer card needs improvement
- If no convective response: Lagrangian model is faster than ALE
- Data to be made available on DesignSafe

Acknowledgments

- ARPA – E
- Kairos Power
- Benjamin D. Kosbab and Kaniel Z. Tilow, SGH
- Technical staff, SEESL, University at Buffalo

Thank you

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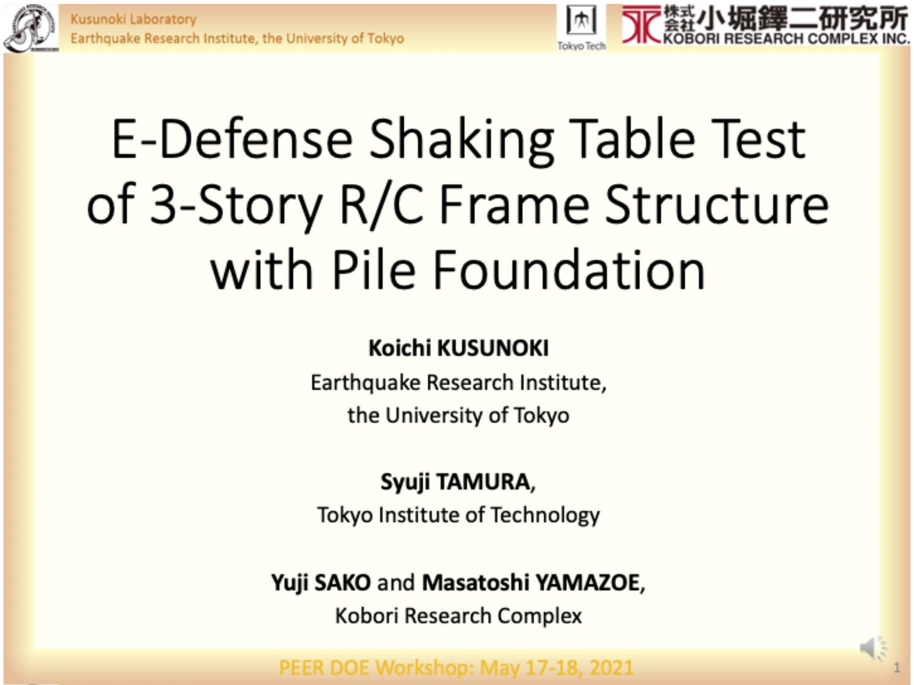
E-DEFENSE SHAKING TABLE TEST OF 3-STORY R/C FRAME STRUCTURE WITH PILE FOUNDATION



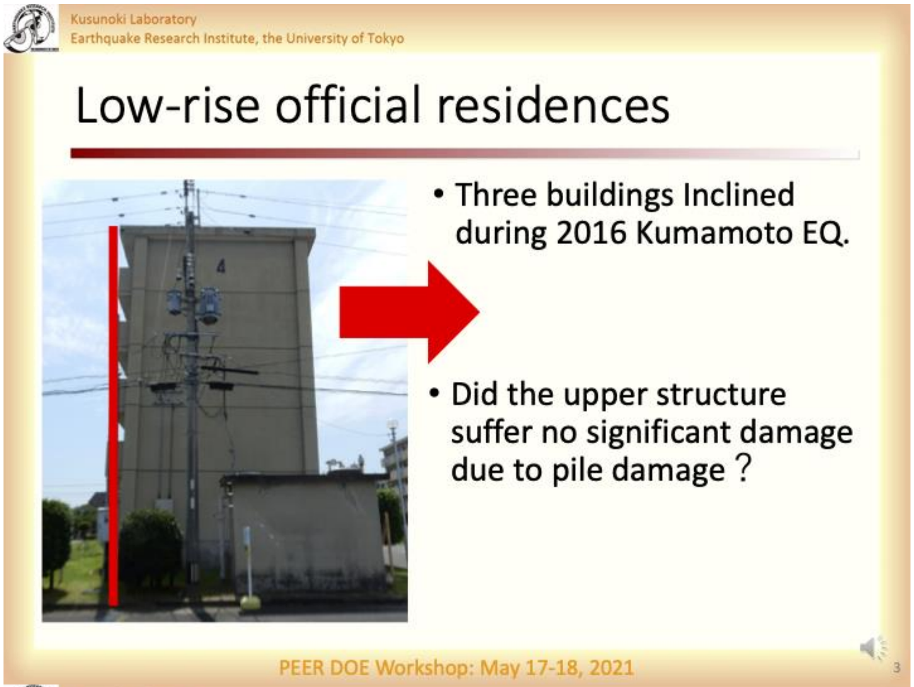
E-Defense Shaking Table Test of 3-Story R/C Frame Structure with Pile Foundation

Koichi KUSUNOKI
Earthquake Research Institute,
the University of Tokyo

Syuji TAMURA,
Tokyo Institute of Technology

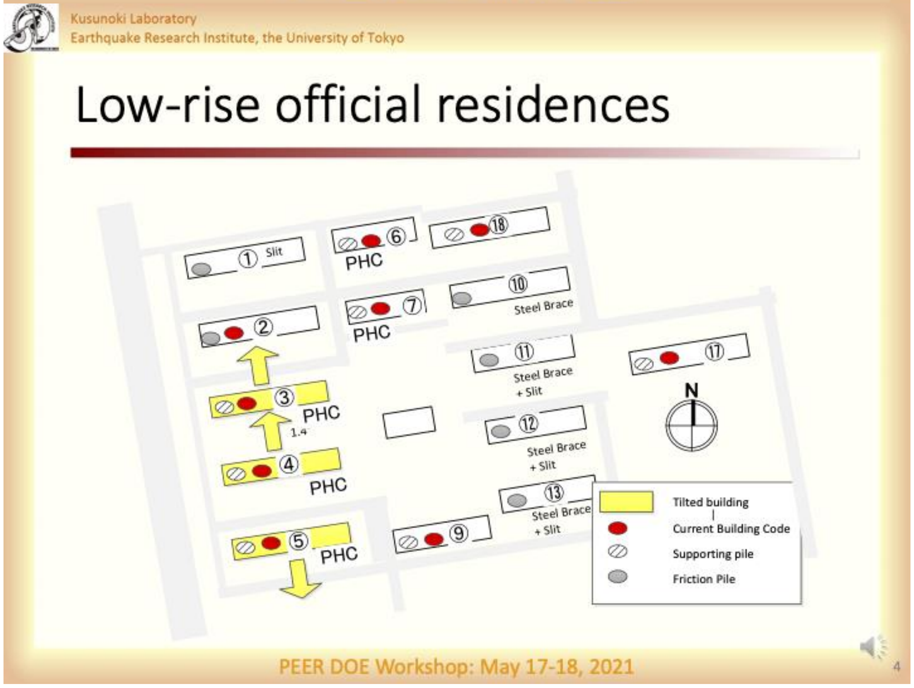
Yuji SAKO and **Masatoshi YAMAZOE**,
Kobori Research Complex

Low-rise official residences



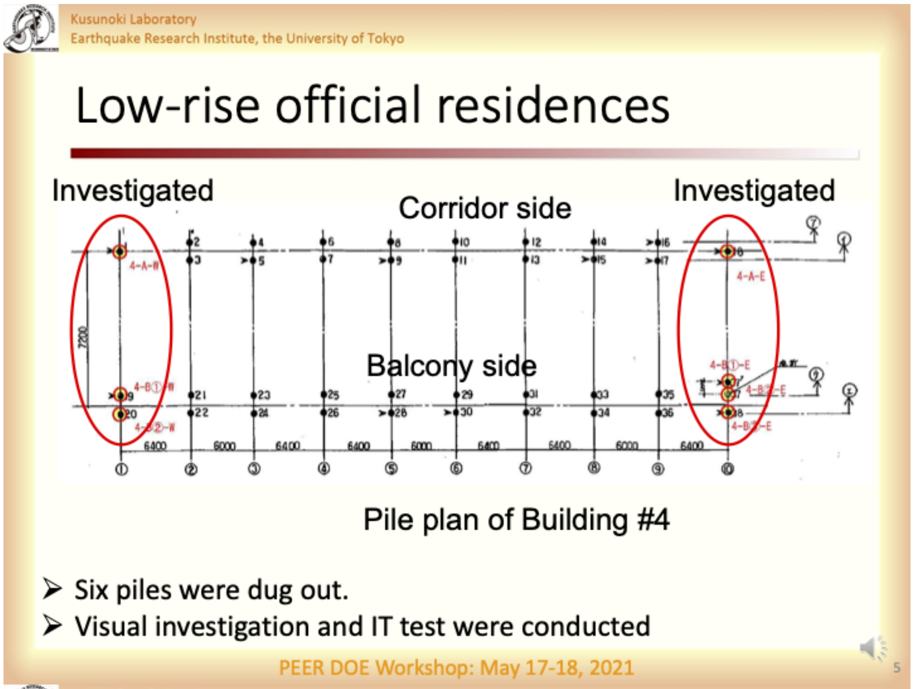
Low-rise official residences

- Three buildings Inclined during 2016 Kumamoto EQ.
- Did the upper structure suffer no significant damage due to pile damage？



Low-rise official residences

🟨 Tilted building
● Current Building Code
▨ Supporting pile
● Friction Pile



Low-rise official residences

Investigated

Corridor side

Investigated

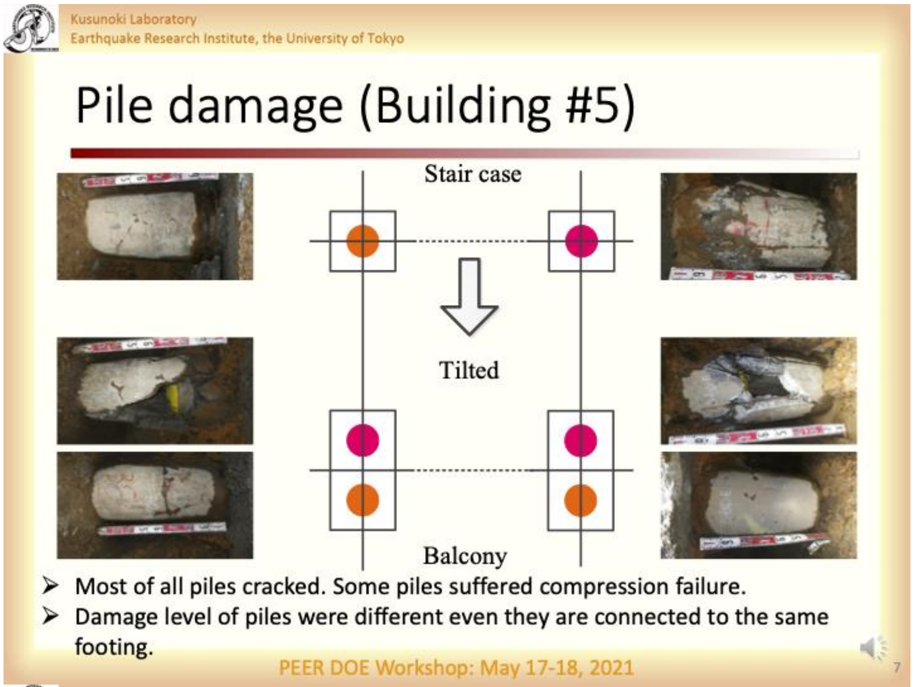
Balcony side

Pile plan of Building #4

- Six piles were dug out.
- Visual investigation and IT test were conducted

Low-rise official residences

Piles were
severely damaged



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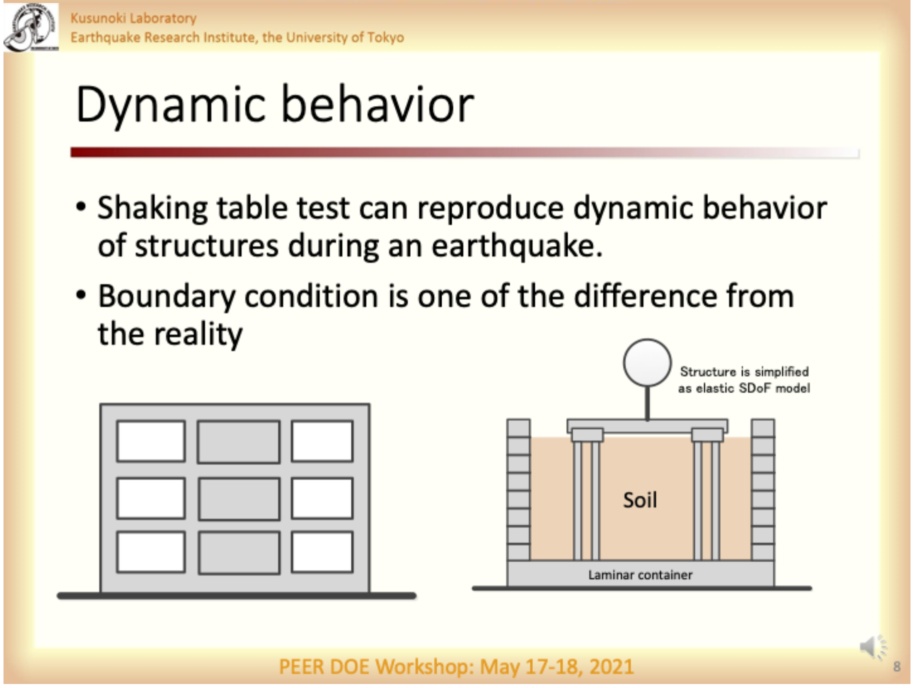
Pile damage (Building #5)

- Most of all piles cracked. Some piles suffered compression failure.
- Damage level of piles were different even they are connected to the same footing.



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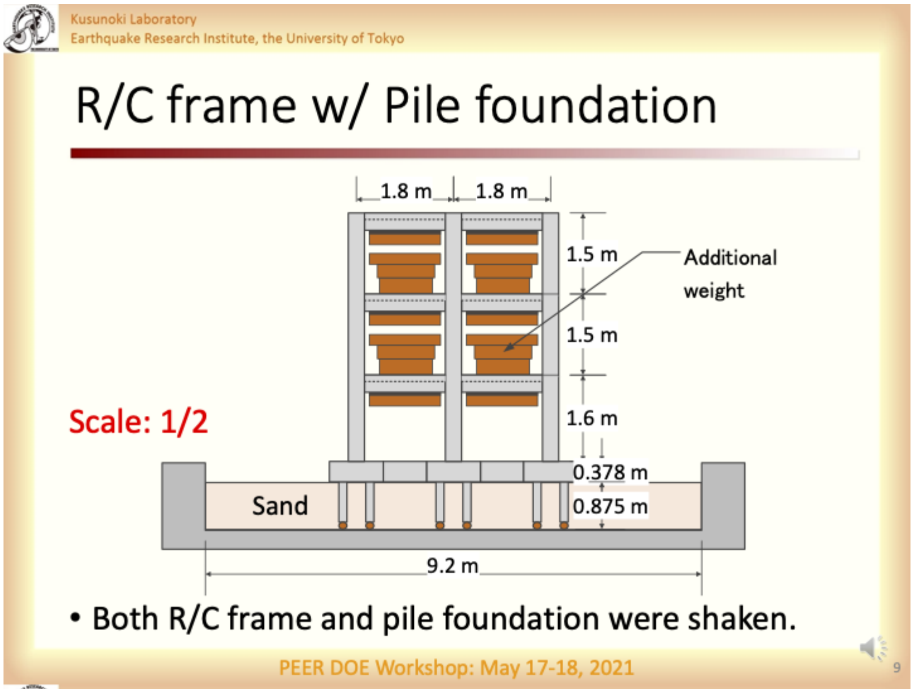
Dynamic behavior

- Shaking table test can reproduce dynamic behavior of structures during an earthquake.
- Boundary condition is one of the difference from the reality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R/C frame w/ Pile foundation

Scale: 1/2

- Both R/C frame and pile foundation were shaken.

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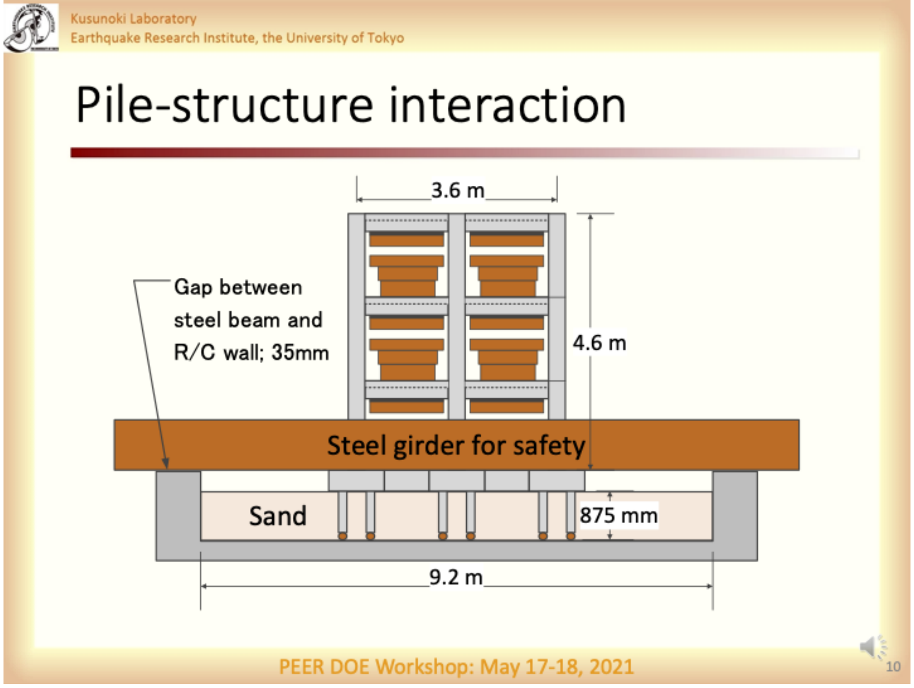
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Pile-structure interaction

Gap between steel beam and R/C wall; 35mm

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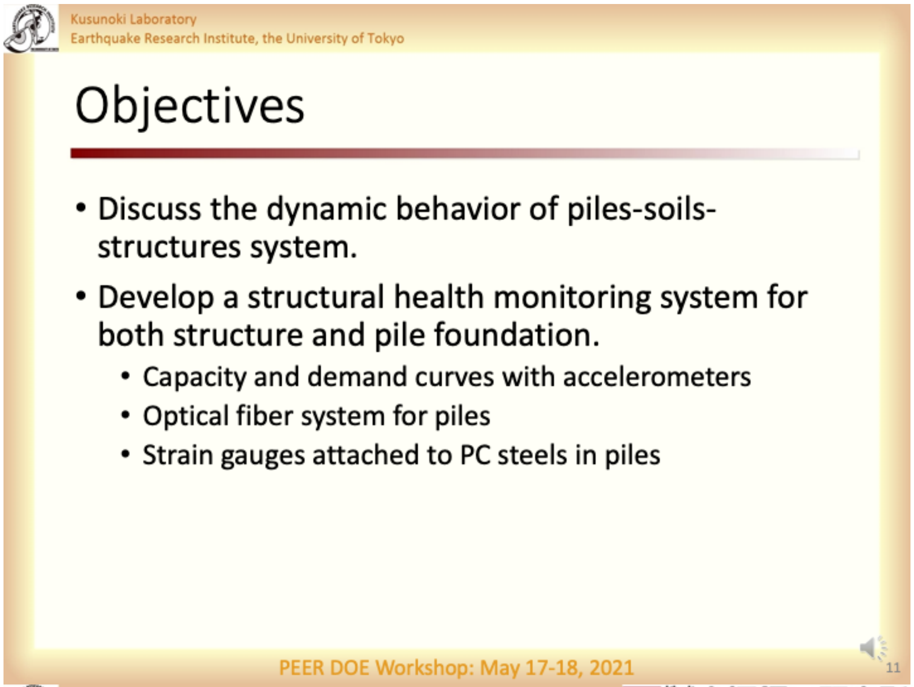
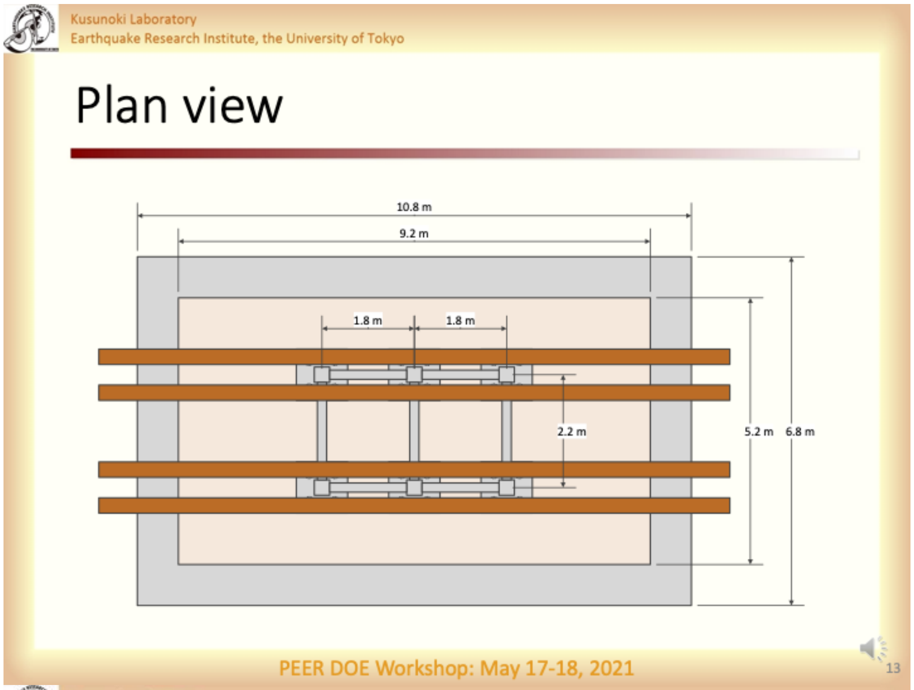


Objectives

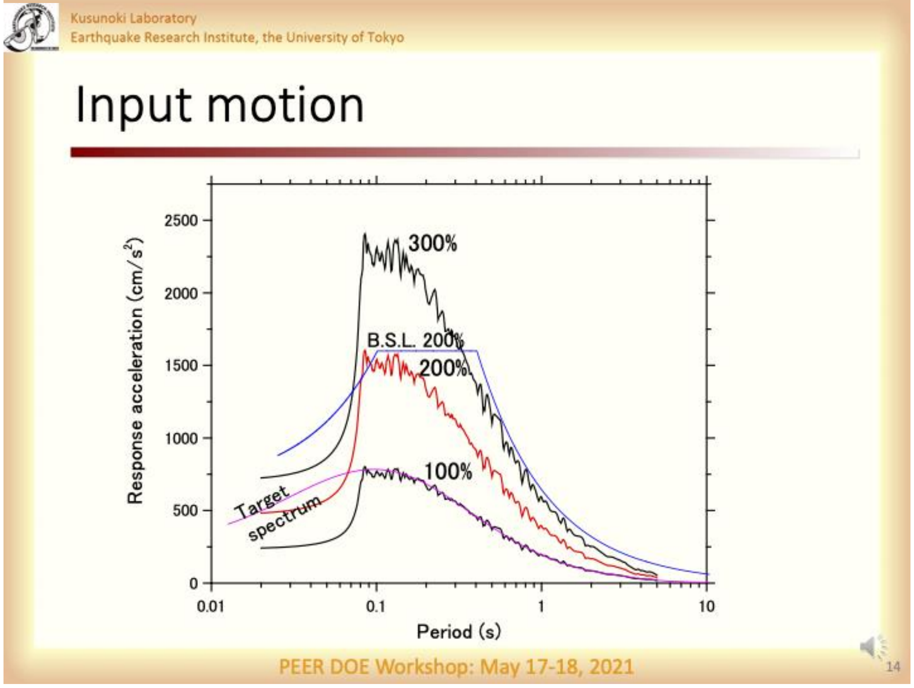
- Discuss the dynamic behavior of piles-soils-structures system.
- Develop a structural health monitoring system for both structure and pile foundation.
  - Capacity and demand curves with accelerometers
  - Optical fiber system for piles
  - Strain gauges attached to PC steels in piles

Team

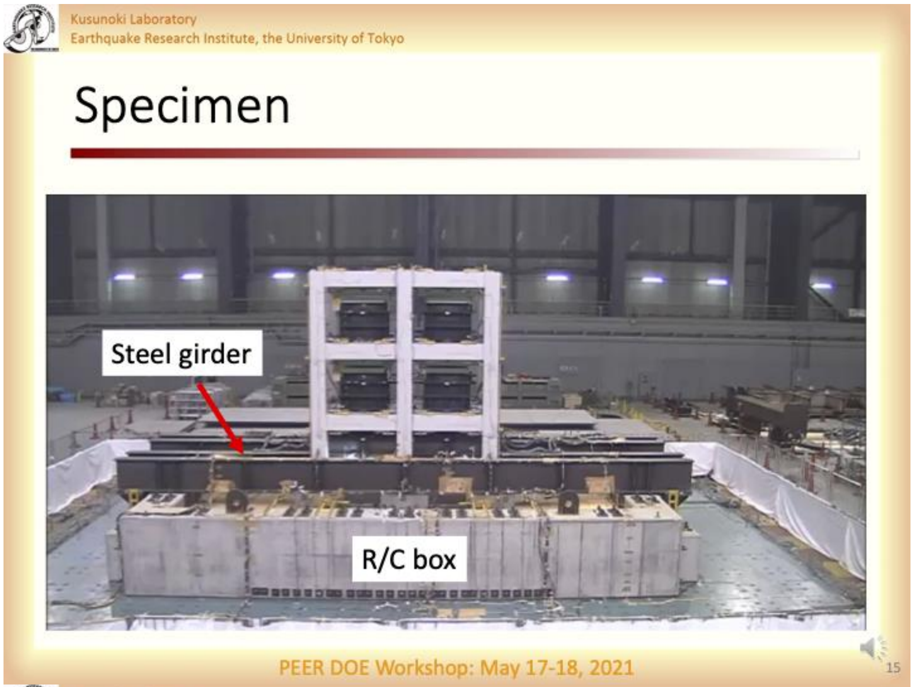
- Y. Sako, N. Kojika, M. Yamazoe, Kobori Research Complex
- K. Kusunoki, ERI, U. of Tokyo
- N. Nakamura, Y. Hibino, Hiroshima Univ.
- K. Sugimoto, Yokohama National University
- H. Okano, Chiba University
- K. Hayashi and M. Abe, Kyoto University
- T. Nagae and Y. Kawamata, E-Defense
- M. Kashiwa, NIRIM
- Y. Morii, Shimizu Corp.
- H. Funabara, Taisei Corp.
- T. Suzuki Takenaka Corp.
- N. Adachi, Kashima Corp.



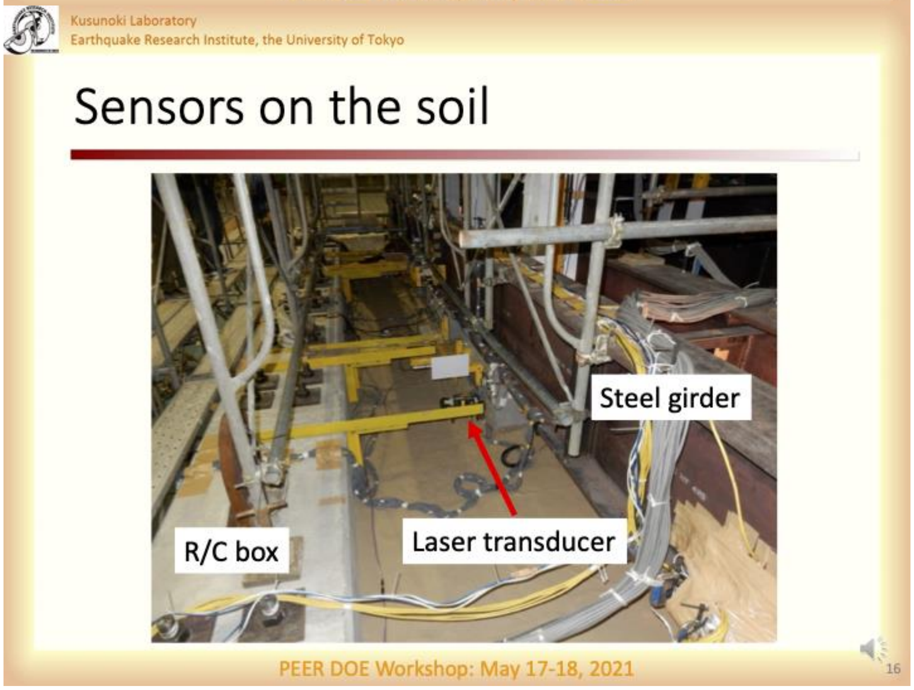
Plan view



Input motion



Specimen

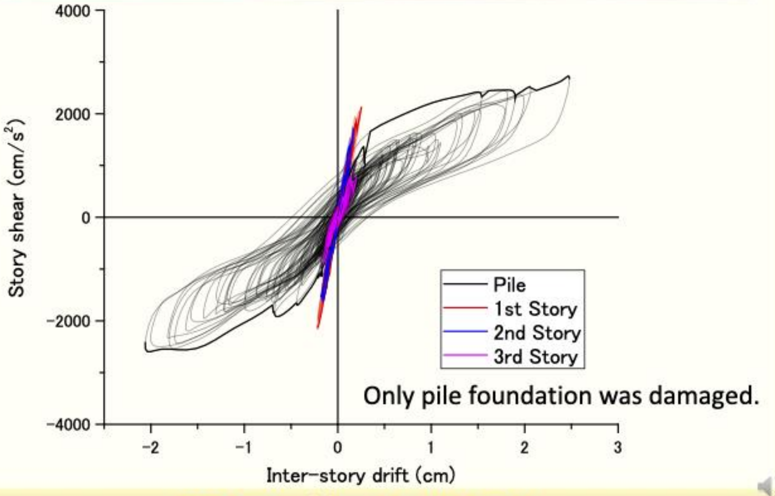


Sensors on the soil

Test results (Pile foundation)

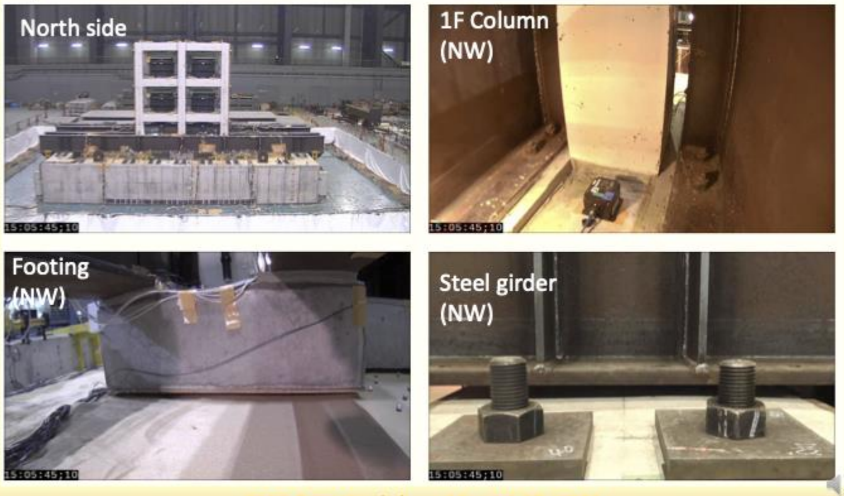
Level	PGA (cm/s^2)	Maximum story drift angle (structure)	Maximum deformation angle (pile)
20%	31	1/9140(1F)	1/2610
60%	91	1/2270(1F)	1/490
100%	180	1/1310(1F)	1/160
200%	410	1/700 (1F)	1/33
300%	680	1/570 (2F)	1/7

Level 200%; story force-story drift



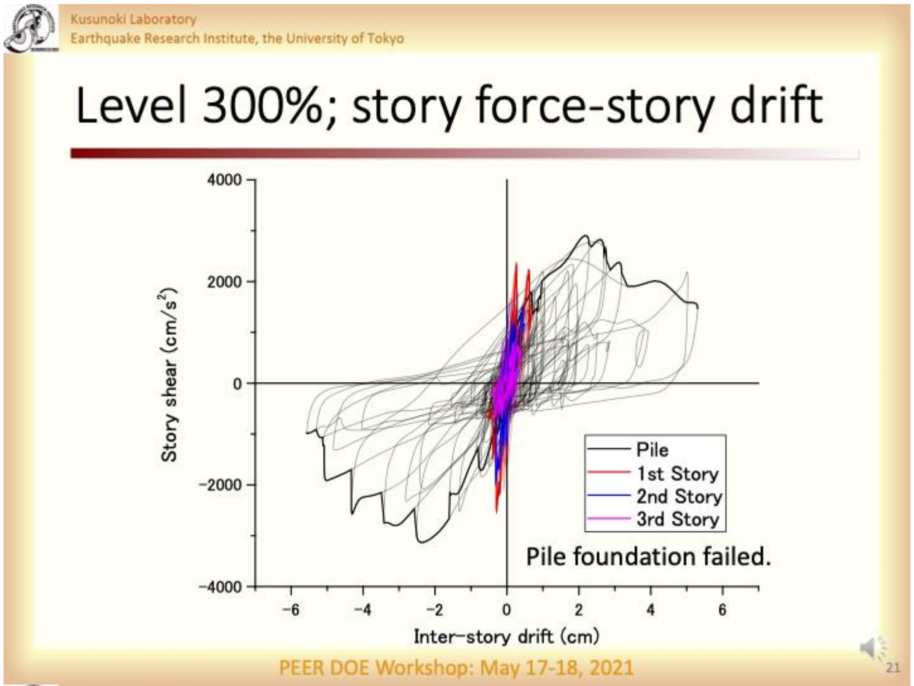
Only pile foundation was damaged.

Level 200% (pile foundation)



W/ Pile foundation Level 200%

- Building response
  - Maximum inter story drift angle was 1/700, which was too small to cause flexural cracks in beams.
- Pile response
  - Maximum lateral displacement was 20mm. The strain of PC steel in the piles exceeded yield strain.
  - **No significant settlement nor residual displacement was observed**.



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Level 300%; story force-story drift

Pile foundation failed.

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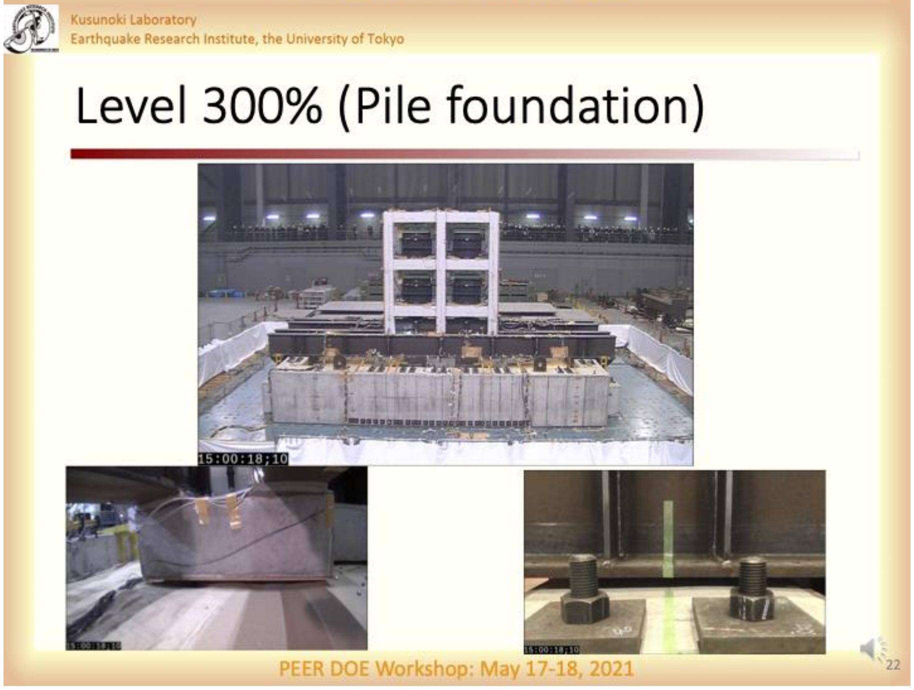

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Level 300% (Pile foundation)

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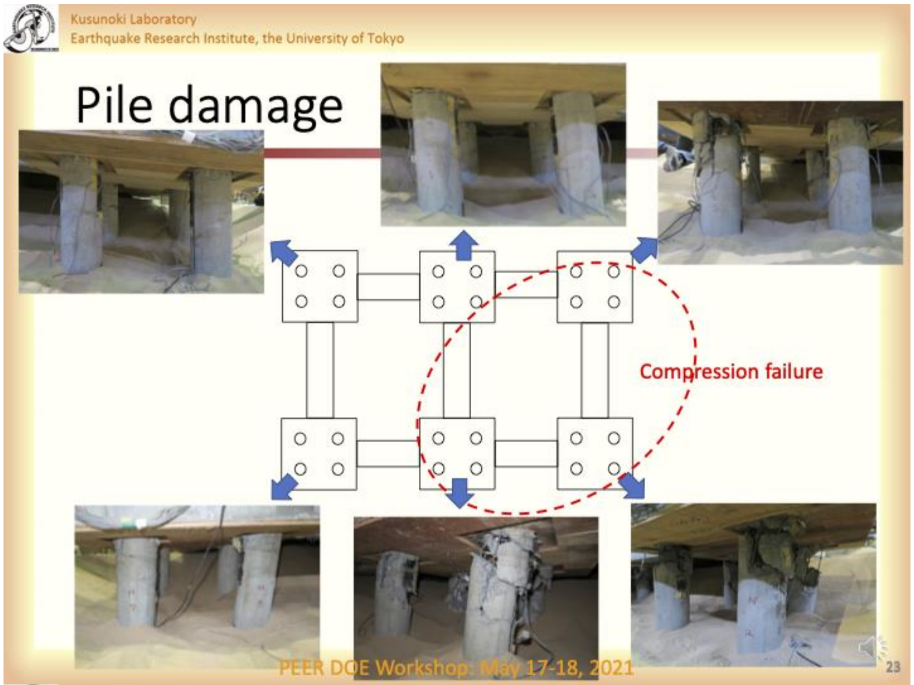

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Pile damage

Compression failure

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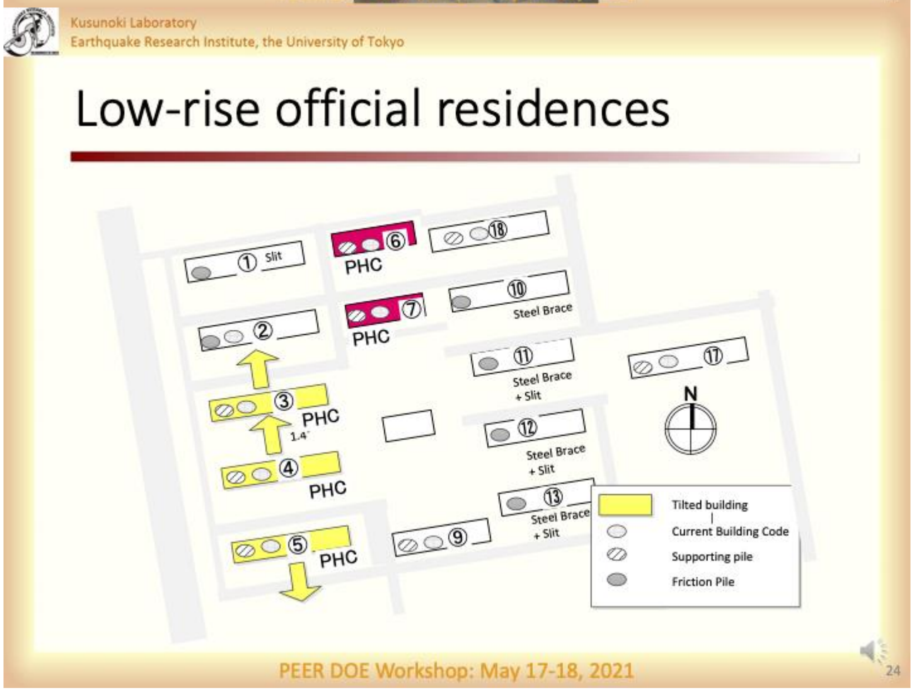

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Low-rise official residences

🟨 Tilted building
|
○ Current Building Code
▨ Supporting pile
● Friction Pile

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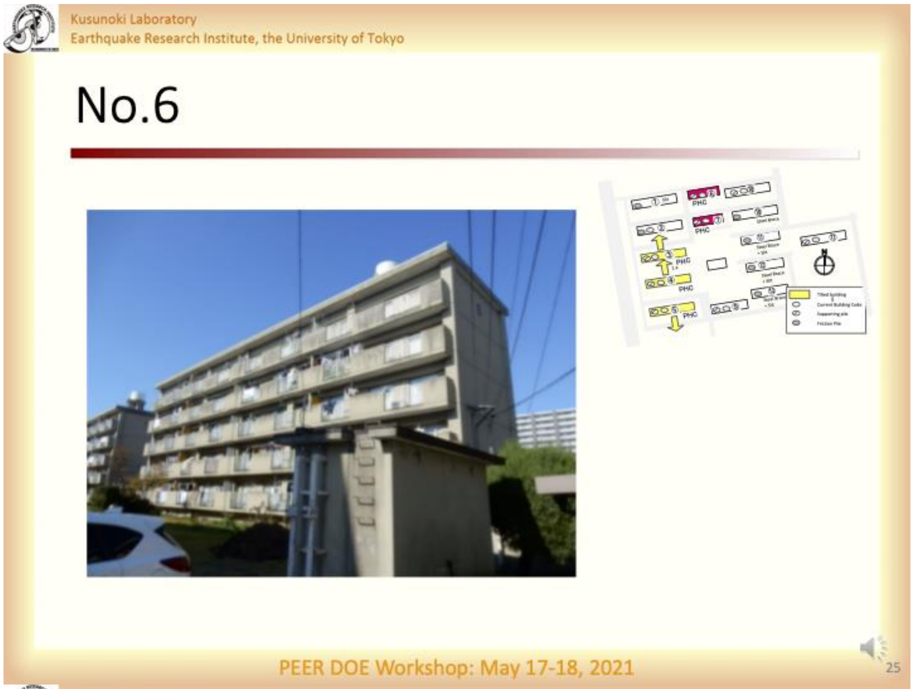

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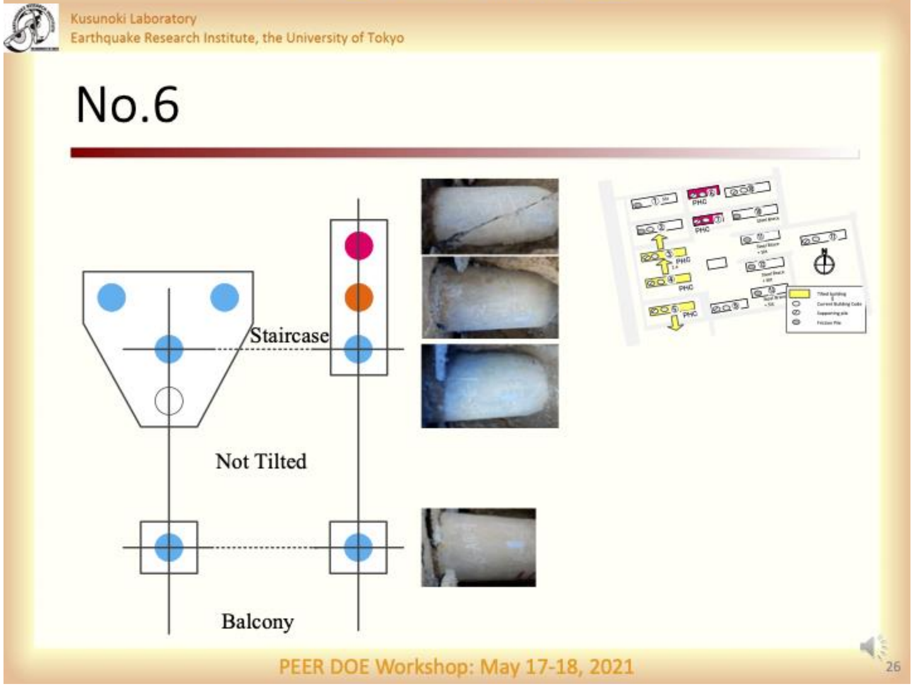

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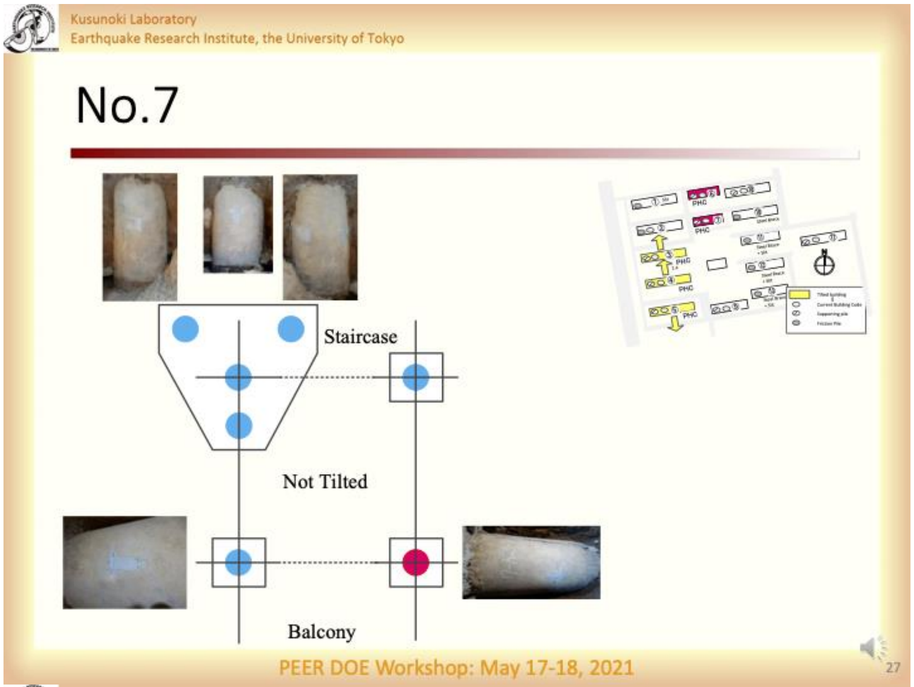
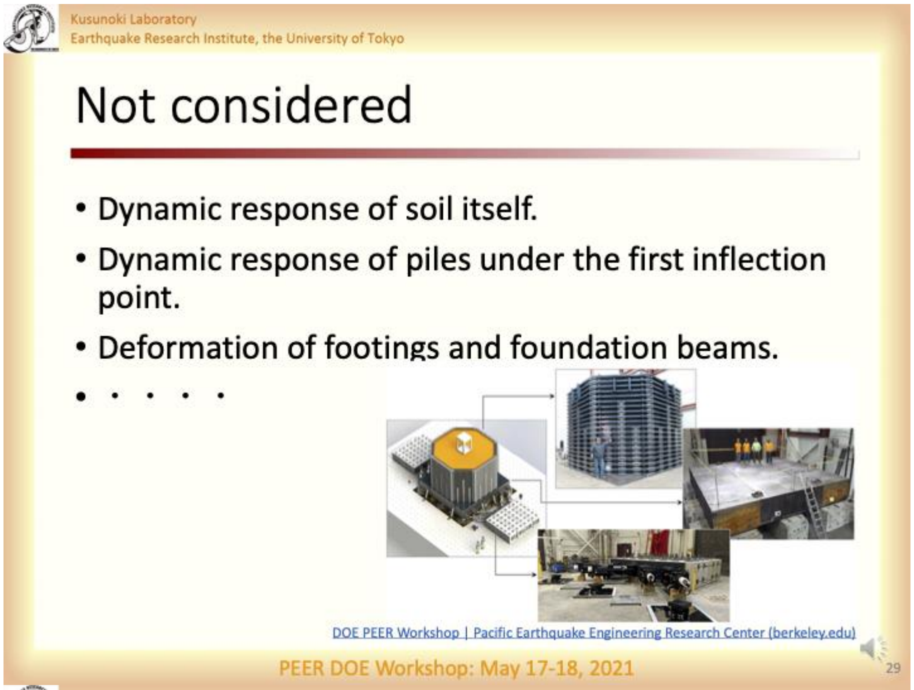
No.7



Concluding remarks

- R/C frame structure with pile foundation was successfully tested on the E-Defense.
- The Pile foundation failed, and the upper structure did not suffer any severe damage.
- Even though the pile foundation suffered severe damage, there was no evidence of the damage on the ground's surface.
- It is required to develop a monitoring system/damage detection method for the pile foundation.



