PACIFIC EARTHQUAKE ENGINEERING RESEARCH CENTER

U.S.–New Zealand–Japan International Workshop

Liquefaction-Induced Ground Movement Effects
University of California
Berkeley, California
2–4 November 2016

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Pacific Earthquake Engineering Research Center
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ABSTRACT

There is much to learn from the recent New Zealand and Japan earthquakes. These earthquakes produced differing levels of liquefaction-induced ground movements that damaged buildings, bridges, and buried utilities. Along with the often spectacular observations of infrastructure damage, there were many cases where well-built facilities located in areas of liquefaction-induced ground failure were not damaged. Researchers are working on characterizing and learning from these observations of both poor and good performance.

The “Liquefaction-Induced Ground Movements Effects” workshop provided an opportunity to take advantage of recent research investments following these earthquake events to develop a path forward for an integrated understanding of how infrastructure performs with various levels of liquefaction. Fifty-five researchers in the field, two-thirds from the U.S. and one-third from New Zealand and Japan, convened in Berkeley, California, in November 2016. The objective of the workshop was to identify research thrusts offering the greatest potential for advancing our capabilities for understanding, evaluating, and mitigating the effects of liquefaction-induced ground movements on structures and lifelines. The workshop also advanced the development of younger researchers by identifying promising research opportunities and approaches, and promoting future collaborations among participants.

During the workshop, participants identified five cross-cutting research priorities that need to be addressed to advance our scientific understanding of and engineering procedures for soil liquefaction effects during earthquakes. Accordingly, this report was organized to address five research themes: (1) case history data; (2) integrated site characterization; (3) numerical analysis; (4) challenging soils; and (5) effects and mitigation of liquefaction in the built environment and communities. These research themes provide an integrated approach toward transformative advances in addressing liquefaction hazards worldwide.

The archival documentation of liquefaction case history datasets in electronic data repositories for use by the broader research community is critical to accelerating advances in liquefaction research. Many of the available liquefaction case history datasets are not fully documented, published, or shared. Developing and sharing well-documented liquefaction datasets reflect significant research efforts. Therefore, datasets should be published with a permanent DOI, with appropriate citation language for proper acknowledgment in publications that use the data.

Integrated site characterization procedures that incorporate qualitative geologic information about the soil deposits at a site and the quantitative information from *in situ* and laboratory engineering tests of these soils are essential for quantifying and minimizing the uncertainties associated site characterization. Such information is vitally important to help identify potential failure modes and guide *in situ* testing. At the site scale, one potential way to do this is to use proxies for depositional environments. At the fabric and microstructure scale, the use of multiple *in situ* tests that induce different levels of strain should be used to characterize soil properties. The development of new *in situ* testing tools and methods that are more sensitive to soil fabric and microstructure should be continued.

The development of robust, validated analytical procedures for evaluating the effects of liquefaction on civil infrastructure persists as a critical research topic. Robust validated analytical procedures would translate into more reliable evaluations of critical civil infrastructure

performance, support the development of mechanics-based, practice-oriented engineering models, help eliminate suspected biases in our current engineering practices, and facilitate greater integration with structural, hydraulic, and wind engineering analysis capabilities for addressing multi-hazard problems. Effective collaboration across countries and disciplines is essential for developing analytical procedures that are robust across the full spectrum of geologic, infrastructure, and natural hazard loading conditions encountered in practice.

There are soils that are challenging to characterize, to model, and to evaluate, because their responses differ significantly from those of clean sands: they cannot be sampled and tested effectively using existing procedures, their properties cannot be estimated confidently using existing *in situ* testing methods, or constitutive models to describe their responses have not yet been developed or validated. Challenging soils include but are not limited to: interbedded soil deposits, intermediate (silty) soils, mine tailings, gravelly soils, crushable soils, aged soils, and cemented soils. New field and laboratory test procedures are required to characterize the responses of these materials to earthquake loadings, physical experiments are required to explore mechanisms, and new soil constitutive models tailored to describe the behavior of such soils are required. Well-documented case histories involving challenging soils where both the poor and good performance of engineered systems are documented are also of high priority.

Characterizing and mitigating the effects of liquefaction on the built environment requires understanding its components and interactions as a system, including residential housing, commercial and industrial buildings, public buildings and facilities, and spatially distributed infrastructure, such as electric power, gas and liquid fuel, telecommunication, transportation, water supply, wastewater conveyance/treatment, and flood protection systems. Research to improve the characterization and mitigation of liquefaction effects on the built environment is essential for achieving resiliency. For example, the complex mechanisms of ground deformation caused by liquefaction and building response need to be clarified and the potential bias and dispersion in practice-oriented procedures for quantifying building response to liquefaction need to be quantified. Component-focused and system performance research on lifeline response to liquefaction is required. Research on component behavior can be advanced by numerical simulations in combination with centrifuge and large-scale soil–structure interaction testing. System response requires advanced network analysis that accounts for the propagation of uncertainty in assessing the effects of liquefaction on large, geographically distributed systems. Lastly, research on liquefaction mitigation strategies, including aspects of ground improvement, structural modification, system health monitoring, and rapid recovery planning, is needed to identify the most effective, cost efficient, and sustainable measures to improve the response and resiliency of the built environment.

ACKNOWLEDGMENTS

Research supported by the U.S. National Science Foundation (NSF) under award number CMMI-1640922 is gratefully acknowledged. However, any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect the views of the NSF.

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The assistance of Grace Kang and Yolanda West of PEER in coordinating planning and logistics for the workshop is appreciated. Jorge Macedo of the University of California, Berkeley, provided logistical support as well.

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1 Introduction

1.1 OVERVIEW

There is much to learn from the recent earthquakes in New Zealand (NZ) and Japan. The 2010–2011 Canterbury Earthquake Sequence and the 2011 Tohoku, Japan earthquake and its aftershocks produced differing levels of liquefaction-induced ground movements that damaged buildings, bridges, and buried utilities. Along with the often spectacular observations of infrastructure damage, there were also many cases where well-built facilities, which were located in areas of liquefaction-induced ground failure, were not damaged. There are numerous cases where current procedures indicated that liquefaction should have occurred, and it did occur. However, there are also numerous cases where current procedures indicated that liquefaction should have occurred, but its occurrence was not evident. Researchers are working on characterizing and learning from these observations of both poor and good performance of the ground, and the engineered facilities built in and atop of the ground. This workshop provided an unprecedented opportunity to take advantage of recent research investments following these events to develop a path forward for an integrated understanding of how structures with differing foundation systems perform with various levels of liquefaction. Efforts will focus on both poor and good performance of engineered facilities to advance performance-based earthquake engineering (PBEE) design procedures.

The objective of the workshop was to identify the empirical, numerical, and analytical methods that hold the greatest potential for advancing insight on the effects of liquefaction-induced ground movements on structures and systems; therefore, they should be considered high priority for further research. The workshop also advanced the development of younger researchers through identifying research approaches that appear to be promising, as well as to promote future collaborations among participants.

The international workshop was held on 2–4 November 2016. Fifty-five leading researchers in the field, two-thirds from the United States (U.S.) and one-third from New Zealand and Japan, convened in Berkeley, California. The workshop objectives were met through a series of activities prior to, during, and after the workshop, including development and distribution of a workshop bibliography that includes recent publications, collection of extended abstracts (one submitted by each participant) outlining the primary issues that need to be addressed to advance understanding of the effects of soil liquefaction, a mix of working group and full workshop discussions, and delivery of workshop outcomes in this report.

The U.S. National Science Foundation (NSF) was the primary sponsor of the workshop. Additional support was provided by the Pacific Earthquake Engineering Research (PEER) Center and the Geotechnical Extreme Events Reconnaissance (GEER) Association. Funding to support participation of New Zealand researchers was provided by the New Zealand Earthquake Commission (EQC), and support for Japanese researchers was provided by their respective organizations.

1.2 NATIONAL ACADEMIES REPORT

The National Academies of Sciences, Engineering, and Medicine recent report on the “State of the Art and Practice in Earthquake Induced Soil Liquefaction” was prepared to examine technical issues regarding liquefaction hazard evaluation and consequence assessment. The Academies’ report was published in late December 2016. Although the contents of the report were not known at the time of the workshop, they were known during the post-workshop report writing effort. The workshop report was written to complement the National Academies Report [2016] by leveraging the work completed by the Academies’ study while avoiding significant duplication.

1.3 WORKSHOP OBJECTIVES

The objectives of this workshop were to identify key challenges and issues related to, describe critical geologic processes and the underlying mechanisms involved in, and develop research approaches to overcome the challenges that remain in understanding, assessing, and mitigating the effects of soil liquefaction. They were grouped into three categories:

I. The effects on structures and lifelines of liquefaction-induced flow slides that are governed by the residual shear strength of liquefied soil;

II. The effects of liquefaction-induced lateral spreading on structures and lifelines; and

III. The effects of liquefaction-induced settlement on structures and lifelines.

With the assumption that cyclic mobility, involving limited to large levels of strain, had developed, workshop participants focused on the manifestations of liquefaction/cyclic softening/cyclic failure and their effects on structures and lifelines. Thus, workshop participants moved beyond issues that have received much attention over the last few decades to issues that have not been explored in the same degree of detail. The effects of soil liquefaction matter most to engineers, city planners, architects, and the public.

Stemming from the workshop objectives stated previously, the workshop organizers facilitated discussions with the aim of identifying empirical, numerical, and analytical methods that hold the greatest potential for advancing insight on the effects of liquefaction-induced ground movements on structures; therefore, they should be considered high priority for further research. This also advanced the development of younger researchers through identifying research approaches and opportunities that appear to be promising, as well as to promote future collaborations among participants.

1.4 WORKSHOP PLAN

The selected "*liquefaction effects*" challenge was addressed by each workshop participant responding to one or more of these prompts:

1. What is the current state-of-the-art for evaluating this problem today?
2. What are the key underlying geologic processes that affect it?
3. What are the primary mechanisms involved in the phenomenon?
4. What are the key challenges to developing better evaluation procedures?
5. What is the best path forward for advancing understanding and procedures to address it?

Speakers were able to choose whether to focus on only one or two of the prompts that they thought were most important. However, in totality, the presentations and discussions focused on addressing all five prompts for each of the three "*liquefaction effects*" challenges. Extended abstracts by workshop participants are provided in Appendix A. Each participant also identified pertinent literature, which are listed in Appendix B.

Focused interactive discussion periods were a key part of this workshop. Significant time was scheduled to discuss the key research thrusts for each "*liquefaction effects*" challenge and to identify the most promising paths forward toward assessing the effects of liquefaction on structures and lifelines.

After the workshop, workshop organizers and a few selected participants, who expanded the group's perspectives, remained in Berkeley for an additional day to distill and synthesize the results of the workshop into this workshop report.

1.5 WORKSHOP ORGANIZATION

1.5.1 Workshop Organizing Committee

The workshop organizing committee includes faculty members from the University of California, Berkeley, and the University of California, Davis, in the U.S. as well as from the University of Canterbury in New Zealand and the Tokyo Institute of Technology in Japan. The lead workshop organizers are delineated below:

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1.5.2 Participants

Fifty-five people participated in the workshop. Invited workshop participants included leading researchers in the area of soil liquefaction and its effects in the U.S., New Zealand, and Japan. Special emphasis was given to inviting researchers involved in NSF-funded research in New Zealand and Japan. Of the participants, about 66% were from the U.S., 15% were from New Zealand, and 15% were from Japan. Of the U.S. participants, about 15% were graduate students or postdoctoral scholars. Of the invited U.S. participants, about 25% were junior academicians, about 65% were senior academicians, and about 10% were from industry/government. Of the invited U.S. participants, nearly 25% were women.

The workshop was announced in the U.S. through the USUCGER mailing list. About 20% of the U.S. participants were selected from those who responded to the announcement. Each applicant was asked to provide a one-page document that confirmed their availability, presented their reason for attending, and listed three papers that best represented their work in the area of soil liquefaction. The U.S. workshop organizers selected approximately six individuals from academia and industry to attend from those individuals who applied. These preliminary selections were based on the quality and responsiveness of their application, and the goal of having a diverse group of workshop participants. Preliminary selections by the U.S. workshop organizers were approved by a majority of the three U.S. workshop advisors before being finalized. International participants were proposed by the New Zealand and Japanese Workshop Co-Organizers. They finalized their selections based on criteria appropriate for their country. Workshop participants are listed in Appendix C.

1.5.3 Conduct of the Workshop

The workshop was held on the University of California, Berkeley campus at the Faculty Club. The workshop was held in early November 2016, which best satisfied the constraints of the New Zealand and Japanese researchers as well as those in the U.S. Significant time was scheduled for open discussions following invited talks and short presentations by selected participants. The workshop agenda is provided in Appendix D.

1.6 REPORT FORMAT

Initially, this report was to be organized similar to the workshop organization with chapters focused on each of the challenges that remain in understanding and assessing the effects of soil liquefaction (i.e., liquefaction-induced flow slides, liquefaction-induced lateral spreading effects, and liquefaction-induced settlement effects). Consideration was also given to reorganizing the chapters of this report to address directly each of the five workshop prompts (i.e., current state-of-the-art, key underlying geologic processes, primary mechanisms, key challenges to developing better procedures, and best paths forward). However, the conduct of the workshop identified five cross-cutting research priorities that need to be addressed to advance our understanding and assessment of the effects of soil liquefaction. Hence, the report was reorganized to capture common key themes, which often emerged during workshop presentations, that provide an integrated approach to addressing liquefaction effects problems. Recalling that the primary objective of the workshop was to identify research priorities that hold the greatest potential for advancing insights and procedures for evaluating the effects of liquefaction-induced ground deformations on structures and lifelines, the remainder of this report is organized into five chapters:

Chapter 2: Case history data

Chapter 3: Integrated site characterization

Chapter 4: Numerical analysis

Chapter 5: Challenging soils

Chapter 6: Effects and mitigation of liquefaction in the built environment and communities

A concluding chapter follows these chapters, with additional useful information provided in the appendices to this report.

2 Case History Data

2.1 INTRODUCTION

Field case histories have played a fundamental role in liquefaction research for more than 50 years. From the first detailed documentation of liquefaction effects during the 1964 Niigata, Japan, and Great Alaskan earthquakes to the more recent 2010–2011 Canterbury Earthquake Sequence in New Zealand and 2011 Tohoku earthquake in Japan, each earthquake provides important and new insights into the liquefaction phenomenon. Documenting liquefaction effects under true field conditions, which include variable soil properties, multi-directional ground shaking, and three-dimensional geometries, is critical to geotechnical earthquake engineering because these case histories provide critical constraints for the approaches we use to evaluate liquefaction effects. In fact, many of the case history observations of liquefaction and its effects are at the core of the techniques used in liquefaction analysis, such as liquefaction triggering relationships, estimates of the post-liquefaction residual strength of soil, and predictive models of displacement associated with lateral spreading.

The more recent earthquake events in New Zealand and Japan have taken advantage of dense networks of instrumentation and new technologies of field documentation (e.g., LIDAR, satellite imagery) to enhance the quantity and quality of field data that is collected at sites that have liquefied. It is not an overstatement to say that the quantity of case history data available for liquefaction research has increased by an order of magnitude over the last five years. In addition, the high resolution of displacement measurements, topographic models, and site characterizations have allowed for more detailed investigations into the various mechanisms at play in liquefaction effects. Finally, the sharing of observational data and site characterization data, particularly in New Zealand through the New Zealand Geotechnical Database, has allowed a larger breadth of researchers to take advantage of these high-resolution datasets and make additional contributions to liquefaction research.

Despite the encouraging experiences from the recent earthquakes in Japan and New Zealand, there are still important field case history needs across the various aspects of liquefaction and its effects on the built environment. These field case history needs are discussed below, along with the required ancillary information required for a well-documented case history. Additionally, a vision is laid out for formal data publishing and citing of case history data in the literature.

2.2 FIELD CASE HISTORY NEEDS

Field case histories have been documented for various types of liquefaction effects, from basic triggering observations to lateral spreading and flow slides. For some aspects of liquefaction effects there are a large number of documented cases histories (e.g., several hundred case histories are available for use in developing liquefaction triggering relationships), while for other liquefaction effects there are only a handful of case histories (e.g., about 30 case histories are available to assess post-liquefaction residual shear strength). Based on discussions at the workshop, specific aspects of liquefaction effects that require more case histories were identified; they are listed below:

- Post-liquefaction residual strength for medium dense soils (i.e., SPT blow counts larger than about 15 blows/30 cm). There are no case history data for these soils, which may or may not indicate that these materials are not susceptible to flow failure.
- Post-liquefaction vertical settlement (Figure 2.1a). Few data are available for post-liquefaction vertical settlement because settlement is difficult to measure. It may be measured from pre- and post-earthquake surveys or LIDAR, or it may be inferred from the settlement around pile-supported structures that are assumed to have not settled.
- Liquefaction-induced lateral spreading displacements (Figure 2.1b). While several hundred displacement measurements at lateral spread sites are available, the published datasets only represent about 10–15 separate earthquakes and 50–75 sites. New case histories may incorporate traditional techniques to measure displacements (i.e., the mapping and measurement of crack widths), but should also take advantage of remote sensing techniques, such as LIDAR and high-resolution satellite imagery.
- Sites with minor ground movements. While much case history data collection is focused on sites with the most dramatic movements, case history data are also needed for liquefied sites with minor to no vertical settlement or horizontal displacement. These data will allow predictive techniques to be calibrated in the range of small displacements.
- Coupled lateral and vertical soil movements induced by soil liquefaction. The mechanisms for lateral spreading also involve settlement and differential settlement induces lateral ground strain. It is not strictly possible therefore to consider lateral ground movement and settlement as independent from each other. It is important to elucidate how lateral and vertical ground movements are interrelated. Exploration of various geomorphological settings is encouraged in which key topographic and stratigraphic controls can be identified and used to quantify the relative magnitudes of horizontal and vertical deformation.
- Performance of infrastructure subjected to liquefaction-induced settlement and lateral spreading (Figure 2.1c). Liquefaction-induced ground movements are important to engineers when they cause damage to infrastructure. Case histories documenting the performance of infrastructure require careful documentation of the movements and the associated damage at various locations of the infrastructure.
- Liquefaction performance of challenging soils, including interbedded soil deposits, intermediate (silty) soils, gravelly soils, crushable (calcareous, pumice) soils, and aged/cemented sands. Few documented case histories for these types of challenging soils exist, thus it is important to document the field performance of these challenging soils/sites

to understand better how these materials respond during earthquakes and to assess the applicability of liquefaction analysis techniques to these soils.

- Regional-scale observations of liquefaction occurrence and effects (Figure 2.1d). Most liquefaction case histories are focused at the scale of a single site. Yet, liquefaction analysis procedures often are applied at the city or regional scale for planning purposes or to evaluate the seismic response of existing distributed systems such as pipeline or transportation networks. Regional-scale observations of liquefaction effects can be used to calibrate regional-scale liquefaction hazard and risk assessments.
- Performance of ground that has been improved by various soil improvement methods. Case histories can provide information regarding the efficacy of different soil improvement methods and be used to evaluate the techniques used to predict the performance of improved ground.

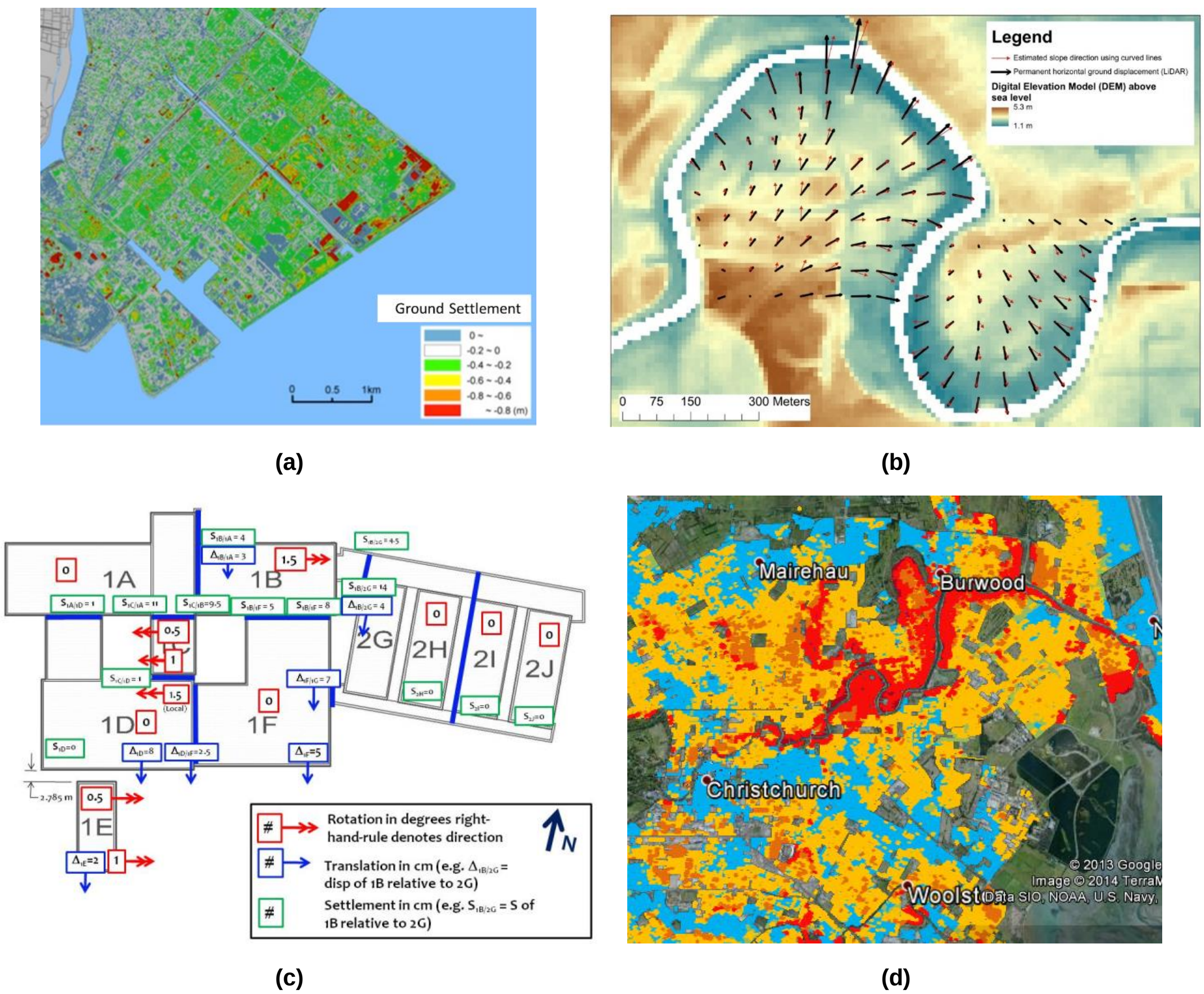


Figure 2.1 **Examples of the different field case history needs: (a) liquefaction-induced ground settlement [K. Tokimatsu, personal communication, 2016]; (b) lateral spread displacements [Cubrinovski and Robinson 2016], (c) response of infrastructure to liquefaction movements [Bray et al. 2012]; and (d) regional-scale liquefaction observations [NZGD].**

Because field case histories play an important role in developing and evaluating liquefaction analysis procedures, it is important to consider the information required to fully characterize a case history for use in analysis. This information includes, but is not limited to, detailed subsurface characterization, geologic interpretations, nearby recorded ground motions, pre- and post-earthquake geometries (e.g., digital elevation or surface models), 2D or 3D ground movements, and quantification of uncertainties for all the information collected. Collecting post-earthquake geometries and measuring ground movements should take advantage of new remote sensing technologies, such as LIDAR, digital photogrammetry, and satellite imagery [Rathje and Franke 2016]. LIDAR, deployed on an aircraft, on a UAV, or on the ground, creates a high-resolution point cloud of a site through laser distance measurements [Figure 2.2 (left)], while digital photogrammetry (e.g., Structure from Motion, SfM) creates high-resolution point clouds and digital terrain models [Figure 2.2 (c)] from a large collection of digital photographs of a study area acquired from different locations [Figure 2.2 (a, b)]. Pre- and post-earthquake point clouds can be used to measure 3D movements across a site; alternatively pre- and post-earthquake satellite imagery can be used to measure 2D horizontal movements. Formal protocols should be developed that describe the information needed to define a high quality case history for the different types of case histories listed above, and also to educate the geotechnical engineering community about the availability of new technologies.

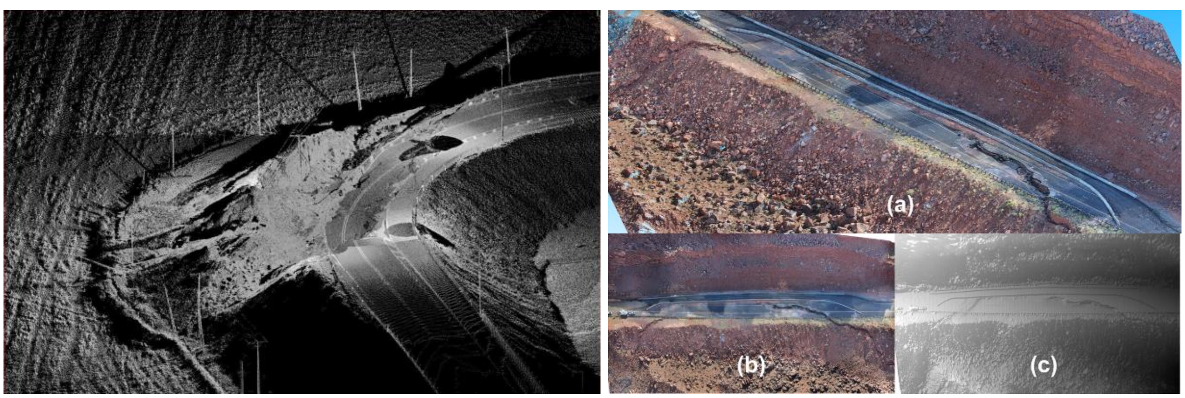


Figure 2.2 Three-dimensional geometries of failures derived from: (left) LIDAR [Frost and Turel 2011] and (right) digital photogrammetry applied to digital photographs collected from a UAV [Rathje and Franke 2016].

The Geotechnical Extreme Events Reconnaissance (GEER) Association has been successful identifying important case histories, collecting perishable data for case histories, and utilizing new technologies to collect data during reconnaissance. Moving into the future, GEER reconnaissance teams, as well as other reconnaissance researchers, should continue to take advantage of new technologies to collect high-resolution datasets for liquefaction case histories. An important recent development is the creation of the National Science Foundation (NSF)-supported post-disaster, rapid response research facility headquartered at the University of Washington. Funded through the Natural Hazards Engineering Research Infrastructure (NHERI), the "RAPID" facility will make state-of-the-art field data collection tools available to the broader research community for use in reconnaissance efforts (https://rapid.designsafe-ci.org/). Finally, researchers should consider installing instrumentation at critical liquefaction sites in highseismicity areas such that high quality, high-resolution data of the liquefaction response at the field scale can be collected when an earthquake occurs. All of these efforts will enable the next generation of contributions to liquefaction research.

2.3 RESEARCH THRUSTS

Until the last ten years, most liquefaction case history datasets were not fully documented, published, or shared. In some cases, only the interpreted data were shared, without the original collected field data or the supporting ancillary information. As a result, others have had to trust the interpretations of the original researcher, despite the fact that the datasets have been disparate in terms of their quality and vetting. The positive experience of sharing liquefaction data through the New Zealand Geotechnical Database after the 2010–2011 Canterbury Earthquake Sequence demonstrated the power of sharing unfiltered subsurface data and field observations with the broader research community.

To accelerate advances in liquefaction research, it is critical that liquefaction case history datasets from across the globe be published in electronic data repositories for use by the broader research community. Importantly, we should not expect researchers to share their data out of simple generosity, but rather researchers should receive academic credit for sharing data and for having other researchers re-use their data. To realize this vision, datasets must be published formally with a permanent Digital Object Identifier (DOI) and with citation language such that other users can cite datasets in their work in the same way that they cite refereed journal articles.

There are several efforts underway to address archiving and publishing of data for the liquefaction and broader natural hazards engineering community. The DesignSafe cyberinfrastructure (https://www.designsafe-ci.org) is a cloud-based environment for natural hazards engineering funded through the NSF-supported NHERI program, and includes the Data Depot data repository for data archiving and publishing [Rathje et al. 2017]. DesignSafe also includes cloud-based tools to analyze, visualize, and integrate different datasets. In addition, the NHERI SimCenter will provide a portfolio of computational modeling and simulation software that can aid in the analyses of case history and related physical modeling data. The Next Generation Liquefaction (NGL) project [Stewart et al. 2016] was recently launched to specifically enhance the accessibility of liquefaction case history data and also to coordinate the development of liquefaction prediction models among different research teams. These activities demonstrate that the research community is realizing the value of sharing datasets via formal publishing mechanisms.

Although this chapter has focused on field case history datasets that have been at the core of liquefaction research for more than 50 years, other datasets have the potential to enhance and accelerate liquefaction research if they are shared with the broader research community. These datasets include experimental data from physical model tests (centrifuge and 1*g* tests) and element-scale laboratory tests (e.g., triaxial, direct simple shear), as well as output data and numerical models from numerical analyses. These datasets also should be documented, archived, and published such that they can be re-used in future research.

3 Integrated Site Characterization

3.1 INTRODUCTION

Site characterization is central to both the development of well-documented case histories (see Chapter 2) and the estimation of model parameters needed for engineering analysis and design (see Chapter 4). Although there are many commonalities in site characterization programs performed for these two purposes, there are also differences. Common among the site characterization programs is the need to integrate qualitative geologic information about the soils at a site and the quantitative information from *in situ* and laboratory engineering tests. The source of the sediments, modes of transport, and the depositional and post-depositional environments directly influence the spatial variability and the fabric and structure of the sediments at a site. Hence, geology has a significant influence on the spatial variability of the engineering properties of the sediments at a site.

Also common to site characterization programs for developing well-documented case histories and estimating model parameters for engineering analysis and design is the underlying objective to identify potential modes of "failure" (i.e., typically associated with poor performance due to excessive deformation). For case history development, the mode(s) of failure are often known to some extent based on field observations and an understanding of the geologic setting of the site. In this scenario, targeted detailed *in situ* tests and sampling for laboratory tests are performed to develop a more complete understanding of the failure mode and to estimate the appropriate engineering properties of the sediments relevant to the failure mode. Both standard and non-standard site characterization methods are often employed for case history development. Non-standard methods for site characterization are often used to assess their viability for characterizing sites, to refine estimates of common soil properties/profile stratigraphy, or to estimate less common soil properties/profile stratigraphy that may be relevant to the observed failure mode.

Ideally, integrated site characterization performed in support of engineering analysis and design is hypothesis driven and iterative, with the level of detail/sophistication employed being commensurate with the potential risk due to failure. Such an integrated site characterization program consists of the following main steps: (1) list of potential failure modes based on understanding of geologic controls operative at the site; (2) selection of engineering models for analysis of failure modes; (3) plan and implementation of site characterization program; (4) revise list of failure modes/modify site characterization plan and implementation; and (5) move forward with design. This approach to integrated site characterization is in line with the Observational Method commonly used in many aspects of geotechnical engineering [Peck 1969]. Unfortunately, schedule and budget constraints rarely permit this type of approach to site characterization to be fully implemented, and often a streamlined version is used, consistent with project requirements and risks. These often consist of the following basic steps: (1) plan and perform site investigation, with or without preliminary geologic model or list of potential failure modes; (2) obtain information needed for models (whether or not models capture all potential failure modes – 1D versus 2D versus 3D models and failure modes); and (3) move forward with design. Also, standard *in situ* test methods that are somewhat insensitive to sediment fabric and microstructure are often used (e.g., Standard Penetration Test, SPT), and sampling for refined characterization of the sediment fabric and microstructure is often *not* performed. This is likely the reason that approximately 45% of all geotechnical-centric legal claims are related to site characterization errors [Lucia et al. 2017].

3.2 ISSUES

A significant amount of discussion among the workshop participants focused on the importance of geology in proper site characterization, ranging from its relevance to the composition, fabric and microstructure of soil at a given location in the profile to the spatial variability of the sediments at a site. It was generally agreed that often site characterizations are performed without consideration of the geologic controls operative at a site, and that most current standard-of-practice *in situ* test methods are not sensitive enough to detect the fabric and microstructure of sediments. Furthermore, disturbed sampling may cause mixing of fine- and coarse-grained fractions, resulting in an incorrect characterization of the engineering properties of the soil.

Some new site characterization approaches/methods are available. For example, two methods presented at the workshop include use of Normalized Rigidity Index (K_G) as a fabric/microstructure index (Schneider and Moss [2011]; Robertson [2016]) and the use of multiple sleeves with varying roughness (Figure 3.1) on a CPT-type device or on self-boring devices (Frost et al. [2014]; Frost et al. [2016]; and Martinez and Frost [2016]). The Normalized Rigidity Index is a function of both small strain shear wave velocity (V_s) and the large strain cone penetration test (CPT) tip resistance; thus, it can provide an indication of aging and cementation (Figure 3.2). Multiple friction sleeve technology has the potential to minimize insertion effects and measure soil properties across multiple scales. Undoubtedly, other approaches for improved site characterization exist or are under development; however, there is a general reluctance by practitioners to embrace these new approaches. The reasons for this relate to: (1) the methods being impractical for commercial use (e.g., too time intensive to use, too difficult to interpret results, or issues with equipment reliability); (2) legal liability for deviating from standard of practice; and (3) general lack of familiarity with the methods by the profession.

Moving from small scale (fabric and microstructure of soil) to a larger scale (variability of deposits across a site), information about the site geologic setting can help guide the locations in which *in situ* tests and sampling are performed (e.g., grid versus nested test locations). Grid test locations, which are often used in the absence of information about a site's geology, can result in erroneous idealization of the spatial variation of the soil profile. This, combined with incomplete/improper characterization of a sediment's fabric and microstructure, often leads to incomplete or misinterpreted case histories or an incomplete identification of potential failure modes for engineering analysis and design.

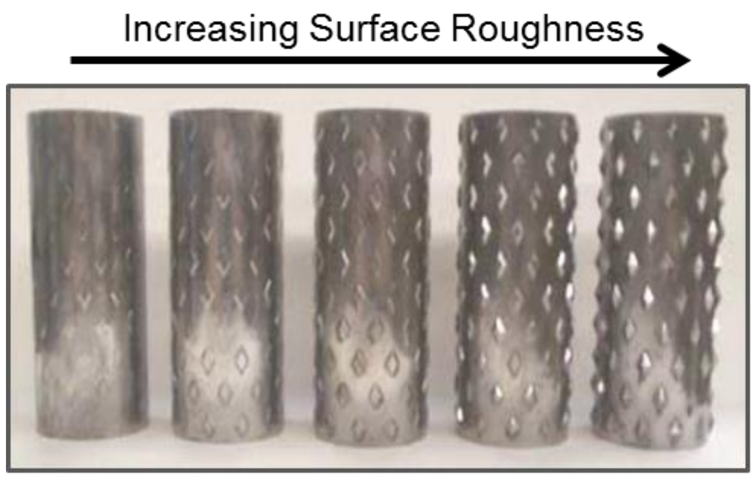


Figure 3.1 **Friction sleeves of variable roughness that are able to measure soil properties across multiple scales [Frost 2016].**

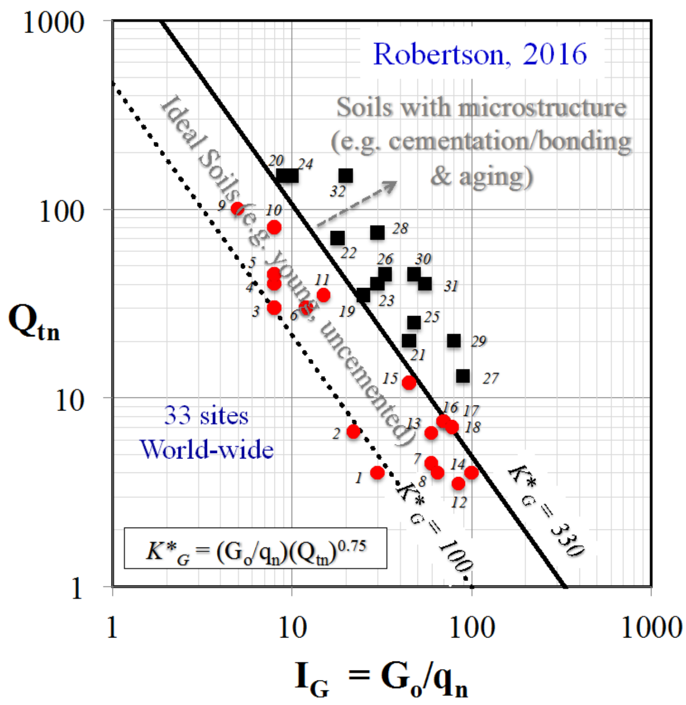


Figure 3.2 **Q_{tn}–I_G chart to identify soils with microstructure. Case history examples: red circles are young uncemented silica-based soils; and black squares are soils with microstructure or calcareous [Robertson 2016].**

3.3 RESEARCH THRUSTS

Better integration of qualitative geologic information about the soils at a site and the quantitative information from *in situ* and laboratory engineering tests is essential for quantifying and minimizing the uncertainties associated with site characterization. This will help identify potential failure modes and guide *in situ* testing. At the site scale, one potential way to do this is to use proxies for depositional environments [Gaskins Baise 2016]. At the fabric and microstructure scale, use of multiple existing *in situ* tests that induce different levels of strain (e.g., V_s and CPT) should be used to characterize soil properties. Also, new *in situ* test methods that are more sensitive to the fabric and microstructure of the soil should continue to be developed and evaluated. In this regard, better education of practitioners on the value of various, non-standard site characterization techniques or the use of multiple tests is needed. Finally, hypothesized potential failure modes at a site should be used to guide the *in situ* testing to ensure appropriate model parameters for engineering analysis and design can be determined from the test results.

4 Numerical Analysis

4.1 INTRODUCTION

The development and rigorous validation of numerical analysis tools and procedures for predicting the effects of liquefaction on the built environment was identified by workshop participants as an overarching research need. This need was repeatedly expressed across the workshop sessions as a priority for addressing key knowledge gaps regarding post-triggering residual shear strengths and the effects of ground deformations on the built environment. Numerical analysis was viewed as critical for several purposes, including obtaining insights on field mechanisms that cannot be discerned empirically, providing a rational basis for developing or constraining practice-oriented engineering models, and providing the essential tool for evaluating complex structures with unique characteristics that are outside the range of empirical observations.

Several key knowledge gaps were identified that will require the development and use of advanced numerical analysis procedures to make significant progress. For example, the ability to simulate localizations and strength loss due to pore water diffusion in heterogeneous deposits (e.g., void redistribution or water film formation) is key to advancing our understanding of the "residual" shear strength that a liquefied soil may develop in the field during or after an earthquake. Ground cracking and graben formation often have strong effects on the magnitude of ground or slope deformations that develop in the field, and yet they often can only be indirectly accounted for in analyses. The representation of spatial variability for deposits with complex depositional architectures is central to understanding many case histories, developing guidance on how to deal with spatial variability in practice-oriented engineering models, and predicting performance for major civil structures. The complex local interactions affecting the performance of civil infrastructure, such as the flow of liquefied soils around piles or the uplift of buried tunnels, are often key limitations in our current analyses of these problems. The large deformations and run-out distances associated with flow slides, such as observed for some recent high-profile tailings dam failures, require further advances in simulating large deformations and accounting for other mechanisms (like entrainment or mixing during run-out). Current numerical modeling procedures offer limited capabilities for addressing many of these knowledge gaps; thus, further advances in the numerical modeling procedures will need to be developed.

Numerical analysis offers unique strengths and capability in the engineering assessment of liquefaction problems. This stems from the fact that soil liquefaction involves highly dynamic processes that include interactions both at particle level and macro scale (“system response”) that in turn result in significant variation of effects on the soil microstructure, layers, deposit as a whole, structural components, and the entire soil–foundation–structure system. One may argue that numerical analysis has a unique capacity to incorporate all these effects, from micro- to macro-scale, including the temporal and spatial evolution of processes and consequences, and relate them to prototype scale configurations, natural environment, and reality. The challenges in this context are: (i) the development of robust numerical tools and procedures that can rigorously address all important aspects in the processes; (ii) how to feed the models and procedures with appropriate input and quality data; and (iii) to provide supporting tools for interpretation of the results in the context of a rational engineering assessment.

Numerical modeling procedures were recognized as having a key role for constraining or guiding development of practice-oriented engineering procedures. For example, engineering practice currently utilizes a number of lateral spreading models that utilize empirical case history data combined with mechanistic frameworks of various types. The mechanistic frameworks are often overly simplified (e.g., 1D liquefaction vulnerability indices) and the case history data do not cover the full range of field conditions and seismic hazards encountered in practice. Numerical analysis provides an essential opportunity for supplementing the case history data and developing improved practice-oriented models that are more rationally constrained.

Participants also highlighted the essential role of numerical analysis for evaluating or designing critical civil infrastructure (e.g., major dams, bridges, ports, buildings, and underground structures) in areas of potential liquefaction for which simplified practice-oriented procedures are insufficient. More advanced analysis procedures are routinely used in practice for such types of infrastructure, but participants noted a major need was improving the standards of practice and validation (through improving codes, procedures, and the competence of the user).

There are several major challenges to developing robust validated numerical analysis procedures for the effects of liquefaction on civil infrastructure systems due to the variety of multi-scale, multi-physics coupled nonlinear interactions that come to the forefront in different scenarios where analytical capabilities for liquefaction effects have not been validated (or, worse yet, have been invalidated). These include several related to the previous discussed knowledge gaps: (1) the range of constitutive behaviors exhibited by various geologic materials; (2) tensile cracking and localizations; (3) pore pressure migration and water film formation in heterogeneous profiles; (4) particle size and particle crushing effects; (5) strain softening and physical instability, including flow problems involving large displacements; (6) 3D stress–strain behavior of soils under earthquake loading; and (7) slip, degradation, and strain-softening at soil–structure interfaces. Progress toward robust validation of numerical analysis procedures will require a sustained and coordinated effort across these various fronts, as discussed in the following section. Relevant efforts include the NSF-sponsored Liquefaction Experiments and Analysis Projects (LEAP), which is an international collaboration to produce a set of high-quality experimental data that can be used to establish the validity of existing computational models for soil liquefaction analysis.

4.2 DEVELOPMENT AND VALIDATION

The progressive advancement of numerical analysis procedures for liquefaction effects on civil infrastructure will require a continuous cycle of development and validation to overcome each of the several major challenges identified at the workshop. Progress will require advances related to the numerical analysis frameworks, constitutive models for soils and interfaces, efficient and robust validation protocols, use of visualization and presentation tools for scrutiny and interpretation of analytical results, and best practices guidance for utilization and documentation in engineering applications.

4.2.1 Numerical Analysis Platforms

Numerical analysis frameworks or platforms will require further advances to address or overcome a number of limitations in present capabilities. Examples of problems posing numerical challenges include the coupled, large-deformation analysis of strain-softening, localizations, cracking, and interfaces in two or three dimensions with complex constitutive models (e.g., Figure 4.1). Finite element and finite difference procedures, which have advantages for several classes of problems, are the most common procedures used in engineering practice. Although the discrete element method (DEM) and material point methods (MPM) offer the potential for great insights and increased capabilities, both methods have a number of challenges to overcome before they can compete in engineering practice. Regardless of the numerical platform, a major barrier to rapid advancement and improved practices is the fact that neither research nor commercial software platforms are able to incorporate the best available solution techniques/options for each of these different classes of problem. A common community platform with sufficient resources to synthesize past progress and sustain further progress remains a major need for the community.

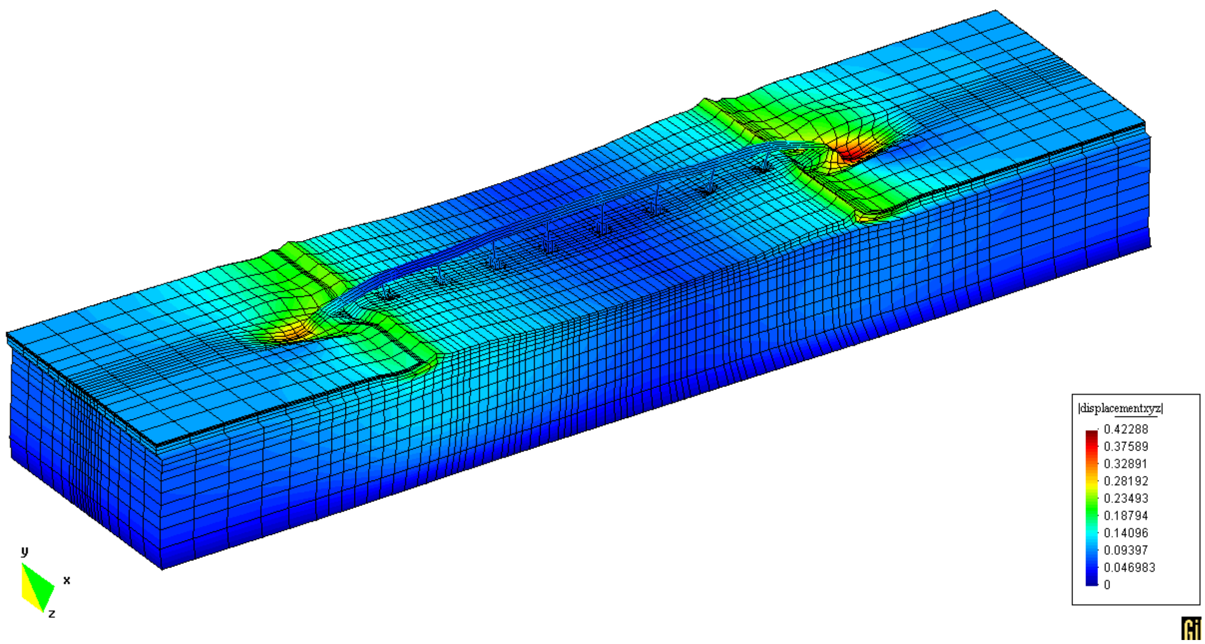


Figure 4.1 High-resolution three-dimensional numerical analysis provides essential abilities for modeling liquefaction effects on major civil structures. Color scale indicates permanent absolute displacement in meters (courtesy A. Elgamal, UCSD).

Soil liquefaction and associated phenomena involve transformational effects and changes in behavior, such as fluidization of loose soils during liquefaction and their transformation from a solid state into a viscous fluid state, or segmentation of the "continuum" through the development of ground fissures and tensile cracking. In this context, strengths and weaknesses of different numerical methods could be further explored including more rigorous definition of limits for various methods with regard to their quantitative predictive capacity. It is important for further advancements and acceptance of numerical tools and procedures to clearly and rigorously establish the realms of quantitative predictive capacity, and associated limits beyond which only qualitative predictions are possible with a given methodology. Development of hybrid approaches in which strengths of different methodologies will be combined to numerically simulate complex phenomena stretching across the boundaries of specific methods may bring a step-change in the predictive capacity of our numerical tools for liquefaction problems.

4.2.2 Constitutive Models

Constitutive models that can simulate seismic responses of the broad range of geologic materials encountered in practice continues to be a major limitation in current practices. Most constitutive models have been developed around frameworks best suited for either clean sand or non-sensitive sedimentary clays. The uncertainties in seismic responses for a host of other challenging soil types (e.g., carbonate soils, gravels, and low-plasticity silts) is discussed in Chapter 5. In all cases, a key challenge is recognizing that complex constitutive models will be required for modeling the full range of possible soil behaviors, but that their adoption in practice depends on sufficient experimental data and the availability of efficient calibration protocols that emphasize ease of use and functionality.

Most of the available models cannot cover all simulation phases from gravity analysis, through seismic analysis to post-liquefaction response in an equally rigorous and integrated manner. We still strive to develop a 3D constitutive model for sandy soils under 3D earthquake excitation. Development of such models including the capacity to model previously identified knowledge gaps will be needed to utilize fully the potential of advanced numerical analyses.

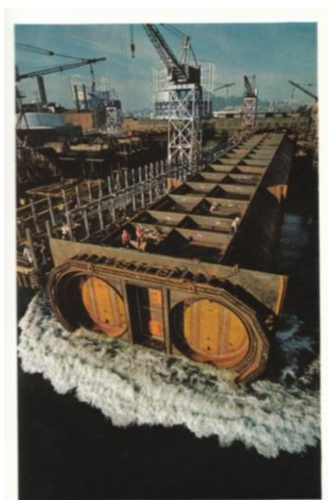
4.2.3 Validation Protocols

The development of more formal validation protocols for nonlinear analysis procedures is important for establishing confidence in, and adoption of, emerging capabilities. Validation protocols need to involve a sufficiently robust set of simulation cases (physical experiments or case histories) to enable an approximate evaluation of bias and dispersion between predicted and measured responses. These efforts will require large sets of experimental data covering a range of related scenarios, soil types, system configurations, loading conditions, and large deformation problems. Ideally, such efforts would also include the comparative evaluation of more than one numerical analysis procedure (e.g., considering alternative constitutive models or calibration protocols). Lastly, validation protocols must be recognized as being dependent on the constitutive model calibration process, the numerical solution parameters, the loading and boundary condition specifications, and the user's experience/capabilities.

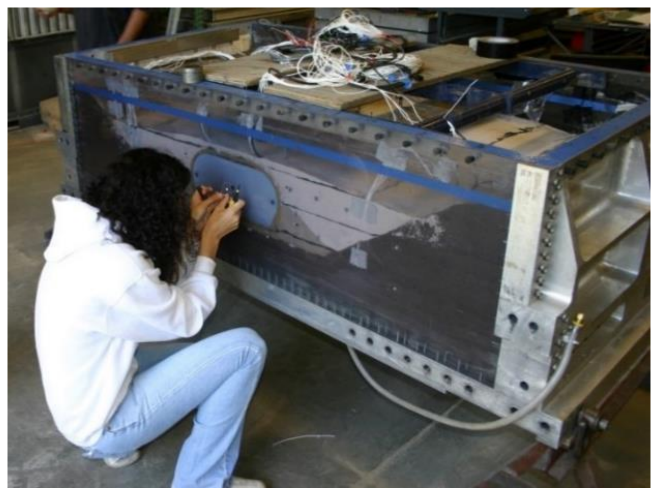
Current constitutive and analytical models cannot be assumed to handle generalized conditions and therefore cannot reliably be extended to new infrastructure systems without structure-specific, soil-specific, and loading-specific validation studies. Centrifuge and shake table modeling provide an essential basis for validation of advanced computational models. The body of archived experimental datasets continues to grow and provide a basis for evaluating new analytical capabilities without necessarily requiring new experimental data, although there are numerous soil/structure/loading conditions not yet examined. Physical models with dense instrumentation arrays enable definition of complex local mechanisms through inverse analyses (e.g., Figure 4.2). Validation against measurements of complex local mechanisms provides a higher-resolution evaluation of computation models and can help identify computational modeling limitations that impact simulation accuracy and generalization at a global scale.

Further validation against well-documented case histories will provide an opportunity to integrate key aspects in the engineering evaluation such as site characterization, soil characterization through laboratory testing, ground motion characteristics, governing mechanisms of deformation (damage), and "system response." Such validations will also elucidate the implementation of numerical tools in engineering practice.

Progress in numerical analysis capabilities will require an integrated approach with a continuous cycle of development and validation. Participants noted the importance of community buy-in, open-access, and baseline resources for maintaining an integrated effort.



(a)



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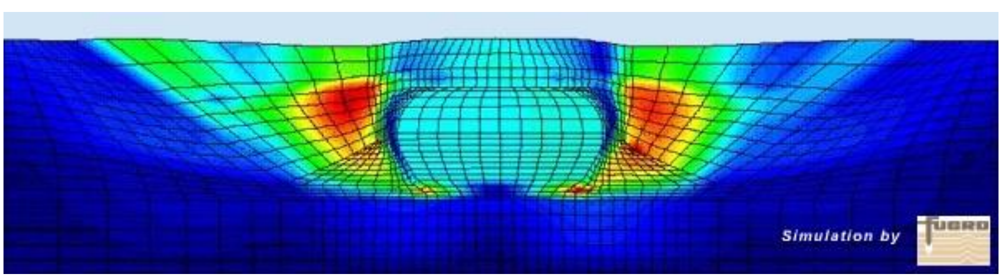

(c)

Figure 4.2 **(a) The effects of liquefaction on the submerged BART tube [National Geographic 1969] was evaluated using (b) 9-m radius geotechnical centrifuge model tests, and (c) nonlinear numerical analysis. The experimental data identified key mechanisms affecting tube uplift and provided essential validation for the computation models used for final design and evaluation of this critical infrastructure.**

4.3 RESEARCH THRUSTS

The development of robust, validated analytical procedures for the effects of liquefaction on civil infrastructure was identified as an overarching research need and priority by workshop participants. Achieving this grand challenge would be transformative for geotechnical engineering at many scales, from advancing our scientific understanding of the most complex phenomena to elevating the standards of practice. Robust validated analytical procedures would translate into more confident evaluations of critical civil infrastructure, support the development of mechanics-based practice-oriented engineering models, help eliminate suspected biases in our current engineering practices, and facilitate greater integration with structural, hydraulic, and wind engineering analysis capabilities for addressing multi-hazard problems.

Collaboration across countries and disciplines was recognized as essential for facilitating rapid progress toward achieving the goal of robust validated analytical procedures. Advancing numerical analysis capabilities would benefit from collaborations with researchers in computer science, applied mathematics, and other engineering disciplines examining the response of particulate, multi-phase systems. Validation protocols and exercises would benefit from multi-country efforts that pool resources, experimental datasets, and enable comparative evaluations of alternative analytical procedures (e.g., constitutive models, calibration procedures, loading and boundary specifications, and numerical parameters).

5 Challenging Soils

5.1 INTRODUCTION

The large majority of liquefaction research has been performed on the traditionally acknowledged liquefiable soil types of clean sands and non-plastic silty sands. The Seed and Idriss [1971] simplified liquefaction triggering procedure was originally developed for clean sands and later work by Seed et al. [1985] added non-plastic silty sands. Current state-of-the-art simplified liquefaction triggering evaluation procedures (e.g., Youd et al. [2001], Moss et al. [2006], and Boulanger and Idriss [2014]) were also developed largely from case histories and laboratory testing programs involving clean sands and non-plastic silty sands. Thus, these semi-empirical procedures are most confidently applied to projects where the seismic response of clean sand and non-plastic silty sands govern performance. Their applicability in the seismic evaluation of other soil types is not known, and their use for soil deposits not composed of clean sand or non-plastic silty sand is often an extrapolation without sufficient data to constrain it. Moreover, the soil constitutive models employed commonly to evaluate liquefaction effects were largely developed and calibrated to capture the seismic response of clean sands. There are few models available that were developed expressly to capture the seismic response of silts, gravel, and other challenging soils.

Characterizing heterogeneous natural deposits or constructed fills across the scale of civil infrastructure systems usually involves a program of *in situ* field testing and laboratory testing of field samples. All currently available *in situ* tests, field sampling tools, and laboratory tests have known limitations when employed in challenging soils. In some cases, there are no reliable *in situ* tests or sampling procedures available. Consequently, the estimation of the engineering properties of these challenging soils remains a dominant source of uncertainty in the application of advanced computational models. There are soil types where the application of current procedures have known biases or major data/knowledge gaps. Examples include finely interbedded sands and fine-grained soils (e.g., effect of inter-bedding on composite response, and lack of resolution of *in situ* test data for thin layers), intermediate soils (e.g., interpretation of *in situ* test data in clayey silts), mine tailings (e.g., evaluation of flow potential), gravelly and cobbly soils (e.g., particle size effects for *in situ* tests and loading responses), crushable soils (e.g., characterization of soils with low penetration resistance due to particle breakage), and aged or cemented soils (e.g., tests that capture their role in liquefaction triggering and its consequences).

The paucity of applicable physical data or case histories for many of these challenging soil types means that their expected responses under generalized loading are poorly understood, and the procedures for estimating their properties lack appropriate validation. Thus, there are critical issues that need to be addressed to improve the methods employed for characterizing the engineering response of these challenging soils. These issues include characterization in the field and laboratory, correlating material responses to index properties, and robust modeling of their dynamic responses in numerical analysis. These issues are discussed for some of the challenging soil types identified previously, and recommendations for future research to advance our understanding and capabilities in engineering design are presented.

5.2 INTERBEDDED SOIL DEPOSITS

Chapters 3 and 4 identified interbedded soil deposits as particularly challenging to characterize physically and to model numerically. Our site characterization tools and numerical procedures have been developed largely for relatively thick soil deposits, whose variability is at a scale that can be captured reasonably with these approaches. Yet, we often encounter in engineering practice highly stratified soil deposits that are difficult to characterize or to model. Some depositional environments lead routinely to the formation of highly stratified soil deposits (e.g., overbank deposits and swamps, and mine tailings ponds). Important issues in characterizing thinly interbedded soil deposits include the role of hydraulic conductivity contrasts in soil deposit response, compressibility of fine-grained soil layers, and limitations in modeling the often small-scale geometry of theses deposits. New, robust approaches are required to address the important issues resulting from interbedded soil deposits.

Relatively small variations in the particle sizes of the finer fraction of adjacent soil layers can lead to large contrasts in their respective hydraulic conductivities. Thus, vertical water flow in highly stratified soil deposits is often restricted as the effective vertical hydraulic conductivity of the deposit is an order or two of magnitudes less than its effective horizontal hydraulic conductivity. Characterization and modeling of the aggregate effects of the contrasts in hydraulic conductivity in the seismic response of highly stratified soil deposits are difficult. Currently, there are no widely accepted methods for identifying cases when this issue may govern site response and its effects on structures and for modeling it when it is thought to be important.

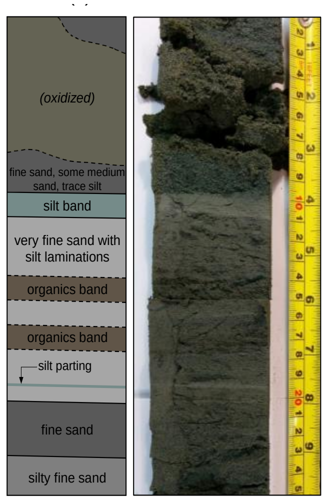

Figure 5.1 Highly interbedded soil deposit in Christchurch, which did not exhibit surface manifestations of liquefaction although intensely shaken [C.Z. Beyzaei, personal communication, 2016].

Interbedded soil deposits containing layers of fine-grained soil are challenging to characterize because of the high compressibility of these soils relative to that of quartz sands. Thus, penetration tests often underestimate the cyclic resistance of a thin sand layer located between two compressible fine-grained soil layers. Conversely, the penetration resistance of a highly compressible thin layer of soil can be overestimated if sandwiched between two stiff layers. Methods for addressing the former case are available (e.g., Youd et al. [2001]), but they are highly uncertain and often applied in an overly conservative manner. Methods for addressing the latter case are not available.

Numerical efficiency remains a challenge in advancing the profession's ability to evaluate the seismic response of highly stratified soil deposits composed of thin layers. Today, we simply cannot afford to model each thin layer in our numerical analysis. Thus, we need robust methods for capturing the composite effects of thinly layered soil deposits, while we develop more efficient numerical procedures to capture directly key effects of highly stratified soil layers. These methods must consider the geologic and depositional environment factors that are integral to developing the fabric of the highly stratified soil deposits.

5.3 INTERMEDIATE (SILTY) SOILS

The liquefaction or cyclic softening of soils intermediate to those of sands and clays (i.e., those soils with significant fines contents or nonzero plasticity indices, such as sandy silts, silts, and low-plasticity clayey, silty sands) have produced devastating ground, building, and infrastructure damage. For example, the cyclic softening of young, low plasticity silts and clayey silts was responsible for lateral displacement, settlement, and tilting of numerous buildings in Adapazari, Turkey, during the 1999 Kocaeli earthquake (e.g., Bray et al. [2004]) and in Wufeng, Taiwan, during the 1999 Chi-Chi earthquake (e.g., Chu et al. [2004]). However, there are also cases of silt deposits liquefying but not undergoing lateral spreading (e.g., Youd et al. [2009]), and sites where silty soils did not produce surface manifestations of liquefaction under intense levels of ground shaking even though state-of-the-art liquefaction triggering procedures indicated that they should have liquefied (e.g., Beyzaei et al. [2015]). In addition, evaluating the potential for catastrophic flow failures in fine-grained tailings materials is complicated by their distinct differences from naturally-deposited fine-grained soils and chemical evolution over time. Therefore, considerable research is required to better understand, characterize, and model the cyclic response of low-plasticity silty and clayey "intermediate" soils.

The cyclic responses of clean sands have largely guided the development of our liquefaction evaluation procedures. For example, the widely used Zhang et al. [2002] post-liquefaction reconsolidation settlement procedure is based on the laboratory testing of clean sand specimens presented in Ishihara and Yoshimine [1992]. This procedure and similar procedures (e.g., Tokimatsu and Seed [1987]) are commonly used to estimate post-liquefaction reconsolidation settlement in silty soils, although none of the empirical data were developed on silty soil test specimens. Penetration resistances in silty soils are typically adjusted by fines-content "corrections" to reflect "clean-sand equivalent" penetration resistances, and these clean sand-based procedures are then used to estimate liquefaction-induced settlements. There is not a sound theoretical basis for this commonly applied adjustment to the Zhang et al. [2002] procedure and little empirical data to support it. Additionally, the higher compressibility of silts relative to sands implies that their penetration resistances may not track as we commonly assume with a fines-content correction. Moreover, the plasticity of the fines does not influence the liquefaction triggering procedures or methodologies that only consider the amounts of fines and not the types of fines. Thus, the lack of a sound theoretical framework and insufficient empirical data are key restrictions in advancing liquefaction evaluation procedures for silty soils.

The issues discussed above (as well as additional issues) provide challenges to characterizing and modeling silty soils. Advancements in material characterization techniques in both the field and the laboratory are required with parallel advancements in the development of robust soil constitutive models specifically calibrated to capture the key aspects of silty soil response during undrained cyclic loadings. As slightly plastic silty soils have been shown to be sampled effectively without inducing significant disturbance with some high-quality samplers, there is merit to performing laboratory tests on silty soil specimens instead of relying solely on *in situ* penetration tests. Alternative field and lab testing tools and simplified penetration test procedures that capture the particularly relevant responses of the full range of intermediate soil types should be explored.

5.4 GRAVELLY SOILS

Field observations from historic earthquakes (e.g., Harder [1988]) and recent earthquakes (e.g., Nikolaou et al. [2014]) have demonstrated that liquefaction of gravelly soils can produce significant damage to civil infrastructure. Research on the triggering and consequences of liquefaction of gravelly soils has been limited, in part, because of the challenges in sampling or *in situ* testing of these soil types. Characterizing gravelly soils in a reliable, cost-effective manner is challenging. Even high-risk, complex projects with sizeable site characterization budgets, such as dams built atop gravelly alluvium, often struggle to evaluate gravel liquefaction and its effects. Due to these limitations and the uncertainties derived from them, expensive liquefaction mitigation measures are often undertaken to mitigate the possible risks from unacceptable performance of a critical facility such as a dam. Thus, there is a pressing need to develop robust characterization and modeling methodologies for gravelly soils.

The cyclic response of gravelly soils is not fully understood due to few well-documented field case histories of gravel liquefaction and the limited availability of the large-scale laboratory test devices required to perform tests satisfactorily on specimens composed of gravel-sized particles. Some large-scale cyclic triaxial testing of gravel has been performed; however, such testing is complicated by issues such as membrane compliance, which increases the uncertainty in the interpretation of test results. Consequently, field testing of gravelly soil deposits is currently preferred. Although not widely available at present, larger penetration devices have been employed with some success with the instrumented Becker Penetration Test (iBPT), potentially making it the preferred *in situ* tool; see Figure 5.2. Shear wave velocity (V_s) methods are a potential alternative. However, V_s measurements may not discriminate adequately between gravel deposits that may or may not liquefy at design levels of earthquake shaking. Moreover, it is challenging to evaluate the consequences of liquefaction with all of the available techniques, whose development has focused primarily on liquefaction triggering of gravels and not its effects. For example, methods for estimating the post-liquefaction residual shear strength of gravelly soils are lacking. There is not a field case history of a flow slide involving gravel liquefaction that can used to back-calculate the post-liquefaction residual shear strength of gravelly soils. There is insufficient understanding of gravel liquefaction at this time to conclude that the absence of such case histories implies that gravels cannot flow. Lastly, some research suggests that the cyclic response of soils with large-sized gravel particles is more dependent on the mineralogy and shape of gravel particles than it is for sand particles. For example, the cyclic response of test specimens composed of crushed rock gravel-size particles differs from that of test specimens composed of river-run hard, rounded gravels [Rollins et al. 1998].

A special challenge in characterizing gravelly soils is the influence of large-size particles on the evaluation of the finer fraction of the matrix material, which may govern the overall response of some gravelly soils depending on the grain size distributions within various lenses in a natural deposit. With the limited empirical data available on gravelly soils, it is currently difficult to develop the validated, robust soil constitutive models required to perform reliable numerical analysis.

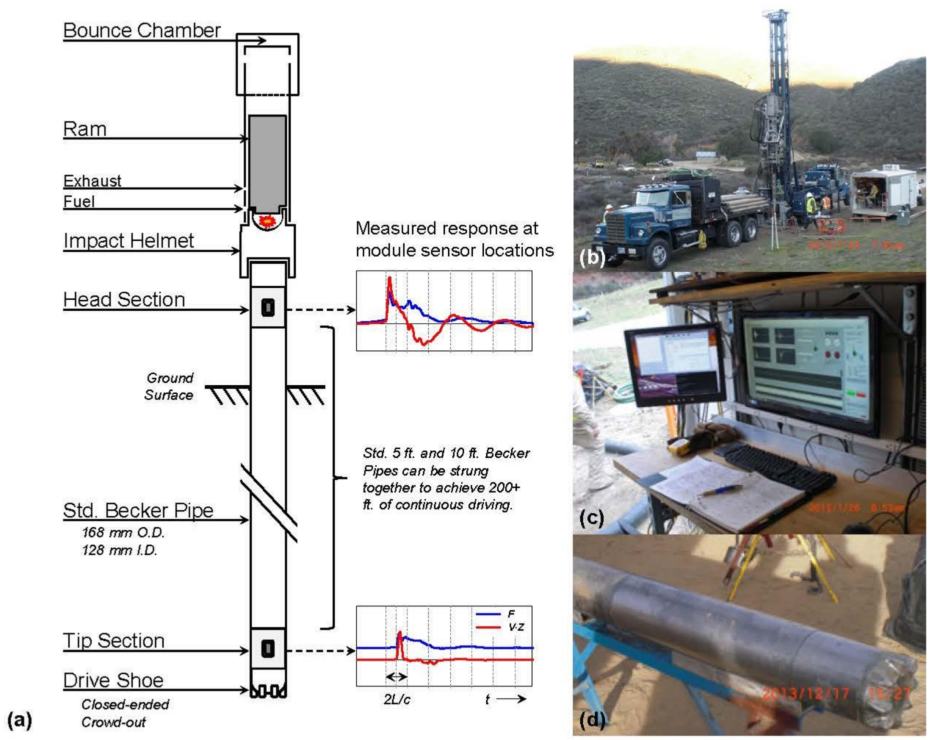

Figure 5.2 **(a) Schematic of the iBPT system including the diesel hammer and sample measurements at the head and tip; (b) drill rig used to conduct Becker testing; (c) above-ground data control system for the iBPT; and (d) iBPT tip section and closed-ended drive shoe [DeJong et al. 2015].**

5.5 CRUSHABLE SOILS

There are significant deposits of crushable soils worldwide that require sound liquefaction assessment tools and methods. Calcareous sands, volcanic pumice, coal fly ash waste materials, and a variety of other soils are susceptible to particle crushing at low to intermediate stresses. The engineering properties of these materials are less understood than the traditionally studied soils composed of hard, quartz, rounded/subrounded particles. This is especially true in terms of the undrained cyclic response of crushable soils and its effects on structures and infrastructure. The consequences of underestimating the effects of liquefaction of crushable soils for critical structures such as oil platforms demands conservatism in the use of current methods that possess great uncertainty regarding their application to crushable soils. The excessive costs involved in overestimating the potential effects of liquefaction of crushable soils due to the use of penetration-based liquefaction evaluation methods that were not developed nor calibrated for crushable soil deposits are also daunting.

Soils with crushable particles have divergent responses from soils with hard, quartz, rounded/subrounded particles, which are the materials that form the primary basis of our current understanding and empirically based liquefaction evaluation methods. The results of penetration tests in crushable soil deposits cannot be interpreted with existing empirical methods that were developed largely on clean, quartz sands. It is not clear if penetration resistance is a suitable indicator of the cyclic resistance and the consequences of liquefaction of crushable soils. Although the limited available data and research on crushable soils have identified and demonstrated these and other critical issues, they have not yet led to well-accepted alternative procedures. Consequently, there are great opportunities to advance understanding and engineering design tools for crushable soils.

5.6 AGED OR CEMENTED SOILS

The liquefaction case histories from which Cyclic Resistance Ratio (CRR) correlations have been developed are almost exclusively for Holocene-age soils, with many being less than 500 years old. However, engineers are often asked evaluate the liquefaction potential of older (or “aged”) deposits, which raises questions about the appropriateness of the existing correlations. It is well known that aging effects are important to soil liquefaction triggering (e.g., Youd and Perkins [1978] and Seed [1979]). Although a full understanding of the underlying mechanisms for aging is lacking, micro-scale particle reorientation and cementation have been proposed as leading mechanisms, with the former becoming more in favor than the latter. As discussed previously in Section 3.2, a combination of *in situ* characterization tools may help to identify when cementation and aging effects may be important (e.g., Figure 3.2). Adding to the challenge faced by engineers tasked with evaluating the liquefaction potential of aged deposits is that most of the studies on the influence of aging on liquefaction resistance provide qualitative results (e.g., Youd and Perkins [1978]). One of the few exceptions to this is the method proposed by Hayati and Andrus [2009], which uses shear-wave velocity (V_s) in combination with penetration resistance to quantify aging effects on liquefaction resistance. Although this procedure shows promise (e.g., Maurer et al. [2014]), further field validation is needed. Recent earthquakes have presented several case histories of earth fills of various ages that have or have not liquefied. These cases histories should be investigated fully to advance the understanding of aged soils. Additionally, CRR correlations and soil constitutive models do not currently incorporate directly aging effects, although most engineers believe these effects can be important in some cases.

The evaluation of the liquefaction potential of cemented soils also poses a challenge to engineers. Cementation typically increases penetration resistance as well as V_s measurements. If the earthquake loading is sufficient to break the cementation of contractive, saturated soils, liquefaction of these soils is possible. The assessment of liquefaction and its consequences for these soils is challenging due to its brittle response once cementation bonds are broken. The development of reliable procedures to discern if liquefaction will occur and the consequences if it occurs will be challenging. As noted by workshop participants, the pressing need in several major projects to evaluate the cyclic response of cemented soils illustrates that this research is warranted to advance the state-of-the-practice on this topic.

5.7 RESEARCH THRUSTS

There are soils that are challenging to characterize, to evaluate, and to model because their responses differ significantly from those of clean sands and silty sands, which are materials that have been comparatively well studied. Of these soils, the greatest needs and potential for advancing insight on the effects of soil liquefaction-induced ground deformations on structures and lifelines are through research on interbedded soil deposits, intermediate (silty) soils, mine tailings, gravelly soils, crushable soils, aged soils, and cemented soils.

Workshop participants supported a major research thrust regarding challenging soil types that should include field and laboratory testing to characterize the responses of these materials to earthquake loadings, physical experiments (e.g., centrifuge tests) to explore mechanisms, and the development of new soil constitutive models that are implemented in advanced robust and efficient numerical analysis. Additionally, well documented case histories where the poor and good performance of engineered systems at sites whose response is governed by these materials should be of high priority.

Research on any one challenging soil type is likely to benefit strongly from coordinated efforts across case history studies, laboratory testing, physical modeling, and numerical analysis efforts. An example scenario would be large-scale physical model tests on a challenging soil type, wherein various *in situ* test measurements are performed prior to earthquake loading (e.g., CPT and V_s), samples are obtained for laboratory testing, inverse analyses of shaking records are used to describe material responses, and numerical models are developed for and evaluated against the recorded responses. The efficiency of such coordinated efforts would be strengthened by international collaborations and the inclusion of expertise from the geologic sciences (e.g., sedimentology and geophysics) and material science (e.g., particle characteristics related to macro behaviors).

6 Effects and Mitigation of Liquefaction on the Built Environment and Communities

6.1 INTRODUCTION

As demonstrated by recent earthquakes in Japan and New Zealand, liquefaction can have widespread effects on communities due to loss of critical infrastructure, buildings, and residential structures. Recent and ongoing research following the Canterbury Earthquake Sequence [CES] [van Ballegooy et al. 2014], provides extraordinary insights about:

- Regional susceptibility to liquefaction;
- Geomorphic and topographic controls on patterns of liquefaction-induced settlements and lateral spreading [Cubrinovski and Robinson 2016];
- Impact of liquefaction-induced ground deformation on underground water supply, wastewater conveyance, gas distribution [O'Rourke et al. 2014], and electric power systems [Tang et al. 2014];
- Regional liquefaction impact on residential structures and commercial buildings [Bray et al. 2014; van Ballegooy et al. 2014]; and
- Liquefaction-induced ground deformation effects on roads and bridges [Cubrinovski et al. 2014].

Of particular importance is the social impact of widespread liquefaction and its effects on land use planning. In the Christchurch area, the spatial variability of liquefaction severity has had important ramifications on rebuilding and insurance coverage, including requirements for residential building foundations, shallow ground improvement techniques, and eligibility for insurance. Of particular importance is the effect of severe liquefaction on restricting services and real estate development through the designation of "red" zones in which buildings, infrastructure, and eligibility for insurance are removed or restricted to avoid exposure to future liquefaction-induced damage.

Likewise in the U.S., liquefaction plays a key role in city planning and the development of critical infrastructure. In San Francisco, for example, the backbone for fire protection is the Auxiliary Water Supply System (AWSS), which is zoned with cut-off valves to isolate portions of the system likely to be damaged by liquefaction during a severe earthquake. The vulnerability of San Francisco to fire following earthquake is well known [Scawthorn, et al. 2006]. The fire following the 1906 San Francisco earthquake is still the greatest single fire loss in U.S. history. The size and severity of the 1906 San Francisco fire is related in large part to the loss of water due to liquefaction-induced failure of water distribution pipelines [O'Rourke et al. 2006]. After the 1989 Loma Prieta earthquake, San Francisco came very close to a major conflagration from fire that began in the Marina District at the same time that damage caused by liquefaction to the AWSS and the potable water supply systems left the Marina and large portions of the central business district without water. Fortunately because of NSF-supported research, a portable water supply system was in place that successfully put out the Marina fire [O'Rourke, 2010].

Given the close interaction between liquefaction and the performance of critical infrastructure servicing large, diverse communities, it is essential to cover the effects and mitigation of liquefaction on the built environment as a core part of engineering and geoscience research programs focused on seismic hazards. The spatially variable effects of liquefaction on buildings, lifelines, and the communities that rely upon them, and their mitigation through ground improvement and seismic resilient design and retrofit, needs to be an essential part of the research agenda for liquefaction.

6.2 BUILT ENVIRONMENT

The built environment is composed of buildings and spatially distributed infrastructure, often referred to as lifelines. These systems are intricately linked to the health, economic well-being, security, and social fabric of the communities they serve. As pointed out by ATC [2016], when earthquakes or other hazards strike lifeline systems, they disrupt the flow of resources and provision of services that sustain communities. In the worst cases, these disruptions can lead to regional, national, and even global social and economic impacts, such as the devastating consequences of the 2011 Tohoku earthquake and tsunami impacts at the Fukushima-Daiichi Nuclear Power Plant. Not only did the loss of this essential lifeline facility result in the loss of 30% of Japan's electric power supply, it also contributed to a global crisis of confidence regarding the safety and reliability of nuclear power.

Buildings provide the structural core of the built environment, which are serviced by lifelines. Modern communities are characterized by a diverse building stock that encompasses residential housing, commercial and industrial buildings, and public buildings and facilities. There are many different types of buildings with a myriad of structural configurations each subject to different thresholds of tolerable settlements and lateral deformation that are, in turn, linked with limit states associated with architectural, functional, and structural damage. Lifelines are constructed over broad geographical areas and are vulnerable to a wide range of seismic and other natural hazards. Transportation lifelines, for example, often rely on bridges that are founded in the type of liquefaction-susceptible soil that is frequently found along rivers and shorelines. The failure of a particular bridge or of a particular section of pipeline may have consequences that extend far beyond the specific location of the failure. This characteristic has a profound influence on planning and design as compared with similar activities for a building or specific facility that are local in nature.

Large parts of the built environment are more than 50 to 100 years old, with many constructed before modern earthquake codes, standards, and guidelines. Aging and repetitive use reduces infrastructure resilience to hazards such as earthquakes. In 2013, the American Society of

Civil Engineers (ASCE) graded the nation's infrastructure as a D+ across 16 categories [ASCE 2013] . Vulnerability due to aging is an important factor to consider in evaluating both building and lifeline response to liquefaction-induced ground deformation.

6.3 RESILIENCE

According to ATC [2016], the concept of *Resilience* involves the ability of people and communities to adapt to changing conditions and withstand and rapidly recover from disruptions. With respect to lifelines, community resilience involves a complex interaction among the people who depend on lifeline systems and the physical characteristics, operation, and management of those systems. People need access to the resources and services supplied by lifeline systems to withstand and recover from disaster-related disruptions. Also, the rate at which the functionality of lifeline systems is restored can have a major influence on a community's recovery trajectory and outcomes.

The resilience of an organization, community, building, or lifeline system is an overarching attribute that reflects its degree of preparedness and ability to respond to and recover from shocks [ATC 2016]. With regard to hazard events, resilience has been defined as: "*the ability to prepare and plan for, absorb, recover from, and more successfully adapt to adverse events*" [The National Academies 2012].

Resilience has become a governing concept for strengthening communities and the built environment against natural hazards and human threats, including continuity in business operations, emergency planning and response for essential services, hazard mitigation, and the capability of the built environment (e.g., facilities, transportation systems, utilities) to resist physically and rapidly recover from disruptive events [ATC 2016]. Substantial resources have been mobilized to promote research and development for resilient communities by the Department of Homeland Security (DHS), National Institute of Standards and Technology (NIST), and NSF.

6.4 BUILDINGS

Building response to liquefaction depends on the mechanisms of ground deformation and associated soil–structure interaction triggered by liquefaction. Ground deformation mechanisms are complex and related to shear induced soil deformation, including loss of bearing capacity and ratcheting strains linked to rocking and shearing motion; volumetric deformation; and loss of underlying soil caused by ejecta transmitted to the ground surface [Bray and Dashti 2014]. In addition, building response is related to the width and height of the structure, foundation depth, thickness of non-liquefiable crust and underlying liquefiable layers, ground motion characteristics, and severity of liquefaction. Several key parameters have been investigated with respect to building settlement during liquefaction, including the effects of the building aspect ratio (ratio of building width to thickness of liquefiable layer), foundation relative stiffness, mass eccentricity, bearing pressure, number of stories, static factor of safety with respect to bearing capacity, and dynamic overturning moments.

Research is needed to understand better and clarify the complex mechanisms of ground deformation and building performance. The relative effects of the various factors described above have been studied, but clear trends have not been delineated. Simplified procedures for quantifying building response to liquefaction are needed. Of critical importance is the role that the thickness of non-liquefiable crust plays in the deformation of buildings due to underlying liquefaction. Research is needed on the effects of crust type and crust thickness relative to the thickness of the underlying liquefiable layers. The influence of crust thickness on ejecta needs to be investigated, including the crust thickness that effectively blocks or suppresses ejecta and the influence of crust imperfections (structural penetrations, cracks, root holes, trenches, etc.) on the transmission of ejecta to the ground surface.

For deep foundations, research is needed on liquefaction-induced down drag and the effects of liquefaction on battered piles. It is important to understand how liquefaction affects piles and shafts that bear on firm soil layers at depth by developing a better understanding of how side shear is affected by liquefaction, including loss of side shear during liquefaction and negative skin friction triggered in response to post liquefaction-induced loss of volume. The effects of lateral movement and settlement on battered piles and their connections to structures such as wharf decks must also be better understood.

Research methods appropriate for the evaluation of building response to liquefaction include physical modeling, especially in the case of centrifuge studies that are able to isolate various ground deformation mechanisms and investigate the influence of crust and liquefiable layer thickness as well as building parameters. Detailed analyses of well-documented case histories provide invaluable information about actual building response and the means for validating advanced numerical modeling with material and geometric nonlinearities.

6.5 LIFELINES

Lifelines are generally grouped into six principal systems, including electric power, gas and liquid fuels, telecommunications, transportation, water supply, and wastewater conveyance and treatment. Flood and hurricane protection systems are also lifelines in the sense that levees and flood protection structures form geographically distributed lines of defense against inundation, thus providing critical infrastructure and support for communities.

Lifeline response to earthquakes and liquefaction involves two levels of system behavior: (1) component performance for which soil–structure interaction under earthquake loading is evaluated; and (2) system performance for which the integrated behavior of the network is assessed. The two are very different and are governed by issues related to the level of detail in component versus system characterization, spatial variability, uncertainties in material properties and component state of repair, redundancy, network flow laws, operational logic of the system, and direct and indirect impact on communities; see Figure 6.1.

A significant trend in geotechnical engineering has been the implementation of advanced physical testing of lifeline components and facilities for soil–structure interaction. Examples of such testing include centrifuge simulation of pipelines crossing strike–slip and normal faults [O'Rourke et al. 2010] as well as the effects of transient motion and liquefaction in urban environments with multiple adjacent buildings [Hayden et al. 2015]. Large-scale testing has been used to characterize soil–pile interaction during liquefaction at the Japanese National Research Institute for Earth Science and Disaster Prevention [Tokimatsu and Suzuki 2004]. Large-scale tests have also been used to simulate fault movement and abrupt ground rupture effects on underground pipelines and protective vaults [O'Rourke 2010; Jung et al. 2016].

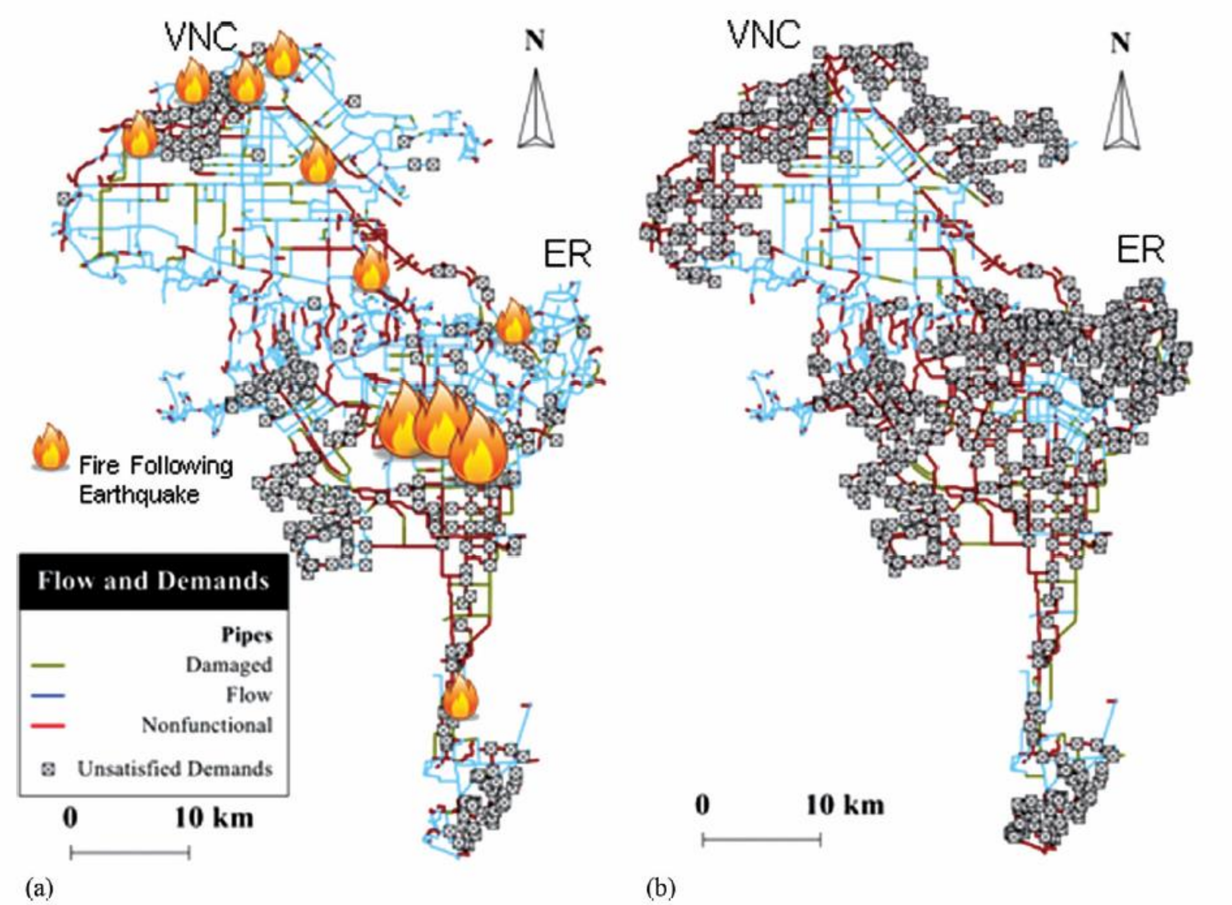

Figure 6.1 Plot of the major water transmission pipelines in Los Angeles indicating flow state and unsatisfied demands for (a) 0 and (b) 24 hours after the 2008 ShakeOut Scenario earthquake. Also shown are the locations of large fires and super conflagrations. These simulations are an example of advanced network modeling for planning and engineering large geographically distributed systems (after Davis and O'Rourke, [2011]).

Large-scale testing plays a critical role in the development of the next generation earthquake and hazard resilient pipelines. With strong encouragement from U.S. water supply owners and operators, numerous pipeline manufacturers are designing and fabricating pipelines that are able to accommodate large ground deformation caused by soil liquefaction, landslides, and faulting. This market-driven research and development program has resulted in many novel designs and new products to improve water distribution system performance during earthquakes and post-earthquakes.

System performance and modeling are important for at least three reasons [O'Rourke, 2010]. First, system performance provides the basis for planning and engineering at a scale commensurate with earthquake or other hazards that have large, geographically distributed effects. Second, system performance is the logical extension of component or individual pipeline response. It entails the outcome of integrated component behavior, and for a pipeline network represents the ultimate expression in terms of service and the consequences of soil–structure interaction. Third, system performance provides the only way by which managers and engineers can gauge the scale and regional impact of an earthquake or similar natural hazard. System performance sets the stage for quantifying the regional economic consequences and community impact of an earthquake, as well as planning for emergency response and system restoration.

Geotechnical engineering is an indispensable part of the modeling and management of large geographically distributed lifeline systems. Research is needed to characterize the effects of liquefaction and other geohazards on large, geographically distributed systems, including modeling of their spatial and temporal variability, as well as methods for making risk-based decisions under conditions of high uncertainty.

Major risk assessments and seismic retrofit projects are underway in the U.S. to improve the seismic performance of electric power grids, gas and liquid fuel facilities, telecommunications facilities, transportation hubs, water supply facilities, wastewater conveyance, and levee systems. There is an urgent need for improved modeling of the effects of liquefaction-induced ground deformation on underground pipelines, conduits, and cables to provide more realistic projections of damage and system restoration times. For example, pipeline repair rates in water distribution networks caused by liquefaction-induced ground deformation are often estimated in accordance with the American Lifelines Alliance Guidelines [ALA 2001]. These guidelines are now well out of date and need to be improved with data and observations from more recent earthquakes, including the 2010–2011 Canterbury, New Zealand, Earthquake Sequence, and the 2011 Tohoku, Japan, and 2010 Maule, Chile, earthquakes.

As pointed out in Chapter 2, collecting post-earthquake geometries and measuring liquefaction-induced ground movements should take advantage of new remote sensing technologies, such as LIDAR, digital photogrammetry, and satellite imagery. These remote sensing technologies are valuable not only for the collection of data for research and engineering synthesis, but are critically important for earthquake recovery. Remote sensing has the capability for quantifying regional damage patterns, including damage to commercial and industrial buildings, residential structures, underground utilities, and transportation systems.

6.6 MITIGATION

Liquefaction hazards may exist in both natural and man-made soil deposits. A variety of natural processes can result in liquefaction-susceptible soils. While widespread recognition of liquefaction over the past 50 years has led to the design and construction of fills that are resistant to liquefaction, many areas of older fills with low liquefaction resistance still exist. To provide resilience at such sites, liquefaction hazards must often be mitigated.

A variety of liquefaction mitigation techniques have been developed and used over the past 50 years; see Figure 6.2. Often described as “soil improvement” techniques, they can be broadly divided into five main categories: densification, reinforcement, grouting/mixing, crust strengthening, and drainage. Some techniques fall into more than one of these categories. The techniques can be further subdivided according to the level of disruption they cause to the area in which they are being applied. Some techniques cause so much vibration, noise, or deformation that they are impractical for use in populated areas or in the vicinity of functioning, existing structures such as buildings, bridges, and pipelines. Other techniques are much less invasive and can be used with little to no disruption or inconvenience to surrounding structures and activities.

Figure 6.2 Jet grouting of cellular wall structure adjacent to sensitive historic piers along Seattle waterfront (http://sdotblog.seattle.gov).

Soil improvement techniques are unique in geotechnical engineering in that frequently they have been developed through the initiative and imagination of contractors as opposed to the more conventional process of research and validation. Design procedures have followed implementation in a number of cases. In some cases, research has uncovered deficiencies in early design procedures; in others, existing design procedures have not yet been fully validated by research or empirical experience. New soil improvement procedures have been developed in recent years but have not been fully developed to the point where they can be relied upon in practice. This situation gives rise to a number of research needs and opportunities, including:

- Many urban areas have developed adjacent to rivers, estuaries, bays, and other large bodies of water and hence have saturated sandy soils that are susceptible to liquefaction. These areas often have considerable critical, operating infrastructure located on or within liquefiable soil deposits, so soil improvement is limited to techniques that cause minimal noise, vibration, and deformation of the soil being improved. Such techniques can be used beneath buildings or around pipelines without taking them out of service. While a number of non-disruptive techniques are available, they are frequently expensive and can be less reliable in terms of the uniformity of the improvement they provide at field scale. In recent years, a number of promising new technologies have been proposed, including desaturation by introduction of air bubbles into liquefiable soils and bio-remediation techniques such as microbially-induced calcite precipitation. Further research into the field-scale level, uniformity, and permanence of these and existing procedures is needed.

- Validation of soil improvement techniques is ultimately accomplished best by observation and documentation of full-scale behavior during actual earthquakes. The development of case histories that compare the response and performance of improved ground with nearby unimproved ground after strong shaking would greatly improve the confidence and economy with which such procedures could be used.
- Verification of soil improvement has been a critical issue for many years. It is common to measure soil parameters, such as penetration resistance or shear wave velocity, before and after improvement, but interpretation of the post-improvement measurements has proven difficult, largely due to time-dependent variations in the measured parameters. Further research into measurements, including both geotechnical and geophysical tests, that correlate well to the degree and uniformity of soil improvement is needed.
- Soil improvement methods vary widely in applicability, speed, disruption, sustainability, and cost. Engineers currently have little quantitative information with which to base comparisons and selections of optimal techniques. Research focused on the development of performance-based design procedures that consider life-cycle costs would lead to more efficient and economical use of often-scarce liquefaction mitigation resources.

Mitigation can also be accomplished by initial construction and retrofitting to enhance resistance to liquefaction-induced ground deformation through strengthening and stiffening of structures, or the provision of either flexibility or ductility so that the structure can adjust to differential movement with minimal loss of integrity or functionality. The foundation of a residential building, for example, can be stiffened with a rigid mat or fortified beam and floor construction. Underground pipelines and conduits can be manufactured with special joints and materials that can accommodate liquefaction-induced ground deformation through axial slip and rotation at specially designed joints or by means of the ductility inherent in welded steel pipe and polyethylene pipelines with thermal fusion connections. The next generation earthquake and hazard resilient pipelines, discussed above, represents an industry-based effort to improve water supply infrastructure by mitigating the effects of liquefaction and other sources of differential ground movement through innovative pipeline design.

6.7 COMMUNITIES

As emphasized by ATC [2014], lifelines are essential for emergency response, restoration of order, and recovery after earthquakes as well as other natural hazards and human threats. Resilience, buildings, and lifelines converge in the communities that are exposed to hazards. Community recovery from disasters depends on the orderly and rapid restoration of the built environment. Community resilience to earthquakes depends on the mitigation and control of liquefaction, especially when the geomorphology and stratigraphy promote widespread liquefaction-induced ground deformation.

The same characteristics affecting performance of buildings and lifelines under seismic conditions—including interdependencies, socioeconomic factors, and institutional constraints—affect them when subjected to other hazards. Research and implementation focused on lifeline and building systems performance are inherently multidisciplinary. The network analysis procedures, metrics and tools developed for modeling lifeline and building response to earthquakes can be adapted to other hazards. Insights gained about lifeline system interdependencies and socioeconomic issues apply to multiple hazards, as do intelligent monitoring and sensor technologies developed to improve lifeline system reliability to earthquakes [ATC 2014].

6.8 RESEARCH THRUSTS

The effects of liquefaction on the built environment and the mitigation of liquefaction were identified by workshop participants as critical topics for research, development, and implementation. The built environment consists of residential housing, commercial and industrial buildings, and public buildings and facilities, as well as spatially distributed infrastructure, referred to in this report as lifelines, including electric power, gas and liquid fuel, telecommunication, transportation, water supply, wastewater conveyance/treatment, and flood protection systems. Liquefaction has had damaging effects on buildings and lifelines, causing widespread disruption of the built environment and threatening the economic well-being and community security in places such as Christchurch, Tokyo, San Francisco, and Los Angeles. Research focused on the improved characterization and mitigation of liquefaction effects on the built environment is an essential and indispensable part of promoting community resilience against natural disasters.

Research to improve the characterization and mitigation of liquefaction effects on the built environment is a broad mandate. Some opportune topics for this research were identified by workshop participants. For example, the complex mechanisms of ground deformation caused by liquefaction and building response need to be clarified and simplified procedures for quantifying building response to liquefaction are needed, including the effects of the building's aspect ratio (ratio of building width to thickness of liquefiable layer), foundation relative stiffness, mass eccentricity, bearing pressure, number of stories, static factor of safety with respect to bearing capacity, dynamic overturning moments, and the effects of crust thickness relative to the thickness of the underlying liquefiable layers. Research is also needed on how liquefaction affects piles and shafts through a better understanding of how side shear is affected by liquefaction, including loss of side shear during liquefaction and negative skin friction triggered in response to post-liquefaction-induced loss of volume.

Research on lifeline response to liquefaction involves: (1) component performance for which soil–structure interaction under earthquake loading is evaluated; and (2) system performance for which the integrated behavior of the network is assessed. Research on component behavior can be advanced by centrifuge and large-scale soil–structure interaction testing. A noteworthy example of large-scale testing is the development of the next generation earthquake and hazard resilient pipelines, whereby the pipeline industry has been engaged in market-driven research to provide pipelines that are able to accommodate large ground deformation caused by soil liquefaction, landslides, and faulting.

Research is needed to characterize the effects of liquefaction on large, geographically distributed systems, including modeling their spatial and temporal variability, as well as methods for making risk-based decisions under conditions of high uncertainty. In particular, there is an urgent need for improved modeling of the effects of liquefaction-induced ground deformation on underground pipelines, conduits, and cables to provide more realistic projections of damage and system restoration times.

Research should be focused on soil improvement techniques, which are broadly divided into five main categories: densification, reinforcement, grouting/mixing, crust strengthening, and drainage. Research is needed on the most effective, cost efficient, and sustainable measures to improve the response of buildings and lifelines to liquefaction-induced ground deformation. A future research agenda should include strategies and protocols for using soil improvement to reduce ground deformation in combination with structural modifications to accommodate movement.

7 Conclusion

7.1 INTRODUCTION

The objectives of this workshop were to identify research needs and approaches to overcome the key challenges resulting from three categories of soil liquefaction effects problems:

I. The effects of liquefaction-induced flow slides that are governed by the residual shear strength of liquefied soil;

II. The effects of liquefaction-induced lateral spreading on structures and lifelines; and

III. The effects of liquefaction-induced settlement on structures and lifelines.

Workshop participants were asked to address these challenges by responding to one or more of these prompts:

1. What is the current state-of-the-art for evaluating this problem today?
2. What are the key underlying geologic processes that affect it?
3. What are the primary mechanisms involved in the phenomenon?
4. What are the key challenges to developing better evaluation procedures?
5. What is the best path forward for advancing understanding and procedures to address it?

During the conduct of the workshop, participants identified five cross-cutting research priorities that need to be addressed to advance our understanding and assessment of the effects of soil liquefaction. Recalling the primary objective of the workshop was to identify research priorities that hold the greatest potential for advancing insights and procedures for evaluating the effects of liquefaction induced ground deformations on structures and lifelines, the report was organized to address five research themes:

- Case history data
- Integrated site characterization
- Numerical analysis
- Challenging soils
- Effects and mitigation of liquefaction in the built environment and communities

These research themes provide an integrated approach to addressing liquefaction effects problems. The primary findings and recommendations presented previously in each chapter of the report are summarized in this concluding chapter of the report.

7.2 CASE HISTORY DATA

Many of the available liquefaction case history datasets are not fully documented, published, or shared. To accelerate advances in liquefaction research, it is critically important that liquefaction case history data sets be published in electronic data repositories for use by the broader research community. Developing and sharing well documented liquefaction datasets are significant research efforts. Therefore, datasets should be published with a permanent DOI, with appropriate citation language for proper acknowledgment in publications that use the data.

There are several efforts underway to address publishing case history data. The *DesignSafe* cyberinfrastructure is a cloud-based environment for natural hazards engineering funded through the NSF-supported NHERI program [Rathje et al. 2017]. It includes the Data Depot data repository for data archiving and tools for making the data accessible. Efforts such as the Next Generation Liquefaction (NGL) project were launched recently to enhance the accessibility of liquefaction case history data. These activities demonstrate that the research community recognizes the importance of collecting and sharing case history data. Such efforts should continue to be supported as important research thrusts in liquefaction engineering. Lastly, these datasets should include also experimental data from physical model tests so they too can inform future research.

7.3 INTERGRATED SITE CHARACTERIZATION

Better integration of qualitative geologic information about the soil deposits at a site and the quantitative information from *in situ* and laboratory engineering tests of these soils is essential for quantifying and minimizing the uncertainties associated with site characterization. Such information is vitally important to help identify potential failure modes and guide *in situ* testing. At the site scale, one potential way to do this is to use proxies for depositional environments. At the fabric and microstructure scale, the use of multiple existing *in situ* tests that induce different levels of strain should be used to characterize soil properties. New *in situ* testing tools and methods that are more sensitive to the fabric and microstructure of the soil should continue to be developed. Engineers should be trained to value non-standard site characterization techniques and understand the benefit of using multiple tests to investigate soil response more fully. Hypothesized potential failure modes at a site should be used to guide the integrated site characterization plan to ensure that appropriate model parameters for engineering analysis and design can be estimated reliably.

7.4 NUMERICAL ANALYSIS

The development of robust, validated analytical procedures for evaluating the effects of liquefaction on civil infrastructure is a critical research need. The pursuit of this important challenge would advance our scientific understanding of the effects of soil liquefaction on the built environment and elevate the standards of engineering practice. Robust, validated analytical procedures would translate into more confident evaluations of critical civil infrastructure, support the development of mechanics-based practice-oriented engineering models, help eliminate suspected biases in our current engineering practices, and facilitate greater integration with structural, hydraulic, and wind engineering analysis capabilities for addressing multi-hazard problems.

Effective collaboration across countries and disciplines is essential for facilitating rapid progress toward achieving the goal of robust validated analytical procedures. Advancing numerical analysis capabilities would benefit from collaborations with researchers in computer science, applied mathematics, and other engineering disciplines examining the response of particulate, multi-phase systems. Validation protocols and exercises would benefit from multi-country efforts that pool resources, experimental datasets, and enable comparative evaluations of alternative analytical procedures. Advancing analytical capabilities in liquefaction engineering can be truly transformative in research and practice. Moreover, these advancements can be incorporated and integrated in other areas of liquefaction effects research.

7.5 CHALLENGING SOILS

There are soils that are challenging to characterize, to model, and to evaluate, because their responses differ significantly from those of clean sands, they cannot be sampled and tested effectively using existing procedures, their properties cannot be estimated confidently using existing *in situ* testing methods, or constitutive models to describe their responses have not yet been developed or validated. Research on these challenging soils—interbedded soil deposits, intermediate (silty) soils, mine tailings, gravelly soils, crushable soils, aged soils, and cemented soils—is required. Field and laboratory testing are required to characterize the responses of these materials to earthquake loadings, physical experiments (e.g., centrifuge tests) are required to explore mechanisms, and new soil constitutive models that are implemented in advanced robust and efficient numerical analysis are required. Additionally, well-documented case histories involving challenging soils where both the poor and good performance of engineered systems are documented are a high priority.

Research on challenging soil types is likely to benefit strongly from coordinated efforts across case history studies, laboratory testing, physical modeling, and numerical analysis efforts. An example scenario would be large-scale physical model tests on a challenging soil type, wherein various *in situ* test measurements are performed prior to earthquake loading (e.g., CPT and V_S), samples are obtained for laboratory testing, inverse analyses of shaking records are used to describe material responses, and numerical models are developed for and evaluated against the recorded responses. The efficiency of such coordinated efforts would be strengthened by international collaborations and the inclusion of expertise from the geologic sciences, material science, and computational mechanics.

7.6 EFFECTS AND MITIGATION OF LIQUEFACTION ON THE BUILT ENVIRONMENT AND COMMUNITIES

The effects of liquefaction on the built environment and the mitigation of liquefaction are high-priority topics for research, development, and implementation. The built environment consists of residential housing, commercial and industrial buildings, and public buildings and facilities, as well as spatially distributed infrastructure, including electric power, gas and liquid fuel, telecommunication, transportation, water supply, wastewater conveyance/treatment, and flood protection systems. Liquefaction has had damaging effects on buildings and lifelines, causing widespread disruption of the built environment and threatening the economic well-being and community security in places such as Christchurch and Tokyo.

Research to improve the characterization and mitigation of liquefaction effects on the built environment is a broad mandate. Some opportune topics for this research were identified by workshop participants. For example, the complex mechanisms of ground deformation caused by liquefaction and building response need to be clarified and practice-oriented procedures for quantifying building response to liquefaction are needed. Research on lifeline response to liquefaction involves: (1) component performance for which soil–structure interaction under earthquake loading is evaluated; and (2) system performance for which the integrated behavior of the network is assessed. Research on component behavior can be advanced by centrifuge and large-scale soil–structure interaction testing in combination with numerical simulations. System response requires advanced numerical and network analyses. Further research is required to characterize the effects of liquefaction on large, geographically distributed systems, including modeling their spatial and temporal variability, as well as methods for making risk-based decisions under conditions of high uncertainty.

Lastly, research on liquefaction mitigation strategies, including aspects of ground improvement, structural modification, system health monitoring, and rapid recovery planning, is needed to identify the most effective, cost efficient, and sustainable measures to improve the response and resiliency of the built environment. Soil improvement techniques are broadly divided into five main categories: densification, reinforcement, grouting/mixing, crust strengthening, and drainage. Research is needed to better quantify the mechanisms of these soil improvement techniques, to develop and validate analytical methods for evaluating their performance during earthquakes, and to develop strategies for efficiently utilizing these techniques in reducing the impacts of liquefaction on our built environment and communities.