Not considered

- Dynamic response of soil itself.
- Dynamic response of piles under the first inflection point.
- Deformation of footings and foundation beams.
- • • • •

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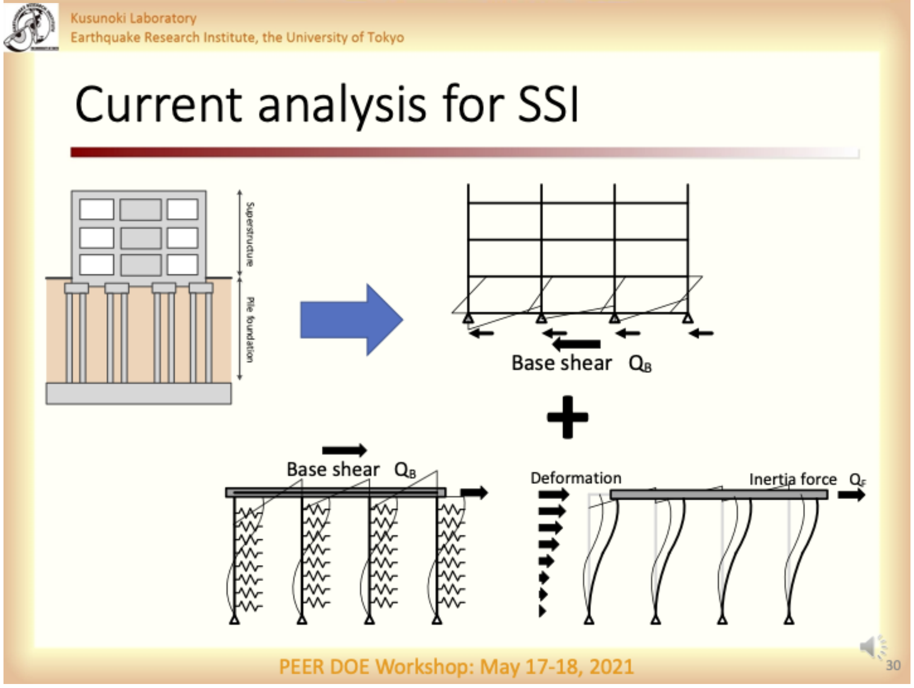
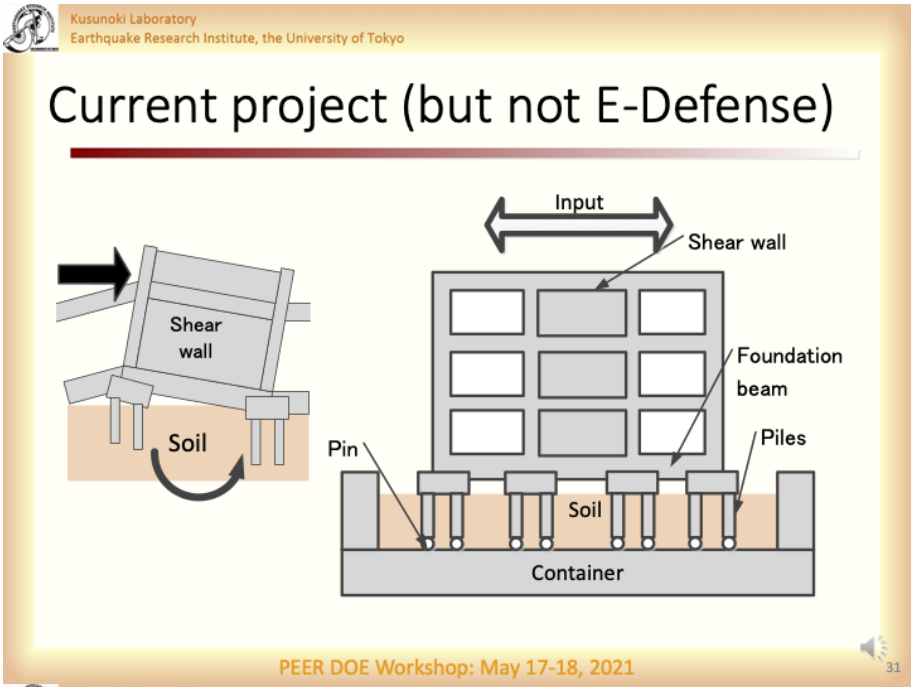


Current analysis for SSI



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Current project (but not E-Defense)

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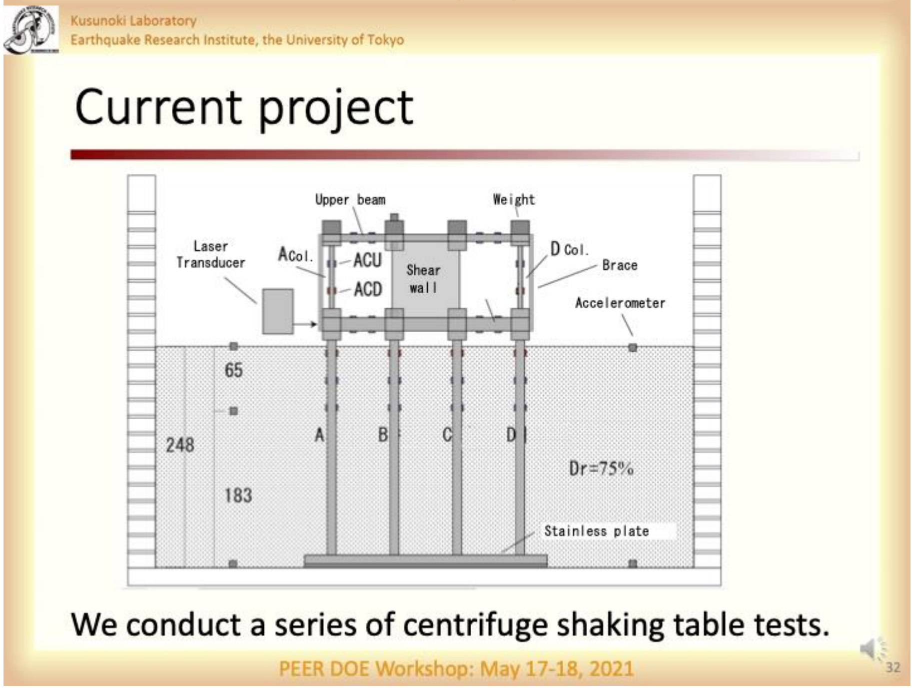

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Current project

We conduct a series of centrifuge shaking table tests.

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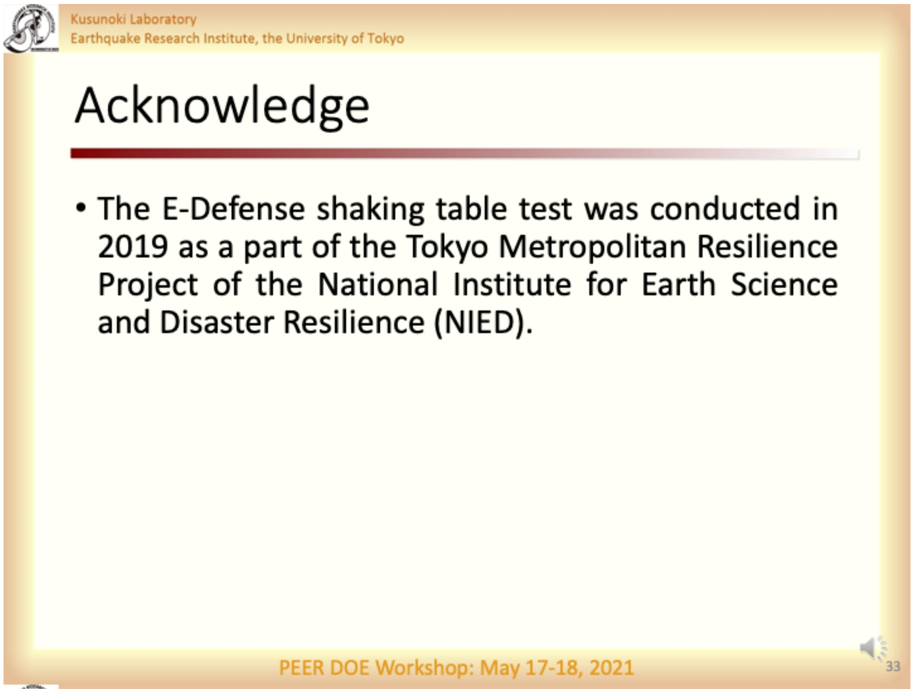

Acknowledge

- The E-Defense shaking table test was conducted in 2019 as a part of the Tokyo Metropolitan Resilience Project of the National Institute for Earth Science and Disaster Resilience (NIED).

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Thank you for your kind attention…

MODELING VERTICAL FREE-FIELD MOTION FOR SSI ANALYSIS CONSISTENT WITH VERTICAL DESIGN MOTION DEVELOPMENT

Modeling Vertical Free-Field Motion for SSI Analysis Consistent with Vertical Design Motion Development

DOE/PEER/UNR Workshop

International Workshop on Large-Scale Shake Table Testing for the Assessment of

Soil-Foundation-Structure System Response for Seismic Safety of DOE Nuclear Facilities

May 18, 2021

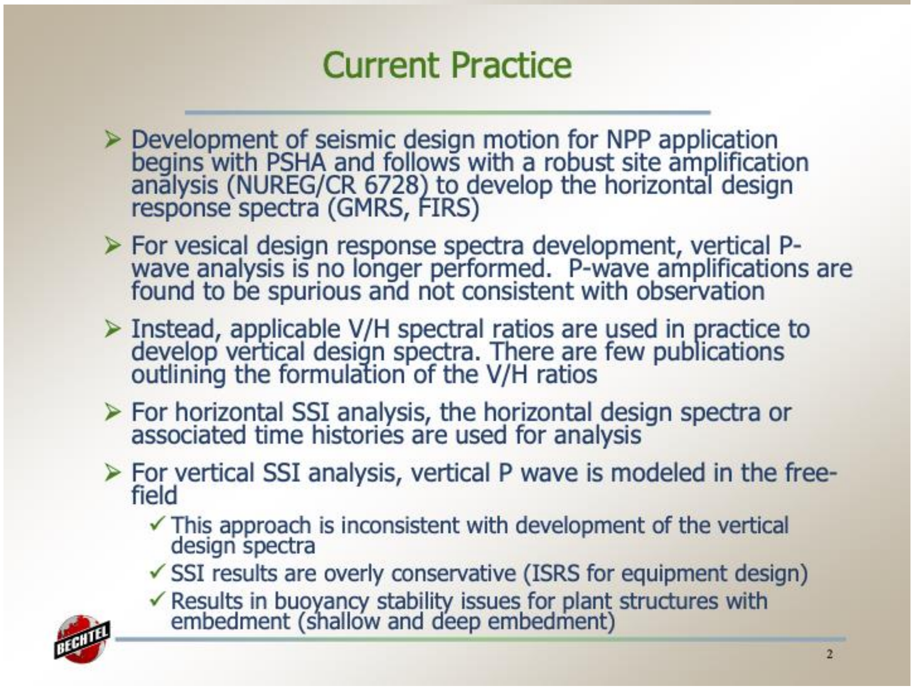
Farhang Ostadan

Manager of Earthquake Engineering Center

Bechtel Corporation

EPRI Report

https://www.epri.com/#/pages/product/3002011804/



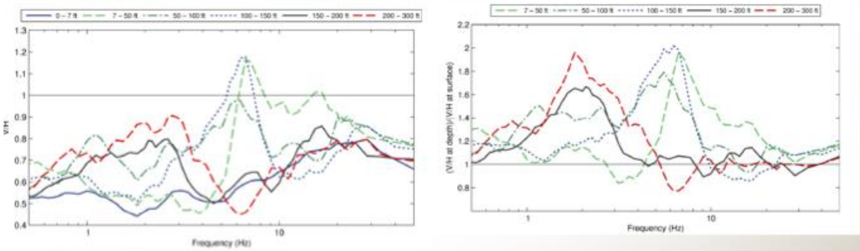
Current Practice

- Development of seismic design motion for NPP application begins with PSHA and follows with a robust site amplification analysis (NUREG/CR 6728) to develop the horizontal design response spectra (GMRS, FIRS)
- For vesical design response spectra development, vertical P-wave analysis is no longer performed. P-wave amplifications are found to be spurious and not consistent with observation
- Instead, applicable V/H spectral ratios are used in practice to develop vertical design spectra. There are few publications outlining the formulation of the V/H ratios
- For horizontal SSI analysis, the horizontal design spectra or associated time histories are used for analysis
- For vertical SSI analysis, vertical P wave is modeled in the free-field
  - ✓ This approach is inconsistent with development of the vertical design spectra
  - ✓ SSI results are overly conservative (ISRS for equipment design)
  - ✓ Results in buoyancy stability issues for plant structures with embedment (shallow and deep embedment)

New Approach

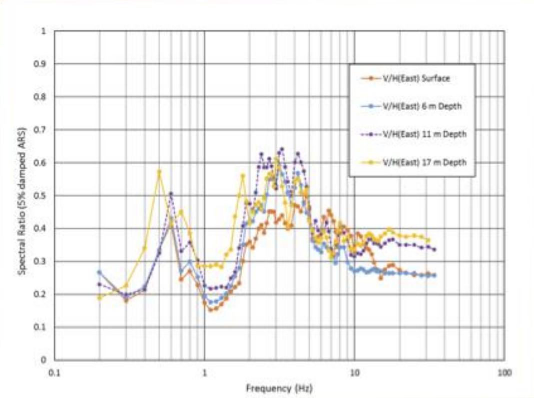
- For SSI analysis of embedded structures using the substructuring methods (SASSI), the free-field motion within the embedment depth of the structures needs to be computed. In other SSI formulation, free-field motion for the full height of the soil column is needed
- In the new approach, the free-field motion at each depth in the model is computed using the horizontal motion at the same depth and applying applicable V/H ratio to get the vertical motion
- This approach is formulated in frequency domain using RVT (random vibration theory) in an iterative process to get he vertical spectra at all depth of interest for SSI analysis
- In this approach, free-field SSI vertical motion is consistent with the approach used for development of the vertical design spectra
- The vertical SSI results are more realistic and are reduced from the results using P-wave analysis

Depth-dependent Vertical-to-Horizontal (V/H) Ratios of Free-Field Ground Motion Response Spectra for Deeply Embedded nuclear Structures (BNL-107612-2015-R, 2/2015)



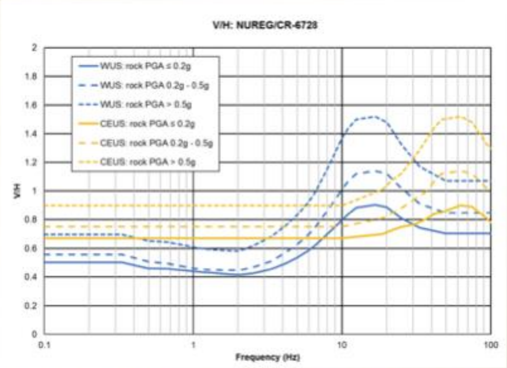
(Data from 45 vertical arrays: California, Japan, Alaska, Taiwan)

Lotung SSI Experiment (1980s)

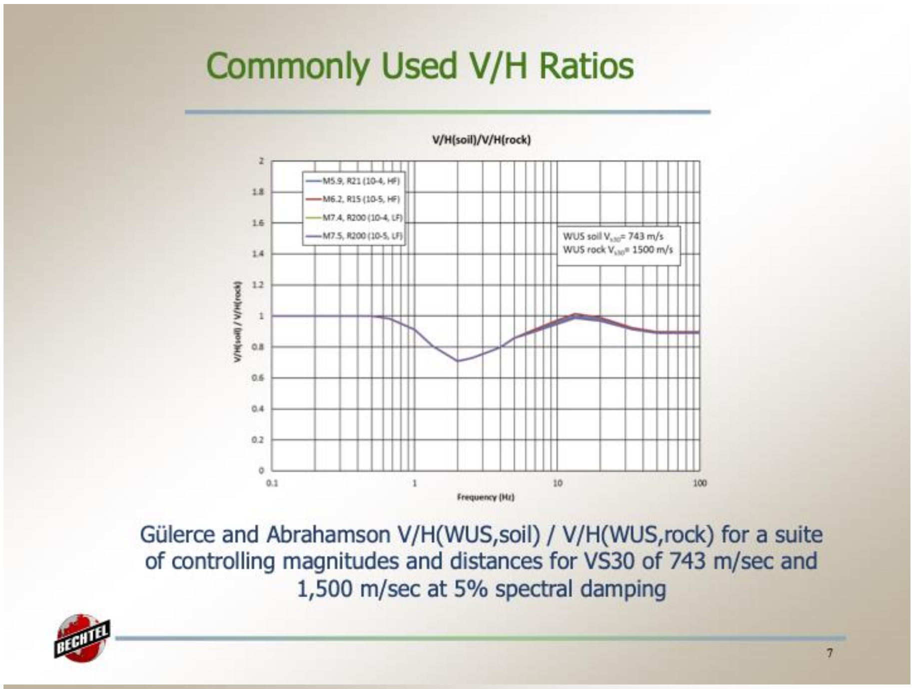


V/H Spectral Ratio from Lotung LSST No. 7 Free-Field records

Commonly Used V/H Ratios

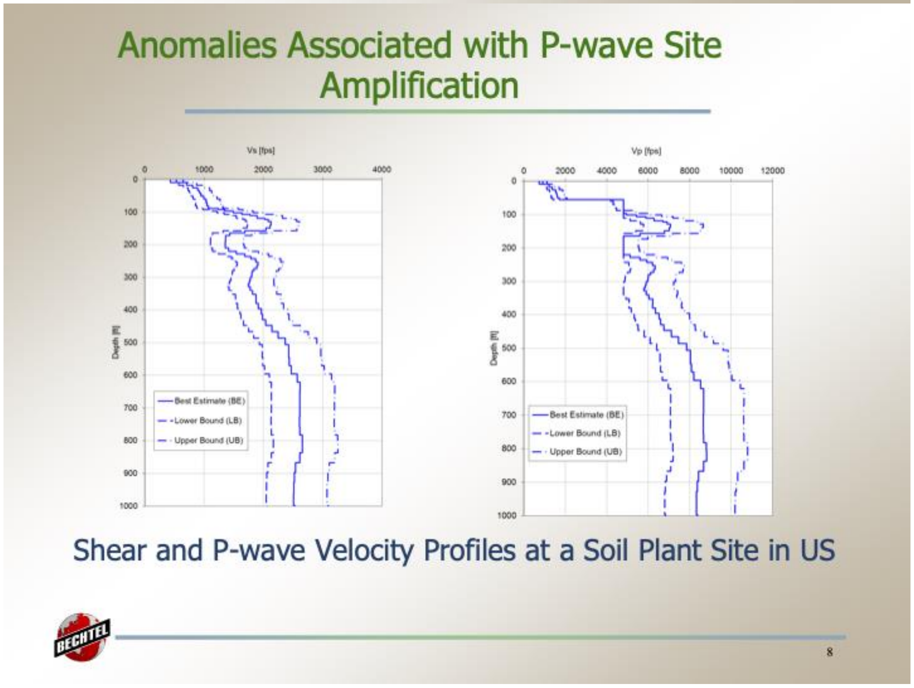


V/H ratios for WUS rock and CEUS Hard Rock Sites,
NUREG/CR-6728 at 5% spectral damping



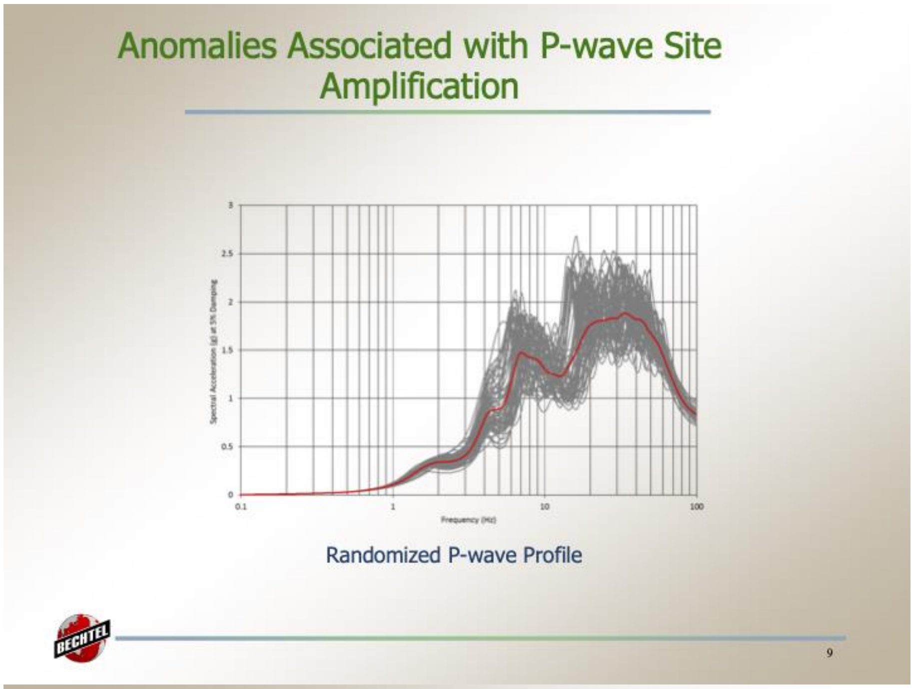
Commonly Used V/H Ratios

Gülerce and Abrahamson V/H(WUS,soil) / V/H(WUS,rock) for a suite of controlling magnitudes and distances for VS30 of 743 m/sec and 1,500 m/sec at 5% spectral damping



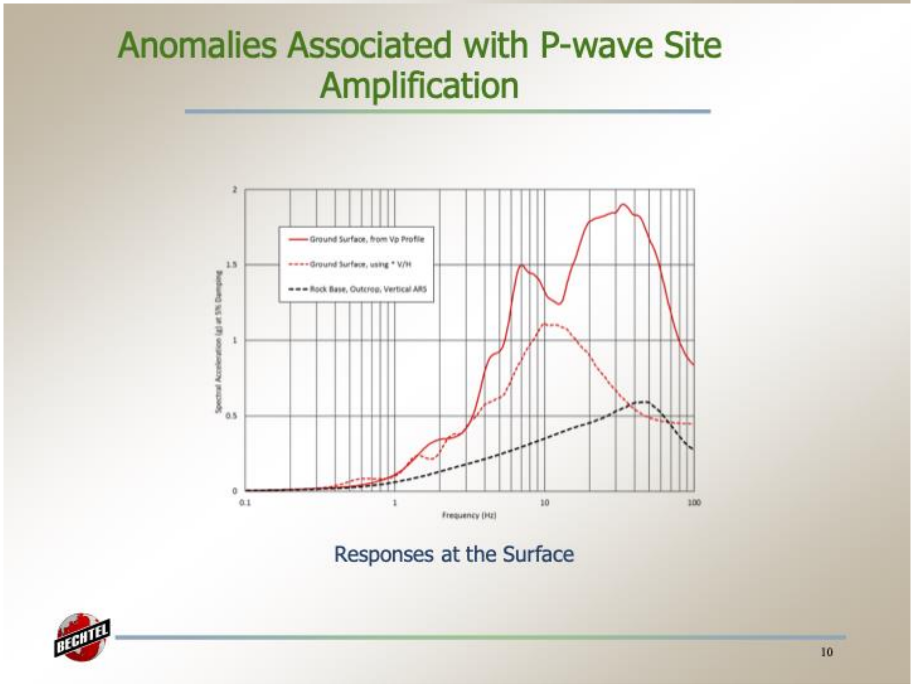
Anomalies Associated with P-wave Site Amplification

Shear and P-wave Velocity Profiles at a Soil Plant Site in US



Anomalies Associated with P-wave Site Amplification

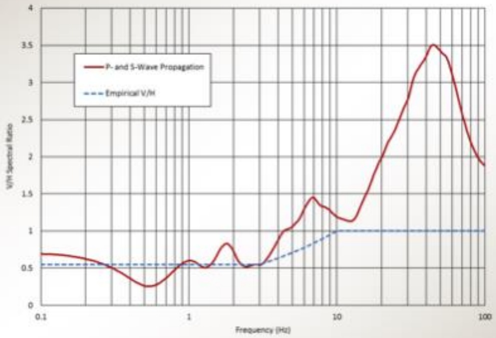
Randomized P-wave Profile



Anomalies Associated with P-wave Site Amplification

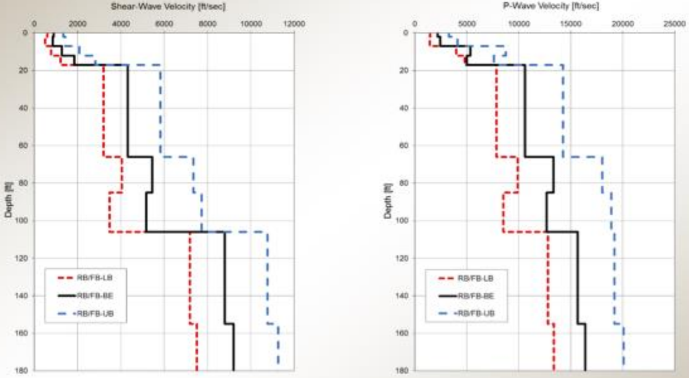
Responses at the Surface

Anomalies Associated with P-wave Site Amplification

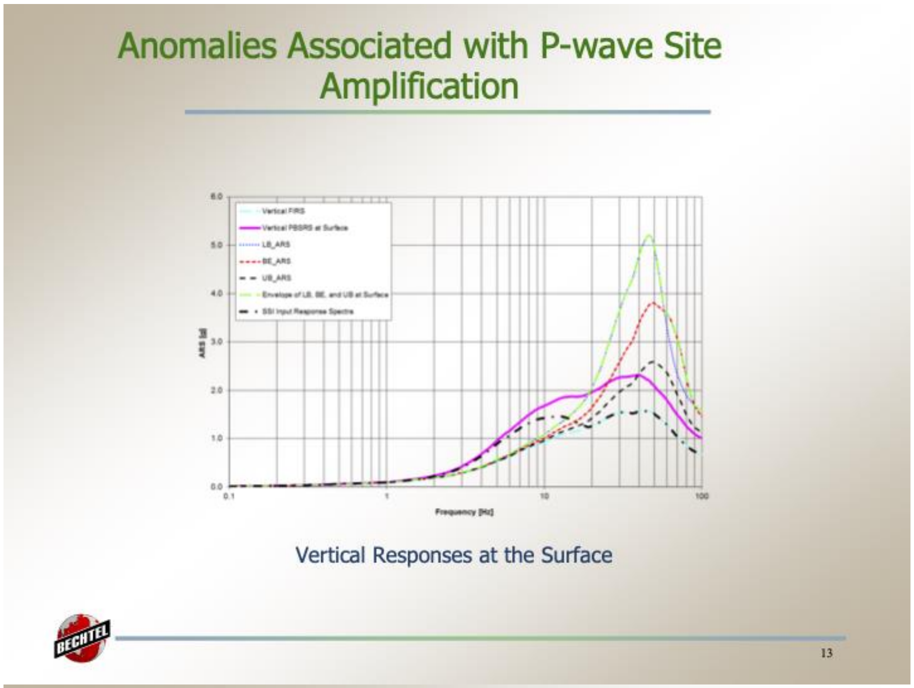


Comparison of V/H Ratio

Anomalies Associated with P-wave Site Amplification

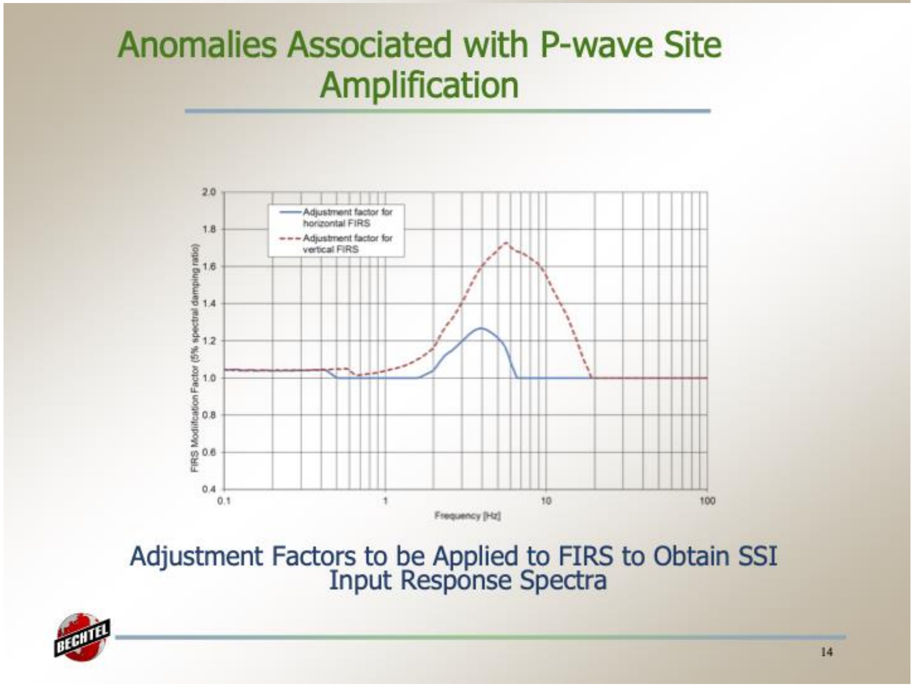


Shear and P-wave Velocity Profiles at a Rock Plant Site in US



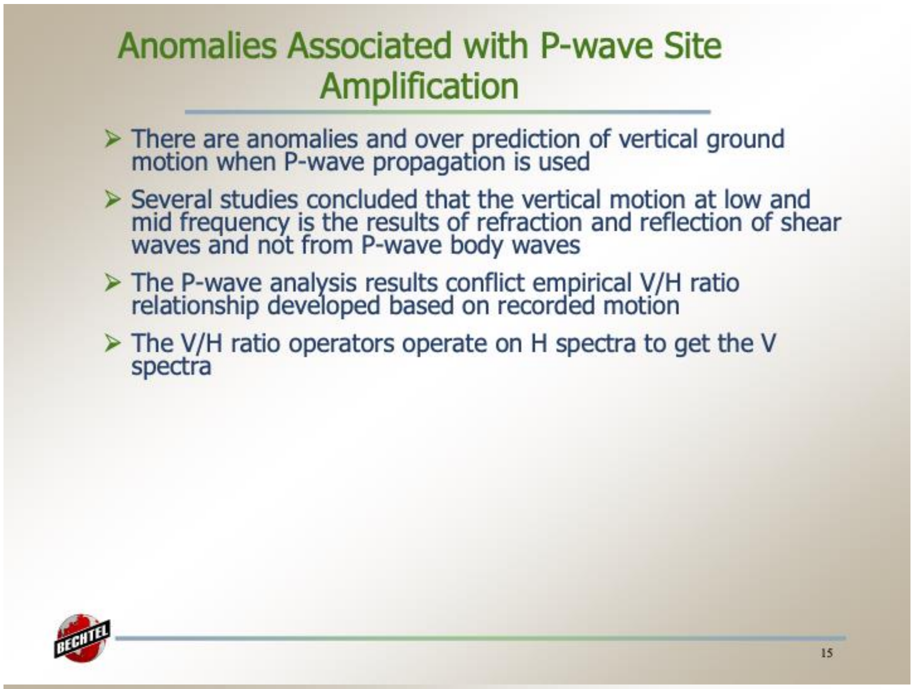
Anomalies Associated with P-wave Site Amplification

Vertical Responses at the Surface



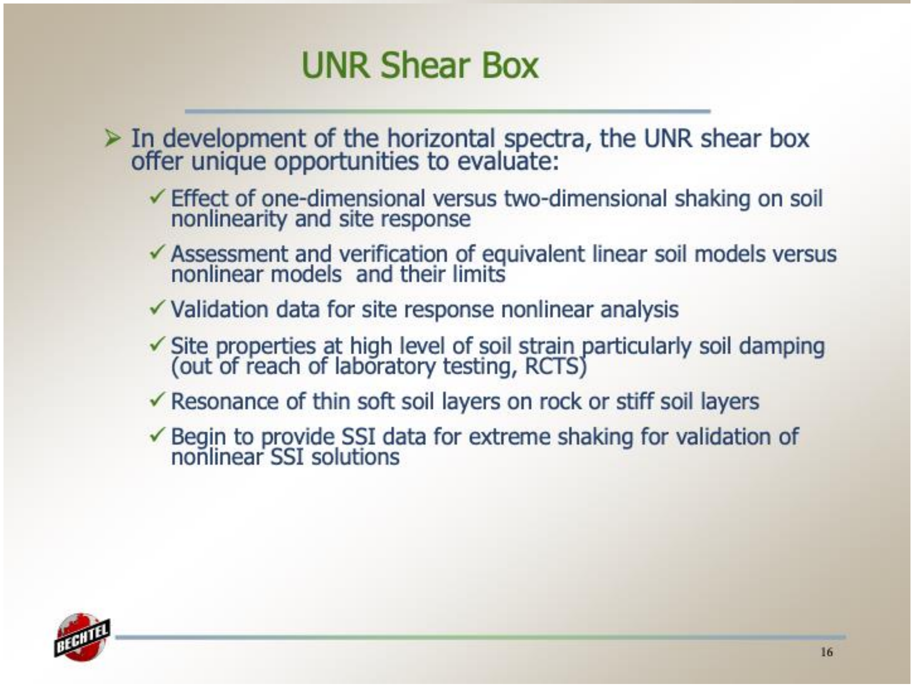
Anomalies Associated with P-wave Site Amplification

Adjustment Factors to be Applied to FIRS to Obtain SSI Input Response Spectra



Anomalies Associated with P-wave Site Amplification

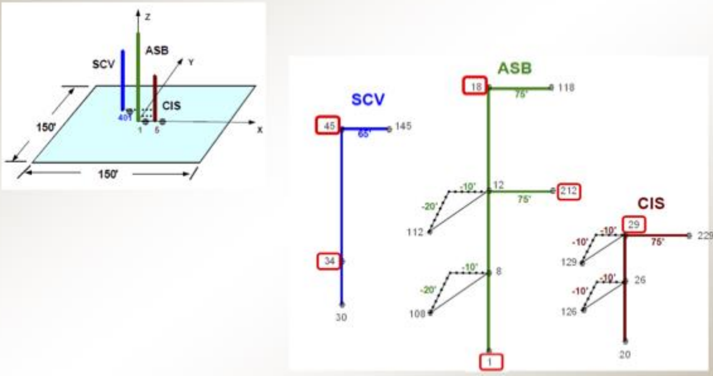
- ➢ There are anomalies and over prediction of vertical ground motion when P-wave propagation is used
- ➢ Several studies concluded that the vertical motion at low and mid frequency is the results of refraction and reflection of shear waves and not from P-wave body waves
- ➢ The P-wave analysis results conflict empirical V/H ratio relationship developed based on recorded motion
- ➢ The V/H ratio operators operate on H spectra to get the V spectra



UNR Shear Box

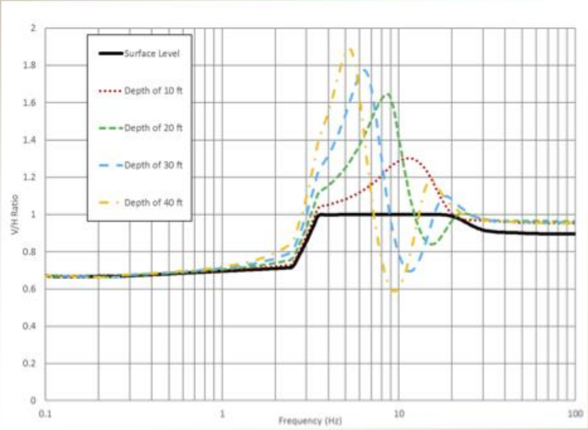
- ➢ In development of the horizontal spectra, the UNR shear box offer unique opportunities to evaluate:
  - ✓ Effect of one-dimensional versus two-dimensional shaking on soil nonlinearity and site response
  - ✓ Assessment and verification of equivalent linear soil models versus nonlinear models and their limits
  - ✓ Validation data for site response nonlinear analysis
  - ✓ Site properties at high level of soil strain particularly soil damping (out of reach of laboratory testing, RCTS)
  - ✓ Resonance of thin soft soil layers on rock or stiff soil layers
  - ✓ Begin to provide SSI data for extreme shaking for validation of nonlinear SSI solutions

SSI Case Study

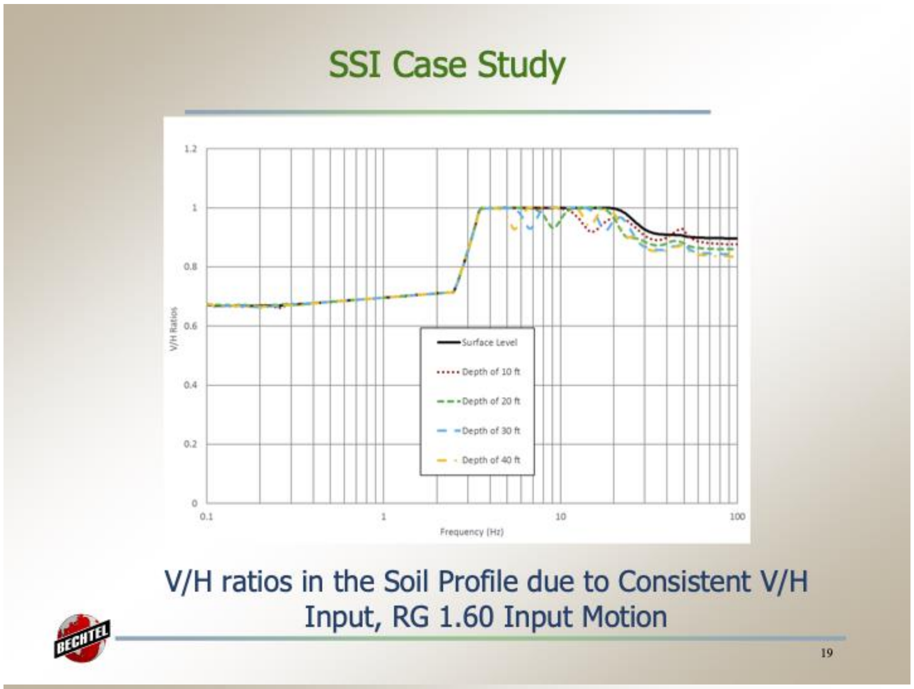


Adopted AP1000 Lumped Mass Stick Model (LMSM)

SSI Case Study

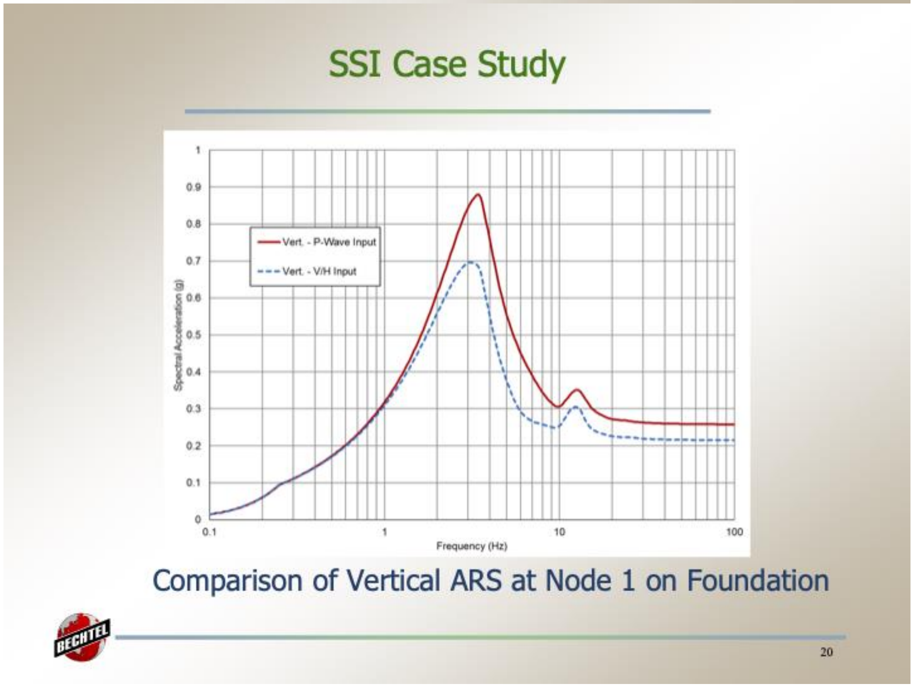


V/H ratios in the Soil Profile due to P-wave Propagation Input, RG 1.60 Input Motion



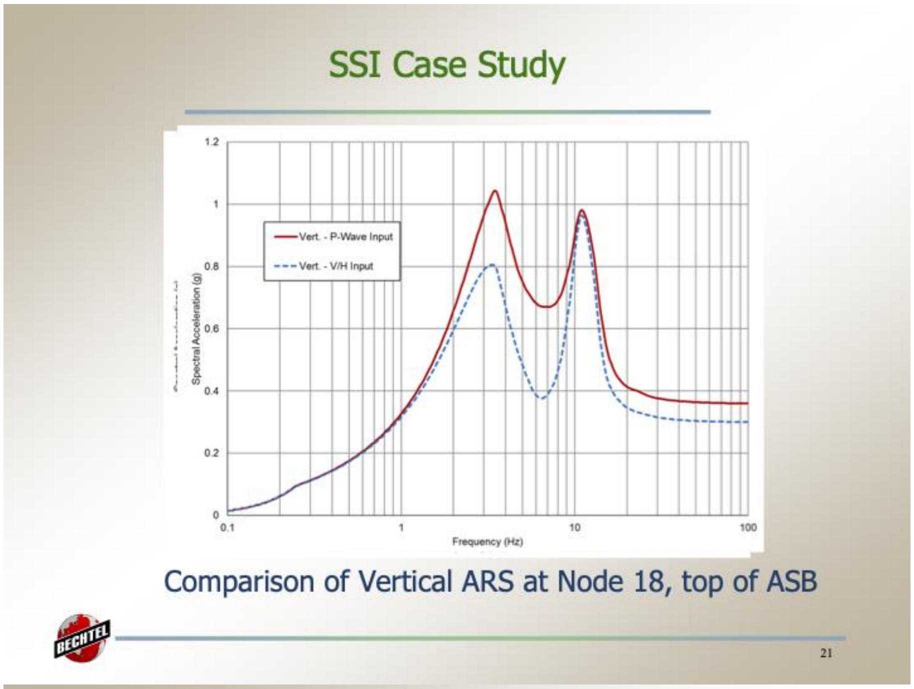
SSI Case Study

V/H ratios in the Soil Profile due to Consistent V/H Input, RG 1.60 Input Motion



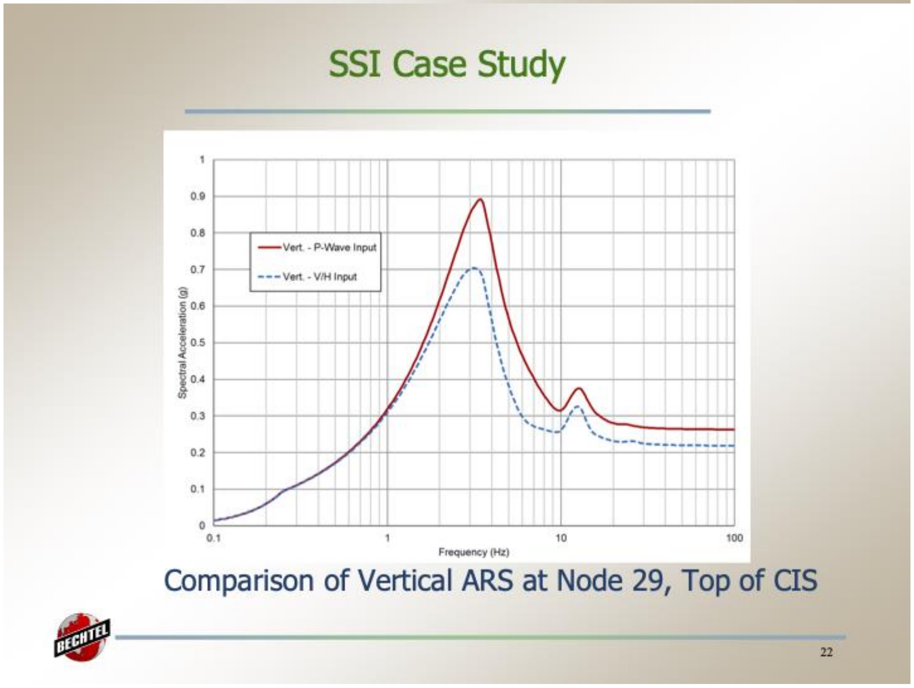
SSI Case Study

Comparison of Vertical ARS at Node 1 on Foundation



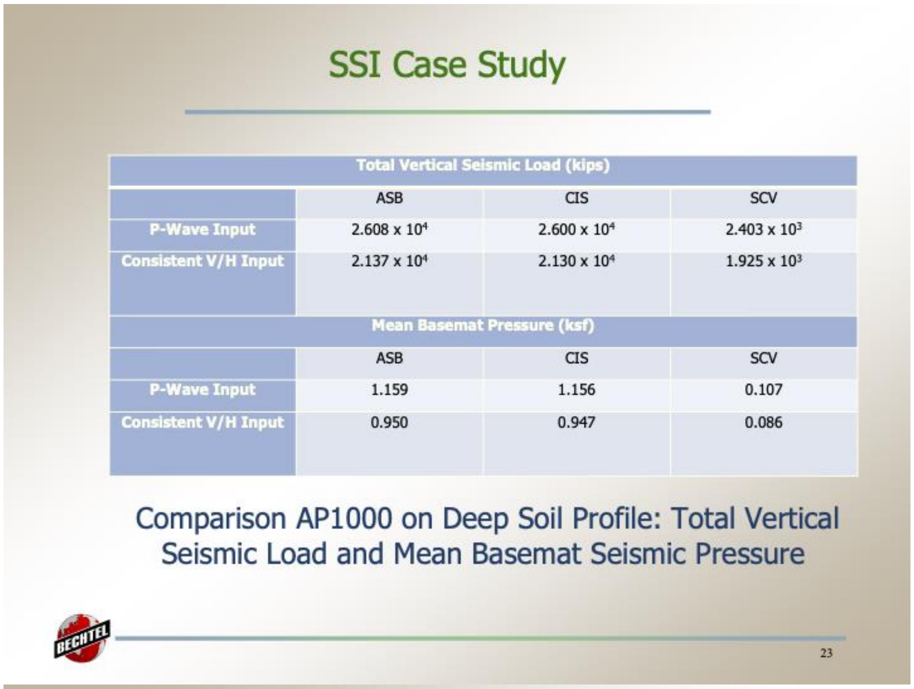
SSI Case Study

Comparison of Vertical ARS at Node 18, top of ASB



SSI Case Study

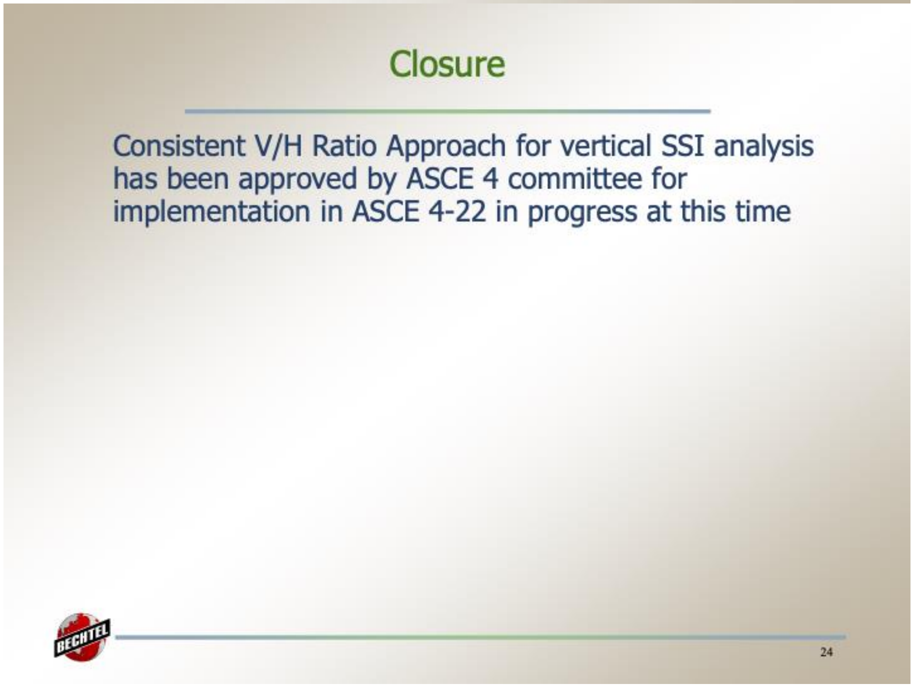
Comparison of Vertical ARS at Node 29, Top of CIS



SSI Case Study

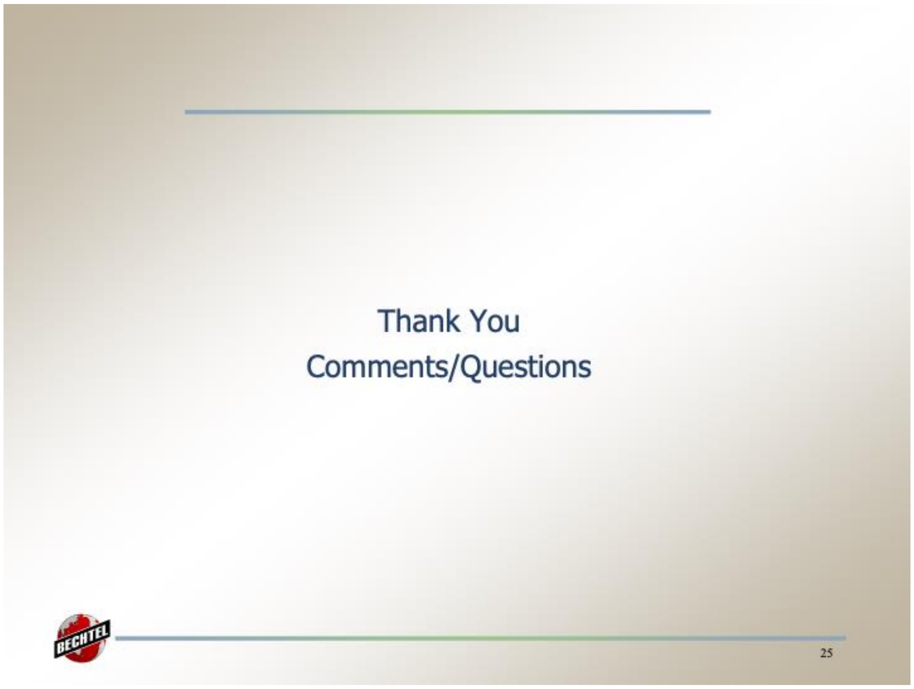
	Total Vertical Seismic Load (kips)		
	ASB	CIS	SCV
P-Wave Input	2.608 x 10^4	2.600 x 10^4	2.403 x 10^3
Consistent V/H Input	2.137 x 10^4	2.130 x 10^4	1.925 x 10^3
	Mean Basemat Pressure (ksf)		
	ASB	CIS	SCV
P-Wave Input	1.159	1.156	0.107
Consistent V/H Input	0.950	0.947	0.086

Comparison AP1000 on Deep Soil Profile: Total Vertical Seismic Load and Mean Basemat Seismic Pressure



Closure

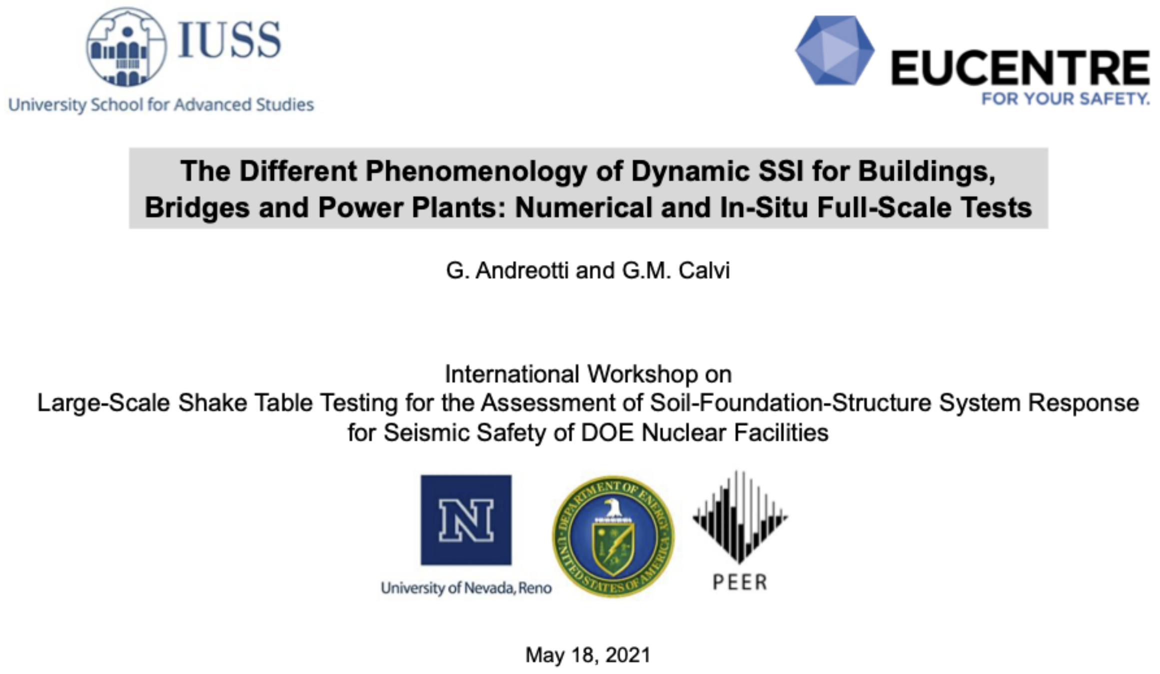
Consistent V/H Ratio Approach for vertical SSI analysis has been approved by ASCE 4 committee for implementation in ASCE 4-22 in progress at this time



Thank You

Comments/Questions

THE DIFFERENT PHENOMENOLOGY OF DYNAMIC SSI FOR BUILDINGS, BRIDGES AND POWER PLANTS: NUMERICAL AND IN-SITU FULL-SCALE TESTS



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EUCENTRE
FOR YOUR SAFETY.