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APPENDIX A: WORKSHOP ABSTRACTS

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POST-LIQUEFACTION RESPONSE OF GRAVELLY SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL

To date, most research in soil liquefaction has focused on sands, as they have been observed to liquefy in the field and can be readily tested under controlled conditions in the laboratory. However, the response of gravelly soils during earthquake loading is not fully understood due to fewer well-documented case histories of field liquefaction as well as the unavailability of large-scale laboratory test devices. Recently, extensive gravel liquefaction was observed at numerous villages and sites in the Chengdu plain during the 2008 Wenchuan, China earthquake and also during a sequence of two relatively smaller earthquakes in Cephalonia, Greece 2014 gravel fill liquefaction resulted in extensive damage associated with settlement and lateral spreading in two ports. Characterizing gravelly soils in a reliable, cost-effective manner is very challenging for routine engineering projects. Even for large projects, such as dams and energy projects, characterization is expensive and problematic. Nevertheless, dam engineers are frequently called upon to assess the potential for liquefaction in gravels and liquefaction mitigation costs often run into millions of dollars.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Data from laboratory testing of gravelly soils is very limited due to the need for large scale testing devices to accommodate gravel particles. Limited large scale cyclic triaxial testing of gravel has been performed, however it has important testing complications due to membrane compliance. Cyclic simple shear testing of gravel has not been conducted, even though the stress path during a cyclic simple shear test is most representative of field conditions. Field testing is typically preferred, but the penetration techniques currently used in the US (i.e. short-interval SPT, CPT and Becker Penetration Test) are either not suited for gravels, or have high-cost of mobilization and testing, large uncertainties in measured resistances due to the distribution of forces and deformations along the shaft and at the tip, and require several corrections and adjustments before they can be used in liquefaction triggering assessment (Ghafghazi, et al 2014). Shear wave velocity (V_s) based methods are a promising alternative or complement for liquefaction assessment (Dobry 2016), however the relationship between V_s and liquefaction susceptibility of gravels is not sufficiently established. To assess post-liquefaction shear strength of gravelly soils, cyclic triaxial testing has been shown to produce higher values compared to cyclic simple shear (Ishihara, 1993), so existing data cannot be used alone for this assessment. Finally, there are no reported back calculated residual (post-liquefaction) shear strengths from sites with liquefied gravels to be used in developing correlations.

2.5 PATHS FORWARD

An integrated approach combining large scale laboratory testing, innovative field testing and back-analysis of case histories is required to develop liquefaction triggering charts and post-liquefaction shear strength recommendations for gravelly soils. At the University of Michigan (UM) we developed a prototype large-scale (12” diameter) cyclic simple shear (CSS) device (with V_s measurement capabilities) and we have produced a unique set of constant volume, monotonic and cyclic simple shear test data for gravels (Hubler et al.

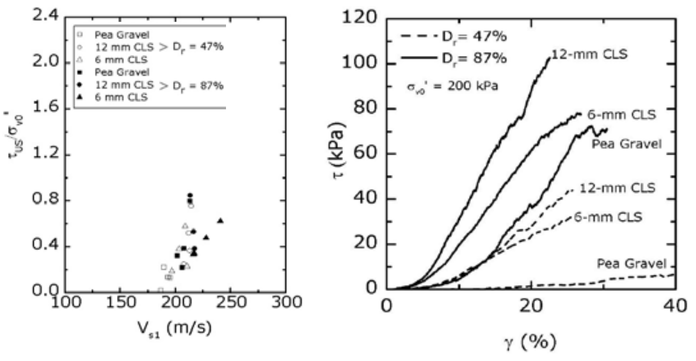

Figure 1. (a) Comparison of 3 Uniform Gravels for τ_{US}/σ_{v0}’ versus V_{s1} and (b) Post-Cyclic Stress-Strain Response (Hubler et

2014 and Hubler et al. submitted) (Fig.1) and gravel-sand mixes.

Despite the relatively small magnitude of the Cephalonia, Greece, 2014 earthquakes, extensive liquefaction of gravelly soils was observed in the coastal front and especially in the two main ports, Lixouri, and Argostoli. A strong motion station located 150 m away from the Lixouri port recorded 0.64g during the second event, whereas in the vicinity of the Argostoli port there were two strong motion stations. We have collected extensive documentation of the gravel liquefaction inland as well as at the sea-front and port-front where significant lateral spreading occurred over a distance of about 500-1000 m in each port, geotechnical investigation data and conducted extensive Dynamic Cone Penetration Tests (DPT) and V_s testing. As illustrated in Figure 2, being able to back analyse a case history having information on stratigraphy (DPT, Vs, profiles), post-event deformation and input ground motion, provides unprecedented opportunities for assessing post-liquefaction shear strength and stability of gravelly soils.

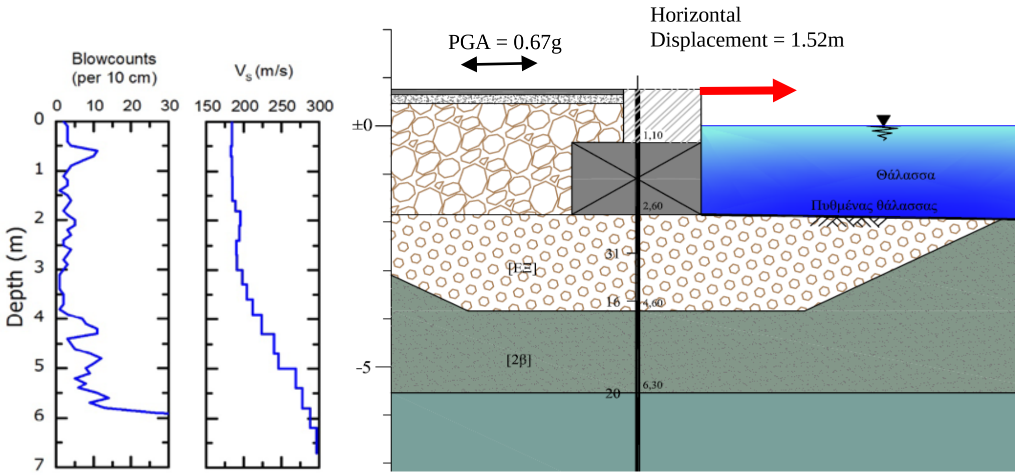

Figure 2. Schematic of the Lixouri port at a location where 1.52m of horizontal displacement was measured, and DPT and Vs profiles with depth for the same location.

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NEED FOR IMPROVED GROUND TRUTH DATA SETS FOR TRAINING AND TESTING OF LIQUEFACTION-INDUCED SETTLEMENT MODELS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED SETTLEMENT

When the goal is to characterize liquefaction effects in terms of vertical and horizontal displacements, the liquefaction prediction methodology needs to include accurate geometry as well as soil behavior.

2.1 CURRENT STATE-OF-THE-ART

The majority of empirical liquefaction assessment techniques rely on point-data such as in-situ penetration measurements and either characterize liquefaction at a point where a sample has been taken or along a line informed by point measurements. Prediction of liquefaction effects often involves integration over the vertical length of a boring (SPT) or cone (CPT).

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

In order to predict accurate settlements across the foundation of a structure or along the length of a lifeline, the evaluation has to include the potential settlement due to the liquefaction of a volume of material. As a result, the prediction requires knowledge of accurate subsurface geometry. The differential settlements across a site are critical for structure and lifeline performance.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Although liquefaction may initiate at a point or at several points in the subsurface; the surface effects due to liquefaction result from the soil and water movement through the subsurface and the resulting settlement.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

This brings up two key challenges that need to be addressed to further the challenge of developing methods to predict liquefaction-induced settlements for future earthquakes:

1. Accurate definition of three-dimensional spatial variability of soil properties and stratigraphy in order to determine the extent and volume of liquefiable material at a site.
2. Availability of training databases for liquefaction induced settlements which include a broader more complete characterization of both the three-dimensional subsurface (#1) and the two-dimensional settlement across the site.

2.5 PATHS FORWARD

Key Challenge #1 – three-dimensional characterization of the spatial variability of soil properties – and the effects on liquefaction. Prior efforts exist to help characterize the spatial variability of soils (e.g. DeGroot and Baecher, 1993; DeGroot, 1996; Phoon et al., 2003; Elkateb et al., 2003; Uzielli et al., 2005) and their effect on liquefaction (Fenton and Vanmarke, 1998; Popescu et al., 2005; Dawson and Baise, 2005; Baker and Faber, 2008; Chakrabortty and Popescu, 2012; Chen et al; 2016); however, three-dimensional characterization of the subsurface is not commonplace. Several prior efforts have investigated this topic in the laboratory or through simulations: how does liquefaction develop in heterogeneous media (Fenton and Vanmarke, 1998; Popescu et al., 2005; Baker and Faber, 2008; Chakrabortty and Popescu, 2012). However, these efforts have not been translated into the ability to predict liquefaction severity. From a prediction point of view: How does volume of liquefiable material in combination with depth correlate with liquefaction-induced settlements? Can we better use knowledge of geology and depositional environment to predict extent and spatial variability of liquefiable materials, and ultimately settlements?

Key Challenge #2 – improve training data quality and quantity.
Our historic databases for liquefaction are limited to three types: 1) binary point data (liquefaction or no liquefaction) with explanatory variables simplified to single values (e.g. Boulanger and Idriss, 2015), 2) descriptive case study data of a particular site or structure (e.g. Bray et al. 2014) or 3) spatial maps of liquefaction occurrence (e.g. Toida and Yamazaki, 2012). The extensive data collection efforts after the 2010-2011 Christchurch earthquakes and the 2011 Tohoku earthquake have resulted in high resolution liquefaction data sets that have defined a new standard. These datasets include detailed geotechnical information, detailed geologic information, and high resolution mapping of liquefaction surface effects. The high resolution mapping data is coming either from visual interpretation of aerial imagery, or automated processing of imagery acquiared using SAR, LiDaR or satelittes (e.g. WorldView-2 or WorldView-3).

The key to moving forward is use all available technologies to redefine our data collection standards so that we optimize the quantity and quality of collected data related to liquefaction effects in future events. To improve the quality of post-event data collection, we need to define the necessary components of future reconnaissance efforts as a community to ensure consistent and complete data collection. High quality data collection will also require resources. With the recent and continuous improvement in sensor technology (both LiDaR and satellite imagery), our ability to map high resolution settlements and surface effects due to liquefaction is steadily improving (Morgenroth et al., 2016; Zhu et al., 2016; Rathje et al., 2016). Figure 1 shows recent work on developing automated image processing methods to map liquefaction using WorldView-2 imagery acquired before and after the event. Progress is being made on image processing for liquefaction mapping; however, many challenges remain (e.g. shadows, image processing due to climate changes, land cover in urban vs rural environments). We need to continue developing the data processing methods to pair with the technology to ensure the accuracy of acquired data.

Future liquefaction data collection should include:

- Consistent data collection: name required elements (e.g. 2D settlement maps, 3D stratigraphy, SPT, CPT, etc.)
- Complete data collection: need to collect data at nonliquefied – and non-settlement sites
- High quality data collection: 2D settlement maps (e.g. LiDaR), high resolution maps of ejected sand

This will require that we as a community embrace the use of remotely sensed data to provide high resolution maps of settlement, horizontal movement, and evidence of surface effects (ejected sand).

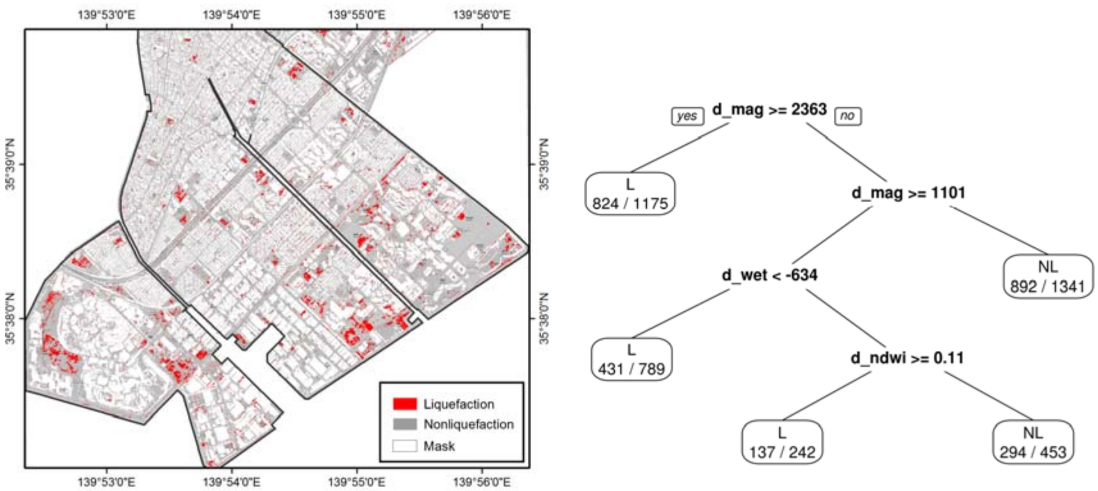

Figure 1. Map of liquefaction induced surface effects in Urayasu City mapped using pre-event and post-event images from WorldView-2. The classification uses a decision tree with spectral information from the Tassled cap transformation and the NDWI.

3.0 ACKNOWLEDGEMENTS

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MITIGATING EFFECTS OF SILTY SOIL THIN-LAYER STRATIGRAPHY ON LIQUEFACTION MANIFESTATION AND CONSEQUENCES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Case histories from the 2010-2011 Canterbury Earthquake Sequence (CES) provide data for level-ground sites where liquefaction triggering and settlement are expected to occur based on assessment using simplified methods, yet no liquefaction manifestations or consequences were observed during the post-earthquake reconnaissance (Beyzaei et al. 2015, Stringer et al. 2015). These "no-liquefaction" sites provide a unique opportunity to investigate: a) potential reasons for discrepancies between the simplified method estimations and post-earthquake observations, b) the range of subsurface environments for which simplified methods are appropriate, and c) how heterogeneous stratigraphy is simplified for analysis.

Recent investigations suggest that thin-layer stratigraphy of liquefiable and non-liquefiable silty soils may be a critical factor in the potential for liquefaction manifestation and consequences at a site, providing insight beyond that of CPT-based assessment and element-scale laboratory testing (Beyzaei et al. 2016). The effects of thin layers on pore water pressure development, dissipation, and movement; hydraulic connectivity; and post-liquefaction reconsolidation may influence liquefaction occurrence, manifestations, and consequences. Thin non-liquefiable layers may prevent development of thick hydraulically-connected strata that enable upward ejecta flow, prohibiting manifestations at the ground surface. Liquefiable layers may be held in place due to sand layers/lenses pinching out or interspersed silt pockets.

Additional issues that arise regarding these sites are the classification of case histories for *triggering* correlations as "liquefaction" or "no liquefaction" based on observed *manifestations* at the ground surface and the application of a 1D or 2D mindset to subsurface environments that may truly require 3D understanding. With liquefaction assessment methodologies for clean-sand sites relatively well-established, the profession would benefit from incorporation of more borderline and marginal liquefaction case histories arising from differing subsurface conditions.

2.1 CURRENT STATE-OF-THE-ART

Liquefaction assessments (simplified methods or numerical analysis) typically account for uncertainty and variability in soil characterization and estimated consequences through the use of the median $\pm\ \sigma$ estimates or upper and lower bound estimates of the key soil parameters. Research does not provide much guidance regarding fully probabilistic approaches. Additionally, the available methods are often based on a 1D interpretation of a 3D problem. While this simplification may work in many cases, for thinly-layered stratigraphy in a heterogeneous environment, this approach appears to fall short.

Popescu et al. (1997, 2005) and Boulanger & Montgomery (2015) examine spatial variability within relatively consistent deposits and geologic units, presenting results and guidance on selecting representative properties by stratum. However, sites with thin-layer stratigraphy such as the CES case histories may have interlayering of liquefiable/non-liquefiable soils that varies over short distances, vertically and laterally. These sites require evaluating the effects of variability within non-uniform deposits, considering the scale and characterization of thin layers within a stratum rather than a stochastic-type variability. The influence of thin-layer stratigraphy and stratum non-uniformity on liquefaction occurrence and manifestation can then be considered in the context of the complex 3D subsurface depositional environment and stratigraphy. For these cases, it may not be appropriate to select a single N-value or CPT properties to represent a stratum for liquefaction assessment or considerable judgement in selecting those parameters will likely be required.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

Geology is critical in understanding development of subsurface stratigraphy, the potential for liquefaction, and interaction between thin layers and the stratum as a whole. Working from a background of geology allows engineers to begin to distinguish differences between sites and strata that are not readily apparent from CPT data or parameters such as relative density and fines content.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

- Availability of case histories with discrepancies between liquefaction estimates and observations
- Difficulty identifying marginal liquefaction sites during post-earthquake reconnaissance efforts
- Recent CES case histories are largely from free-field/low-rise construction sites; different liquefaction manifestations may have occurred with different building stock in place

For the available "no-liquefaction" case histories, determining:

- If liquefaction did or did not occur at depth (given no manifestations at the ground surface)
- Which factor(s) limited surface manifestation if liquefaction did occur at depth (e.g. thin-layering, partial saturation, non-liquefiable crust, or multiple factors)
- Range of estimated CRR and CSR
- Which stratum is the "critical layer" (sites with thin-layering and complex stratigraphy may not have a clearly defined critical layer to incorporate in databases/correlation development)
  - Seemingly consistent critical layers can have thin layers not identified by the CPT or SPT
- At what scale vertical and lateral heterogeneity become homogeneity, how that should be characterized, and what representative properties should be selected for those layers in analysis
- Appropriate investigation techniques for characterization of thin-layer stratigraphy in more heterogeneous environments and for large areas such as those covered by infrastructure networks

2.5 PATHS FORWARD

Paths forward: development of additional no-manifestation liquefaction case histories, greater emphasis on documenting borderline case histories, field/laboratory testing programs and numerical sensitivity analysis. Goals: assessing the scale at which heterogeneity becomes homogeneity for the purpose of liquefaction assessment, guidance on selecting homogeneous properties representing the vertically and laterally heterogeneous system (especially for numerical analysis), identification of critical parameters influencing simplified assessments, and determining appropriate and consistent application of simplified methods across a broader range of sites, including identification of sites for which simplified methods may not be applicable.

3.0 ACKNOWLEDGEMENTS

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DETERMINING PROPERTIES OF CHALLENGING SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1, 2 & 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES, LATERAL SPREADING & SETTLEMENT

An overarching challenge in assessing liquefaction and ground deformation effects on infrastructure is the development and validation of procedures for determining the properties of a range of challenging soil types. Challenging soil types, in this context, are those for which there are significant knowledge gaps or limitations in currently used procedures, including issues related to the available empirical data, in-situ testing methods, field sampling capabilities, laboratory testing capabilities, or fundamental understanding of material responses. Examples of challenging soil types are intermediate soils (e.g., clayey sands, sandy silts), gravels and gravelly soils, calcareous soils, lightly cemented soils, thinly interbedded soil deposits, coal fly ash materials, tailings materials, and quick clays.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The key challenges to developing better evaluation procedures for some of these challenging soil types can be grouped in three categories. First, each challenging soil presents unique obstacles to the development of reliable evaluation procedures or adaptation of the evaluation procedures currently used for most sand or ordinary clay deposits. Second, there is generally a shortage of sufficiently detailed case histories or physical model tests for guiding improvements to evaluation procedures or validating new procedures. Third, there is often an inadequate theoretical framework for rationally interpreting all aspects of the procedures (e.g., from in situ test results to performance under seismic loading), such that the basis for extrapolation to a broader range of conditions often remains highly uncertain.

The unique nature of each challenging soil type results in its own unique obstacles regarding some component of an engineering evaluation. For intermediate soils, how well can the effects of sampling disturbance be managed for various combinations of fines content and fines plasticity? For gravels and gravelly soils, how reasonable is it to predict their seismic loading behavior based on equivalent $(N_1)_{60}$ values from iBPT tests with correlations developed primarily for sands? For calcareous soils, can the effects of sampling disturbance be managed and/or do we have enough information to develop correlations between cyclic responses/strengths and the results of in situ tests over a broad range of overburden stress conditions? For lightly cemented sands, how does the cementation affect their cyclic responses/strengths, the results of in situ tests, and hence the correlations between cyclic responses and in situ test results. For interbedded sand, silt, and clay deposits, how can we evaluate the cyclic strengths of sand/silt beds less than 20 or 30 cm thick given they are below the resolution limits of most in situ test (CPT, SPT, or V_s) measurements? For sensitive or quick clays, are we adequately evaluating the degree of remolding and strength loss or the potential for localizations to develop during seismic shaking? These and other challenges have persisted despite research efforts to date, illustrating the need for perhaps more concerted efforts and new approaches.

The paucity of detailed physical modeling or case history data for many soil types means that their expected behaviors under generalized loading are poorly understood and the procedures for estimating their properties based on in situ tests or other characterization data lack appropriate validation.

Theoretical frameworks and constitutive models for interpreting the results of in situ tests, relating the results to the soil's cyclic responses, or simulating seismic responses at the system level are also often lacking for various challenging soil types. Limitations in our theoretical frameworks contribute to uncertainties in extrapolating the limited available data to a broader range of field conditions.

2.5 PATHS FORWARD

Advancing evaluation procedures for many challenging soil types would be greatly aided by obtaining field case histories or physical model tests with the characterization data (e.g., vane shear, T-bar, CPT, dynamic penetrometers, V_s, V_p, lab tests on samples) and seismic performance data obtained for the same site or geotechnical structure/model. In this regard, physical modeling using centrifuge or 1-g shaking tables offers a potential path forward for supporting the more timely development or validation of evaluation methods for various challenging soil types.

Physical modeling at sufficiently large scales offers the potential for performing these characterization tests in models with realistic levels of system complexity (including geologic complexity, such as inter-bedded sand and silt deposits) and minimizing scale effects (e.g., ratio of penetrometer size to particle or interlayer size). Methods for preparing centrifuge or shaking table specimens would need to be developed for each challenging soil type, while also recognizing that certain characteristics of natural deposits (e.g., age) cannot be simulated with reconstituted models. Additional in-flight in-situ testing tools would need to be developed and the capabilities of existing in-flight tools expanded. Inverse analyses of dense instrumentation arrays can be used to quantify cyclic responses/strengths under different shaking motions or sequences of motions. The combination of in-situ testing data and back-calculated cyclic responses/strengths can then be used to evaluate correlations between these behaviors; e.g., Darby et al. (2016) demonstrated the utility of this approach by examining the correlations between cone penetration resistance and cyclic resistance ratio for sand specimens subjected to multiple shaking events (Figure 1).

Lastly, these experimental efforts rely on the parallel development of theoretical frameworks and constitutive models for interpreting the results of in situ tests, relating the results to the soil's cyclic responses, and simulating seismic responses at the system level. Progress for any one challenging soil type is more likely to be achievable through group research efforts bringing together teams with the appropriate range of technical strengths.

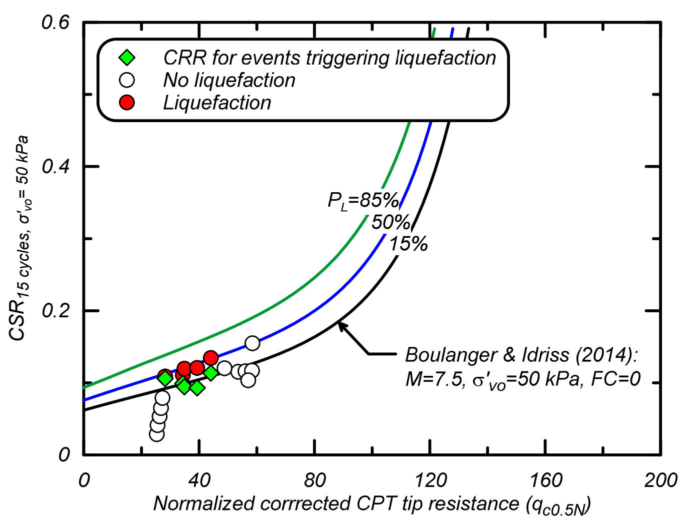

Figure 1. Example of centrifuge tests used to examine the correlation between cyclic resistance ratio and cone penetration resistance (at an overburden stress of 50 kPa and 15 uniform cycles of loading) across a sequence of shaking events (Darby et al. 2016; Geo-Congress, Phoenix, ASCE)

PATHS FORWARD FOR ASSESSING LIQUEFACTION IMPACTS: INCREASED PHYSICS, MODEL VALIDATION, AND EXPLICT UNCERTAINTIES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

The development and effects of liquefaction-induced settlement on structures and lifelines (but also lateral spreading and flow failures) are complex, and subject to ongoing uncertainties in our collective understanding of the salient physics, as well as uncertainties in site-specific characterization, and potential future seismic loading.

2.1 CURRENT STATE-OF-THE-ART

Liquefaction-induced impacts resulting from settlements (as well as lateral spreading and flow failure) are assessed in a principally empirical manner, based on stress-based procedures and the factor of safety concept. Empirical extensions from triggering models to impacts are inherently tied back to the factor of safety concept. The consideration of numerical analyses are rare, and usually reserved for research endeavors. Recent models for triggering have begun to explicitly provide uncertainty quantification (in the regression, but not in the underlying data itself), but impact models for settlements or other impact indices (such as LPI or LSN) do not consider uncertainties (resulting from uncertainty in triggering models, the use of alternative triggering models, or triggering-to-impact metrics such as how volumetric strains are computed).

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Scientific understanding naturally starts with observations. The collection ('data assimilation'), and first-order interpretation of observations can lead to anecdotal trends and the subsequent development of empirical models (requiring implementation 'verification') for understanding physical phenomena and predicting future behavior, and over time the incorporation of additional physics yields greater predictive power and assimilation of data from observations. These models are then used for prediction, and subsequently these predictions are compared with observations, via either retrospective, or ideally prospective validation. Figure 1 illustrates the inference spiral of system science which embodied these ideas. It is a spiral because of the fact that processes of data assimilation, model development and verification, prediction, and model validation are processes which are ongoing. Outward movement on the spiral implies increasing data assimilation, improved model theories and required computation, increased predictions, and increases in predictive capabilities as evident through validation.

There are several aspects that are presently hindering the predictive capabilities of liquefaction-induced impact models from moving outward on the inference spiral:

1. A focus on empirical prediction models, limiting the amount of information that can be assimilated from case-histories; as well as information about the nature of seismic loading and site characterization that can be utilized in forward predictions; a complicating a unified consideration of laboratory, field, and numerical modeling insights;
2. A lack of architecture (both conceptual framework and computational systems) to provide validation data, and to enable coordinated validation activities to assess, in a transparent manner, the predictive capabilities of existing prediction models, as well as models under development.

3. Explicit consideration of uncertainties that exist in prediction models as well as seismic loading and site characterization.

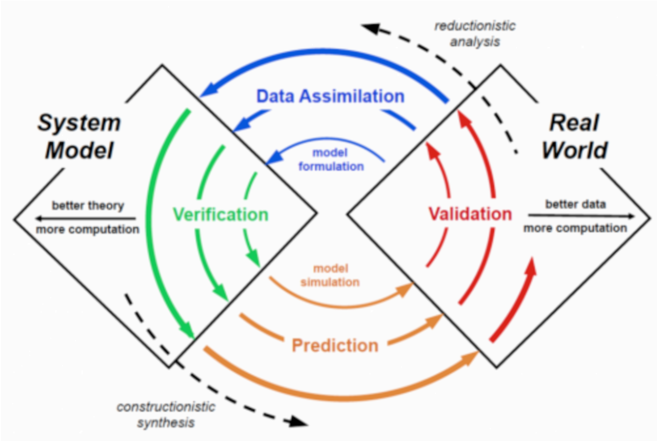

Figure 1. The inference spiral of system science illustrating the concepts of data assimilation, model verification, model prediction, and model validation. Moving outward on the circle implies more data, theories and computation (after Jordan, 2015).

2.5 PATHS FORWARD

As noted above, a predominant focus on empirical models, lack of systematic validation, and only implicit consideration of uncertainties are hindering advances in liquefaction impact models. Possible paths forward in each of these areas are summarized below.

The consideration of empirical models leads to over-simplification in both the representation in the seismic loading as well as the geotechnical site conditions (and importantly the interaction between different geotechnical layers and any overlying structures during seismic shaking). Seismic demands are conventionally considered simply via PGA and Mw. With earthquake-induced ground motion modeling now providing ground motion time series (i.e. acc vs. time), a more explicit means by which this complex time series is directly utilized is needed (either via determining an equivalent loading or by using the time series directly). The currently simplistic treatment of seismic loading immediately limits the precision of liquefaction impact models. On the capacity side of the equation, relatively detailed geotechnical information is often collected to characterize the site conditions of a particular soil column (e.g. SPT, CPT, etc.), however, the simplified procedure by which the cyclic strength of soil deposits are considered neglects the 'system' interactions that occur during dynamic loading, both in terms of the migration of excess pore pressures, but also the manner in which the dynamic response of layers that undergo stiffness reduction changes the loading transmitted to surrounding layers. Directly capturing such features requires physics-based explicit site response modeling, and such integrated modeling (in which soil behavior is described via constitutive models rather than simply 'liquefaction resistance') also offers the opportunity of a more explicit integration of laboratory-, field-, and numerical-simulation-based research endeavors.

The consideration of more advanced physics-based models requires sufficient validation to ensure predictive capabilities which are superior to empirical models. While validations of liquefaction models have occurred in the past, they are often performed by the same people who have developed the models, and with a subset of available validation data- this results in a lack of transparency in the process. To develop a better understanding of predictive capabilities, advance research on identified weaknesses, and achieve greater utilization, a more formal framework and computational architecture for validation is needed to evaluate existing and under-development models. The PEER NGL database and LEAP projects are current activities in this direction, but substantially more is needed.

Finally, explicit treatment of uncertainties in liquefaction impact models are needed; where current treatment lags behind that seen in engineering seismology and structural engineering models. A greater emphasis on validation, as described above, will allow for an improved assessment of model uncertainties, and we will also start to see that some uncertainty, e.g. in the form of spatial variations in soil properties (present in reality), is in fact necessary to improve the quality of our model predictions.

ARE STANDARD OF PRACTICE PILE PINNING PROCEDURES FUNDAMENTALLY INCORRECT?

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED LATERAL SPREADING ON STRUCTURES AND FOUNDATIONS

Deep foundations are often analyzed for lateral spreading effects by imposing a free-field ground displacement profile on the free ends of p-y elements attached to the pile beam element. A number of case histories of bridges founded in liquefiable soils indicate that the free-field displacement is excessively conservative, as piles are predicted to fail in cases where they did not. Standard-of-practice procedures have not been adequately validated with field case history observations, and involve assumptions that are suspect in my opinion.

2.1 CURRENT STANDARD-OF-PRACTICE

Rather than "State-of-the-art", as requested in the prompt, we have chosen to address the "Standard-of-practice" here. The current standard-of-practice is:

1. Perform a pseudo-static limit equilibrium slope stability analysis corresponding to a particular pile pinning force, compute the yield acceleration (k_y) and Newmark displacement, repeat for a number of pinning force values, and plot Newmark displacement versus pinning force,
2. Perform a pushover analysis by imposing soil-displacements on the free-ends of the p-y elements, and plot shear force in the piles at the depth of the sliding plane versus soil surface displacement.
3. Find the point of intersection of 1. and 2. This is considered the "pinning-compatible" displacement appropriate for computing demands on the pile foundation.

Although there are many aspects of this procedure that warrant attention of the workshop participants, we would like to focus on one specific item. The areal extent of the lateral spread feature relative to the size of the pile foundation is treated in vastly different manners by different researchers. In some cases pinning is only applied when the out-of-plane width of the spreading soil is small relative to the width of the pile group (e.g., Boulanger et al. 2007, Ashford et al. 2011), while other documents (NCHRP 472) suggest that the out-of-plane width of the spreading soil should be taken as being equal to the width of the pile group. Turner and Brandenberg (2015) hypothesize that pinning is appropriate only when the areal extent (i.e., length and/or width) of the lateral spread feature is small enough to influence mobilization of the load transfer mechanism between the foundation elements and the spreading soil.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Fig. 1a shows a single pile in a lateral spread that is large in both horizontal directions relative to the foundation. In this case, the areal extent of the lateral spread feature is large enough to fully encompass the zone of influence of soil-pile interaction, therefore the free-field soil displacement is the appropriate input for the free ends of the p-y elements. The soil displacement in the vicinity of the pile will be smaller than the free-field displacement, and the pile might be considered to have "pinned" back the soil. We reject this definition of pinning, and define pinning as being a reduction in demand imposed on the foundation element due to soil-structure interaction. This latter definition is consistent with the manner in which most people use the term "pile pinning". We contend that the free-field displacement is the correct input when the areal extent of the lateral spread is so large that the full load transfer mechanism is allowed to develop. Note that the load transfer stiffness between the nonliquefied crust and the pile may be significantly softer than for a nonliquefied profile due to the influence of the underlying liquefied layer on the distribution of stresses within the nonliquefied crust (Brandenberg et al. 2007).

Fig. 1b shows a lateral spread feature that has essentially infinite with (perpendicular to the direction of lateral spreading), but a finite length (parallel to the direction of lateral spreading). If the length is small relative to the width of the foundation elements, the passive failure mechanism behind the piles may be altered. This introduces a pinning effect because the full load transfer mechanism cannot develop in this case. Imposing a free-field displacement on the pile foundation in this case would over-estimate the demands imposed on the foundation. We argue that the ultimate capacity and stiffness of the p-y elements should be altered in this case to account for the alteration to the load transfer mechanism compared to what would occur in a lateral spread with infinite extent. Fig. 1c shows a lateral spread feature that has a finite width and length, which further contributes to pinning.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The size of a lateral spread feature relative to the size of the foundation elements is a key consideration in assessing pile pinning. Challenges to properly accounting for the behavior are:

1. The size of lateral spread features can be challenging to define a priori. These features are often controlled by minor geological details that are difficult to predict before an earthquake, and sometimes even challenging to identify after a lateral spread has occurred.
2. Procedures for estimating the ultimate capacity and load transfer stiffness of pile foundations interacting with finite width and/or length lateral spread features currently do not exist, but would need to be formulated to implement a method that accounts for this behavior.
3. The pile pinning procedure has already gained widespread acceptance among the geotechnical earthquake engineering community, and it is not likely to be replaced quickly by a procedure that better accounts for fundamentals. This is particularly true if a new procedure predicts worse performance than the existing pinning methods.

2.5 PATHS FORWARD

Clarifying the fundamental load transfer mechanisms between deep foundations and lateral spreads of finite areal extent is crucial for the safe design of structures to resist lateral spreading demands, and also to reduce excessive conservatism. Existing pinning procedures may be unconservative, or overly conservative depending on the specifics of a particular problem. An effort is needed to bring together researchers who have studied this issue using physical modelling studies, field case studies, and numerical simulations to better understand the underlying mechanisms, and develop more rational procedures for analysis of pinning effects.

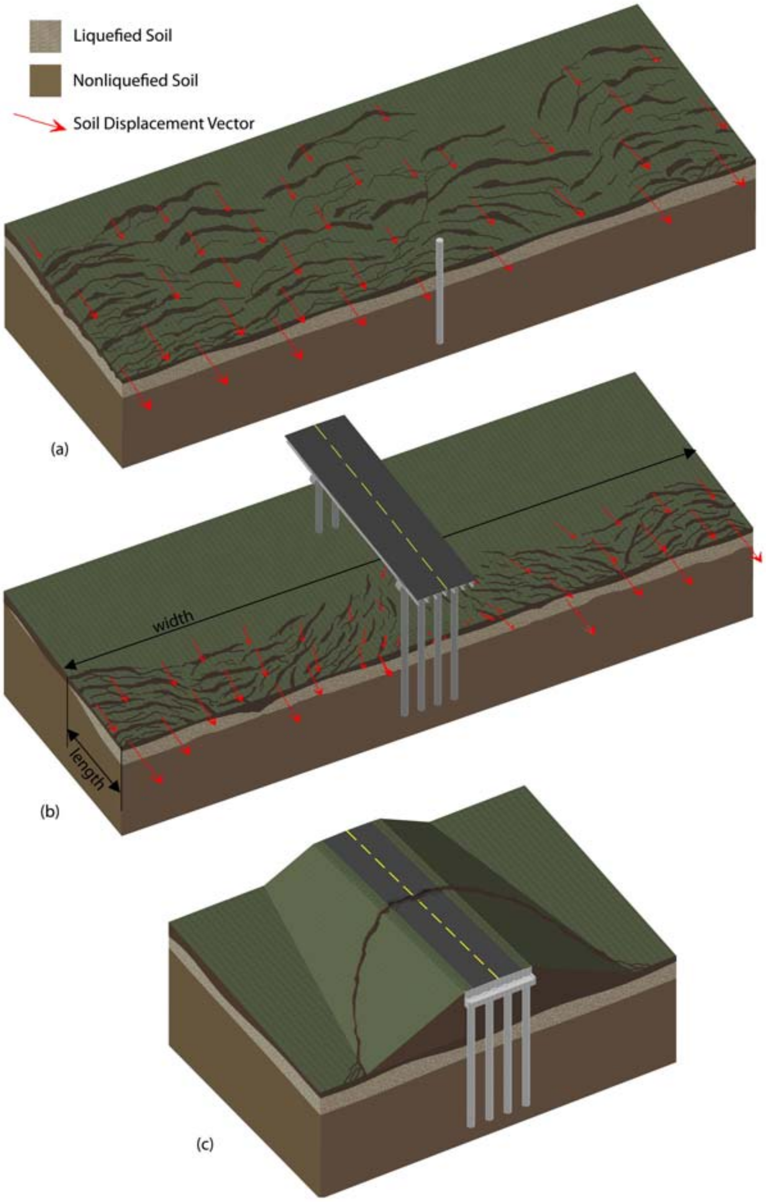

Figure 1. Relationship between liquefaction-induced lateral spreading and soil type.

3.0 ACKNOWLEDGEMENTS

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LIQUEFACTION-INDUCED BUILDING SETTLEMENT

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Shallow foundations at sites with a shallow liquefiable layer can undergo shear-induced ground settlement as well as settlement due to the removal of soil from beneath its foundation through sediment ejecta. These mechanisms are not captured and hence cannot be estimated using only 1D post-liquefaction reconsolidation procedures. The 1D procedures are applicable for level-ground sites with no influence of the overlying structure. Therefore, 1D procedures should not be relied upon solely for evaluating the seismic performance of shallow foundations at potentially liquefiable sites.

2.1 CURRENT STATE-OF-THE-ART

Recommendations for evaluating the seismic performance of shallow-founded structures at liquefiable soil sites are provided in Bray et al. (2016). The engineer should gain insight through these steps:

1. Perform liquefaction triggering assessment and calculate 1-D post-liquefaction volumetric reconsolidation settlements.
2. Estimate the likelihood of sediment ejecta developing at the site by using ground failure indices such as Ishihara (1985) and LSN (van Ballegooy et al. 2014). Estimate the amount of foundation settlement as a direct result of loss of ground due to the formation of sediment ejecta. Use relevant case histories to estimate the amount of ejecta, and assume the ejecta have been removed below the building foundation.
3. Perform bearing capacity analyses using post-liquefaction strengths of liquefied soils. If the post-liquefaction bearing capacity factor of safety (FS) is less than about 1.5 for light to medium size buildings or the post-liquefaction bearing capacity FS is less than about 2 for heavy or tall buildings, large movements are possible, and the building's seismic performance is likely unsatisfactory.
4. Perform nonlinear effective stress soil-structure-interaction (SSI) analyses to estimate building movements that includes shear-induced deformation.
5. Use engineering judgment. Through identification of the key mechanisms of liquefaction-induced building movement, simplified and advanced analyses can be used to provide valid insights. However, case histories and judgment are equally important to consider.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

There are numerous important geologic processes and details. The depositional processes determine the soil fabric which in turn affects pore water pressure generation and migration and hence ultimately building settlement. Some of these effects can be partially erased by previous liquefaction events. Soil layering at a site at several scales also affects pore water pressure dissipation. Many of the case histories where dramatic liquefaction-induced settlement occurs are relatively thick clean sand sites. However, silty soil deposits can be troublesome if loaded by heavy or tall buildings (e.g., Bray and Sancio 2009).

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Liquefaction-induced building movements result from volumetric-induced deformation, shear-induced deformation, and loss of supporting ground due to the formation of sediment ejecta (e.g., Bray and Dashti 2014). Some of these mechanisms are shown in Figure 1, which include: (a) ground loss due to soil ejecta; (b) shear-induced partial bearing capacity failure due to cyclic softening; (c) SSI shear-induced building ratcheting during earthquake loading; (d) volumetric strains due to sedimentation of the soil structure after liquefaction; and (e) post-liquefaction reconsolidation settlement. All of these mechanisms can contribute to liquefaction-induced building settlement.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The primary challenges to developing reliable evaluation procedures are: (a) characterization of the soil profile including its spatial variation, (b) modeling all key features of soil response with a robust constitutive model, (c) the inherent "brittleness" of the liquefaction phenomenon wherein small changes in soil response can lead to vastly different responses, (d) characterization of earthquake ground motions, (e) the challenges in modeling SSI, and (f) the need for efficient, calibrated analytical methods, among other factors.

2.5 PATHS FORWARD

Developing well documented case histories that can be used to critique and to develop analytical procedures is necessary. Centrifuge studies are helpful in exploring mechanisms. Efficient, robust numerical simulations that consider the inherent variability of natural soil deposits are required.

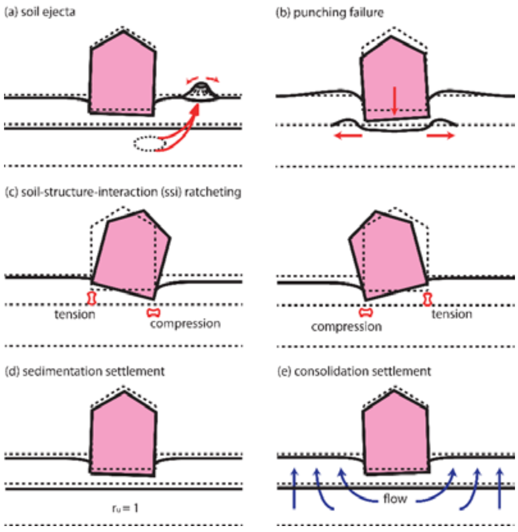

Figure 1. Liquefaction-induced displacement mechanisms: (a) soil ejecta; (b) punching failure, (c) SSI shear-induced ratcheting; (d) sedimentation and (e) consolidation (adapted from Bray and Dashti 2014).

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UNDERSTANDING THE MECHANICS OF EARTHQUAKE-INDUCED FLOW LIQUEFACTION: FROM OBSERVATIONS TO PREDICTIONS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL

Recent major seismic events in New Zealand and Japan have shown that a significant portion of earthquake-induced damage to the natural and built environment is related to ground failure associated with soil liquefaction, a phenomenon that mostly occurs in saturated loose sandy soils during earthquakes. The catastrophic effects of liquefaction are most evident in sloped ground, where the liquefaction-induced total loss of soil shear strength and stiffness results in very large horizontal ground deformation (flow liquefaction). While the consequences of such flow liquefaction have been well documented, there is still a lack of knowledge as to the mechanics of earthquake-induced flow liquefaction due to the insufficient understanding of the combined effects of key factors such as slope ground conditions, earthquake characteristics, confining stress level, soil density, fines content, soil structure and fabric etc., which limits the ability to foresee susceptible soils in advance and thus potentially catastrophic failures occurring.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Spanning from purely theoretical standpoint to practical applications, there is a particular interest to enhance understanding of the effects of static shear (i.e. slope inclination) on the failure mechanisms (i.e. failure induced by liquefaction and/or brought about a large deformation extent) of sand subjected to undrained cyclic shear loading.

In an attempt to address this issue, the Author performed a preliminary series of undrained cyclic torsional simple shear tests on saturated Toyoura sand specimens under various combinations of static and cyclic shear stresses (Chiaro et al., 2012). This was possible because of the use of an innovative large-strain torsional shear apparatus developed at the University of Tokyo (Kiyota et al., 2008). Compared with conventional triaxial and simple shear devices, this apparatus is capable of realistically simulating the large deformation behavior that liquefied soil exhibits during earthquakes. Through this preliminary study a more rational and accurate evaluation of liquefaction potential of sandy soils in sloped ground was obtained and a method for the assessment of earthquake-induced flow liquefaction was developed (Chiaro et al. 2015, 2016).

It is known that the resistance to liquefaction of sands depends on the soil properties as well as on the stress conditions such as confining pressure, cyclic shear stress and initial static shear stress. In order to take the above factors into account, the proposed predictive method (Fig. 1) is defined by means of three fundamental parameters namely: (i) static stress ratio, SSR (= τ_{static}/p_0'), which corresponds to the driving shear force induced by the inclination of slopes; (ii) cyclic stress ratio, CSR (= τ_{cyclic}/p_0'), that represents the inertial force exerted by earthquakes; and (iii) undrained shear strength ratio (USS = τ_{und}/p_0'), where τ_{und} is expected to vary depending on initial relative density (D_r) and effective mean principal stress level (p_0'), among other factors. Moreover, by plotting the experimental data (Chiaro et al., 2012) in terms of η_{max} (= (SSR+CSR)/USS) vs. η_{min} (= (SSR-CSR)/USS), a four-zone graph with well-defined boundary conditions was established. Each zone corresponds to a distinct liquefaction/failure behaviors observed in the laboratory (Chiaro et al., 2012, 2015), namely flow liquefaction (severe liquefaction zone); cyclic liquefaction (moderate liquefaction zone); failure induced by accumulation of large plastic deformation (shear failure zone); and no failure and no liquefaction (safe zone).

Using the proposed predictive method, the liquefaction-induced failure of a very gentle sloped ground that occurred in Ebigase (Japan) during the 1964 Niigata Earthquake (Mw = 7.5 and a_{max} = 0.16g) was carefully evaluated (Chiaro et al., 2016). Similar to field observation, predictions confirmed that given the sloped ground conditions, under such strong earthquake, severe liquefaction could happen only within the intermediate loose sandy soil layer, approximately at a depth in-between 3.5 m and 6.5 m below the ground surface. On the other hand, for the denser soil elements, liquefaction could not be triggered by the earthquake.

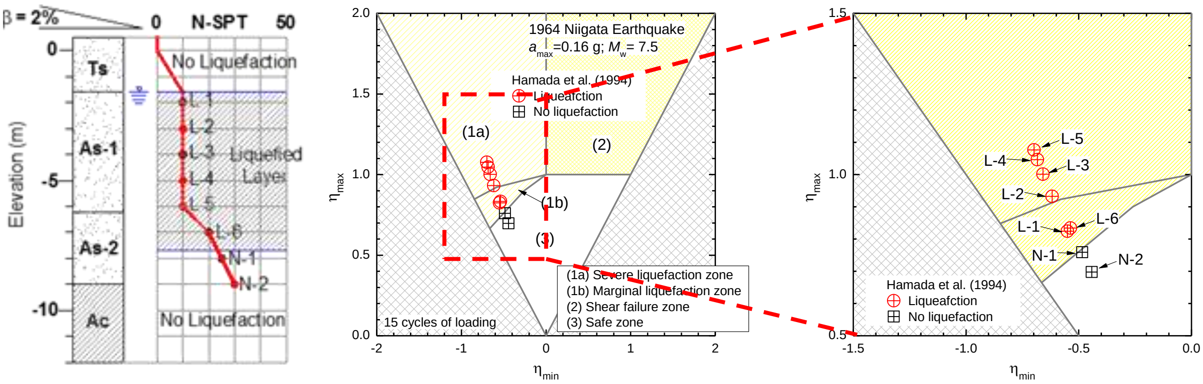

Figure 1. Comparison between observed and predicted failure behaviors of a very gentle slope in Ebigase during the 1964 Niigata Earthquake (Chiaro et al., 2016)

Note that this study was conducted on an idealized sand (i.e. Toyoura sand), and thus the obtained findings are not exhaustive. Moreover, the predictive method is not always directly applicable to all types of liquefiable sandy soil, which have different fines content and structure/fabric compared to that of an idealized clean sand. Accordingly, further comprehensive investigations on fines-containing sands are now planned to supplement past studies undertaken by the Author and yield new insights on the fundamental mechanics of earthquake-induced flow liquefaction of sandy soils not previously possible.

3.0 ACKNOWLEDGEMENTS

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PUZZLING PATTERNS OF LIQUEFACTION MANIFESTATION (OR LACK THEREOF) FOLLOWING THE 2011 CHRISTCHURCH EARTHQUAKE

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Generally speaking, CPT-based simplified methods reliably back-predict the occurrence of liquefaction triggering at sites in which surface manifestations of liquefaction (sand boils, excessive settlement, etc.) were observed during the 2011 Christchurch earthquake. However, CPT-based methods have resulted in a significant number of "false positive" back-predictions across the city for which significant liquefaction manifestation was predicted but no liquefaction damage was observed. This over-prediction of liquefaction effects has costly consequences for rebuilding efforts and insurance claims throughout Christchurch, and is potentially driving overly-conservative liquefaction design in similar soil types worldwide.

2.1 CURRENT STATE-OF-THE-ART

The state-of-the-art for predicting/estimating the overall liquefaction "severity" at level-ground sites revolves around the calculation of several liquefaction severity parameters (e.g., LPI, LPI_{ISH}, LSN, S_{V1D}) and their perceived correlation to liquefaction–induced damage. These liquefaction severity parameters generally require some sort of integration and/or summing of the factor of safety down to a common depth reference in order to establish a single number that represents the expected severity of liquefaction. For example, the 1-D vertical reconsolidation settlement [S_{V1D}; Yoshimine 1992, Zhang et al. 2002] is commonly used to determine if ground improvement and/or a more robust foundation system is required at a site. However, S_{V1D} (and other liquefaction severity parameters) do not always correlate well with actual liquefaction severity/manifestations. For example, consider the S_{V1D} estimates calculated from 12 CPT's at a case history site in Christchurch (refer to Figure 1 and Table 1). Nine of the 12 CPT's are located in areas where surficial manifestations of liquefaction did not occur following the 2011 Christchurch earthquake. However, significant liquefaction triggering and high S_{V1D} values (135mm < S_{V1D} < 175mm) are predicted from all 12 CPT's.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Detailed documentation and subsequent analysis of key liquefaction and no-liquefaction case histories are required to advance the state-of-practice in predicting the severity of liquefaction effects. We need to go beyond liquefaction triggering as a yes or no answer and truly scrutinize performance. Toward this end, our research team has collected detailed observations of liquefaction severity (surface manifestations, structural distress, etc.) and high-quality subsurface data, including SCPT, direct-push crosshole (DPCH) measurements of Vs and Vp, and continuous sonic coring, at 30-plus case history sites in Christchurch. False-positive liquefaction sites were purposely targeted as a means to investigate why we are struggling to properly predict liquefaction triggering and subsequent liquefaction effects (or the lack thereof) in soils with: (a) high fines contents, (b) significant inter-layering, and (c) partially saturated zones below the hydrostatic water table.

2.5 PATHS FORWARD

We have attempted to tackle the problem of false-positive liquefaction predictions using a combined CPT-Vs-Vp approach. While the CPT is extremely sensitive to density (one of the main factors influencing liquefaction triggering and subsequent effects), it is not very sensitive to microstructure and/or issues with partial saturation, which can also play an important role in triggering and manifestation. We are currently analysing data from our 30-plus new case history sites in light of this combined in-situ testing approach. We hope this combined approach will help shed light on puzzling patterns of liquefaction manifestation that we cannot currently understand based on CPT alone.

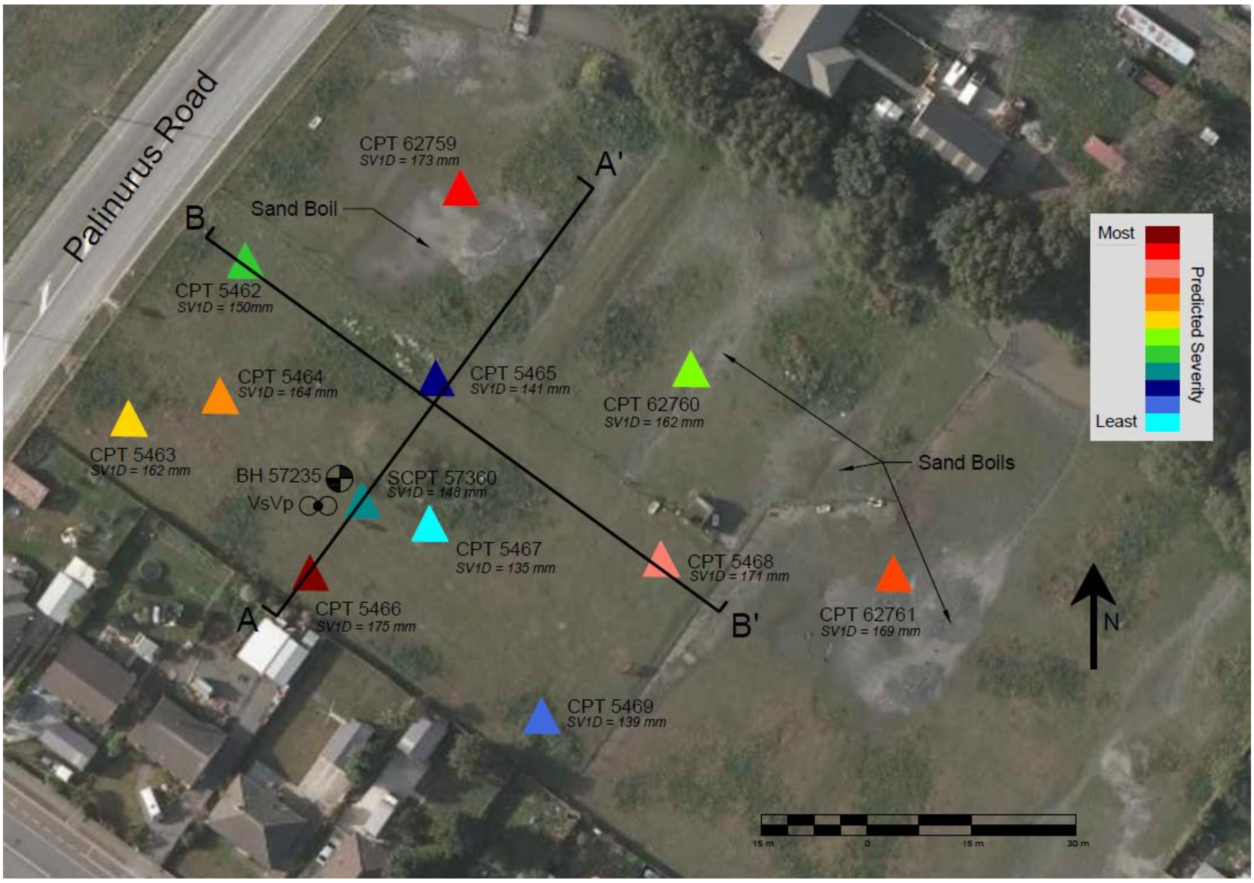

Figure 1. Site plan for Palinurus Road showing extent of liquefaction surface manifestations triggered by the February 2011 Christchurch earthquake (image taken on 24 February 2011, base layer from NZGD). Note that 9 of the 12 CPT's exist in areas where no surface manifestations of liquefaction occurred. However, liquefaction triggering would have been predicted to occur across the entire site, with SV1D values ranging from 135mm – 175mm.

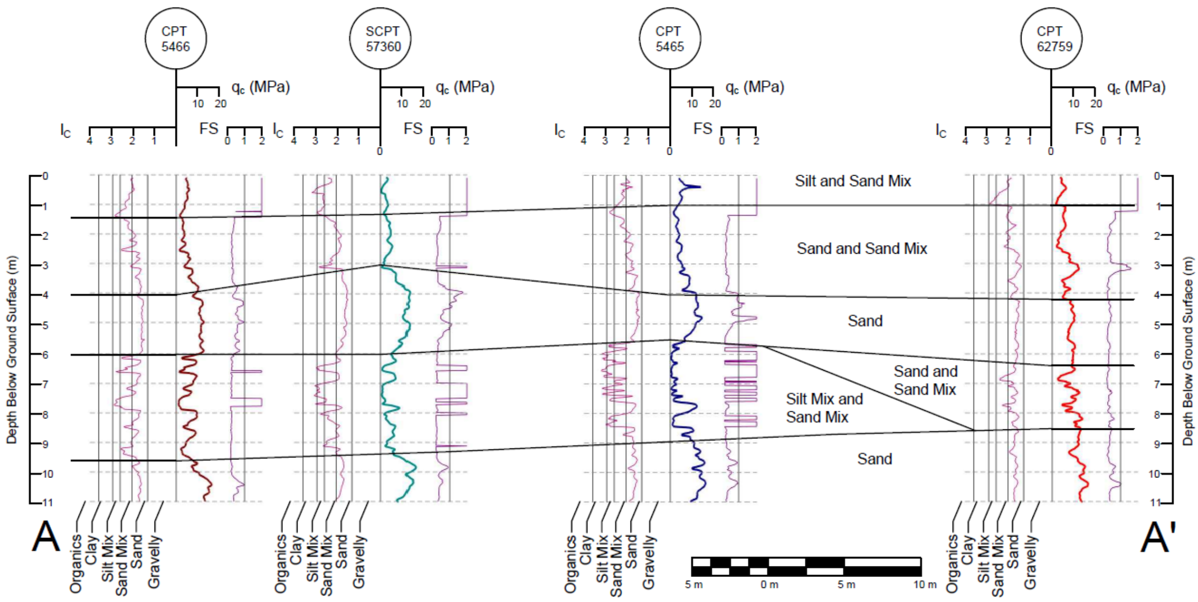

Figure 2. Subsurface cross section A-A' running parallel to Palinurus Road. I_C is shown for February groundwater table conditions and factor of safety is shown for the February 2011 Christchurch earthquake with median PGA and a P_L =15%.

Table 1. Summarized liquefaction severity parameters for all CPTs at Palinurus Road for PGA associated with the February 2011 Christchurch earthquake and a P_L = 15%

Earthquake	Liquefaction	Value	LPI	LPI_{ISH}	LSN	S_{V1D} (mm)	CTL (m)
February 2011	Not Observed	Range	29 - 38	25 - 31	33 - 41	135 - 175	6.2 - 8.2
		Mean	34	28	37	154	7.2
		σ	3.4	2.3	2.8	15	0.7
	Observed	Range	35 - 39	29 - 31	39 - 41	162 - 173	8.2 - 8.6
		Mean	37	30	40	168	8.3
		σ	2	1.2	1	5.6	0.3

3.0 ACKNOWLEDGEMENTS

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LIQUEFACTION-INDUCED LATERAL SPREADING

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

2.1 CURRENT STATE-OF-THE-ART

Current state-of-the-art for lateral spreading assessment is based on empirical methods developed from interpretation of case studies of lateral spreading (e.g. Youd et al., 2002; Zhang et al., 2004; Tokimatsu and Asaka, 1998). These methods essentially provide means for estimating the magnitude and spatial distribution of lateral spreading displacements, and use measures for the seismic demand, density state of soils (approximated through the penetration resistance) and ground geometry as inputs in the assessment.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Lateral spreading is a very complex phenomenon that involves all the complexities of soil liquefaction, plus an intricate interplay of gravity-induced and earthquake-induced mechanisms of ground deformation. It involves a highly dynamic process that is affected by the temporal and spatial evolution of liquefaction, resulting changes in the stress-strain characteristics of soils, and combined effects of ground shaking and gravity-imposed shear stresses that provide the driving mechanism for permanent lateral ground displacements. There are clear dynamic effects in this process that contribute to the resulting magnitude of ground displacements.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The manifestation of lateral spreads in the field is often extremely complex and difficult to characterize. Spreads may involve global movement patterns that are easy to depict by aerial surveying methods, such as LiDAR, satellite images, aerial photography, but may also manifest substantial non-uniformity of ground distortion and variability of ground displacements including size and spatial distribution of ground cracks and fissures, which are observed in local field (ground) surveying measurements (inspections). So, the first challenge is to consistently characterize lateral spreads in the field, and estimate the magnitude and spatial distribution of ground displacements that occurred at these sites (Cubrinovski and Robinson, 2016).

2.5 PATHS FORWARD

A more rigorous characterization of lateral spreads is needed to capture important characteristics of manifested ground displacements, in particular, their magnitudes and spatial distribution. Such approach will allow classification of lateral spreads into different categories, and then development of class-specific predictive models for lateral spreading. Quantification of dynamic effects to lateral spreading displacements can then be attempted.

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EVALUATION AND MITIGATION OF LIQUEFACTION EFFECTS ON BUILDING PERFORMANCE: AN INTEGRATED OBSERVATIONAL-EXPERIMENTAL-NUMERICAL-STATISTICAL APPROACH

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Soil liquefaction regularly causes major damage to buildings and other civil infrastructure in earthquakes. Existing procedures (all deterministic) to evaluate building settlement on softened ground and to design mitigation techniques do not account for: (1) the influence of structure and foundation properties on pore pressure generation and soil settlement; (2) the combined effects of liquefaction-induced soil deformation and ground shaking on structural performance (e.g., drifts and damage); or (3) the influence of fine content and spatial variations in hydraulic conductivity on excess pore pressures, displacements, and accelerations imposed on structures. Further, existing numerical tools to develop more risk-informed, probabilistic methods have not been validated adequately against field or laboratory studies for a range of structure, soil, mitigation, and ground motion characteristics. These shortcomings need to be addressed to realize the benefits of performance-based engineering in the evaluation and mitigation of the liquefaction hazard near structures. A sound predictive model for liquefaction effects on building performance is also desperately needed in catastrophe risk modeling.

2.1 CURRENT STATE-OF-THE-ART

An effective mitigation of soil liquefaction requires a thorough understanding of the consequences of liquefaction in the context of building performance. If soil liquefaction is judged to be likely at a site (e.g., using empirical procedures such as Youd et al. 2001), and if overall stability is not a concern, the consequences of liquefaction are evaluated in terms of settlement at the location of a structure. Today, empirical procedures (e.g., Tokimatsu and Seed 1987) are still primarily used to estimate settlement on liquefiable ground. If the settlements are excessive, mitigation techniques are considered to reduce settlements to an acceptable level or to avoid soil liquefaction.

The existing empirical procedures were developed to capture only volumetric settlements in the free-field. The building's presence is either ignored or considered only as an increased surcharge both in estimating settlement and in designing mitigation strategies. These methods ignore mechanisms that damage the building and its surroundings, e.g., deviatoric deformations and volumetric strains due to partial drainage (Dashti et al. 2010), and were repeatedly proven inadequate in previous earthquakes. Further, the influence of permanent settlement, tilt, and shaking on structural response and damage is unclear because existing models rarely, if ever, consider the structure and liquefying soil as a system. When modeling the building as a rigid foundation or a linear-elastic structure, one cannot represent damage propagation and period elongation, which are important particularly when using mitigation. Lastly, the influence of fine content and variations in hydraulic conductivity (present in many case histories and sites) on the resulting deformations and effectiveness of different mitigation techniques in terms of building performance is poorly understood. Improved procedures are needed to estimate and mitigate liquefaction damage considering variations in site conditions and presence of fines, building nonlinearities and performance objectives, and the inherent uncertainties.

2.5 PATHS FORWARD

The robust evaluation of the performance and damage potential of constructed facilities on potentially liquefiable soils poses a critical gap in earthquake engineering knowledge. An integrated observational, experimental, numerical, and statistical approach is proposed to investigate the behavior of inelastic buildings on liquefiable soils with a range of mitigation measures and to develop robust, performance-based procedures for the evaluation and mitigation of effects of liquefaction on buildings. A primary goal is to evaluate the performance of the soil-mitigation-foundation-structure system holistically and to evaluate potential tradeoffs of liquefaction mitigation, which reduce pore pressure generation and settlement but increase ground shaking intensity, aggravating building damage. To evaluate these effects, the structures need to be designed to be capable of damage (nonlinear and potentially inelastic). With this better understanding, engineers can design mitigation to improve performance at a system level.

Previous case histories have shown satisfactory performance of some (not all) mitigation techniques. Although valuable insight can and must be drawn from case history data, the complex interactions among the soil, foundation, mitigation, and the superstructure cannot be studied systematically via case histories alone. However, case-history observations can guide and validate experimental and numerical approaches. It is proposed to build on recent case histories and physical model studies by examining selected independent variables, with the aim of developing a reliable performance-based mitigation design methodology (see Fig. 1) following these steps:

1. Experimentally evaluate the combined effects of building geometry and properties, soil and mitigation (e.g., drains) properties, and characteristics of ground motion on soil-foundation-structure interaction (SFSI) and structural performance in terms of key engineering demand parameters (EDPs) that control damage and loss: settlement, tilt, flexural drift ratio, and plastic hinging, etc.
2. Calibrate and validate nonlinear numerical models of the soil-foundation-structure systems that were studied experimentally for a range of soil, structural, and mitigation properties.
3. Perform a comprehensive numerical parametric study for the conditions not considered experimentally. These studies will improve the understanding of underlying mechanisms, trends, and key damage predictors. Parallel computing and access to super computers is essential for this task.
4. Use the data from the parametric study to develop probabilistic predictions of building damage (e.g., permanent settlement and tilt; peak transient interstory drift ratio) on liquefiable ground and propose mitigation levels that achieve a desired level of system performance. The probabilistic predictive models need to be validated with case-history observations.

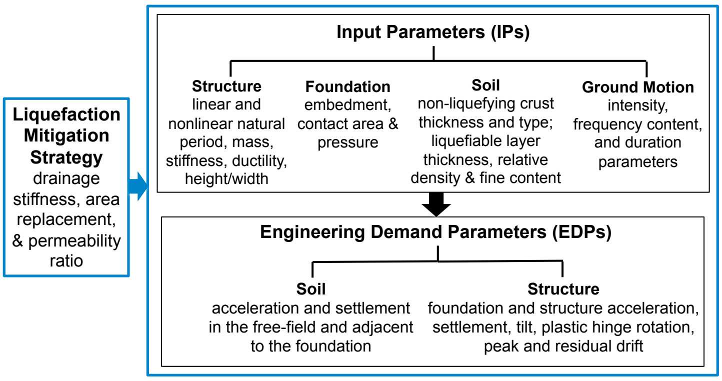

Figure 1. The proposed framework for the performance-based assessment of liquefaction effects.

3.0 ACKNOWLEDGEMENTS

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OPTIMIZING SITE CHARACTERIZATION TO EVALUATE AND INCORPORATE SPATIAL GEOLOGIC STRUCTURE IN 2-D/3-D ANALYSIS OF CASE HISTORIES

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1.0 LIQUEFACTION EFFECTS CHALLENGE:
2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED LATERAL SPREADING

[2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED SETTLEMENT]

2.1 CURRENT STATE-OF-THE-ART

The analysis of liquefaction induced lateral spreading (and to a similar extent settlement analyses) often includes oversimplifications of the geologic structure which have recently been shown to contribute, in some cases significantly, to over estimation of lateral movements following earthquake loading. While this simplification is most explicitly embedded in simplified 1-D analysis methods where infinite lateral continuity of layers is a fixed assumption when analyzing a single CPT profile, it also often exists in more complex 2-D analyses because there is limited CPT data available and/or there is a lack of understanding of the spatial geologic structure.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

The spatial geologic structure of natural deposits directly influences the fraction and connectivity of liquefiable zones as well as the characteristics of any non-liquefiable materials that comprise the balance of the subsurface. Liquefaction induced lateral spreading, and to a lesser degree settlements, often occur in alluvial depositional environments whose local structure is dependent on factors including, for example, channel width, meander rate, avulsion frequency, river gradient, canyon/valley width, and flow variability.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Significant deformations often manifested at the surface in case histories are often due to connected zone(s) of localized deformations that form through layers/zones/lenses of different materials. In the case of lateral spreading, one continuous zone through liquefiable sands and interbedded silts/clays may be sufficient for significant deformations to be realized. It is the composite soil resistance along this zone that provides the resistance to deformation.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Two of the challenges limiting current procedure advancement is (1) the site characterization that is completed when the case history is investigated in the field is often inadequate, and (2) the subsequent conditioning of the subsurface model on the site investigation data available when performing analyses.

2.5 PATHS FORWARD

I am interested in (1) developing an approach to optimize, in near real-time, the number and spacing of CPT soundings, for example, that are performed at a case history or project site based on the overall geologic model and conditioned on site data (CPT, SPT, geophysics) as it is obtained and (2) using conditioned realizations of the subsurface based on reconnaissance data and the geologic model to better define the location and continuity between zones where liquefaction may have occurred. An example of the former is presented in Figure 1 for a hypothetical case of a simulated alluvial deposit

(Figure 1A). Conditioned realizations based on synthetic CPT profiles obtained performed on a grid pattern and when performed in a nested grid to better map the correlation lengths present in the geologic structure are presented in Figure 1B and C. The improved definition of a realistic spatial structure is evident in Figure 1C. For the latter idea, Figure 2 contains an example where 34 CPT profiles along the planned alignment of a new dam have been used in combination with expected loading conditions to identify zones where liquefaction is probable.

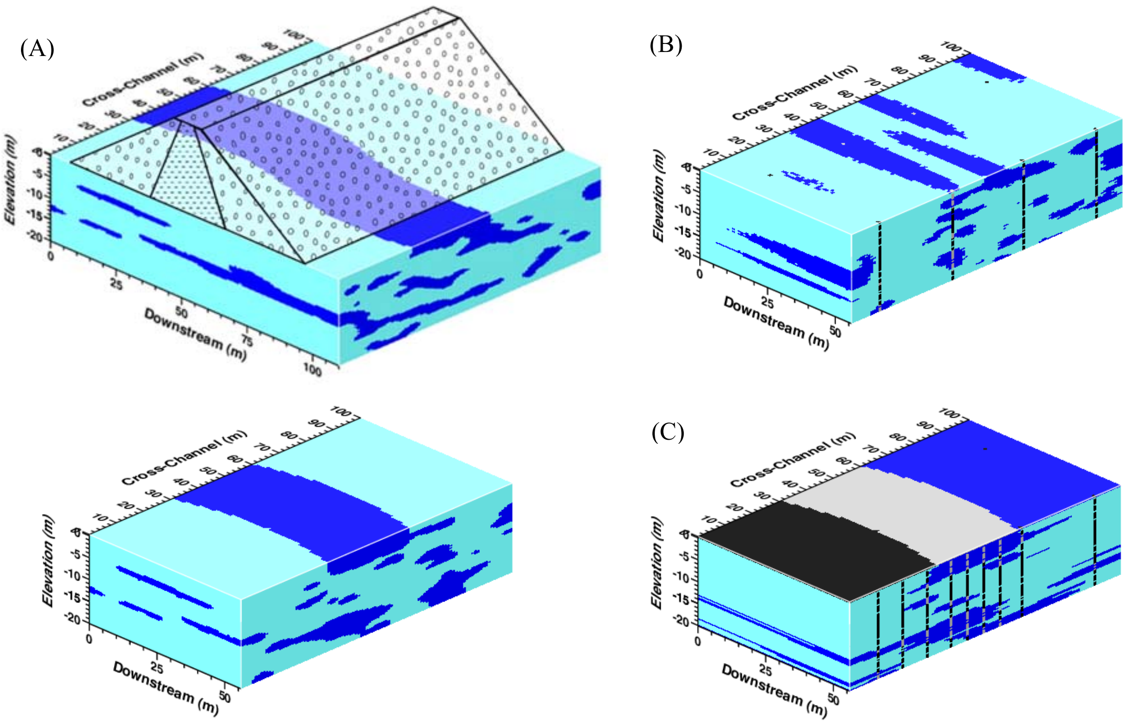

Figure 1. (A, left side) Simulated alluvial river deposit as full realization (top left) and as vertical cut along river (bottom left), (B, top right) Conditioned realization based on wide grid CPT coverage, and (C, bottom right) Conditioned realization based on nested grid coverage defined based on assumed geologic model.

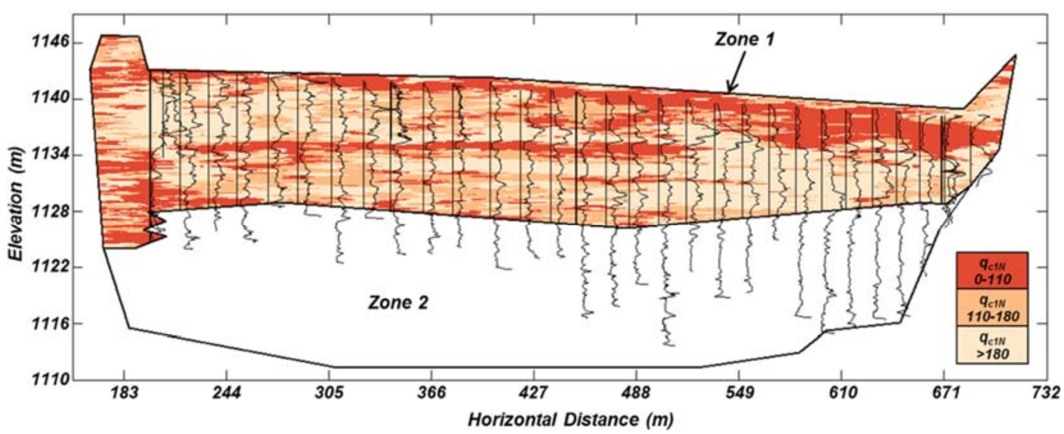

Figure 2. Conditioned realization based on 34 CPT profiles identifying zones susceptible to liquefaction.

3.0 ACKNOWLEDGEMENTS

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LIQUEFACTION-INDUCED SOIL SETTLEMENT AND CONSEQUENCES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – Development and effects of liquefaction-induced settlement on structures and lifelines.

Settlement associated with lateral spreading
Settlement due to post-liquefaction re-solidification
Settlement in presence of shallow foundation structures
Settlement in presence of deep foundations and down-drag effects
Differential settlement due to liquefaction
Inhomogeneous and/or stratified ground
Influence of sand boils
Mitigation approaches

2.1 CURRENT STATE-OF-THE-ART

Correlation of settlement in liquefied ground to peak shear strain
Correlation of settlement in liquefied ground to other "damage" parameters

2.2 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Generation of excess pore-pressure
Degree of disturbance to the soil skeleton
Influence of stratification and consequences

2.3 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Quantification of the mechanism and correlation to appropriate damage parameters
Site Investigation techniques and knowledge about the existing ground condition

2.4 PATHS FORWARD

Physical Testing (large-scale)
Evidence from Earthquake Reconnaissance
Calibrated numerical simulation tools
Mitigation strategies

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ADVANCING THE EMPIRICAL AND SEMI-EMPIRICAL PREDICTION OF LATERAL SPREAD DISPLACEMENTS THROUGH BETTER DEFINITIONS, IMPROVED DATABASES, AND PERFORMANCE-BASED MODEL IMPLEMENTATION

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

This abstract focuses on three key challenges related to the improvement of empirical and semi-empirical lateral spread prediction. Three paths forward are suggested for each of these challenges, and are, in the author's opinion, achievable within the next three to five years.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

<u>Challenge #1: Reaching consensus on the definition of "lateral spread displacement</u>" – Based on the author's observations, there is disagreement and confusion regarding the definition of a lateral spread among many in the geotechnical community. For example, what some engineers would call "lateral spread" others might call "seismic slope displacement." Though this issue may simply seem one of semantics, it has significant consequences in design and leads to many inconsistencies in engineering practice. Displacement predictions from seismic slope displacement models can differ substantially from displacement predictions from regional lateral spread displacement models. This challenge also has implications on the development of lateral spread case histories. For example, many engineers define lateral spread as the horizontal ground deformations that occur in or above liquefied soil due to cyclic mobility during an earthquake. According to this definition, post-liquefaction horizontal ground accelerations are necessary to develop the lateral spread. However, critics of this definition point to eye-witness cases where horizontal ground deformations were observed to occur *after* the strong ground shaking ended. Should these latter cases therefore be classified as flow failures and removed from our lateral spread databases? What if a water film is suspected to have contributed to the displacements?

<u>Challenge #2: Updating and improving the case history databases</u> – Many of the empirical and semi-empirical lateral spread prediction models that are used widely in practice today are based on case histories that were developed prior to 1999. Since that time, numerous earthquakes and lateral spread events have been documented throughout the world, though to varying degrees of quality and reliability. In addition, existing lateral spread databases have significant data gaps including lateral spreads from subduction zone events, small magnitude events, and small- or zero-displacement events. Faster and more reliable methods for measuring lateral spread displacements in the field are needed. Greater consistency between SPT- and CPT-based model lateral spread predictions is needed. Quantification and incorporation of the various uncertainties associated with a lateral spread case history remains a significant challenge.

<u>Challenge #3: Better implementation of our prediction models in engineering design</u> – Conventional analysis procedures for liquefaction effects such as lateral spread tend to be performed in a pseudo-probabilistic manner (Rathje and Saygili 2008), which assumes that the return period of the computed liquefaction effect is the same as the causative probabilistic ground motion. Numerous researchers (e.g., Kramer and Mayfield 2007; Rathje and Saygili 2008; Franke and Kramer 2014) have shown that this assumption is false and that the conventional pseudo-probabilistic approach produces inconsistent and unreliable hazard estimates because it: (1) considers only a single return period for the ground motion, (2) neglects the uncertainty associated with our prediction models, and (3) assigns 100% of the probabilistic ground motion contribution to a single earthquake scenario. A probabilistic or

performance-based analysis approach for lateral spreading (e.g., Franke and Kramer 2014) is built to overcome these challenges, but is difficult for most engineers to implement in practice because of their unfamiliarity with probabilistic approaches and the numerous iterative calculations that are required.

2.5 PATHS FORWARD

Overcoming Challenge #1 – Sadly, there is no "easy" answer for overcoming this challenge. Engineers have been trained in different ways, and local standards of practice will continue to dictate the methods used to predict lateral spread displacements until greater consensus is first obtained in the research community. Fortunately, the development of the NGL lateral spread database provides a unique opportunity to address this challenge in the near future. If definitions of different lateral spread "failure modes" are defined (e.g., slump zone failure, suspected water film, etc.,) and incorporated into the database, then future NGL model developers may naturally focus their models on predicting a particular type of failure mode.

Overcoming Challenge #2 – The pending development of the NGL lateral spread database and linking it with the NGA ground motion database provides a significant opportunity to overcome Challenge #2. Continued contribution of case histories to the NGL database by researchers in the future will be key to keeping the database updated and to preventing "data stagnation." Remote sensing methods and technologies for documenting lateral spreads in the field have improved significantly during the last 15 years (Rathje and Franke 2016) and include the use of LiDAR, satellite-based imagery analysis, and 3D modelling from aerial images collected from unmanned aerial vehicles (UAVs or drones; Franke et al. 2017a).

Overcoming Challenge #3 – A new simplified performance-based lateral spread procedure that uses lateral spread reference parameter maps (Ekstrom and Franke 2016; Franke et al. 2017b) closely approximates the results from the full performance-based procedure (Franke and Kramer 2014), but is sufficiently user-friendly that it can be applied on even the most routine of projects. This procedure was recently applied successfully on a massive pipeline design project in Alaska, and was praised by all of the geotechnical engineers involved because of its simplicity, flexibility, and power (personal communication, T.L. Youd, September 2016).

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EXPLORING NEW APPROACHES TO EVALUATE PARTICLE LEVEL RESPONSES IN LIQUEFIABLE SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

The most common approaches used to estimate liquefaction induced settlements on structures and lifelines typically involve the use of in-situ penetration tests (e.g. SPT, CPT) to assess liquefaction potential and are then complemented by results from laboratory tests that correlate post-liquefaction volumetric strain with Factor of Safety for liquefaction. Along with subsequent empirical corrections for density and fines content, amongst others, the results are then used to estimate 1D free field settlements of level ground due to post-liquefaction reconsolidation. Estimates using this general approach do not account for shear induced displacements nor do they account for building movement.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

A key challenge to developing better evaluation procedures today is that all methods begin with the premise that a penetration test as represented by a resistance value (e.g. N value or Qc) is a "given" and so while many valuable efforts that seek to enhance penetration test based methods have either recently been conducted or are in process, the degree to which they can overcome all the inherent limitations that result from methods rooted in a penetration resistance is ultimately a critical constraint. Such limitations include factors such as the disturbance effects that occurs as a result of the penetration of the device itself (Frost et al., 2016) and the contrast in the volume of soil represented in the penetration measurement versus that reflected in a pore pressure measurement amongst others.

2.5 PATHS FORWARD

While ongoing studies such as those noted above should, without question, be continued in the absence of alternative approaches, it is also timely that efforts to explore new approaches that remove the reliance on current penetration values and their inherent limitations, be initiated. As an example, ongoing field and laboratory studies as well as complementary numerical simulations at Georgia Tech have been exploring the role of a new generation of self-boring devices with textured sleeves that can be loaded axially and torsionally. These studies have shown that different volumes of soil are engaged as a function of loading orientation and that differences in global measurements are directly related to the different manner in which the sleeve texture interacts with the adjacent soil during axial and torsional loading (Figure 1). This is significant given that current penetration devices only measure an average penetration resistance value for a bulb of soil that is of the order of 16 times the diameter of the penetrometer whereas a sequence of textured sleeves such as used in the prototype system can provide 8 measurements of a much smaller volume of soil (Frost et al., 2014) being sheared as a

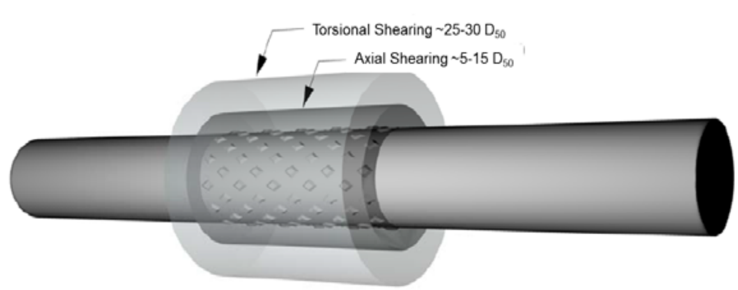

Figure 1: Volume of Soil Influenced/Disturbed by Axial and Torsional Shear.

function of sleeve texture and loading orientation. These multiple measurements can thus be inverted to provide particle level insight (Figure 2) into volume change characteristics and thus a direct measurement of shear induced volumetric strains and thus displacements.

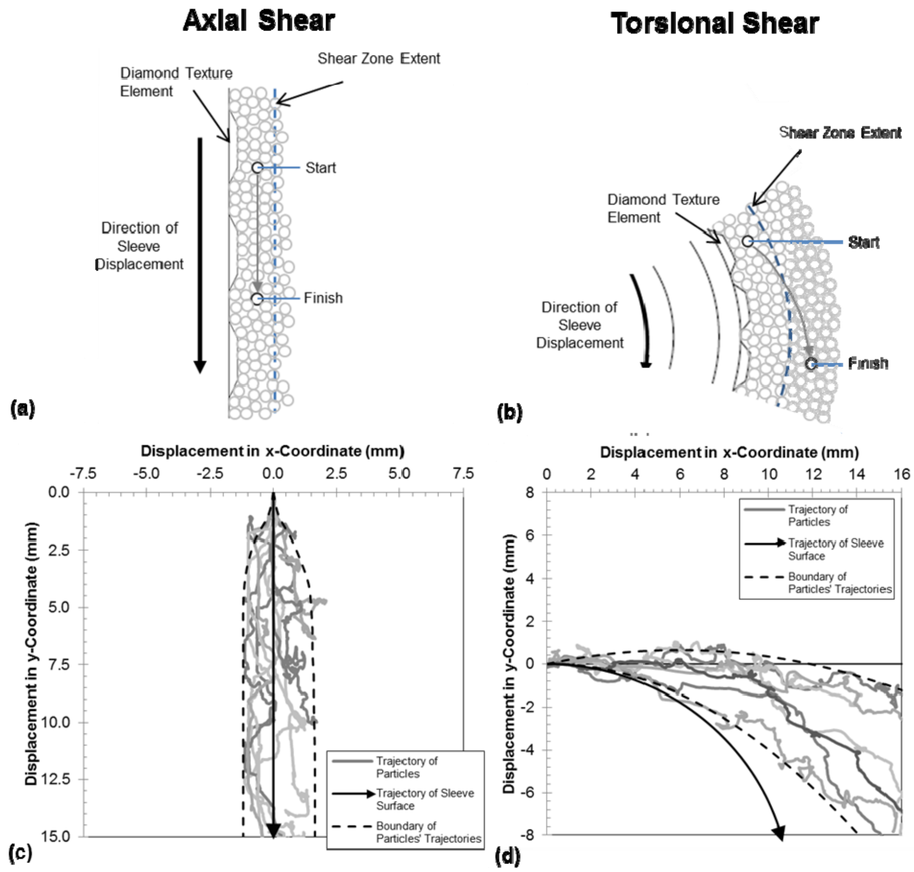

Figure 2. Proposed micro-mechanisms taking place during (a) Axial and (b) Torsional shear. Particle trajectories from DEM simulations during (c) Axial and (d) Torsional simulations against surfaces of R_{max} = 1.00 mm.

3.0 ACKNOWLEDGEMENTS

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USE OF VOLUMETRIC STRAIN IN LIQUEFACTION DAMAGE INDEX FRAMEWORKS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Liquefaction damage indices are used to predict the severity of surficial liquefaction manifestations, which relates to the damage potential of liquefaction to structures and lifelines. These indices bridge the gap between liquefaction triggering predictions and the resulting consequences. Recent damage index frameworks (e.g., Liquefaction Severity Number: LSN) have been proposed that incorporate estimates of post-liquefaction volumetric strain. The motivation for this is that contractive and dilative tendencies of the soil can be taken into account, where these tendencies significantly influence the magnitude of post-liquefaction deformations and, hence, the damage potential of liquefaction. However, the analysis of data from the 2010-2011 Canterbury Earthquake Sequence (CES) shows that the correlation between predicted and observed severity of surficial liquefaction manifestations using these new frameworks is not as efficacious as some alternative frameworks that do not incorporate post-liquefaction volumetric strain (e.g., Maurer et al., 2015). The reasons for this are explored herein.

2.1 CURRENT STATE-OF-THE-ART

van Ballegooy et al. (2012, 2014) proposed the LSN framework to reflect the damaging effects of shallow liquefaction on residential land and foundations based on observations made following the CES. LSN is computed as:

$$LSN = 1000\int\frac{\varepsilon_v}{z}dz \tag{1}$$

where: ε_v is the calculated volumetric strain and z is the depth to the layer of interest. In this framework ε_v is used an index that accounts for the contractive and dilative tendencies of the soil as a function of density.

2.2 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

LSN incorporates the contractive and dilative tendencies via ε_v (i.e., the one-dimensional post-liquefaction consolidation strain) estimated using the method of Ishihara and Yoshimine (1992) (Figure 1). As can be observed from Figure 1, for a given factor of safety against liquefaction (FS_{liq}), soils having a higher relative density (D_r), and thus a greater tendency to dilate upon shearing, undergo smaller post-liquefaction volumetric strains than do looser soils.

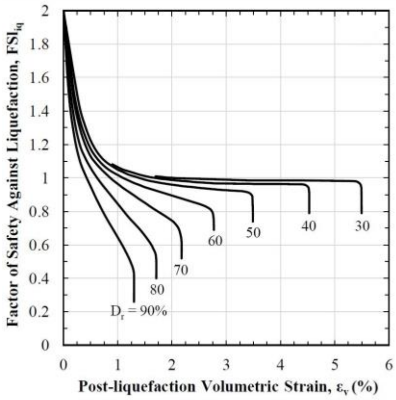

Figure 1. Post-liquefaction volumetric strains versus the FS_{liq} for clean sands as a function of initial D_r (Maurer et al., 2015; after Ishihara and Yoshimine, 1992).

Analyzing data from the 2010-2011 CES, Maurer et al. (2015) showed that assuming a constant value of ε_v of 1% within the LSN framework when $FS_{liq} < 1$ and $\varepsilon_v = 0\%$ when $FS_{liq} \geq 1$ is more efficacious than computing ε_v using the relationship shown in Figure 1. This contradicts the intended purpose of incorporating ε_v into a damage index framework.

The authors hypothesize that the reason for this contradiction is that the upward curvature of the Cyclic Resistance Ratio (CRR) curve with increasing penetration resistance in the simplified liquefaction evaluation procedure (e.g., Idriss and Boulanger, 2008) inherently accounts for the contractive and dilative tendencies of the soil. A similar hypothesis regarding the shape of the CRR curves was put forth by Dobry (1991) in relation to the lack of surficial liquefaction manifestations in a dense soil deposit predicted to have liquefied during the 1979 Imperial Valley, California earthquake. As a result, including ε_v in a damage index framework that uses the FS_{liq} computed via the simplified procedure inherently double counts the contractive and dilative tendencies of the soil. Note that the correlations shown in Figure 1 relating FS_{liq}, D_r, and ε_v were based on laboratory test results in which measured excess pore water pressures were used to define liquefaction. The correlations were not based on a CRR curve derived from field observations where the liquefaction response of a deposit was defined based on the presence or absence of surficial liquefaction manifestations.

2.3 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The authors hypothesize that the lack of efficacy of LSN despite the incorporation of ε_v within its framework is due to the double counting of the contractive and dilative tendencies of the soil. However, this does not imply that these tendencies are optimally accounted for in damage index frameworks that do not incorporate ε_v just because FS_{liq} is computed using the simplified procedure. The challenge for developing a better liquefaction damage framework is to optimally account for the contractive and dilative tendencies of the soil, while keeping the simplicity of implementing the framework on par with LSN.

3.0 ACKNOWLEDGEMENTS

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ABSENCE OF RESIDUAL SHEAR STRENGTH CASE HISTORES FOR MEDIUM DENSE SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL

Excess pore water pressures generated by liquefaction of cohesionless soils reduce the shear strength of the soils and can result in flow slides in sloping ground. Due to limitations in laboratory testing approaches, the undrained residual shear strengths of liquefied soils are generally estimated using empirical correlations developed from flow slide case histories. The correlations developed from the case histories relate penetration resistance to the back-calculated residual shear strength of the liquefied soil in the flow slide. Unfortunately, the number and quality of the case histories is very limited and this has resulted in extrapolating the data and correlations from loose and very loose soil conditions where flow slides have occurred to higher soil densities where no flow slides have been observed. The concern is that this extrapolation results in overly conservative and costly determinations of potential deformation and failure for critical structures.

2.1 CURRENT STATE-OF-THE-ART

The current state-of-the-art and current practice for estimating the undrained residual shear strength of cohesionless soils once they are predicted to liquefy employ empirical correlations developed from flow slide case histories. The residual shear strength of liquefied soil, S_r, is determined as a function of penetration resistance, typically equivalent clean sand Standard Penetration Test (SPT) values of $[(N_1)_{60}]_{cs}$. Some correlations relate S_r directly to SPT blowcount, others relate a normalized S_r/σ_{vo}' value to SPT blowcount, and still others use a hybrid approach. While these different correlations can often predict very different S_r values for a specific SPT blowcount, the principal problem with them is that they are all based on generally the same case histories and these case histories have the following limitations:

1. There are only about 30 case histories employed in the different correlations, and more than half of these case histories are very incomplete with respect to key parameters such as actual penetration resistance measured, level of accelerations sustained at the sites, post-slide slope geometries, and the shear strengths of adjoining soil materials. As a result, there are only about 10 or so case histories considered to be well-documented. This is a very limited number to base critical determinations on.
2. The vast majority of the flow slide case histories involve very loose to loose soils with SPT blowcounts that are less than 10. There are only about 3 to 7 case histories, depending upon the correlation interpretations, with SPT blowcounts greater than 10 and no case histories with SPT blowcounts greater than 15.
3. Several of the key case histories for SPT blowcounts greater than 10 (e.g. 1938 Fort Peck Dam construction slide) are associated with static flow slides, and not seismic loadings. Consequently, the potential for void ratio redistribution may not be the same.
4. All of the case histories are for flow slides that actually happened. There are no case histories associated with non-failure conditions – that is, where soils were triggered to liquefy, but the residual shear strength was high enough to prevent a flow slide failure. As such, the nature of these correlations introduces a conservative bias.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

One of the key mechanisms postulated to occur during liquefaction is the upward flow of excess pore water and the potential for void ratio redistribution beneath impeding soil layers of less permeable material. It is well established that the generation of high excess pore pressures during liquefaction leads to an upward gradient and flow of pore water. However, if less pervious layers or lenses of silt or clay exist within or above the liquefied deposits, they will impede this flow. In such an event, there is the potential for pore water pressures to increase at the base of impeding layer or lens and for the void ratio to then increase. The areas of increased void ratio would then have reduced residual shear strengths. It has even been postulated that water films or blisters might develop beneath the impeding soils leading to localized areas of zero residual shear strength. The reduced residual shear strengths in these localized areas result in an overall reduction in the average residual shear strength of the liquefied soil mass. This process is not captured in laboratory testing.

A key challenge is that there are no known flow slides in the case history databases for conditions where the SPT blowcount exceeded 15 – yet the correlations are often used for soils with blowcounts higher than 15. Many practitioners, when determining potential residual shear strength values of medium dense soils for critical projects such as large dams, end up extrapolating the correlations for soils with blowcounts higher than 15, presumably because of the high consequences associated with dam failure. Are we creating a problem when nature is suggesting that there isn't one? Is it possible that denser soils that have been triggered to liquefy do not generate enough volume of flow to create significant void ratio redistribution? Or are these denser soils sufficiently dilative to remain stable even with limited void ratio redistribution? When discussing this challenge, Professor James Mitchell likes to employ a quote attributed to Carl Sagan who reportedly said "*absence of evidence is not evidence of absence.*" Professor Mitchell implies that even though there are no <u>observed</u> case histories of flow slides occurring in such medium dense soils, it does not mean that they haven't happened, or could happen in the future. Nevertheless, the total absence of flow slides for SPT blowcounts greater than 15 may indicate that current practice is overly conservative and that there should be a high priority to further pursue the residual strength values of medium dense soils.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The key challenges to improving residual strength correlations include the following:

- Strong seismic events are relatively rare and areas where soils have been triggered to liquefy beneath sloping ground are even more limited. Flow slides are even rarer.
- In evaluating the performance of slopes that performed well following an earthquake, it is not immediately obvious that a slope that performed well must have had soils with high residual shear strengths. The slope may not have liquefiable soils, or the liquefiable soils may not have triggered if the ground shaking was not high enough. This has lead to a reluctance to investigate slopes that performed well. These non-failure case histories are not dramatic and there may be a reluctance to use a limited exploration budget on sites that may turn out not to have liquefiable soils present.

2.5 PATHS FORWARD

To improve our understanding of residual shear strength for medium dense soils, it is imperative that medium dense soils under sloping ground conditions be investigated following large seismic events, especially if they performed well. Current liquefaction triggering correlations indicate that sandy soils with SPT blowcounts between 15 and 25 can be triggered to liquefy if the ground shaking is strong enough. The path forward would be to go back to areas which recently sustained strong earthquake shaking (e.g. 2011 Tohoku Earthquake, Japan) and look for sloping ground areas where saturated sandy soils with SPT blowcounts between 15 and 20 exist, and the earthquake loading was sufficient to trigger liquefaction. The conditions and performance of such sites can investigated and non-failure strengths back-analysed for an improved set of correlations.

NUMERICAL MODELING OF LIQUEFACTION AND ITS CONSEQUENCES

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1.0 LIQUEFACTION EFFECTS CHALLENGE:

2 - Development and effects of liquefaction-induced lateral spreading
3 - Development and effects of liquefaction-induced settlement on structures and lifelines

2.1 CURRENT STATE-OF-THE-ART

Empirical approaches are often used to evaluate liquefaction induced lateral spreading and settlement. Numerical tools are now being increasingly relied upon to simulate the effect of soil failure, including liquefaction, on structures. This is facilitated by significant enhancements in analysis speeds due to improvements in available hardware and software. This reliance is likely to continue to increase in support of performance based engineering approaches to infrastructure design.

While important contributions have been made in this field, available numerical analysis tools are far too limited and extensive verification and validation remains elusive. Evaluation of constitutive model performance relative to empirical liquefaction triggering criteria and consequences of liquefaction is limited.

2.2 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Available numerical tools rely on constitutive models that represent the soil as a continuum, and hence are unable to model the transition of the soil to fluid-like response as porewater pressures increase and liquefaction is triggered, and then the transition back to a continuum response as excess pore water pressures are dissipated. This transition is relevant to the estimate of liquefaction-induced settlements or the forces generated due to lateral spreading.

Despite these limitations, existing tools are able to capture important aspects of these phenomena as illustrated in Figures 1 and 2.

2.3 PATHS FORWARD

Improvements in existing constitutive models will help improve the reliability of available numerical tools. Nevertheless, there are a number of future opportunities that can pursued:

1) Systematic validation of numerical tools relative to extensively used empirical procedures.
2) Improve numerical simulation codes to significantly improve analysis speeds.
3) Use of alternative numerical formulations that allow for the transition from solid-like to fluid-like behavior and back.
4) Integrating discrete element modelling with continuum models such that discrete particle interaction can be represented when phase transition occurs.

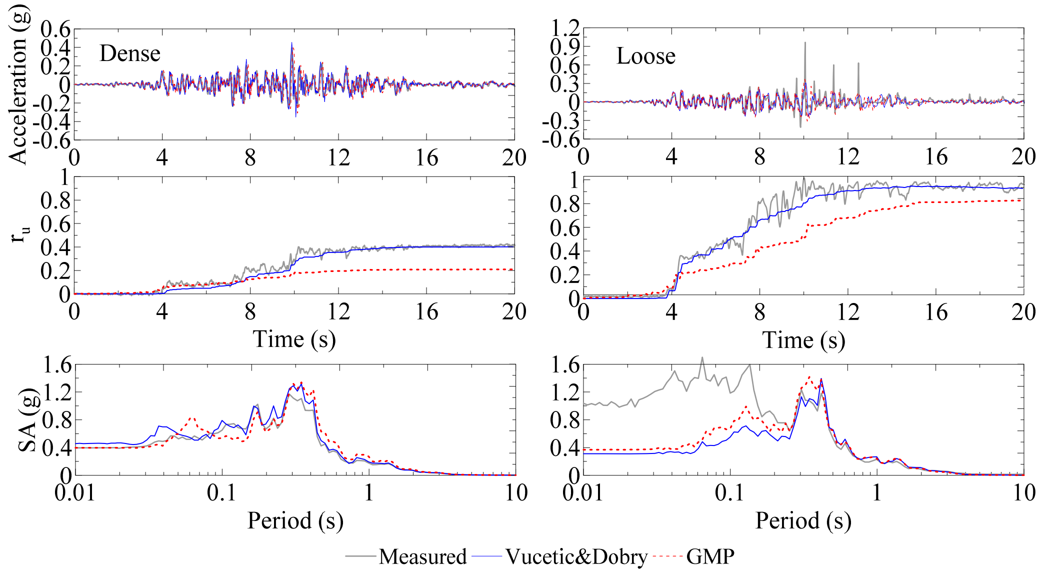

Figure 1. Comparison of measured and computed results in dense and loose layers of centrifuge test subjected to strong ground motion .

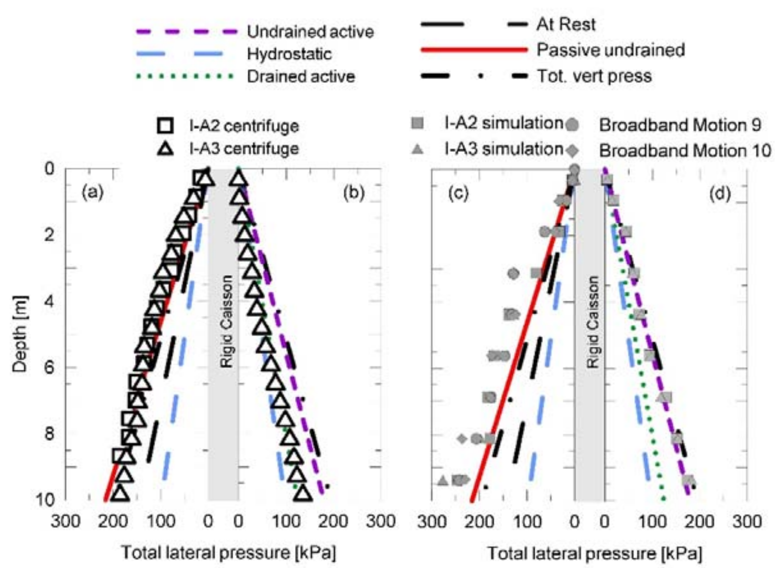

Figure 2. Computed and centrifuge measured lateral spreading induced earth pressures against a stiff caisson.

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PILE FOUNDATIONS IN LATERALLY SPREADING SOILS: BUILDING A MECHANISM-BASED FRAMEWORK?

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED LATERAL SPREADING

The response of pile foundations in laterally spreading soil during strong earthquakes is a complex and intense dynamic process, affected by kinematic and inertial interactions between the soil, foundation and superstructure, temporally and spatially varying pore pressures, and highly non-linear soil response. The essence of the problem is the interaction between the soil, piles, and superstructure, both during and after strong shaking. Extensive post-earthquake field investigations, numerical studies, and physical model tests (many involving the testing of complete (reduced-scale) laterally spreading soil-foundation systems in geotechnical centrifuges) have contributed to the development of our understanding of lateral spreading demands on piles and pile groups. Such understanding can play an important role in informing and advancing design practice, be it through the identification of response mechanisms, the calibration of numerical tools, or the evaluation of design procedures or proposed engineering solutions. Nonetheless, some of this work has so far had limited impact on design practice (Finn, 2005).

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

At the system level, the soil-foundation-superstructure response can be conceptualised as a complex interaction between several concurrent or competing mechanisms (including, for example, lateral translation (with or without significant pile damage), rigid overturning, or foundation back-rotation), as illustrated in Figure 1 below. Many previous studies have made important advances in our understanding of the soil-pile interaction for one or another of these mechanisms, while a few studies have gone further to identify key parameters and details of the soil, foundation, and superstructure that appear to have a controlling influence on which mechanism prevails for different soil conditions and foundation/structural designs.

Specific parameters that might influence the mechanism of response include (but are not limited to): the relative soil-pile stiffness, the presence or not of a non-liquefied crust layer, the severity of the lateral spreading demand, the pile tip fixity conditions and available support from the foundation soil, the connection details between the foundation and superstructure, and the longitudinal strength and stiffness of the superstructure (for the case of bridges).

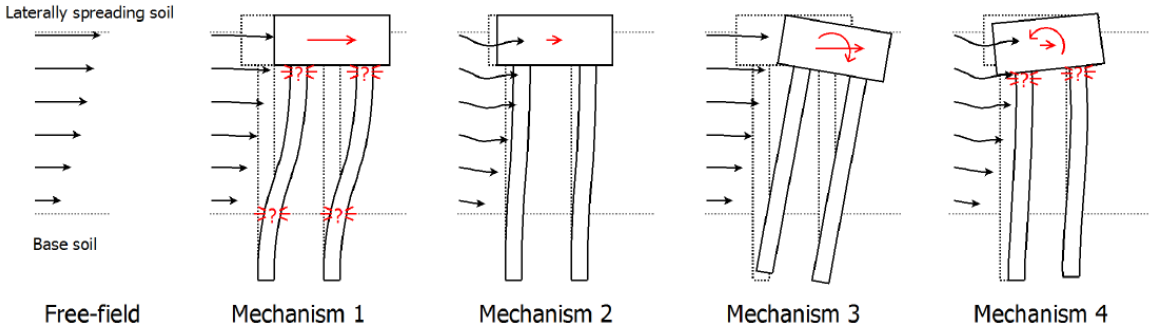

Figure 1. Example mechanisms of pile foundation response when subjected to liquefaction-induced lateral spreading (from Haskell et al., 2012)

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Although our understanding of the lateral spreading demands on piles has improved considerably thanks to recent research efforts it is still somewhat piecemeal in nature, with many of the mechanisms and trends that have been identified being essentially scenario-specific, their application to unfamiliar soil-pile systems being burdened by uncertainty. Rather than being a consequence of an inherent lack of numerical and physical modelling results or field evidence, it perhaps reflects more a weakness in the linking of changes to the characteristics of the superstructure-foundation-soil system to gross changes in the mechanism of interaction between the soil, foundation, and seismic demand, and the linking of this understanding to the overall system behaviour and foundation performance.

From a practical standpoint, the current fragmented understanding precludes the consistent and reliable design of pile foundations in soils likely to undergo lateral spreading – it is unclear even the mechanism of response to ‘target’ and any prediction of foundation performance is burdened by considerable uncertainty.

2.5 PATHS FORWARD

One possible path forward, specifically targeted at addressing the fragmented understanding of pile response in laterally spreading soil, is to attempt to pull together and synthesise the findings from the large body of previous studies on different aspects of lateral spreading effect on pile foundations. This would be done with the specific aim of identifying the range of possible response mechanisms that can develop and the governing/controlling parameters and design details that influence which mechanism ultimately prevails. The key output from this effort would be the development of a comprehensive mechanism-based framework for describing and anticipating which mechanism(s) might develop for a given scenario. In developing such a framework, it is anticipated that significant uncertainties and areas for more specific investigation might be identified, and that these might be mechanism-dependent, due to the different characteristics of the interaction between the soil, foundation, and superstructure associated with each mechanism. Such a framework might also be useful for developing mechanism-specific design solutions and damage mitigation options for existing foundations (Haskell, 2014).