The Different Phenomenology of Dynamic SSI for Buildings, Bridges and Power Plants: Numerical and In-Situ Full-Scale Tests

G. Andreotti and G.M. Calvi

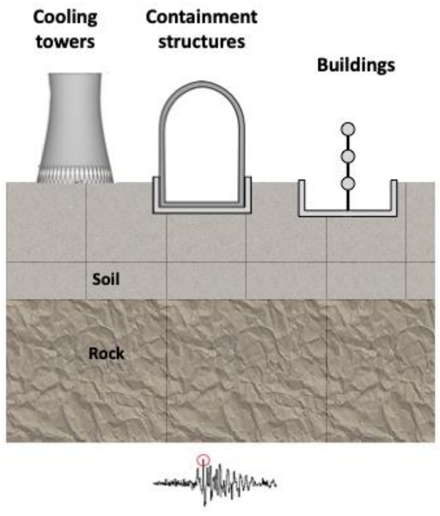
International Workshop on
Large-Scale Shake Table Testing for the Assessment of Soil-Foundation-Structure System Response
for Seismic Safety of DOE Nuclear Facilities

University of Nevada, Reno

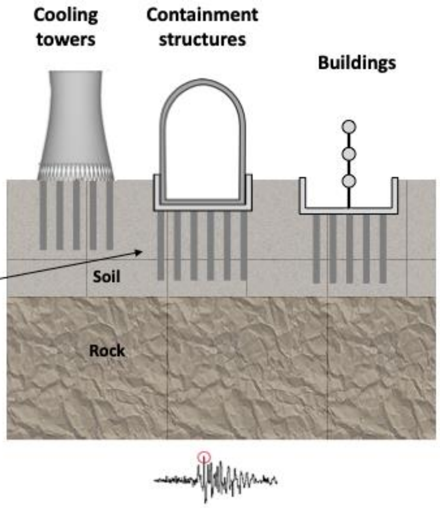
PEER

May 18, 2021

Above-ground structures of NPPs



Above-ground structures of NPPs

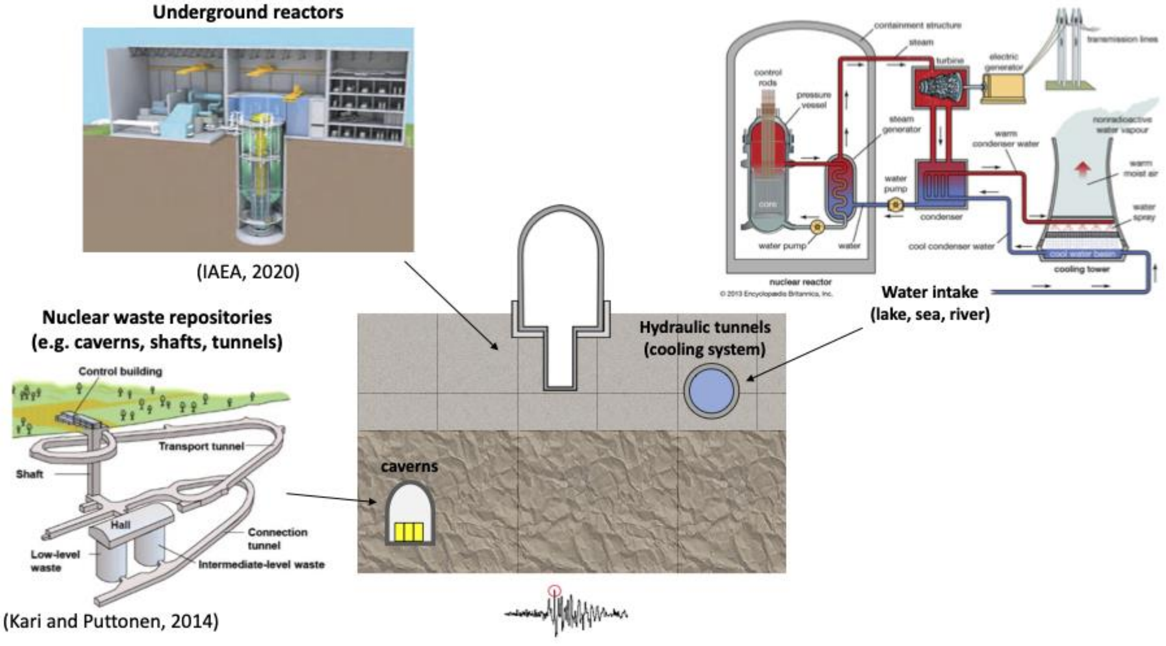


Containment on pile foundations
e.g.

- Point-Beach NPP (USA)
- H.B. Robinson NPP (USA)
- Angra NPP (Brazil)
- Gosgen-Daniken NPP (Switzerland)

(Zou et al., 2020)

Underground structures of NPPs



(IAEA, 2020)

(Kari and Puttonen, 2014)

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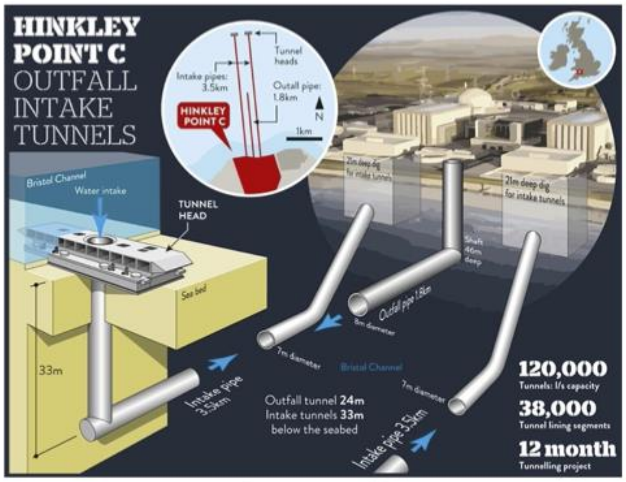
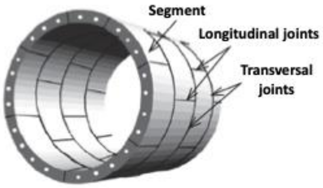
Examples of tunnels and shafts of the cooling system in NPPs



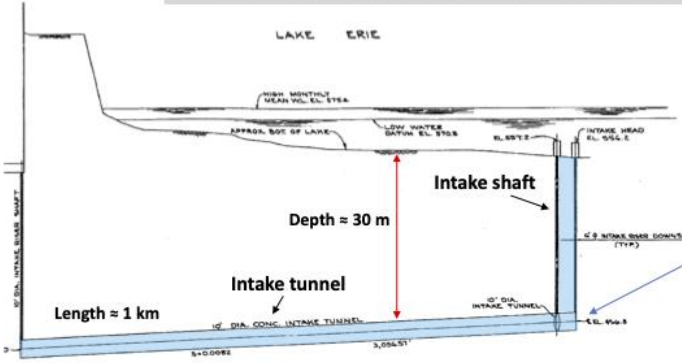
(Kennedy, 2019)



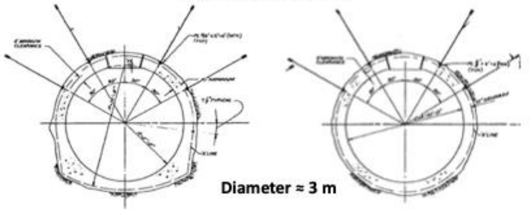
Segmental tunnels (precast segments)

Diameter = 7 m

Examples of tunnels and shafts of the cooling system in NPPs

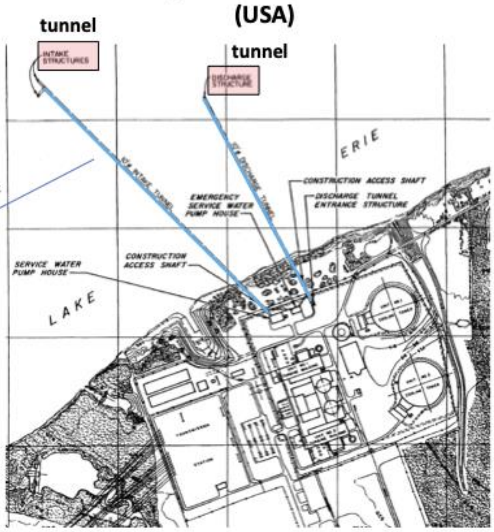


Traditional tunnel



Diameter ≈ 3 m

Perry Nuclear Power Plant (USA)



Nonlinear SSI analysis
(Structural nonlinearity)

ASCE 4-16: Seismic Analysis of Safety-Related Nuclear Structures

Chapter 5 - SSI

(5.1 GENERAL REQUIREMENTS): (a) **SSI effects shall be considered for all safety-related nuclear structures**.

Chapter 4 - Analysis of structures

(4.1 GENERAL REQUIREMENTS): (a) The seismic analysis of safety-related structures is **typically** performed by analysis of **linearly elastic** mathematical **models**. **Nonlinear analysis may be performed in some cases**, especially for **beyond design basis calculations** or **evaluation of existing facilities**.

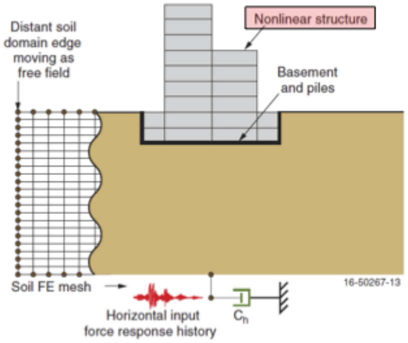
Design peak ground acceleration and recorded peak ground acceleration at NPPs.

	Kashiwazaki-Kariwa, Japan	Fukushima Daiichi, Japan	North Anna, USA
Design value	0.20 g	0.26 g[a]	0.18 g
Recorded value (year)	0.32 g (2007)	0.56 g (2011)	0.26 g (2011)

[a] Design basis updated in 2009 to 0.45 g (The National Diet of Japan, 2012).

(Coleman et al., 2016)

Recorded seismic demand exceeded design value



Nonlinear SSI analysis
(Structural nonlinearity)

ASCE 4-16: Seismic Analysis of Safety-Related Nuclear Structures

Chapter 4 - Analysis of structures

(4.1 GENERAL REQUIREMENTS): (a) The seismic analysis of safety-related structures is **typically** performed by analysis of **linearly elastic** mathematical models. **Nonlinear analysis may be performed in some cases**, especially for **beyond design basis calculations** or **evaluation of existing facilities**. **(or definition of fragility curves)**

Design peak ground acceleration and recorded peak ground acceleration at NPPs.

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Design value	0.20 g	0.26 g[a]	0.18 g
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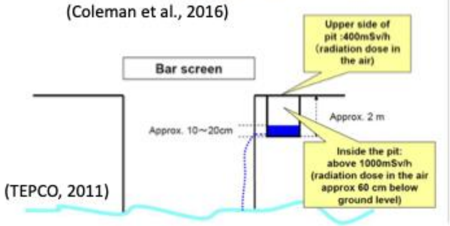
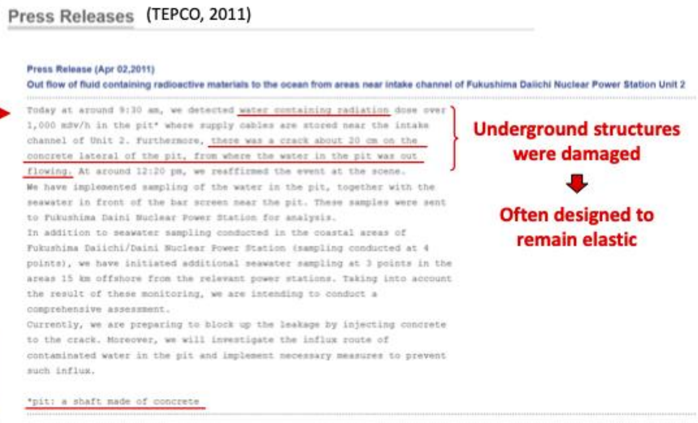


(TEPCO, 2011)



Press Releases (TEPCO, 2011)

Press Release (Apr 02,2011)

Out flow of fluid containing radioactive materials to the ocean from areas near intake channel of Fukushima Daiichi Nuclear Power Station Unit 2

Today at around 9:30 am, we detected water containing radiation dose over 1,000 mSv/h in the pit* where supply cables are stored near the intake channel of Unit 2. Furthermore, there was a crack about 20 cm on the concrete lateral of the pit, from where the water in the pit was out flowing. At around 12:20 pm, we reaffirmed the event at the scene.

We have implemented sampling of the water in the pit, together with the seawater in front of the bar screen near the pit. These samples were sent to Fukushima Daini Nuclear Power Station for analysis.

In addition to seawater sampling conducted in the coastal areas of Fukushima Daiichi/Daini Nuclear Power Station (sampling conducted at 4 points), we have initiated additional seawater sampling at 3 points in the areas 15 km offshore from the relevant power stations. Taking into account the result of these monitoring, we are intending to conduct a comprehensive assessment.

Currently, we are preparing to block up the leakage by injecting concrete to the crack. Moreover, we will investigate the influx route of contaminated water in the pit and implement necessary measures to prevent such influx.

*pit: a shaft made of concrete

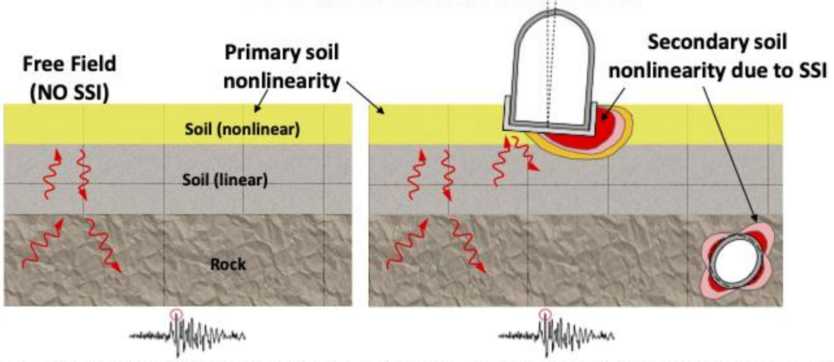
Underground structures were damaged

Often designed to remain elastic

Nonlinear SSI analysis
(Soil nonlinearity)

ASCE 4-16: Seismic Analysis of Safety-Related Nuclear Structures

Chapter 5 – SSI: Nonlinear Behavior of Soil: **Primary and secondary soil nonlinearity**

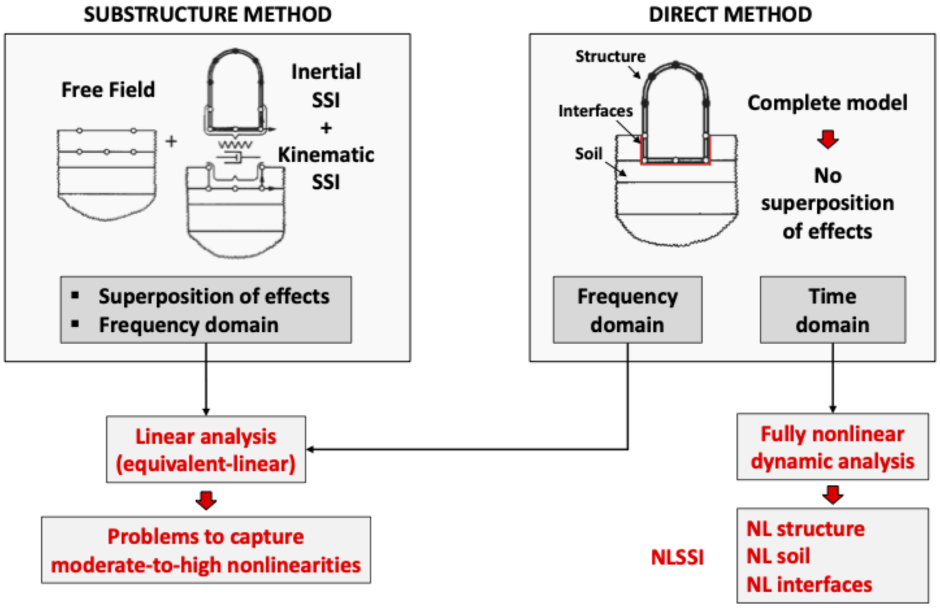


5.1.4 (d) **Primary nonlinearities shall be considered** in the SSI analysis. **Secondary nonlinearities,** including local soil nonlinear behavior in the vicinity of the soil-structure interface, **need not be considered,** except for the calculation of seismic soil pressure.

COMMENTARY: C5.1.4 Nonlinear Behavior of Soil

(…) **rigorous nonlinear analysis** of a typical nuclear structure requires a fully three-dimensional model and an appropriate set of constitutive equations for soil. These requirements **are currently beyond the state of the art for design**.

Solution of dynamic SSI: linear vs nonlinear



SUBSTRUCTURE METHOD

Free Field + **Inertial SSI + Kinematic SSI**

- **Superposition of effects**
- **Frequency domain**

DIRECT METHOD

Complete model

No superposition of effects

Frequency domain

Time domain

Linear analysis (equivalent-linear)

Problems to capture moderate-to-high nonlinearities

Fully nonlinear dynamic analysis

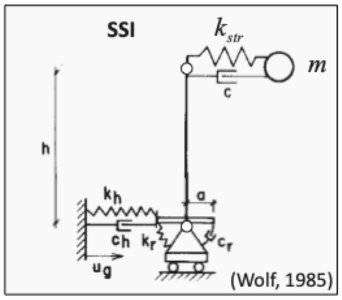
NLSSI

NL structure
NL soil
NL interfaces

Some questions

Soil-structure stiffness ratio $\frac{K_{struct}}{K_{soil}}$ ➡ One of the main variables of SSI

Simple models for above-ground structures



(Wolf, 1985)

⬇

Fixed base

$$\omega_{FixB} = \sqrt{\frac{k_{str}}{m}}$$

$$\xi_{str}$$

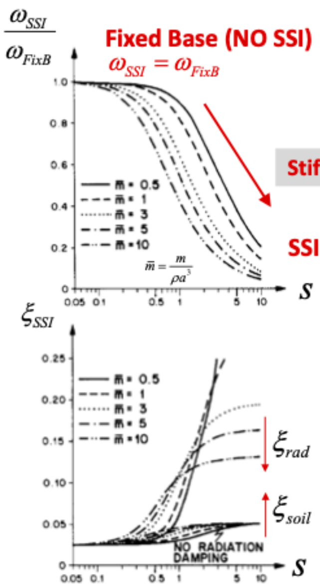
Equivalent SSI

$$\omega_{SSI} = \sqrt{\frac{k_{eq}}{m}}$$

$$\xi_{SSI} = \xi_{str} + \xi_{soil} + \xi_{rad}$$

$$\frac{1}{k_{eq}} = \frac{1}{k_{str}} + \frac{1}{k_h} + \frac{h^2}{k_r}$$

Reference model ➡ **Fixed base**



Stiffness contrast

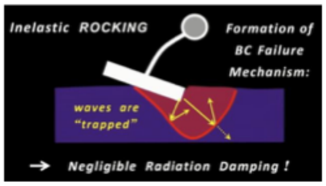
SSI

Structural stiffness

$$s = \frac{\omega_{FixB} h}{V_S}$$

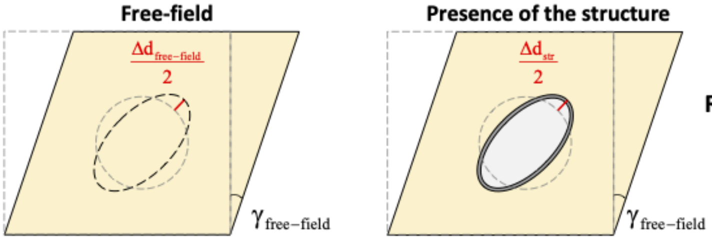
↓

Soil stiffness (shear wave vel)



(Prof. Gazetas, Rankine lecture 2019)

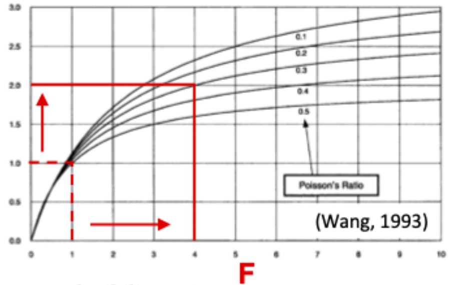
Simple models for underground structures



Reference model ➡ **Free-field (no structure)**

Displacement structural demand

$$\frac{\Delta d_{str}}{\Delta d_{free-field}}$$



Flexibility ratio

$$\uparrow F = \frac{E_{soil} \cdot (1-\nu_{str}^2)r^3}{(1+\nu_{soil}) \cdot 6E_{str}I_{str}} = \frac{K_{soil}}{K_{str}} \downarrow$$

Stiffness contrast

Could not be constant during earthquakes
⬇
Evolution of soil and structural nonlinearity

Some questions

Soil-structure stiffness ratio $\frac{K_{struct}}{K_{soil}}$ ➡ One of the main variables of SSI

NLSSI implies nonlinearity of the whole system (structure + soil) ➡ Stiffness ratio is not constant

- **What happens to the whole system with the evolution of structural damage and soil nonlinearity?**
- **Does structural nonlinearities affect soil nonlinearity?**
- **Does soil properties affect structural capacity?**

YES numerical evidence

Two case studies:

1) Tunnels ➡ 2D fully nonlinear time history analysis
2) CIDH bridge columns ➡ 3D detailed modelling with experimental benchmark

- **Is the gap between geotechnical and structural engineers detrimental to solve NLSSI problems?**

1) Fragility analysis of underground tunnels: fully nonlinear SSI

M_w=7.9
Epicentral distance ≈ 14.2 Km
2008 Wenchuan earthquake (China)

(Li, 2012)

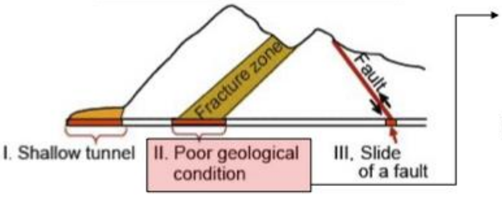
M_w=6.9
Epicentral distance < 15 Km
1980 Irpinia earthquake (Italy)

(Cotecchia, 1986)

M_w=7.4
Epicentral distance < 32 Km
1990 Manjil earthquake (Iran)

(Wieland, 2011)

Critical scenarios for tunnels



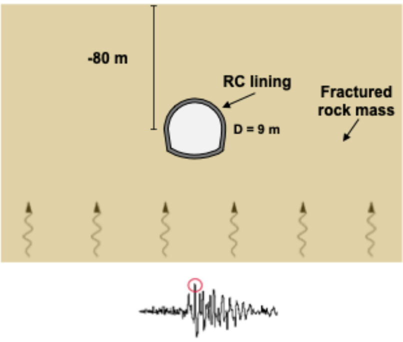
Early objective

Definition of numerical fragility curves considering variability of seismic input, structural and geotechnical nonlinearities, depth of tunnel.
(see Andreotti and Lai, 2017a, 2017b, 2019)

From the suite of analysis: investigation on NLSSI effects

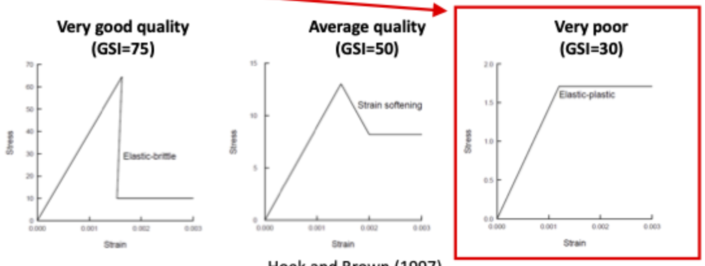
Fragility analysis of underground tunnels: fully nonlinear SSI

Selected scenario to evaluate NLSSI



Rock mass parameters

GSI	E [MPa]	ν	ϕ [°]	dilatancy [°]	c [MPa]	γ [kN/m³]	K₀
25	1100	0.3	22	0	0.5	24	0.6



Hoek and Brown (1997)

Mohr-Coulomb model

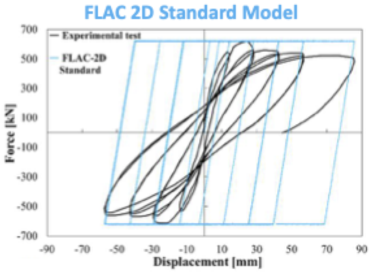
RC Tunnel lining

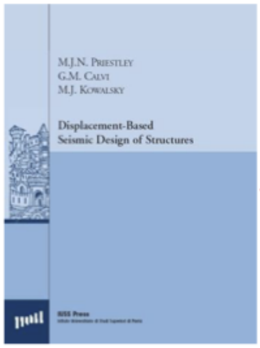
Type		Reinforcement		Concrete			Steel	
Section	Thickness [m]	Steel rebars [kg/m³]	Stirrups [cm²]	E_c [MPa]	f_c [MPa]	f_t [MPa]	E_s [MPa]	f_s [MPa]
S1	0.7	50	14.13	33000	25.5	2.55	210000	450
S2	1	80	14.13					

FLAC 2D

Problems to model cyclic nonlinear behaviour of RC structures

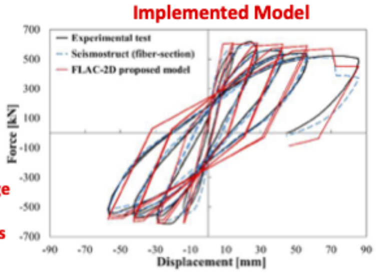
Modelling nonlinear cyclic behaviour of structural elements and damage assessment



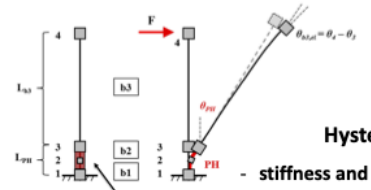


Application of concepts

Transfer of knowledge from structural to geotechnical systems



Elementary part

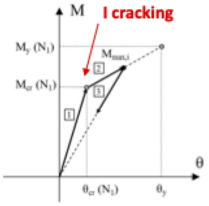


Finite length inelastic zone (plastic hinge region)

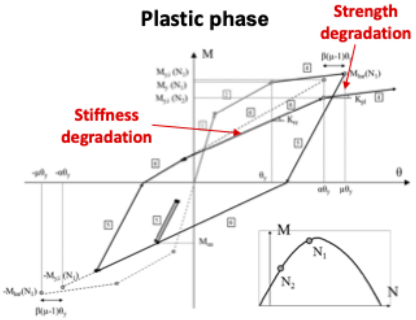
Hysteretic law

- **stiffness and strength degradation**
- **Axial interaction (M-N)**

Bi-linear elastic phase



Plastic phase

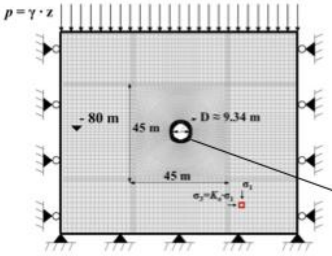


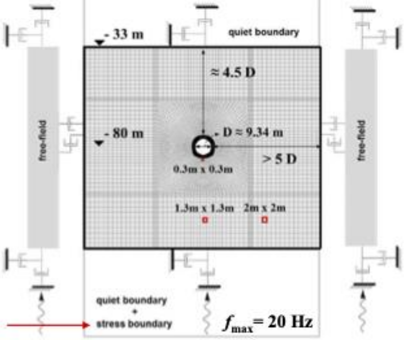
FLAC 2D numerical model

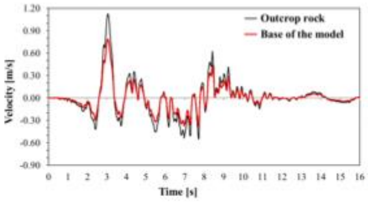
Multi-step static analysis

Initial conditions

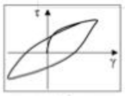
Nonlinear time history analysis



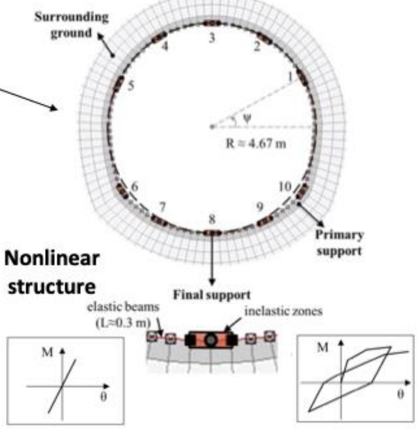




Nonlinear soil

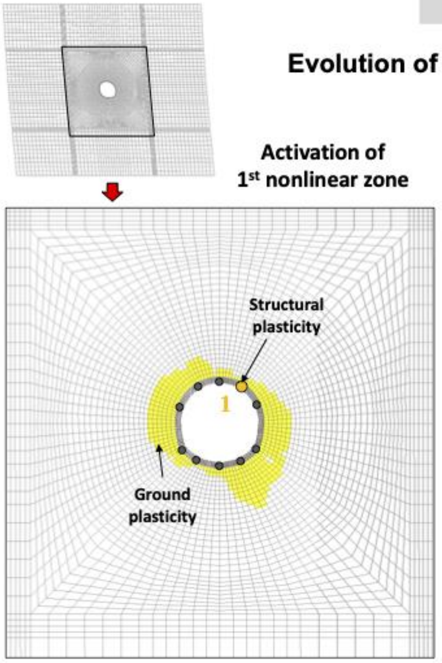


Nonlinear structure



Results: NLSSI tunnel

Evolution of ground and structural nonlinearity



Ground Plasticity Index

$$\mathrm{GPI} = \frac{N^{pl}_{dyn}}{N^{pl}_{static}} - 1$$

n° plastic zones dyn analysis

n° plastic zones static analysis

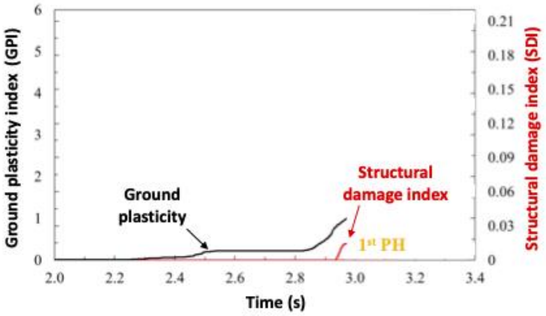
Structural Damage Index

$$\mathrm{SDI} = \sum_{i=1}^{n} \frac{\theta^{i}_{pl}}{\theta^{i}_{pl,u}}$$

n° of PH

Plastic rotation

Plastic capacity



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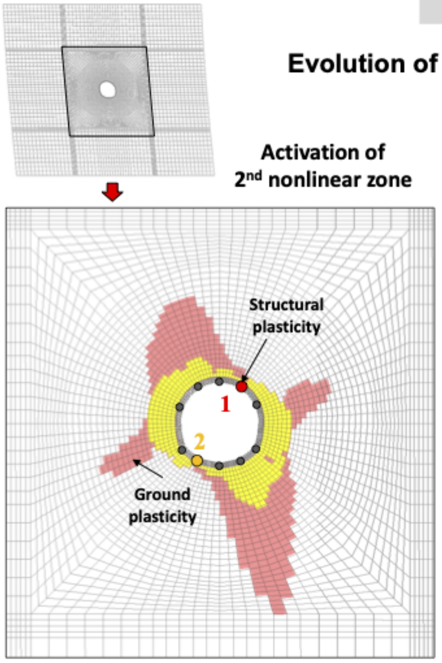

Ground Plasticity Index

$$\mathrm{GPI} = \frac{N^{pl}_{dyn}}{N^{pl}_{static}} - 1$$

n° plastic zones dyn analysis

n° plastic zones static analysis

Structural Damage Index

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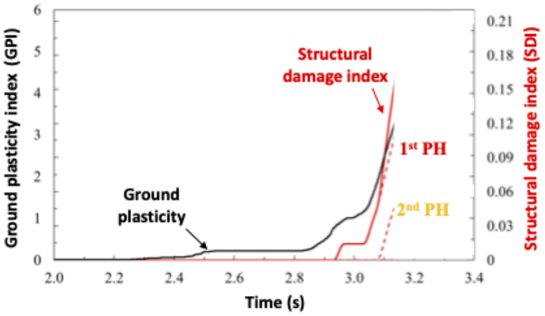
n° of PH

Plastic rotation

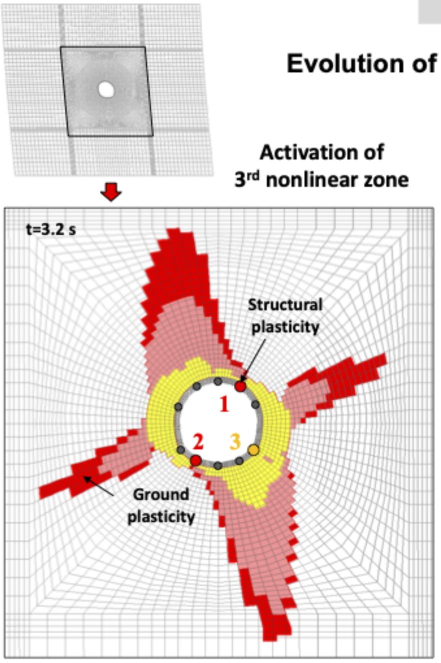
Plastic capacity



Results: NLSSI tunnel

Evolution of ground and structural nonlinearity

Activation of 3rd nonlinear zone



Ground Plasticity Index

$$GPI = \frac{N^{pl}_{dyn}}{N^{pl}_{static}} - 1$$

n° plastic zones dyn analysis

n° plastic zones static analysis

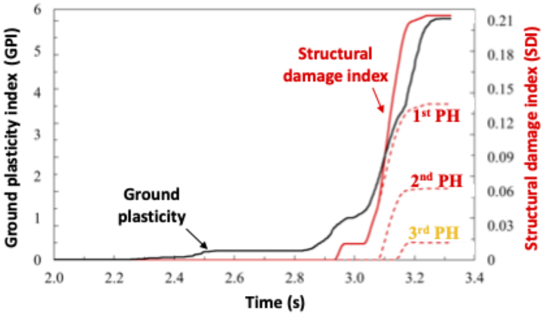
Structural Damage Index

$$SDI = \sum_{i=1}^{n} \frac{\theta^{i}_{pl}}{\theta^{i}_{pl,u}}$$

n° of PH

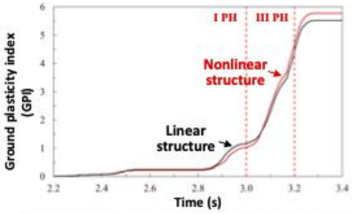
Plastic rotation

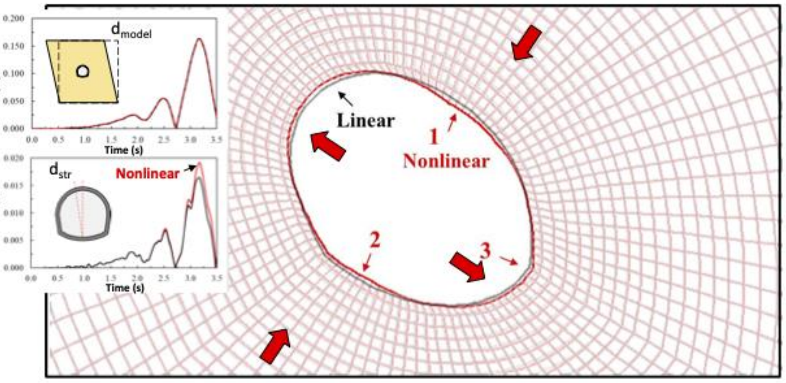
Plastic capacity

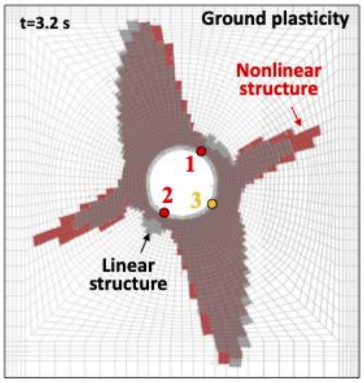


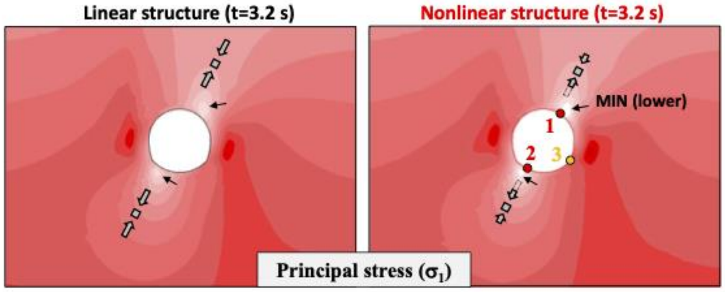
Results: NLSSI tunnel

Linear vs Nonlinear structure



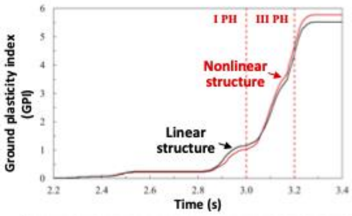


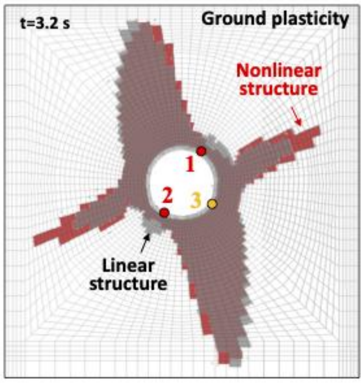


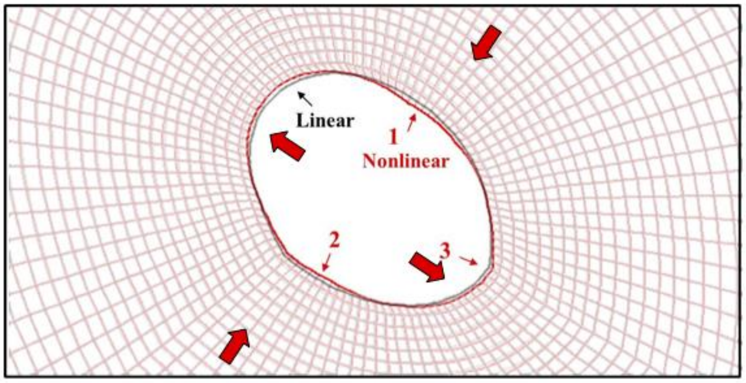


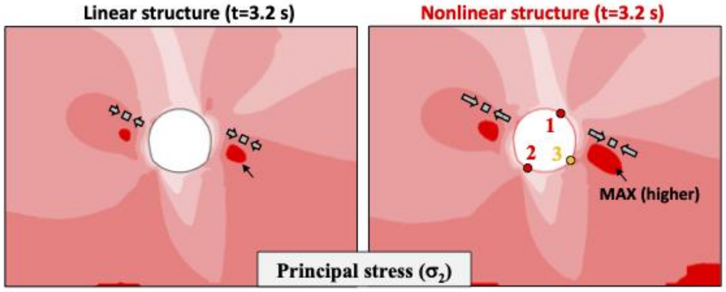
Results: NLSSI tunnel

Linear vs Nonlinear structure







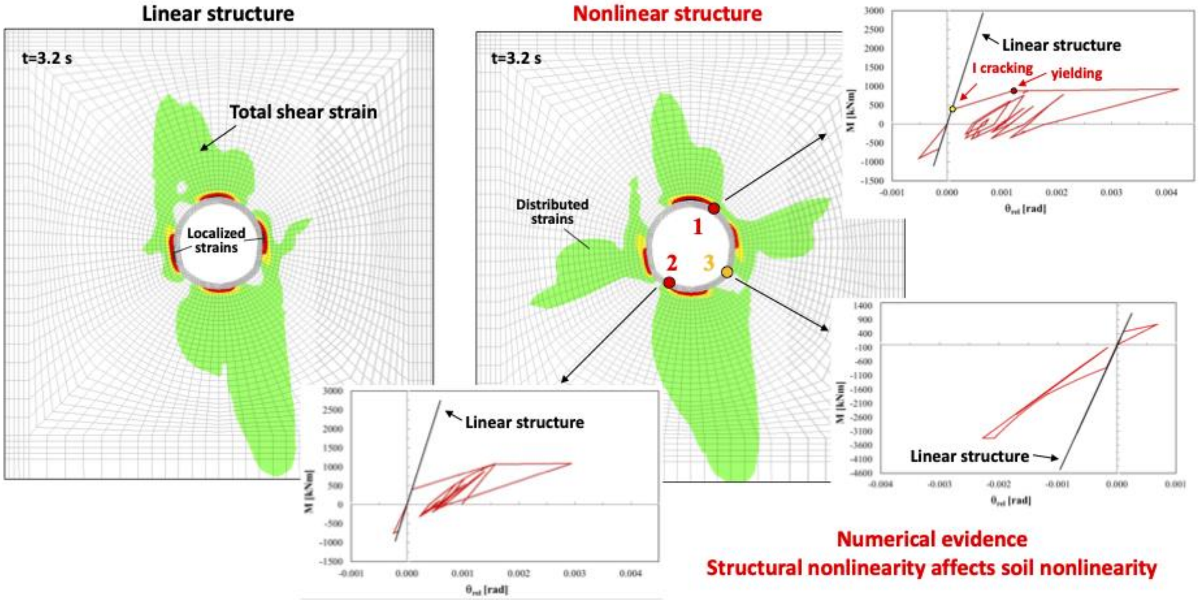

Linear structure (t=3.2 s)

Nonlinear structure (t=3.2 s)

Principal stress (σ_2)

Results: NLSSI tunnel

Linear vs Nonlinear structure: shear strains and structural deformations


Linear structure

Nonlinear structure

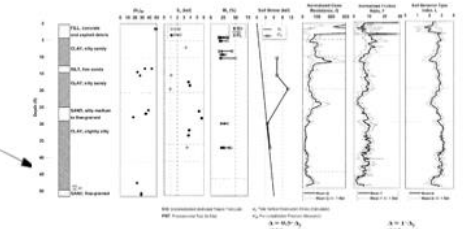
Numerical evidence
Structural nonlinearity affects soil nonlinearity

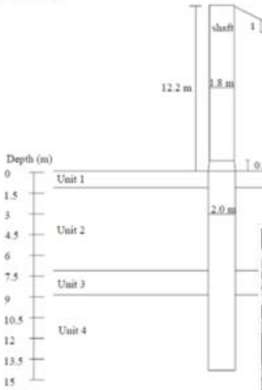
2) Detailed modelling of CIDH bridge column

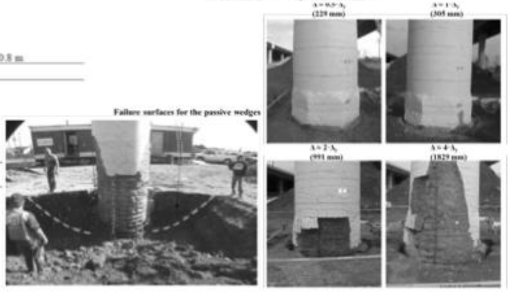
Experimental benchmark: full-scale cyclic test by UCLA (Janoyan, Wallace and Stuart, 2006)

Why this test?

- Severe nonlinearity of both soil and structure
- Full-scale test
- High quality experimental data: soil + structure

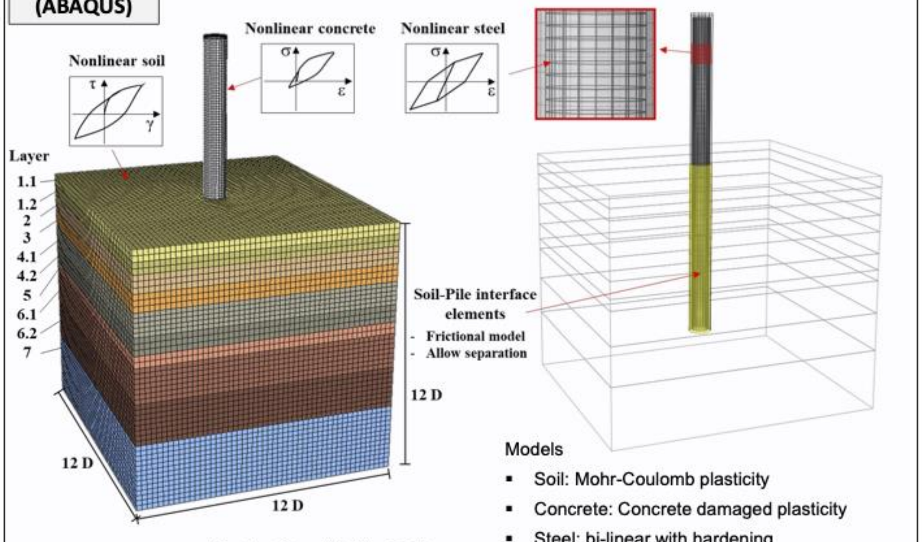






2) Detailed modelling of CIDH bridge column

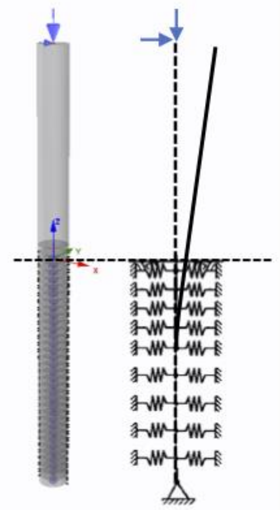
3D MODEL (ABAQUS)



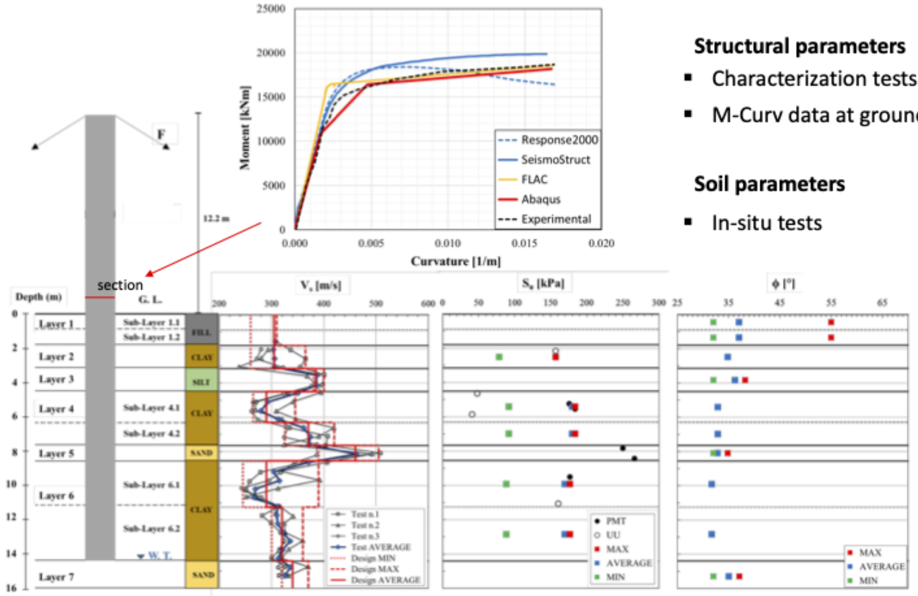
Models
- Soil: Mohr-Coulomb plasticity
- Concrete: Concrete damaged plasticity
- Steel: bi-linear with hardening
- Interface: Frictional model allowing separation

(Andreotti and Calvi, 2021)

Fiber sections and p-y curves (SeismoStruct)



Calibration of model parameters



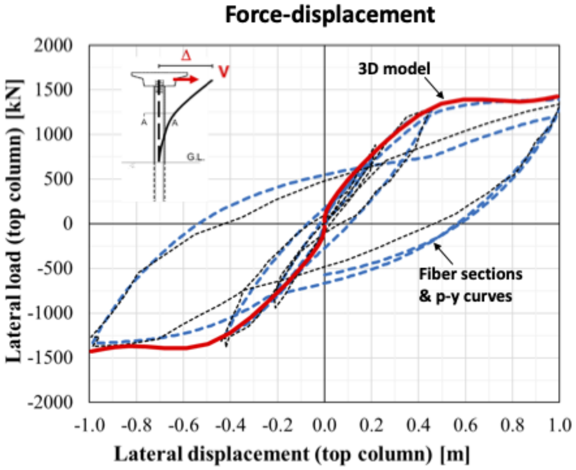
Structural parameters

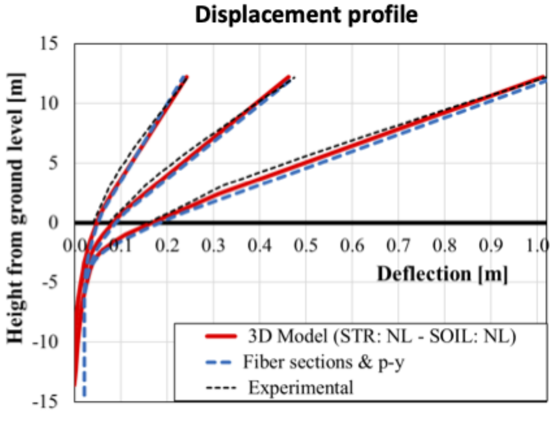
- Characterization tests small specimens
- M-Curv data at ground level

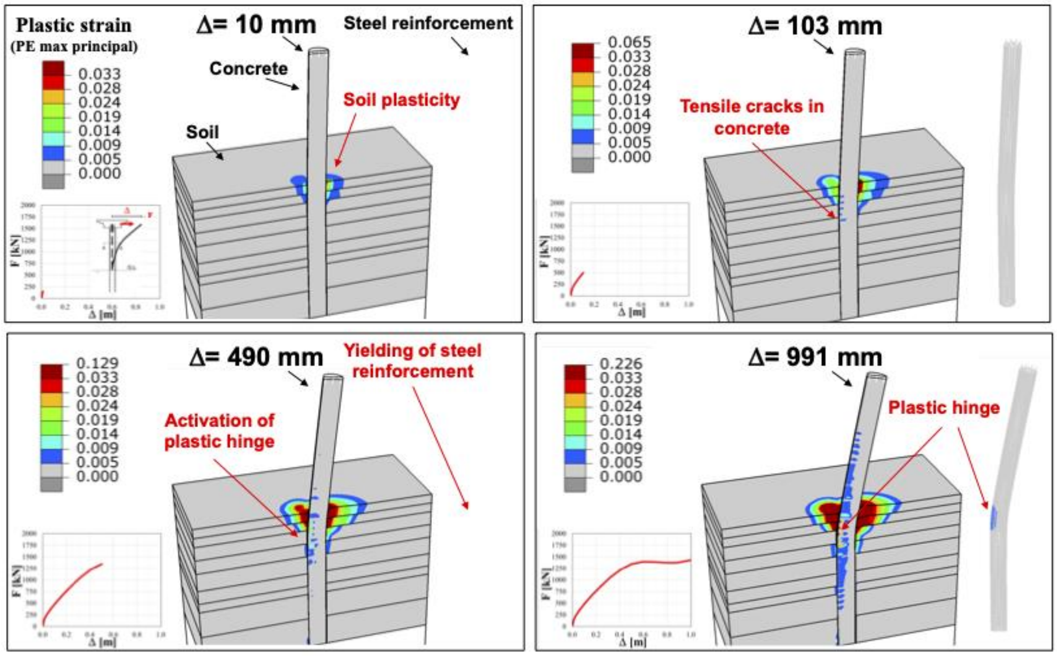
Soil parameters

- In-situ tests

Comparison 3D model and fiber sections/p-y curves


Force-displacement

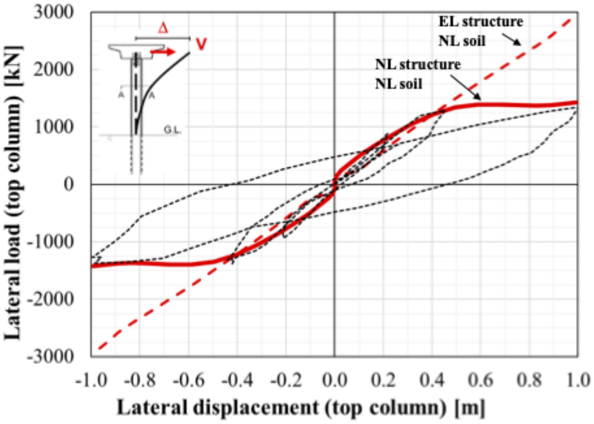

Displacement profile



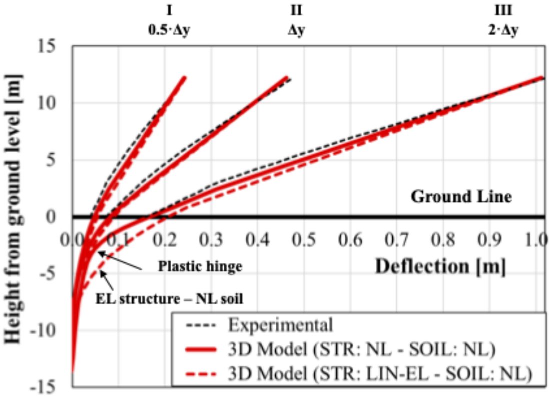
3D model: Elastic vs Nonlinear structure

Results with identical soil and different structural behaviour (LIN vs NL)

Force-displacement

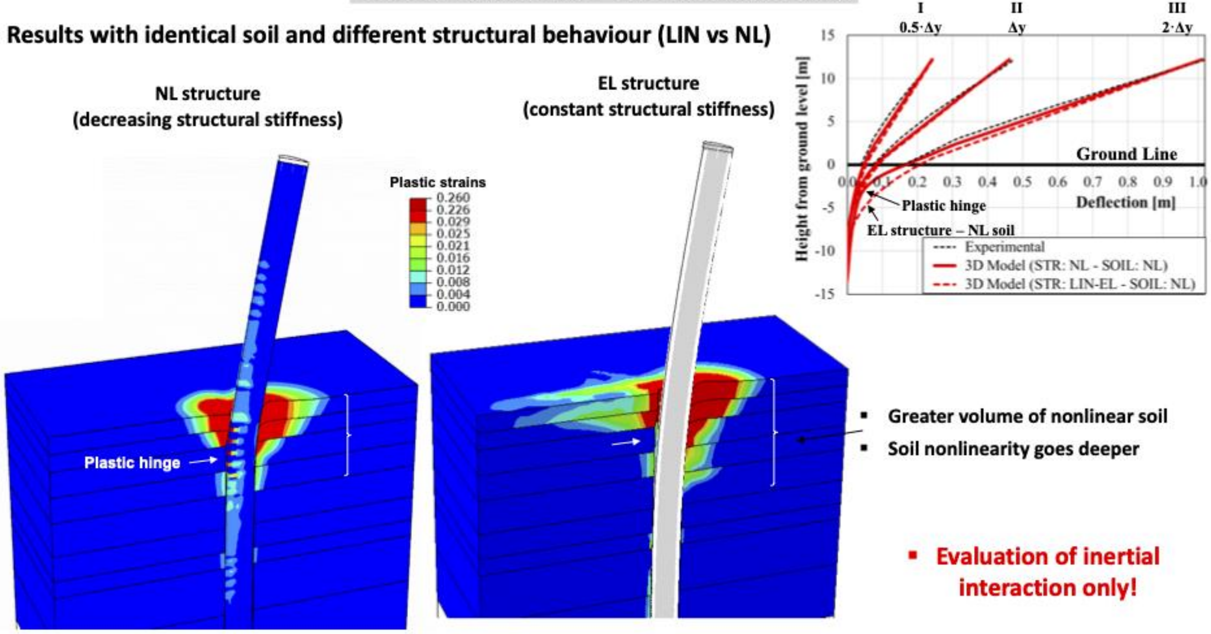


Displacement profile



Structural influence on soil nonlinearity

Results with identical soil and different structural behaviour (LIN vs NL)

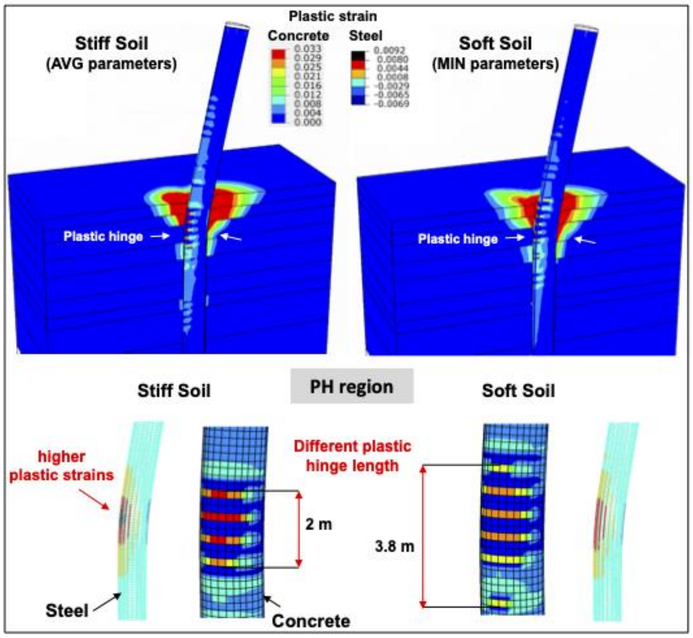


- **Greater volume of nonlinear soil**
- **Soil nonlinearity goes deeper**

- **Evaluation of inertial interaction only!**

Identical structure different soil

Influence of soil stiffness on structural nonlinearity



Stiff soils

- Concentration of strains (soil & structure)
- Smaller plastic hinge length
- Higher curvature demand

Soft soils

- Distribution of strains
- Larger plastic hinge length
- Lower curvature demand

Numerical evidence
Soil properties affect structural capacity

Seismic demand is modified by secondary soil nonlinearity

Seismic risk
- **Hazard**
- **Vulnerability**
- **Exposure**

NLSSI

How much?