3.0 ACKNOWLEDGEMENTS

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PATHS FORWARD TOWARD ASSESSING THE EFFECTS OF LIQUEFACTION

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED FLOW SLIDES

Liquefaction-induced flow failure of geotechnical structures, such as shown in Figure 1, is associated with the steady state of sand on the order of 10 kPa. Although the initial confining stress does not affect the steady state of sand, it is very sensitive to the fines content and void ratio. The steady state strength of Masado, a decomposed granite in Kobe, is about 10 kPa at a void ratio as low as $e = 0.4$. The shear resistance at the quasi-steady state, which corresponds to the transition over the phase transformation line that can induce a large shear strain in engineering sense, is highly dependent and almost proportional to the initial confining stress. The combination of the effects of cyclic loading and the steady state will be a challenge in the coming decades (Iai and Ichii, 2010).

Figure 1. Damage to a river dike at the Shiribeshi-Toshibetsu river, Hokkaido, Japan, during Hokkaido-Nansei-oki earthquake of 1993 (Iai and Ichii, 2010)

2.5 PATHS FORWARD

The effects of pore water migration can be significant in inter-layered structure of clay and sand. In order to approach this aspect of the study, appropriate modelling of behaviour of clay, in particular appropriate modelling of stress-induced anisotropy of the steady state (i.e. critical state), becomes important (Iai et al., 2015a). Formation of water film underneath the capping clay layer and long term effects of dissipation of excess pore water pressure from the capping clay layer can be reasonably simulated within the framework of the currently available effective stress analysis scheme, provided that the scheme is numerically robust and reasonably accurate.

A new challenge is combined hazards, such as the combination of earthquake motions and tsunamis observed during 2004 Sumatra, Indonesia, and 2011 East Japan earthquake. The rapid seepage flow through the rubble mound of a breakwater can cause complete bearing capacity failure. The mechanism of the failure is common to the liquefaction-induced flow slides (Iai et al., 2015b).

Studies to elucidate the mechanics of partially saturated sands are in progress, but many challenges remain in the coming decades. There are two factors that are not considered in the fully saturated or fully drained (dry) sands: high compressibility of air phase, and addition of suction (or air pressure) as additional variable. Studies considering these additional factors will lead us to more generalized understanding of the mechanical behaviour of sands and clays. Some of the recent results on this subject will be presented during the workshop.

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WHAT WILL THE NEXT GENERATION OF LIQUEFACTION VULNERABILITY PARAMETERS LOOK LIKE?

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Liquefaction vulnerability parameters (or liquefaction severity indices) are a useful tool for providing a simplified appreciation of the complex problem of predicting liquefaction-induced ground damage. Rather than attempting to predict the physical settlement directly, these parameters provide an index that can be used to understand the likelihood that different levels of severity of liquefaction-induced ground damage might occur. They are often used for area-wide liquefaction studies where efficient and consistent analysis of ground information (typically CPT or SPT data) is required to identify relative differences in performance between different areas. They can also provide valuable insights to help guide more detailed engineering assessment on a site-specific basis.

With the wealth of data now available, a new generation of liquefaction vulnerability parameters will soon emerge. So now is a good time to develop a shared understanding of the desirable features of vulnerability parameters, and the key mechanisms they should aim to capture.

2.1 CURRENT STATE-OF-THE-ART

There are a range of liquefaction vulnerability parameters commonly used in engineering practice, such as volumetric reconsolidation settlement, Liquefaction Potential Index (LPI) and Liquefaction Severity Number (LSN). Tonkin & Taylor (2015) and van Ballegooy et al. (2015) provide an overview of several parameters and comparison to ground damage in the Canterbury earthquakes. It should be appreciated that there is no one vulnerability parameter that is intrinsically better than the others. Rather, each provides a different perspective to help guide engineering judgement.

Many of these parameters are in effect a simple "bottom-up" summation of the effects from liquefied soil throughout the soil profile up to the ground surface. There are two particular limitations to this approach that the next generation of vulnerability parameters may be able to overcome:

- For the most part they only take account of the negative effects from soil that is liquefied, and do not directly account for the positive effects of interlayering with soil that is not liquefied (except sometimes in a simplified way, e.g. depth-weighting or surface crust thickness).
- The contribution from liquefaction at depth becomes "locked in" to the index value regardless of the nature of the overlying soil. For example a site would have the same index value regardless of whether the non-liquefied surface crust comprised dense gravel or loose sand, but the gravel may be more effective at suppressing surface effects from liquefaction beneath.

This suggests that perhaps the calculation methodology for the next generation of liquefaction vulnerability parameters should have the ability to respond to the specific details of layering within the soil profile, to not only "add" to the index value but also to "subtract".

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Conceptually, there are two primary mechanisms that a liquefaction vulnerability parameter needs to capture: the ***accumulation*** and the ***suppression*** of liquefaction-induced ground damage that manifests at the surface. This provides a useful framework for classifying the characteristics of a soil profile and to quantify their effects in a systematic way, as demonstrated in Tables 1 and 2.

Table 1. Characteristics that contribute towards the ***accumulation*** (or lack thereof) of ground surface damage.

<table>
<tr><th>Soil profile characteristic</th><th>Contribution to the accumulation of ground surface damage</th></tr>
<tr><td>Relative density of liquefied soil.</td><td rowspan="2">Influences volumetric consolidation strain (and thus the magnitude of settlement) and volume of pore water expelled (and thus the severity of scour).</td></tr>
<tr><td>Cyclic shear strain induced in liquefied soil.</td></tr>
<tr><td>Thickness of liquefied layers.</td><td rowspan="2">Thicker and more continuous liquefied layers expel a greater volume of pore water, increasing the likelihood that ejecta will be able to burst through an overlying confining layer.</td></tr>
<tr><td>Lateral continuity of liquefied layers.</td></tr>
<tr><td>Soil type.</td><td>Different types of soil exhibit different cyclic and post-liquefaction behaviors (e.g. more severe effects for clean sand vs silt).</td></tr>
</table>

Table 2. Characteristics that contribute towards the ***suppression*** (or lack thereof) of ground surface damage.

Soil profile characteristic	Contribution to the *suppression* of ground surface damage
Thickness of non-liquefied soil layers.	For thicker layers there is less potential for cracks or weaknesses to extend completely through the layer and provide pathways for ejecta, or for high pore pressures to burst through a layer.
Depth of liquefied soil below ground.	For shallow liquefied soils, if ejecta ruptures to the surface then localized ground volume loss can occur, resulting in severe differential settlement. For deeper liquefied soils there is greater opportunity for overlying non-liquefied layers to stop ejecta reaching the surface (Swiss cheese model). Where there is spatial variability of liquefaction at depth (e.g. localized lenses or changing thickness), soil arching can even-out the effects at the ground surface, reducing differential settlement and flexural distortion.
Lateral spreading and dynamic ground lurch.	Ground cracking may open up pathways for ejecta, reducing the ability of non-liquefied layers to suppress damage.
"Toughness" of non-liquefied soil.	Dense or cohesive soils are better able to resist rupture from high pore pressures than loose granular soils. Plastic soils that are able to tolerate deformation without extensive cracking provide fewer pathways for ejecta. If a soil is resistant to internal erosion then there is less potential for cracks to widen due to scour.
Relative density of non-liquefied soil.	Soil arching and stiffness is greater in denser soils, which can reduce differential settlement and flexural distortion.

2.5 PATHS FORWARD

Some thoughts for discussion about desirable features of the next generation of vulnerability parameters:

1. Captures the primary mechanisms outlined in Section 2.3, particularly those associated with ejecta (as it tends to result in the most severe ground surface damage).
2. Takes into account the specific details of layering within the soil profile at a site.
3. Explicitly and transparently evaluates the ***accumulation*** and ***suppression*** of liquefaction-induced ground surface damage, addressing each of the contributing mechanisms in turn to provide insights about which specific factors have the greatest influence for a particular site.
4. Able to provide useful information about ground settlement damage in areas where lateral spreading may occur; not necessarily quantifying the additional damage from lateral ground movement, but at least recognizing the exacerbation of ejecta due to ground cracking.
5. Can run automatically over a database of ground information without manual intervention.
6. Any additional information the analysis algorithm needs to supplement ground testing data is simple to collate in a consistent and standardized manner.

7. The relationship between index values and ground damage is consistent between different earthquake events and different ground conditions.
8. The index value is appropriately sensitive to variations in parameters, but not overly so – e.g.:
   - A “soft-start” at lower PGA values (as pore pressures start to build up) may be preferable to a step-change when the triggering FOS = 1.0.
   - A plateau at high PGA values may be preferable to a continual increase as FOS decreases.
   - It should avoid unstable calculations (e.g. as the depth of liquefaction approaches zero).
9. Is able to clearly convey the probabilistic basis and uncertainties of the underlying correlations (e.g. the likelihood that various levels of ground damage could occur).

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I'VE DONE GROUND IMPROVEMENT, SO I DON'T NEED TO WORRY ABOUT LIQUEFACTION ANY MORE, RIGHT…?

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

In Christchurch, now that the potential effects of liquefaction-induced ground deformation are more widely appreciated across the engineering profession and society in general, there is an increasing use of ground improvement for liquefaction mitigation. Whereas in the past ground improvement was typically only contemplated for large structures of high importance or particular sensitivity to movement, it is now routinely used for a broad range of structures large and small.

It can be tempting for building designers and owners to assume that once the ground improvement is completed, they don't need to give any further thought to liquefaction. As they see it, they have paid extra to get a "liquefaction-proof" building. But it is usually not as simple as that, and as earthquake geotechnical engineers it is our responsibility to clearly communicate how the improved ground can be expected to perform.

So how do we work out what the expected performance is…?

2.1 CURRENT STATE-OF-THE-ART

- Research based on dynamic numerical analysis (usually linear-elastic) and centrifuge testing. Typically focused on triggering and ground deformation for free-field situation rather than consequences for overlying structures.
- Day-to-day engineering practice tends to focus on effects from liquefiable soil below or surrounding improved ground block, rather than effects from within the improved ground. Design is often based on the concept of a robust raft or block of improved ground which acts to control flexural distortion and lateral strain and maintain bearing capacity.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

- Settlement due to liquefaction of underlying ground (e.g. consolidation, punching, squeezing).
- Build-up of excess pore pressures and cyclic strain within the improved ground.
- Weakening of the improved ground due to liquefaction of surrounding or underlying ground (e.g. due to migration of pore pressures and loss of support).
- Interaction between unimproved ground, improved ground, foundation and structure.
- Complex combinations of demands to simultaneously control liquefy and carry foundation loads (especially for ground improvement using reinforcement/stiffening elements).
- A range of mechanisms associated with lateral spread. These are not discussed further here, but it may be useful to consider how they impact improved ground as part of Challenge 2.

Figure 1 and Table 1 overleaf provide an example of the performance of ground improvement beneath the main stands at AMI Stadium over the course of the 2010 – 2011 Canterbury earthquake sequence. The site was subjected to several large earthquakes, including shaking above the design PGA. The ground improvement performed its primary function as designed, maintaining the bearing capacity and preventing overturning of the stands. However, significant settlement of the structure still occurred. An overview is provided in Wotherspoon et al. (2014).

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

- Few relevant case studies (i.e. with extensive liquefaction in surrounding area).
- Incomplete understanding of the mechanisms by which some ground improvement methods control liquefaction within the improved ground (e.g. reinforcement, stiffness, lateral pressures).
- Complexities in analyzing co-seismic and post-seismic interactions between improved ground, surrounding ground and structure.

2.5 PATHS FORWARD

- Invest additional effort to better understand the available case studies and field trials.
- Encourage instrumentation of ground improvement sites, for the possible high-quality case studies of the future.
- Further field trials and centrifuge testing.
- Further numerical analysis to allow broader application of the results in practice (including nonlinear models).
- Better communicate the expected performance of ground improvement so building owners and designers better appreciate the residual risks.

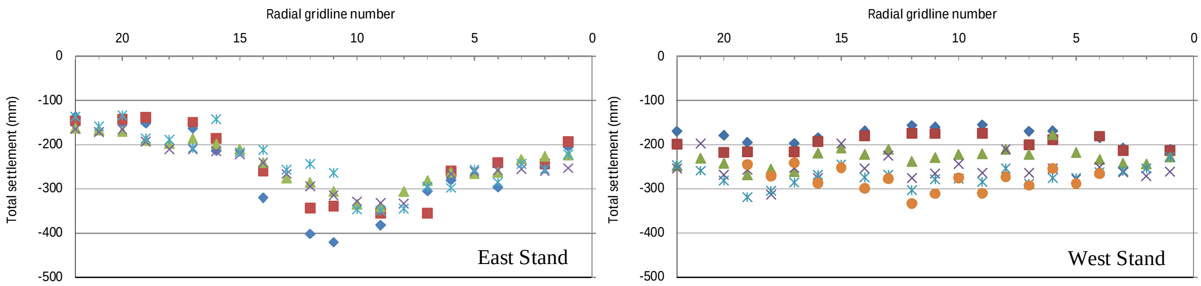

Figure 1. Measured settlement of east and west stands at AMI Stadium following 22-Feb-2011 earthquake.

Table 1. Settlement prediction based on CPT data and estimated PGA for 22-Feb-2011 earthquake.

	Calculated 1-dimensional volumetric reconsolidation settlement (mm)		
	Within improved ground	**Beneath improved ground**	**Total**
East Stand (Deans)	120 average [50 – 210 range]	100 average [70 – 150 range]	220 average [150 – 330 range]
West Stand (Paul Kelly)	75 average [40 – 140 range]	200 average [170 – 240 range]	275 average [210 – 370 range]

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A NEW LOQUEFACTION HAZARD MAP

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED SETTLEMENT

Liquefaction hazard maps have been prepared to assess the impact of liquefaction caused by the scenario earthquake in each target area. The existing maps, however, provide no quantitative indicator of damage extent such as the expected amount of ground subsidence. As a result, it seems that the maps are not effectively used much for disaster-prevention activity.

2.1 CURRENT STATE-OF-THE-ART

Current study investigates the relationship between liquefaction potential, P_L, and liquefaction-induced road subsidence caused by the 2011 Off Pacific Coast of Tohoku Earthquake, Japan. The road subsidence was prepared by comparing a set of Digital Surface Models (DSMs) before and after the earthquake (Konagai et al., 2013). It was found that as shown in Fig. 1, the road subsidence differs for different pavement and roadbed thickness, which is well in accordance with the actual damage. By using the regression lines in Fig. 1 and the P_L value distribution of the investigated area, the subsidence risk was evaluated for the road network in the investigated area for the 2011 earthquake. The new map shown in Fig. 2 is reflecting a fine texture of subsurface soil conditions, and it will be used not only for estimating damage extent but also for local government to determine the best routes for emergency vehicles to take after the large earthquake.
Meanwhile, it is also important to obtain a reliable strength parameter of soils for liquefaction assessment. The author is currently investigating the applicability of reconstituted sample, which the small strain shear moduli are the same with the in-situ value, for the liquefaction assessment. The current result of the effective stress analysis using the soil parameters obtained from the reconstituted sample shows the most probable behavior of the ground at the investigation site for the 2011 earthquake (Fig. 3).

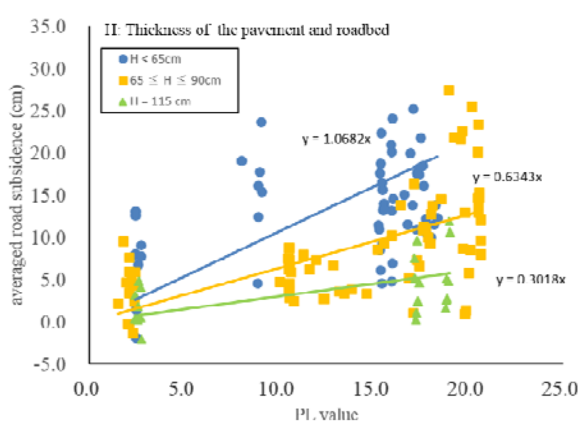

Figure 1. Relationship between P_L value and road subsidence for different pavement and roadbed thicknesses (Kajihara et al., 2016)

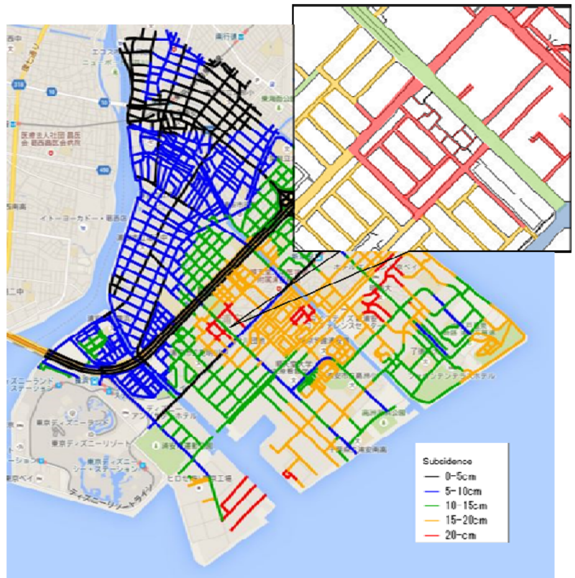

Figure 2. Estimated road subsidence in Urayasu City for the 2011 Tohoku Earthquake (Kajihara et al., 2016)

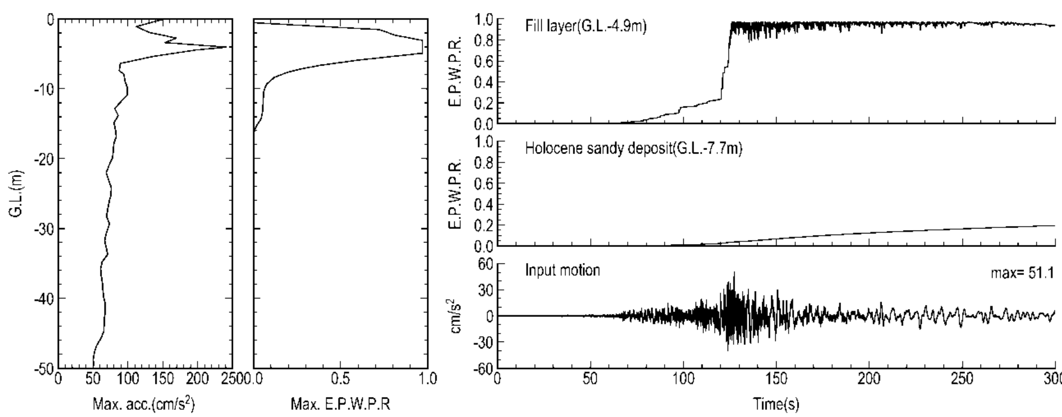

Figure 3. Computed acceleration and excess pore water pressure for the 2011 Tohoku Earthquake based on the test result of reconstituted sample for fill layer (Kiyota et al., 2016)

3.0 ACKNOWLEDGEMENTS

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ABSTRACT TITLE

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LIQUEFACTION EFFECTS CHALLENGE: 2 (also related with 1)– DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED FLOW SLIDES/LATERAL SPREADING

2.0 BACKGROUND

- A simplified in situ stress condition of a soil element consolidated under K_0 conditions and cyclically sheared with no sustained initial shear stresses (i.e. level ground) has been the standard model for evaluating liquefaction triggering mechanisms in engineering practice (Seed and Lee 1966).
- Sands tend to deform laterally and spread during liquefaction and flow under the influence of initial shear stresses, causing significant damage to earth dams, superstructures and buried life lines.
- Casagrande (1971) provided a different view on the liquefaction mechanism focusing on the role of initial shear stress near slopes and superstructures based on Steady-State-Line in State Diagram.
- In order to merge the two different views, quite a few research efforts have been made to date. However, a unified picture of the undrained monotonic and cyclic loading response of saturated sands including the effect of initial shear stress in practical design has not yet been established.

2.1 CURRENT STATE-OF-THE-ART

- In current US practice, the effect of initial shear stress is considered as an influencing factor on liquefaction triggering; $K_\alpha = CRR_\alpha / CRR_{\alpha=0}$ as a function of initial shear stress ratio $\alpha = \tau_s / \sigma'_v$.
- In order to predict liquefaction-induced residual deformations under initial shear stresses, numerical analyses are conducted worldwide assuming the undrained condition, though the results are very much dependent on dilatancy behavior of sands sensitive to the amount of non/low plastic fines and other in situ conditions including void redistribution.
- In current US practice, residual strengths for flow failures are back-calculated from case histories to reflect in situ behavior of heterogeneous sands including void redistribution.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

- In sedimentation process of sandy soils, coarser sand grains tend to deposit first followed by fines, thus in situ sand deposits are normally layered where silt seams are interbedded with sand layers, that may induce void redistribution during liquefaction particularly in hydraulic fill.
- Even uniform sand layers tend to contain more or less fines that may greatly influence their dilatancy behavior according to laboratory tests.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

(a) Clean sands with low F_c is on the dilative side of SSL under effective overburden in normal liquefaction in shallow depths, and flow deformation under initial shear stress occurs gradually with cyclically increasing strain (ductile failure) both in stress-reversal/non-reversal conditions. *CRR* tends to increase with increasing initial shear stress ratio (α) even in the stress-reversal condition.

(b) Sands tend to move from the dilative side of SSL to the contractive side with uniformly increasing F_c, and liquefaction failure under initial shear stress occurs suddenly in completely/partially flow-type (brittle failure). In this condition, *CRR* tends to decrease with increasing α, and more importantly designers have to pay attention to the difference in the failure modes.

(c) In horizontal or gently-inclined loose sand deposits (SPT N_1<10) interbedded with silt seams or covered by silt layers (encountered in situ quite often), void redistribution mechanism different from undrained shearing may trigger another type of time-delayed flow failure under initial stress on both sides of SSL, because it will generate sustained water films with drastically reduced post-liquefaction residual strength.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

How to combine the above three different mechanisms 2.3 (a)~(c) to unify the evaluations of residual shear resistance and flow deformation in simplified formulas on both sides of SSL and also considering the void redistribution effect so that practicing engineers can consider it for Performance-Based-Design.

2.5 PATHS FORWARD

Lab element tests under initial shear stress with varying parameters α, σ_c', D_r, F_c, PI, have to be conducted covering both sides of SSL on the state diagram to study;

(1) How sands will change to be from dilative to contractive with varying parameters.
(2) What is microscopic mechanism of the change above due to increasing F_c.
(3) How the post-liquefaction residual strength (including quasi-steady state strength) is determined associated with various parameters.

Centrifuge model tests on lateral spreading/flow of slopes and bearing sand strata beneath shallow foundations in uniform sand or non-uniform sand (interbedded with silt seams), having boundary conditions as simple as possible with varying slope gradient α, D_r, F_c, PI, have to be conducted covering both sides of SSL on the state diagram to study;

(1) How the failure mode of uniform sand slopes changes with the varying parameters, from ductile cyclic failure to brittle flow-type failure.
(2) How the residual strengths of uniform/non-uniform sand are back-calculated from the test results and how they are compared with soil strengths in the element tests.
(3) How the residual strains (displacements) are correlated with α, D_r, F_c, PI.

Collection & back-calculation of case histories of lateral spreading/flow with well-documented topographical/soil-investigation/earthquake data covering both sides of SSL on the state diagram to study;;

(1) How the residual strengths of in situ soils are back-calculated from case histories and how they are compared with soil element strengths.
(2) How the residual strains (displacements) are correlated with α, D_r, F_c, PI.

Based on the above investigations, unified simplified evaluation procedures on both sides of SSL concerning not only *CRR* for a particular induced residual strain but also the distinction of ductile and brittle failure modes together with associated lateral spreading/flow strain (displacement) has to be established for simplified PBD.

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TOWARD MORE ACCURATE ESTIMATION OF LATERAL SPREADING DEFORMATIONS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

Lateral spreading is known to be one of the most common and important effects of liquefaction, having caused damage to bridges, piers, retaining structures, and foundations in many past earthquakes. The damage caused by lateral spreading depends on the deformations that develop at and below the surface of the spreading soils. Design of structures to resist damage from lateral spreading requires accurate prediction of those deformations. Experience gained in recent earthquakes shows deficiencies in the procedures most commonly used to estimate ground surface displacements caused by lateral spreading – existing procedures can significantly overpredict (and underpredict) observed ground surface displacements. The development of improved procedures for estimation of lateral spreading deformations is an important challenge facing geotechnical earthquake engineers.

2.1 CURRENT STATE-OF-THE-ART

Lateral spreading deformations are generally estimated in current practice using empirical procedures. These procedures have developed over the past 25 years or so and generally fall into two main categories – purely empirical and semi-empirical. The purely empirical procedures include those of Bartlett and Youd (1992) and Youd et al. (2002) – they were developed from regression analyses of a large database of lateral spreading case histories accumulated, and generously made public, by Prof. Les Youd. These procedures predict displacements based on a set of loading, geometry, and material parameters that were found to provide the best fit to the case history data. Due to the almost complete lack of nearby ground motion recordings, earthquake loading is expressed in terms of source parameters (M and R) instead of actual ground motion intensity measures. Slope geometry is divided into two binary categories – ground slope and free-face sites – that many sites do not fall neatly into. Material properties are defined on the basis of layers for which $(N_1)_{60} < 15$ with soils of higher blowcounts treated as not contributing to deformations and soils with $(N_1)_{60} < 15$ contributing equally regardless of the value of $(N_1)_{60}$. While convenient and useful based on the existing database, the material characterization is not consistent with basic principles of soil mechanics and observations of soil behavior in laboratory tests. The semi-empirical procedures include those of Zhang et al. (2004) and typically integrate potential shear strains over the thickness of the liquefiable layers to obtain a lateral displacement index. The potential shear strains are based on laboratory tests with constant-amplitude, harmonic loading, and are predicted independently for each layer on the basis of relative density and a factor of safety. In reality, the layers in even a simple, one-dimensional soil profile do not respond independently of each other, and strains as large as the potential strain used in the integration process do not all develop in all liquefiable layers. Nevertheless, the semi-empirical procedures provide an indication of subsurface displacements in addition to ground surface displacement. The lateral displacement index is then combined with a geometric parameter, again using the binary distinction between site geometries, to estimate the actual ground surface displacement.

In recent years, improved understanding of the mechanical behavior of liquefiable soils, along with improved appreciation of pore pressure and void redistribution, has led to improved numerical models for analysis of liquefiable soils. Numerical models can account for the detailed characteristics of a soil profile and for its interaction with the detailed characteristics of a ground motion.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Obviously, lateral spreading is affected by the mechanisms that control the generation of excess pore pressure under cyclic loading. Of specific importance, however, is the stiffening that occurs when a soil dilates due to phase transformation behavior. That dilation, particularly in the presence of an initial, static shear stress, will limit the amount of deformation that occurs in each cycle of loading. That rate of stiffening, however, changes as the fabric of the soil degrades during and after triggering of liquefaction. Unfortunately, very little data on the post-triggering behavior of liquefiable soils exists, which makes it difficult to calibrate constitutive models for that range of behavior. Pore pressure and void redistribution are other mechanisms that can strongly influence lateral spreading deformations; in fact, some component of the displacements in lateral spreading case histories may be of hydraulic, rather than inertial, origin.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Most research on liquefaction and lateral spreading has focused on the resistance of a soil to liquefaction and subsequent deformation; significantly less attention has been paid to the loading applied to liquefiable soils, and particularly to loading metrics that correlate well to lateral spreading deformations. As previously discussed, current procedures relate lateral spreading displacements to source parameters rather than ground motion intensity measures or assume that the intensity measures that correlate well to factor of safety against triggering (i.e., PGA and M_w) also correlate well to lateral spreading displacement. A soil deposit after triggering of liquefaction is so much softer than it was before triggering that its response should not be expected to correlate well to intensity measures that predict triggering. Other intensity measures, in particular those sensitive to lower frequencies, should be investigated for their potential to improve lateral spreading deformation prediction. Also, laboratory and physical model testing have shown that the great majority of deformations of liquefiable soil profiles occur after liquefaction has been triggered. Therefore, the relevant loading for lateral spreading deformation should be that which occurs after triggering (Kramer et al., 2015).

A second important challenge, particularly in light of the apparent influence of pore pressure and void redistribution, is site characterization. If permeability gradients strongly influence liquefaction and lateral spreading, we need to improve our ability to identify and characterize them in the field.

2.5 PATHS FORWARD

Further research is needed to investigate the role of void redistribution relative to inertial forces in producing lateral spreading deformations. Further development of subsurface investigative tools that can identify and characterize permeability gradients is also needed. Finally, research that identified post-triggering ground motion intensity measures and correlates them to laterals spreading deformations from actual case histories is required.

3.0 ACKNOWLEDGEMENTS

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"RESIDUAL SHEAR STRENGTH" CANNOT BE UNIQUELY CORRELATED TO PENETRATION RESISTANCE

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE "UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL"

1.1 Brief overview of the challenge

One challenge with regard to understanding and assessing the "residual strength" is that the term is a misnomer. The term "residual strength of liquefied soil" implies that it is a property of liquefied soil. Kulasingam et al. (2004) established that the back-calculated shear resistance from slope failures depends on void redistribution, which depends on many factors (such as shaking duration, shaking history, sand layer thickness, thickness of the localized zone of failure, the presence of impermeable layers, slope geometry, and relative density).

Due to its dependence on so many factors, attempts to correlate the back-calculated shear resistance (or shear resistance ratio) with a single parameter such as SPT N value will produce poor correlations with a very large scatter.

Kramer and Wang (2015) admitted that "residual shear strength" is not a material property; they suggested that "residual strength" should be recognized as a system response parameter instead. Therefore, it would be more accurate if the title of their paper and of this workshop topic was "residual strength of slope systems" instead of "residual shear strength of liquefied soil". For this reason, throughout this paper, the term "residual shear strength" will appear in quotation marks.

2.1 CURRENT STATE-OF-THE-ART

One body of literature (e.g. Poulos et al. 1985) determines undrained steady state strength based only on the initial void ratio and the steady state strength corresponding to this void ratio. Similar to critical state soil mechanics, there is a convincing body of evidence that the true constant volume (undrained) strength of sand is uniquely related to the initial void ratio and the location of the steady state or critical state line. Figure 1 shows a Steady State Line for Nevada sand measured for us by Castro (2001) as reported in Kulasingam et al. (2004). One may see that even for relative density of 20%, the effective minor principal stress at steady state would be about 100 kPa, which would produce a large frictional steady state shear resistance. For a relative density of 30% (see dotted arrows in Figure 1), steady state strength would be measured in MPa. However, in the Kulasingam et al. (2004) centrifuge tests, the back-calculated shear strength of slopes that failed was always between 5 and 10 kPa and this strength was uncorrelated to the initial density. It is clear therefore that the mobilized shear resistance in a liquefying slope is much smaller than the undrained steady state strength. Furthermore, it was found that the mobilized strength of slopes that suffered large deformations was insensitive to the initial density (if the slope failed, the mobilized shear stress was between 5 and 10 kPa, regardless of D_r).

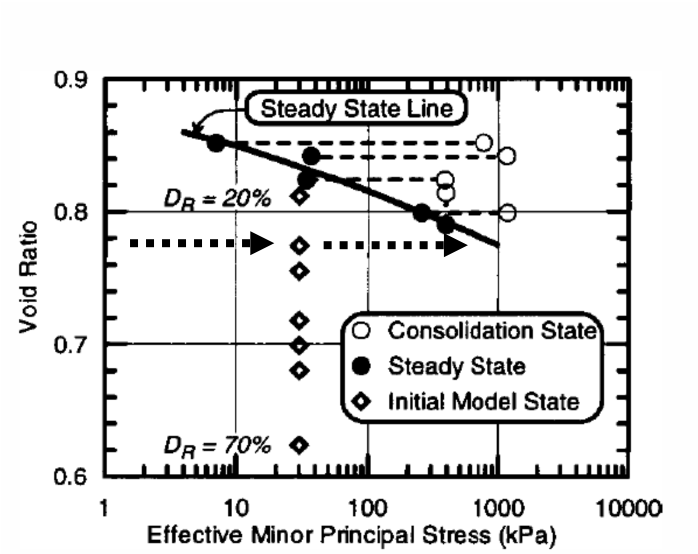

Figure 1. Steady State Line for Nevada Sand measured in triaxial compression and initial states of Nevada sand in a series of centrifuge tests (diamonds) (after Kulasingam et al 2004).

Another body of literature has attempted to empirically correlate "residual strength" and "normalized residual shear strength" back-calculated from case histories with penetration resistance (e.g., Seed (1987), Idriss and Boulanger (2007), and Kramer and Wang (2015)). Figure 2 shows a relationship proposed by Idriss and Boulanger (2007).

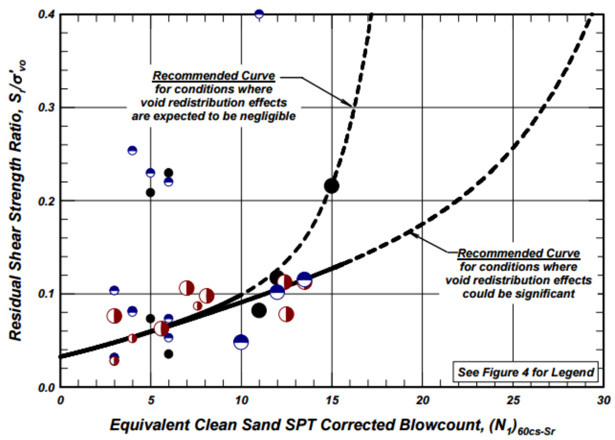

Figure 2. Empirical correlation between Residual Shear Strength Ratio and SPT blowcount from Idriss and Boulanger (2007).

For medium dense soils, the "residual strengths" determined from Figure 2 are orders of magnitude smaller than those that would be determined from Figure 1. The reason why back-calculated "residual strengths" are so much smaller than true undrained steady state strengths was pointed out by Kramer and Wang (2015): "as a flow failure develops in the field, drainage can occur, leading to changes in effective stresses, volume, and density..." Intermixing of soils and loosening due to void redistribution are important reasons why the back-calculated shear resistance of the soil during liquefaction flow slides can be much smaller than the true undrained steady state strengths. Idriss and Boulanger acknowledge that void redistribution can affect residual strengths (see dashed lines in Figure 2). The notations in Figure 2, however, imply that sometimes void redistribution may not be important despite the fact that void redistribution could have occurred in every case history represented in Figure 2. As Kramer and Wang (2015) pointed out, two important reasons why "residual strengths" are smaller than steady state strengths are void redistribution and particle mixing.

It should also be acknowledged that the scatter in Figure 2 (and other) empirical relationships is quite large. For a blow count $(N_1)_{60}$ = 10, the residual strength ratio for the presented case histories varies by almost a factor of ten: 0.05 to 4.0. Despite this large scatter, two unique curves are recommended, without addressing the uncertainty issue. Why is there so much scatter in relationships like Figure 2? One obvious explanation is that it is ludicrous to expect a unique relationship between $(N_1)_{60}$ (a material property) and mobilized shear resistance (a system response parameter).

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Kulasingam et al (2008) explained how the mobilized shear resistance of liquefied soil depends on slope angle, soil density, liquefiable layer thickness and the presence of lower permeability layers (Figure 3). Water collects under the low permeability layer, the void ratio increases, and when the state corresponding to the static shear stress hits the CSL in Figure 3d, undrained failure at critical state can occur.

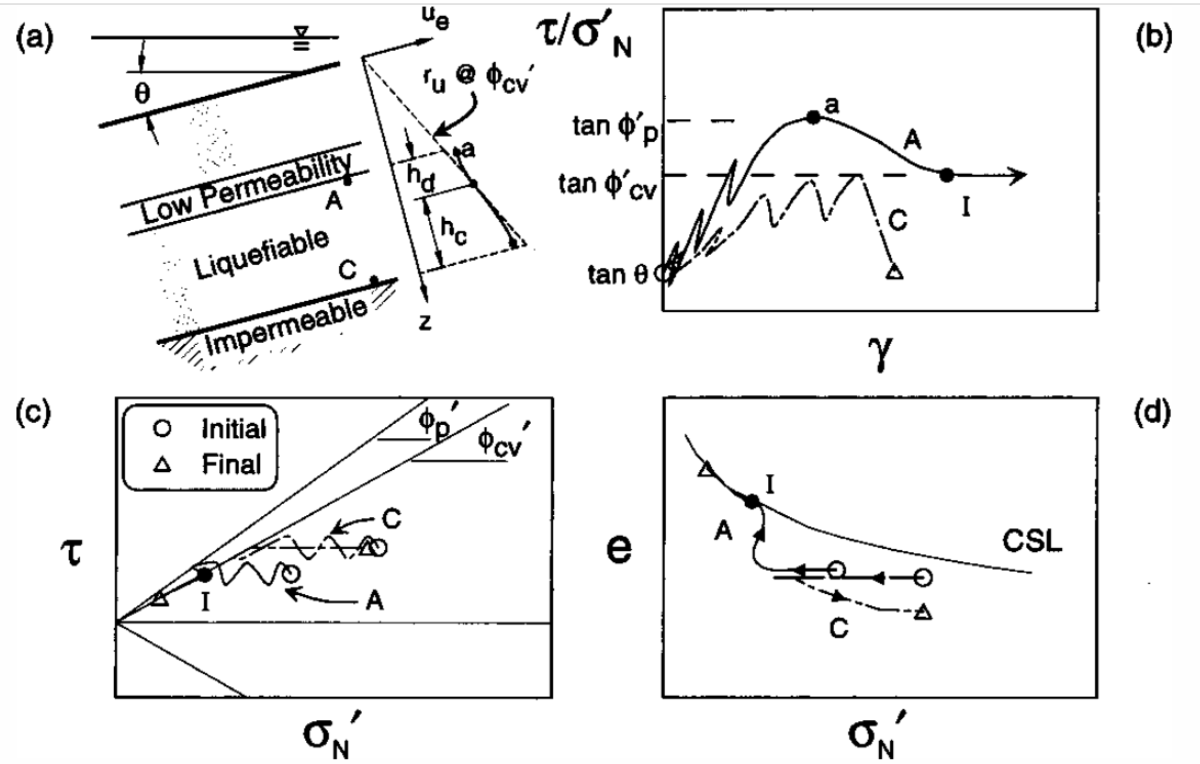

Figure 3. Schematic stress paths of elements near the top and bottom of liquefiable sand layer to illustrate mechanism of softening due to void redistribution (Kulasingam et al. 2004)

In Test 9 of the Kulasingam et al. (2004) centrifuge tests, a strong ground motion is followed by a series of five identical aftershocks. From this data it is apparent that the rate of displacement increases for successive aftershocks (top of Figure 4). This suggests that each aftershock causes additional weakening. It appears that each shake that triggers significant shearing tends to loosen the dilating shear zone a little bit more. The sensor locations for Test 9 are summarized in Figure 5.

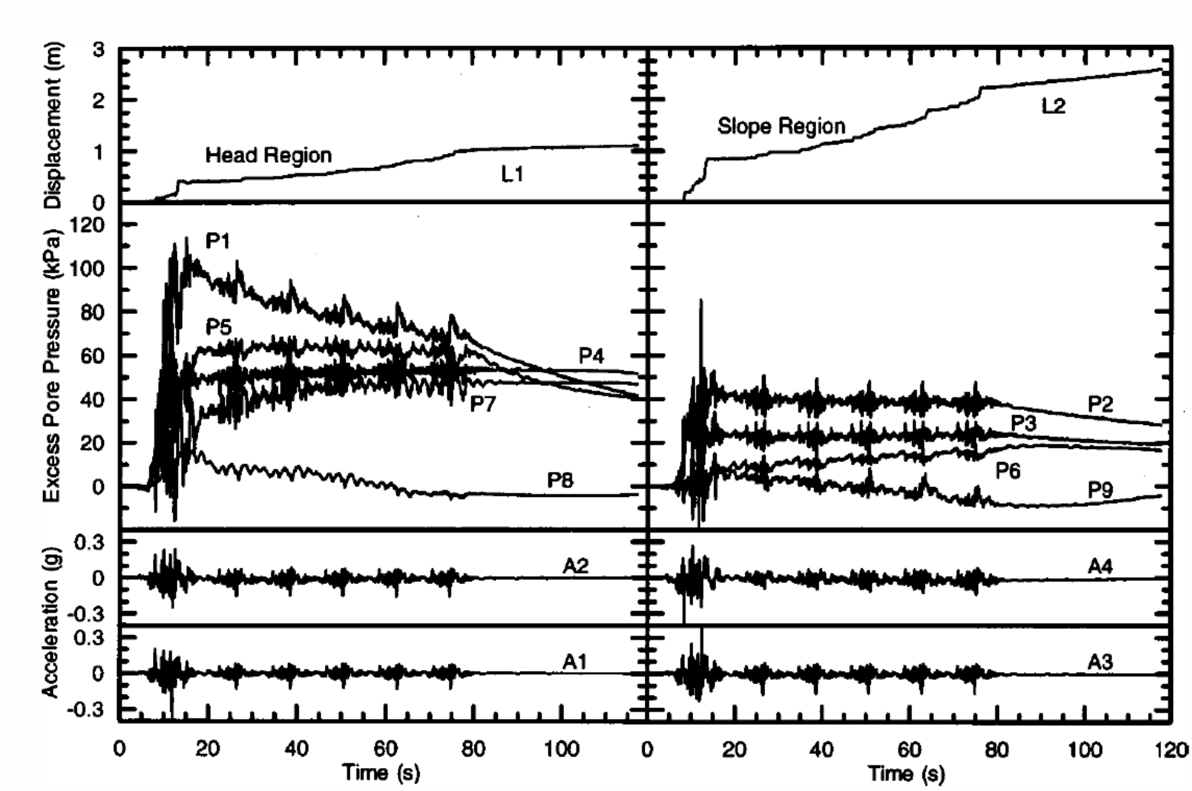

Figure 4. Recordings from Test 9 (D_r = 50%) during event 1 (Motion C) (after Kulasingam et al. 2004).

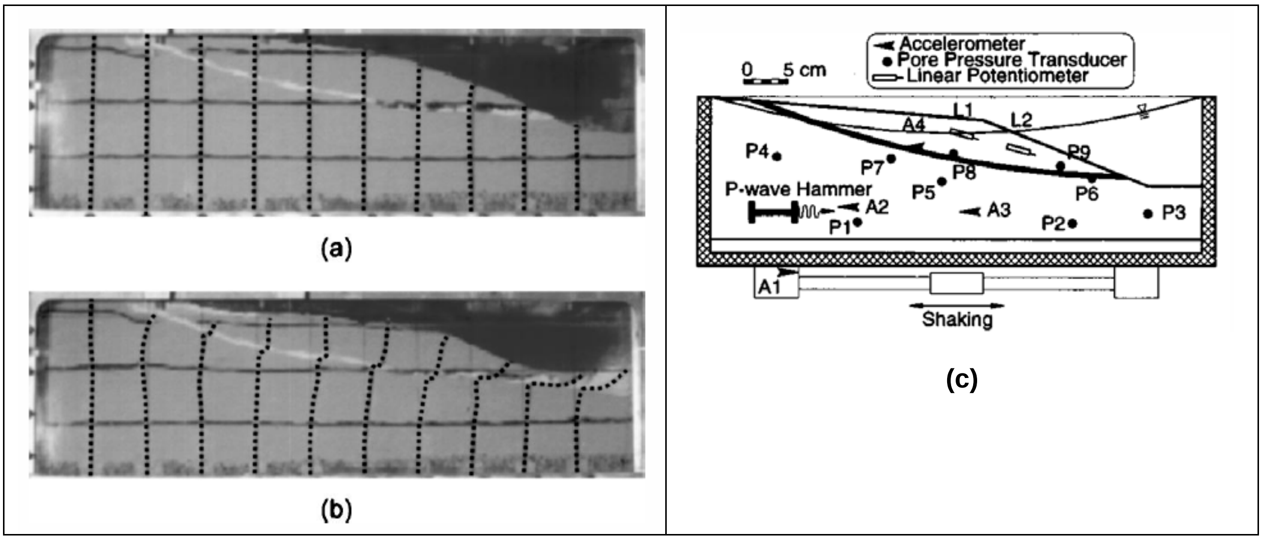

Figure 5. Before shaking (a) and after shaking (b) photographs of Test 9. Sensor locations for test 9 are shown in (c). (After Kulasingam et al. 2004.)

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Some challenges to developing better empirical procedures for "residual strength" are:

1. Back-analysis of flow slides allows one to show that the mobilized sliding resistance dropped below the static stress, but it does not provide information on the minimum or fully softened sliding resistance. Kulasingam et al. (2004) showed that soil may continue to soften with

repeated seismic loading. Flow failure in a case history might occur before the soil is fully softened. Therefore, it is not possible to deduce the fully softened residual strength by back-analysis of a failure.

2. Kulasingam et al. (2004) pointed out that the shear resistance mobilized in a liquefying slope depends on many factors including density, layering, static shear stress, seismic history, and the characteristics of the ground motion. Kramer and Wang (2015) recognized that "residual shear strength" is a system property. No one has laid out any logic to explain why the mobilized strength observed during failure of one system is applicable to the fully softened shear resistance of another slope system.

3. "Residual shear strength" is a deceptive terminology that propagates misunderstanding. The term strength in engineering is used to denote a material property that can be safely used in design calculations. However, in the presence of loosening due to void redistribution, the shear resistance of soil can continue to degrade until failure occurs. The amount of softening depends on many system geometry parameters and ground motion characteristics, and is therefore not a material property of liquefied soil.

4. The term "undrained residual strength" should not be applied to flow failures because there is no way to know that the material in a flow failure is undrained. Local drainage (void redistribution) within sublayers of a large deposit may occur in a fraction of the time required for global pore pressure dissipation; the adjective "undrained" is not justified. On the contrary, there is evidence from the field (e.g. San Fernando Dam) and from centrifuge tests (e.g., Kulasingam et al. 2004) that partial drainage (void redistribution) plays an important role in strength loss of liquefied soil.

5. Since there is no such thing as a "unique undrained residual strength of liquefied soil", the search for a unique relationship between the so called "residual strength" and the SPT N or the CPT q_c will never be successful.

2.5 PATHS FORWARD

The prediction of the flow failure phenomenon depends on our ability to determine if a critical failure plane reaches the critical state under the static stresses acting on the failure plane. If the void ratio increases to the critical state void ratio, large deformations are likely. Instead of a strength-based assessment of slope stability, the assessment should take into account the progressive nature of softening associated with dilation (loosening) of soils in the critical shear zones. In the case of the void redistribution mechanism, the approach to the critical state may be figured out by calculating how much water is being expelled by the zones of densification, and how much of this water contributes to loosening of the soil involved in the failure mechanism.

Advanced numerical simulations are not yet capable of accurately modeling the complex migration of pore voids in stratified liquefying soils, but urgent intense effort could solve this problem. Work is needed on:

1) realistic constitutive models for soils,
2) solution schemes that can predict strain softening, localization of shear strains, and large deformations,
3) multi-physics modeling capabilities could predict void redistribution; this may require the ability to account for water that escapes through cracks and boils, as well as water that accumulates in water films dilating shear zones.

Once these numerical tools are in place, Monte Carlo simulations using a large number of realizations of soil layering and input motions would allow one to predict the probability that flow failure would occur at a particular site. Empirical correlations with SPT or CPT will never be able to account for all of the important factors that contribute to flow failure.

3.0 ACKNOWLEDGEMENTS

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NEXT-GENERATION LIQUEFACTION: OPEN-SOURCE LIQUEFACTION DATABASE AND MODEL DEVELOPMENT

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1.0 LIQUEFACTION EFFECTS CHALLENGE: (1) DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL; (2) DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING; (3) DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Our topic is related to all three challenges. Field case histories play a vital role in the assessment of undrained residual strength, lateral spreading, and settlement effects. One of the core objectives of NGL is to produce a community database of case histories with and without various types of ground failure. This will allow researchers working on these topics to draw upon common data sources.

2.1 CURRENT RESEARCH PROCEDURES

Liquefaction effect evaluation procedures for engineering assessment are based to a large extent on the interpretation of field performance data from sites that have or have not experienced ground failure attributable to liquefaction. However, the number of case histories supporting these liquefaction procedures is remarkably small. For example, while nearly 200–400 case histories support most modern liquefaction triggering procedures, typically only a few dozen of these most tangibly affect the position of the threshold curve. Empirical procedures for analysis of undrained residual strength of liquefied soils are also controlled by only a few dozen case histories. Given the small number of most relevant case histories, it is no surprise that existing databases are incomplete, meaning they cannot constrain important components of engineering predictive models.

In addition, research on liquefaction triggering and effects has occurred within the framework of individual or small groups of researchers assembling and interpreting case history data to support the development of predictive models. Typically only the team of researchers that assembled a particular database has had access to its source data. As a result, the databases have been of different size, breadth, and quality, and their vetting by only small groups of researchers has complicated the identification of potentially problematic data. The groups that assemble case history databases also develop empirical predictive models which have often indicated different behavior due to different data, different data interpretation, potential errors in the interpretation, different approaches to constraining model behavior under data-poor conditions, and different philosophies of model development. Frequently subjective and philosophical decisions regarding the interpretation of case history data are not documented.

This situation serves as a barrier to new investigators engaging in these important topics and complicates the process of understanding differences between models.

2.5 PATHS FORWARD

As described by Stewart et al. (2016), the Next-Generation Liquefaction (NGL) project was established to support the development of a community database for liquefaction case histories, to facilitate studies on key effects poorly constrained by the database, and to establish a collaborative framework for model development by distinct teams drawing upon common resources. Our vision is that the process of database development, supporting studies, and model development would be undertaken with regular communication among investigators via project coordination meetings and with public workshops to enable community engagement and input. A major benefit of this approach is that the resulting model predictions would reflect genuine, 'apples-to-apples', epistemic variability associated with alternate methods of interpreting a common data set. The database expansion is in part associated with the 2011 earthquakes in Japan and New Zealand, which caused a great deal of damage attributable to liquefaction and its effects.

The NGL project deliverables are anticipated to consist of data resources and engineering predictive models. The data resources will be documented in a GIS-type database. The database is web-based, with a front page shown in Figure 1. This database was developed as an archive for *objective* data including, but not limited to, geotechnical in-situ tests (SPT and CPT), invasive and non-invasive geophysical tests (e.g., down-hole, SASW), index- to advanced- laboratory tests, earthquake event information (e.g., magnitude, fault mechanism), ground motions at sites, field performance observations as recorded in various forms (field notes, high resolution mapping -- e.g., LiDAR), and geology and hydrology maps. We store the data following the file format by Association of Geotechnical and Geoenvironmental Specialists (AGS4; http://www.agsdataformat.com/datatransferv4/intro.php).

A major phase of work in NGL has recently been launched with support from the Nuclear Regulatory Commission to develop the NGL database. Additional work with support from PEER is undertaking supporting studies related to the effects of fines content, overburden stress, ageing, and critical layer selection. These activities will be undertaken over a 2-3 year period, to be followed by model development.

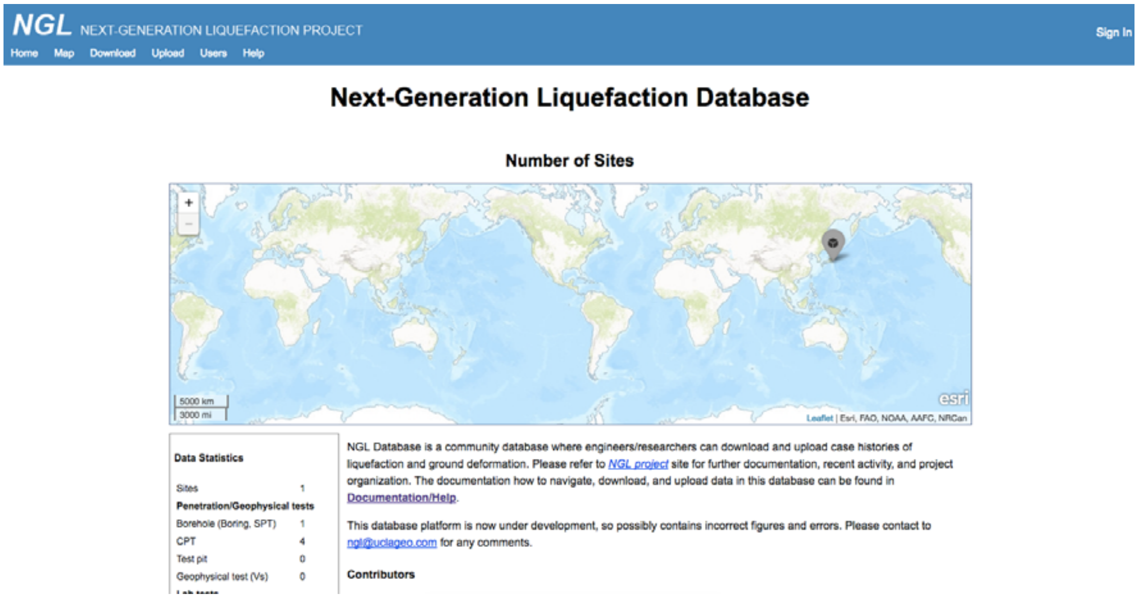

Figure 1. Prototype NGL database interface (http://www.uclageo.com/NGL/database)

3.0 ACKNOWLEDGEMENTS

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STANDARDIZED, OBJECTIVE, AND ECONOMY-FOCUSED PERFORMANCE ASSESSMENT OF LIQUEFACTION DAMAGE INDICES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Liquefaction damage indices are used to predict the severity of soil liquefaction manifested at the ground surface, which serves as a pragmatic proxy for damage potential to structures and lifelines. By way of this simplifying proxy, liquefaction damage indices bridge the gap between liquefaction triggering predictions and the resulting consequences. Iwasaki et al. (1978) proposed the first such index: the liquefaction potential index (*LPI*). Though widely adopted, evaluations of *LPI* following recent liquefaction events, such as the 2010-2011 Canterbury Earthquake Sequence (CES), show that it performs inconsistently. This inspired the development of new damage indices, to include a modified *LPI*, termed LPI_{ISH} (Maurer et al., 2015a), and the liquefaction severity number (*LSN*) (van Ballegooy et al., 2014). Inevitably, additional indices will be developed in the future, each aiming to better predict the damage potential of liquefaction. However, with the goal of developing a more reliable liquefaction damage index, it is critical that researchers adopt a standardized, objective, and economy-focused approach to assessing index performance. Towards this end, the use of receiver operating characteristic (ROC) methodology is herein demonstrated and promoted to serve this need.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

There are two key outcomes relating to the performance evaluation of liquefaction damage indices: (1) identification of optimal index values for classifying hazard; and (2) quantification of index efficiency. Importantly, a standard approach to assessing these outcomes is lacking, complicating comparisons among studies. To demonstrate, seven studies of *LPI* performance are compiled from the literature. These studies, each using case histories from different earthquakes, propose thresholds for predicting any liquefaction manifestation of 5, 5, 13, 14, 14, 13.5, and 5, respectively. While this discrepancy has significant implications, the cause is not easily investigated because the methods used to develop these thresholds differ. In addition, only one of these studies quantifies *LPI* efficiency, hindering performance comparisons across studies or against other damage indices. Moreover, the performance of any hazard assessment is intimately tied to the site-specific consequences, or "economies," of misprediction (e.g., two damage indices can have equal overall efficiency but very different efficiencies in particular economies). Accordingly, damage indices should also be assessed within a framework that considers the significance and variability of misprediction consequences.

2.5 PATHS FORWARD

ROC analyses, which are widely used in medical diagnostics, can provide a standard, objective, and economy-focused performance assessment of liquefaction damage indices, thus meeting each of the aforementioned needs. ROC curves plot the rates of true positives (R_{TP}) (e.g., liquefaction manifestation is observed, as predicted) versus the rates of false positives (R_{FP}) (e.g., liquefaction is predicted, but is not observed) for thresholds ranging from $-\infty$ to ∞. While the reader is referred to Maurer et al. (2015b) for full coverage of the ROC methodology, its utility is henceforth briefly demonstrated for quantifying prediction efficiency (outcome #2 above) using two different approaches. The first evaluates overall efficiency via the area under a ROC curve (*AUC*), where *AUC* is equivalent to the probability that sites with manifestations have higher computed index values than sites without manifestations. Increasing *AUC* thus indicates better performance. However, because *AUC* is an average efficiency across all misprediction economies, it could be misleading in certain cases. Thus, the second approach is to assess efficiency for particular misprediction economies via the prediction *Cost*, defined as *Cost* = R_{FP} x *CR* + R_{FN}, where: R_{FP} is as previously defined; R_{FN} = 1 - R_{TP}; and *CR* = C_{FP}/C_{FN}, where C_{FP} and C_{FN} are the costs of false positives and false negatives, respectively. Within this definition, *CR* is synonymous with "misprediction economy." Using these approaches, it can be determined not only which damage index performs best overall, but also which is best for particular economic scenarios. To demonstrate, the *LPI*, LPI_{ISH}, and *LSN* damage indices were used to predict the surficial manifestation of liquefaction for two case history datasets: (i) ~ 10,000 cases resulting from the CES ("CES Dataset"); and (ii) 265 cases resulting from 23 global earthquakes ("Global Dataset"). For each case, the Boulanger and Idriss (2014) procedure was used to predict liquefaction triggering. Overall efficiencies in terms of *AUC* are presented in Figure 1a, from which it can be seen that LPI_{ISH} is slightly more efficient than *LPI* and *LSN* for both datasets. In Figure 1b, the optimal index is identified at various *CR* values as that for which *Cost* is minimum. Also, because multiple indices could have nearly equivalent performance, any index whose *Cost* is within 1% of minimum is likewise treated as "optimal." From Figure 1b, it can be seen that the optimal index strongly depends on the consequences of misprediction. *LPI*, LPI_{ISH}, and *LSN* are each optimal for different scenarios that could be encountered in practice.

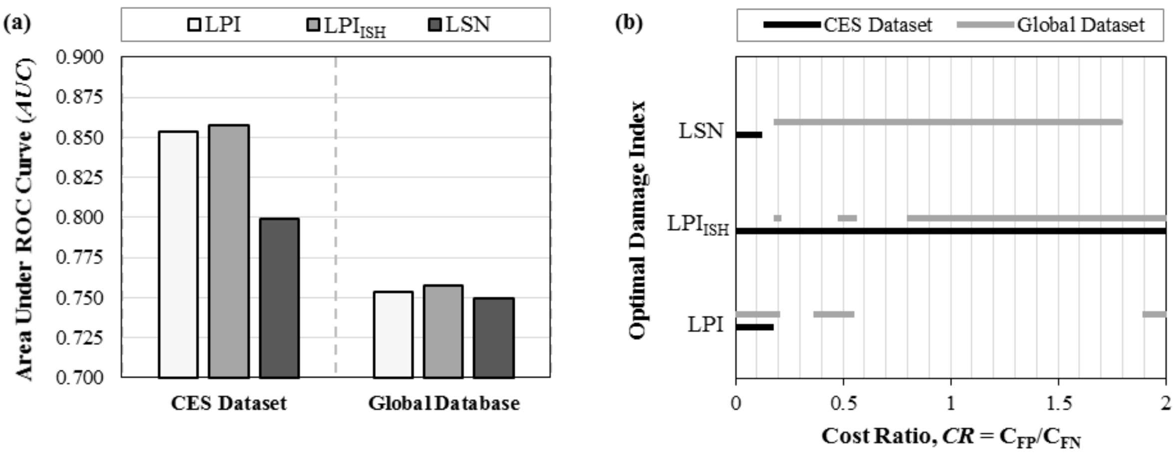

Figure 1. (a) *AUC* values computed from ROC analyses of the CES and global datasets using the *LPI*, LPI_{ISH}, and *LSN* liquefaction damage indices, where *AUC* is a popular measure of overall prediction efficiency; **(b)** Optimal damage index as a function of *CR*, as determined from the prediction cost and described in the text.

3.0 ACKNOWLEDGEMENTS

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DEVELOPMENT AND EFFECTS ON LIQUEFACTION-INDUCED LATERAL SPREADING ON STRUCTURES AND LIFELINES

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1.0 CURRENT STATE-OF-THE-ART

Damaged caused by strong motions on different types of soils have been made clear by numerous natural disasters such as the Alaska and Niiagata 1964 earthquakes, San Fernando 1971 earthquake, Hawaii 2006 earthquake, Haiti 2010 earthquake, New Zealand 2010 earthquake and more recently the Ecuador and Italy 2016 earthquakes. In each of these cases the soil experienced liquefaction (e.g. sand boils, ground cracks and lateral spreading) that resulted in extensive damaged to major infrastructure and lifelines such as homes, hospital, roads, bridges, government, utilities, port facilities and offshore structures. Liquefaction-induced lateral spreading can be defined as a lateral displacement of a gently sloping ground (0.3 to 5%) due to a build-up on pore pressure or liquefaction in relatively shallow soil deposits (e.g loose sands) subjected to strong motions. These horizontal displacements vary widely in magnitude and can range up to several meters. A lot of research has been conducted over the past 50 years to provide a better understanding of the liquefaction-induced lateral spreading phenomenon. All of the findings and lessons learned along the way have considerably developed the state-of-the-art. Currently, the state-of-the-art relies mostly on laboratory tests such as centrifuge test, shaking table test and cyclic triaxial tests and advanced soil-structure interaction models and finite element analyses. Centrifuge modeling has been identified as a key tool to identify and quantify mechanisms, calibrate analyses and evaluate retrofitting strategies for pile foundations (Abdoun and Dobry, 2002). Laboratory tests provide an excellent alternative to test soils specimens of different dimensions under different seismic loads and boundary conditions. Several centrifuge tests and models have been developed to investigate pore pressure build up and the response of pile foundations subjected to strong motions and the effects of lateral spreading including single piles, pile groups and multiple layers soil profiles. Also, several non-linear finite element analyses have been developed and proposed over the years to evaluate the liquefaction-induced lateral spreading phenomena. These models usually require constitutive stress-strain relationships and reliable undrained strength data, which we understand can be very difficult to obtain given all the issues associated with undisturbed sampling of cohesionless soils (e.g. loose sands and non-plastic silts) and making identical reconstituted laboratory specimens that mimic in situ soil fabric. Also, there is a tremendous amount of uncertainties estimating liquefaction-induced lateral spreading using the current available methodology arising from uncertainties in determining appropriate soil properties, selecting representative ground motions when data is not available and difficulties with ground characterization.

2.0 PATHS FORWARD

All of the advances made throughout the years in the state-of-the-art for assessing liquefaction-induced lateral spreading needs to be integrated with the current state-of-the-practice, which basically relies on field measurements like the Standard Penetration Test, Cone Penetration Test and Shear Wave Velocity Measurements and empirical correlations that are calibrated with case histories during earthquakes. Shear wave velocity is a parameter that can be easily measured in both the laboratory and the field and could be used to link lab and field behavior as it captures soil fabric, which we know is crucial to replicate when preparing samples in the laboratory, and overburden stress effects.

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TOWARDS THE PREDICTION OF FLOW SLIDES

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1.0 DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES THAT ARE GOVERNED BY THE UNDRAINED RESIDUAL SHEAR STRENGTH OF LIQUEFIED SOIL.

Seismic induced flow failures tend to result in large deformations and significant damage. Predicting the triggering, volume, and runout of flow failures has proven to be quite difficult because of the complex physics of the problem. Recent earthquakes in New Zealand and Japan may provide detailed flow failure case histories that can help better constrain forward analysis of this type of liquefaction effect.

2.0 CURRENT STATE-OF-THE-ART

State-of-the-art flow failure analysis involves several steps that are rife with uncertainty. The first step is assessing triggering based on penetration or other measurements (e.g., Moss et al., 2006; Yazdi and Moss, 2016). If triggering is likely the next step is estimating the post-liquefaction residual strength (e.g., Kramer and Wang, 2015; Weber et al., 2015) and comparing that to the static driving shear stresses. If the static driving shear stresses are in excess of the liquefied residual strength then lateral spreading or flow failure is likely. Deformation magnitude or runout distance of a flow failure can be quite difficult to estimate and has been based on empirical-correlations, progressive limit equilibrium method (LEM) analyses, or time domain effective stress finite difference modeling.

2.2 CONDITIONS FOR FLOW FAILURE

Flow failures occur in metastable saturated granular soils where triggering results in contractive behavior and rapid generation of excess pore pressures (Ishihara, 1996). Triggering can be caused by seismic or static stress conditions. Flow failures are therefore not unique to seismic events but can be caused by rapid pore pressure changes due to rainfall infiltration or drawdown, load changes due to construction or landslides, or other triggering mechanisms. Debris flows and flow failures can be lumped into the same hazard category based on the physics of the phenomena.

Metastable soil conditions are usually a result of natural deposition in a low energy environment (river point bars, colluvial deposits in an arid environment, etc.) or manmade deposits in a low energy environment (hydraulic fills, tailings slurries, etc). Examining prior flow failures we find that they generally occur on slopes greater than 6% (Youd et al., 2002) and the unconstrained deformations, sometimes on the order of 100's of meters, are differentiated from the relatively constrained deformations of lateral spreads by the threshold of 3 m (Park, 2013) to 5 m (Youd et al., 2002). The figure shows a histogram of penetration resistance from prior flow failures (Yazdi and Moss, 2016).

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

More and better detailed case histories of flow failures are always needed. Funding can be the limiting factor in collecting the necessary subsurface measurement, survey/lidar, lab testing, and detailed data analysis that it takes to fully develop a flow failure case history. Hence the current liquefied residual strength database is composed of ~30 case histories, which hasn't changed appreciable since correlations between penetration resistance and liquefied residual strength were first proposed (e.g., Seed and Harder, 1985). From the perspective of deformations, there is a disconnect between static and seismic induced flow failures even though the only difference is the triggering mechanism. Existing research into rainfall induced debris flows and landslide induced debris flows could inform the extents and limits of runout for predicting consequences. Limited modeling of flow failures has been carried out to date, and although the problem is sometimes intractable from a numerical modeling perspective, improved finite difference and more likely discrete element approaches could approach a reasonable simulation.

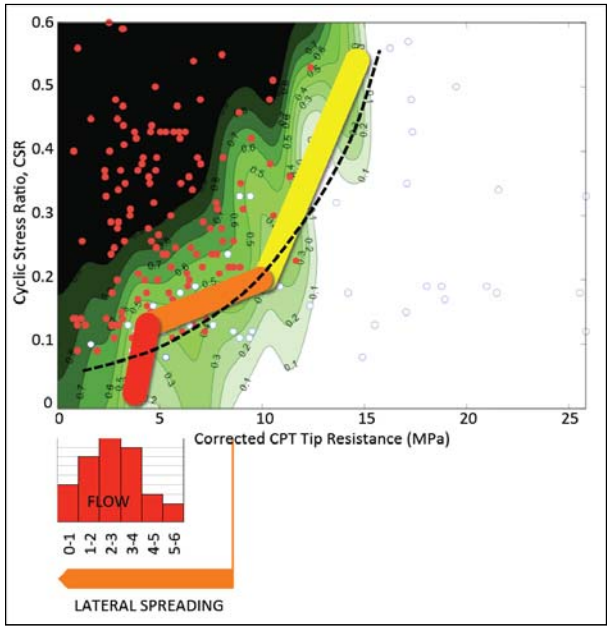

Figure 1. Histogram of CPT measurements from flow failures shown with respect to the range of lateral spreading CPT measurements and liquefaction triggering curves (from Yazdi and Moss, 2016).

2.5 PATHS FORWARD

- Champion and fund the collection of high quality case histories of flow failures. This requires in situ measurements of the soil (SPT, CPT, V_S), survey of the pre- and post-geometry, lab testing of high quality samples, and careful detailed data analysis.
- Develop simple means of quantifying the driving shear stress in typical situations where flow failures occur.
- Push for semi-theoretical deformation models drawing also from work done in related fields studying debris flows.
- Populate a database that includes all static shear stress driven failures (large shear strains, lateral spreads, flow failures, debris flows, turbidites) that will stimulate further statistical analysis and predictive model fitting.
- Encourage numerical modelling of large-strain liquefaction failures which focus on deformations and properly capture the influence of driving shear stresses.

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EMPIRICAL EVALUATION OF LIQUEFACTION-INDUCED LATERAL SPREADING DISPLACEMENTS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

Liquefaction-induced large ground deformations are one of the major causes of damage to deep foundations and waterfront structures in high seismic areas worldwide. Notable recent examples of such damage include but not limited to the 2010 Chile, the 2010 Haiti, the 2010-2011 Canterbury sequence earthquakes ([1],[2],[3]). Field reconnaissance of these events have reported the liquefaction-induced lateral spreading displacements up to several feet which are consistent with the observations from previous earthquakes such as the 1995 Kobe earthquake ([4],[5]).

In order to analyze and design structures such as deep foundations when subjected to liquefaction-induced lateral spreading, prediction of the maximum probable lateral soil displacements is one of the important steps. Once the displacement of the liquefied soil is estimated, it can be applied to the deep foundations using available computer programs such as LPILE which has been developed to evaluate the lateral response of the deep foundations. This procedure is commonly used in engineering practice [6]. As a result, the reliability of the available tools for the design of deep foundation subjected to liquefaction-induced lateral spreading highly depends on the ability to accurately estimate the anticipated lateral ground displacements. This abstract discusses current state of practice to predict the extent of liquefaction-induced lateral soil displacements and path forward for developing more rigorous prediction methods and tools.

2.1 CURRENT STATE-OF-THE-ART

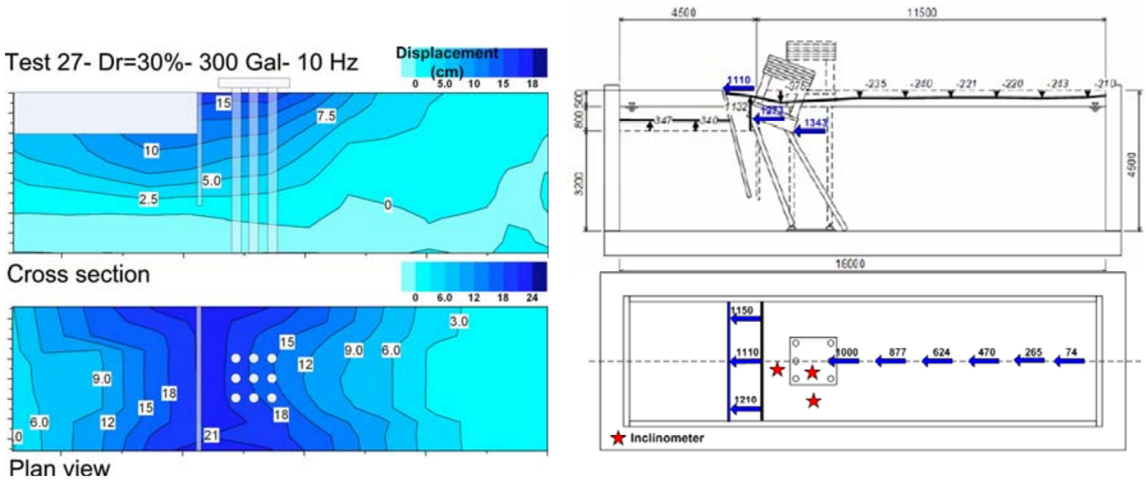
Researchers have investigated this phenomenon using case histories, experimental methods and numerical simulations. Numerical methods such as OpenSees, though available, still have limited application in engineering practice due to the complexity of the tools and their limited soil models. Available empirical methods for the prediction of liquefaction-induced lateral spreading can be roughly divided into three main categories based on the type of required input parameters:

1. Methods based on seismological parameters (e.g. Youd & Perkins [7], Ambraseys [8]).
2. Methods developed using only ground configuration (i.e. ground slope) (e.g. Hamada et al. [9]).
3. Correlations based on stratigraphy and properties of subsurface soil as well as the ground motion characteristics. Recently proposed methods mainly fall under this category such as Shamoto et al. [10]; Hamada [11]; Youd et al. [12]; Faris [13], and Valsamis et al. [14]).

The current state-of-art empirical methods often result in displacement predictions with a high level of uncertainty. For instance, Youd et al. [12] reported a factor of 2 in the majority of the predictions.

2.5 PATHS FORWARD

Developing more rigorous prediction methods and tools should be a high priority research topic in the area of liquefaction-induced lateral spreading. In my opinion, the methods based on subsurface soil properties as well as ground motion characteristics similar to Category 3 described above have the potential to yield the best predictions. However, developing such prediction methods should be based on a comprehensive database which comprises not only field observations but also includes experimental data. The advantages of experimental methods such as shake table or centrifuge tests are (1) ability to conduct experiments under prescribed conditions and controlled motions, (2) well-characterized soil information, (3) ability to explore the effects of different parameters on the response through parametric study. The author has utilized shake table experiments with different scales (from large to small) in the past to study the response of pile foundations subjected liquefaction-induced lateral spreading as illustrated in Figure 1. The results have demonstrated the efficiency of scaled shake table experiments to reproduce the overall behavior.



(a) (b)

Figure 1. Examples of shake table experiments to reproduce liquefaction-induced lateral spreading (a) 1-g scaled experiment [15], (b) large-scale experiment at E-Defense [16].

3.0 ACKNOWLEDGEMENTS

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LIQUEFACTION EVALUATION IN INTERMEDIATE SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1, 2 & 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED FLOW SLIDES, LATERAL SPREADING & SETTLEMENT

Recent case history analysis from the Canterbury Earthquake Sequence indicated a large proportion of "false-positive" cases where liquefaction evaluation predicted severe damage and yet no liquefaction was observed (van Ballegooy et al. 2015). The over-prediction of liquefaction damage was observed for many soils that had high fines content (FC) and no to low plasticity ("intermediate soils"). These soil types are not sufficiently represented in the liquefaction case history databases to inform liquefaction correlations. Recent work on fundamental intermediate soil behavior has shown that the addition of a small amount of plasticity to non-plastic fine grained soils can have a significant affect on cone tip resistance (q_t) and CRR correlations. Liquefaction evaluation methods can be improved with a CPT-based evaluation method that applies to a broad range of soil types, including sands, clays, and intermediate soils.

2.1 CURRENT STATE-OF-THE-ART

A limited number of liquefaction case histories are from intermediate soil sites, and very few of those cases report the soil plasticity index (PI). Without a theoretical understanding of how q_t and CRR relate across intermediate soil types, CPT-based liquefaction triggering correlations (e.g. Robertson and Wride 1998, Moss et al. 2006, and Boulanger and Idriss 2014) in intermediate soil conditions (i.e. FC > 50% and PI < 12) are constrained by limited case histories and can show large discrepancies across the methods. Additionally, most CPT interpretations require that the soil is treated as either sand-like (I_c<2.6) or clay-like (I_c>2.6); therefore, there is a notable discontinuity of CRR across the I_c=2.6 boundary (Figure 1).

Recent research work has focused on developing an informed CPT interpretation in intermediate soils. Price et al. (2015) tracked CRR and q_t for fine grained soils, and showed that the addition of a small amount of plasticity to non-plastic silt resulted in a significant decrease in cone tip resistance (q_t) and an increase in CRR values (Figure 2).

2.5 PATHS FORWARD

Future work should focus on developing theory-based models for CPT-based estimation of CRR across a broad range of soil types. Measureable soil properties, such as FC, PI, V_s, V_p, permeability, may warrant being incorporated into the models, which will require understanding their influence on CRR. Additionally, the relationships for r_d, C_N, K_σ, and MSF should be examined for intermediate soils. The interpretation of CPT data in interbedded deposits (e.g., thin layer and transition effects) with and without graded bedding needs examination. Updated CPT models will need to continue to be validated against available case history data.

A fundamental understanding of intermediate soil behavior related to the cone penetration test can be gained with lab experimentation, centrifuge modeling, and numerical simulations. Lab testing such as the work described in Price et al. (2015, 2016) can track the role of PI and FC on CRR. A numerical penetration model with a unified constitutive model that is applicable across a range of conditions (e.g. MIT-S1 in Pestana and Whittle, 1999) may also be used to study the role of properties such as FC, PI, overburden stresses, state on q_t. Centrifuge models provide opportunities to directly measure q_t and dynamic loading responses on a range of soil types to obtain data to supplement field case histories.

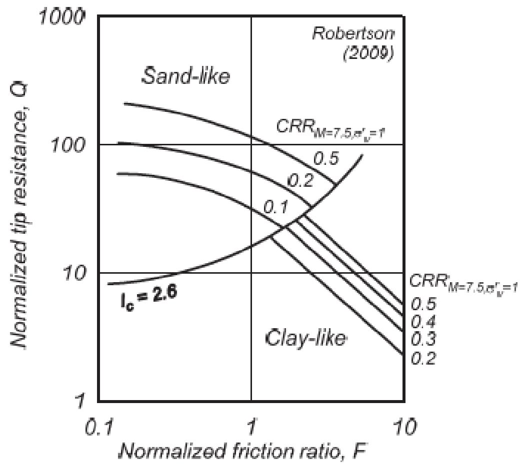

Figure 1. CPT interpretation CRR for sand-like soils and clay-like soils (after Robertson 2009)

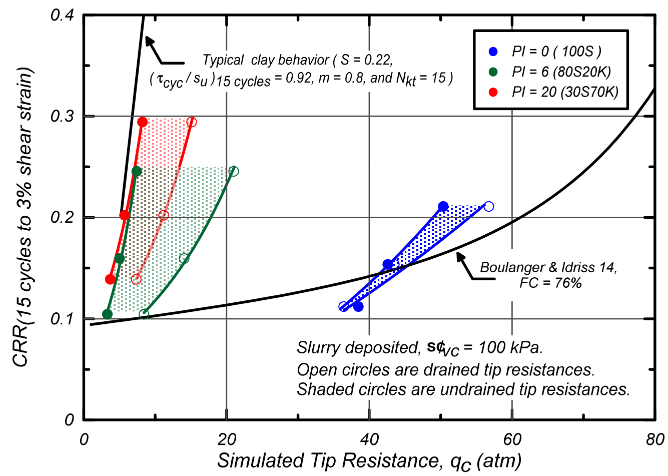

Figure 2: CRR from cyclic DSS tests for PI=0, 6, and 20 mixes of silica flour and kaolin clay versus simulated q_t from cylindrical cavity expansion (from Price et al. 2015)

3.0 ACKNOWLEDGEMENTS

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LIQUEFACTION AND SPREADING OF THIN SAND LAYERS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

More than 2000 river levees were damaged by the 2011 Off the Pacific Coast of Tohoku Earthquake and liquefaction of thin soil layers in levees is considered to be the fundamental mechanism of about 80% of the damaged levees, though we do not pay much attention to thin liquefiable soil layers. Following three issues which have been risen from the newly realized mechanism will be next challenges.

Liquefaction assessment on damaged and undamaged levees revealed that the currently used liquefaction assessment method provides factors of safety, FL, for relatively thin saturated layers in the levees excessively on the safe side. Estimated factors FL for not only the damaged levees but also all the undamaged levees were lower than unity. Improvement of accuracy of the assessment methods is needed.

Several slopes of levee spread laterally more than 20m. The heights of the levees were approximately 7 to 9 m with the slope angles approximately 1:2.5 or gentler and all the levee had relatively thin liquefiable soil layer, from 1 to 3m, near the base of the levees (Fig.1). It is uncommon that such thin liquefied layers with relatively gentle slopes developed very large spreading. Prediction of such large defamation is a challenge.

Integrity of the damaged levees due to the liquefaction induced lateral spreading as riverine structures to fight against flooding water is also an important issue.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Practical liquefaction assessment procedures invoke the undrained condition of soil. This is the case of the drainage condition for thick soil layers in earthquake durations. However, for cases of liquefaction of thin layers, partially drainage during earthquake shaking may play an important role and this has to be properly taken into account.

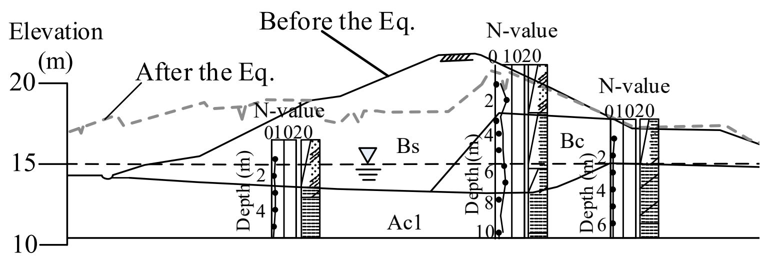

Figure 1. Damaged levee of the Naruse river

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UNDERSTANDING AND CHARACTERIZING LIQUEFIED SHEAR STRENGTH

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED FLOW SLIDES

Flow failures arguably are the most catastrophic consequence of liquefaction. As such, it is fortunate that these failures are relatively uncommon. Despite the fairly well-understood mechanics of flow liquefaction under controlled laboratory conditions, defining in the field the liquefied shear strength for evaluating consequences in ground subjected to a static shear stress (e.g., slopes, embankments, and structure foundations) remains challenging, in part because of the poorly understood role of drainage/porewater pressure migration, void redistribution, and soil fabric and compressibility.

2.1 CURRENT STATE-OF-THE-ART

The current state-of-the-art for evaluating liquefied shear strength remains quite similar to the approaches developed in the late 1980s/early 1990s. These approaches involve one of two options: (1) estimating the liquefied shear strength based on shear strengths mobilized in liquefaction flow failures and bearing capacity failures; and (2) measuring the liquefied shear strength in laboratory tests using a combination of carefully sampled, yet still partially disturbed, specimens and reconstituted specimens (e.g., Castro et al. 1985). Seed (1987) developed the first case-history based estimates for liquefied shear strength based on measured or estimated values of overburden stress-normalized standard penetration test blow count, $(N_1)_{60}$. As illustrated in Figure 1, the state-of-the-art now involves using liquefied shear strength ratios, $s_u(liq)/\sigma'_{vo}$ and either $(N_1)_{60}$ or overburden stress-normalized cone penetration test tip resistance, q_{c1} (or q_{T1} or Q_{tn}) (Olson and Stark 2002; Idriss and Boulanger 2007).

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

The primary mechanism involved in flow liquefaction is the generation of excess porewater pressures in saturated, contractive soil as a result of rapid loading. However, what is often poorly incorporated in evaluation procedures is the fact that the soil must be contractive to experience flow liquefaction and mobilize a liquefied shear strength, as well as the role that the pre-existing static shear stress plays in triggering flow liquefaction and . When the combined effects of pre-existing static shear stress and generated excess porewater pressure cross the yield strength envelope, flow liquefaction is triggered in the saturated, contractive soil. Olson (2015) summarized the state-of-the-art and primary mechanisms involved in flow liquefaction in greater detail than can be provided here.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

There are numerous challenges to better defining and evaluating the liquefied shear strength of saturated, contractive soils. These can be summarized as follows.

- Improving our ability to identify and characterize contractive soils in-situ using the state parameter (Jefferies and Been 2015) or empirical procedures (Olson 2009; Robertson 2010), and understanding the role of soil fabric and compressibility in this identification process.
- Improving documentation for statically- and seismically-induced liquefaction flow/bearing capacity failures. This challenge stems largely from the lack of well-documented case studies of such flow failures, which results in considerable uncertainties in defining $s_u(liq)$.
- Developing novel approaches to define and measure liquefied shear strength in the laboratory and in model-scale (e.g., centrifuge) settings.
- Improving our ability to characterize settings where void redistribution may occur and influence the shear strength mobilized during liquefaction.

2.5 PATH FORWARD

The path forward must include a combination of improvements in field characterization, field documentation, and especially novel field and laboratory experimentation to provide practitioners with a critical parameter for many geotechnical problems that include earthen dams and building foundations subjected to strong (and perhaps not so strong) seismic shaking as well as mine tailings facilities subjected to more common loading conditions.

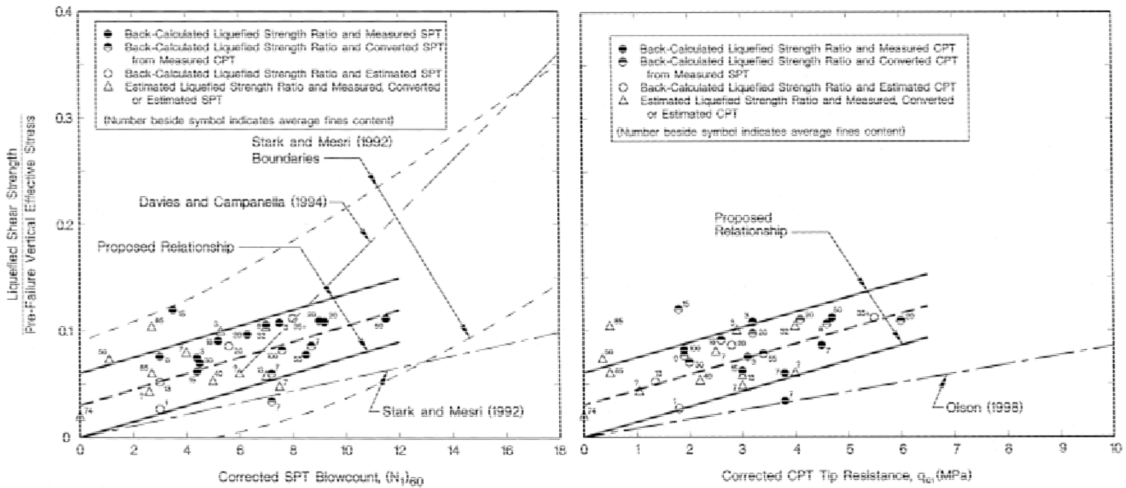

Figure 1. Olson and Stark (2002) relationships between liquefied shear strength ratio and normalized SPT blow count and normalized CPT tip resistance (modified from Olson and Stark 2002).

3.0 ACKNOWLEDGEMENTS

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POST-LIQUEFACTION BEHAVIOUR OF SANDS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED FLOW SLIDES THAT ARE GOVERNED BY UNDRAINED RESIDUAL STRENGTH

While the factors affecting the liquefaction resistance of sands have been studied by many researchers, their behaviour post liquefaction needs to be further examination. More specifically, understanding the effects of various parameters on the stress-strain relation of sands during the post-liquefaction stage is important not only for the purpose of assessing the magnitude of ground deformations induced by liquefaction, but also in investigating the impact of these deformations on buried structures, such as pipelines and pile foundations (e.g. possible conversion to p-y curve to analyse soil-structure interaction using Winkler method). In this paper, the post-liquefaction behaviour of sand is examined through a simplified stress-strain relation. The post-liquefaction stress-strain behaviour of sandy soils is generally of interest because of their high susceptibility to liquefaction; however, that of other local soils, such as the highly crushable pumice sands (in the North Island of New Zealand) need to be equally addressed because many engineering projects are constructed in areas underlain by these deposits. A question that arises is whether the post-liquefaction stress-strain behaviour of hard-grained sands is similar to those of crushable soils.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

In order to shed on light on the above, several series of advanced element tests using a cyclic triaxial apparatus were first conducted on different sandy soils: two commercially available sands (Redhill-110 sand and Japanese silica sand No. 8) and two natural sands from India (Assam sand and Ganga sand) with the aim of investigating the effect of relative density on the post-liquefaction stress-strain behaviour. These sands were reconstituted in the laboratory with different relative densities and tested under various effective confining pressures and different levels of cyclic stress ratio (CSR). In the tests, undrained stress-controlled sinusoidal cyclic loading with frequency of 0.1 Hz was initially applied in order to liquefy the soil sample. When the onset of liquefaction was monitored, the cyclic load was stopped and strain-controlled monotonic load was then applied under undrained condition to obtain the stress-strain relation of the liquefied sand. The post-liquefaction behaviour of the sands is then modeled in the form of bi-linear stress-strain curves, and is characterised by three parameters: the initial shear modulus (G_1), the critical state shear modulus (G_2); and the post-dilation shear strain ($\gamma_{post-dilation}$), defined as the shear strain level when the soil starts to dilate in the post-liquefaction state. These parameters are shown in Figure 1, while the variations of these parameters with respect to relative density are illustrated in Figure 2. Further details are provided by Rouholamin et al. (2016).

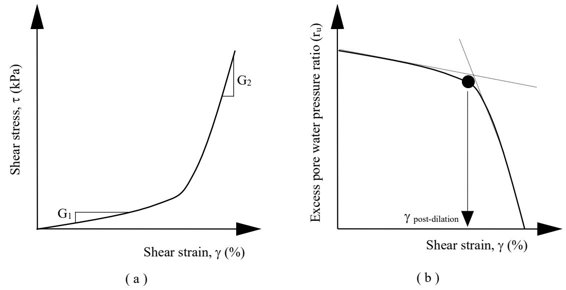
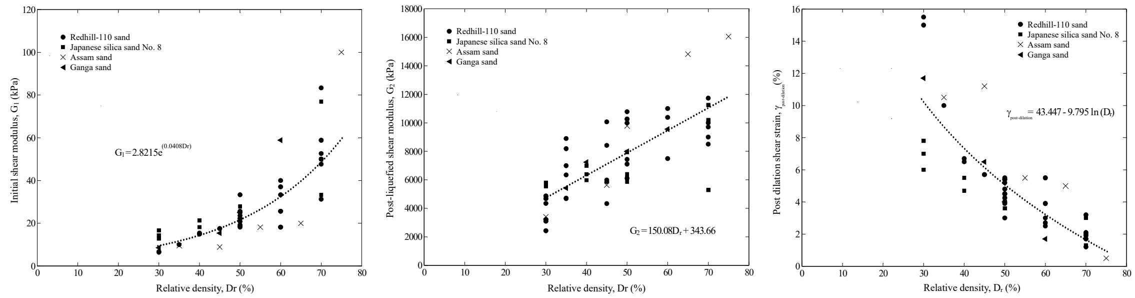


Figure 1. Post-liquefaction behaviour of liquefied sand: (a) shear strain versus shear stress; and (b) shear strain versus excess pore water pressure ratio.



(a) (b) (c)

Figure 2. Variation with respect to relative density of: (a) G_1; (b) G_2; and (c) ($\gamma_{post\text{-}dilation}$) for σ'_c=100 kPa.

Next, similar series of tests were performed on reconstituted natural pumiceous deposits (from Waikato Region, NZ) and Toyoura sand. Figure 3 illustrates the effect of relative density on the post liquefaction behaviour of hard-grained Toyoura sand and crushable pumice deposits. As seen from the figure, when the pumiceous material (regardless of relative density) liquefied and reached excess pore water pressure of 100%, it has a much more noticeable $G_1$when compared to that of Toyoura sand. This can be attributed, partly at least, to the high angularity of pumice particles which induced interlocking between them when monotonically sheared. In addition, it is clearly seen that $\gamma_{post\text{-}dilation}$ is smaller for pumice materials, owing to its more dilative response which resulted in an early decrease in excess pore water pressure. Further analyses of the results are presented by Asadi et al. (2016).

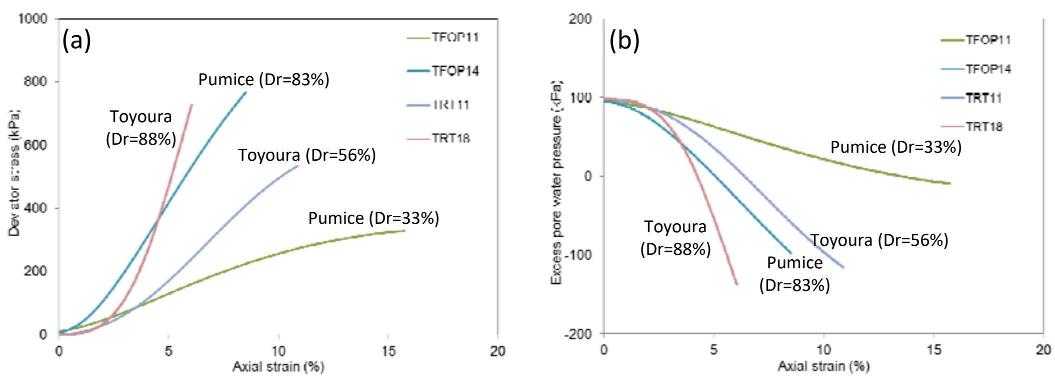


Figure 3. Comparison of the post-liquefaction undrained monotonic behaviour of Toyoura sand and pumice sand: (a) stress-strain relation; and (b) pore water pressure response.

3.0 ACKNOWLEDGEMENTS

A part of this abstract is based on research conducted while the author was on sabbatical leave at the University of Surrey, U.K. in 2015.

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Effects of Liquefaction-Induced Lateral Spreading on Structures and Lifelines

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Abstract

Current State of the Art

This abstract focuses on underground lifelines, particularly pipelines, conduits, cables, vaults, and tunnels. The current state of the art recognizes that underground infrastructure is affected by both differential lateral ground movement, often expressed as lateral ground strain, and differential vertical ground movement, often expressed as angular distortion or deflection ratio. Underground lifelines are affected by both simultaneously. Although horizontal ground displacement is regarded as the principal movement associated with lateral spreading, significant vertical ground movements accompany lateral speading. Thus, lateral spreads generate lateral and vertical movements, both of which affect buried infrastructure and need to be characterized for an appropriate assessment of lifeline response.

Earthquake-induced ground deformation is also representative of extreme conditions of soil-structure interaction that accompany floods, hurricanes, landslides, large soil movements caused by tunneling and deep excavations, and subsidence resulting from dewatering and/or withdrawal of minerals and fluids during mining and oil production. Hence, lifeline performance during earthquakes provides a framework for the analysis and design of underground infrastructure that is resilient to a variety of natural and construction-related hazards.

There has been substantial research performed on the effects earthquake-induced permanent ground deformation on pipelines and conduits as well as the effects of ground deformation on tunnels and vaults. Both large-scale and centrifuge testing facilities have provided valuable experimental data to characterize how pipelines respond to abrupt ground deformation and to validate numerical models for soil-pipeline interaction. Three types of numerical models have been developed. In the most widely used modeling process the pipe is modeled as a beam, often with nonlinear material and geometric properties, and soil-pipe interactions orthogonal and parallel to the pipeline longitudinal axis are modeled by linear, multi-linear, and nonlinear relationships. Three dimensional models have been developed and successfully applied using shell elements in combination with multi-linear spring-slider elements and with continuum elements that use various constituent laws to represent soil behavior.

The earthquake response of pipeline systems, incorporating pipeline response to liquefaction-induced ground movement, has been modeled and used for engineering, planning, and policy for water supplies in California and elsewhere. These pipeline system simulations involve hydraulic network models and allow for estimates of system reliability regarding flow levels in various parts of the hydraulic network.

Investigations of the earthquake response of the Christchruch, NZ water supply, wastewater conveyance, and gas distribution systems during the Canterbury Earthquake Sequence have led to important findings. The fusion welded MDPE gas distribution system sustained virtually no damage in response to the combined effects of all CES earthquakes and associated liquefcation-iduced ground deformation. In contrast, the jointed pipelines in the water distribution system sustained several thousand repairs during the

CES. The superior performance of the gas distribution system is related primarily to the strength and ductility the polyethylene piping. High resolution LiDAR before and after each main CES earthquake has allowed water distribution system damage to be correlated with lateral ground strain and angular distortion in liquefied soils, thus for the first time quantifying the combined effects of lateral ground strain and differential settlment on pipeline damage.

The next generation earthquake resilient pipeline systems are under development, with the pipeline industry designing and manufacturing new products that accommodate liquefaction-induced ground movement. The core concept of earthquake resilient pipelines is geometric nonlinearity. The pipelines are designed to change in length and shape through axial extension/compression and deflection to adjust to differential ground movements without loss of continuity and internal pressure. These pipelines are being validated at large scale, and the next generation soil-pipeline interaction models (that incorporate geometric nonlinearity) are being developed.

Key Underlying Geologic Processes

Lateral spreading is often modeled or correlated with SPT and CPT values on the basis of slope and free face condtions either with level or sloping ground. This type of modeling is simplistic and ignores the gradient at the base of the liquefiable deposit. Moreover, it only takes account indirectly of the thickness of the liquefiable deposit. Key geologic features contributing to lateral spreading include stratigraphy, geomorphology, and topographic features. Often, lateral spreading is observed in a radial pattern around a landform, which involves locally elevated topography, such as a bend in a river or a sand dune. When underlain by a thick deposit of liquefiable soil, the landform tends to sink down and spread out.

Primary Mechanisms of Deformation

The lateral movement is actually driven in part by bearing capacity failure of the landform, and the lateral displacement is inherently coupled with vertical movement. As cracks develop in spreading ground, soil ejecta is lost to the ground surface, thereby exacerbating differential settlements. Lateral spreading at slopes or free faces that extend for long distances parallel to a river or abandoned river channel involve two-dimensional mechanisms of slope deformation, which also involve coupled horizontal and vertical soil movement.

Underground pipeline response to lateral spreading involves both differential vertical and lateral movement and needs to be analyzed for the effects of both. Ground deformation affects individual pipelines, such and trunk and transmission pipelines, and affects networks of distribution pipelines with cross-connections, tees, bends, and service connections.

Key Challenges to Better Evaluation Procedures

The mechanisms driving liquefaction-induced ground deformation are complex and diverse. Lateral spreading may be influenced locally by two-dimensional mechanisms of slope deformation or may be affected by more widespread deformation patterns associated with the three-dimensional sinking down and spreading out of landforms. It may be affected simultaneously both by local two-dimensional mechanisms and more widespread three-dimensional mechanisms of movement. Geometric nonlinearities in spreading ground, such as cracks propagating to the surface, slip surfaces, and ejecta pathways, are difficult to model and predict. The amounts of ejecta, corresponding volume losses, and coupled horizontal and vertical ground movement patterns are not possible to predict with current modeling procedures.

The performance of individual pipelines, conduits, and cables subject to liquefaction-induced ground deformation involve may different patterns of movement, different force-displacement interactions that depend on the orientation of movement relative to the longitudinal axis of the lifeline, and the frequency and orientation of abrupt ground movement that intersects the lifeline. The performance of a pipeline system depends on the complex patterns of ground deformation that are distributed spatially throughout the interconnected network.

Best Path Forward

The best path forward is a four-part process that involves: 1) evaluation of well-documented case histories, 2) physical modeling and experiments using large-scale testing and centrifuge facilities, 3) development of numerical models for soil-pipeline and soil-tunnel interaction validated by the large-scale and centrifuge testing as well as case history data, and 4) development of netwok models that simulate system performance. Well-documented case histories include the CES, its triggering of liquefaction and spatial distribution of liquefaction-induced ground deformation, and its effects on lifeline systems throughout Christchurch. Large-scale testing should be focused on the next generation earthquake-resilient pipelines, and both large-scale and centrifuge tests should be used to quantify fundamental soil-pipeline interaction mechanisms for numerical modeling. Two-dimensional numerical models should be refined and improved, and three-dimensional shell and continuum modeling should be advanced for the interaction between pipes/tunnels/vaults and liquefaction-induced ground deformation. Network models should be enhanced to simulate lifeline system performance under liquefaction-induced soil movements.

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PREDICTING LATERAL SPREADING DISPLACEMENTS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

An important challenge to practicing engineers is the prediction of the areal extent of a lateral spread and the associated distribution of displacement. This is an important issue because these displacements cause damage to overlying structures, as well as subsurface infrastructure (e.g., pipelines), and the level of displacement influences the design of the infrastructure, the decision to potentially perform soil improvement, and the areal extent of that soil improvement.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

We need to better understand how the geomorphology and geologic depositional processes influence lateral spread displacements. While we may assume that these geologic processes are represented in our subsurface characterization (e.g., CPT), there are examples where there may only be subtle differences in the CPT resistance but the geomorphology is very different and the observed displacements are different. It is not clear how to reconcile and combine the quantitative engineering analyses and qualitative geologic analyses, but we certainly have to a better job bringing geologic interpretations into our engineering analyses.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Displacement Datasets
The dataset of lateral spread displacements is relatively large (~450 displacement vectors in the Youd et al. 2002 dataset), but they represent fewer than 55 sites and are from only about 10 different earthquakes. Additionally, about 300 of these displacement vectors are from 14 sites from the 1971 Niigata earthquake. Thus, the dataset poorly samples the range of site and ground motion characteristics that influence lateral spread displacements. This dataset includes a significant number of older, lower quality case histories in an effort to increase the available data. However, these older, lower quality case histories do not have any ground motion information, which limits their usefulness in applying current liquefaction triggering techniques.

Predictive Displacement Models
Current techniques used to predict lateral spread displacements are mostly empirical (e.g., Youd et al. 2002) or semi-empirical (e.g., Zhang et al. 2004), and they are based on field measurements of displacement at lateral spread sites, as noted above. Because the underlying displacement datasets poorly sample the range of site and ground motion characteristics that influence lateral spread displacements, they become inaccurate when extrapolating beyond their limits. This is particularly true for the purely empirical predictive models that do not have constraints based on underlying physics and do not include any quantification of liquefaction triggering.

Finally, it is important to note that, in their development, these models treat each measurement location as an independent data point, even if they are located within the same lateral spread site. Thus these techniques ignore the interactions between ground conditions within the same lateral spread when predicting displacement.

2.5 PATHS FORWARD

Displacement Datasets

We need to significantly expand the dataset of well-characterized lateral spread case histories, and remove from the dataset case histories for which ground shaking information is not available.

New lateral spread case histories should take advantage of remote sensing measurements of displacements through either optical image correlation of satellite imagery or three-dimensional differencing of point clouds derived from LIDAR or digital photogrammetry (i.e., Structure from Motion, SfM). A recent paper that describes the use of these techniques is Rathje and Franke (in press). An example of the displacement details that can be captured by remote sensing is shown in Figure 1 for an area in eastern Christchurch affected by the 2010-2011 Canterbury Earthquake Sequence in New Zealand (Martin and Rathje 2014, Rathje et al. 2015).

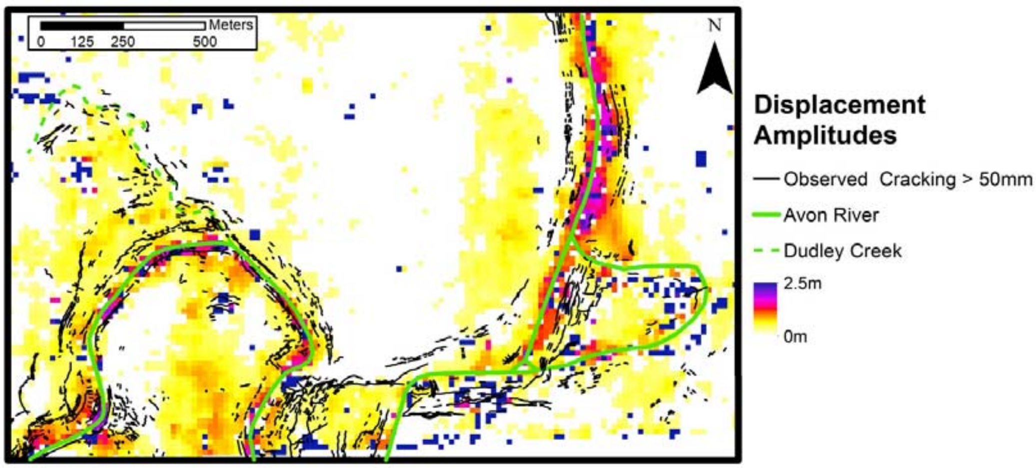

Figure 1. Amplitudes of horizontal displacement from optical image correlation (Martin and Rathje 2014).

Predicting Lateral Spread Displacements

We need to retire the purely empirical displacement models that are do not explicitly account for the triggering of liquefaction at a site because they do not properly incorporate the intensity of shaking at a site. Semi-empirical models that incorporate liquefaction triggering and strain potential should continue to be improved and refined. Improvements could include the treatment of interactions between ground conditions within the same lateral spread and the incorporation of geologic constraints.

We need to move towards applying more advanced modelling techniques (i.e., finite element analysis) to the prediction of lateral spread displacements. This requires well-documented case histories that can be used for validation purposes. Both field case histories and physical models should be considered for validation. Field case histories have the benefit of including real site conditions with spatial variability, but there will always be uncertainty regarding the proper input ground motion time histories to use. Physical models have the benefit of a known input motion, but the geometry and spatial variability often will not fully mimic field conditions.

3.0 ACKNOWLEDGEMENTS

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ROLE OF MICROSTRUCTURE IN LIQUEFACTION EVALUATION

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 & 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED SETTLEMENT & LATERAL SPREADING

The current case history database is composed of soils that are predominately young (Holocene-age), silica based and uncemented. However, in many parts of the world the simplified liquefaction evaluation methods are applied to a much wider range of soils resulting in conservative evaluations.

2.1 CURRENT STATE-OF-THE-ART

The current liquefaction evaluation methods are based on case histories that are bias toward sites that have experienced liquefaction and in soils that are young (i.e. Holocene-age), silica based and uncemented. Hence, they tend to result in conservative results when applied to soils outside of the existing database (e.g. older and/or lightly cemented soils).

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Many natural soils have some form of structure that can make their in-situ behavior different from those of very young uncemented soil. The term structure can be used to describe features either at the deposit scale (macrostructure), e.g. layering and fissures, or at the particle scale (microstructure), e.g. bonding/cementation. Older natural soils tend to have some microstructure caused by post depositional factors, of which the primary ones tend to be age and bonding (cementation). Microstructure tends to give a soil a strength and stiffness that cannot be accounted for by void ratio and stress history alone.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Key challenges are (1) how to identify and quantify the existence of soil microstructure and (2) how to incorporate microstructure into current liquefaction evaluation methods.