Structural capacity affected by soil properties

Experimental tests: issues of NLSSI

- Centrifuge tests

Artificial gravity field

Small scale structural models (1/20 – 1/50)

➡ **Correct reproduction of soil properties and confinement**

Problems to reproduce detailing and damage of reinforced concrete elements

- Shake tables with small soil boxes

Larger structural models (scale 1/10 – 1/5)

Small soil vertical stress

➡ **Less problems to reproduce RC elements**

Problems to reproduce soil properties and confinement

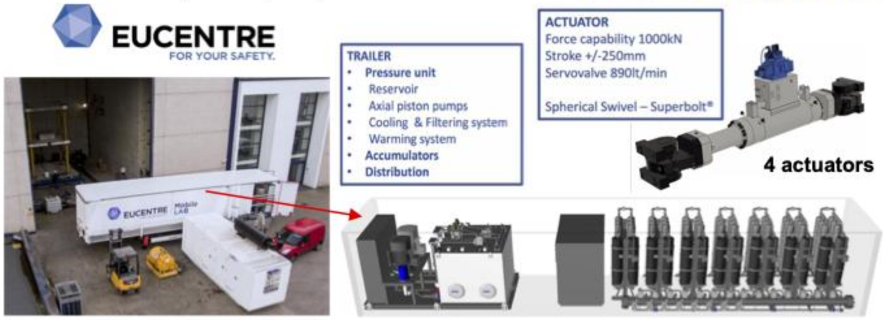
- Mobile laboratory for in-situ dynamic tests

(Calvi et al., 2021)

➡ **Full scale tests in-situ (dynamic inertial SSI interaction)**

Feasibility study to reproduce kinematic SSI interaction

EUCENTRE
FOR YOUR SAFETY.

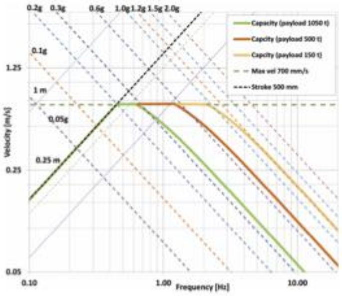


TRAILER
- **Pressure unit**
- Reservoir
- Axial piston pumps
- Cooling & Filtering system
- Warming system
- **Accumulators**
- **Distribution**

ACTUATOR
Force capability 1000kN
Stroke +/-250mm
Servovalve 890lt/min

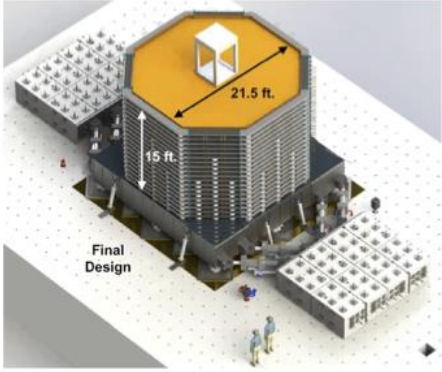
Spherical Swivel – Superbolt®

4 actuators



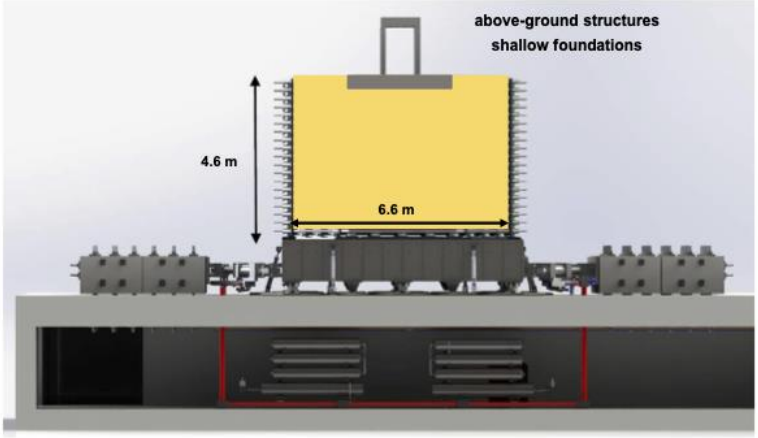
Experimental tests on NLSSI: fill the gap between research and practice

New shake table with big soil box
University of Nevada, Reno



Important contributions to dynamic NLSSI ➡

- Structural damage of RC elements
- Failure mechanisms of the system
- Kinematic and inertial interaction
- Soil properties
- Primary and secondary soil nonlinearity
- Damping (e.g. radiation)



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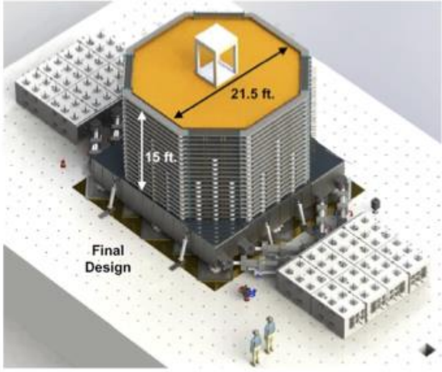

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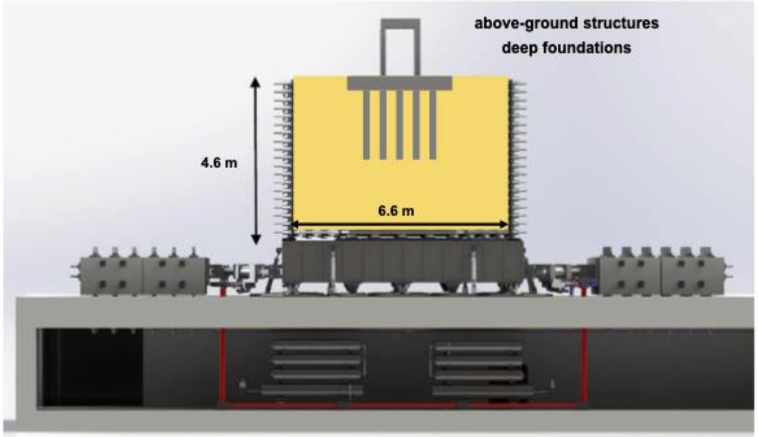

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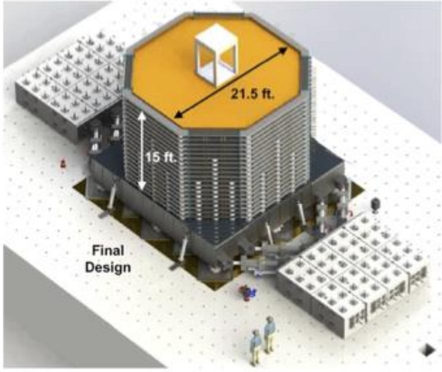

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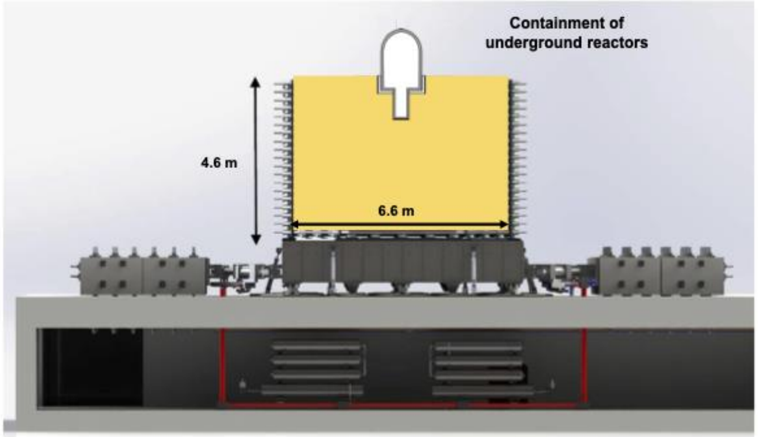

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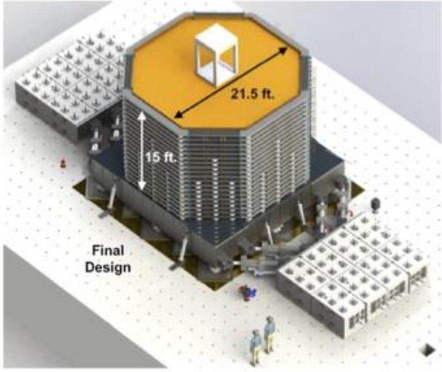

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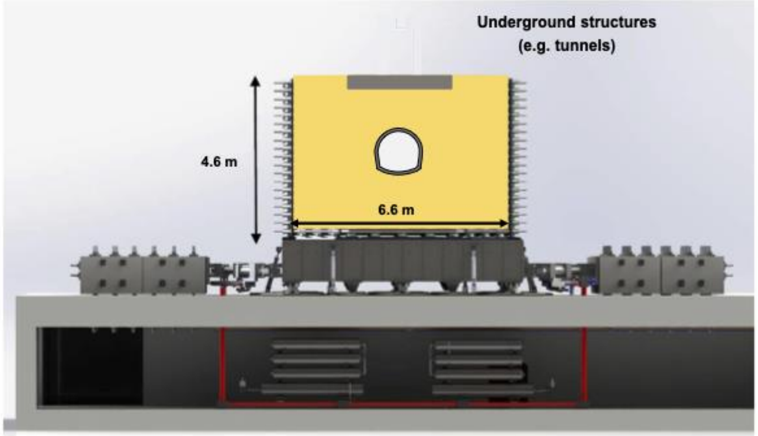

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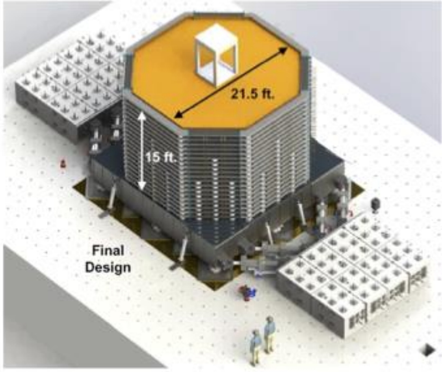

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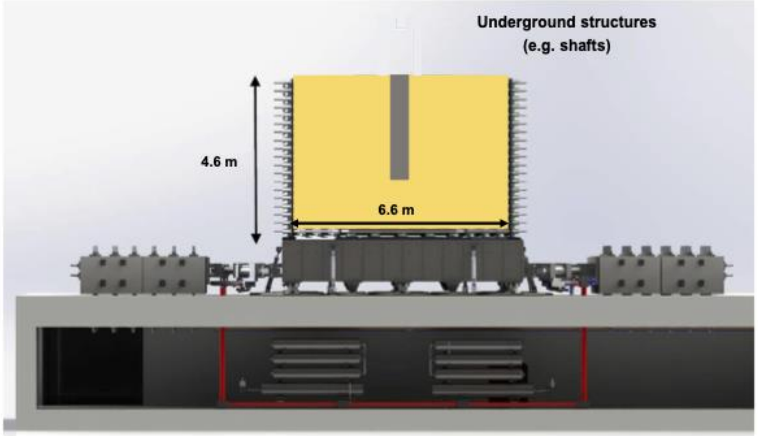

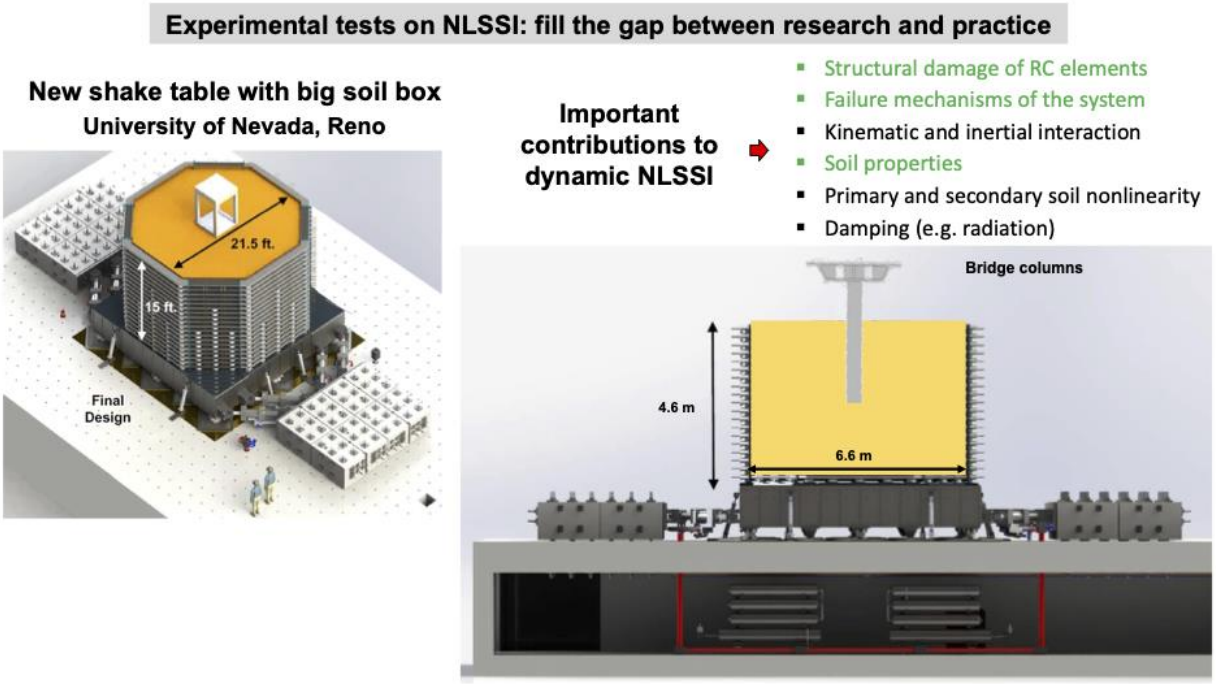

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Is the gap between geotechnical and structural engineers detrimental to solve NLSSI problems?

> *"If our small minds, for some convenience, divide this glass of wine, this universe, into parts -- physics, biology, geology, astronomy, psychology, and so on -- remember that nature does not know it! So let us put it all back together, not forgetting ultimately what it is for. Let it give us one more final pleasure; drink it and forget it all!"*
>
> Prof. Richard P. Feynman

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[2] European Centre for Training and Research in Earthquake Engineering, Via A. Ferrata 1, 27100 Pavia, Italy

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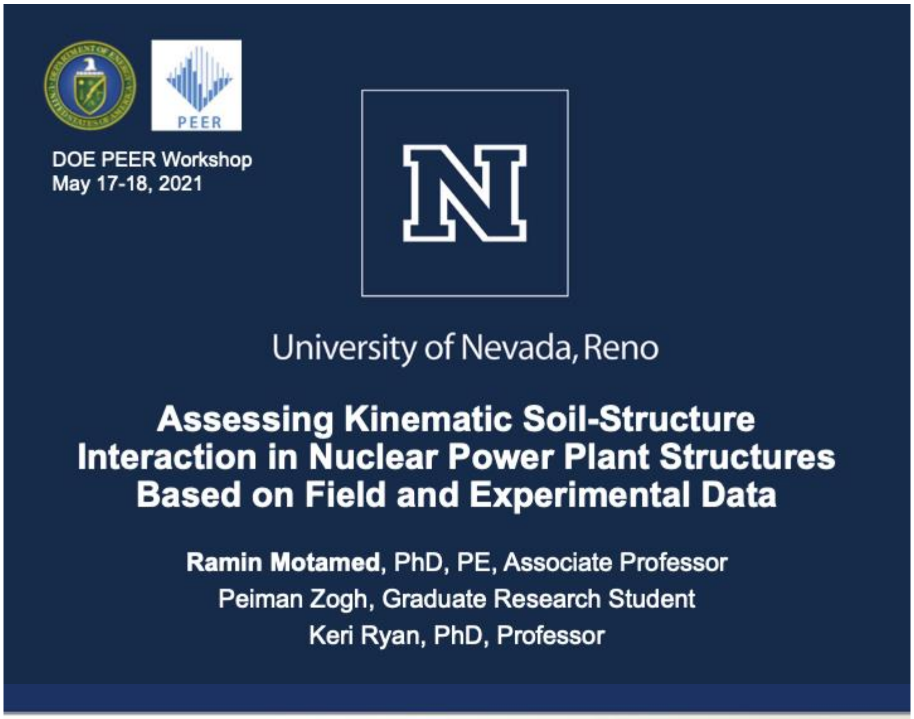
ASSESSING KINEMATIC SOIL-STRUCTURE INTERACTION IN NUCLEAR POWER PLANT STRUCTURES BASED ON FIELD AND EXPERIMENTAL DATA

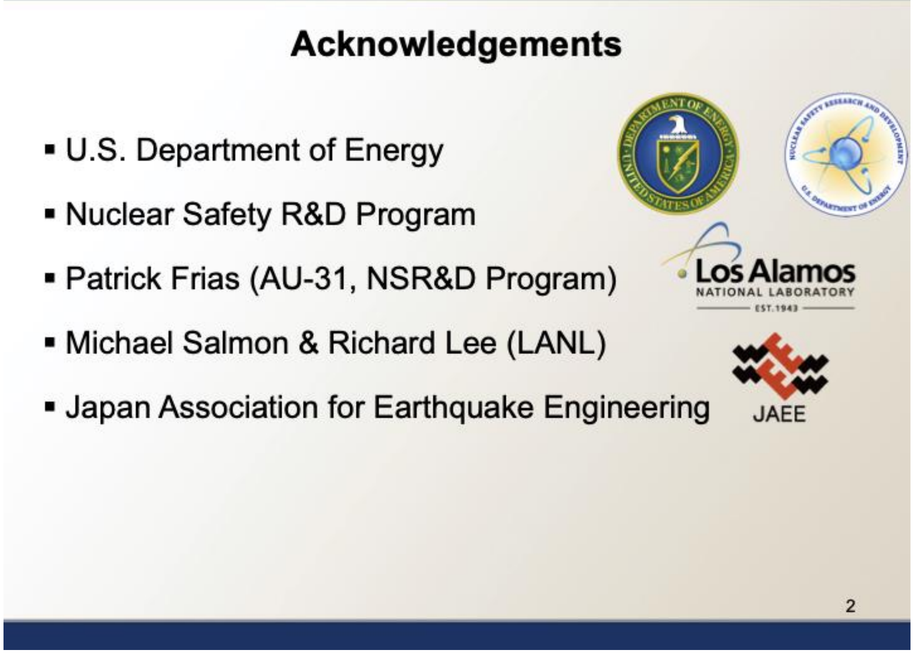


DOE PEER Workshop
May 17-18, 2021

University of Nevada, Reno

Assessing Kinematic Soil-Structure Interaction in Nuclear Power Plant Structures Based on Field and Experimental Data

Ramin Motamed, PhD, PE, Associate Professor
Peiman Zogh, Graduate Research Student
Keri Ryan, PhD, Professor



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- U.S. Department of Energy
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- Patrick Frias (AU-31, NSR&D Program)
- Michael Salmon & Richard Lee (LANL)
- Japan Association for Earthquake Engineering

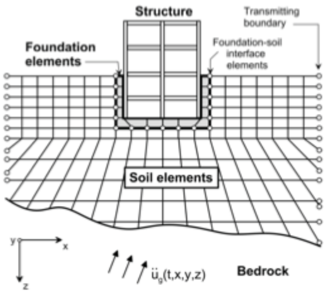
Presentation Outline

- Soil-Structure Interaction and Kinematic Effects
- Two Example Instrumented NPP Sites
- Code-Based Simplified Procedure (ASCE 41/17)
- Kinematic SSI Assessment at Select NPP Sites
- On-going Work
- Concluding Remarks

Soil-Structure Interaction (SSI)

❑ The response of a structure to earthquake shaking is affected by interactions between three linked components (ATC-83):

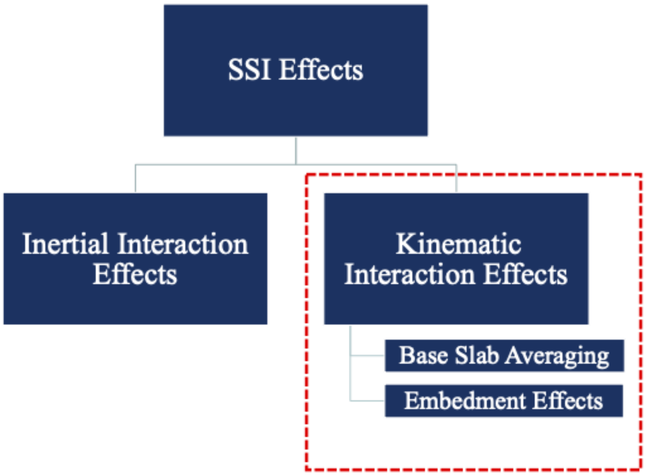
- Structure
- Foundation
- Underlying soil



Soil-structure interaction analysis →

- The collective response of these systems to a specified ground motion

Soil-Structure Interaction (SSI)


SSI Effects

Inertial Interaction Effects

Kinematic Interaction Effects

- Base Slab Averaging
- Embedment Effects

Soil-Structure Interaction (SSI) Terminologies

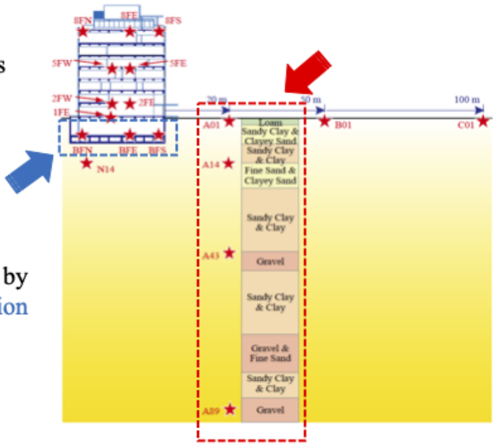
☐ Free-Field Motion (FFM):

A recorded motion by accelerometers at the ground surface

- Not too far from structure
- Not too close to the structure

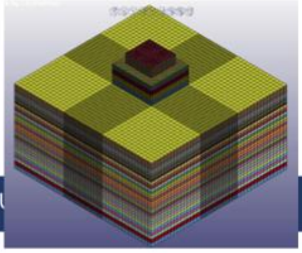
☐ Foundation Motion (FM):

A recorded motion by accelerometers at the foundation level of the structure



Soil-Structure Interaction (SSI) Effects

- Inertial interaction effects tend to produce narrow-banded ground motion modification near the fundamental frequency of the soil-structure system
- Kinematic effects are relatively broad-banded and concentrated at high frequencies
- Kinematic interaction effects can be predicted using (1) relatively costly finite element analyses or (2) simplified models
- Simplified models are semi-empirical in nature => basis for seismic design guidelines such as ASCE 41



KKNPP
Unit 7

Review of ASCE 41-17

- The simplified procedure in ASCE 41-17 to predict foundation-level motions is based on free-field motions multiplied by transfer functions (in terms of RRS) [ATC-83 project by NIST]
- Based on regular building datasets: relatively small footprints (< 196 ft) & non-embedded or shallowly embedded foundations
- Based on 41 pairs of data recorded at 29 sites

$$\mathit{Transfer\ Function}\ (TF) = \frac{\mathit{Motion\ at\ Foundation\ Level}}{\mathit{Motion\ at\ Free\ Field\ (Same\ direction\ with\ Fndn\ motion)}}$$

Transfer Functions Based on:

1) Acceleration Response Spectra - Ratio of Response Spectra (RRS)
2) Fourier Amplitude Spectra

Kinematic Effects in Nuclear Facilities

- Nuclear facilities ⟶ {
  1) Much larger footprints
  2) Higher fundamental frequencies
  3) Deeper embedment depths
- Design guidelines and standards that can benefit:
  - ASCE 4
  - DOE-STD-1027-2018
  - DOE-STD-1020-2016

DOE STANDARD
Natural Phenomena Hazards Analysis and Design Criteria for DOE Facilities

U.S. Department of Energy
Washington, D.C. 20585

DOE STANDARD
HAZARD CATEGORIZATION OF DOE NUCLEAR FACILITIES

U.S. Department of Energy
Washington, DC 20585

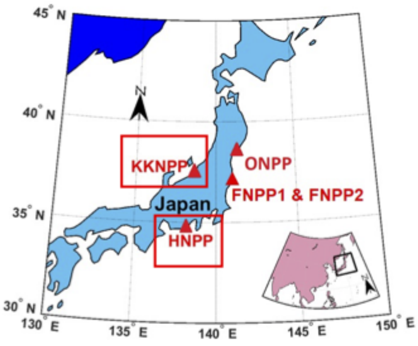
4-16

Seismic Analysis of Safety-Related Nuclear Structures

ASCE

Instrumented NPP Sites Used in This Study

- Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP)
- Hamaoka Nuclear Power (HNPP)
- 114 pairs of FFM & FM data already analyzed
- Three additional sites have been processed and added to our dataset



Kashiwazaki-Kariwa Nuclear Power Plant (KKNPP) Site Map

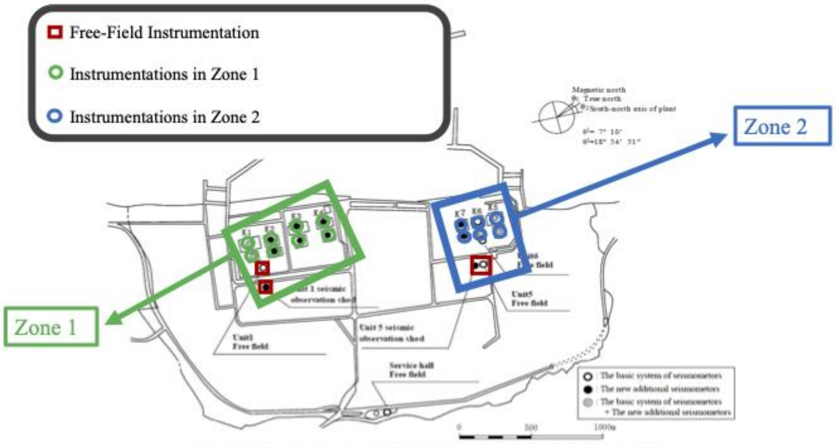


Fig. 1: Location map of seismic observation points in the Kashiwazaki-Kariwa Nuclear Power Station

KKNPP Unit 1 within Zone 1

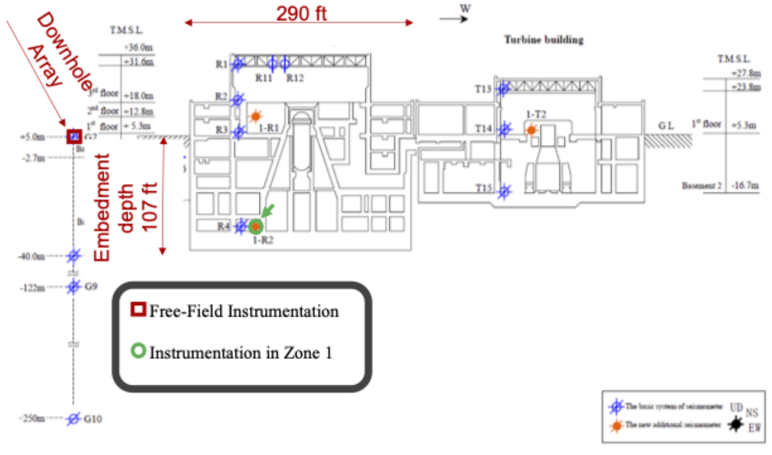
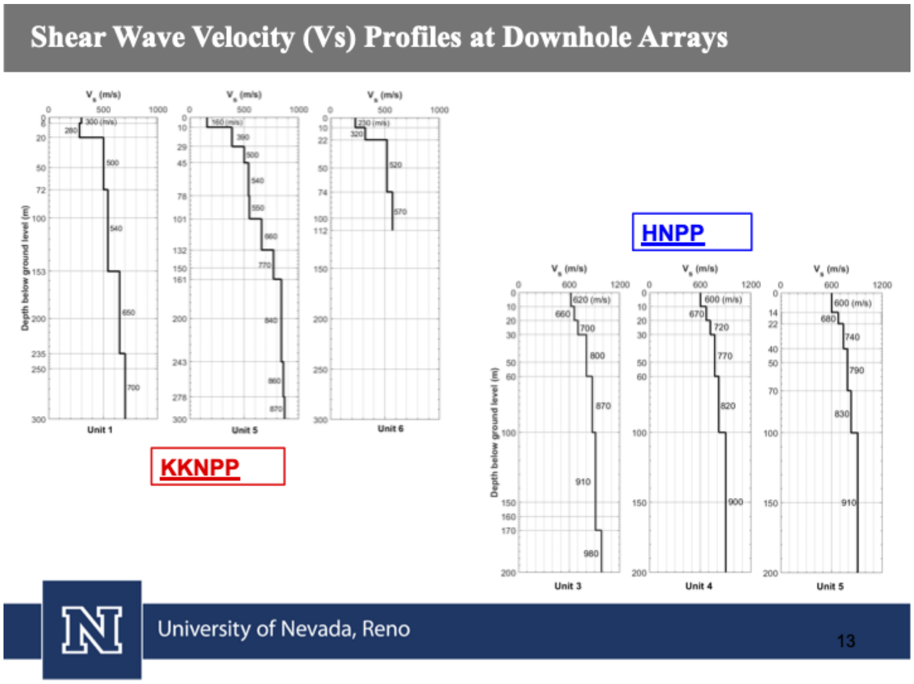


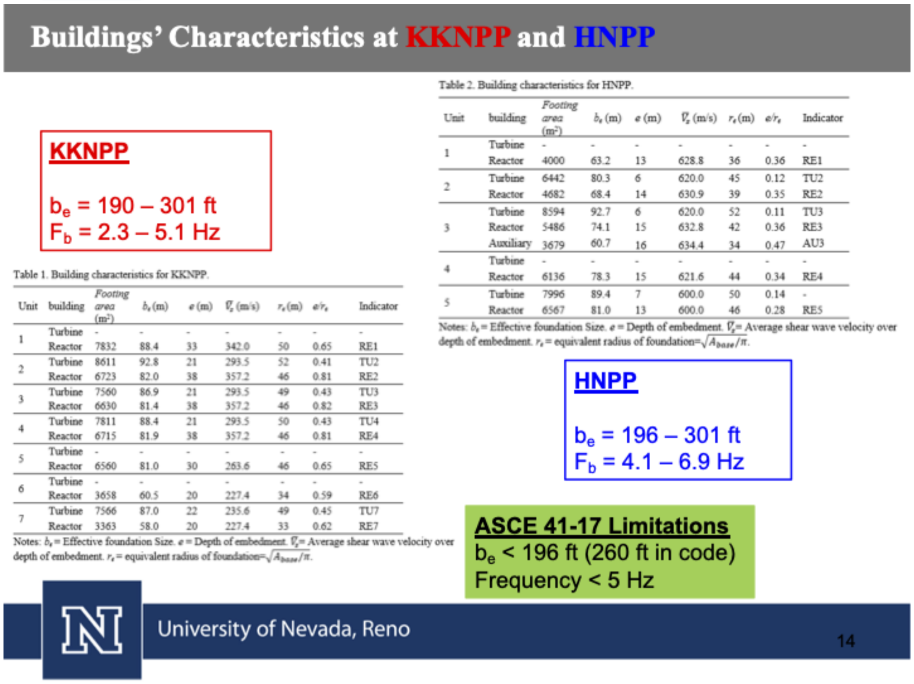
Fig. 3 Cross-sectional view showing the Location map of seismometers in Unit 1



Shear Wave Velocity (Vs) Profiles at Downhole Arrays

KKNPP

HNPP



Buildings' Characteristics at KKNPP and HNPP

KKNPP

b_e = 190 – 301 ft
F_b = 2.3 – 5.1 Hz

Table 1. Building characteristics for KKNPP.

Unit	building	*Footing area* (m²)	b_e (m)	e (m)	$\bar{V}_s$ (m/s)	r_e (m)	e/r_e	Indicator
1	Turbine	-	-	-	-	-	-	-
	Reactor	7832	88.4	33	342.0	50	0.65	RE1
2	Turbine	8611	92.8	21	293.5	52	0.41	TU2
	Reactor	6723	82.0	38	357.2	46	0.81	RE2
3	Turbine	7560	86.9	21	293.5	49	0.43	TU3
	Reactor	6630	81.4	38	357.2	46	0.82	RE3
4	Turbine	7811	88.4	21	293.5	50	0.43	TU4
	Reactor	6715	81.9	38	357.2	46	0.81	RE4
5	Turbine	-	-	-	-	-	-	-
	Reactor	6560	81.0	30	263.6	46	0.65	RE5
6	Turbine	-	-	-	-	-	-	-
	Reactor	3658	60.5	20	227.4	34	0.59	RE6
7	Turbine	7566	87.0	22	235.6	49	0.45	TU7
	Reactor	3363	58.0	20	227.4	33	0.62	RE7

Notes: b_e = Effective foundation Size. e = Depth of embedment. $\bar{V}_s$= Average shear wave velocity over depth of embedment. r_e = equivalent radius of foundation=$\sqrt{A_{base}/\pi}$.

Table 2. Building characteristics for HNPP.

Unit	building	*Footing area* (m²)	b_e (m)	e (m)	$\bar{V}_s$ (m/s)	r_e (m)	e/r_e	Indicator
1	Turbine	-	-	-	-	-	-	-
	Reactor	4000	63.2	13	628.8	36	0.36	RE1
2	Turbine	6442	80.3	6	620.0	45	0.12	TU2
	Reactor	4682	68.4	14	630.9	39	0.35	RE2
3	Turbine	8594	92.7	6	620.0	52	0.11	TU3
	Reactor	5486	74.1	15	632.8	42	0.36	RE3
	Auxiliary	3679	60.7	16	634.4	34	0.47	AU3
4	Turbine	-	-	-	-	-	-	-
	Reactor	6136	78.3	15	621.6	44	0.34	RE4
5	Turbine	7996	89.4	7	600.0	50	0.14	-
	Reactor	6567	81.0	13	600.0	46	0.28	RE5

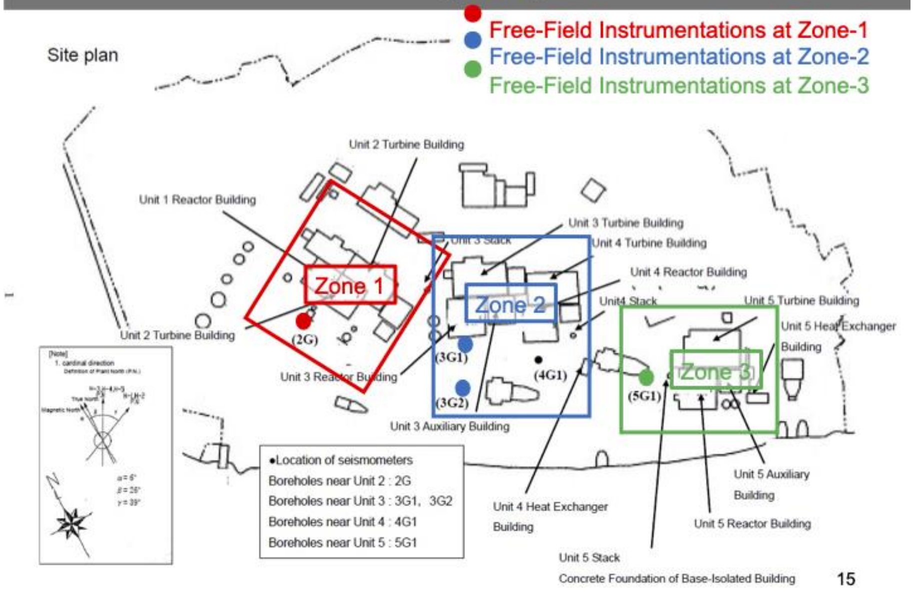
Notes: b_e = Effective foundation Size. e = Depth of embedment. $\bar{V}_s$= Average shear wave velocity over depth of embedment. r_e = equivalent radius of foundation=$\sqrt{A_{base}/\pi}$.

HNPP

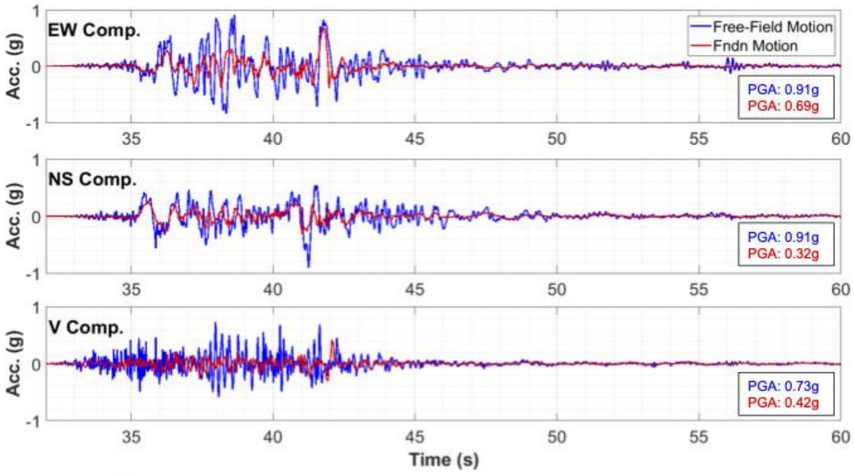
b_e = 196 – 301 ft
F_b = 4.1 – 6.9 Hz

ASCE 41-17 Limitations
b_e < 196 ft (260 ft in code)
Frequency < 5 Hz

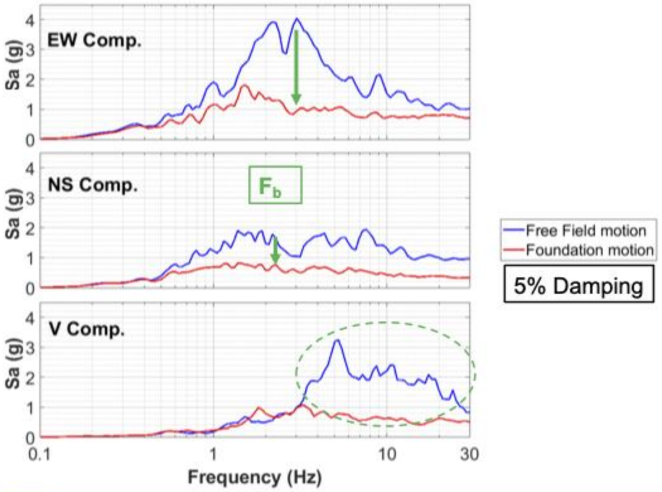
Hamaoka Nuclear Power Plant (HNPP)



Example of Recorded Time Histories of FFM vs FM at KKNPP



Acceleration Response Spectra



ASCE 41-17 Procedure

❑ Base-Slab Averaging Effect

$$\boxed{RRS_{bsa} = 0.25 + 0.75 \times \left\{ \frac{1}{b_0^2} \left[1 - \exp\left(-2b_0^2\right) \times B_{bsa} \right] \right\}^{1/2}} \quad (8\text{-}15)$$

where

$$B_{bsa} = \begin{cases} 1 + b_0^2 + b_0^4 + \dfrac{b_0^6}{2} + \dfrac{b_0^8}{4} + \dfrac{b_0^{10}}{12} & b_0 \le 1 \\ \exp\left(2b_0^2\right) \left[\dfrac{1}{\sqrt{\pi} b_0} \left(1 - \dfrac{1}{16 b_0^2} \right) \right] & b_0 > 1 \end{cases} \quad (8\text{-}16)$$

$$b_0 = 0.0001 \times \left(\frac{2\pi b_e}{T} \right) \quad (8\text{-}17)$$

b_e = effective foundation size in feet;

$$b_e = \sqrt{A_{base}} \le 260\text{ft.}; \quad (8\text{-}18)$$

T = fundamental period of the building in seconds, computed based on a model with a flexible base condition per 8.4.2, which shall not be taken as less than 0.20 s when used in Eq. (8-17); and

A_{base} = area of the foundation footprint if the foundation components are interconnected laterally (ft^2).

❑ Embedment Depth Effect

$$\boxed{RRS_e = 0.25 + 0.75 \times \cos\left(\frac{2\pi e}{T v_s} \right) \ge 0.50} \quad (8\text{-}19)$$

where e = foundation embedment depth in feet. A minimum of 75% of the foundation footprint shall be present at the embedment depth. The foundation embedment for buildings located on sloping sites shall be the shallowest embedment;

T = fundamental period of the building in seconds, computed based on a model with a flexible base, which shall not be taken as less than 0.20 s when used in Eq. (8-19);

v_s = effective shear wave velocity for site soil conditions, taken as average value of velocity over the embedment depth of the foundation (ft/s), or approximated as nv_{s0}

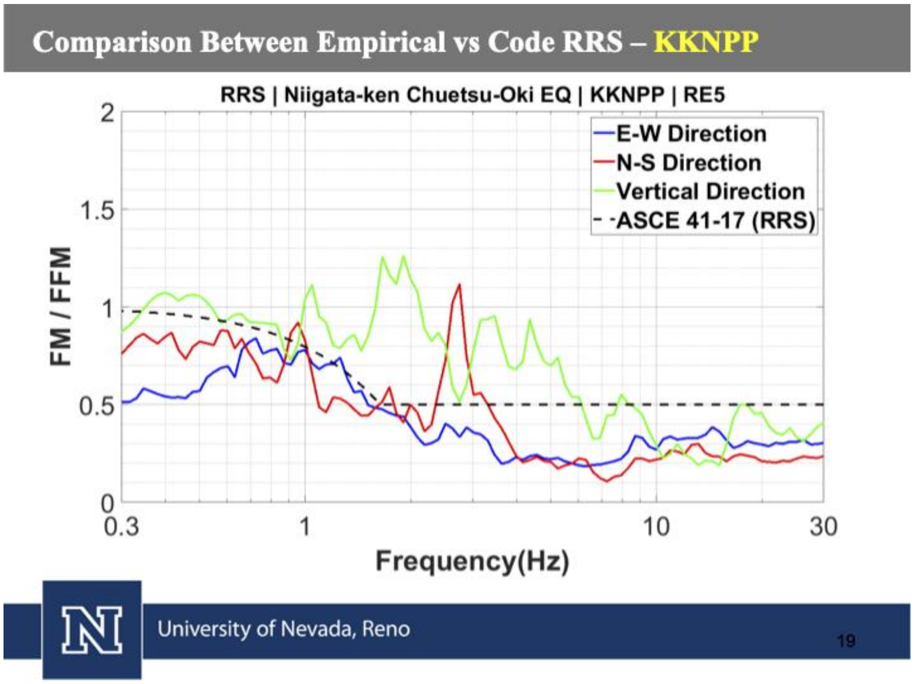
v_{s0} = shear wave velocity for site soil conditions at low strains, taken as average value of velocity over the embedment depth of the foundation (ft/s);

n = shear wave velocity reduction factor;

$n = \sqrt{G/G_o}$; and

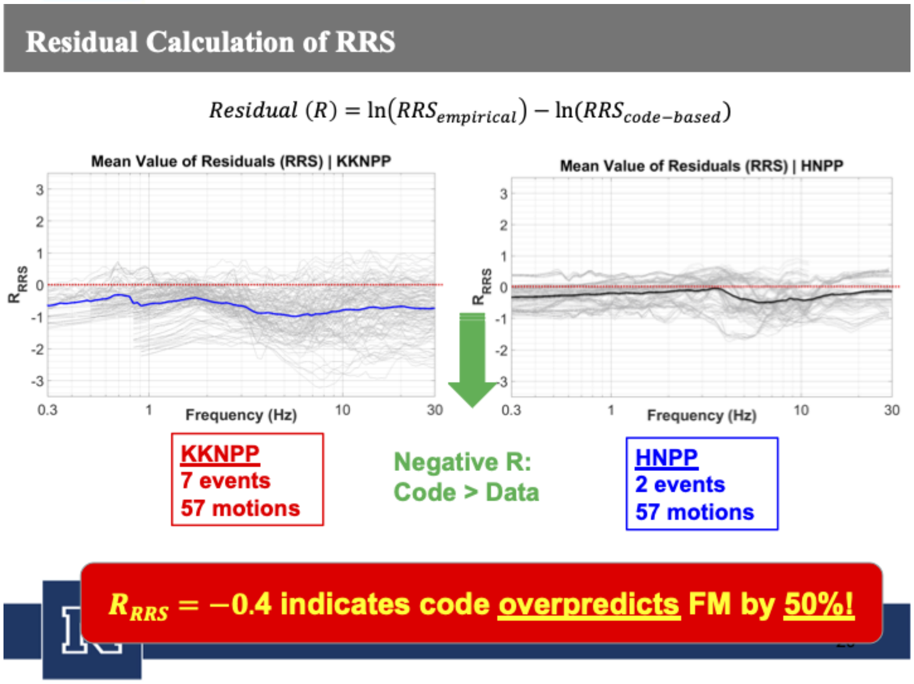
G/G_o = effective shear modulus ratio from Table 8-2.

$$\boldsymbol{RRS = RRS_{bsa} \times RRS_e \ge 0.5}$$



Comparison Between Empirical vs Code RRS – KKNPP

University of Nevada, Reno



Residual Calculation of RRS

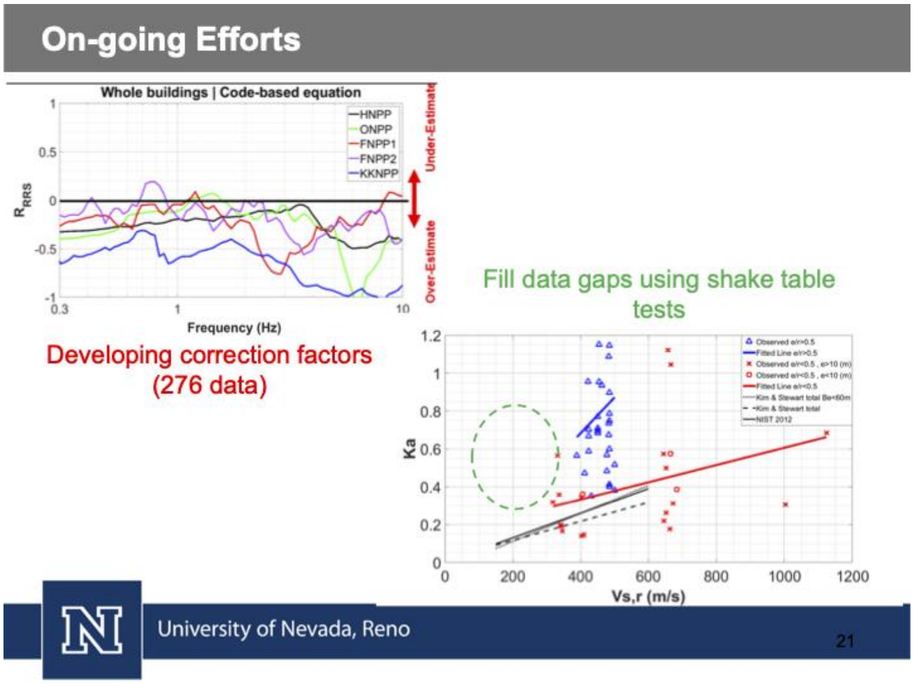
$$Residual\ (R) = \ln(RRS_{empirical}) - \ln(RRS_{code-based})$$

KKNPP
7 events
57 motions

Negative R:
Code > Data

HNPP
2 events
57 motions

$R_{RRS} = -0.4$ indicates code overpredicts FM by 50%!

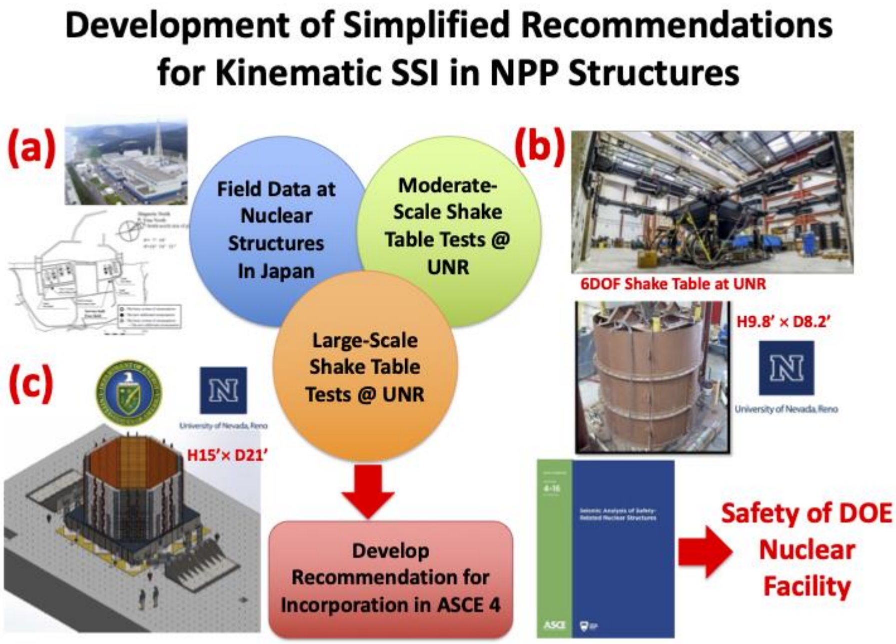


On-going Efforts

Developing correction factors (276 data)

Fill data gaps using shake table tests

University of Nevada, Reno



Development of Simplified Recommendations for Kinematic SSI in NPP Structures

Field Data at Nuclear Structures In Japan

Moderate-Scale Shake Table Tests @ UNR

Large-Scale Shake Table Tests @ UNR

Develop Recommendation for Incorporation in ASCE 4

6DOF Shake Table at UNR

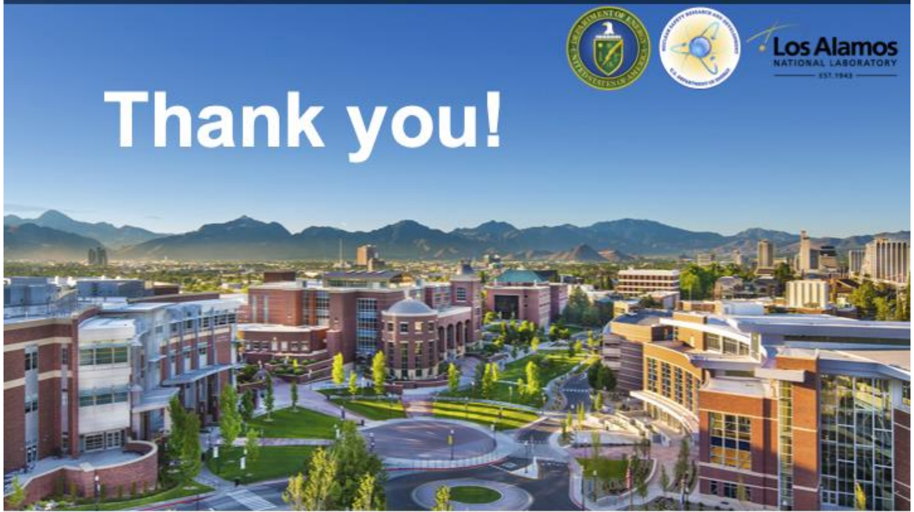
H9.8′ × D8.2′

H15′× D21′

Safety of DOE Nuclear Facility

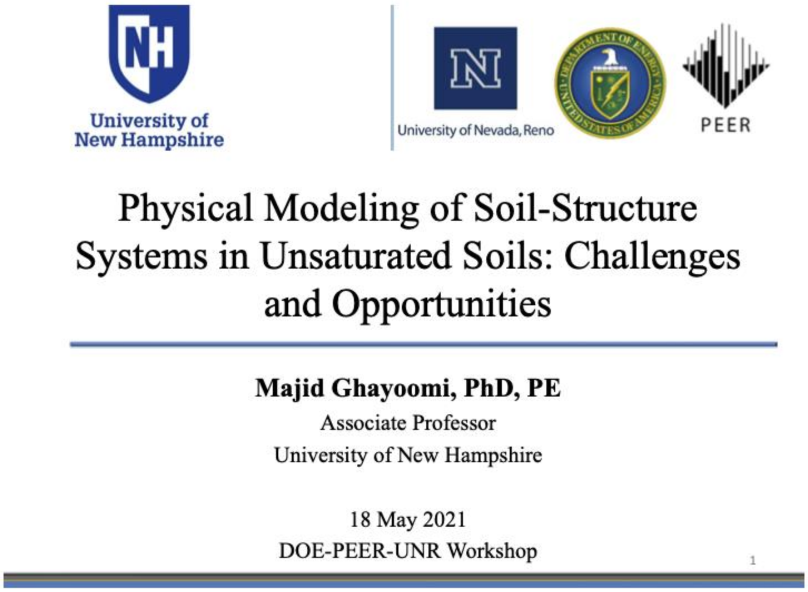
Concluding Remarks

- Code-Based methods for considering kinematic SSI effects include simplified equations for approximation of transfer functions and are limited to horizontal component
- Horizontal empirical transfer functions are generally consistent with code-based ones in terms of shape, but adjustments are needed for NPP facilities
- Current code-based equations were calibrated for regular buildings in terms of foundation size and embedment depth => NOT applicable to NPP structures
- Code-based RRS given by ASCE 41 consistently overpredicted the FM over the frequency range => need for a simplified procedure applicable to NPP structures
- Data gaps can be filled with shake table experimental results



Thank you!

PHYSICAL MODELING OF SOIL-STRUCTURE SYSTEMS IN UNSATURATED SOILS: CHALLENGES AND OPPORTUNITIES



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University of Nevada, Reno

PEER

Physical Modeling of Soil-Structure Systems in Unsaturated Soils: Challenges and Opportunities

Majid Ghayoomi, PhD, PE
Associate Professor
University of New Hampshire

18 May 2021
DOE-PEER-UNR Workshop

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- ☐Sahar Ghadirianniari*
- ☐Matthew Turner*

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- ☐Dr. Kyohei Ueda, KU
- ☐Dr. Ali Khosravi, OSU
- ☐Dr. John McCartney, UCSD

* Past or Current students

University of New Hampshire

NSF

DPRI-KU

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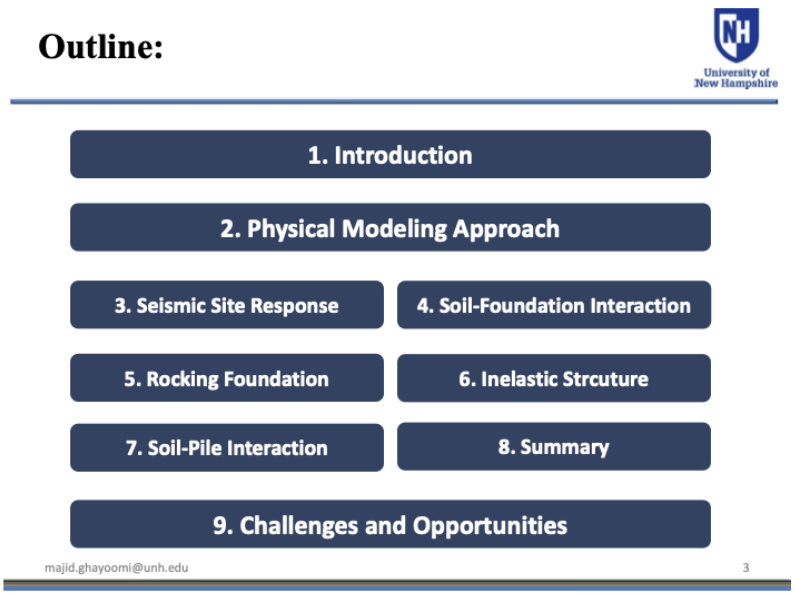

Outline:

1. Introduction

2. Physical Modeling Approach

3. Seismic Site Response

4. Soil-Foundation Interaction

5. Rocking Foundation

6. Inelastic Strcuture

7. Soil-Pile Interaction

8. Summary

9. Challenges and Opportunities

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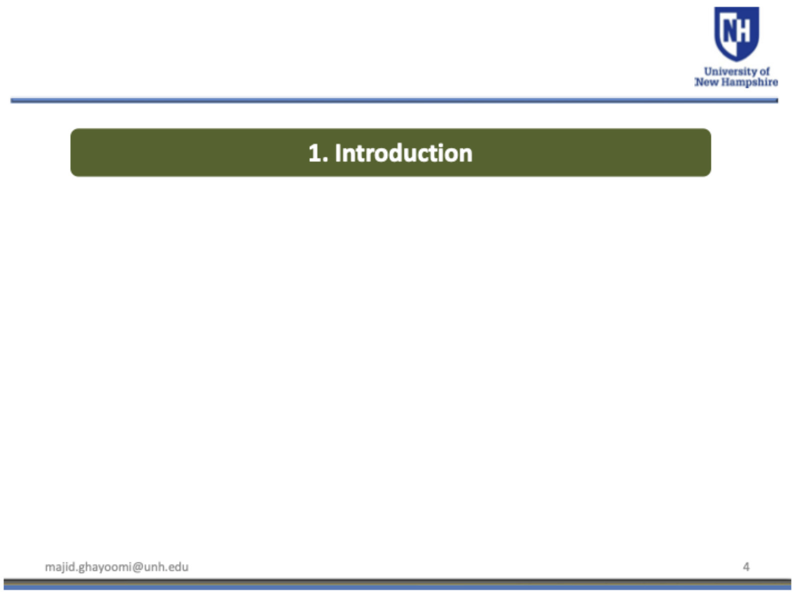

1. Introduction

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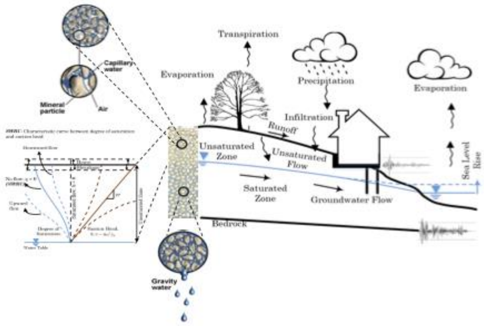
Seismic Design and Unsaturated Soils:

❑Many geotechnical systems involving unsaturated soils must be designed for seismic loads.



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Seismic Design and Unsaturated Soils:

❑Many geotechnical systems involving unsaturated soils must be designed for seismic loads.

❑**Should the degree of saturation be considered in seismic design of such systems**?

- ➢ Current practice is based on dry or saturated soil systems.
- ➢ Current procedures do not account for moisture variability.

- o Is it conservative?
- o Is it safe?
- o Is it cost-effective?
- o Is it mechanistically accurate?

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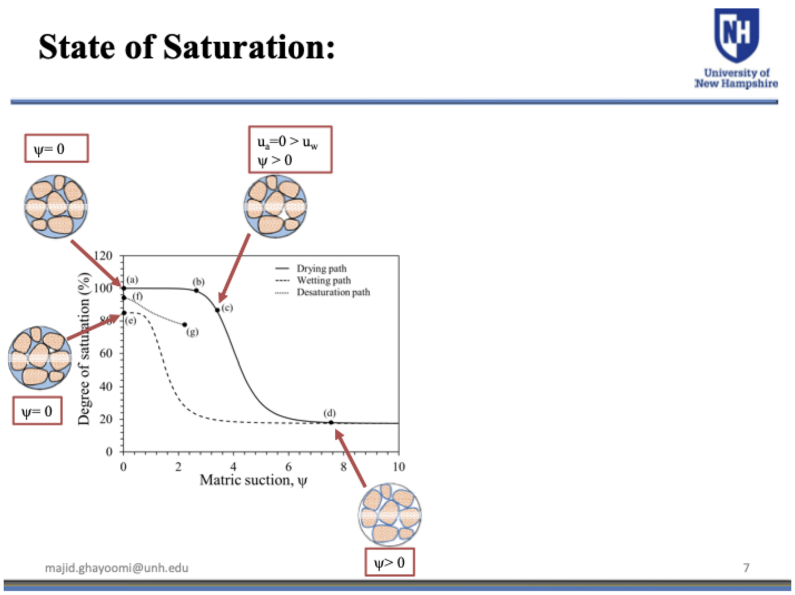


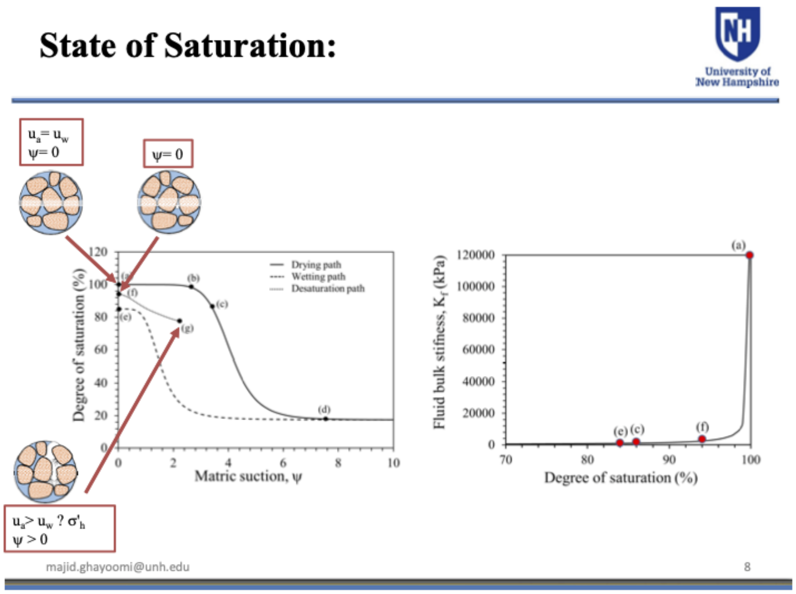
State of Saturation:

$\psi = 0$

$u_a = 0 > u_w$
$\psi > 0$

$\psi = 0$

$\psi > 0$



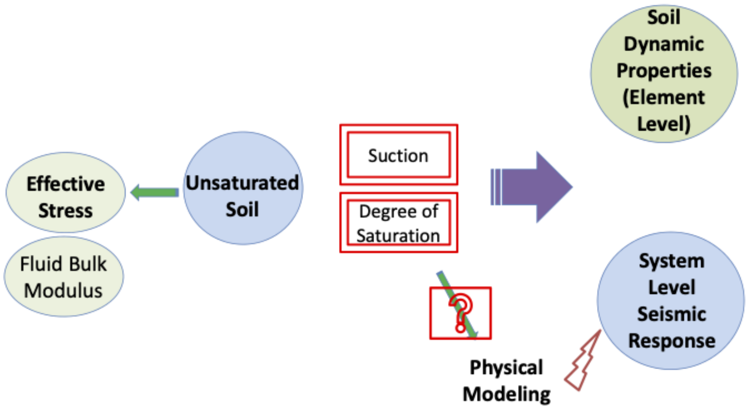
State of Saturation:

$u_a = u_w$
$\psi = 0$

$\psi = 0$

$u_a > u_w$? σ'_h
$\psi > 0$

Research Approach:



Soil Dynamic Properties (Element Level)

Effective Stress

Fluid Bulk Modulus

Unsaturated Soil

Suction

Degree of Saturation

System Level Seismic Response

Physical Modeling

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1. Introduction

2. Physical Modeling Approach

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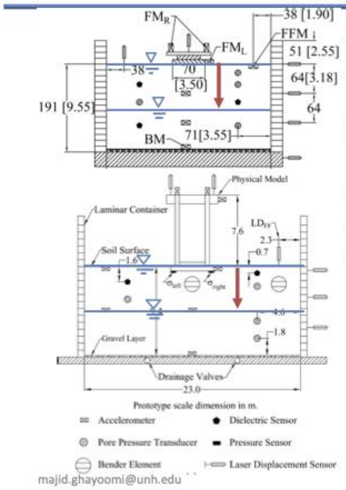
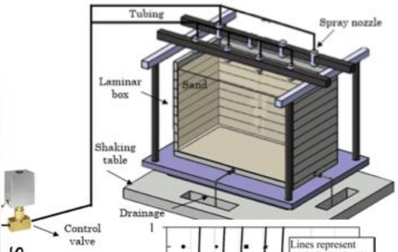
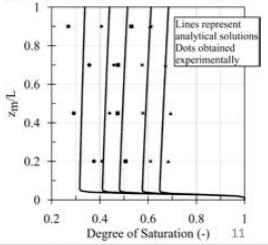
Centrifuge Modeling – Control Saturation:



Setting Ground Water Level

Steady State Infiltration





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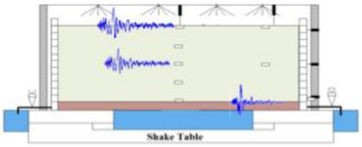
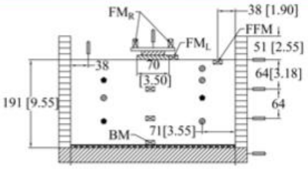
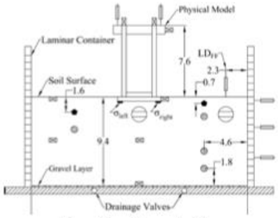
Testing Systems:



Site Response



Soil-Foundation Interaction



Rocking Foundation

Inelastic Structure

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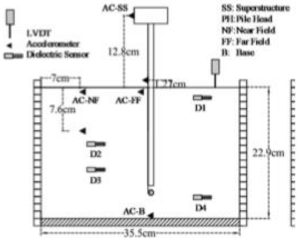
Testing Systems:



Pile-Soil Interaction

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1. Introduction

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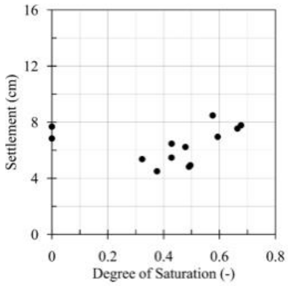
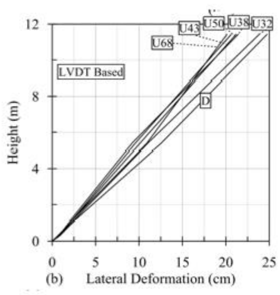
Site Response:

Free Field Settlement



Lateral Deformation Profile



Results of Steady State Infiltration – Constant Degree of Saturation
Mirshekari and Ghayoomi (2017) – SDEE

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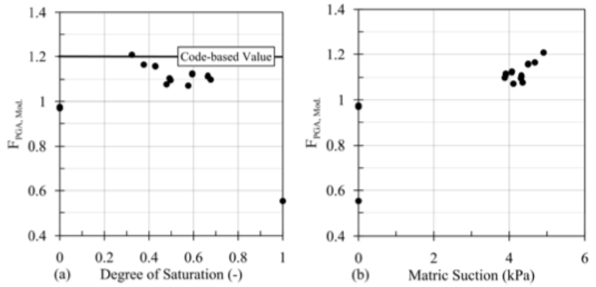
Site Response:

PGA Amplification



$$\boxed{F_{PGA} = PGA/PGA_{Base}}$$

Results of Steady State Infiltration – Constant Degree of Saturation
Mirshekari and Ghayoomi (2017) – SDEE

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Outline:

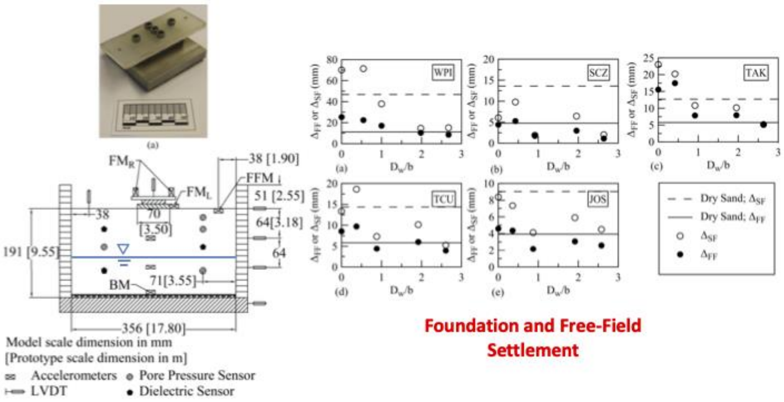
1. Introduction

2. Physical Modeling Approach

3. Seismic Site Response

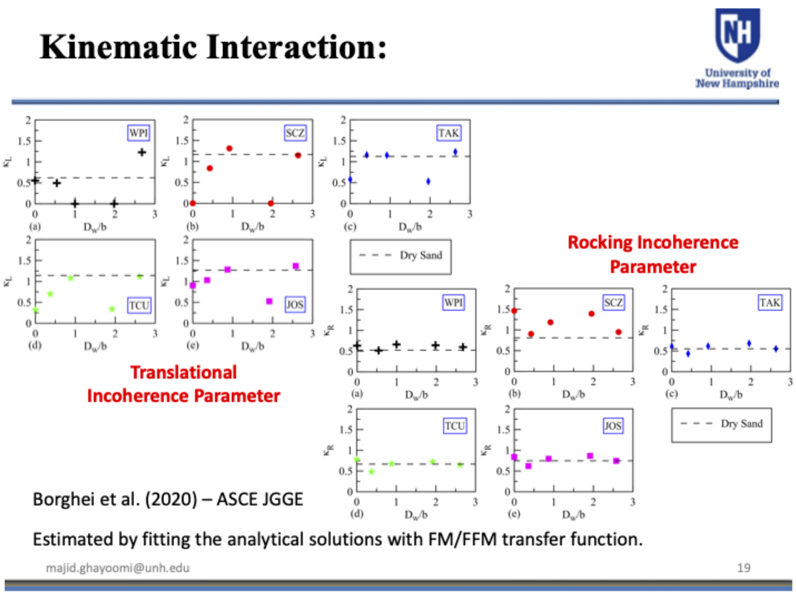
4. Soil-Foundation Interaction

Soil-Foundation Interaction:



Foundation and Free-Field Settlement

Borghei et al. (2020) – ASCE JGGE



Kinematic Interaction:

University of New Hampshire

Translational Incoherence Parameter

Rocking Incoherence Parameter

Borghei et al. (2020) – ASCE JGGE

Estimated by fitting the analytical solutions with FM/FFM transfer function.

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University of New Hampshire

1. Introduction

2. Physical Modeling Approach

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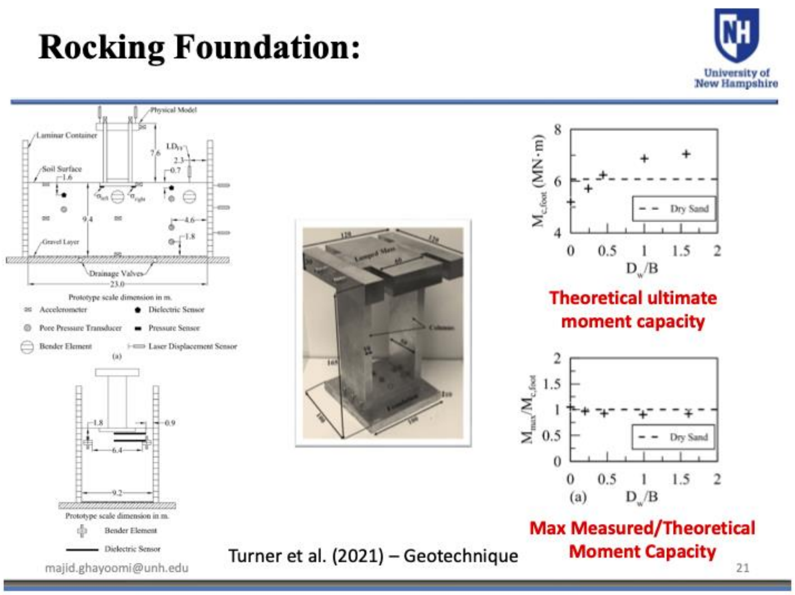

Rocking Foundation:

Theoretical ultimate moment capacity

Max Measured/Theoretical Moment Capacity

Turner et al. (2021) – Geotechnique

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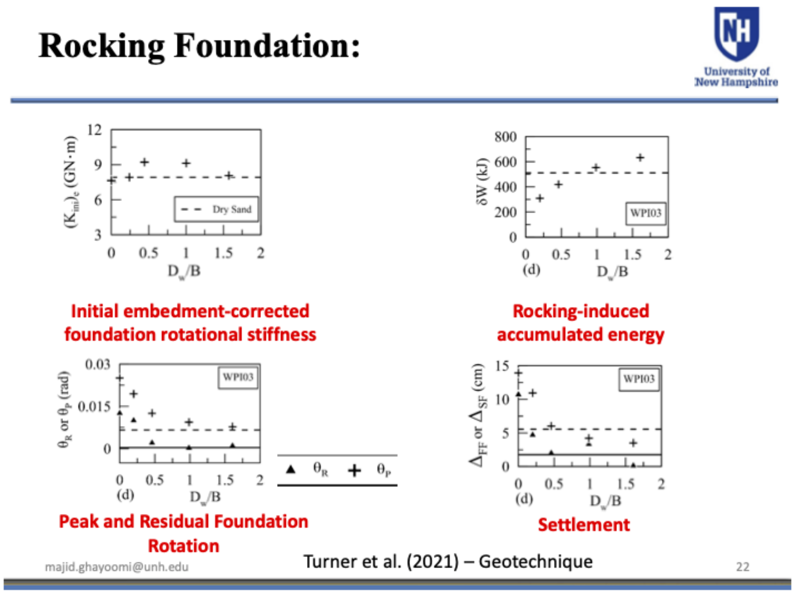

Rocking Foundation:

Initial embedment-corrected foundation rotational stiffness

Rocking-induced accumulated energy

Peak and Residual Foundation Rotation

Settlement

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Turner et al. (2021) – Geotechnique

Outline:

1. Introduction

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4. Soil-Foundation Interaction

5. Rocking Foundation

6. Inelastic Structure

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1. Introduction

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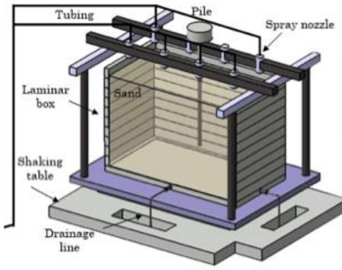
4. Soil-Foundation Interaction

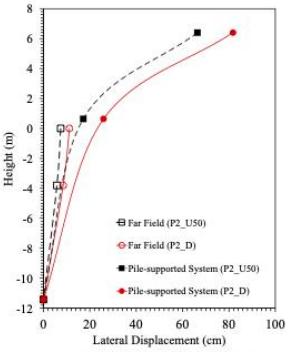
5. Rocking Foundation

6. Inelastic Structure

7. Soil-Pile Interaction

Soil-Pile Interaction:



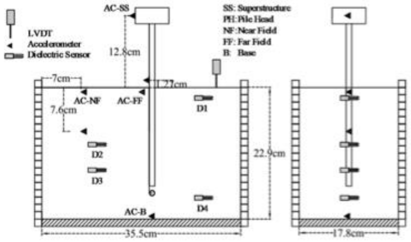


Lateral Deformations

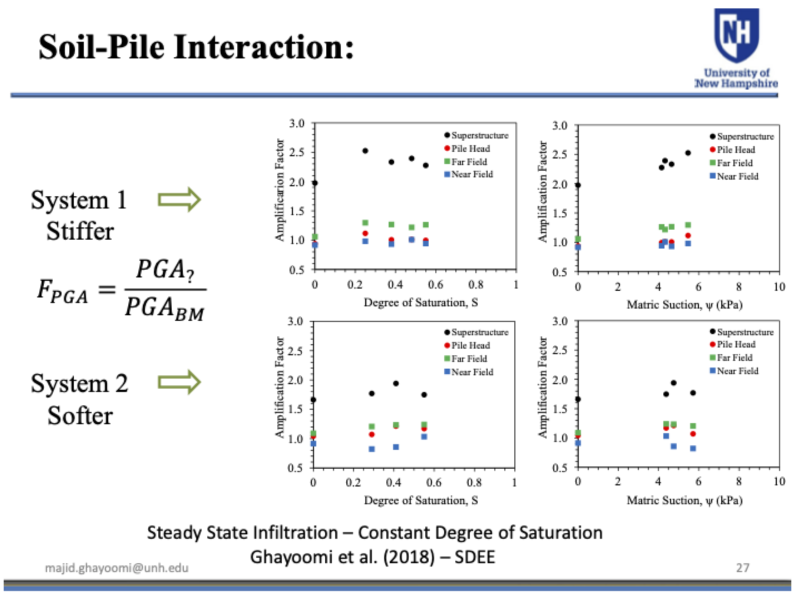
Steady State Infiltration – Constant Degree of Saturation
Ghayoomi et al. (2018) – SDEE

Soil-Pile Interaction:

$$F_{PGA} = \frac{PGA_?}{PGA_{BM}}$$



Steady State Infiltration – Constant Degree of Saturation
Ghayoomi et al. (2018) – SDEE



Soil-Pile Interaction:

University of New Hampshire

System 1
Stiffer

$$F_{PGA} = \frac{PGA_?}{PGA_{BM}}$$

System 2
Softer

Steady State Infiltration – Constant Degree of Saturation
Ghayoomi et al. (2018) – SDEE

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University of New Hampshire

1. Introduction

2. Physical Modeling Approach

3. Seismic Site Response

4. Soil-Foundation Interaction

5. Rocking Foundation

6. Inelastic Strcuture

7. Soil-Pile Interaction

8. Summary

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Summary:

☐ There are proven effects of degree of saturation on dynamic soil properties and consequently on the response of geotechnical systems.

☐ The overall system response is influenced by combined effects of damping, modulus, density, and wave propagation mechanisms.

☐ Unsaturated soil could lead to higher motion amplification and impose larger seismic demands on surface structures.

☐ Stiffer unsaturated soils lead to lower settlements/deformations, with minimum at middle range degree of saturation.

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1. Introduction

2. Physical Modeling Approach

3. Seismic Site Response

4. Soil-Foundation Interaction

5. Rocking Foundation

6. Inelastic Strcuture

7. Soil-Pile Interaction

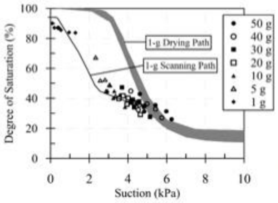
8. Summary

9. Challenges and Opportunities

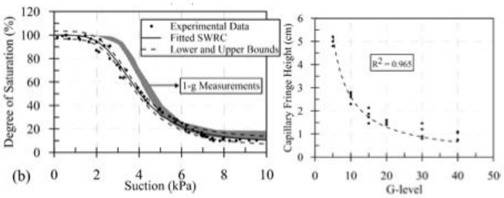
Challenges:

☐Controlling the Degree of Saturation in Physical Models

Infiltration



Capillary Rise



Compaction

Mirshekari et et al. (2018) – ASTM GTJ

Challenges:

☐Other Challenges:

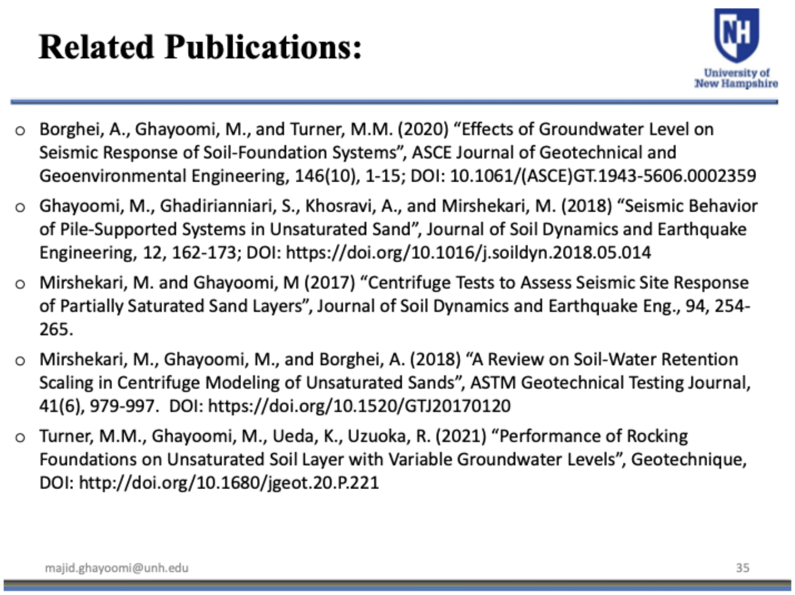
- Compacted soils specially ones with higher permeability
- Modeling larger/wider structures using infiltration method
- Limited space to observe variation in water content
- Difficulties in modeling multi-layered soils
- Limited options on tensiometers
- Challenges in implementing liquefaction mitigation techniques using IPS

Opportunities:

- ❑ SFSI on compacted unsaturated soils to study
  - Seismic Demand
  - Seismic Compression
- ❑ Modeling layered soils with different water retention regimes
- ❑ Versatile monitoring of water content and suction
- ❑ Better distribution of moisture in depth

Thank you!

Durham, NH



Related Publications:

University of New Hampshire

- Borghei, A., Ghayoomi, M., and Turner, M.M. (2020) “Effects of Groundwater Level on Seismic Response of Soil-Foundation Systems”, ASCE Journal of Geotechnical and Geoenvironmental Engineering, 146(10), 1-15; DOI: 10.1061/(ASCE)GT.1943-5606.0002359
- Ghayoomi, M., Ghadirianniari, S., Khosravi, A., and Mirshekari, M. (2018) “Seismic Behavior of Pile-Supported Systems in Unsaturated Sand”, Journal of Soil Dynamics and Earthquake Engineering, 12, 162-173; DOI: https://doi.org/10.1016/j.soildyn.2018.05.014
- Mirshekari, M. and Ghayoomi, M (2017) “Centrifuge Tests to Assess Seismic Site Response of Partially Saturated Sand Layers”, Journal of Soil Dynamics and Earthquake Eng., 94, 254-265.
- Mirshekari, M., Ghayoomi, M., and Borghei, A. (2018) “A Review on Soil-Water Retention Scaling in Centrifuge Modeling of Unsaturated Sands”, ASTM Geotechnical Testing Journal, 41(6), 979-997. DOI: https://doi.org/10.1520/GTJ20170120
- Turner, M.M., Ghayoomi, M., Ueda, K., Uzuoka, R. (2021) “Performance of Rocking Foundations on Unsaturated Soil Layer with Variable Groundwater Levels”, Geotechnique, DOI: http://doi.org/10.1680/jgeot.20.P.221

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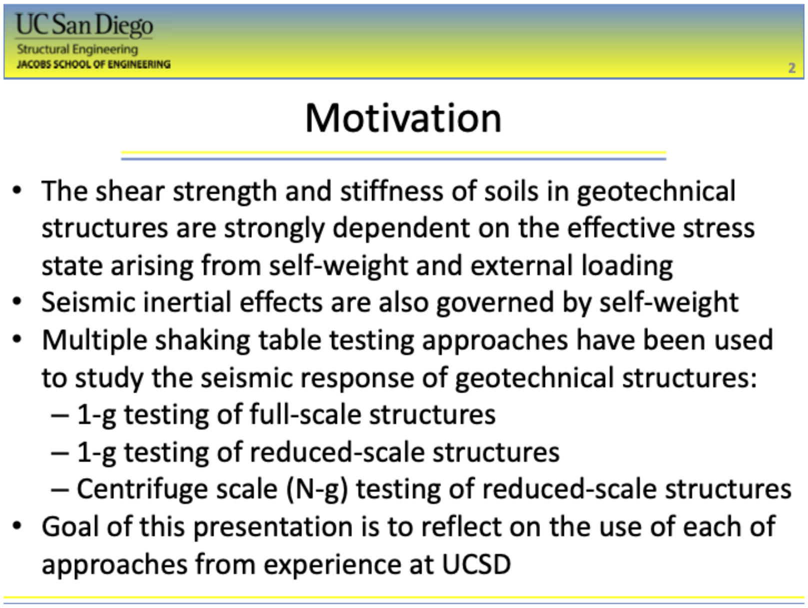
RESULTS FROM SHAKING TABLE TESTS ON FULL-SCALE RAIL EMBANKMENTS

UC San Diego
Structural Engineering
JACOBS SCHOOL OF ENGINEERING

Shake Table Tests on Geotechnical Structures at Multiple Scales

John S. McCartney, Ph.D., P.E., F.ASCE
Professor and Department Chair
Department of Structural Engineering
University of California San Diego
UNR-DOE-PEER Workshop
May 17-18, 2021



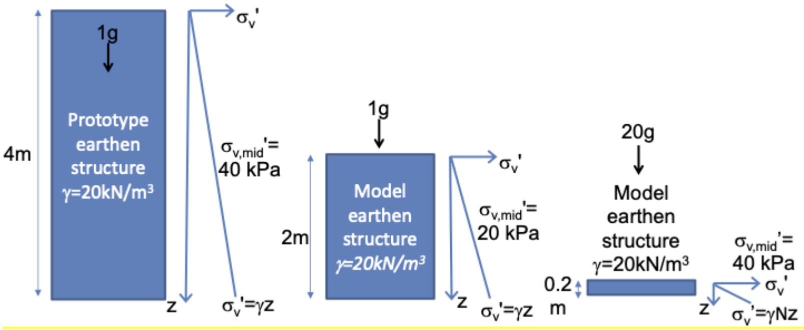
UC San Diego
Structural Engineering
JACOBS SCHOOL OF ENGINEERING

Motivation

- The shear strength and stiffness of soils in geotechnical structures are strongly dependent on the effective stress state arising from self-weight and external loading
- Seismic inertial effects are also governed by self-weight
- Multiple shaking table testing approaches have been used to study the seismic response of geotechnical structures:
  - 1-g testing of full-scale structures
  - 1-g testing of reduced-scale structures
  - Centrifuge scale (N-g) testing of reduced-scale structures
- Goal of this presentation is to reflect on the use of each of approaches from experience at UCSD

Need for Scaling in Reduced Scale Testing

- When testing a model at 1-g with a geometry that is N times smaller than a prototype, the self-weight is still proportional to the height of the soil layer and is N times smaller than the prototype
- When testing a model at N-g with a geometry that is N times smaller than a prototype, the effective stress at mid-height will be the same as the prototype structure (assuming model height is small compared to the arm length)



Why Perform 1-g Shake Table Testing?

- Shake table testing has been used successfully to investigate the seismic performance of various geotechnical structures:
  - Earthen embankments (Wartman et al. 2005)
  - MSE walls/slopes with no surcharge (El-Emam and Bathurst 2004, 2005, 2007; Ling et al. 2005, 2012; Tatsuoka et al. 2009, 2012)
  - MSE bridge abutments (Helwany et al. 2012; Zheng et al. 2019a, 2019b)
  - Foundations (shallow, deep, helical anchors, etc.)
- Can use actual (or similar) materials used in the field (backfill soil, geosynthetic reinforcements, facing blocks, reinforced concrete)
- Can use similar construction techniques/details to the field
- No scaling conflicts for time (dynamic/diffusion)
- Dense instrumentation array can be used without altering overall system response

Why Perform N-g Shake Table Testing?

- Centrifuge modeling with shake table testing of geotechnical structures is well-established with validated scaling relationships
- Idealize complex geometry to focus on key mechanisms
- Smaller models with lower cost may permit multiple tests
  - Ease of preparation and instrumentation placement
  - Soil preparation may not be same as the field, but can be more controlled
  - Can dissect specimens after testing
- Dense instrumentation arrays, but requires miniature sensors
- Use containers to reduce boundary effects (laminar or FSB container), help visualize deformations (acrylic facing), help saturate soil layers for liquefaction testing
- Time scaling conflict for dynamic/diffusion problems
- Can use actuators to load systems to failure

Reduced-Scale 1g Shake Table Testing?

- Motivations:
  - Full-scale structures may be too heavy to test on a shake table
  - When testing foundations, the dimensions necessary to reduce boundary effects may be too large for the extents of a shake table
- Scaling relationships must be used so that the stress-strain curve of the model soil leads to a similar deformation response to the prototype
  - Most common approach is to use a lower relative density for the soil to reduce the shear strength and stiffness
  - Small dynamic/diffusion time scaling conflict may be present
- Reduced scale testing may require less effort while still using the same construction methods as fule-scale structures
  - May permit more test configurations to be evaluated

Presentation Scope

- Overview of shake table facilities at UCSD for geotechnical structures and examples of past projects
- **Project 1**: ½-scale MSE bridge abutment shaking table tests
  - Project sponsor: Caltrans, FHWA Pooled Fund
  - Research team: Yewei Zheng (now Prof. at Wuhan University), Prof. Pat Fox (Dept. Heat at Penn State), Prof. Benson Shing (UCSD)
- **Project 2**: Full-scale rail embankment shaking table tests
  - Project sponsor: BART
  - Research team: Axel Yarahuaman , Chih-Yen Wang, Prof. Ken Loh
- **Project 3**: Centrifuge-scale rocking foundation study
  - Project sponsor: PEER
  - Research team: Jeffrey Newgard, Prof. Tara Hutchinson

Research Foci In Geotechnical Engineering at UCSD

- Geotechnical hazard characterization and mitigation
  - Earthquake response of buried structures
  - Rocking foundations with integrated ground improvement
  - Effects of liquefaction and lateral spreading on pile foundations
  - Post-wildfire mudflows
- Energy geotechnics
  - Energy piles
  - Thermal energy storage
  - Enhanced geothermal systems
- Material characterization
  - Unsaturated soils
  - Hydrophobic soils affected by wildfires
  - Geosynthetics
  - Tire derived aggregates

Ahmed Elgamal, Professor

Tara Hutchinson, Professor

John McCartney, Professor

Ingrid Tomac, Assistant Professor

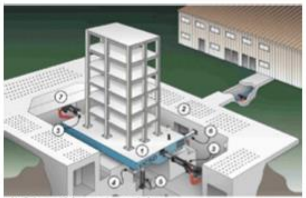
Geotechnical Engineering Research Approach at UCSD

- Constitutive modeling
  - Quantify soil behavior under complex loading or environmental interactions
  - Models for nonlinear, elasto-plastic behavior
  - Cyclic shearing, high stresses, wetting/drying, temperature cycles
- Numerical simulations
  - Continuum simulations (Finite elements, finite difference)
  - Micromechanics: Discrete element modeling (DEM) coupled with computational fluid dynamics (CFD)
- Physical modeling (needed for validation)
  - Full-scale testing
  - Reduced-scale testing
  - Centrifuge-scale testing
- Development of simplified design tools

UCSD Large High Performance Outdoor Shaking Table

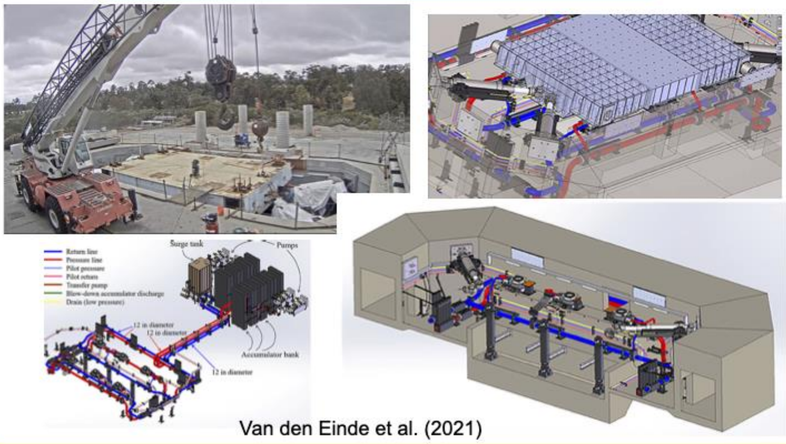
http://nees.ucsd.edu/facilities/shake-table.shtml

Size	7.6 m × 12.2 m
Peak acceleration: bare table, 400 ton payload	4.2 g, 1.2 g
Peak velocity	1.8 m/s
Stroke	±0.75 m
Maximum gravity (vertical) payload	20 MN
Force capacity of actuators	6.8 MN
Maximum overturning moment: bare table, 400 ton specimen	35 MN-m, 50 MN-m
Frequency bandwidth	0 — 33 Hz



1) *7.6 m × 12.2 m moving steel platen*
2) *Reinforced concrete surrounding reaction block*
3) *Hydraulic actuators (2, 6.8 MN combined force capacity)*
4) *Hydrostatic bearings (6, sliding mechanism)*
5) *Vertical hold-down struts (overturning restraint)*
6) *Hydrostatic bearings (2 slaved pairs, yaw restraint)*
7) *Cover plates to protect actuators*

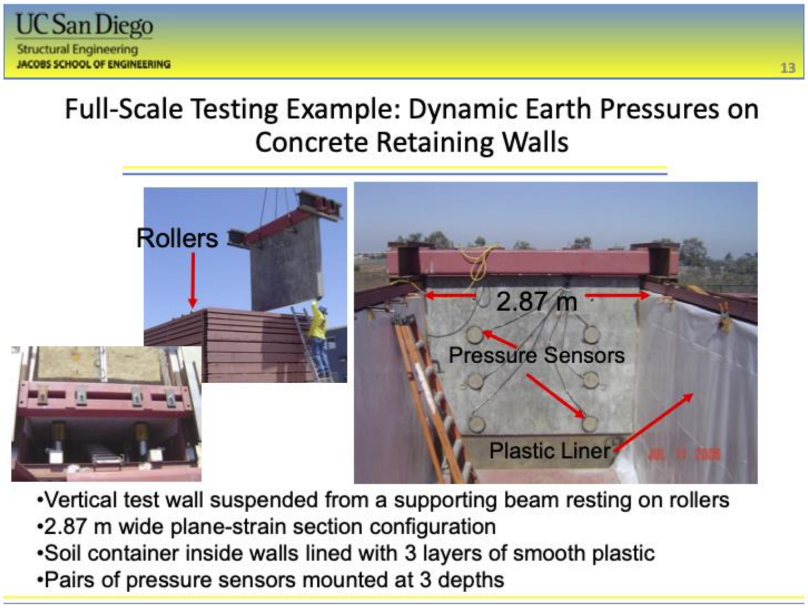
UCSD LHPOST 6 Degree of Freedom Upgrade (October 2021)



Van den Einde et al. (2021)

Large Laminar Container for LHPOST

Length of 6.7 m (22 ft), width of 3 m
(9.6 ft) and height of 4.7 m (15.2 ft)

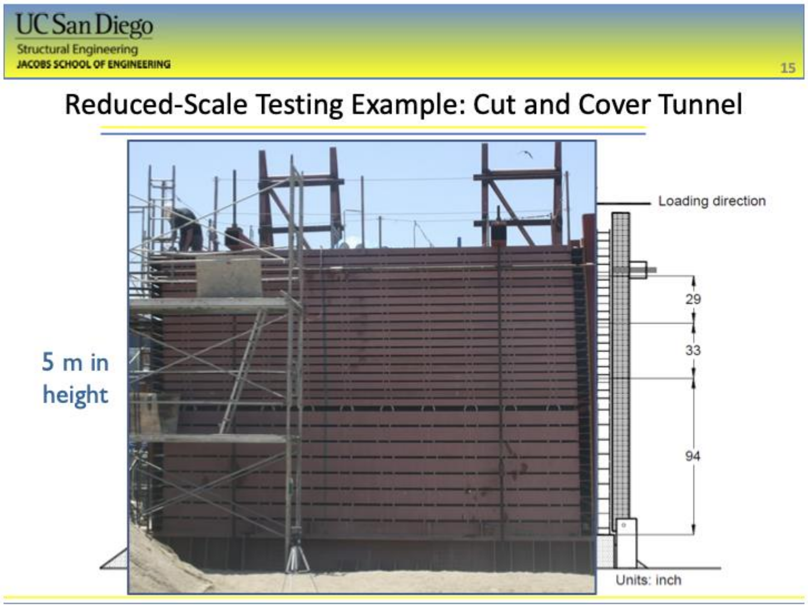


Full-Scale Testing Example: Dynamic Earth Pressures on Concrete Retaining Walls

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

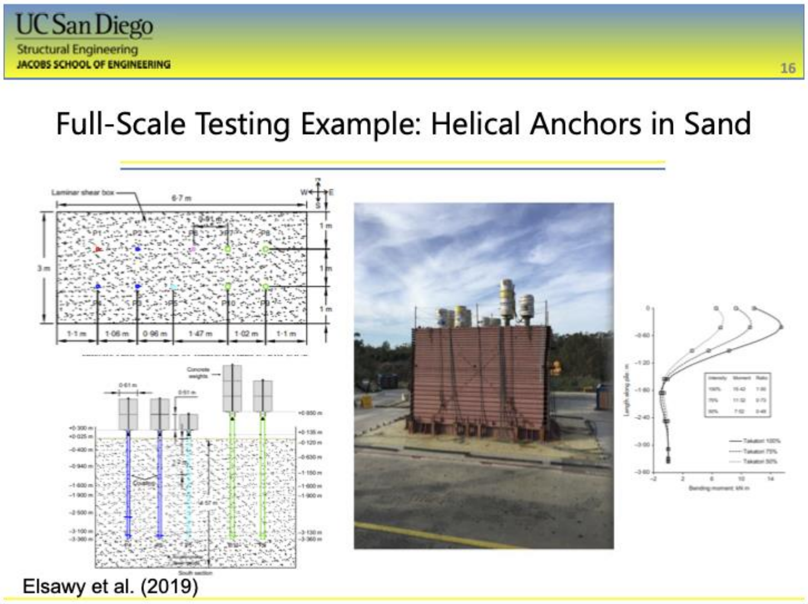
Full-Scale Testing Example:
Use of Actual Construction Techniques