2.5 PATHS FORWARD

Andrus et al (2001), Robertson (2015, 2016) recently published methods to identify and quantify soil microstructure based on seismic CPT (SCPT) data and related this to liquefaction resistance. However, the current database from sites where the soils have some microstructure is limited. Hence, there is a need to collect additional case history data from a wider range of sites where factors such as aging and/or cementation exist to guide in developing improved scaling factors to account for microstructure.

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DRAGLOAD AND DOWNDRAG ON PILES FROM LIQUEFACTION INDUCED GROUND SETTLEMENT

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3.0 DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES.

1.0 CURRENT STATE-OF-THE-ART

There is significant controversy regarding appropriate procedures to account for liquefaction-induced dragloads from negative friction and resulting downdrag settlement. Although the neutral plane concept is likely the most theoretically sound approach for assessing behavior, there is considerable uncertainty about what negative skin friction develops in liquefied soil at equilibrium and how dragload should be considered in assessing the behavior of the pile. AASHTO procedures assume negative friction develops to the base of the liquefied layer, multiply skin friction with load factors and require the positive skin friction and end-bearing resistance multiplied by a resistance factor to exceed the negative friction and applied load (Strength-based approach). Others (Fellenius & Siegel, 2008, Rollins & Strand, 2006) argue that the neutral plane should be located by trial and error without factoring loads or resistance and that acceptable performance should be based on settlement (Settlement based approach). The strength based approach can often lead to expensive increases in pile length that would not be required using the settlement based approach.

2.0 KEY UNDERLYING GEOLOGIC PROCESSES

3.0 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Deep foundations can bypass liquefiable layers and bear in more competent strata at depth. Loads imposed on the pile foundation are typically resisted by positive skin friction acting on the side of the pile and by end-bearing resistance at the toe of the pile. However, when liquefaction occurs in a layer along the pile, settlement of that layer and the soil above it could exceed the settlement of the pile leading to negative skin friction along that length of the pile down to the neutral plane. Negative skin friction acting on the pile creates a “dragload” on the pile in addition to the permanent pile head load. The neutral plane is the depth where the settlement of the pile equals the settlement of the soil and also where the load in the pile is the greatest. Below the neutral plane, the positive skin friction and end-bearing pressure provide upward resistance which decreases the load in the pile. End-bearing resistance is dependent on the settlement of the toe which influences the location of the neutral plane. Although these concepts are well known, very few field or laboratory measurements are available to document the magnitude of negative skin friction that would develop in a liquefiable layer. In contrast to non-liquefiable layers, where the negative skin friction might simply be equivalent to the positive skin friction, the negative skin friction immediately following liquefaction is likely to be a very small fraction of the pre-liquefaction value or perhaps zero as observed in a blast liquefaction downdrag test shown in Fig. 1(Rollins & Strand, 2006). Nevertheless, as the excess pore pressures dissipate in the liquefiable layer, the skin friction at the pile-soil interface is likely to increase. In the blast liquefaction test, the negative skin friction after settlement increases to about 50% of the positive skin friction in this zone. Similar results were obtained from blast liquefaction tests in New Zealand.

4.0 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The key challenge to understanding downdrag behaviour and better predicting performance at present is primarily the lack of well-documented case histories. In the absence of this data, it will be very difficult to develop a consensus about which approach is correct. The major difficulty in developing case histories is that a variety of key measurements are necessary to understand behaviour. Variables include: liquefied thickness or pore pressure ratio versus depth, soil settlement versus depth, pile load versus depth, ultimate end-bearing resistance, end-bearing resistance versus toe displacement (q-z curve), and unit side resistance. It is unlikely that all this information will be available from a post-earthquake investigation. However, they could be obtained from a centrifuge test or from a blast liquefaction test. Lastly, improved understanding of q-z curves is critical in predicting behaviour.

5.0 PATHS FORWARD

It seems reasonable to expect that a combination of large-scale blast liquefaction testing and centrifuge testing could provide the necessary case histories to understand behavior and evaluate predictive methods. Researchers at BYU have recently partnered with T&T in New Zealand, INGV in Italy, and Univ. of Arkansas in the US to measure negative friction and downdrag settlement after inducing liquefaction around driven piles, auger-cast piles, micro-piles and drilled shafts. These test results should be valuable. Because the pore pressure time histories in earthquakes generate slower and often remain liquefied longer than in blast liquefaction tests, it would be valuable to perform some additional test results form a centrifuge model to confirm behavior from blast liquefaction tests.

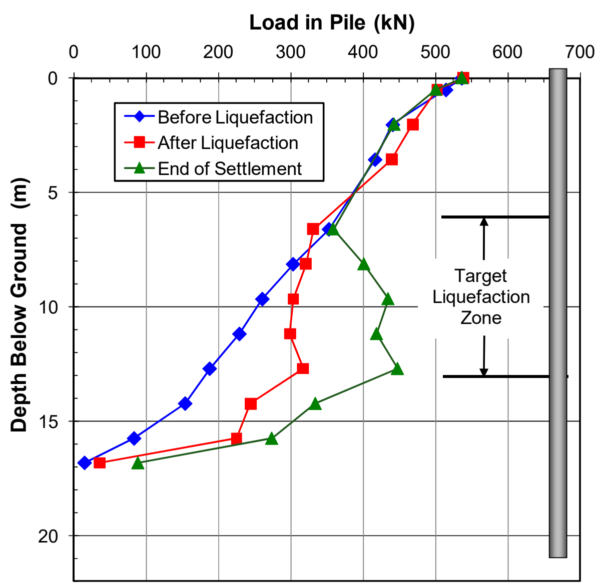

Figure 1. Pile load vs. depth curves before blasting, immediately after blasting and after settlement of the liquefied layer..

6.0 ACKNOWLEDGEMENTS

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LATERAL SPREAD DAMAGE TO BRIDGES AND PORT FACILITIES

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3.0 DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED LATERAL SPREADING ON STRUCTURES AND LIFELINES;

1.0 CURRENT STATE-OF-THE-ART

Commonly, a lateral spread displacement profile is determined using one of several methods for predicting lateral spread, such as that proposed by Youd et al (2002). The free-field displacement profile is connected to the bridge abutment and abutment piles using p-y springs within a computer model such as LPILE, for example. The p-y springs must adequately account for behavior within liquefied sand layers as well as passive versus deflection in non-liquefied soil adjacent to the abutment. The computer model is then used to determine the bending moment demand on the foundation and the displacement of the bridge foundation

2.0 KEY UNDERLYING GEOLOGIC PROCESSES

3.0 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Although research on lateral spread damage to structures originally focused on developing reasonable p-y curves in liquefiable sand, experience has taught us that the passive force-deflection relationship often governs the lateral force and resulting foundation displacement. Research conducted by a number of researchers (Rollins and Cole, 2005, Shamsabadi et al. 2007, Lemnitzer et al. 2008), has shown that the passive force in well compacted granular approach fills is best predicted using a log-spiral method. Maximum passive force develops with a displacement equal to 3 to 5% of the abutment backwall height with a hyperbolic curve force-deflection relationship. Passive force is significantly higher for gravel than for sand owing to higher friction angle and greater unit weight. More recent lab and large-scale testing has shown that passive force on abutments is significantly reduced as skew angle is increased (Rollins and Jessee, 2013; Marsh et al 2014) as shown in Fig. 1. For example, passive force is reduced by 50% with a skew angle of 30º. A reduction

4.0 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

A review of lateral spread displacement prediction equations relative to measured displacements at ports in Chile (Tryon et al, 2017) showed that most are methods only accurate with a factor of two. Furthermore, accuracy of several strain based methods required the Cetin depth correction factor in order produce accuracy within a factor of two. The Youd et al (2002) approach is extremely difficult to apply for large magnitude earthquakes (M8+) because the Joyner and Boore R value is zero in many of these cases. This yields unreasonably high displacement values. More sophisticated ground motion prediction equations are needed for these large earthquake events. Tryon et al. 2017 found that an approach which used the spectral acceleration at a period of 0.5 seconds gave better agreement than using the simple M & R approach proposed by Youd et al (2002).

5.0 PATHS FORWARD

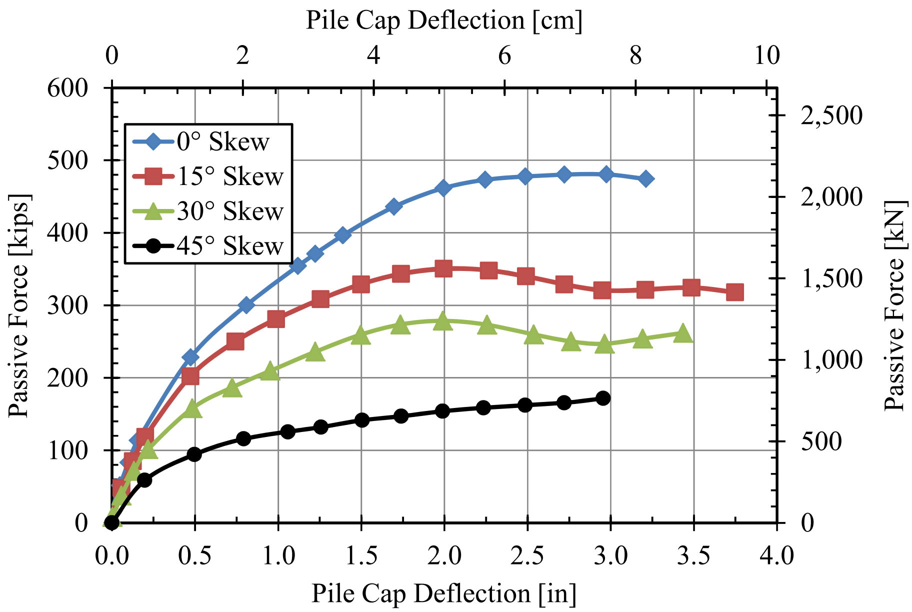

Figure 1. Passive force-deflection relationships from large-scale abutment load tests at skew angles of 0, 15º, 30º, and 45º

6.0 ACKNOWLEDGEMENTS

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CYCLIC SOFTENING OF FINE GRAINED SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1.0 DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED FLOW SLIDES

This abstract focuses on the cyclic softening of ***fine-grained, plastic, clay-like soils***, not transitional soils, or mixtures of sands and fine-grained soils. Although ground failures occur less frequently in plastic soils than in sand-like soils; the consequences of these failures are severe and result in substantial damage and fatalities. Examples of such failures are: the 4th Avenue, and Turnagain Height slides during the 1964 Prince William Sound, Alaska earthquake; bearing failures in Mexico City during the 1985 Michoacan, Mexico earthquake; bearing failures in the city of Adapazari in the 1999 Kocaeli, Turkey earthquake; bearing failures in the city of Wufeng during the 1999 Chi Chi, Taiwan earthquake; and bearing failures in the city of Bhuj, 2001 India earthquake. Understanding the mechanisms and factors affecting cyclic softening of clays and plastic silts under earthquake loading have great immediacy, as several seismically active major metropolitan areas in the world are underlain by plastic soils similar to those that have experienced ground failures in the past. In practice, engineers need reliable tools to predict the softening behavior of plastic soils.

1.1 CURRENT STATE-OF-THE-ART

Since 1999, researchers have proposed three different criteria to differentiate fine-grained soils that are susceptible and not susceptible to liquefaction (Boulanger and Idriss, 2006; Bray and Sancio, 2006; Seed et al., 2003), and one method to predict softening of fine-grained soils (Boulanger and Idriss, 2007). Idriss and Boulanger (Idriss and Boulanger 2008; Boulanger and Idriss 2006; Boulanger and Idriss 2004) have developed a stress-based approach that is currently the procedure widely used for predicting softening in plastic soils. Addition method uses a strain-based approach was developed by Mejia et al. (2009) and implemented by Tsai et al. (2014).

The Idriss and Boulanger method divides the analysis into two parts: 1) Initiation of softening- when soils strain by 3%; and 2) Consequences-the post-initiation displacements. For the initiation, the Idriss and Boulanger method presents the softening behavior of fine-grained soils using the normalized cyclic strength, which can be described by the equation: $\tau_{cyc}/S_u = a \cdot N^{-b}$, where a and b are curve fitting parameters, and N is the number of loading cycles. The two values, $\left(\tau_{cyc}/S_u\right)$, and b are critical values in the Boulanger and Idriss softening predictions, and the average values are assumed to be representative for all plastic soils. The developed approach is originally based upon data from cyclic direct simple shear tests (DSS) and cyclic triaxial (TX) tests performed at different times in laboratories over 20 years on remolded and undisturbed soil specimens. Dahl (2011) and, Dahl and Boulanger (2014) recently conducted a series of laboratory tests on transitional soils and clayey soils, and results were used to refine the existing Idriss and Boulanger approach.

Kaya and Erken (2015) conducted a series of cyclic triaxial tests of Adapasari soils and found some dependency of soil plasticity on the cyclic softening behavior. Dr. T. Leslie Youd (personal communication) observed that softening in clay caused serious ground failure during some earthquakes, notably, the 1964 Prince William Sound earthquake and landslides at Turnagain Heights and 4th Avenue. In other earthquakes, he has carefully searched for evidence of clay ground failures, and found none. Particularly, he found no physical evidence or historical records from the 1906 San Francisco earthquake and other smaller earthquakes induced ground failures in San Francisco Bay Mud.

Various factors affecting cyclic softening including but not limited to PI, LL, stress paths, sensitivity, age, cementation, fabric and structure, and a means to account for their effects in softening predictions remain unclear.

1.2 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

We must address the questions: *Why do some clays and plastic silts exhibit more softening than others? What are the factors affecting the softening behavior? What are the mechanisms involved in clays and plastic silts during earthquake loading?* In practice, engineers do not have access to expensive laboratory testing and they need a tool to connect index soil properties with the cyclic softening behavior of fine-grained soils similar to the liquefaction evaluation of sands using SPT or CPT-based liquefaction susceptibility criteria. Are index soil properties alone sufficient to screen and predict the cyclic softening behavior?

One controversy among geotechnical engineers is whether the mechanisms that lead to failures in fine-grained soils are the same as the mechanisms that cause liquefaction in sand. Completely understanding all of the mechanisms are very important, however the difficulty in isolating the overlapping influences of multiple factors remains a challenge. Field observations are challenging as the cyclic softening of clays and plastic silts are difficult to document.

3.0 ACKNOWLEDGEMENTS

This abstract is based on the literature review on cyclic softening of fine-grained soil developed collaboratively with Dr. James Bay at Utah State University and verbal communication with Dr. T. Leslie Youd at Brigham Young University. The author greatly appreciates their contribution.

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EVALUATION OF BRIDGE LOADING DUE TO LATERAL SPREADING

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CHALLENGE 2: DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED LATERAL SPREADING ON STRUCTURES AND LIFELINES

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Evaluation procedures generally fall into one of two broad classes: pseudo-static limit equilibrium or dynamic FEM. Recommendations for pseudo-static evaluation are provided in Ashford et al. (2011). These recommendations were extended by Caltrans and adopted as a formal design procedure (MTD 20-15). These procedures, with modest variation in implementation, have achieved broad usage in California transportation applications. Use of dynamic FEM is much less common and implementation is quite varied. Implementations range from 2D-FLAC based models that use DESRA-2 to estimate excess pore pressure ratios to advanced OPENSEES based fully coupled pressure dependent multi-yield surface models (Yang et al., 2003).

Opportunities to improve pseudo-static based procedures are limited by the gross approximations that comprise these procedures. Generally, if the ground displacement demand pushing on the bridge foundation were known, the corresponding estimates of structural demand are thought to be reasonably reliable. The ground displacement demand, unfortunately, is highly uncertain. The displacement estimate, typically based on some function of the ratio of the seismic coefficient k_h and the yield coefficient k_y, should be considered more of an *index* than a true estimate. At Caltrans, this index is used to separate displacement demand into broad categories of small, medium, and large. Since most bridges are ductilely designed and tolerant of substantial footing displacement (~10% of column height) reasonable design decisions can be reached despite large uncertainty in displacement demand. Buildings and pipelines represent challenges since their tolerance for ground displacement is often small.

The primary challenge for dynamic FEM models is developing an analytical framework for project design. Currently, these methods are primarily used in a research setting where the goal is to tune model parameters and try to match lab experiments. This kind of work is critical for model development but falls short in forward application. A second challenge is accounting for sample disturbance when laboratory testing to determine constituitive parameters. How are lost fabric effects accounted for?

2.5 PATHS FORWARD

The gold standard for any model is validation against observed performance. The pseudo-static method recommended by Ashford and implemented at Caltrans was the subject of validation by Brandenberg (2014) and by Ashford (unpublished) as well as some unpublished in-house evaluations. Generally, the procedure seems to do reasonably well at predicting small displacement demands in cases where small demands were observed. However, suitable case-histories of large displacement demand are not yet available to test the method's ability to predict large demand. Developing such case-histories should be a priority.

Another opportunity to improve pseudo-static based procedures would be to use dynamic FEM and one or more fully coupled constituitive models that include dilative effects (e.g. Yang, 2003) to calibrate displacement estimating procedures that are based on k_h/k_y (e.g. Bray and Travasarou, 2007).

These procedures were not developed for application to liquefaction but have been adopted to that use anyway. A calibration study would help identify possible biases and means to correct them.

Moving dynamic FEM into design practice will require the development of probabilistic based methods to account for the many sources of uncertainty including input motion. Priorities should include the development of constituitive model parameters in the form of probability distributions and covariance matrices to describe correlations between parameters.

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THE ROLE OF DEPOSITIONAL ENVIRONMENT AND FABRIC ON LIQUEFACTION SUSCEPTIBILITY

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1,2 & 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION …

All three topics of the workshop seem to take it for a given that the state of the art of evaluating liquefaction potential is at a point that it does not have to be addressed. However, the scattergrams presented in the most recent conferences show a great degree of uncertainty especially when it comes to borderline materials. While there are many sources of uncertainty that come into play, a glaring omission is the almost complete absence of the recognition of the role of the depositional fabric and depositional environment.

Terzaghi throughout his career emphasized the importance of geology, geologic setting and geologic detail in geotechnical engineering (see e.g. Terzaghi 1950). Clearly, the depositional environment and the source material influence the grain shape, grain composition, depositional fabric, and details of stratigraphy, which all must play a role in the liquefaction susceptibility and in the ultimate response of the deposit be it liquefaction induced lateral spreading, flow failure, or settlement.

2.1 CURRENT STATE-OF-THE-ART

There is actually extensive literature on the role of fabric on the liquefaction susceptibility of samples prepared in the laboratory. In fact this issue has been well recognized early on and revisited regularly by many investigators over the last 30+ years (see e.g. Mulilis et al. 1977, Ishihara 1993, Zlatovic et al. 1997, Vaid et al. 1999, Wood et al. 2008). Yet, the same level of recognition does not seem to have permeated into general field practice in terms of developing or employing site investigation tools capable of distinguishing the differences in material consistency to variations in density and packing with a sufficient degree of accuracy. Specifically, SPT has been the norm due to its easy availability and simplicity, even though it is a crude tool incapable of allowing investigators to make the distinctions in fabric that directly affect the strength of the material and hence its liquefaction resistance.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

Depositional Fabric – Depositional fabric is a combination of grain shape, degree of rounding, grain orientation, and packing. The grain shape and rounding are very much a function of the source rock area and the distance of transport. Grains produced from foliated metamorphic or thinly bedded sedimentary rocks will tend to retain elongate shape for a significant distance from the source, whereas grains from rocks with a random arrangement of grains without preferred shape will tend to be more equidimensional and rounded. As a result, the depositional fabric can be, and usually will be, very much controlled by the grain shape and strength of the current at the time of deposition, as illustrated in the photograph in Figure 1.

The photograph on the left clearly illustrates how grain shape influences the fabric in a natural deposit. Such fabric cannot be reproduced in any laboratory test using reconstituted materials and it cannot be recognized from downhole SPT tests. Moreover, split spoon samples of material like this would be disturbed to the point that the fabric would be unrecognizable on casual inspection. The photograph on the right is an excellent illustration of the influence of packing and grain orientation in

Figure 1. Left: Elongate gravel particles arranged in an imbricate (shingle-like) fabric in glacial outwash gravels, Utah. Right: Oval, well rounded boulders arranged in a tight packing stand at a much steeper slope than randomly deposited boulders of the same type, Japan (N. Sitar originals).

the apparent strength of the material, since both stacks have the identical materials with the same material friction angle. The only difference is packing and grain orientation.

Another aspect of the fabric and packing is illustrated in Figure 2. This fine to medium sand was deposited in a beach environment and the continuous rolling of grains back and forth gave them very well rounded shape and also produced a very tight, interlocking fabric. Again this type of fabric cannot be reproduced in the laboratory nor it can be recognized by any standard field test; however, it will clearly affect the strength and, hence, the liquefaction potential of the deposit. These are just two simple examples of the multitude of the potential geometries of grain-to-grain contact and grain orientation resulting from deposition of different types of sediments in different geologic settings.

Figure 2. Photograph of the fabric in fine to medium beach sand showing the intimate packing of the grains with concave-convex mating surfaces (N. Sitar original)

Stratigraphic Detail – Grain size distribution, i.e. percent of fines, and the plasticity of the fines has become one of the cornerstones of the currently debated approaches mentioned above. However, the problem lies in the size of the "representative" sample from which this data is derived. Figure 3 is a photograph of a clean, glacial outwash sand deposit. The drying of the cut surface nicely exposed the details of stratification within this deposit, which shows that there are distinct layers with clearly different grain size. Nevertheless, if one considers the length of the sampling interval for a typical split spoon sample (18 in), it is clear that the grain size distribution will be very similar if not identical over any depth interval one may choose in that section. Hence, the conclusion based on grain size analysis would be that this particular deposit is quite uniform, which obviously is not the case.

Figure 1. Fine layering in a glacial sand deposit (N. Sitar original)

The issue of fine stratigraphic detail becomes acutely important the moment the focus shifts on the issue of % of fines and their plasticity. Historically, the various liquefaction triggering analyses focused on the % of fines and their plasticity in order to determine which deposits are or are not likely to liquefy. The inherent assumption in these analyses is that the fines are uniformly distributed throughout the respective sample intervals, which may be the case for artificial fills, but is highly unlikely in most fluvial depositional environments. The plasticity of the fines, while clearly an important parameter from material behavior standpoint, is similarly affected, i.e. uniformly distributed plastic fines represent a completely different setting than fines confined to a very narrow band, which again is typical of fluvial environments.

The fact that thin layers of liquefiable, or extremely strain softening material can cause significant problems is well recognized, e.g. the Turnaigan Heights landslide triggered by the 1964 Alaska Earthquake (Seed 1968, Stark and Contreras 1998) and there are many others. Thus, whatever method we use to evaluate site stratigraphy and material properties, the method has to be capable of identifying such layers a priori rather than in a post-failure investigation.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES AND PATH FORWARD

Probably the most important challenge is to recognize the shortcomings of the past approaches and the field techniques used to get the legacy data. The next challenge is to start parsing the available data on the basis of the depositional environment, depositional fabric and grain shape, and stratification. In parallel there is a need for a detailed mechanical study of the depositional fabrics formed in the different environments and their mechanical response to cyclic loading on the scale of individual layers rather than in aggregate across the full depth of the deposit. In addition, it is time to start looking at the overall geologic setting, i.e. is it appropriate to equate the behaviour of granular soils derived from young volcanics in Japan to granular soils in floodplains of glacial rivers draining the granitic batholiths in California etc.

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INFLUENCE OF SOIL-STRUCTURE-INTERACTION ON GROUND FAILURE IN FINE-GRAINED SOILS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Ground failure case histories often occur in broadly distributed areas involving failure of soil both in the free-field and beneath structures. These are straightforward cases in which cause and effect can clearly be established: The earthquake *caused* failure of the soil, which exerted a negative *effect* on the structure. However, there have been a number of case histories in which ground failure occurred in the soil only beneath the structure, and not in the free-field. These cases tend to occur in fine-grained soils that might be considered non-susceptible to liquefaction by certain criteria. In these cases, the cause and effect relationship is not so straightforward: The earthquake *caused* the ground and the structure to shake, and ground failure and the accompanying negative impact on the structure were the *effect* of the combination of stresses from free-field wave propagation, and from stresses imposed by the vibrating structure. This is fundamentally a soil-structure-interaction issue. Traditional procedures for evaluating liquefaction triggering and its effects typically are formulated for free-field conditions, neglecting the increment of stress imposed by the structure.

2.1 CURRENT STATE-OF-THE-ART

The current state of the art for cyclic failure of fine-grained soils involves assessment of their failure potential based on either (i) a combination of index properties, such as liquid limit, plastic limit, clay fraction, and water content (Bray and Sancio 2006, Wang 1979), or (ii) liquid limit and plastic limit combined with an assessment of whether the soil will behave in a "clay-like" or "sand-like" manner (Boulanger and Idriss 2007). However, cyclic shear stress (CSR) in these methodologies is formulated for one-dimensional shaking conditions in which the CSR is a function of horizontal shaking intensity at the ground surface combined with a reduction factor for depth and adjustments for magnitude and vertical effective stress. These procedures routinely ignore the stress increments induced by soil-structure-interaction.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

The geologic processes controlling susceptibility to ground failure in these cases are the same as those in the free-field. There is ample evidence from case studies of the limitations of susceptibility criteria that are based on index tests. Our position is that testing that reveals fundamental aspects of soil behavior (e.g., undrained strength normalization) should be the first choice for critical projects in lieu of index test-based methods. However, the relative merits of such an approach remain to be confirmed on a statistical basis. An excellent opportunity in this regard is a large number of liquefaction and no-ground failure observations from Mihama Ward, Chiba Japan during the 2011 Tohoku earthquake.

This case study is described by Sekiguchi and Nakai (2012) and is being investigated further as part of the NGL project.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Several key mechanisms are involved in the phenomenon, as described by Rollins and Seed (1990). Seismic waves propagating through the soil interact with a structure, which vibrates and propagates waves back down through the soil. The phasing of these waves depends on the frequency content of the motion, stiffness of the soil, and the characteristics of the structure and its foundation. The phasing between these sources is currently not well understood. Another pertinent aspect is that the structure exerts static vertical stresses on the soil that alter its shear strength, stiffness, and susceptibility to liquefaction or cyclic softening. For example, the stress history (consolidation stress and overconsolidation ratio) is different beneath a structure than in the free-field. This alters the cyclic resistance ratio of the soil due to overburden and static shear effects, and also potentially the propagation of seismic waves due to the stiffening effect due to increased stiffness beneath the structure.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

There are two primary challenges to developing better evaluation procedures. The first key challenge is that the database of cyclic strength of fine grained soils from case histories and laboratory tests is very limited. More data would be very helpful for constraining this phenomenon and for identifying key aspects of behavior. The second challenge is that analytical solutions for stresses induced by surface loading are abundant for static loading conditions, but relatively sparse for dynamic conditions. Heidarzadeh (2015) recently developed solutions using a boundary element code for vertical harmonic surface point loading (Fig. 1). These solutions represent the components of the Cauchy stress tensor along with phase lag relative to the surface load as a function of dimensionless frequency and distance parameters. These solutions are amenable to integration to obtain various types of surface loading, but this work has not yet been performed. Numerical solutions, of course, can capture such behavior, but simpler solutions are needed for routine use in practice.

2.5 PATHS FORWARD

Laboratory testing of fine-grained soils is needed to better understand their cyclic failure potential. The existing database is rather sparse, and is generally accessible in digital form only to the owners of the data. A publicly available database of laboratory test data is needed to help advance knowledge in this area. Furthermore, there is a need for researchers to develop solutions for components of the Cauchy stress tensor beneath vibrating structures for a range of different foundation conditions (rigid, flexible, intermediate stiffness, shallow foundations, deep foundations, etc.).

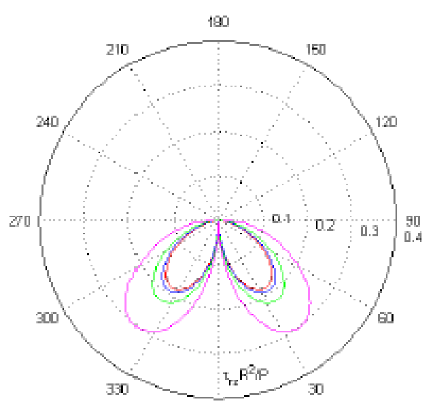

Figure 1. Contours of shear stress (τ_{rz}) for a harmonic vertical point load with dimensionless frequency $\omega R/V_{S=}4$ (Heidarzadeh 2015).

3.0 ACKNOWLEDGEMENTS

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IMPROVEMENTS IN FIELD SEISMIC AND LABORATORY MEASUREMENTS USED TO IDENTIFY AND CHARACTERIZE LIQUEFIABLE SOILS AND PREDICT CONSEQUENCES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED LATERAL SPREADING

Prediction of the final stage leading to the development of a highly-softened to fluidized soil and then prediction of the lateral spreading over various distances that depend upon the subsurface conditions, continuity of layers and surface topography and finally concluding with prediction of effects of the lateral spreading on the natural and built environments is a complex problem yet to be solved.

2.1 CURRENT STATE-OF-THE-ART

Over my career, I have been involved with small-strain field seismic measurements and small- to moderate-strain laboratory measurements using torsional resonant column and cyclic torsional shear tests (Figs. 1 through 4). The field seismic measurements up to this time are used to evaluate 1-D V_S profiles and sometimes create pseudo 2-D V_S profiles by stitching 1-D profiles together. This type of field seismic testing is not adequate for evaluating the lateral continuity of liquefiable materials that is required to predict lateral movements. Laboratory testing such as cyclic triaxial and cyclic simple shear tests apply rather simple anisotropic states of stress and have limited control over large following deformations that are required to predict large lateral movements in liquefaction-induced flow slides.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

One of the key challenges to adequately evaluating the lateral continuity of liquefiable materials that is required to predict lateral movements is 3-D V_S profiling. This challenge requires improvements in field seismic measurements and computational modeling. One of the key challenges in laboratory testing is to develop the capability of making large deformational measurements in the soften material while continuously monitoring the pore pressure and material-skeleton stiffness.

3.0 ACKNOWLEDGEMENTS

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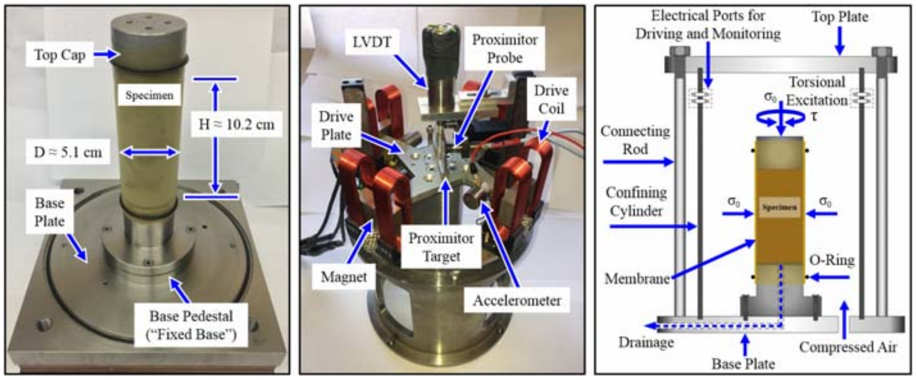

(a) Reconstituted sand specimen on base pedestal and confined using a vacuum

(b) Driving and monitoring systems on support cylinder

(c) Simplified schematic of RCTS confining system (driving system not shown)

Figure 1. Reconstituted sand specimen (a), photographs of the coil-magnet drive system and accelerometer, proximitor and LVDT monitoring system (b) and simplified schematic of RCTS confining system (c) (Wang et al., 2017).

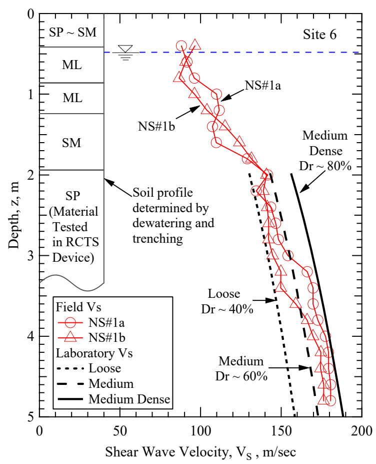

Figure 2. Comparison of small-strain shear wave velocities from field and laboratory measurements (Wang et al., 2017).

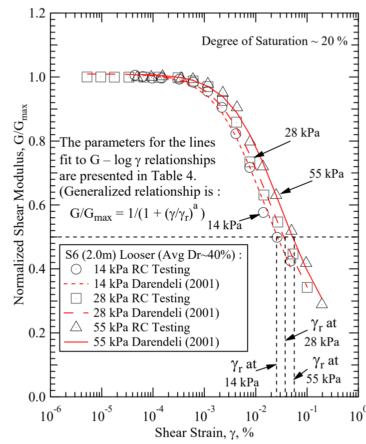

Figure 3 Variations in the G/G_{max} – log γ relationships at values of σ_0' of 14, 28 and 55 kPa from RC testing of a reconstituted, loose sand specimen from 2 m (Wang et al., 2017).

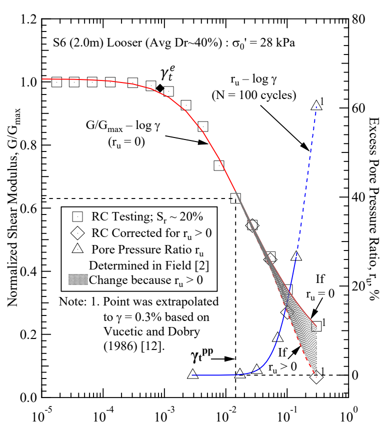

Figure 4 Example showing the change in the G/G_{max} – log γ relationship when excess pore pressure is generated (Wang et al., 2017).

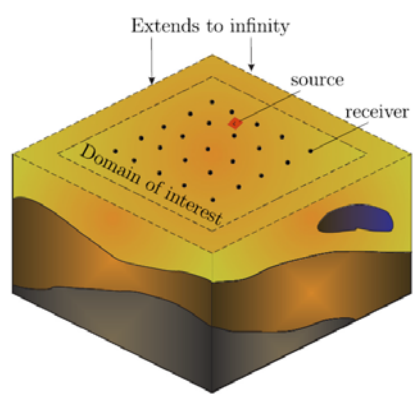

Figure 5 Problem definition: (a) interrogation of a heterogeneous semi-infinite domain by an active seismic source (computational model truncated from the semi-infinite medium via the introduction of perfectly-matched-layers (PMLs)) (Fathi et al., 2016).

POST-LIQUEFACTION BEHAVIOUR OF SILTY SANDS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

Following the sequence of earthquakes which affected Christchurch during 2010 and 2011, it was noted that the ground performance was better than expected in a number of areas across the city, particularly where layers of silty soil existed in the upper regions of the soil profile. Detailed investigation was carried out at a number of research sites, incorporating areas where liquefaction triggering was expected and that the response was both better-than-expected, and as-expected. Data from the cyclic triaxial testing (Stringer et al. 2015, Beyzaei et al. 2015) has indicated that the resistance of these soils against liquefaction was not large enough to be able to explain discrepancies between the predicted and observed responses, and in some cases was similar to those expected from simplified triggering methods. Hence, it is expected that liquefaction or significant softening would have occurred in some of the soil layers at these sites during the Christchurch earthquakes.

For residential buildings built on shallow foundations, total settlements are likely to be less important than differential settlements which cause tilting or cracking of the structure. In cases where the crust is able to withstand the shearing loads of the superstructure during the strong ground motion (i.e. where liquefaction is triggered at depth), the release of excess pore pressures from the liquefied layers may become important in understanding the likely damage occurring as ground settlements develop.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Liquefaction in sand deposits is often associated with sand boils which provide the surface “evidence” that liquefaction has occurred. These form as the result of water draining out of a liquefied soil layer and carrying soil grains with it as a result of the viscous drag forces and/or pressure difference between the liquefied soil layer and the surface.

One of the characteristics of some Christchurch silty soils which were investigated was the high percentage of non-plastic fines. The dissipation of excess pore pressures after cyclic loading in these soils often took place over an extended period, often upwards of several minutes for a 10cm specimen. This is contrasted by the dissipation of excess pore pressures in cleaner sands which occurs on the order of seconds.

This difference (as a result of permeability) may lead to a significant change in the development of ground settlement. In the case of a clean sand where fluid flow can be very rapid, an initial flow channel to the surface may form. The ensuing flow through the channel is fast enough to carry with it ejecta material, and material is carried to the surface, resulting in chaotic redistribution of material at depth, and potentially greater differential settlements to a structure located in the same area. However, where the material is more silty, the flow out of the “liquefied” layer is much slower, and as a consequence, flow channels may begin to form at depth but is ultimately limited by the low flow of water from other parts of the soil layer. Hence, sand boils might be restricted, and the ensuing settlement might be much more one dimensional, leading to better performance of the overlying structures.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

In considering the cyclic behavior of soils, there is a tendency to classify soils as either being prone to liquefaction in the case of “sandy” materials, or to cyclic softening, in the case of “clayey” materials.

Having distinguished a material as being prone to liquefaction, the focus in a routine analysis (i.e. not involving detailed numerical modelling calibrated against element test data) is on making "corrections" to various parameters so that the behavior can be compared to a clean sand. This approach has been useful and necessary in the development of tools which allow the economic assessment of liquefaction susceptibility. However, as described in section 2.3, this approach may not be appropriate for establishing what happens to soil behavior once liquefaction has been triggered, with some mechanisms appropriate to clean sands not being applicable to a silt or silty sand. In this regard, a key challenge in improving our evaluation procedures centers on our desire to make use of existing case histories, test results and behavioral frameworks, while at the same time needing to understand the behavior of a particular material in a particular system.

2.5 PATHS FORWARD

To improve our ability to evaluate the development of settlements and their effect on structures, we need to understand better the specifics of how materials behave after liquefaction has been triggered and how the soil behaves within the larger soil system that it is part of.

3.0 ACKNOWLEDGEMENTS

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SPATIAL VARIABILITY OF SOIL AND IMPACT ON LIQUEFACTION-INDUCED DIFFERENTIAL SETTLEMENTS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED SETTLEMENT ON STRUCTURES AND LIFELINES

In the absence of sufficient crust thickness such that shear-induced vertical movements are minimized, liquefaction-induced differential settlements, δ_d, arise from: (1) spatial variability in strata thickness, (2) spatial variability of the soil within a given stratum, or (3) both types of variability. *Bray et al.* (2014) described specific case histories in detail associated with (1), focusing on deformation performance of structures that experienced shear- and ejecta-induced deformations arising from near-surface liquefiable layers and large magnitudes of δ_d. Differential settlements arising from differences in stratigraphic thickness can be predicted with sufficient exploration of the subsurface and indices such as the LSN (*van Ballegooy et al.* 2014). A significant challenge remains in identifying the potential for damaging δ_d in cases where a relatively uniform thickness of a liquefiable layer exists across a site (2), and the key to addressing this particular challenge lies in connecting the geological deposition process with the subsequent inherent autocorrelation of spatially-varying soil characteristics.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

Although the processes associated with deposition are not random, they can often be modeled using random field model (RFM) parameters. For example, beach sands are worked by wind and waves, and the predominant wind speed, and wave height and speed over a given geologic time period will dictate the sorting of grain sizes, fines content (*FC*), and other characteristics of interest that are inferable using in-situ tests. Beach sands and desiccated clays are expected to exhibit relatively short autocorrelation, whereas deltaic topset or lacustrine deposits are expected to exhibit much longer autocorrelation.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

We generally separate the spatial variability in an in-situ measurement into: (i) a deterministic trend function; (ii) a randomly fluctuating component representing the variance in the measurement, typically the coefficient of variation (COV); and (iii) a measure of autocorrelation, typically the scale of fluctuation (Phoon and Kulhawy 1999). In sufficient numbers, in-situ tests can be used to characterize and link a deposition process to its inherent spatial variability. *Stuedlein et al.* (2016a) and *Gianella and Stuedlein* (2016b) describe a ground improvement test site characterized with 25 pre-densification CPTs and 10 borings within a 3 m × 26 m plan area. The test site and soil layers were horizontal and uniform; however, the site experienced significant δ_d (Figure 1*a*) following blasting conducted to evaluate a liquefaction mitigation method. During the course of blasting, many of the PPTs indicated instances of instantaneous liquefaction and these instances were distributed in a relatively spatially uniform manner (though none indicated residual post-blasting pore pressures associated with sustained liquefaction). Inspection of sample-based *FC*, q_t, and the site-specific

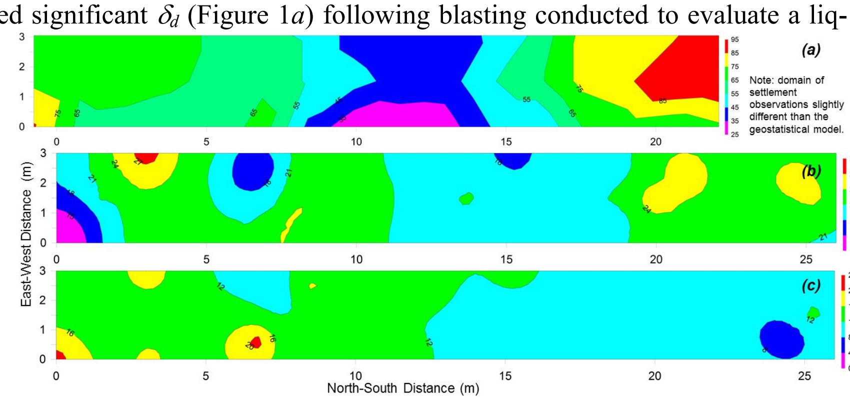

Figure 1. Counter maps of (a) observed excess PWP dissipation-induced settlements (mm), (b) liquefaction-induced settlements (mm) computed for $PGA = 0.12g$ and $M_w = 7.5$, and (c) vertically-averaged *FC* (%) in the liquefiable layer (2.5 to 11m depth). *Note different scales.*

CPT-based FC for individual CPTs would not have readily suggested the possibility of significant δ_d. However, a 3D model of the test volume developed using calibrated geostatistical models in kriging operations (on a 0.25 m grid) and in terms of q_t, f_s, FC, and q_{c1N} and q_{c1NCS} (using *Boulanger and Idriss* 2015) allowed computation of volumetric strains (using Yoshimine et al. 2006) and the variation in settlement with PGA (e.g., Figure 1*b*, PGA = 0.12*g*, M_w = 7.5). Although not directly comparable, regions of higher and smaller δ_d are similarly distributed across the plan area, indicating that the pattern of deformation could be predicted using the 3D geostatistical model of the test site.

In order to assess the likelihood of poor structure performance, the δ_d of structure bays of various widths founded on 1 x 1 m wide footings were computed for each kriging grid and compared to allowable, serviceability limit state, and ultimate limit state (ULS) angular distortions, α_d, of 1/500, 1/300, and 1/170, respectively (*Skempton and MacDonald* 1956). Interestingly, the lower PGA events produced the largest α_d exceedance percentages, as those pockets of looser, siltier sands exhibited smaller CRRs and greater compressibility. As the PGA increases, more of the stratum liquefies and experiences strain, leading to more uniform settlements, and therefore smaller instances of exceeding a given α_d threshold. **Significantly, it is noted that the median horizontal scale of fluctuation of q_t, f_s, and FC of these beach sands is about 3.25 m, and it is the 3 m bay width (common for residential structures) that most frequently exceeds the ULS α_d threshold.** *Thus our key challenge to improving our assessments is to understand the link between spatial variability and the effective scale of a given structure.*

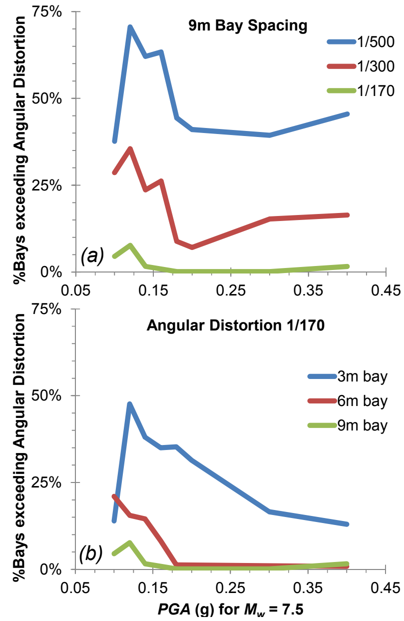

Figure 2. Percent of hypothetical structure bays exceeding angular distortion limits for test area as *f*(PGA): (a) 9m bay spacing, (b) the ULS limit of 1/170 for R/C frames.

2.5 PATHS FORWARD

The profession needs to improve the incorporation of geologic knowledge into site-specific plans for engineered structures. Quantifying the effect of the deposition process on spatial variability presents a fruitful avenue for understanding the spatial distribution of differential settlement-induced building damage, such as that described by *van Ballegooy et al.* (2014), and for its prediction. Post-earthquake reconnaissance efforts that collect sufficient data in programs specifically executed for the purpose of quantifying spatial variability (as done here) represent the best path forward for improving the pre-earthquake assessment of liquefaction–induced ground failure, and such efforts are *strongly* encouraged.

3.0 ACKNOWLEDGEMENTS

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Seismic Performance of Geotechnical Structures in Consideration of Seepage-induced Deterioration

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1.0 Liquefaction Effects Challenge: 1 – Development and Effects of Liquefaction-induced Flow Slides That Are Governed by the Undrained Residual Shear Strength of Liquefied Soil

The liquefaction-induced flow slide of earthwork is caused not only by rather uniform shear deformation of large liquefiable portion of the earthwork but also by localised shear deformation in a certain zone of the earthwork. Most of all the reported physical model tests show the former failure mode unless thin weak and/or less permeable layer is introduce in the slope or foundation ground to cause the localised shear deformation (Kulasingam *et al.*, 2004; Maharjan & Takahashi 2014). This failure mode can reasonably be captured by the finite element analysis with an appropriate soil model. However, majority of the liquefaction-induced severe damage of the actual earthwork in the past earthquakes seems to have occurred in the latter failure mode. One of the possible reasons is existence of weak and/or less permeable layer in the slope or foundation ground. Detection of this kind of layer prior to the event is difficult and perhaps such a zone has been formed due to *deterioration of the soil in the long term*. If this is the case, it is important to know (1) how natural process alters the soil fabric and (2) how the soil fabric change affects vulnerability of the soil against liquefaction.

2.2 Key Underlying Geologic Processes

Significant number of high embankments of mountainside roads has been damaged in the past earthquakes. Those embankments have been typically constructed on catchment topography such as swamps or valleys and have showed flow slide due to the liquefaction of the lower portion. Enomoto and Sasaki (2015) tried to mimic the flow slide of the high embankment in the 2007 Noto-Hanto Earthquake using the geotechnical centrifuge. They could reproduce the similar deformation pattern using sand of small fines content, which was different from the soil in the actual site. When they built the model embankment with the actual soil from the site, permanent deformation of the embankment was rather small. These facts suggest that the state of the soil was different from the actual one.

It is possible that the embankments built on the catchment topography have suffered from years of seepage-induced internal erosion, because of the environment they are subjected to. This process chronically makes the soil packing loose and consequently makes the embankments vulnerable to earthquake. Such soil deterioration should be properly considered in the seismic performance evaluation.

2.5 Paths Forward

The seepage-induced internal erosion can be one of the causes of soil deterioration. Ke and Takahashi (2014) developed a triaxial internal erosion apparatus capable of investigating not only the hydraulic characteristics of soils at the onset and in the progress of internal erosion but also the mechanical behaviour of the internally eroded soils sequentially. Using this apparatus, responses of the internally eroded soils in the triaxial compression were investigated under both drained condition (Ke & Takahashi, 2015) and undrained condition (Ouyang & Takahashi, 2016). In their tests, only the responses of gap-graded soils under monotonic compression were examined. To understand liquefaction potential and deformation characteristics of the internally eroded soils, liquefaction resistance of various erodible soils should be examined.

3.0 ACKNOWLEDGEMENTS

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EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING ON PILE FOUNDATIONS DURING A SUBDUCTION-ZONE EARTHQUAKE

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

Probability of occurrence of a huge subduction-zone EQ (M8-9) in west Japan is more than 70 % in 30 years. The seismic design of pile foundations for liquefaction induced lateral spreading of AIJ was based on case histories of 1995 Kobe EQ. The phenomenon of the lateral spreading of a huge subduction-zone EQ can be different from that of a near-field earthquake such as Kobe EQ. However, little is known about the lateral spreading caused by a huge subduction-zone earthquake.

2.1 CURRENT STATE-OF-THE-ART

Case histories of 1995 Kobe EQ showed that the failure modes of piles caused by the lateral spreading were different from those caused by liquefaction. The failure modes of piles at a sea side were different from those at a mountain side as shown in Fig. 1. Therefore, the structures were easy to be declined. The failure modes of the piles can be explained by the soil deformation.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Duration of a huge subduction-zone earthquake would be very long, more than 3 minutes. The lateral spreading occurred after the shaking in the case of Kobe EQ. Therefore, pile's damage caused by the soil deformation. The lateral spreading can occur during a shaking in the case of the subduction-zone earthquake. This suggests that not only the soil deformation but also the inertial force of a structure can affect piles.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

I'm interested in what is the best method to mitigate damage for the lateral spreading. A combination of piles and soil improvements?

2.5 PATHS FORWARD

Large scale centrifuge tests such as UC Davis?

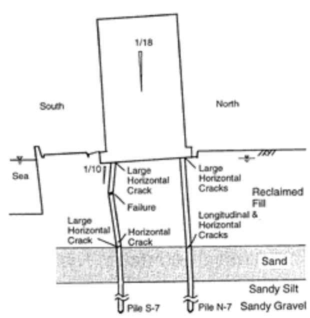

Figure 1. Deformation of piles caused by the lateral spreading, Kobe EQ. (Tokimatsu et al. 1998).

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LIQUEFACTION-INDUCED SETTLEMENTS OF BUILDINGS WITH SHALLOW FOUNDATIONS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 –LIQUEFATION-INDUCED SETTLEMENTS OF STRUCTURES WITH SHALLOW FOUNDATIONS

2.1 CURRENT STATE-OF-THE-ART

Extensive soil liquefaction occurred in reclaimed land of the Tokyo Bay area and the Tone River basin during the 2011 Tohoku Earthquakes, causing large settlement and titling of many wooden houses and low-rise reinforced concrete (RC) buildings both founded on shallow foundations. Similar liquefaction-induced damage to buildings took place during past strong earthquakes including the recent Christchurch and Kumamoto earthquakes. It seems that liquefaction-induced settlements of buildings with shallow foundations can be affected by various factors, relative effects of which have not been clearly identified, and that there exists no reliable method to estimate those values.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Based on reconnaissance studies regarding the 1964 Niigata earthquake and 1G shaking table tests, Yoshimi and Tokimatsu (1977) showed that the settlement of a RC building decreased as the ratio between building width and thickness of liquefied layer (W_B/H_L) increased. Tokimatsu et al (1992) suggested that the settlement and tilting of RC buildings during the 1990 Luzon earthquake were significantly affected by the contact pressure (q_B) and shear stress imposed by the buildings and their adjacent structures. Sancio et al (2004) showed that the settlement of RC buildings founded on thin liquefiable deposits during the 1999 Kocaeli earthquake were controlled not only by the contact pressure of the buildings (q_B) but also by their aspect ratios (H_B/W_B).

Ishihara (1985) suggested after the 1983 Nihonkai-chubu earthquake that the non-liquefied crust overlying a liquefied deposit, if its thickness (H_{NL}) exceeds 2-3 m, could reduce liquefaction-induced structural damage to wooden houses during earthquakes with peak ground acceleration of the order of 2 m/s^2. Based on reconnaissance studies on the 2011 Tohoku earthquakes, Tokimatsu et al (2012) found that: (1) Even where wooden houses settled or tilted, few upper structures suffered damage, which contrasts well with those observed about 35 years ago. This is because many buildings had adopted mat foundations or highly rigid foundations based on the lesson learned; (2) RC houses, and houses whose first floor or semi-basement was made of reinforced concrete, suffered relatively heavy settlement. This is probably because their ground contact pressure (q_B) was greater; and (3) The liquefaction-induced tilting angles of wooden houses tended to increase with increasing liquefaction-induced ground settlement.

Dashti et al (2010) suggested, based on centrifuge experiments simulating the performance of RC buildings, key parameters controlling liquefaction-induced building settlement are such factors as seismic demand, liquefaction layer thickness (H_L), foundation width (W_B), static shear stress ratio, building aspect ratio (H_B/W_B), building weight (q_B), and 3D drainage.

Based on similar centrifuge studies but simulating the performance of lighter buildings like wooden houses, Hino et al (2015) found that: (1) the relative settlement and tilt angle of a building increased as the crust thickness (H_{NL}) and the density of liquefied soil (D_{rL}) decreased or the thickness of the liquefied soil (H_L) and building self-weight (q_B) and mass eccentricity ratio increased; and (2) the effects of soil liquefaction below the crust on building damage were well accounted for by the safety factors against vertical force and static and dynamic overturning moments of the building together with the liquefaction severity of the underlying liquefiable deposit, represented by the integration of liquefaction-induced volumetric strain with depth.

A brief literature survey described above indicates that the liquefaction-induced settlement of buildings with shallow foundations could be affected by various factors, relative effects of which have not been clearly identified, and suggests the need for further study.

2.5 KEY CHALLENGES AND PATHS FORWARD

(1) Compilation of well documented case histories of liquefaction-induced ground and building settlements during resent earthquakes.
(2) Centrifuge experiments to identify relative effects of key parameters on building settlements.
(3) Clarification of effects of the key parameters described above as well as of other possible factors such as pore pressure migration and dissipation, earthquake sequence, and geological environment on ground and building settlements based on laboratory element and/or centrifuge tests.
(4) Refinement of numerical and design procedures to estimate ground and building settlements, which should be substantiated by well-documented case histories and centrifuge experiments.
(5) Development of cost effective mitigation techniques taking into account the key parameters controlling settlement and tilting of buildings.

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CONSIDERATIONS FOR LIQUEFACTION-INDUCED LATERAL SPREADING DEMANDS ON STRUCTURES – AN INDUSTRY PERSPECTIVE

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED LATERAL SPREADING

2.1 CURRENT STATE-OF-THE-ART

State-of-the-art approaches in industry involve, primarily, two-dimensional nonlinear dynamic effective stress analyses, with appropriately calibrated and validated constitutive models. Three-dimensional pseudostatic analyses are also performed, when warranted, primarily to assess residual (immediately post-shaking) conditions. Three-dimensional dynamic analyses with total stress models are performed on occasion, however, soil properties should be calibrated to ensure similar response between a 3D representative section and 2D effective stress analyses. Three-dimensional dynamic effective stress analyses are not currently common, due to the limited number of available constitutive models for liquefaction implemented in three dimensions.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The primary challenge associated with developing better evaluation procedures is having to describe a very complex phenomenon, involving excess pore pressure generation/dissipation, soil deformations, and soil/structure interaction, with a limited number of simplified assumptions and input parameters. For the majority of cases in practice, actual project conditions may vary significantly from idealized assumptions in simplified evaluation methods. Hence, there is a need for the evaluation procedures to encompass a wide range of soil conditions, foundation types, and structures. Oftentimes, spatial heterogeneity, discontinuous layering, interlayering, and localized phenomena control the liquefaction-induced demands and system response. These factors are difficult to capture by simplified methods.

2.5 PATHS FORWARD

The recommended path forward, to advance the community's understanding and to allow development of appropriate evaluation procedures, should involve a combination of experimental and analytical work. Furthermore, as understanding is improved for what are considered most common conditions, less commonly encountered soils and a wider range of foundation types and structures should also be considered.

Specifically, an expanded database of cases should be developed to help calibrate and validate numerical models and guide the development of simplified methodologies. This database should include both high-quality model-scale experiments and well-documented field case histories. On the numerical analyses front, additional work should be conducted to improve existing numerical models to capture key mechanisms associated with liquefaction, especially in relation to sloping ground conditions and across a wide range of relative densities and different soil types. Development of 3D tools and implementation of constitutive models in three dimensional space is needed, to allow for modeling of foundation types and conditions not adequately modeled in two dimensions (e.g., pile foundations, asymmetrical structures, etc.). The above 2D and 3D numerical tools can also be used to generate a database of hypothetical case histories, to fill-in data gaps in experimental and field observations, and allow for easier parametrization and development of simplified methods.

Finally, areas where additional research would be beneficial include, among others: a) assessing liquefaction-induced kinematic demands on pile foundations associated with soil lurching, b) near shore environments and structures (e.g., port structures retaining liquefiable soils), c) fine-grained low-plasticity and interlayered soils, and d) calcareous soils usually encountered in nearshore environments.

3.0 ACKNOWLEDGEMENTS

This abstract is mostly based on work conducted by Fugro's Earthquake Engineering group.

Ageing of soils affecting liquefaction triggering and undrained shear strength of soils

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 1 – Development and effects of liquefaction-induced flow slides that are governed by the undrained residual shear strength of liquefied soil

The author's group has been continuing to conduct the earthquake geotechnical reconnaissance investigations on the areas and sites affected by soil liquefaction during 2011 Great East Japan Earthquake, by using Swedish weight sounding tests. The current objectives of the investigations include liquefaction-induced river levee failures. In the fall of 2015, the authors have visited one of the sites, where the liquefaction-induced flow slides were observed during 2011 earthquake, and conducted some Swedish weight sounding tests. The outcome of the investigations will be published in the coming PBD conference, (Hyodo et al. 2017). What has intrigued the authors was that one particular section of the river levee has suffered from flow slides, despite the fact that the river levee of similar outer appearance extended over a distance. In addition, the values of Swedish penetration resistance were also similar to each other.

2.1 CURRENT STATE-OF-THE-ART

The author's group has been engaged in developing the procedures to estimate the undrained shear strength of soils governing liquefaction-induced instability of flow slides as well as the post-liquefaction settlement of silty sands. These outcomes were published recently in the two technical papers of Tsukamoto et al. (2009) and Tsukamoto et al (2010). These procedures are based primarily on Swedish weight sounding tests. However their principles can be used for other types of penetration tests.

2.2 KEY UNDERLYING GEOLOGIC PROCESSES

It was found that the failed section was located on the micro landform of old river channel, while the other non-failed sections were located on terrace. Therefore, the only difference between the failed and non-failed sections has to be found on their different micro landforms, which would suggest that ageing of soils should have certainly affected the occurrence of flow slides.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

Following 2011 earthquake in Japan, where reclaimed soil deposits were exclusively liquefied, one of the current issues affecting the practice of evaluating soil liquefaction triggering is unanimously how "ageing of soils" would affect soil liquefaction triggering. The accumulating research outcomes in US, including Andrus and Stokoe (2000), are quite impressive and promising.

The other issue should be found on the recurrence of soil liquefaction. In other words, would liquefaction occur again at the same sites where liquefaction occurred during past earthquakes ? This issue is quite sensitive to the residents of the reclaimed land along Tokyo bay, where extensive liquefaction was observed during 2011 earthquake. In fact, the recurrent liquefaction was confirmed at a number of sites during 2011 earthquake. This could be the old and new issue, (Finn et al. 1970). So the question arises as to the recurrent liquefaction resistance. The past studies indicated that the recurrent liquefaction resistance could be lower than the original resistance.

Therefore, the liquefaction resistance of soils could certainly be increased by ageing of soils, and also could be reduced by previous history of liquefaction, i.e. recurrent liquefaction. So the overall issue can be summarised as the converse effects of ageing and history of liquefaction.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

There is a myth to the author's understanding regarding the liquefaction resistance of silty sands and silts, that can be summarised as follows.

(a) It is known and the fact that laboratory reconstituted specimens of silty sands and silts tend to exhibit lower resistance to liquefaction that those of clean sands, though density should matter herein.
(b) The liquefaction resistances of silty sands and silts are estimated to be greater than clean sands, when they are evaluated based on SPT blow counts.
(c) It is reasonable and agreeable that the SPT penetration resistances of silty sands and silts tend to be lower than clean sands, due most probably to more contractive dilatancy of silty sands and silts than clean sands.
(d) One might criticize that large-strain SPT N-values would not suit to examining ageing effects of soils that would form fragile inter-granular structures, hence comes in the shear wave velocity measurement, (ex. Andrus and Stokoe 2000).
(e) One might assume that ageing inter-granular structures tend to be more easily developed or developable within silty sands and silts than clean sands, though their mineralogical origins should matter herein.

The author is aware of many publications linked to each topic described above especially in ASCE Journal of Geotechnical Engineering. With all these topics considered, the author is still interested in evaluating the liquefaction resistance and undrained shear strength of soil from field penetration tests, incorporating effectively the ageing of soils. However, the authors is not aware of what could be the most probable scenario on what degree the effects of ageing are important in estimating the liquefaction resistance and undrained shear strength of silty sands and silts.

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PREDICTION OF LIQUEFACTION-INDUCED DIFFERENTIAL GROUND SURFACE SETTLEMENT

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

The 2010 – 2011 Canterbury Earthquake Sequence (CES) affected the Canterbury region of New Zealand, resulting in widespread liquefaction ejecta, which was observed on approximately 50,000 residential properties (approximately one third of the urban residential flat land area of Christchurch). The residential building damage was closely correlated to the severity of liquefaction ejecta, i.e., the residential building damage was greater in areas with larger quantities of liquefaction ejecta surrounding each building. The main mechanism causing the liquefaction-induced building damage was differential ground surface settlement, mainly caused by:

- Liquefaction ejecta causing localised volume loss;
- Subsurface variation in liquefaction-induced volumetric densification;
- Topographic relevelling in areas with locally raised building platforms and incised roads; and
- Lateral spreading.

The liquefaction-induced differential ground surface settlement damaged approximately 15,000 residential buildings beyond economic repair. The total economic losses from the CES were in the order of NZ$40 billion, with approximately one third to one half of the economic losses being directly attributable to damage caused from liquefaction-induced differential ground surface settlement.

Being able to accurately predict whether liquefaction ejecta will occur and the resultant liquefaction-induced differential ground surface settlement for a given level of earthquake shaking, is important for the following reasons:

- Appropriate land zoning decisions can be made;
- Building foundations can be designed to withstand appropriate levels of differential ground surface settlement; and
- Building damage can be more accurately predicted for catastrophe loss modelling purposes.

2.1 CURRENT STATE-OF-THE-ART

There are no direct methods to evaluate the exact locations of where liquefaction ejecta will occur, and neither are there methods to directly evaluate the quantum of ejecta that is likely to be expelled at the ground surface. This makes it very challenging to predict the differential ground surface settlement that is likely to occur across a building footprint, particularly in areas away from rivers which are not affected by lateral spreading.

Liquefaction vulnerability parameters (e.g., S_{V1D}, LPI, LPI_{ISH} and LSN and the Ishihara H_1/H_2 ratio) are useful tools for providing an indirect method for predicting the likelihood of liquefaction-induced ground damage. Rather than attempting to predict the physical differential ground surface settlement directly, these parameters provide an index that can be used to indicate the likelihood of severity of liquefaction-induced differential ground surface settlement that might occur for a given level of earthquake shaking. They are typically used for area-wide liquefaction studies where efficient and consistent analysis of geotechnical investigation information (typically CPT or SPT data) can be used to identify relative differences in expected performance between different areas. These vulnerability parameters can also provide valuable insights to help guide more detailed engineering assessment on a site-specific basis.

While there is a reasonable correlation between these liquefaction vulnerability parameters and the observed liquefaction from the CES events, studies have shown that there can be a lot of bias and dispersion between the vulnerability parameters and the observed land damage (van Ballegooy et al., 2015 and Maurer et al., 2015). This is because each of the liquefaction vulnerability parameters have significant inherent simplifications which mean that certain physical soil attributes and mechanisms causing liquefaction-induced settlement and differential ground surface settlement are not explicitly accounted for including:

- Competency of the non-liquefying crust layer;
- Loss of soil ejected to the ground surface;
- Soil stratification (i.e. uniform to highly stratified);
- Three dimensional effects including spatial variability of soil properties and the lateral discontinuity of the soil strata;
- Soil saturation levels;
- Dynamic response of the soil profile;
- Influences from artesian groundwater pressures in underlying aquifers;
- Soil grain size, grain type, plasticity, fabric, aging effects; and
- Previous stress history of the soil.

These simplifications make the assessment of liquefaction vulnerability easier and less expensive to perform, but they also contribute to the bias and dispersion in the correlation between these parameters and actual liquefaction-induced differential ground surface settlement.

2.2 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

The liquefaction vulnerability parameters (e.g., S_{V1D}, LPI, LPI_{ISH} and LSN and the Ishihara H_1/H_2 ratio) all use the simplified CPT or SPT-based liquefaction triggering evaluation procedures (e.g. Robertson and Wride, 1998, Moss et al., 2006, Idriss and Boulanger, 2008 and Boulanger and Idriss, 2014) as the first step to determine which soil layers are likely to liquefy for a given level of earthquake shaking. These simplified liquefaction evaluation procedures all comprise a set of complex empirical equations.

Many of the above factors listed in the bullet points in Section 2.1 are being examined by numerous researchers and will result in potential “patch” equations for inclusion into the simplified liquefaction triggering and vulnerability assessment frameworks in an attempt to reduce some of the bias and dispersion (i.e. false positive and false negative predictions). However, there are significant challenges in incorporating these additional factors into the existing simplified liquefaction triggering and vulnerability assessment frameworks, with a risk that over time these frameworks may become significantly more complex and convoluted with the incorporation of “patching” equations.

2.3 PATHS FORWARD

An alternative and potentially simpler approach for improving the assessment of the potential for liquefaction ejecta and liquefaction-induced differential ground surface settlement is to develop and review a case history database. The concept is to develop a matrix of the typical geomorphologic characteristics, sediment characteristics, soil profile characteristics, groundwater levels and ground motion characteristics that do and do not result in liquefaction-induced differential ground surface settlement.

In addition to the 2010-2011 Canterbury earthquakes, there are eight other earthquakes in New Zealand that are known to have triggered liquefaction for which there are good observation records available. These earthquakes include the 1855 Wairarapa, 1929 Murchison, 1931 Hawkes Bay, 1968 Inangahua, 1987 Edgecumbe, 1991 Hawks Crag, 2008 Gisborne and 2014 Seddon earthquakes. Research is being undertaken to:

- Collate the liquefaction observations from all these earthquakes and digitize the extent of liquefaction from these observations;
- Back-calculate PGA contour estimates and modelled depths to groundwater at the time each of the earthquakes occurred for the areas affected by liquefaction;
- Compile all available geotechnical data to the New Zealand Geotechnical Database;
- Back calculate the liquefaction vulnerability parameters for each of the earthquakes and compare the predicted extent of liquefaction with the digitized extents of liquefaction to identify areas where the predicted extent of liquefaction is both consistent and inconsistent with that observed; and
- Compare the geomorphic setting, paleo-depositional setting, the subsurface sediment characteristics, sediment characteristics, soil profile characteristics, groundwater levels and ground motion characteristics where the simplified CPT and SPT-based prediction methods are correctly and incorrectly predicting liquefaction-induced land damage.

3.0 ACKNOWLEDGEMENTS

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PREDICTION OF LIQUEFACTION-INDUCED DAMAGE TO LIGHT WEIGHT, TIMBER FRAMED, SINGLE STOREY RESIDENTIAL BUILDINGS.

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFACTION-INDUCED SETTLEMENT ON STRUCTURES AND LIFELINES

The 2010 – 2011 Canterbury Earthquake Sequence (CES) caused widespread liquefaction ejecta across the Canterbury region of New Zealand. The liquefaction resulted in extensive differential ground surface settlement causing extensive liquefaction-induced damage to the light weight, timber framed, single storey residential building portfolio typically founded on shallow foundations. The liquefaction-induced differential ground surface settlement damaged approximately 25,000 residential buildings, of which 15,000 were beyond economic repair. The main cause of the building damage was that the residential building foundations had insufficient strength and stiffness to withstand the differential ground surface subsidence, resulting in deformation of the foundations supporting the building. For the buildings which only underwent planar differential settlement (i.e. differential subsidence without flexural distortion), in many cases the foundation systems of those buildings had insufficient strength and stiffness to be able to cost effectively relevel them.

It is important to be able to accurately predict liquefaction-induced differential ground surface settlement and the corresponding Building Foundation Differential Settlement (BFDS), for a given level of earthquake shaking, so that building foundations can be designed with sufficient strength and stiffness. The design objectives for residential buildings on shallow foundations would be to minimise the flexural distortions (which minimise the structural and cosmetic damage to the superstructure) and improve the re-levelability if planar differential settlement occurs. If the predicted liquefaction-induced differential ground surface settlement and the corresponding BFDS is too high, resulting in structural foundations solutions that are not cost effective, then alternatively, shallow ground improvements could be undertaken to reduce the expected ground surface flexural distortions and enable a more cost effective foundation solution to be built on the improved land.

Extensive residential building damage data sets have been collected following the CES including assessments of the mode and severity of BFDS for approximately 65,000 residential buildings in areas where liquefaction occurred. In addition, comprehensive floor level surveys have also been undertaken on approximately 2,500 buildings in areas where liquefaction occurred to measure the differential settlements and angular distortions of the building floors.

2.1 CURRENT STATE-OF-THE-ART

Correlations have been developed using data collected during and after the CES between the BFDS datasets and the liquefaction-induced vertical and horizontal ground surface deformations derived from LiDAR surveys and satellite imagery (collected after each of the major CES earthquakes). These correlations indicate a strong indirect link between the total vertical and horizontal liquefaction-related ground surface movements and BFDS (i.e. larger higher vertical and horizontal liquefaction-related ground surface movements result in an increased likelihood of liquefaction-induced differential ground surface settlement and corresponding BFDS). Such correlations have a useful application following an earthquake event, whereby rapid assessment of liquefaction-induced horizontal movements from satellite imagery and vertical movements from airborne LiDAR surveys can provide the information to more accurately and rapidly estimate the liquefaction-induced damage to buildings on a portfolio basis. This information is useful for estimating metrics such as financial losses and the quantum of people that are likely to be displaced due to building damage. The rapid assessment of these metrics after a disaster is essential to inform planning for an effective post-disaster recovery phase.

While these correlations have significantly less scatter compared with correlations with the CPT and SPT-based liquefaction vulnerability parameters (discussed below), these correlations cannot be easily used to predict BFDS for a given level of earthquake shaking. This is because there are no direct methods to predict liquefaction-induced differential ground surface settlement for a given level of earthquake shaking.

An extensive geotechnical investigation dataset comprising approximately 5,000 boreholes with laboratory test data and 22,000 CPT have been collated for the Christchurch area and is available in the New Zealand Geotechnical Database. Further, approximately 1,000 shallow standpipe piezometers have been installed and monitored across Christchurch to develop depth to groundwater models for each of the major CES earthquakes.

This has enabled the development of preliminary correlations between the foundation differential settlement and angular distortion of residential buildings on shallow foundations, and CPT and SPT-based liquefaction vulnerability parameters (such as LPI and LSN) based on the inferred earthquake shaking contours. These correlations can be used for predicting the liquefaction-induced BFDS of residential buildings located on liquefaction susceptible soil deposits.

2.2 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

While these preliminary correlations work well at predicting building damage on a portfolio basis, they do not work as well when trying to predict the likely damage of an individual building. This is because there is a lot of scatter in the preliminary correlations for the following reasons:

1. Uncertainty in predicting free-filed liquefaction-induced differential ground surface settlement in areas not affected by lateral spreading;
2. Approximately 10 to 20% of the portfolio of residential buildings that were assessed for building differential settlement were also in areas affected by lateral spreading;
3. The assessed building differential foundation settlements were as a result of the cumulative effect of multiple earthquakes from the CES; and
4. Assessments were conducted on a portfolio of varying:
    - Building geometries;
    - Foundation stiffness and strength as well as superstructure stiffness; and
    - Building weights.

The uncertainty in predicting liquefaction-induced differential ground surface settlement (item 1) is being investigated and addressed by various research teams (as discussed in the accompanying abstract). Aside from the uncertainties in predicting liquefaction-induced differential ground surface settlement, the other key challenge for improving the BFDS correlations is to isolate and identify the influence of the various elements affecting BDFS (items 2 to 4) by sub grouping the larger BFDS dataset into smaller datasets.

2.3 PATHS FORWARD

The large BFDS dataset needs to be grouped into smaller datasets to identify:

- Buildings within areas where lateral spreading occurred and areas where lateral spreading did not occur to quantify the effect of lateral spreading on BDFS; and
- Buildings in areas where the liquefaction-induced differential ground surface settlement occurred predominantly due to one earthquake and areas where it occurred as a result of multiple earthquakes to quantify the effect of multiple liquefaction triggering earthquakes on BDFS;

- The influence of building area, irregularity (i.e. building footprint complexity), foundation type, construction age, and building weight on BFDS.

Quantifying and incorporating these effects into the BDFS correlations should help reduce some of the large uncertainties in predicting BFDS.

3.0 ACKNOWLEDGEMENTS

This abstract is based on research funded by the New Zealand Earthquake Commission.

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EFFECT OF LIQUEFACTION INDUCED LATERAL SPREADING ON BRIDGES

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 2 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INDUCED LATERAL SPREADING

This abstract will focus on the performance of bridges in Christchurch that were subjected to liquefaction induced lateral spreading and two main aspects (1) characterizing the extent of lateral spreading displacement; and (2) capturing the bridge system response to these displacements.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

Estimation of the amount of free-field lateral spreading displacement that would develop at a site is a key input into infrastructure design. A range of methods to assess the total lateral spreading displacement and the region affected by these displacements in Christchurch during the Canterbury Earthquake Sequence (CES). This has included ground surveying (Cubrinovski & Robinson 2015, Robinson 2016), aerial photography and LiDAR (Martin & Rathje 2014). Robinson (2016) showed that the empirical model for lateral spreading of Youd et al. (2002) was unable to capture the measured lateral spreading response in Christchurch, typically overestimating the amount of displacement. The complex nature of the lateral spreading process and the influence of the characteristics of a wider region is the focus of current QuakeCoRE research to link the geomorphic and geotechnical characteristics of a region to better assess the potential for lateral spreading (van Ballegooy et al.).

Focusing in from free-field lateral spreading estimation, the subsequent step in the assessment of the effect of lateral spreading on bridges is estimation of the actual displacements that are experienced at the bridge abutments. In Christchurch the bridges affected by lateral spreading were short to moderate length (up to 65 m) with short spans and a very stiff deck. The strut effect due to the presence of the bridge and the flow around mechanism of lateral spreading displacements suggests that the displacements will be smaller, and this has been demonstrated by a case study from Christchurch (Cubrinovski et al. 2014a, 2014b) where displacements were shown to be as little as 40% of the free-field values. However there is still significant uncertainty as to what level of displacement should be used in design given the relatively small number of well documented case histories.

The deformation mechanism for the short span Christchurch bridges was characterized by the pinning of the deck at the top of the abutment due to the stiff deck and subsequent back rotation of the abutments (Wotherspoon et al. 2011, Cubrinovski et al. 2014a, 2014b). This resulted in abutment slumping and placed high demands on the abutment piles resulting in plastic hinge development. Bridges on approach abutments and bridges constructed at the natural river bank levels both exhibited this behaviour. Psuedo-static analysis by Cubrinovski et al. (2014b) was able to capture this response, as well as highlighting the sensitivity of the model to the controlling factors of crust properties and lateral spreading displacement estimates. These lessons have been used in the development of guidelines for bridge design for the New Zealand Transport Agency (NZTA), with a key emphasis on the uncertainties involved (Murashev et al. 2014). Even based on this recent evidence the level of uncertainty in the input to design still can result in a wide range of assessment and design outputs.

A key factor in the assessment of lateral spreading effects is the move from component to system level response. By developing a detailed understanding of the response of abutments to lateral spreading we can capture this mechanism by itself, but there also needs to be an understanding of how loads will propagate into other parts of the structure. Cubrinovksi et al. (2014b) were able to demonstrate this for a case study, showing the importance of modelling the global response on capturing both abutment and pier displacement and damage characteristics.

Using centrifuge models Stergiopoulou et al. (2016a) were able to capture the abutment displacement and pile damage mechanism of a case study bridge in Christchurch. Extending beyond this Stergiopoulou et al. (2016b) performed centrifuge testing of the retrofit solution implemented at this bridge, where large robust replacement piles. In terms of abutment performance, no damage and minimal displacement or back-rotation developed. However, this resulted in a significant increase in the loads transferred to the bridge superstructure (the bridge deck model buckled in the retrofit case, not noted in the above papers). Testing is ongoing to assess the increase in axial load due to this retrofit approach, and performance of landspans. This again indicates the importance of taking a global view to response, and how strengthening and reduction in the back-rotation response can change the distribution of actions throughout the bridge beyond just the abutment.

3.0 ACKNOWLEDGEMENTS

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CHALLENGES IN PREDICTING POST-LIQUEFACTION RECONSOLIDATION STRAINS AND SETTLEMENTS: A MULTITUDE OF IMPLICATIONS

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1.0 LIQUEFACTION EFFECTS CHALLENGE: 3 – DEVELOPMENT AND EFFECTS OF LIQUEFATION-INUCED SETTLEMENT ON STRUCTURES AND LIFELINES

One of the persistently pursued and yet unsolved challenges in the evaluation of liquefaction effects is the quantification of post-liquefaction reconsolidation settlements. These effects have been extensively studied during the last 30 years via numerous methods (experimental, analytical, semi-empirical, and numerical). Yet, the simulation and thus prediction and preparation for such effects is hindered by the lack of a consistent methodology able to capture the effects of post-liquefaction reconsolidation in soil and structure systems. Semi-empirical approaches still used in research and practice cannot replicate phenomena observed in the field (e.g. sand boils and soil cracking). Furthermore, empirically developed charts correlating post-liquefaction settlement to the relative density of sand or the surface manifestation of liquefaction to the relative thickness of non-liquefiable crust layer to the liquefiable one, have been very appealing to practice. However, recent case histories and on-going experimental research has proven them to be limited in their application since they are based on a small number of case histories, do not cover a large number of earthquake events, do not account for the crust type or integrity and do not account or for the existence of a building (footing type, contact pressure). The latter effect has been extensively shown to be affecting the post-liquefaction behavior of soil deposits and in the topic of a significant amount of undertaken research. Accurate prediction of post-liquefaction volumetric strains can also be important for capturing the post-shaking pore pressure redistribution and its effect on slope deformations. Last but not least, the emerging recognition of the importance of spatial variability or interlayering of soil deposits, the multitude of systems affected (e.g. shallow foundations, pipelines, slopes) as well the challenge of residual strength (as set forward within this effort) emphasize the importance of the chosen topic.

2.1 CURRENT STATE-OF-THE-ART

Currently available approaches for predicting the magnitude of post-liquefaction reconsolidation settlements can be categorized as: (1) numerical analyses in the form of finite element or finite difference techniques (e.g. Martin et al. 1975, Seed et al. 1976, Booker et al. 1976, Finn et al. 1977), (2) semi-empirical models developed based on laboratory, field test and case history data (e.g. Lee and Albaisa 1974, Tokimatsu and Seed 1987, Ishihara and Yoshimine 1992, Shamoto et al. 1998, Zhang et al. 2002, Wu and Seed 2004, Tsukamoto et al. 2004), and (3) analytical methods (e.g. Scott 1976, Kamai and Boulanger 2010, Adamidis and Madabhushi 2016). More recently a probabilistic approach was developed by Cetin et al. 2009 who developed a maximum likelihood framework for the probabilistic assessment of cyclically induced reconsolidation settlements of saturated cohesionless soils.

The phenomenon of post-liquefaction reconsolidation has been studied closely either in the framework of lab element tests whereby researchers study the amount of reconsolidation strains experienced by a soil specimen or in the framework of physical model tests and the problem of void redistribution (Whitman 1985). The two are mechanistically similar in the sense that in both cases the developed excess pore pressures need to dissipate and thus lead to migrating water and the associated reconsolidation of the soil. Physical model tests involving liquefiable sands with lower-permeability interlayers have demonstrated how various factors can influence the degree to which void redistribution can affect shear strength losses and slope deformations (e.g. Kulasingam et al. 2004, Malvick et al. 2008). The potential for void redistribution to cause strain localizations and associated deformations in liquefied soil depends on the soil properties (initial relative density, cyclic resistance ratio, permeability), slope geometry (layer thicknesses, slope angle, continuity of interfaces), and ground motion characteristics (shaking intensity, shaking duration, shaking history).

Similarly, numerical simulations using advanced constitutive models for liquefiable soils have been shown to reasonably reproduce the patterns of void redistribution (Ziotopoulou et al. 2012, Boulanger et al. 2013), but they cannot reproduce the delayed timing of the post-shaking scope deformations that were observed in the experiments that provided a validation basis.

2.3 PRIMARY MECHANISMS INVOLVED IN THE PHENOMENON

The Element Level: Experimental Data

Saturated sandy soils have been observed to liquefy during earthquakes and in laboratory experiments. Following liquefaction, the soil grains settle out, and the material solidifies from the base up. To understand the post-liquefaction behavior of liquefied ground, it is important to get a better understanding of a more suitable characterization of the variation of excess pore pressure after liquefaction. The history of pore pressure in a reconsolidating liquefied sand has also been obtained in centrifuge experiments. As the excess pore pressures dissipate, the soil is sedimenting and reconsolidating. In the system level of a soil deposit, this is experienced as a reconsolidation settlement. If the seepage of the water is restricted due to permeability contrasts due to interlayering of soils of lower permeability, then this phenomenon is described as void redistribution leading to localized strength loss and deformation.

Ishihara and Yoshimine (1992) observed that the volumetric strains that occur during post-liquefaction reconsolidation of sand samples are directly related to the maximum shear strains that developed during undrained cyclic loading and to the initial relative density (D_R) of the sand. They compiled laboratory results and developed the relationships shown in Figure 1. The experimental data show post-liquefaction reconsolidation volumetric strains ranging from 1 to 4% for most relative densities. Their results also showed that a significant portion of the reconsolidation strains occurred early in the reconsolidation phase when the effective stresses were still small.

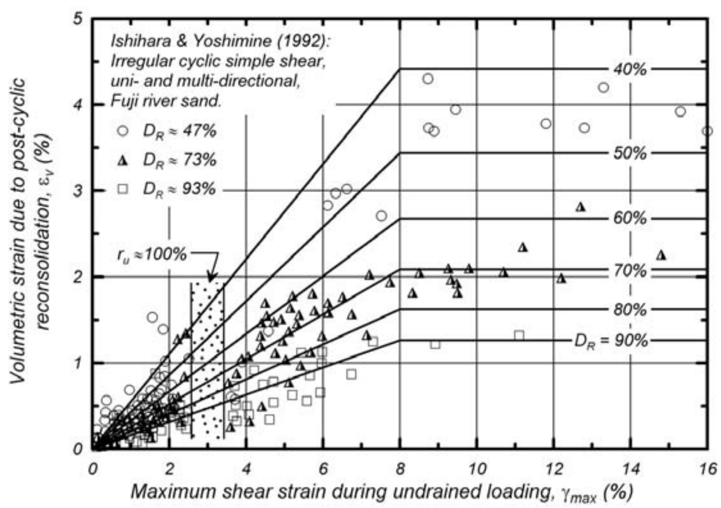

Figure 1. Relationship between post-liquefaction volumetric strain and the maximum shear strain induced during undrained cyclic loading of clean sand (after Ishihara and Yoshimine 1992; redrawn in Idriss and Boulanger 2008).

Other laboratory studies of post-liquefaction reconsolidation strains (e.g., Tokimatsu and Seed 1987, Sento et al. 2004) produced results similar to those of Ishihara and Yoshimine (1992). Sento et al. (2004), however, showed that post-liquefaction reconsolidation strains are more uniquely related to the accumulated shear strain (summing the absolute magnitude of shear strain increments) that develops during the undrained cyclic shearing process than to the maximum shear strain that develops during undrained shearing. When seeking to numerically simulate such effects, we need to recognize that in practice, experimental results for post-liquefaction reconsolidation strains are unlikely to be available for an individual site but any numerical model should at least give results that are consistent with the range of experimental data in the literature.

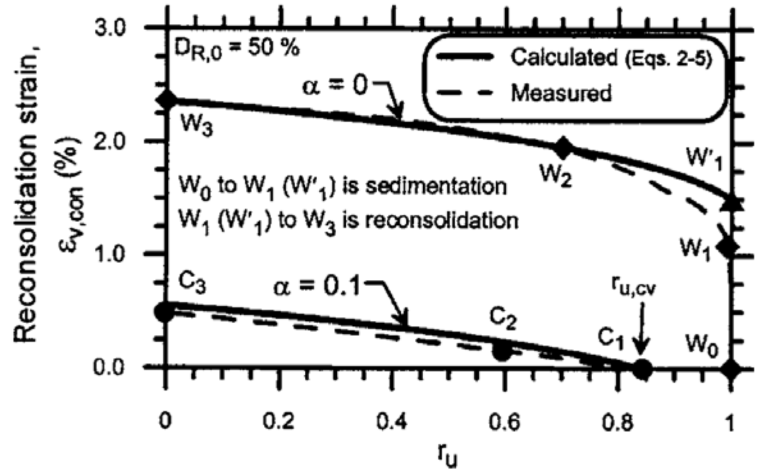

Figure 2. Reconsolidation volumetric strain accumulation for excess pore pressure dissipation from r_u=$r_{u,cv}$ to r_u=0 for level ground (α=0) and sloping ground conditions (α=0.1) whereby α is the static shear stress ratio (α=τ_{st}/σ_{vo}) (triaxial test data from Dismuke 2003)

The System Level: Implications for level ground deposits

Moving to the system level effects of this phenomenon, settlement caused by the one-dimensional reconsolidation of liquefied soils, along with any settlement caused by lateral spreading, is an important possible consequence of liquefaction. Settlement caused by post-liquefaction reconsolidation results from volumetric strains that develop throughout the underlying soil profile. Such volumetric strains are not easy to numerically model. The conventional numerical separation of strains into elastic and plastic components cannot capture post-liquefaction volumetric strains that are due to sedimentation under essentially zero effective stress. For this reason, it is common for numerical models to underestimate liquefaction-induced one-dimensional settlements.

The System Level: Implications for sloping ground deposits

The ability to numerically model post-liquefaction reconsolidation strains can also be important for predicting deformation and stability of slopes. Diffusion of the earthquake-induced excess pore water pressures leads to outward seepage from zones that are reconsolidating. If this transient seepage is impeded by an overlying lower permeability soil layer, the accumulation of water near such an interface can lead to void redistribution and subsequent localized loosening, strength loss and possibly water film formation (e.g., Kokusho 1999, Kulasingam et al. 2004). These mechanisms can all locally diminish the shear resistance in a loosening zone leading to greater deformations and possibly instability.

The Constitutive Level

Constitutive models are most often based on the additive decomposition of strains (or strain rates) into elastic and plastic components. For example, volumetric strain increments $d\varepsilon_v$ are composed of an elastic volumetric strain increment $d\varepsilon_{vol}^{el}$ and a plastic volumetric strain increment $d\varepsilon_{vol}^{pl}$:

$$d\varepsilon_{vol} = d\varepsilon_{vol}^{el} + d\varepsilon_{vol}^{pl} \qquad (1)$$

Elastic strains are computed from Hooke's law and the stress increment, and plastic strains are computed from the yield criterion and flow rule. For PM4Sand Version 3 for example (Boulanger and Ziotopoulou 2015) and models within the family of the Dafalias and Manzari (2004) constitutive model, the elastic volumetric strain increment $d\varepsilon_{vol}^{el}$ is equal to the mean effective stress increment *dp'* divided by the bulk modulus K and the plastic volumetric strain increment $d\varepsilon_{vol}^{pl}$ is equal to the dilatancy *D* multiplied by the plastic shear strain increment $d\gamma_{vol}^{pl}$:

$$d\varepsilon_{vol} = d\varepsilon_{vol}^{el} + d\varepsilon_{vol}^{pl} = \frac{dp'}{K} + D \cdot \left| d\gamma^{pl} \right| \tag{2}$$

One-dimensional post-liquefaction reconsolidation strains are the result of integrating volumetric strain increments as the vertical effective stress σ_v' recovers from zero (i.e., the liquefied state) to its initial value prior to cyclic loading (σ_{vo}') (see also Figure 2)

$$\varepsilon_{vol-1D} = \int_0^{\sigma_{vo}'} \left(\frac{d\varepsilon_{vol}}{d\sigma_v'} \right) d\sigma_v' \tag{3}$$

The nature of this integration depends on the details of the constitutive model.

For stress-ratio controlled models, like PM4Sand, a narrow open cone-type yield surface is used with its apex at the origin and obeying rotational hardening. This means that only changes of the stress ratio (i.e. the ratio of deviatoric stress q over the mean effective stress p') can cause plastic shear and volumetric strains, while constant stress-ratio loading induces only elastic strains. The stress-ratio based dilatancy, critical state and bounding surfaces of the Dafalias and Manzari (2004) model, upon which PM4Sand was developed, are illustrated in Figure 3. Other examples of stress-ratio based models include the ones developed by Papadimitriou et al. (2001), Andrianopoulos et al. (2010) and Yang et al. (2003).

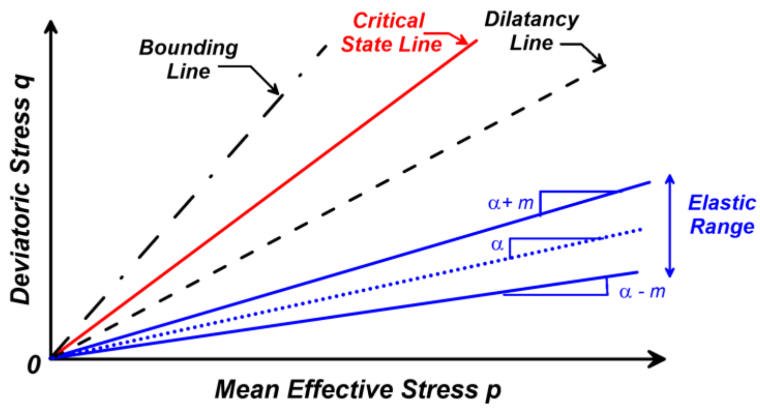

Figure 3. Schematic of the yield, critical, dilatancy, and bounding lines in q-p space (after Dafalias and Manzari, 2004)

In simulations of one-dimensional reconsolidation of a liquefied sand, the stress-ratio will remain constant for a stress-ratio based constitutive model with a constant Poisson's ratio ν. Reconsolidation strains, as computed using Equation 3, will therefore be integrated along the K_o consolidation line. The bulk modulus K will vary with confinement and thus, the reconsolidation response will be non-linear elastic. A simple analytical calculation can also show this discrepancy (e.g. Ziotopoulou and Boulanger 2013).

Other constitutive models include plastic strains during mean stress increases at a constant stress ratio by the introduction of additional plastic loading mechanisms, such as a cap type of loading surface or a more generalized yield surface (e.g., Taiebat and Dafalias 2008; Wang et al. 1990). These features may, or may not, improve predictions of post-liquefaction volumetric strains because they are not necessarily formulated to account for sedimentation strains.

More details on this topic as well as the topic of delayed flow failures can be found in Ziotopoulou and Boulanger (2013), Boulanger et al. 2013 amongst others.

2.4 KEY CHALLENGES TO DEVELOPING BETTER EVALUATION PROCEDURES

There are a number of technical challenges that need to be addressed for improving the ability to numerically simulate the process of post-liquefaction reconsolidation strain development as well as the process of void redistribution and its effects on residual shear strengths as well as deformations:

1) **Major challenge for the numerical simulation and thus prediction of reconsolidation**

volumetric strains and settlements: volumetric strains and settlements that develop during post-liquefaction reconsolidation of sand are difficult to numerically model using the conventional constitutive separation of strains into elastic and plastic components since a large portion of the post-liquefaction reconsolidation strains are due to sedimentation effects which are not easily incorporated into either the elastic or plastic components of behavior. Single element simulations using various constitutive models show that they generally predict post-liquefaction reconsolidation strains that are an order of magnitude smaller than observed in various experimental studies (Ziotopoulou and Boulanger 2013). Thus, these are expected to cause an underestimation of the predicted settlements experienced by a liquefied soil deposit and an underestimation of the magnitude of void redistribution. These lead to difficulties in simulating delayed slope deformations. So far, advanced constitutive models deal with this issue by implementing phenomenological adjustments, that albeit relatively reasonable and successful, still do not capture the fundamental aspects of the behaviour. Thus, constitutive models are needed that can better simulate post-liquefaction reconsolidation strains because these volumetric strains directly affect the volume of water given off by consolidating zones. Constitutive models that offer a better framework towards this direction (e.g. the so-called models with a "cap") need to be examined and validated against documented case histories and tests but it still needs to be recognized that their fundamental background still does not capture the effects of sedimentation.

Even with better constitutive capabilities, challenges that remain towards the system level simulation and prediction of phenomena related to the post-liquefaction volumetric strains are:

2) The length scale for any eventual localization needs to be consistently accounted for and better understood for conditions where geologic contacts are less distinct and/or have gradually varying soil properties (e.g., permeability in a fining-upward sequence).
3) Simulation of localization processes is challenging when using any continuum model. Whether these are void redistribution or cracking of overlying crusts with associated graben formation during the sliding process, the challenges are still the same. This is followed by the important need to evaluate the extent to which any numerical modeling procedure can differentiate between cases (e.g., physical model tests) where void redistribution has and has not led to significant concentrations of shear strains and contributed to the associated deformations.
4) Even with capturing the fundamental mechanistic aspects of the phenomenon, the challenge of improving our ability to characterize geologic heterogeneities, geology contacts, in-situ soil properties, and ground motion characteristics remains. Geologic details can be expected to have a major influence on the thickness and/or continuity of any loosening zones and on the formation of ground cracks or soil boils that may reduce the progression of loosening in certain zones.

2.5 PATHS FORWARD

In moving forward, the obvious solution lies in tackling the challenges set forward in Section 2.4 of this abstract. Interestingly enough, the path forward is similar to the paths recognized for other challenges (e.g. for residual strength estimation as delineated by Professor Bruce Kutter in his abstract). Advanced numerical simulations are still challenged in their capacity to provide accurate predictions of observed settlements, lateral displacements, as well as the complex migration of pore voids in stratified liquefying soils, but intense, methodical and coordinated effort could solve this problem.

Work is needed on:

1) More realistic constitutive models for soils: if not fundamentally accurate (e.g. able to capture sedimentation processes via particle to particle simulations), then constitutive models should be at least carefully validated against the numerous available testing data and case history data;
2) Solution schemes that can predict strain softening, localization of shear strains, and large deformations (also mentioned by Bruce Kutter);
3) Systematic evaluation and validation against the effects from structures and different ground motion characteristics;
4) Improved methods for capturing the spatial variability and interlayering properties of soils;

5) Clarification of terminology amongst the various terms involved in liquefaction evaluations (sedimentation, solidification, post-liquefaction, post-triggering, post-shaking).

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APPENDIX B: BIBLIOGRAPHY

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Liquefaction Workshop Bibliography (References List)

Prior to the workshop, participants were asked to provide relevant references that would inform and facilitate discussion during the workshop. The references provided by workshop participants are listed here. This list is not a comprehensive record of all references on the workshop topics, nor is it meant as a recommendation on the best available references.

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APPENDIX C: WORKSHOP PARTICIPANTS

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Table C-1. Workshop Participants

#	First Name	Last Name	Affiliation	Country
1	Adda	Athanasopoulos-Zekkos	Univ. of Michigan	US
2	Christine	Beyzaei	Univ. of California, Berkeley	US
3	Ross	Boulanger	Univ. of California, Davis	US
4	Brendon	Bradley	Univ. of Canterbury	NZ
5	Scott	Brandenberg	Univ. of California, Los Angeles	US
6	Jonathan	Bray	Univ. of California, Berkeley	US
7	Gabriele	Chiaro	Univ. of Canterbury	NZ
8	Brady	Cox	Univ. of Texas at Austin	US
9	Misko	Cubrinovski	Univ. of Canterbury	NZ
10	Shideh	Dashti	Univ. of Colorado, Boulder	US
11	Jason	DeJong	Univ. of California, Davis	US
12	Ahmed	Elgamal	Univ. of California, San Diego	US
13	Kevin	Franke	Brigham Young University	US
14	David	Frost	Georgia Tech	US
15	Laurie	Gaskins Baise	Tufts University	US
16	Russell	Green	Virginia Tech	US
17	Les	Harder	HDR Engineering	US
18	Youssef	Hashash	Univ. of Illinois	US
19	Jennifer	Haskell	Univ. of Canterbury	NZ
20	Susumu	Iai	Kyoto University	Japan
21	I.M.	Idriss	Consulting Geotechnical Engineer	US
22	Mike	Jacka	Tonkin + Taylor, Ltd.	NZ
23	Takashi	Kiyota	Univ. of Tokyo	Japan
24	Takaji	Kokusho	Chuo University	Japan
25	Steve	Kramer	Univ. of Washington	US
26	Bruce	Kutter	Univ. of California, Davis	US
27	Dong Youp	Kwak	Univ. of California, Los Angeles	US
28	Brett	Maurer	Virginia Tech	US
29	Alesandra	Morales-Vélez	Univ. of Puerto Rico at Mayaguez	US
30	Robb	Moss	Cal Poly, San Luis Obispo	US
31	Ramin	Motamed	Univ. of Nevada, Reno	US
32	Diane	Moug	Univ. of California, Davis	US
33	Thomas	O'Rourke	Cornell University	US
34	Mitsu	Okamura	Ehime University	Japan
35	Scott	Olson	Univ. of Illinois	US
36	Rolando	Orense	Univ. of Auckland	NZ
37	Ellen	Rathje	Univ. of Texas at Austin	US
38	Peter	Robertson	Gregg Drilling & Testing	US
39	Kyle	Rollins	Brigham Young University	US
40	Inthuorn	Sasanakul	Univ. of South Carolina	US
41	Tom	Shantz	Caltrans	US
42	Nick	Sitar	Univ. of California, Berkeley	US
43	Jonathan	Stewart	Univ. of California, Los Angeles	US
44	Ken	Stokoe	Univ. of Texas at Austin	US
45	Mark	Stringer	Univ. of Canterbury	NZ
46	Armin	Stuedlein	Oregon State University	US

47	Akihiro	Takahashi	Tokyo Institute of Technology	Japan
48	Shuji	Tamura	Tokyo Institute of Technology	Japan
49	Kohji	Tokimatsu	Tokyo Institute of Technology	Japan
50	Thaleia	Travasarou	Fugro Consultants	US
51	Yoshimichi	Tsukamoto	Tokyo University of Science	Japan
52	Sjoerd	van Ballegooy	Tonkin + Taylor, Ltd.	NZ
53	Liam	Wotherspoon	Univ. of Auckland	NZ
54	Katerina	Ziotopoulou	Univ. of California, Davis	US

Observer: Rick Fragaszy, U.S. National Science Foundation (NSF)

APPENDIX D: WORKSHOP AGENDA

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U.S. – New Zealand – Japan International Workshop

Liquefaction-Induced Ground Movements Effects

The Faculty Club, UC Berkeley, California

2-4 November 2016

WORKSHOP AGENDA

Wednesday, 2 November 2016

6:00 pm	**Welcome Reception** (Seaborg Room & Patio)
7:00 pm	Dinner (Seaborg Room)

Thursday, 3 November 2016

8:00 am	Breakfast (Heyns Room & Patio)
9:00 am	**Welcome:** Introduction & Workshop Objectives – J. Bray (Heyns Room)
9:15 am	**Session I: Liquefaction-induced flow slides that are governed by the residual shear strength of liquefied soil**; Chair: S. Kramer, Recorder: R. Boulanger (Heyns Room)
	Invited presentations (15-minute talks): S. Kramer, R. Orense, Y. Tsukamoto, & R. Boulanger
10:15 am	Additional presentations (5-minute talks): S. Olson, G. Chiaro, A. Athanasopoulos-Zekkos, A. Takahashi, B. Kutter, & L. Harder
10:45 am	Break
11:15 am	Discussions of identified key challenges/issues
12:45 pm	Lunch (Heyns Room & Patio)
2:00 pm	**Session II: Liquefaction-induced lateral spreading and its effects on structures and lifelines**; Chair: E. Rathje, Recorder: M. Cubrinovski (Heyns Room)
	Invited presentations (15-minute talks): E. Rathje, T. Kokusho, M. Cubrinovski, & T. O'Rourke
3:00 pm	Additional presentations (5-minute talks): K. Stokoe, M. Okamura, K. Franke, L. Wotherspoon, T. Travasarou, & N. Sitar
3:30 pm	Break
4:00 pm	Discussions of identified key challenges/issues
5:30 pm	Summarize first day activities
5:45 pm	Break
6:00 pm	Dinner (Seaborg Room)

U.S. – New Zealand – Japan International Workshop

Liquefaction-Induced Ground Movements Effects

The Faculty Club, UC Berkeley, California

2-4 November 2016

WORKSHOP AGENDA

Friday, 4 November 2016

8:00 am	Breakfast (Heyns Room & Patio)
9:00 am	**Session III: Liquefaction-induced ground settlement and its effects on structures and lifelines**; Chair: R. Green, Recorder: J. Bray (Heyns Room)
	Invited presentations (15-minute talks): R. Green, S. van Ballegooy, K. Tokimatsu, & J. Bray
10:00 am	Additional presentations (5-minute talks): L. Gaskins-Baise, M. Jacka, S. Dashti, S. Tamura, B. Cox, & K. Rollins
10:30 am	Break
11:00 am	Discussions of identified key challenges/issues
12:30 pm	Lunch (Heyns Room & Patio)
1:45 pm	**Session IV: Paths forward toward assessing the effects of liquefaction on structures and lifelines**; Chair: T. O'Rourke, Recorder: K. Tokimatsu (Heyns Room)
	Invited presentations (15-minute talks): P. Robertson, S. Iai, B. Bradley, & I.M. Idriss
2:45 pm	Additional presentations (5-minute talks): J. DeJong, K. Ziotopoulou, T. Kiyota, J. Haskell, D. Y. Kwak, & D. Frost
3:15 pm	Break
3:45 pm	Discussions of identified key challenges/issues
5:15 pm	Summarize last day activities
5:30 pm	Adjourn

Saturday, 5 November 2016

9:00 am	**Workshop Report Preparation:** Organizers & Selected Participants (542 Davis Hall)

WORKSHOP PROMPTS

1. What is the current state-of-the-art for evaluating this problem today?
2. What are the key underlying geologic processes that affect it?
3. What are the primary mechanisms involved in the phenomenon?
4. What are the key challenges to developing better evaluation procedures?
5. What is the best path forward for advancing understanding and procedures to address it?

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PEER 2017/02 *U.S.–New Zealand–Japan Workshop: Liquefaction-Induced Ground Movements Effects, University of California, Berkeley, California, 2-4 November 2016*, Jonathan D. Bray, Ross W. Boulanger, Misko Cubrinovski, Kohji Tokimatsu, Steven L. Kramer, Thomas O'Rourke, Ellen Rathje, Russell A. Green, Peter K. Robinson, and Christine Z. Beyzaei, March 2017.

PEER 2017/01 *2016 PEER Annual Report*, Khalid Mosalam, Amarnath Kasalanati, and Grace Kang, March 2017.

PEER 2016/11 *Seismic Design Guidelines for Tall Buildings*, Members of the Committee for the Tall Buildings Initiative, December 2016.

PEER 2016/10 *Performance-Based Robust Nonlinear Seismic Analysis with Application to Reinforced Concrete Bridge Systems,* Xiao Ling and Khalid M. Mosalam, December 2016.

PEER 2016/09 *Resilience of Critical Structures, Infrastructure, and Communities, Resilience of Critical Structures, Infrastructure, and Communities*, Gian Paolo Cimellaro, Ali Zamani-Noori, Omar Kamouh, Vesna Terzic, and Stephen A. Mahin, December 2016.

PEER 2016/08 *Processing and Development of Iran Earthquake Ground-Motion Database*, Tadahiro Kishida, Sahar Derakhshan, Sifat Muin, Yousef Bozorgnia, Sean K. Ahdi, Jonathan P. Stewart, Robert B. Darragh, Walter J. Silva, and Esmael Farzanegan, December 2016.

PEER 2016/07 *Hybrid Simulation Theory for a Classical Nonlinear Dynamical System*, Paul L. Drazin and Sanjay Govindjee, September 2016.

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PEER 2015/09 *Structural Behavior of Column-Bent Cap Beam-Box Girder Systems in Reinforced Concrete Bridges Subjected to Gravity and Seismic Loads. Part I: Pre-Test Analysis and Quasi-Static Experiments*. Mohamed A. Moustafa and Khalid M. Mosalam. September 2015.

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