- Well graded sand with 7% silt and up to 7% fine gravel (SW-SM) was used in all tests that meets Caltrans structural backfill specifications
- 95% relative compaction (around OMC) Verified by nuclear gauge measurements



Reduced-Scale Testing Example: Cut and Cover Tunnel

5 m in height

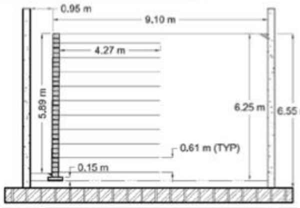


Full-Scale Testing Example: Helical Anchors in Sand

Elsawy et al. (2019)

Full-Scale Testing Example: MSE Wall Dynamic Response

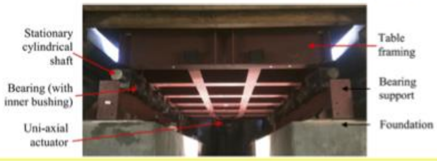
Sander et al. (2014), Fox et al. (2015)

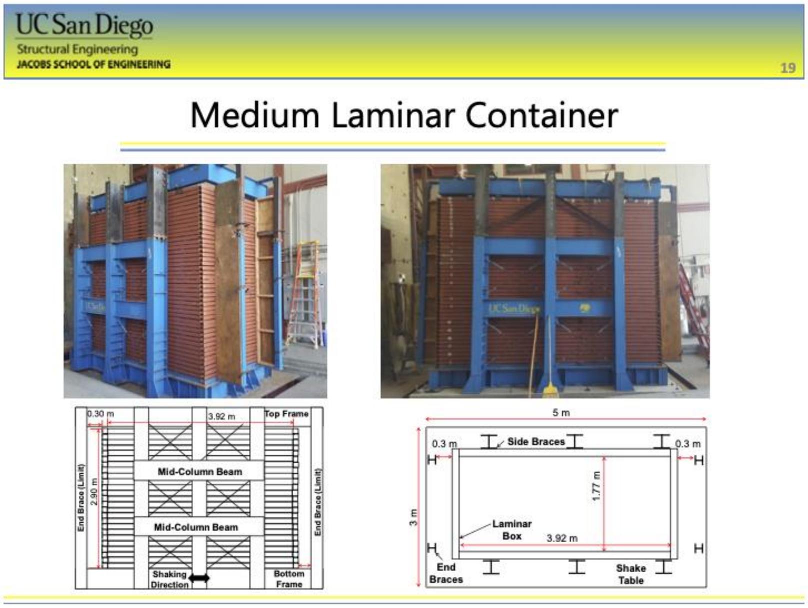


UCSD South Powell Laboratory Shake Table

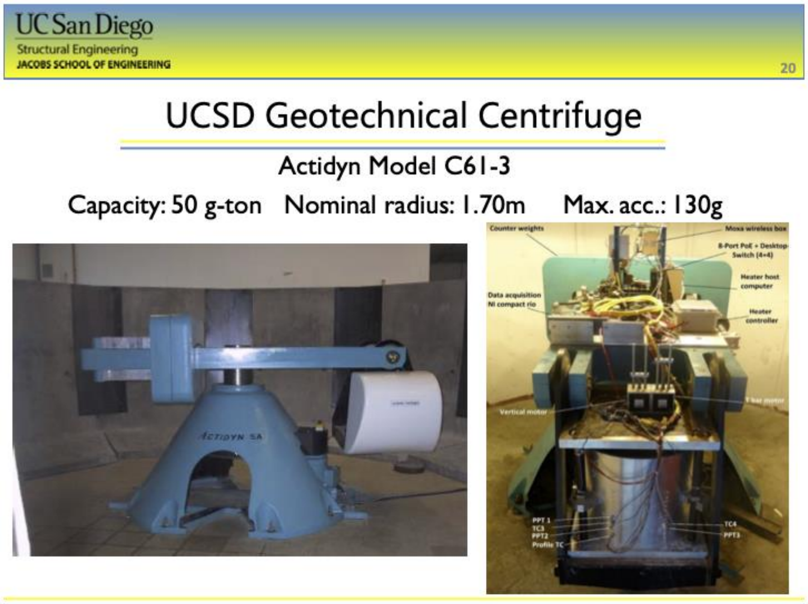
UCSD South Powell Structural Laboratory Shake Table:

- Dimension: 10 ft. x 16 ft.
- Shaking DOF: 1D in N-S direction
- Maximum gravity load: 80 kips
- Dynamic stroke: ± 6 in.
- Dynamic capacity: 90 kips





Medium Laminar Container

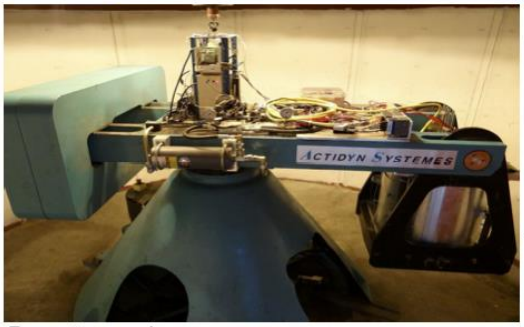


UCSD Geotechnical Centrifuge

Actidyn Model C61-3

Capacity: 50 g-ton Nominal radius: 1.70m Max. acc.: 130g

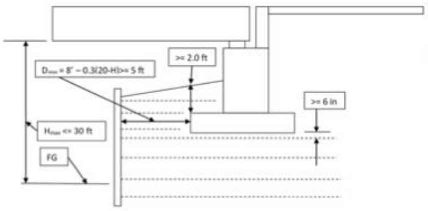
UCSD Geotechnical Centrifuge



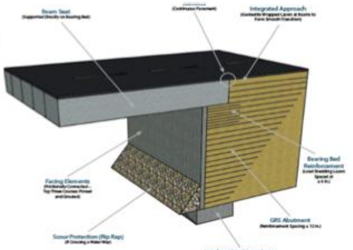
Recent upgrades:
- Control room
- New data acquisition system and actuators
- Miniature shaking table
- Containers (2 laminar, 3 self-reacting clay tanks)

Project 1: Seismic Response of MSE Bridge Abutments

Bridge seat with joint



Caltrans MSE Bridge Abutment



FHWA GRS-IBS Bridge Abutment

MSE and GRS-IBS bridge abutments have many advantages over pile-supported bridge abutments, including cost savings, easier and faster construction, and smoother transition

Research Motivation

MSE bridge abutments have been widely used in US, but there are concerns regarding the seismic performance:

- Geotechnical: backfill settlements and facing displacements
- Structural: bridge deck and seat movements, impact force between bridge deck and seat, and interaction between bridge superstructure and abutment

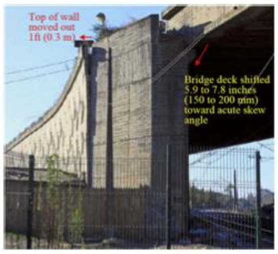
MSE wall performance in Maule Earthquake, Chile

Research Motivation

MSE bridge abutments have been widely used in US, but there are concerns regarding the seismic performance:

- Geotechnical: backfill settlements and facing displacements
- Structural: bridge deck and seat movements, impact force between bridge deck and seat, and interaction between bridge superstructure and abutment

MSE bridge abutment performance in Maule Earthquake, Chile

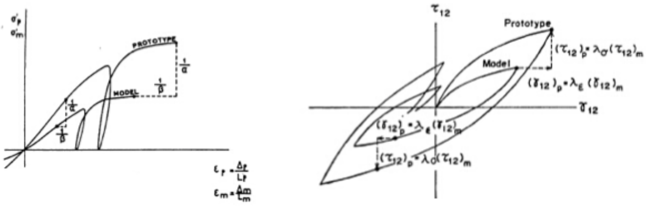


Project Objectives and Approach

- Objectives are to understand:
  - Bridge seat movement and rocking
  - Volumetric compression of the backfill soils
  - Bridge beam impact forces on the bridge seat and retained fill
  - Effects of transverse vs. longitudinal shaking
  - Effects of design details (reinforcement spacing, reinforcement stiffness, reinforcement type)
  - Wall face displacements during static and seismic loading
  - Reinforcement strains during static and seismic loading
- Approach
  - Reduced-scale (N=1/2) shake table testing
  - Reduced-scale testing permitted several models to be constructed with different configurations, materials, and shaking directions

Scaling in Reduced Scale 1g Tests

- Shear strength and stiffness of soils depend on the effective stress
  - Shear strength is typically linearly related to the effective stress
  - Stiffness is nonlinearly related to the effective stress
- The stress-strain curve may change as a function of effective stress (peak values may not occur at the same strain)
- Scaling relationships are thus required to design a reduced scale model so that results can be extrapolated from model to prototype



Monotonic and cyclic stress-strain relationships for model and prototype
(Rocha 1957; Roscoe 1968)

1-g Similitude Relationships

- ☐ Appropriate similitude relationships are needed for the design of reduced-scale model so that experimental results from reduced scale 1*g* shaking table tests can be extrapolated to full-scale conditions
- ☐ Most widely used set of 1*g* similitude relationships - **Iai (1989)**
  - ○ Basis: equilibrium and mass balance of soil, structures, and pore water
  - ○ Assumption: scaled stress-strain relationships for soil are independent of confining stress if appropriate scaling factors are selected
  - ○ Three independent scaling factors:
    - Geometry scaling factor λ – most important for reduced scale model design
    - Density scaling factor λ_ρ – typically assumed to be 1 for the same soil
    - Strain scaling factor λ_ε – can be determined using shear wave velocity measurements, typically assumed to be 1
  - ○ Applicability: applicable to deformation analysis prior to failure, not applicable to the ultimate state of stability due to large deformations or loss of soil contact

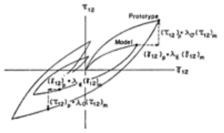
---

1-g Similitude Relationships

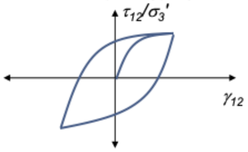
Similitude relationships (Iai 1989)

Variable	Scaling factor	$\lambda_\rho = 1$ $\lambda_\varepsilon = 1$	$\lambda = 2$
Length	λ	λ	2
Density	λ_ρ	1	1
Strain	λ_ε	1	1
Mass	$\lambda^3\lambda_\rho$	λ^3	8
Acceleration	1	1	1
Velocity	$(\lambda\lambda_\varepsilon)^{1/2}$	$\lambda^{1/2}$	1.414
Stress	$\lambda\lambda_\rho$	λ	2
Modulus	$\lambda\lambda_\rho/\lambda_\varepsilon$	λ	2
Stiffness	$\lambda^2\lambda_\rho/\lambda_\varepsilon$	λ^2	4
Force	$\lambda^3\lambda_\rho$	λ^3	8
Time	$(\lambda\lambda_\varepsilon)^{1/2}$	$\lambda^{1/2}$	1.414
Frequency	$(\lambda\lambda_\varepsilon)^{-1/2}$	$\lambda^{-1/2}$	0.707

Goal: Choose soil conditions to have a similar *normalized* stress-strain response in model and prototype for $\lambda_\varepsilon = 1$



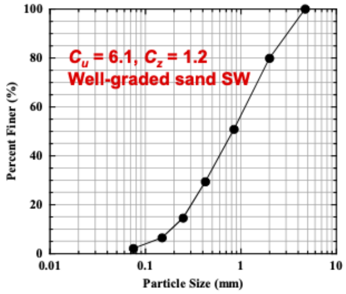
Original stress-strain relationships for soil in the model and prototype (Rocha 1957)

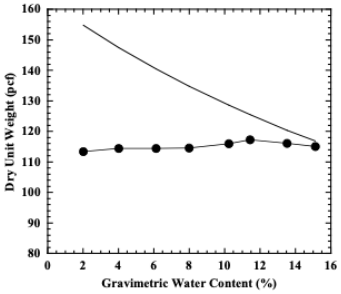


Normalized stress-strain relationships for soil in the model and prototype for $\lambda_\varepsilon=1$

Backfill Soil

- Sieve analysis – Gradation curve
- Standard Proctor compaction curve (not sensitive to water content)

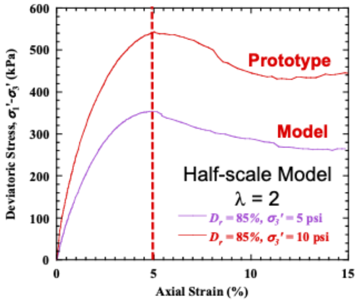




Selection of Compaction Conditions

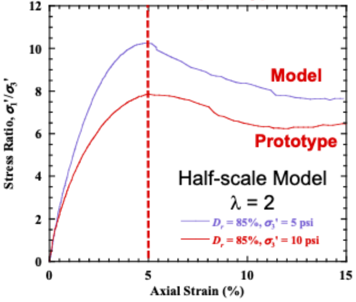
- Typical relative density (D_r) for prototype structures = 85% (RC = 96%)

Peak stresses occur at approximately same axial strain (5%), which indicates that the assumption of λ_ε = 1 is valid

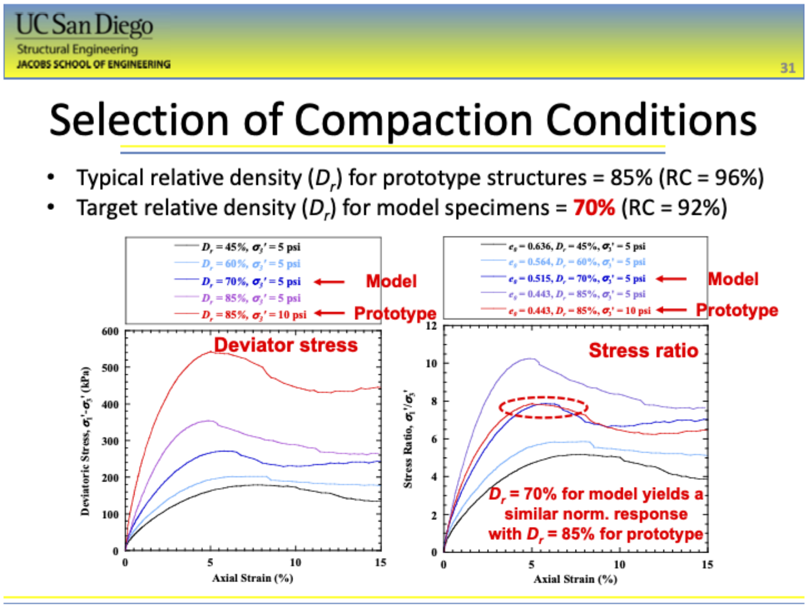


Deviator stress

However, due to the nonlinear effects of confining stress on shear strength and stiffness, the normalized curves are not similar for model and prototype for same D_r



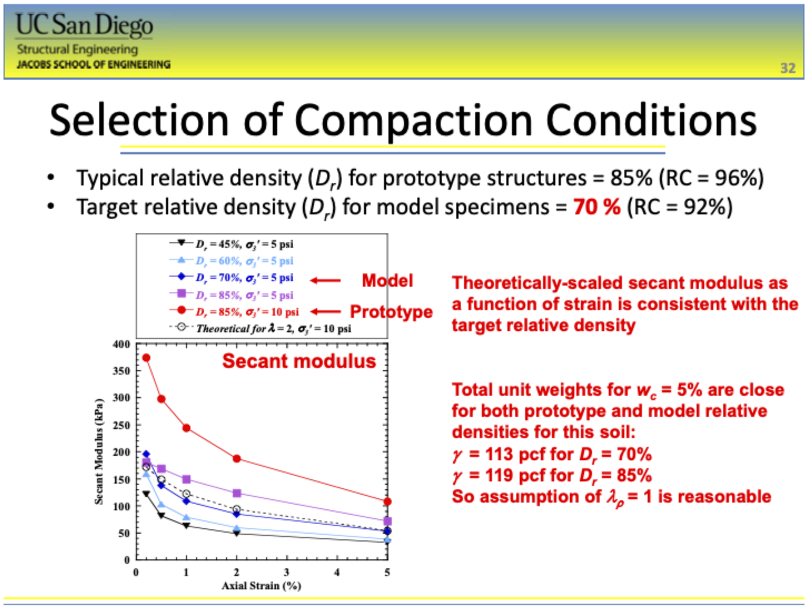
Stress ratio



Selection of Compaction Conditions

- Typical relative density (D_r) for prototype structures = 85% (RC = 96%)
- Target relative density (D_r) for model specimens = **70%** (RC = 92%)

D_r = 70% for model yields a similar norm. response with D_r = 85% for prototype



Selection of Compaction Conditions

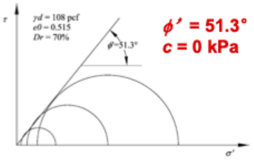
- Typical relative density (D_r) for prototype structures = 85% (RC = 96%)
- Target relative density (D_r) for model specimens = **70 %** (RC = 92%)

Theoretically-scaled secant modulus as a function of strain is consistent with the target relative density

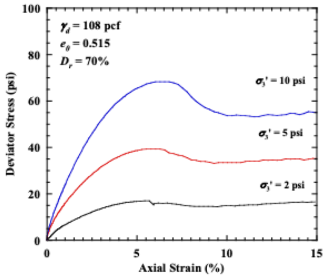
Total unit weights for w_c = 5% are close for both prototype and model relative densities for this soil:
γ = 113 pcf for D_r = 70%
γ = 119 pcf for D_r = 85%
So assumption of λ_ρ = 1 is reasonable

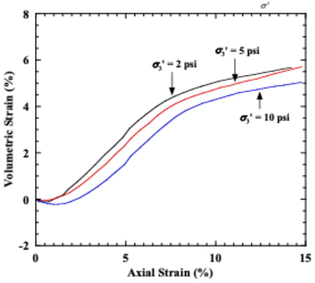
Backfill Soil

- Shear strength and volumetric behavior for $D_r = 70\%$



$\phi' = 51.3°$
$c = 0$ kPa



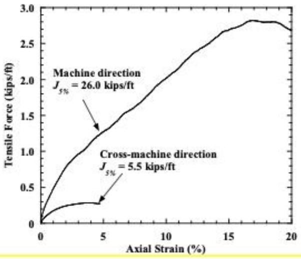


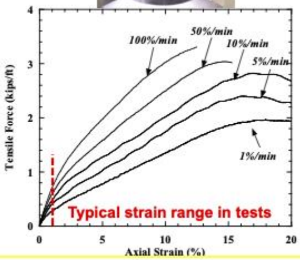
Geogrid Reinforcement

Prototype: Tensar UX 1700

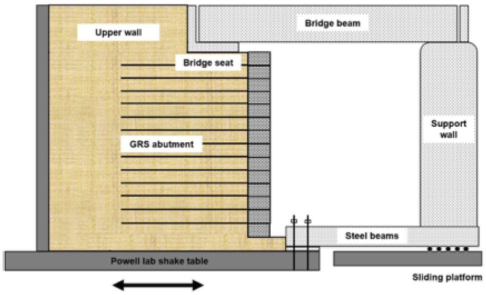
Model: Tensar LH 800

- Index stiffness = 26 kips/ft
- Stiffness scaling factor = 4





Target Prototype and Model Design



Model geometry scaling for $\lambda=2$

	Prototype	Model
Wall height (ft)	14	7
Bridge seat thickness (in)	12	6
Clearance height (ft)	15	7.5
Wall length (ft)	15.6	7.8
Wall width (ft)	14	7
Bridge width (ft)	6	3

Block scaling

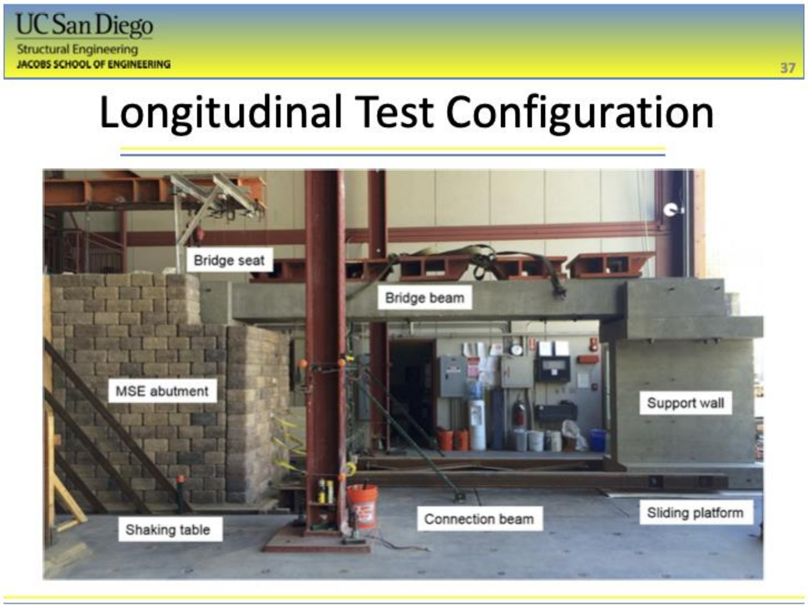
	Prototype	Model
Product	-	Keystone
Dimensions (L x W x H)	24 in x 12 in x 12 in	12 in x 10 in x 6 in

Reinforcement scaling

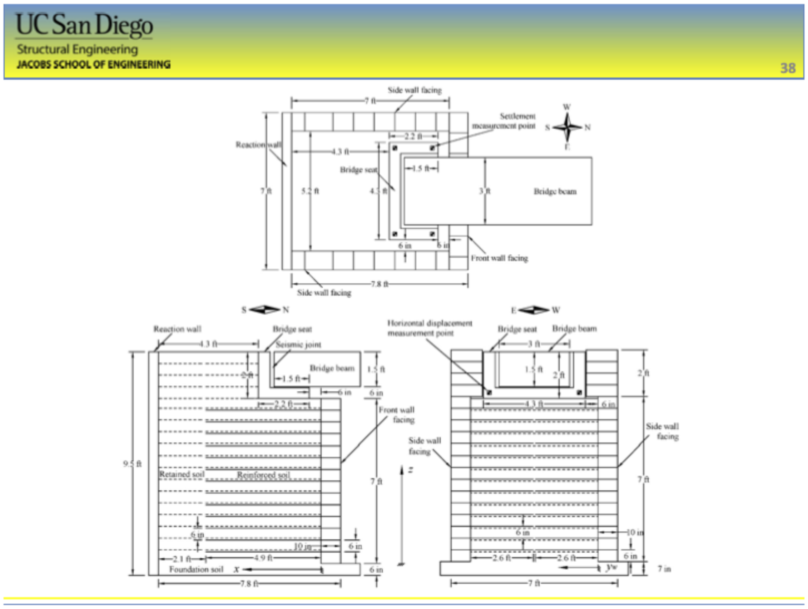
	Prototype	Model
Product	UX1700	LH800
Stiffness (kips/ft)	100	26

Testing Plan

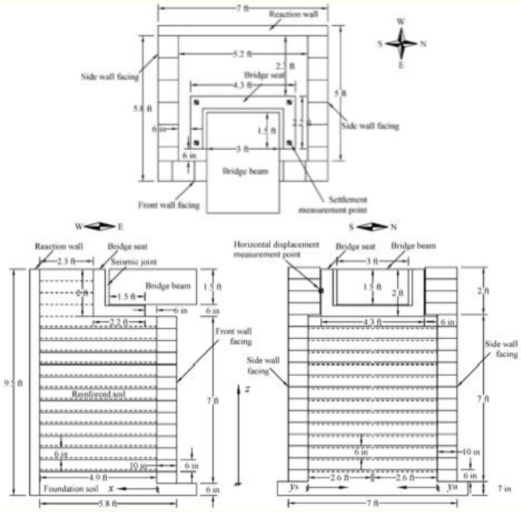
Test No.	Variable	Bridge Surcharge Stress (psf)	Reinforcement Spacing (in)	Reinforcement Stiffness (kips/ft)	Shaking Direction
1	Baseline	1380	6	26	Longitudinal
2	Bridge Surcharge Stress	900	6	26	Longitudinal
3	Geogrid Reinforcement Spacing	1380	12	26	Longitudinal
4	Geogrid Reinforcement Stiffness	1380	6	13	Longitudinal
5	Steel mesh Reinforcement	1380	6	330	Longitudinal
6	Shaking Direction	1380	6	26	Transverse



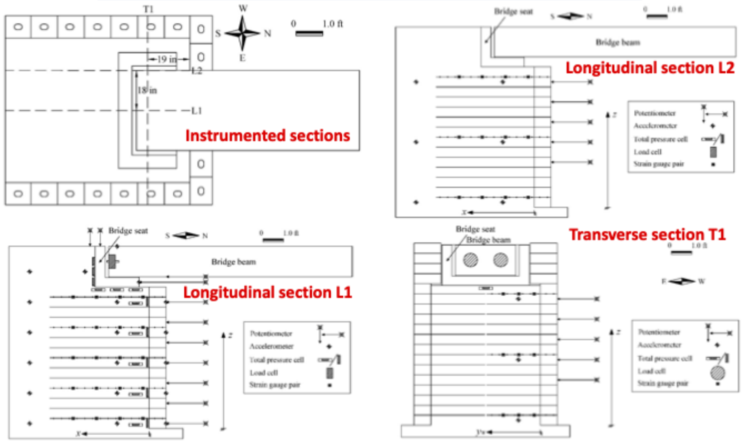
Longitudinal Test Configuration



Transverse Test Configuration



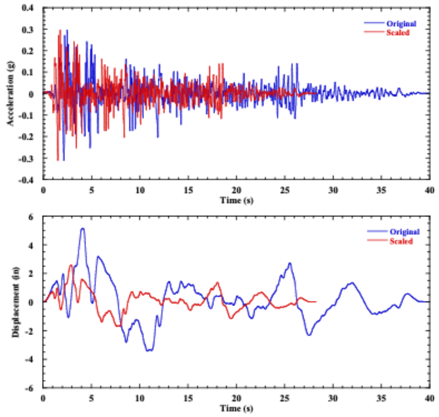
Instrumentation - Longitudinal Test



Scaling of Earthquake Motions

For a ½ scale model:

- Frequency of motion is increased by √2, which shortens the duration
- Acceleration amplitude stays the same as the original motion
- Displacement amplitude is scaled by ½

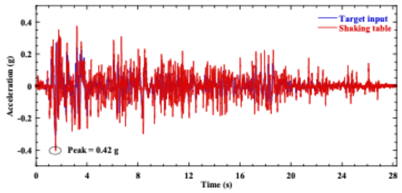


Imperial Valley Motion

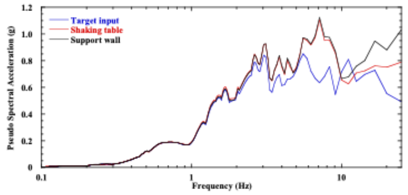
Input Motions

Shaking event	Motion	PGA (g)	PGD (in)
1	White Noise	0.10	0.11
2	1940 Imperial Valley	0.31	2.57
3	White Noise	0.10	0.11
4	2010 Maule	0.40	4.25
5	White Noise	0.10	0.11
6	1994 Northridge*	0.58	3.49
7	White Noise	0.10	0.11
8	*Sin @ 0.5 Hz*	*0.05*	*1.97*
9	*Sin @ 1 Hz*	*0.10*	*0.98*
10	*Sin @ 2 Hz*	*0.20*	*0.49*
11	*Sin @ 5 Hz*	*0.25*	*0.10*
12	*White Noise*	*0.10*	*0.11*

Longitudinal Testing System



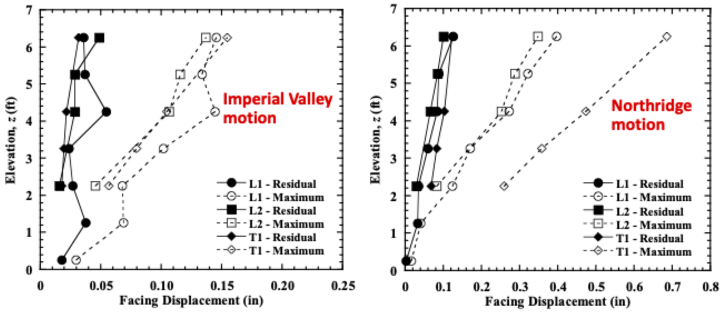
Imperial Valley Earthquake Acceleration Time History



Response Spectra

- Measured displacement time histories for the shaking table, reaction wall, and support wall are identical with the target input displacement time history
- Actual shaking table acceleration time history in general matches well with the target input accelerations
- Actual pseudo-spectral accelerations for the shaking table agree reasonably well with the target values

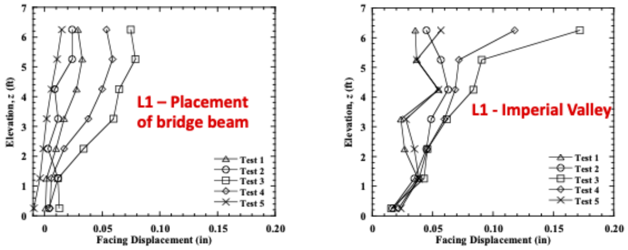
Facing Displacements



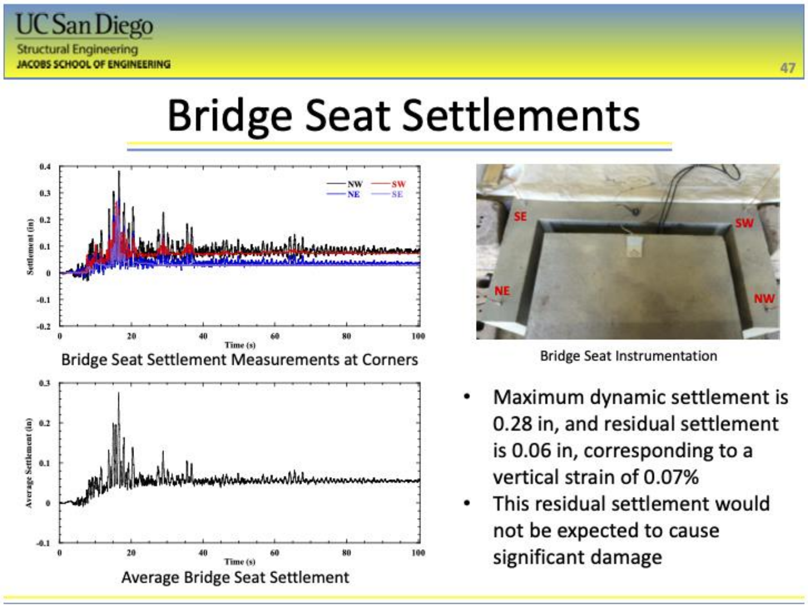
- Seismic displacements at the top are larger than the bottom
- Residual displacements are generally small (max 0.14 in for the Northridge motion)
- Longitudinal shaking resulted in displacements in transverse direction

Facing Displacements

Test No.	1	2	3	4	5
Description	Baseline	Reduced Bridge Load	Increased Reinforcement Spacing	Reduced Reinforcement Stiffness	Steel Reinforcement



- Reinforcement spacing and stiffness have the most significant effects
- Greater bridge load resulted in larger displacements under static loading, but smaller residual displacements from seismic loading



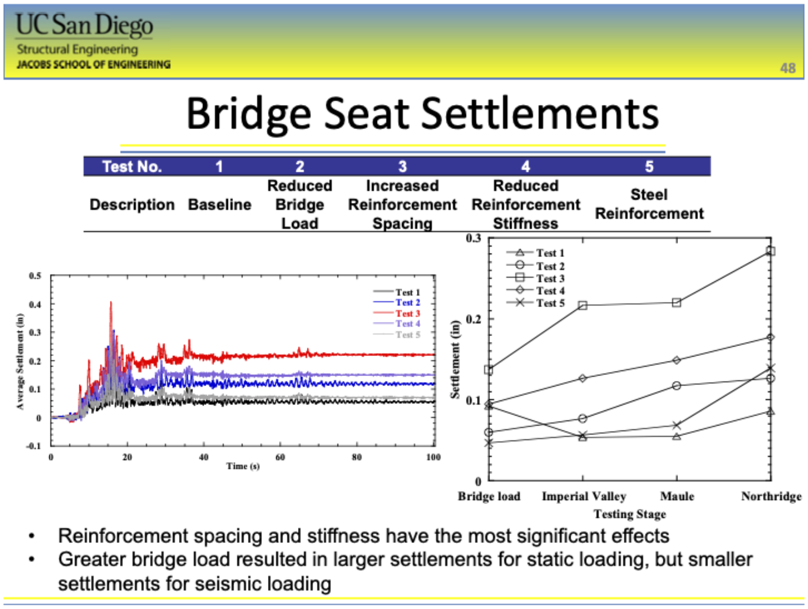
Bridge Seat Settlements

Bridge Seat Settlement Measurements at Corners

Bridge Seat Instrumentation

Average Bridge Seat Settlement

- Maximum dynamic settlement is 0.28 in, and residual settlement is 0.06 in, corresponding to a vertical strain of 0.07%
- This residual settlement would not be expected to cause significant damage



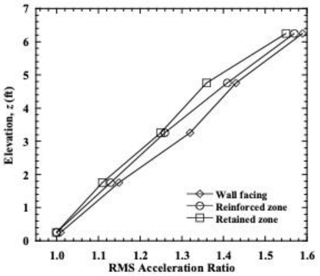
Bridge Seat Settlements

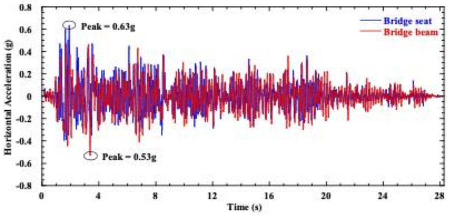
Test No.	1	2	3	4	5
Description	Baseline	Reduced Bridge Load	Increased Reinforcement Spacing	Reduced Reinforcement Stiffness	Steel Reinforcement

- Reinforcement spacing and stiffness have the most significant effects
- Greater bridge load resulted in larger settlements for static loading, but smaller settlements for seismic loading

Acceleration Responses

Accelerations for the Imperial Valley motion in Test 1



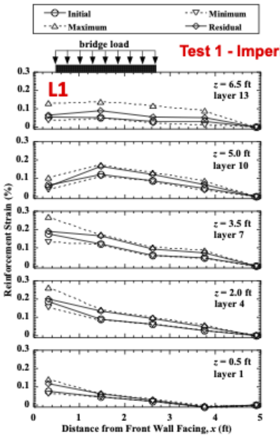


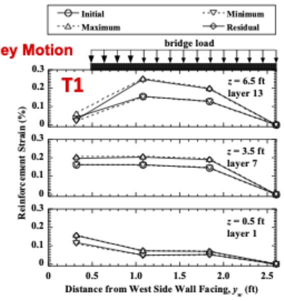
Amplification ratio = 1.60 for bridge seat
Amplification ratio = 1.80 for bridge beam

- Acceleration amplification increases with elevation in the MSE bridge abutment
- Amplification ratios increase from retained zone to reinforced zone to wall facing
- Amplification ratio for bridge beam is larger than bridge seat

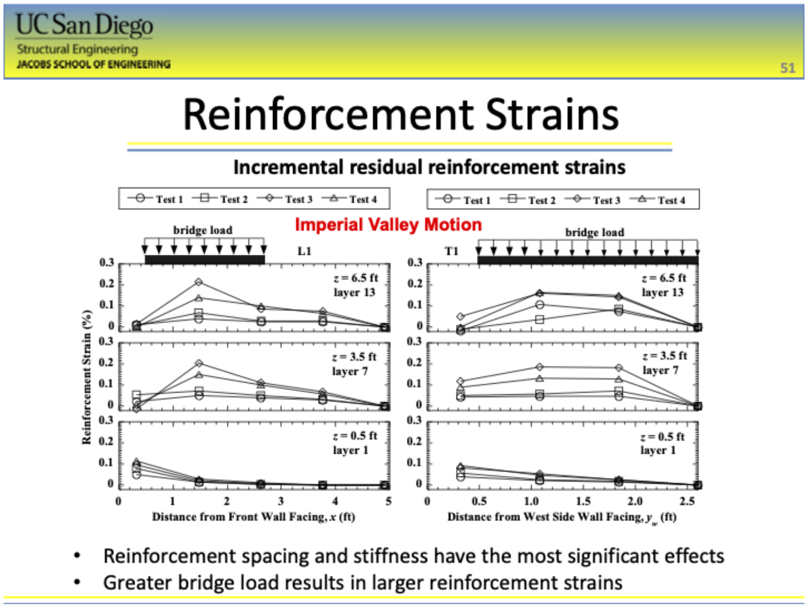
Reinforcement Strains

Test 1 - Imperial Valley Motion



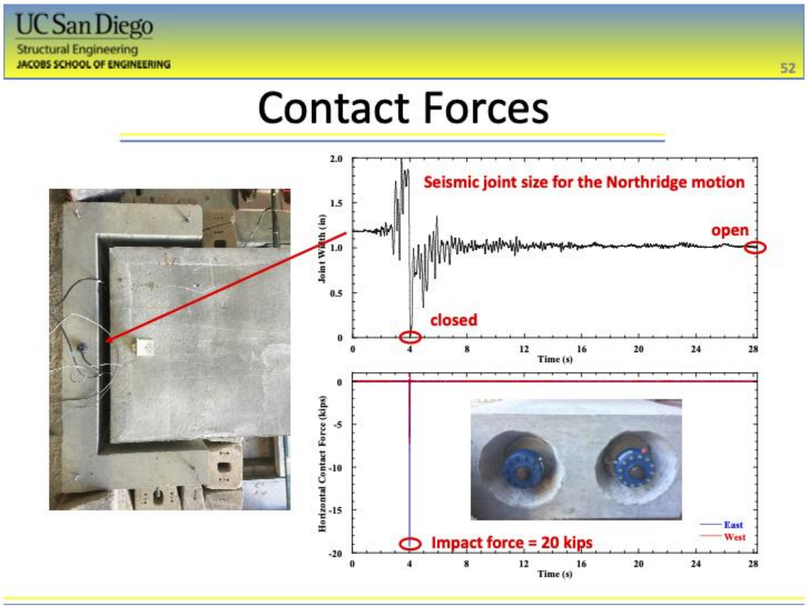


- Max strains are small and far from the ultimate capacity of the geogrids
- Location of max strains under the bridge seat in upper layers and near connections in lower layers
- Longitudinal shaking resulted in strains in the transverse direction

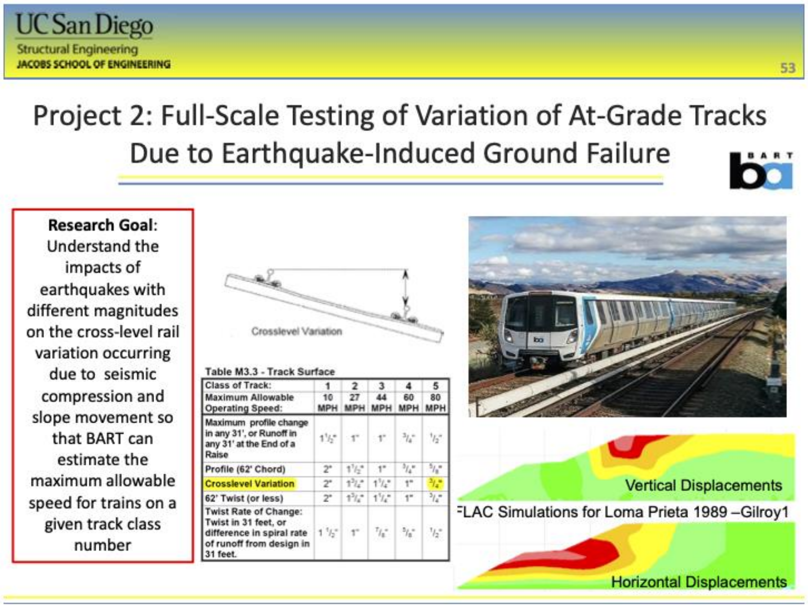


Reinforcement Strains

- Reinforcement spacing and stiffness have the most significant effects
- Greater bridge load results in larger reinforcement strains



Contact Forces



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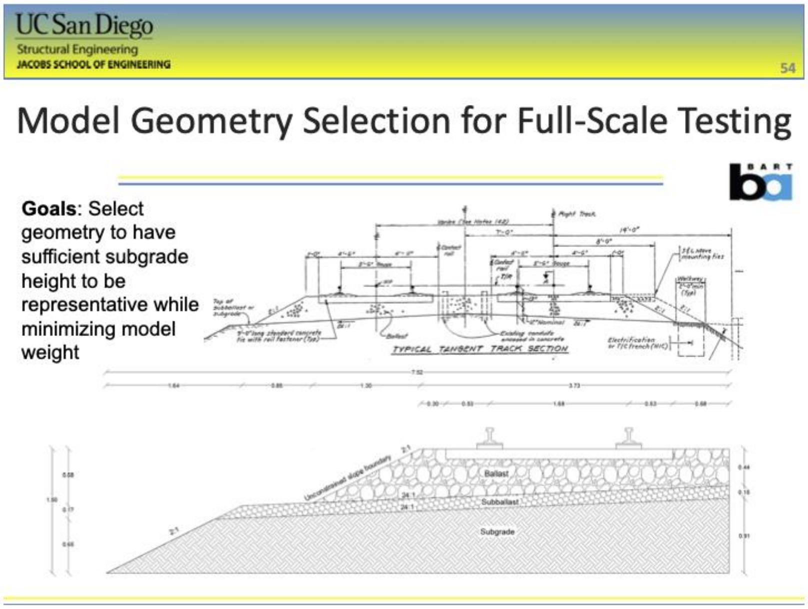
Project 2: Full-Scale Testing of Variation of At-Grade Tracks Due to Earthquake-Induced Ground Failure

Research Goal: Understand the impacts of earthquakes with different magnitudes on the cross-level rail variation occurring due to seismic compression and slope movement so that BART can estimate the maximum allowable speed for trains on a given track class number

FLAC Simulations for Loma Prieta 1989 –Gilroy1

Table M3.3 - Track Surface

Class of Track:	1	2	3	4	5
Maximum Allowable Operating Speed:	10 MPH	27 MPH	44 MPH	60 MPH	80 MPH
Maximum profile change in any 31', or Runoff in any 31' at the End of a Raise	1½"	1"	1"	¾"	½"
Profile (62' Chord)	2"	1½"	1"	¾"	⅝"
Crosslevel Variation	2"	1¾"	1¼"	1"	¾"
62' Twist (or less)	2"	1¾"	1¼"	1"	¾"
Twist Rate of Change: Twist in 31 feet, or difference in spiral rate of runoff from design in 31 feet.	1 ½"	1"	⅞"	⅝"	½"



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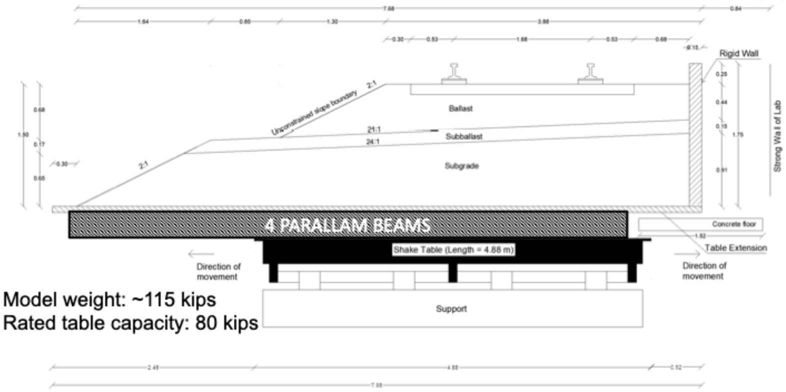
Model Geometry Selection for Full-Scale Testing

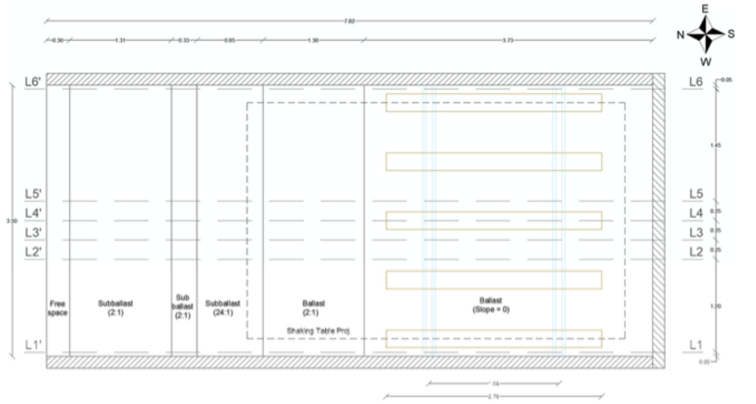
Goals: Select geometry to have sufficient subgrade height to be representative while minimizing model weight

Shake Table Modification For Full-Scale Rail Embankment Testing

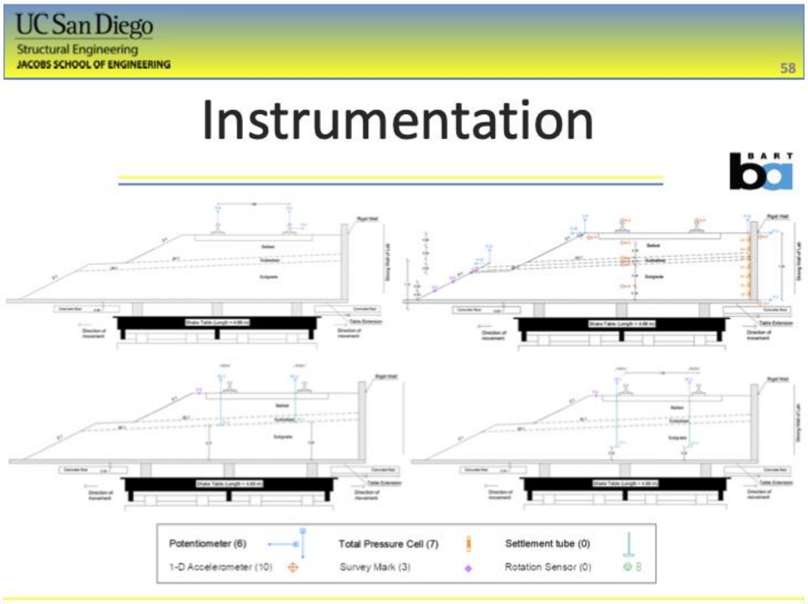


Model weight: ~115 kips
Rated table capacity: 80 kips

Shake Table Modification For Full-Scale Rail Embankment Testing



Project 2: Full-Scale Testing of Variation of At-Grade Tracks Due to Earthquake-Induced Ground Failure



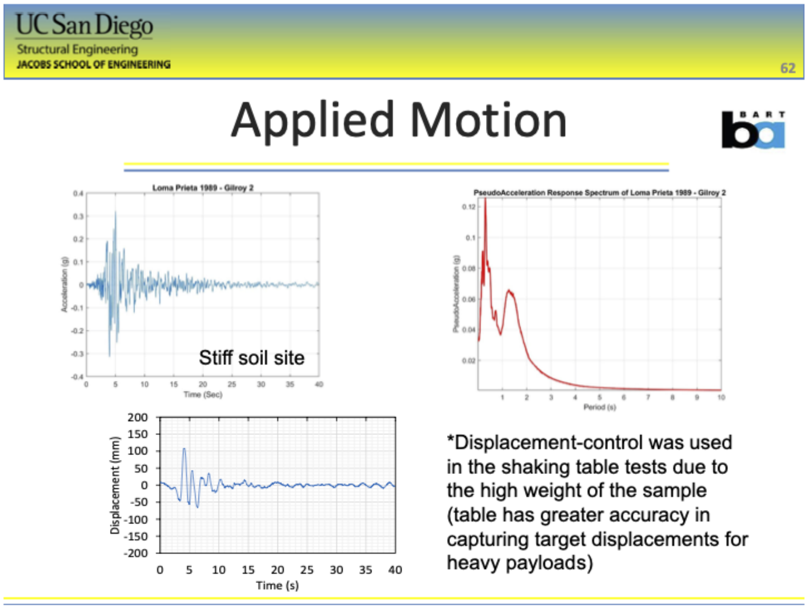
Instrumentation

Potentiometer (6)
Total Pressure Cell (7)
Settlement tube (0)
1-D Accelerometer (10)
Survey Mark (3)
Rotation Sensor (0)

Instrumentation Rack

Instrumentation

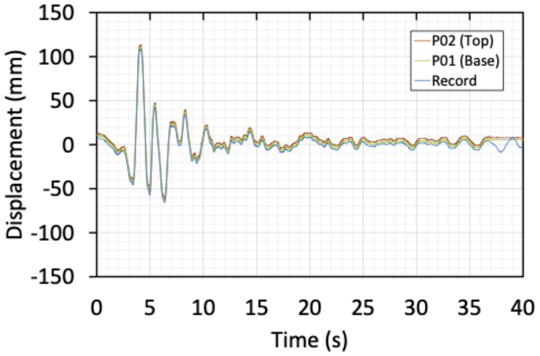
Instrumentation



Applied Motion

*Displacement-control was used in the shaking table tests due to the high weight of the sample (table has greater accuracy in capturing target displacements for heavy payloads)

Empty Box Shakes

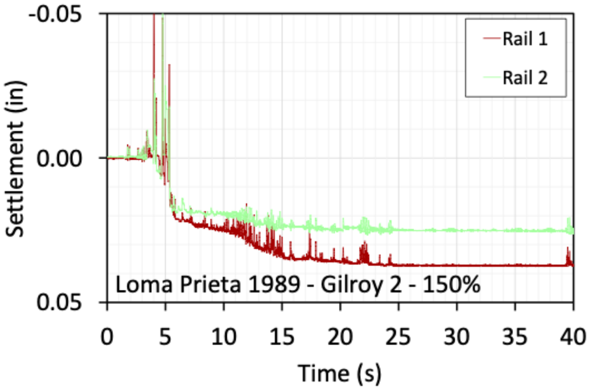


Typical Shaking Sequence

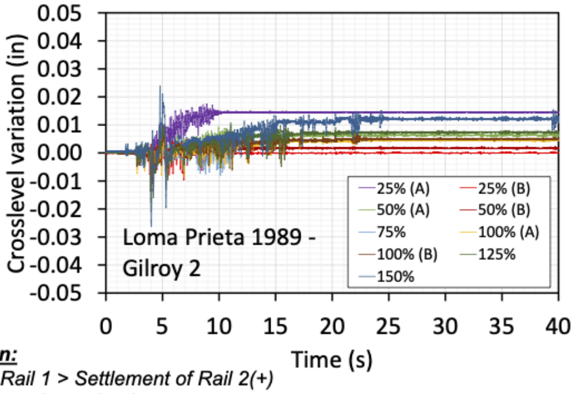
Order	Motion	Peak acceleration (g)	Amplitude	Dominant Frequency (Hz)	Duration (s)	Performed
1	White Noise ST5-01				120	
2	Loma Prieta 1989 – Gilroy 2	0.08	25%	0.68	40	
3	White Noise ST5-02				120	
4	Loma Prieta 1989 – Gilroy 2	0.16	50%	0.68	40	
5	White Noise ST5-03				120	
6	Loma Prieta 1989 – Gilroy 2	0.24	75%	0.68	40	
7	White Noise ST5-04				120	
8	Loma Prieta 1989 – Gilroy 2	0.32	100%	0.68	40	
9	White Noise ST5-05				120	
10	Loma Prieta 1989 – Gilroy 2	0.40	125%	0.68	40	
11	White Noise ST5-06				120	
12	Loma Prieta 1989 – Gilroy 2	0.48	150%	0.68	40	
13	White Noise ST5-07				120	
14	Loma Prieta 1989 – Gilroy 2	0.08	25%	0.68	40	
15	White Noise ST5-08				120	
16	Loma Prieta 1989 – Gilroy 2	0.16	50%	0.68	40	
17	White Noise ST5-09				120	
18	Loma Prieta 1989 – Gilroy 2	0.24	75%	0.68	40	
19	White Noise ST5-10				120	
20	Loma Prieta 1989 – Gilroy 2	0.32	100%	0.68	40	
21	White Noise ST5-11				120	

Repeat shakes

Typical Results

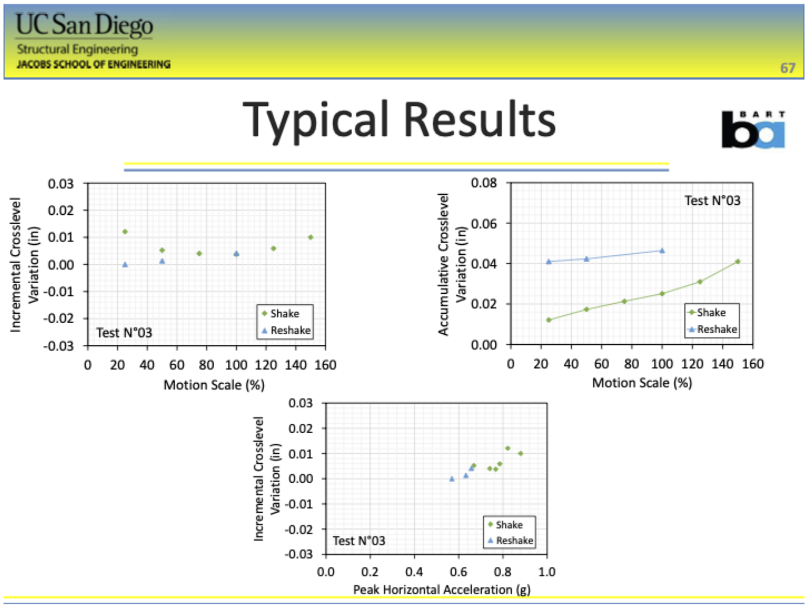


Typical Results



Sign Convention:

- *Settlement of Rail 1 > Settlement of Rail 2(+)*
- *Rail 1 is closer to the embankment crest*



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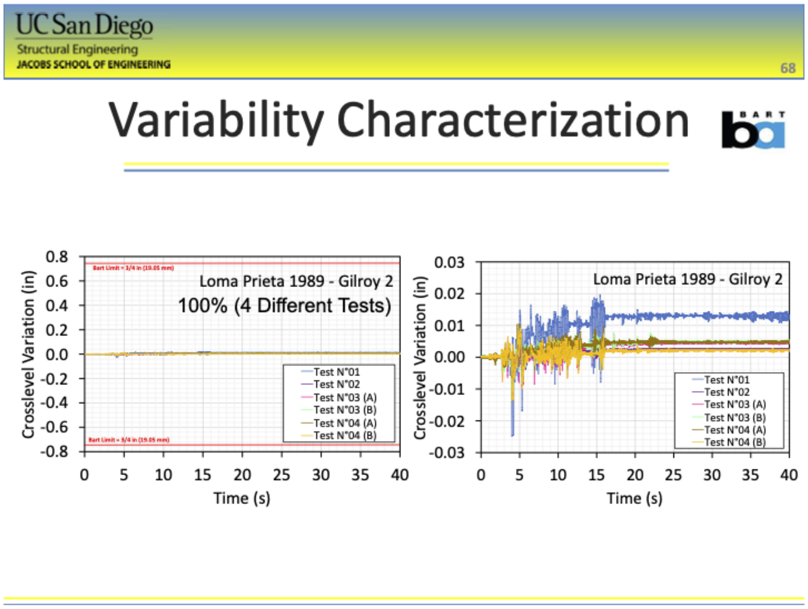
Typical Results



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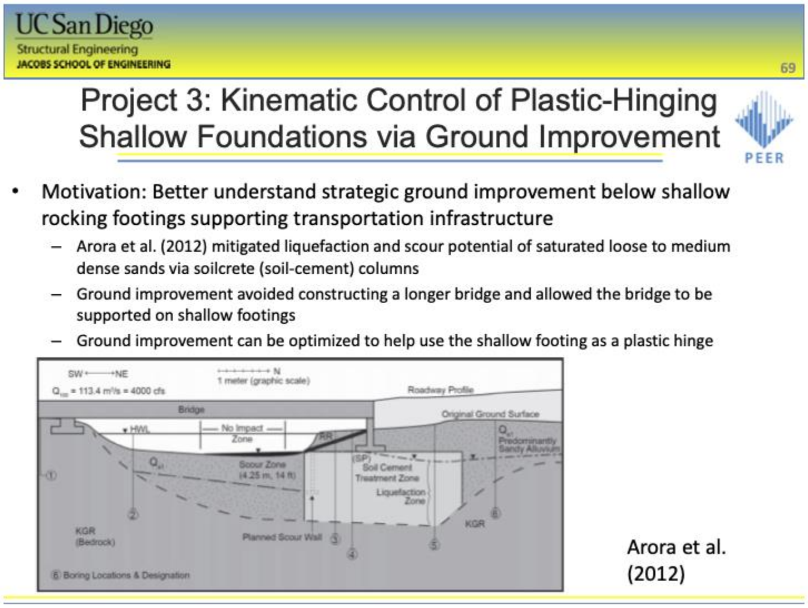
Variability Characterization

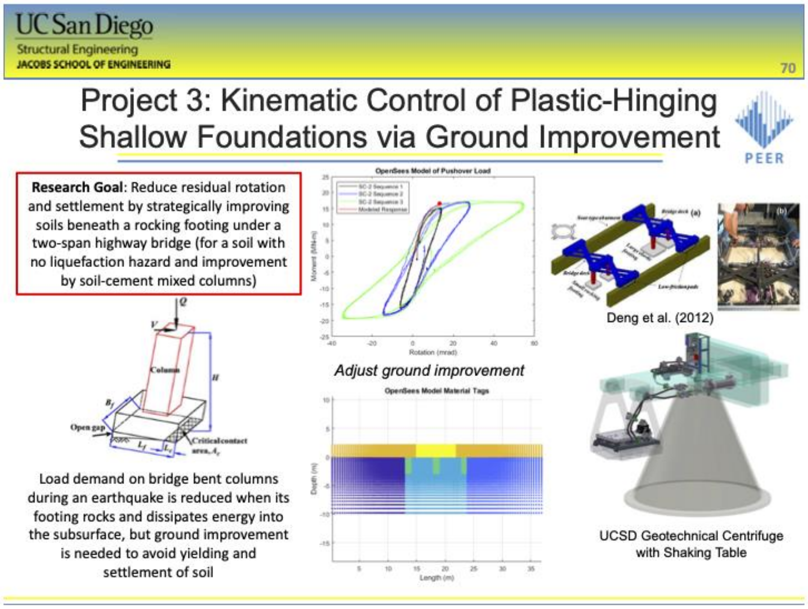


Project 3: Kinematic Control of Plastic-Hinging Shallow Foundations via Ground Improvement

PEER

- Motivation: Better understand strategic ground improvement below shallow rocking footings supporting transportation infrastructure
  - Arora et al. (2012) mitigated liquefaction and scour potential of saturated loose to medium dense sands via soilcrete (soil-cement) columns
  - Ground improvement avoided constructing a longer bridge and allowed the bridge to be supported on shallow footings
  - Ground improvement can be optimized to help use the shallow footing as a plastic hinge

Arora et al. (2012)



Project 3: Kinematic Control of Plastic-Hinging Shallow Foundations via Ground Improvement

PEER

Research Goal: Reduce residual rotation and settlement by strategically improving soils beneath a rocking footing under a two-span highway bridge (for a soil with no liquefaction hazard and improvement by soil-cement mixed columns)

Load demand on bridge bent columns during an earthquake is reduced when its footing rocks and dissipates energy into the subsurface, but ground improvement is needed to avoid yielding and settlement of soil

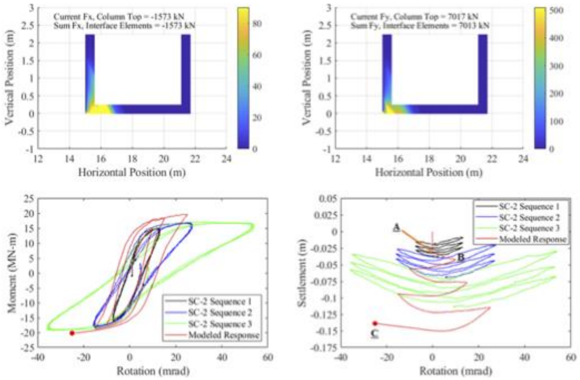
Adjust ground improvement

Deng et al. (2012)

UCSD Geotechnical Centrifuge with Shaking Table

Project 3: Kinematic Control of Plastic-Hinging Shallow Foundations via Ground Improvement

after final pushover load has been applied, point C



Centrifuge Modeling of Rocking Foundations

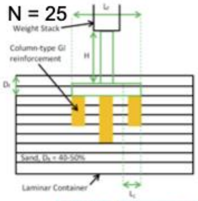
Centrifuge Scaling Relationships for
Dynamic Testing from Kutter (1992)

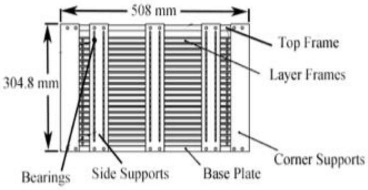
Quantity	Symbol	Units	Scale Factor
Length	L	L	N^{-1}
Volume	v	L^3	N^{-3}
Mass	M	M	N^{-3}
Gravity	g	LT^{-2}	N
Force	F	MLT^{-2}	N^{-2}
Stress	σ	$ML^{-1}T^{-2}$	1
Moduli	E	$ML^{-1}T^{-2}$	1
Strength	s	$ML^{-1}T^{-2}$	1
Acceleration	a	LT^{-2}	N
Time (dynamic)	t_{dyn}	T	N^{-1}
Frequency	f	T^{-1}	N
Time (diffusion) *	t_{dif}	T	N^{-1} or N^{-2}

Time scaling conflict

*The diffusion time scale factor depends on whether the diffusion coefficient (e.g., coefficient of consolidation) is scaled. If the same soil is used in model and prototype, $t_{dif}^* = N^{-2}$.

Centrifuge Modeling of Rocking Foundations

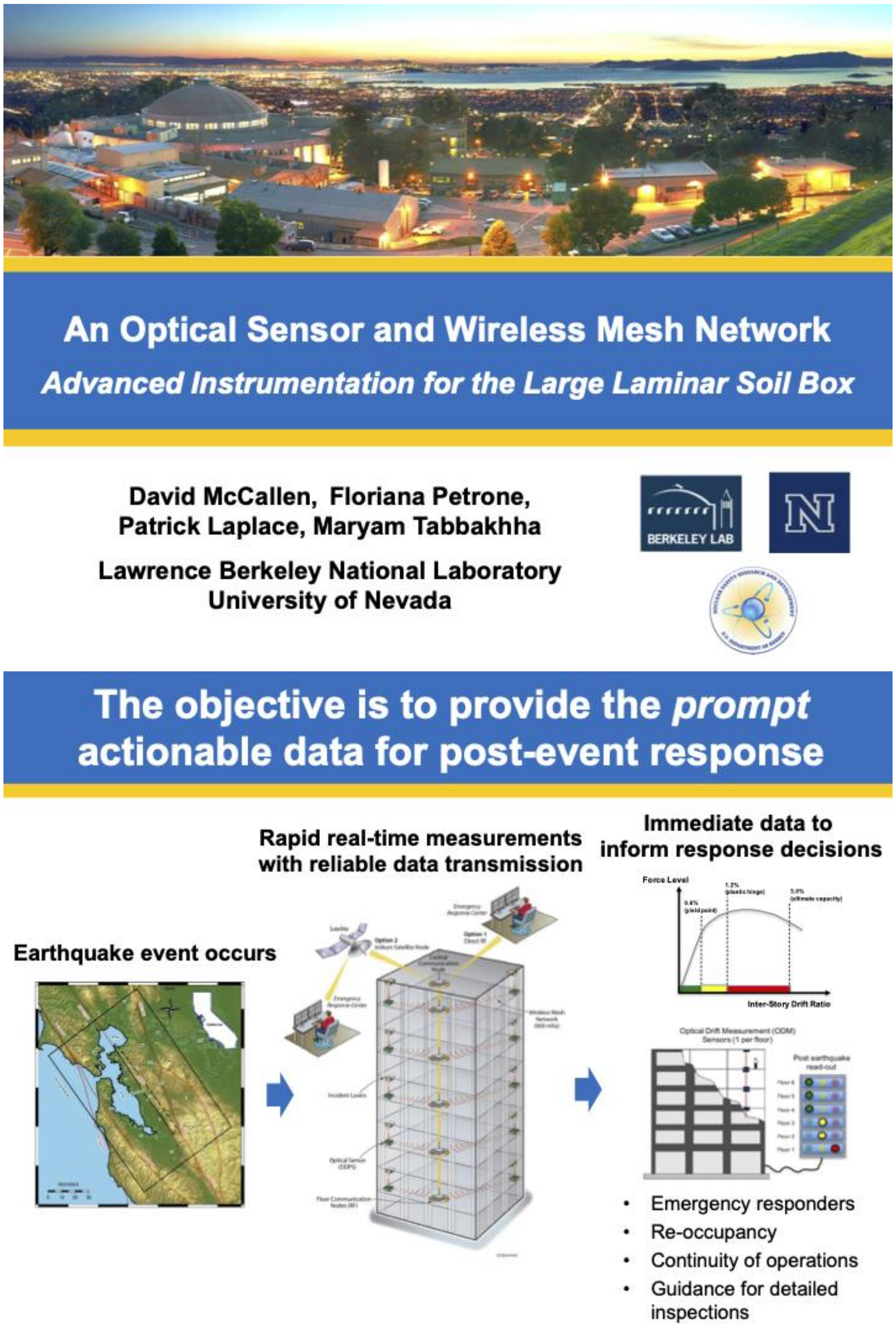




Conclusions

- Shaking table tests can be performed a different scales if similitude relationships are carefully considered
- Full-scale 1-g tests may be suitable for characterizing behavior under field compaction conditions with actual construction procedures
  - Can push the limits of table capacity
  - Need to decide how big is big enough?
- Reduced-scale 1-g tests can permit multiple tests to be performed to understand effects of key performance variables
  - Can come close to field conditions, but may need to compromise on some details
- Centrifuge models should be simplified to focus on key mechanisms,
  - Useful to guide full-scale testing
- Numerical modeling with carefully calibrated constitutive relationships should compliment shake table experiments

ADVANCED INSTRUMENTATION FOR LARGE-SCALE LAMINAR SOIL BOX SHAKE TABLE TESTING



An Optical Sensor and Wireless Mesh Network

Advanced Instrumentation for the Large Laminar Soil Box

David McCallen, Floriana Petrone,
Patrick Laplace, Maryam Tabbakhha

Lawrence Berkeley National Laboratory
University of Nevada

The objective is to provide the *prompt* actionable data for post-event response

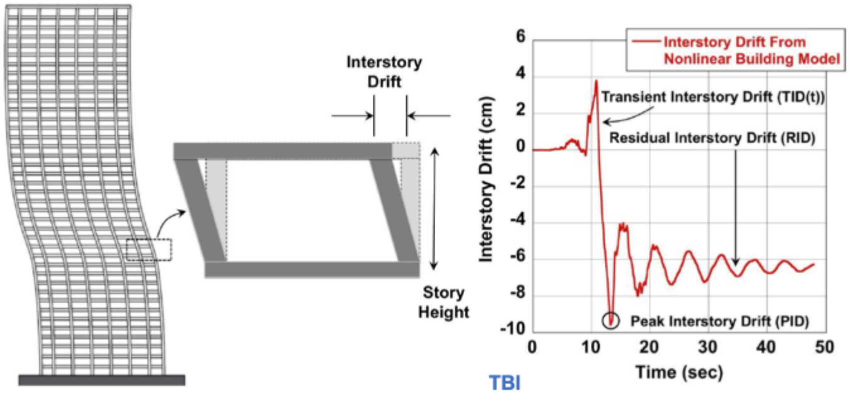
Earthquake event occurs

Rapid real-time measurements with reliable data transmission

Immediate data to inform response decisions

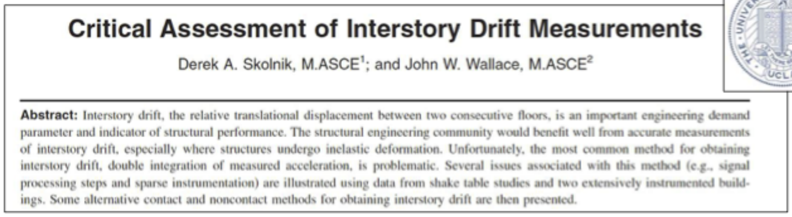
- Emergency responders
- Re-occupancy
- Continuity of operations
- Guidance for detailed inspections

Interstory drift is an essential earthquake demand parameter



Current uses of interstory drift in engineering standards	**ASCE 43-05** **Limit States** **(PID)**	**TBI** **ASCE 7-02** **EN 1998-01** **Not-to-Exceed Drift Limits** **(PID)**	**FEMA P58** **Damage State Drift Limits** **(RID)**

The challenges of accurately measuring interstory drift have been well documented



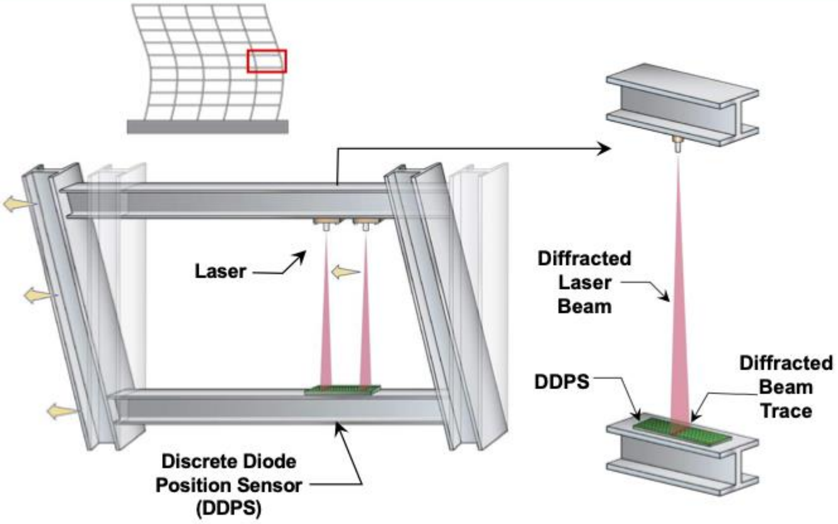
Critical Assessment of Interstory Drift Measurements

Derek A. Skolnik, M.ASCE[1]; and John W. Wallace, M.ASCE[2]

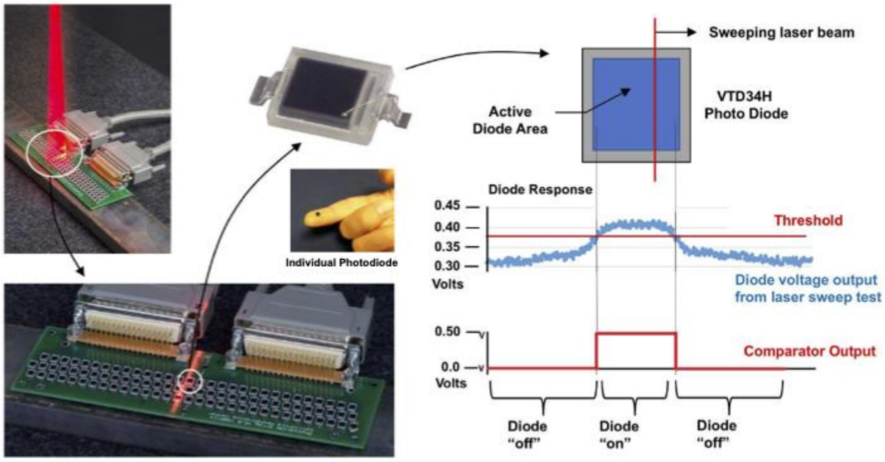
Abstract: Interstory drift, the relative translational displacement between two consecutive floors, is an important engineering demand parameter and indicator of structural performance. The structural engineering community would benefit well from accurate measurements of interstory drift, especially where structures undergo inelastic deformation. Unfortunately, the most common method for obtaining interstory drift, double integration of measured acceleration, is problematic. Several issues associated with this method (e.g., signal processing steps and sparse instrumentation) are illustrated using data from shake table studies and two extensively instrumented buildings. Some alternative contact and noncontact methods for obtaining interstory drift are then presented.

"Interstory drift, the relative translational displacement between two consecutive floors, is an important engineering demand parameter and indicator of structural performance. The structural engineering community would benefit well from accurate measurements of interstory drift, especially when structures undergo inelastic deformation. ***Unfortunately, the most common method for obtaining interstory drift, double integration of measured acceleration is problematic***"

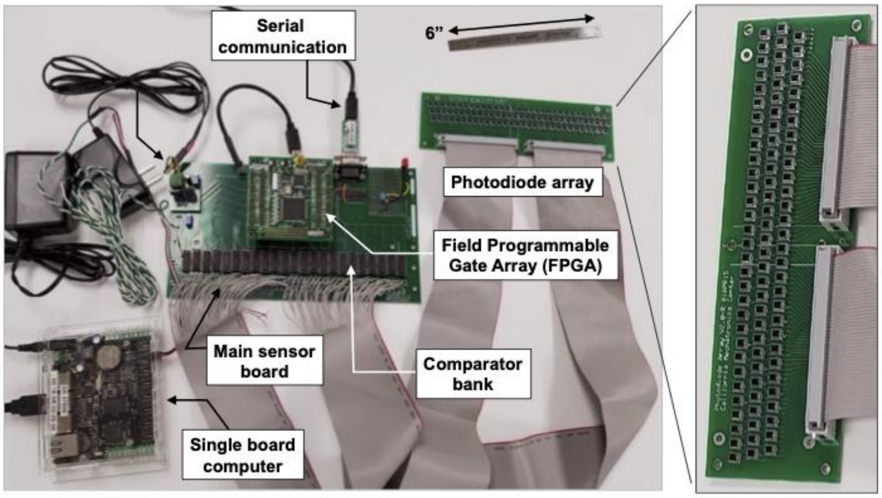
The concept - exploit the physics of light for direct, broad-band drift measurement



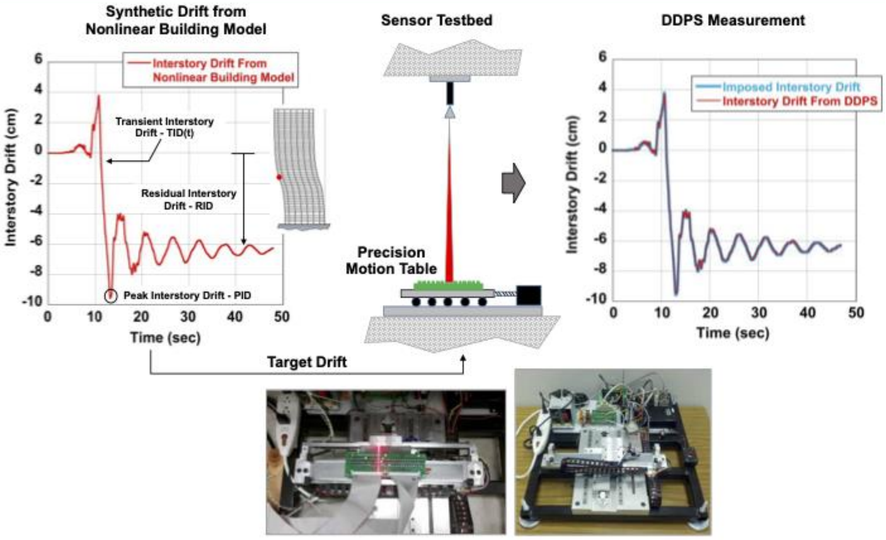
Inexpensive light-sensitive diodes are at the heart of our sensor concept



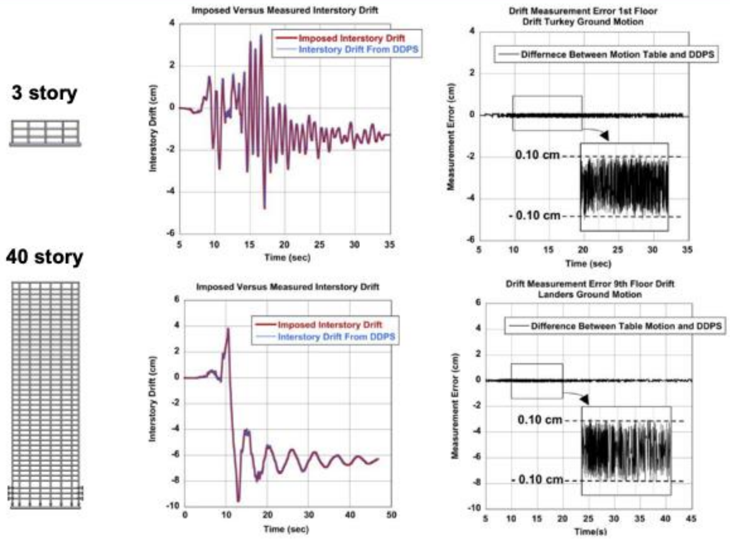
Generation 1 - a prototype sensor for proof of concept



Testbed #1 - a motion table for generating realistic interstory drifts was developed

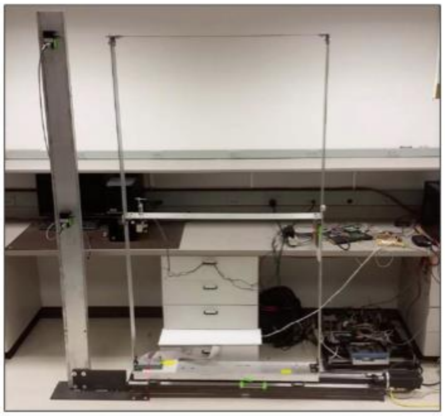


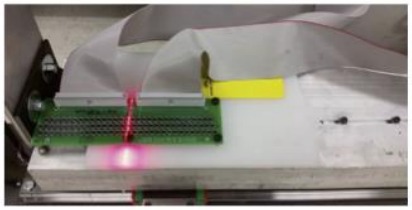
Prototype sensor measurements demonstrated excellent drift measurement



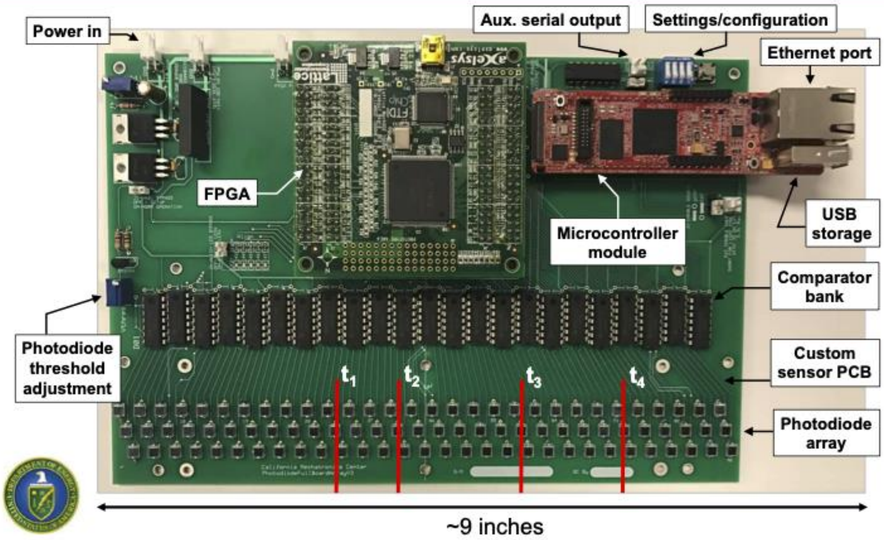
Testbed #2 - a simple laboratory scale frame structure

El Centro motion test



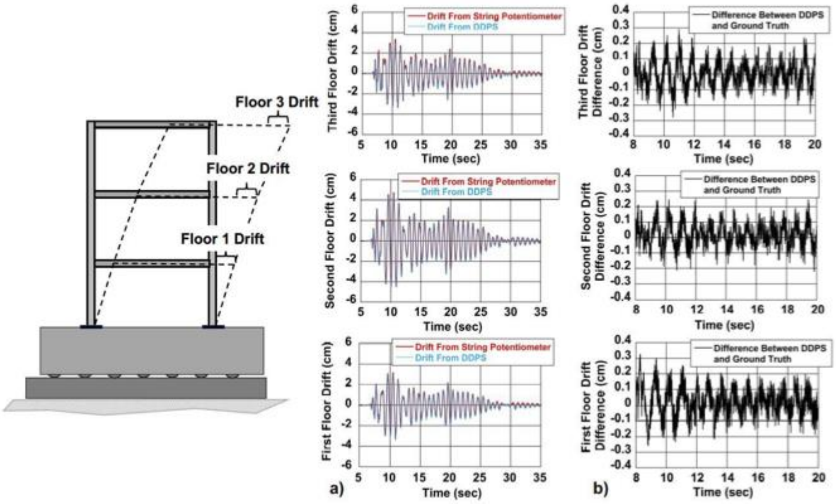


Generation 2 - an integrated sensor on a single circuit board (NSRD)

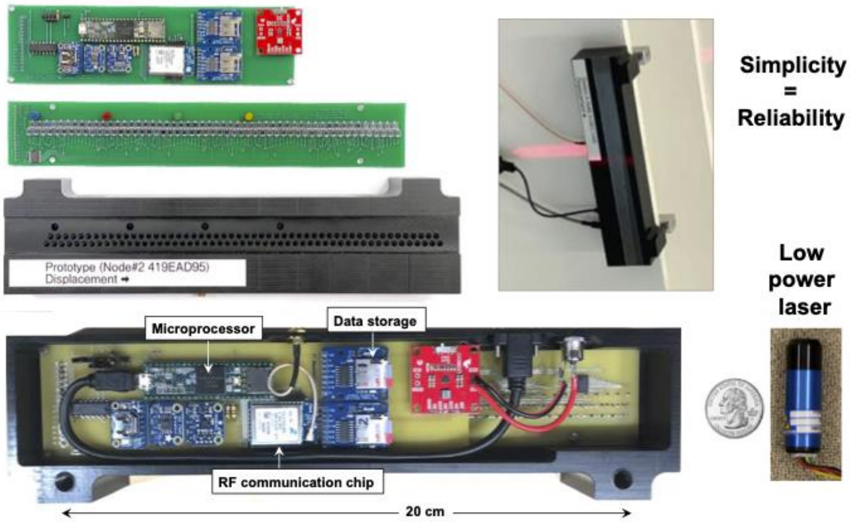


Testbed #3 - the DOE Office of Nuclear Safety supported a larger 3D test at UNR in 2017

Drift measurements - ground truth versus DDPS



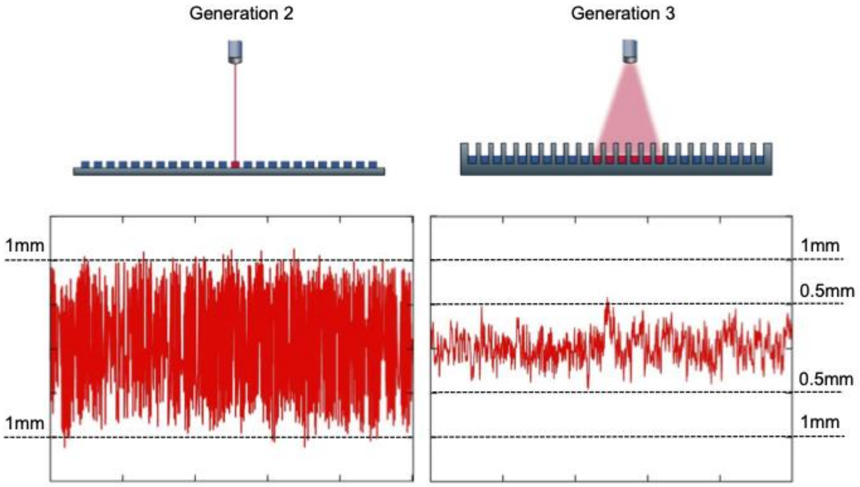
Generation 3 - a deployable sensor based on value engineering and lessons learned



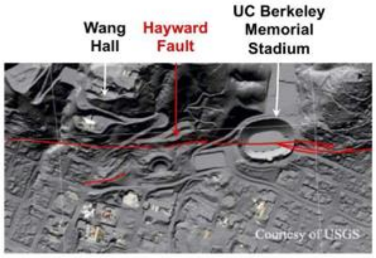
Simplicity = Reliability

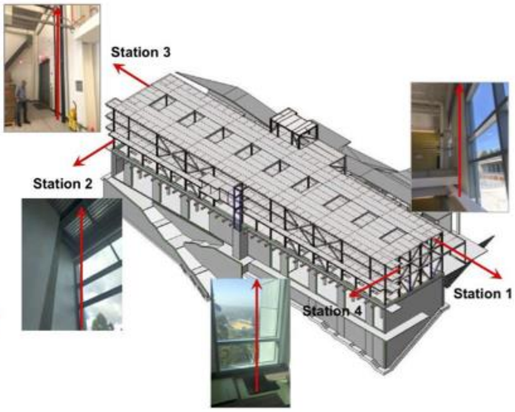
Low power laser

Discovery – a diffuse laser is much better than a sharply focused laser

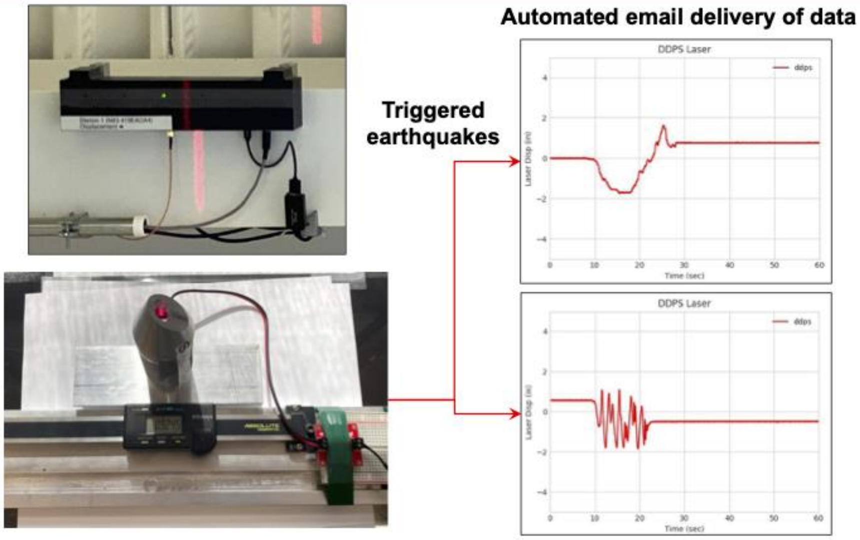


Four sensors were deployed in Wang Hall at Berkeley Lab in September 2019

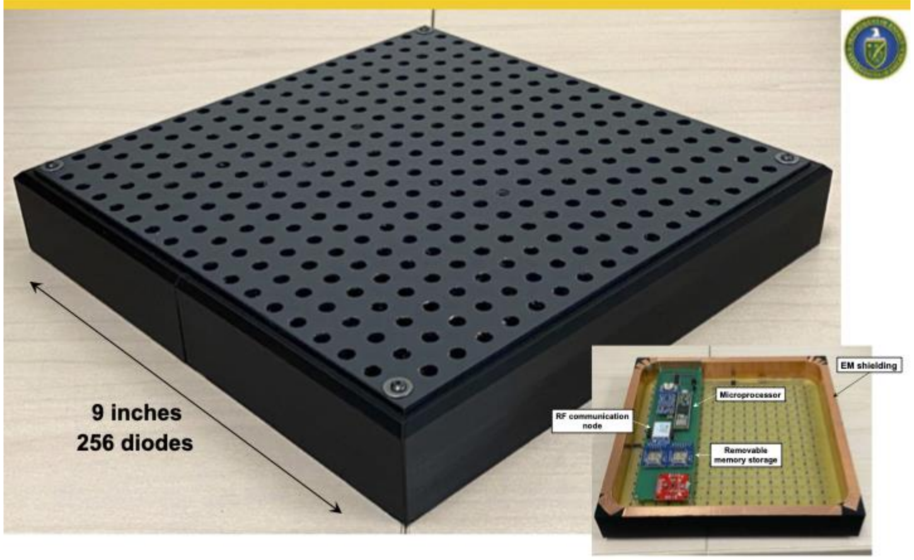




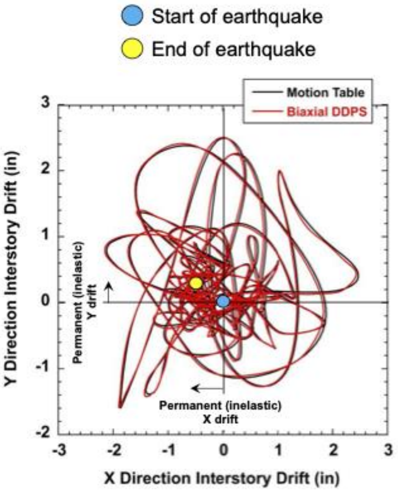
The sensors have continuously operated very reliably for 14 months

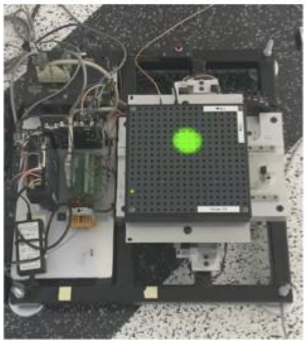


Hardware for the first prototype of a new bi-axial sensor version has been completed



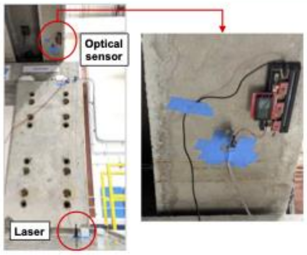
Orbit plot for 20 story building subjected to strong near-field motions

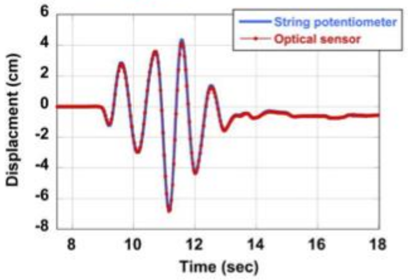




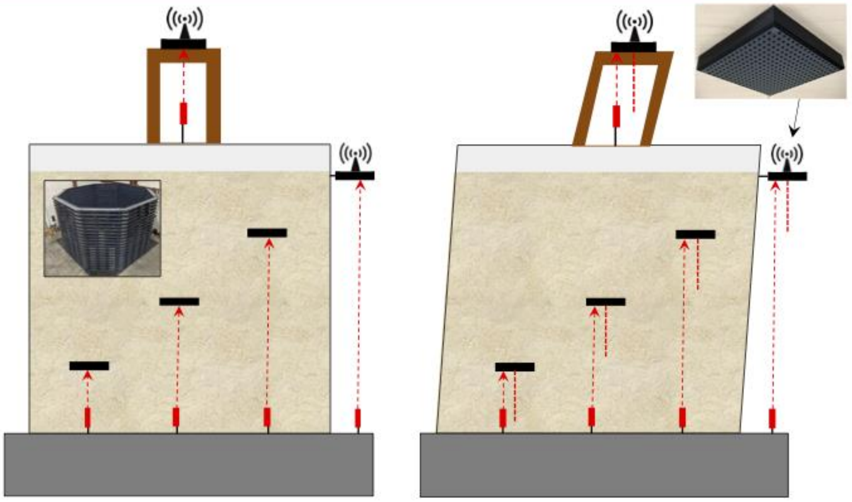
Another use case - we have tested the optical sensor concept as an agile laboratory sensor

Multi-table bridge system test

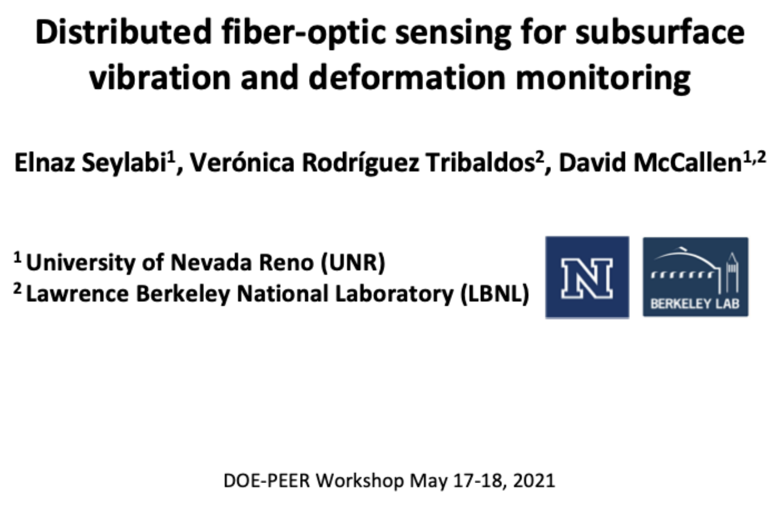




There is the potential to deploy as part of the soil box system diagnostics



DISTRIBUTED FIBER-OPTIC SENSING FOR SUBSURFACE VIBRATION AND DEFORMATION MONITORING

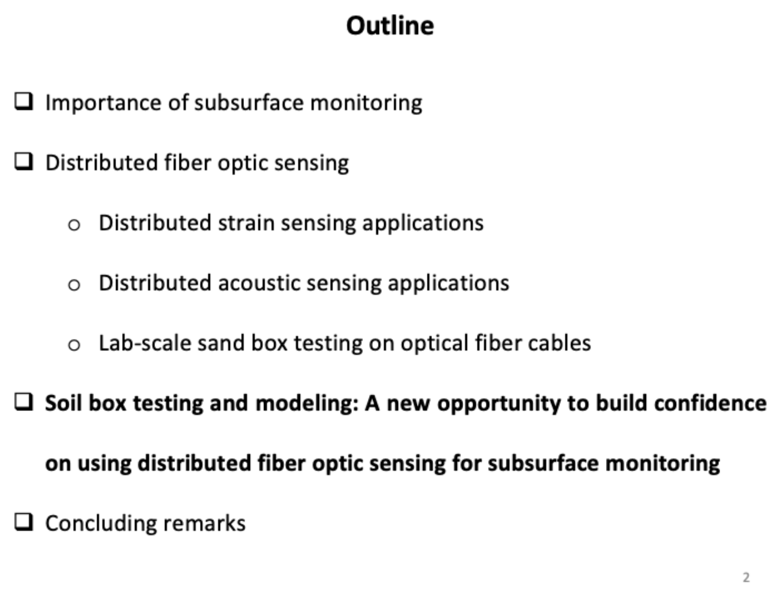


Distributed fiber-optic sensing for subsurface vibration and deformation monitoring

Elnaz Seylabi[1], Verónica Rodríguez Tribaldos[2], David McCallen[1,2]

[1] **University of Nevada Reno (UNR)**
[2] **Lawrence Berkeley National Laboratory (LBNL)**

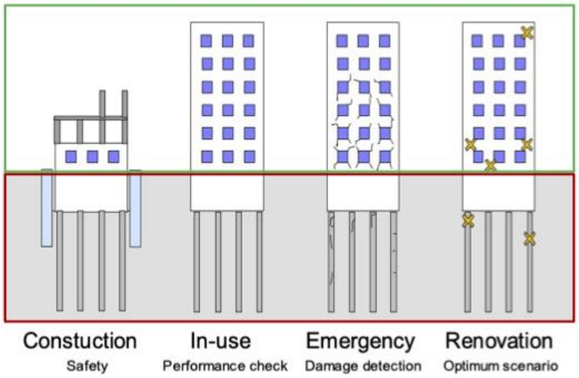
DOE-PEER Workshop May 17-18, 2021



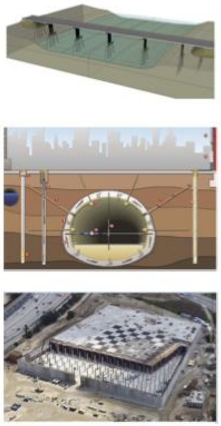
Outline

- ☐ Importance of subsurface monitoring
- ☐ Distributed fiber optic sensing
    - ○ Distributed strain sensing applications
    - ○ Distributed acoustic sensing applications
    - ○ Lab-scale sand box testing on optical fiber cables
- ☐ **Soil box testing and modeling: A new opportunity to build confidence on using distributed fiber optic sensing for subsurface monitoring**
- ☐ Concluding remarks

Subsurface Monitoring



Mita (1999)

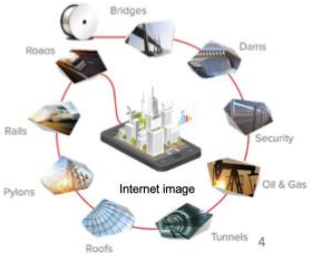


Internet images

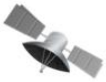
Measurement Sparsity, Inference Uncertainty, and Inaccessibility

- Sparse **point sensors** (e.g., strain gauges, accelerometers) do not provide continuous field measurement, can result in large uncertainties, and are not optimized for long-term monitoring.
- **Vision and remote sensing-based methods** cannot help much with monitoring of inaccessible/deep subsurface infrastructure and subsurface monitoring.
- **Fiber optic sensing-based methods** have been long explored as a promising sensing candidate for health monitoring of civil infrastructure.

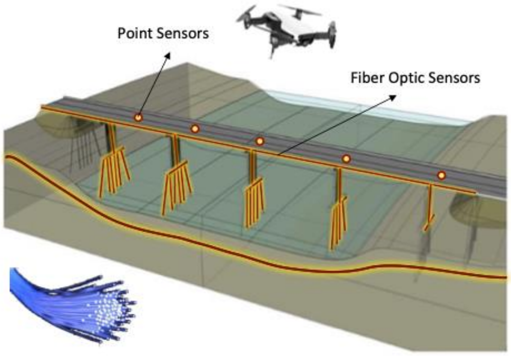
Internet image

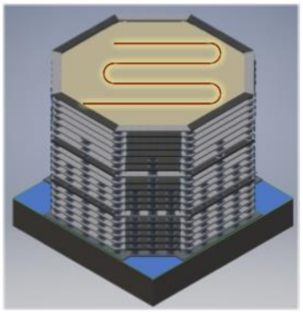


Internet image



Can distributed fiber optic sensing provide robust solutions for subsurface monitoring?

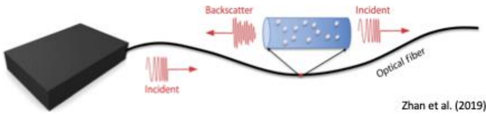




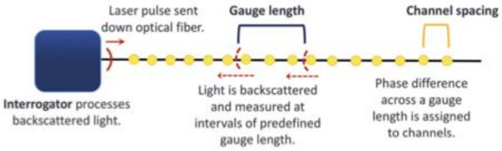
Distributed Fiber Optic Sensing

- Distributed fiber-optic sensing technologies measure **light backscattered from every point along a continuous fiber**
- Changes in the environment around the fiber (temperature, strain) will alter the scattering profile - **<u>The cable is the sensor</u>**

Distributed fiber-optic sensing converts conventional fiber-optic cables into massive arrays of strain, temperature and/or vibration sensors.



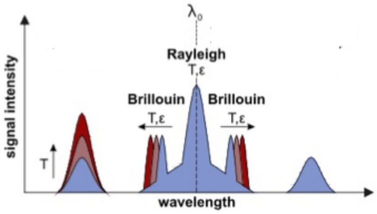
Zhan et al. (2019)



Stork et al. (2020)

Brillouin-Based and Rayleigh-Based Distributed Sensing

	Brillouin-Based Sensing	Rayleigh-Based Sensing
Measurement	Brillouin frequency shift	Rayleigh phase shift
Temporal Sampling	Typically, minutes	mHz to kHz
Sensitive to	**Strain** (and Temperature)	**Dynamic Strain** (and Temperature)
Gauge length (spatial averaging)	Typically, 1 m	Typically, 1 -10 m
Spatial Sampling	< 1 m	< 1 m

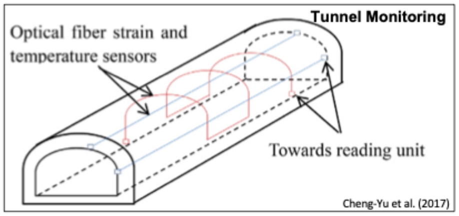


Monsberger et al. (2020)

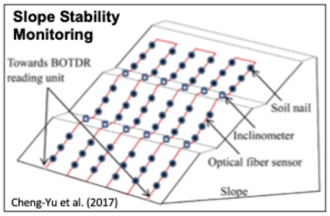
DAS can provide:

- Increased sensitivity
- Increased temporal resolution >> possibility to measure **dynamic processes**

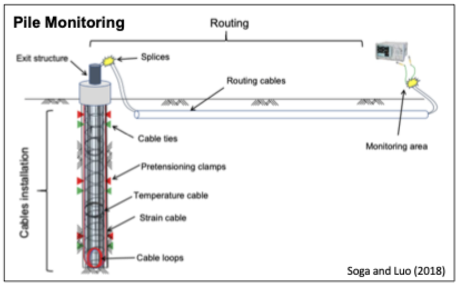
Brillouin-Based Distributed Strain Sensing Applications


Tunnel Monitoring

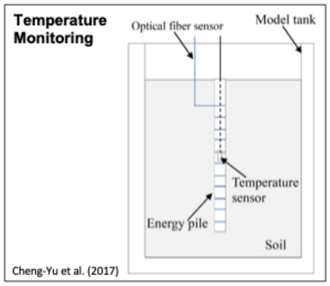
Cheng-Yu et al. (2017)


Slope Stability Monitoring

Cheng-Yu et al. (2017)

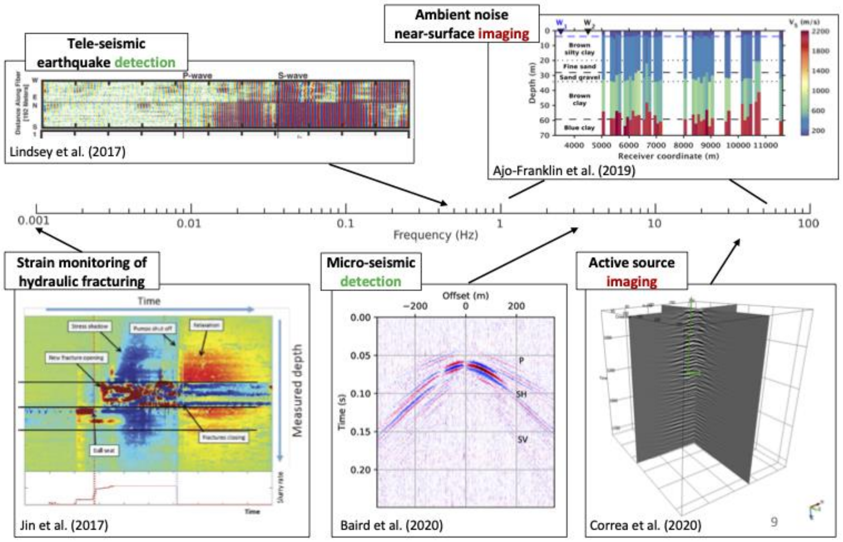

Pile Monitoring

Soga and Luo (2018)


Temperature Monitoring

Cheng-Yu et al. (2017)

Rayleigh-Based Distributed Acoustic Sensing (DAS) Applications



Tele-seismic earthquake detection

Lindsey et al. (2017)

Ambient noise near-surface imaging

Ajo-Franklin et al. (2019)

Strain monitoring of hydraulic fracturing

Jin et al. (2017)

Micro-seismic detection

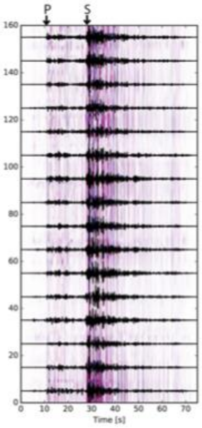
Baird et al. (2020)

Active source imaging

Correa et al. (2020)

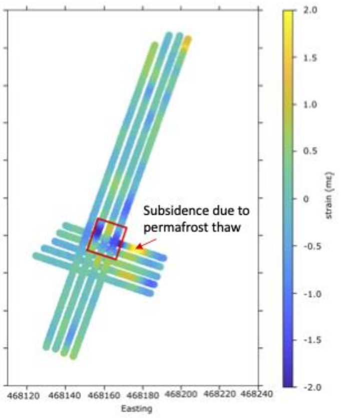
Earthquake & Deformation Monitoring Along the Same Array

Strain Rate



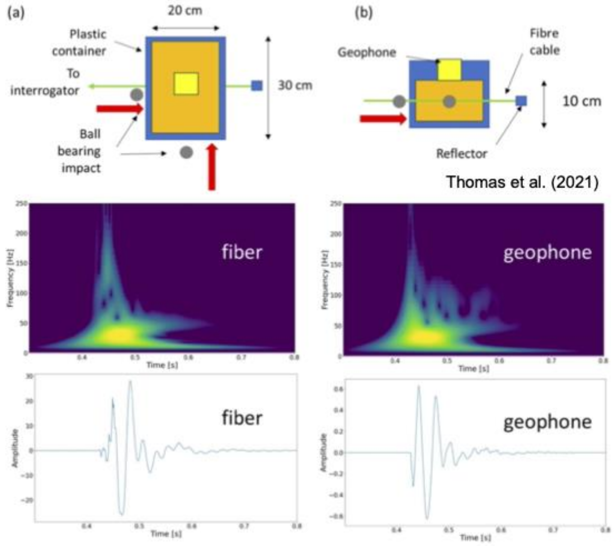
Lindsey et al. (2017)

Strain



Rodriguez Tribaldos et al. (2018)

Lab-scale Sand Box Testing on Optical Fiber Cables

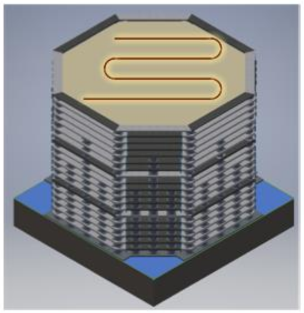


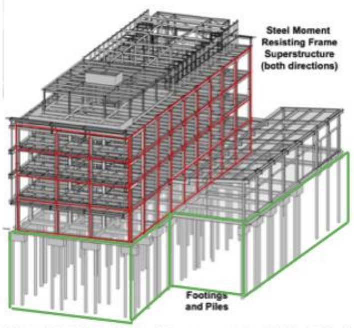
Thomas et al. (2021)

Soil Box Testing & Modeling: A New Opportunity to Build Confidence on Using Distributed Fiber Optic Sensing in Lab- and Real-scale Problems

Objective 1
Advanced Instrumentation Solutions & Lab-Scale Subsurface Monitoring

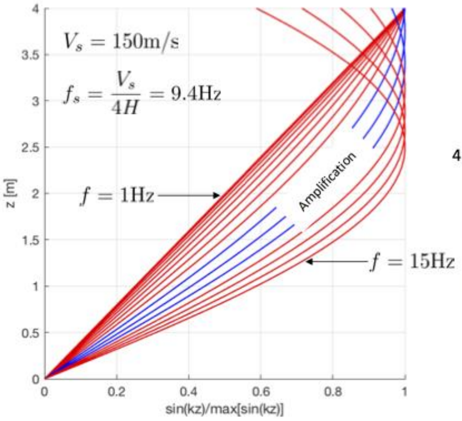
Objective 2
Real-Scale Subsurface Monitoring Solutions

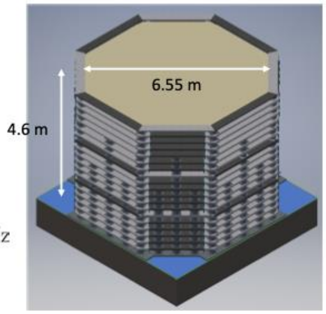




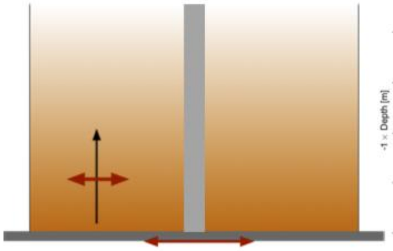
LBNL BioEPIC Building (Courtesy of David McCallen)

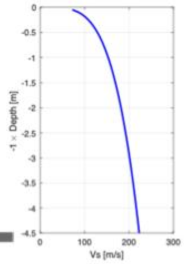
Soil Box Dynamic Characteristics

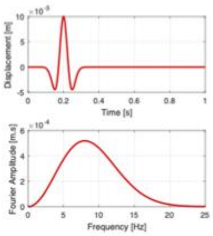




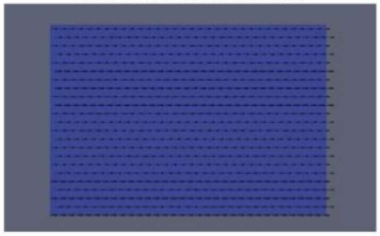
Soil Box Dynamic Characteristics (plane-strain approximation)



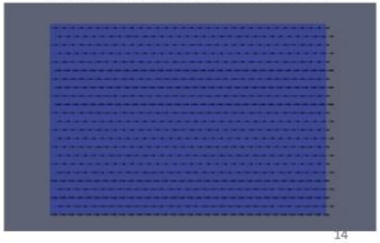




Free field response (no structure)



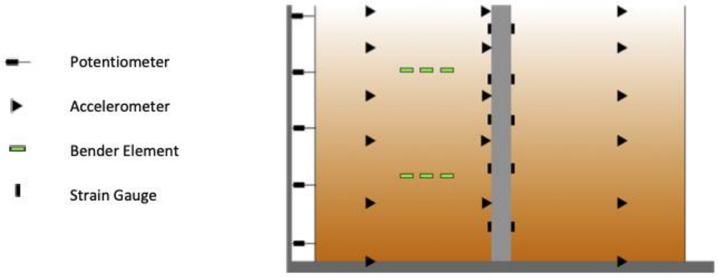
Scattered field response (with structure)



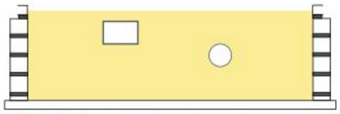
Typical Point Sensors for Lab-Scale Instrumentation of Soil-Structure Interaction Problems

Quantities of Interest:

- Structure deformation and strain
- Soil deformation and strain
- Strain dependent soil properties
- Soil-structure interface characteristics

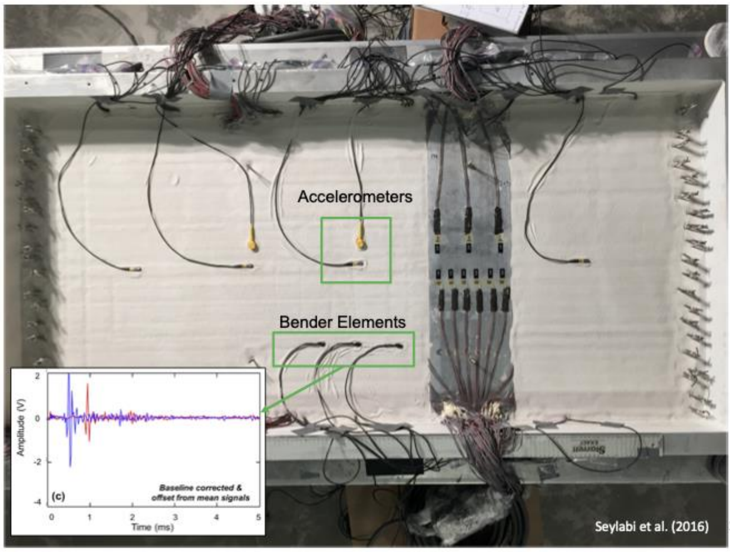


Exemplar Instrumentation (Centrifuge Testing)



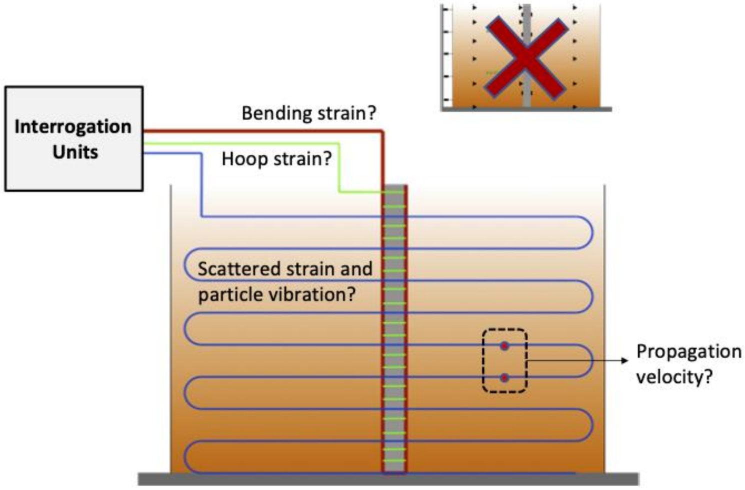
Seylabi et al. (2016)

Exemplar Instrumentation (Centrifuge Testing)



Seylabi et al. (2016)

Potential Fiber Optic Sensing Instrumentation Layout

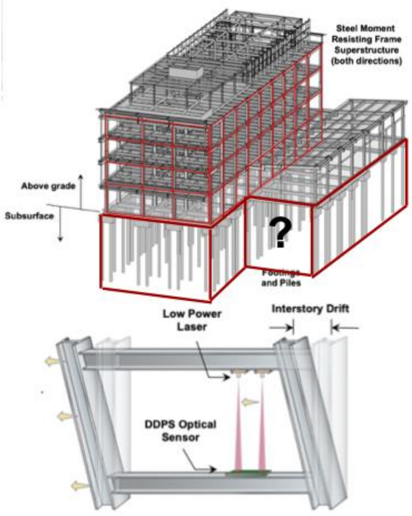


Research Questions

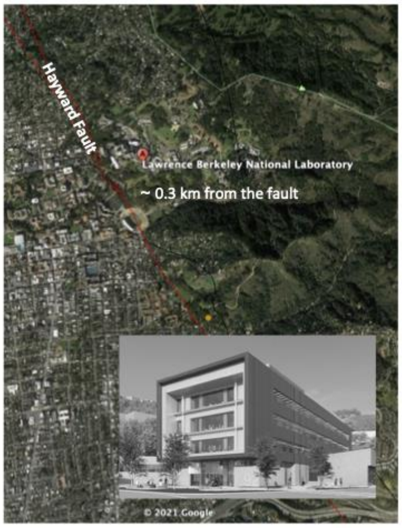
We need to perform systematic numerical and experimental feasibility studies to answer the following research questions:

- Spatial & temporal resolutions in different environments
- Possible designs to resolve different components of the strain tensor
- Consistency with point sensor measurements
- Soil-cable-structure coupling effects and strain transfer mechanisms
- Embedment, densification, and soil nonlinearity effects
- Integrity and resistance to large deformations and damages
- Required numerical modeling techniques
- Real-scale deployment challenges and opportunities for long-term/real-time monitoring

Concluding Remarks

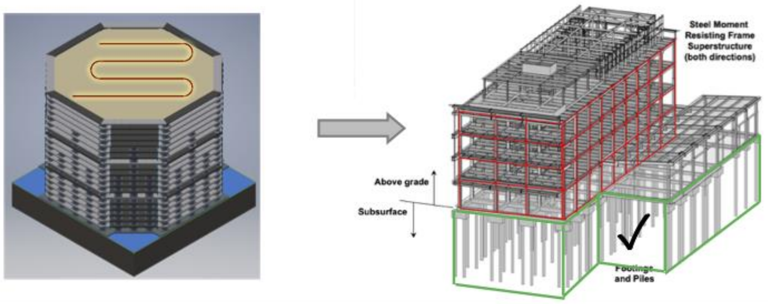


Courtesy of David McCallen



Concluding Remarks

Towards robust solutions for subsurface vibration and deformation monitoring



Thank you for your attention!

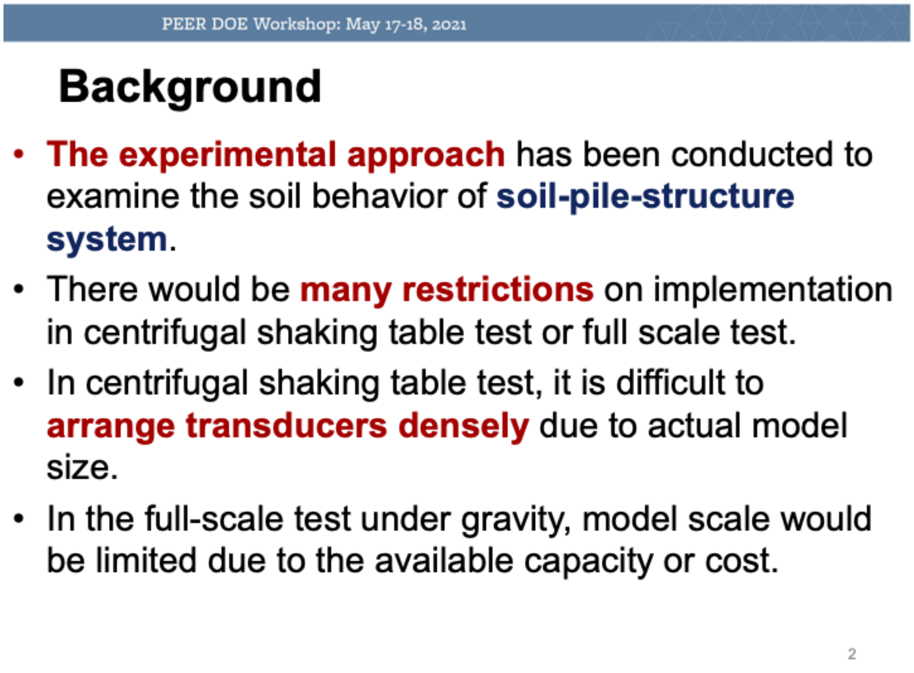
3D EFFECTIVE STRESS ANALYSIS ON SOIL BEHAVIOR AROUND CLOSELY SPACED PILE GROUP IN CENTRIFUGAL SHAKING TABLE TESTS

DOE PEER Workshop
Tuesday May 18, 2021
7:30am – 2:30pm Pacific time

A virtual workshop, funded by the U.S. Department of Energy (DOE) and PEER

3D Effective Stress Analysis on Soil Behavior Around Closely Spaced Pile Group in Centrifugal Shaking Table Tests

○ Yuta Nakagama	Tokyo Electric Power Services Co., Ltd. Tokyo, Japan.
Yukio Tamari	Ditto.
Hiroyuki Yoshida	Ditto.
Minoru Yamamoto	Ditto.
Hiroko Suzuki	Chiba institute of technology. Chiba, Japan.



Background

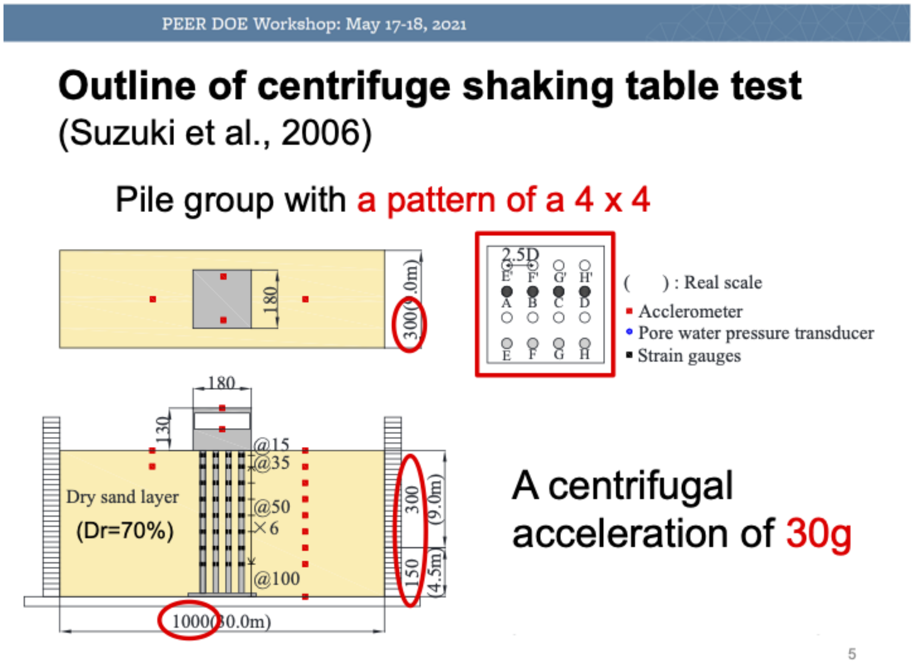
- **The experimental approach** has been conducted to examine the soil behavior of **soil-pile-structure system**.
- There would be **many restrictions** on implementation in centrifugal shaking table test or full scale test.
- In centrifugal shaking table test, it is difficult to **arrange transducers densely** due to actual model size.
- In the full-scale test under gravity, model scale would be limited due to the available capacity or cost.

Objective

- Examine the **soil behavior in pile groups**.
- **Consider the presumed mechanism** from the numerical analysis results.

- Numerical analysis on the already implemented shaking table test of soil-pile-structure system (Suzuki et al., 2006)
- 3D effective stress finite element method

Shaking table tests

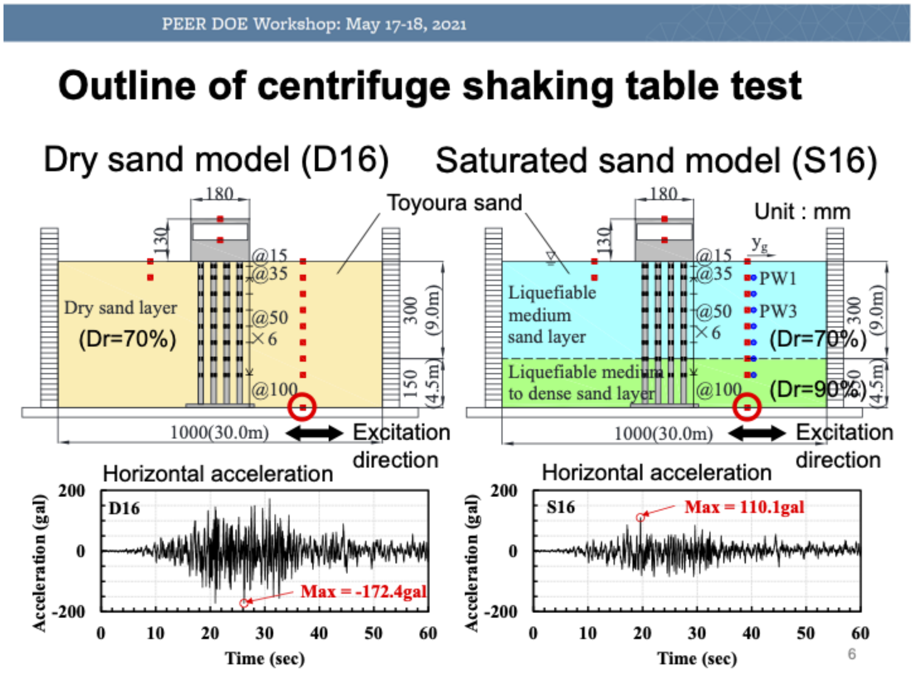


Outline of centrifuge shaking table test

(Suzuki et al., 2006)

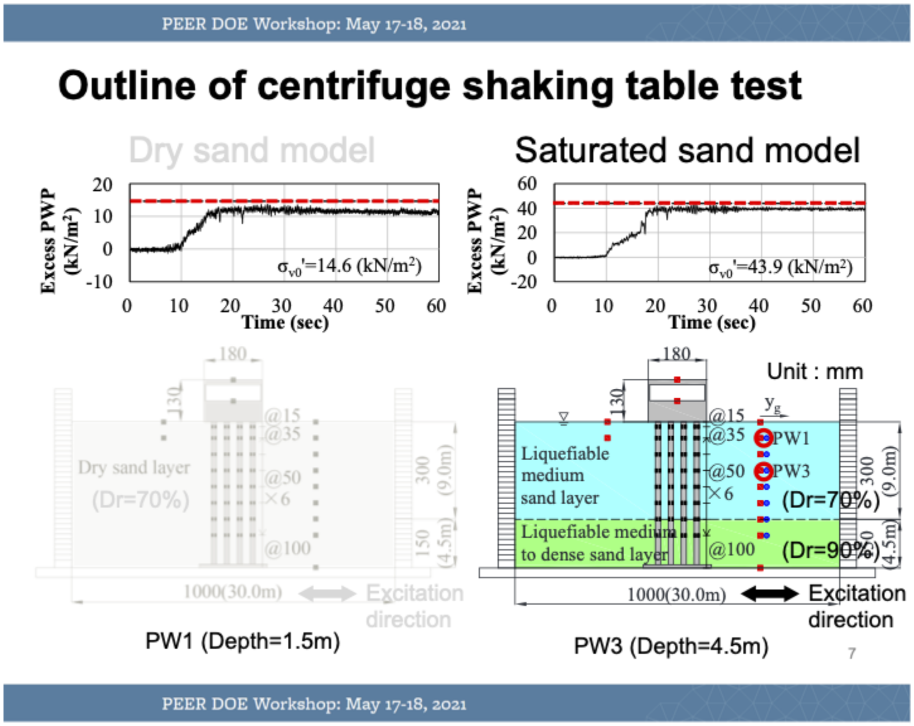
Pile group with a pattern of a 4 x 4

A centrifugal acceleration of 30g



Outline of centrifuge shaking table test

Dry sand model (D16) Saturated sand model (S16)



Outline of centrifuge shaking table test

Dry sand model

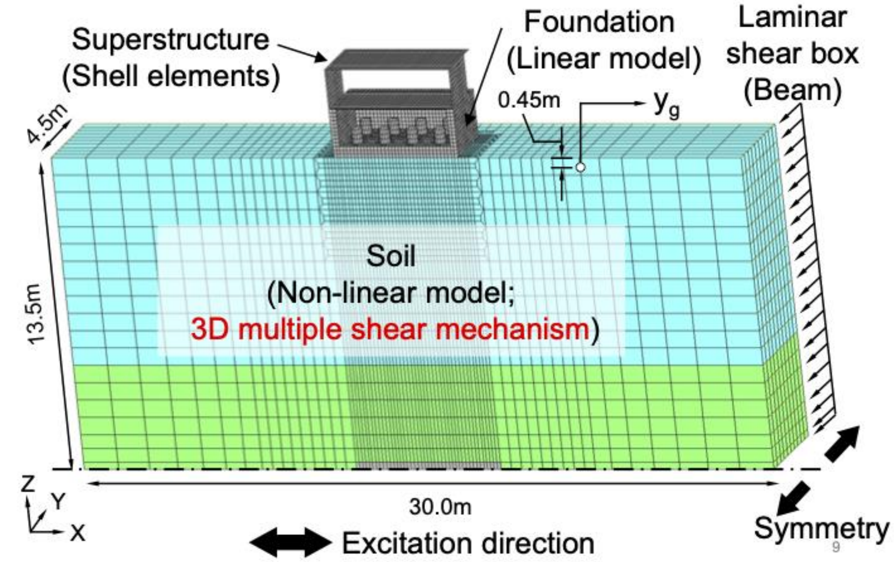
Saturated sand model

PW1 (Depth=1.5m)

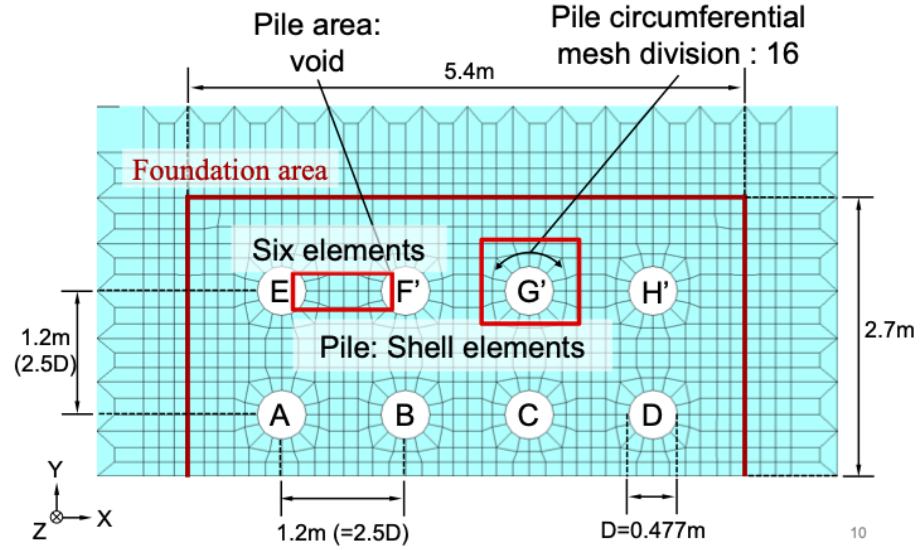
PW3 (Depth=4.5m)

3D finite element model

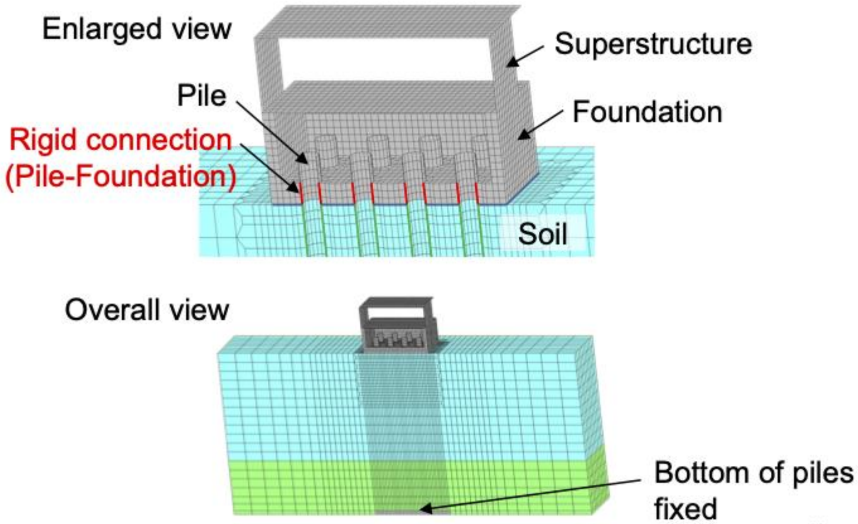
Overall view of Finite Element Model



Finite Element Model around pile group



Restraint conditions of Finite Element Model



Finite Element Method

- Finite element analysis by **FLIP ROSE 3D** ver.1.6 (Iai., 1993).
- Consider a **liquefaction process** and response of **soil-structure system** during earthquakes.
- We implemented a multifrontal massively parallel sparse direct solver (Amestoy et al., 2006) in FLIP ROSE 3D.

Parameters for physical and dynamic deformation characteristics of soils

Layer	ρ (t/m^3)	n	G_{ma} (kPa)	$-\sigma_{ma}'$ (kPa)	φ_f (deg)	v	h_{max}
Dry sand layer	1.57	0.407	59600	29.4	40.8	0.33	0.28
Liquefiable medium sand layer	1.95	0.407	59600	29.4	40.8	0.33	0.28
Liquefiable medium to dense sand layer	2.00	0.377	110300	74.3	44.3	0.33	0.28

ρ : density , n : void ratio

G_{ma} : elastic shear modulus at a confining pressure of ($-\sigma_{ma}'$)
($=8400 \cdot (2.17-e)^2/(1+e) \cdot (\sigma_{ma}')^{0.5}$, Ishihara,. 1996.)

$-\sigma_{ma}'$: reference confining pressure ($=\sigma_v' \cdot (1+2K_0)/3$)

φ_f : shear resistance angle ($\tan\varphi_f=(0.7095e_{min}+0.163)/e$, Moroto,. 1983)

v : poisson's ratio, h_{max} : maximum damping ratio

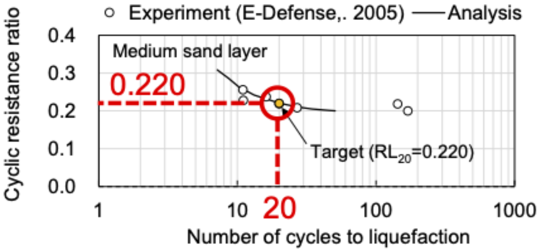
Parameters for liquefaction characteristics

Layer	φ_p (deg)	S_1	w_1	p_1	p_2	c_1	RL_{20}
Liquefiable medium sand layer	28.0	0.005	2.24	0.70	0.80	2.01	0.220
Liquefiable medium to dense sand layer	28.0	0.005	1.38	0.70	0.80	1.93	0.220

φ_p : phase transformation angle, S_1: ultimate limit, w_1: overall of dilatancy

p_1: initial phase, p_2: final phase, c_1: threshold limit of dilatancy

RL_{20}: the cyclic shear strength ratio to 20 cycles (=0.220, E-Defense,. 2005)



Computation conditions

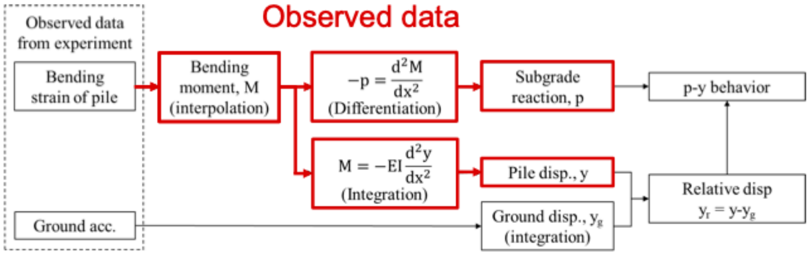
- Undrained conditions
- Wilson-θ method (θ=1.4) for time integration
- Time step of 0.01 seconds
- Rayleigh damping is estimated to be equivalent to damping ratio of primary natural period.

Part	T_G (sec)	h (%)	Rayleigh damping	
			α	β
Soil	0.270	1.0	0.0	0.001
Pile, foundations and Superstructure	0.300	2.0	0.0	0.002

T_G: Primary natural period, h: Initial damping ratio,
β: stiffness proportional Rayleigh damping coefficient (=h・TG/π)

Evaluation method of p-y behavior

Evaluation method of p-y behavior



Observed data

Observed data from experiment

Bending strain of pile

Ground acc.

Bending moment, M (interpolation)

$-p = \frac{d^2M}{dx^2}$ (Differentiation)

$M = -EI\frac{d^2y}{dx^2}$ (Integration)

Subgrade reaction, p

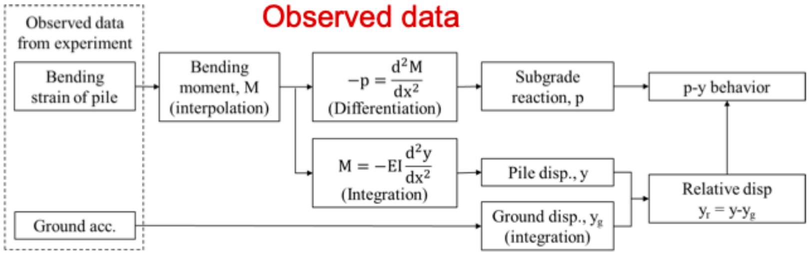
Pile disp., y

Ground disp., y_g (integration)

p-y behavior

Relative disp $y_r = y\text{-}y_g$

Evaluation method of p-y behavior



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p-y behavior

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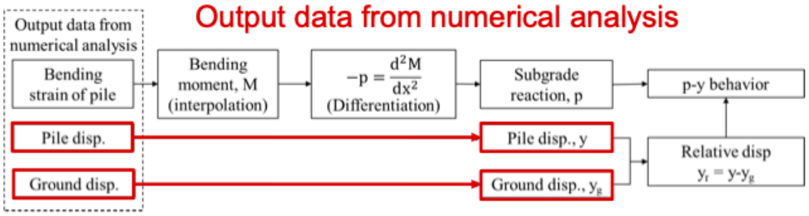


Output data from numerical analysis

Output data from numerical analysis

Bending strain of pile

Pile disp.

Ground disp.

Bending moment, M (interpolation)

$-p = \frac{d^2M}{dx^2}$ (Differentiation)

Subgrade reaction, p

Pile disp., y

Ground disp., y_g

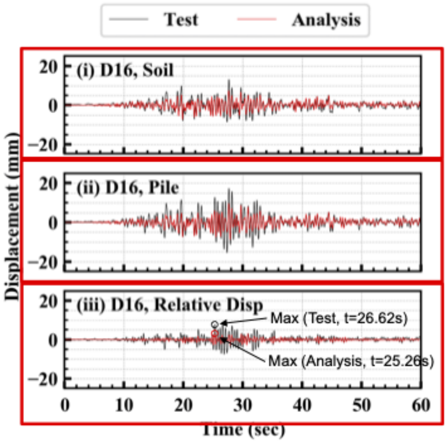
p-y behavior

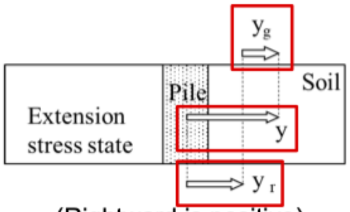
Relative disp $y_r = y\text{-}y_g$

Typical analysis results

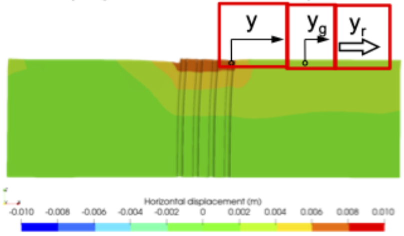
Dynamic response of pile and soil

Dry sand model

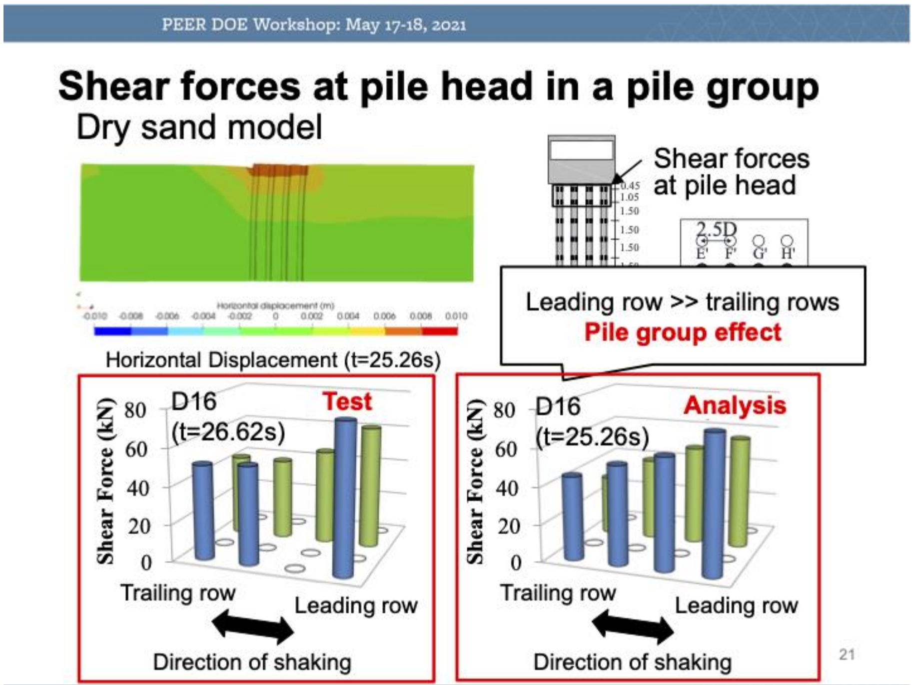




(Rightward is positive)



Horizontal Displacement (t=25.26s)



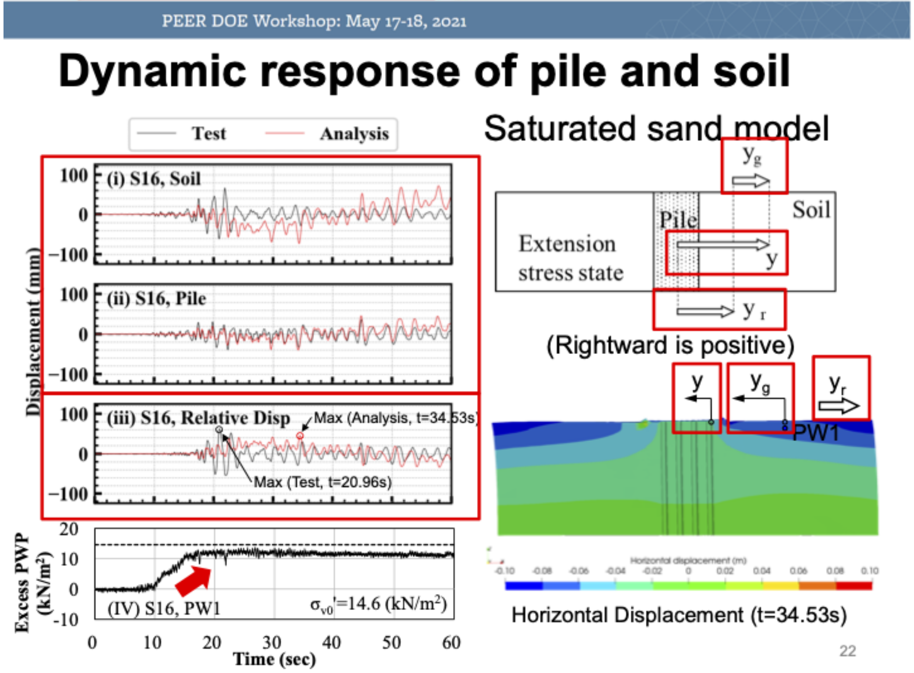
Shear forces at pile head in a pile group

Dry sand model

Horizontal Displacement (t=25.26s)

Shear forces at pile head

Leading row >> trailing rows
Pile group effect

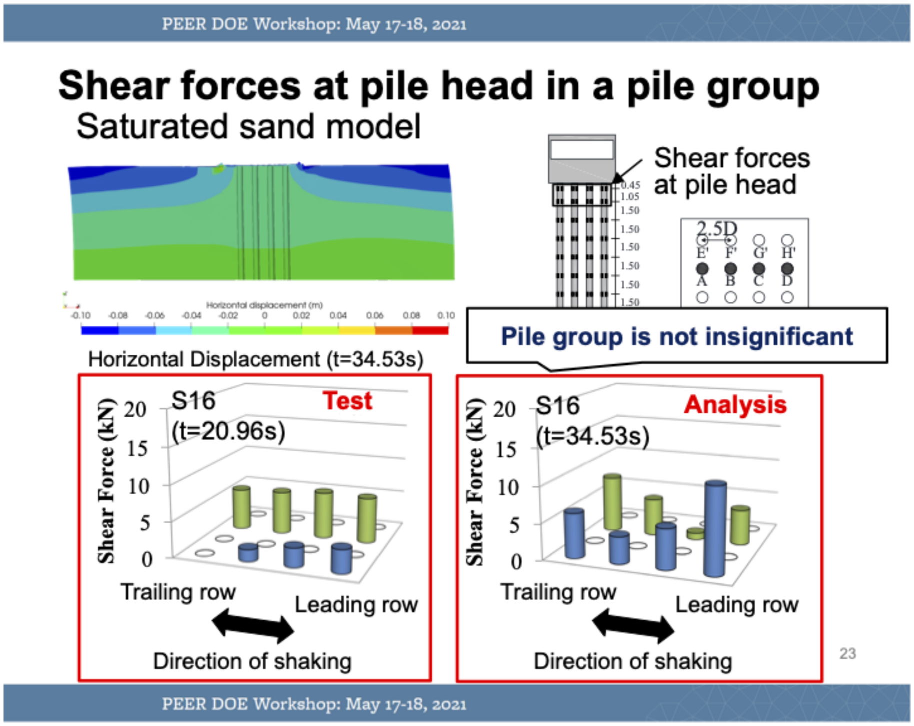


Dynamic response of pile and soil

Saturated sand model

(Rightward is positive)

Horizontal Displacement (t=34.53s)



Shear forces at pile head in a pile group

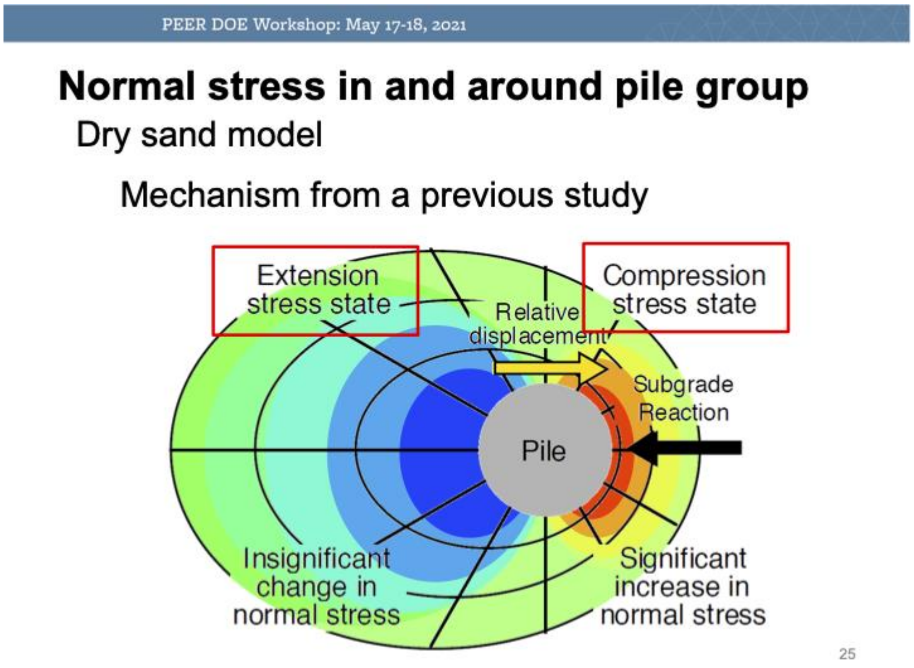
Saturated sand model

Horizontal Displacement (t=34.53s)

Shear forces at pile head

Pile group is not insignificant

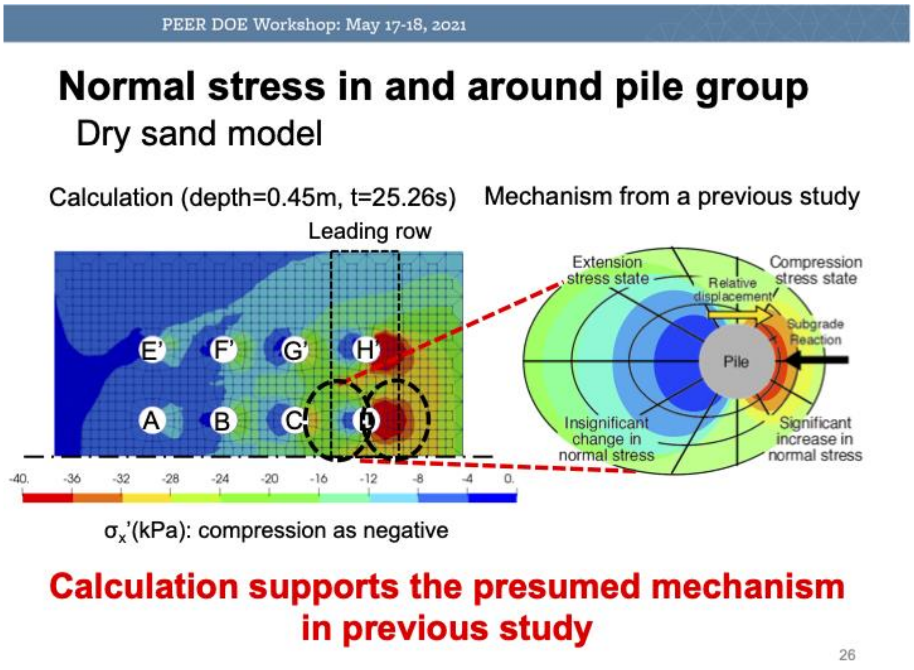
Soil behavior in a pile group



Normal stress in and around pile group

Dry sand model

Mechanism from a previous study



Normal stress in and around pile group

Dry sand model

Calculation (depth=0.45m, t=25.26s) Mechanism from a previous study

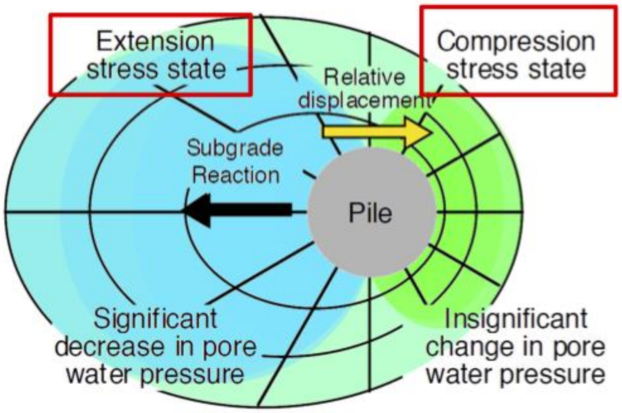
σ_x'(kPa): compression as negative

Calculation supports the presumed mechanism in previous study

Normal stress in and around pile group

Saturated sand model

Mechanism from a previous study



Extension stress state

Compression stress state

Relative displacement

Subgrade Reaction

Pile

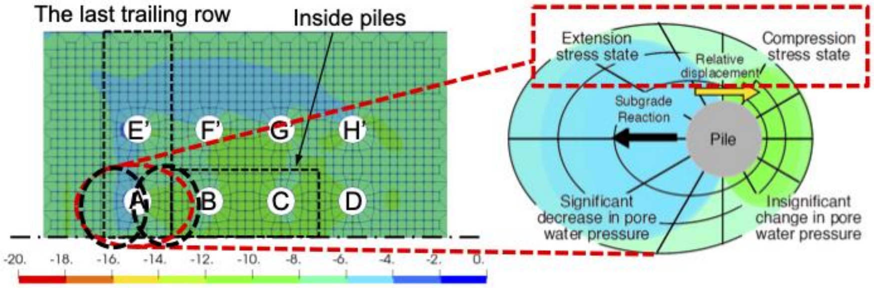
Significant decrease in pore water pressure

Insignificant change in pore water pressure

Normal stress in and around pile group

Saturated sand model

Calculation (depth=0.45m, t=34.53s) Mechanism from a previous study

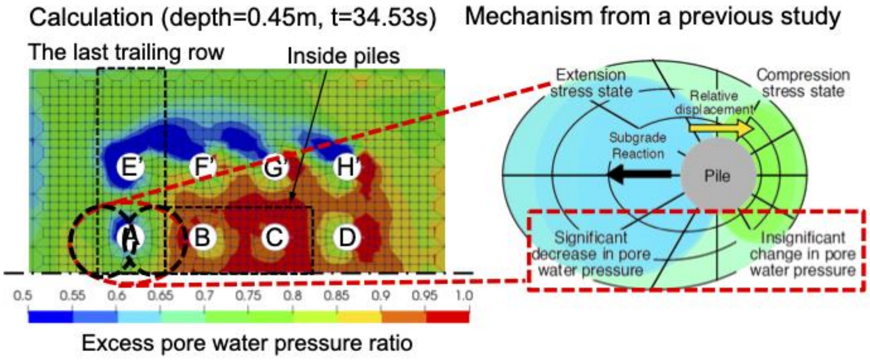


$\sigma_x(=\sigma_x'-P_w)$ (kPa) : compression as negative

Stress state is similar to the mechanism in previous study

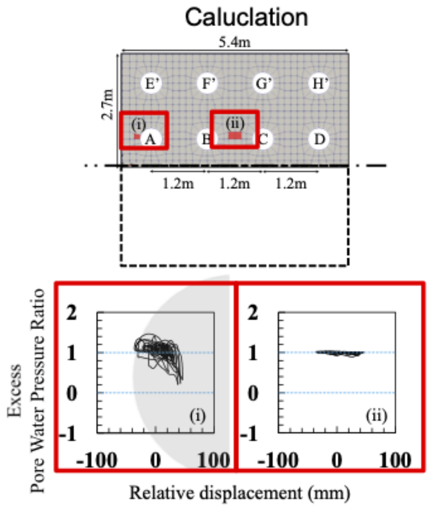
Pore water pressure in and around pile group

Saturated sand model

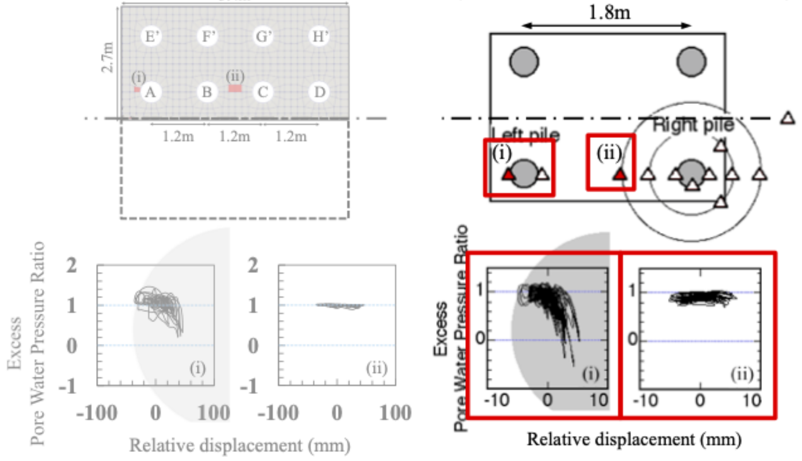


Pore water pressure state is similar to the mechanism in previous study

Hysteresis of excess pore water pressure ratio and relative displacement



Hysteresis of excess pore water pressure ratio and relative displacement



Caluclation

Previous study (Tokimatsu and Suzuki., 2004)

Conclusion

- Pile group effect was successfully simulated in dry sand model.
- The behavior of horizontal normal stress was consistent with the previous study by Suzuki et al. (2006).
- Pile group effect in saturated sand model was not as remarkable as that in dry sand model, being consistent with the previous findings.
- The calculated behavior of pore water pressure and pile relative displacement was qualitatively consistent with previous study (Tokimatsu and Suzuki., 2004).

Thank you very much for your kind attention.

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