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Seismic ground motions for evaluation of liquefaction triggering

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ABSTRACT: Simplified liquefaction triggering evaluation procedures typically use the peak ground acceleration (PGA) and magnitude (M) as parameters representing the seismic demand at a site. The values of PGA and M at a site can vary depending upon the return period in a probabilistic approach or associated expected level of performance. Evaluation of liquefaction triggering may significantly change depending upon the level of the ground motions selected. This paper will present and discuss different current approaches and guidelines dealing with the level of ground motions, and liquefaction triggering evaluation. These guidelines include ASCE 7-16, the 2016 National Academies of Sciences, Engineering, and Medicine report, Naval Facilities Engineering Command (NAVFAC), California Department of Transportation (Caltrans), Port of Los Angeles (POLA), California High Speed Train, California Geological Survey Note 48, and Los Angeles City. International guidelines such as Eurocode, Japanese codes and New Zealand are also incorporated in the discussion. Additionally, new ground motions related parameters for the evaluation of liquefaction such as duration, cumulative absolute velocity (CAV) and others will be discussed. Discussion of various levels of ground motions and parameters will be oriented toward performance-based design approaches in the evaluation of liquefaction. Finally, some clarifying conclusions will be proposed.

1 INTRODUCTION

In the evaluation of liquefaction triggering and associated consequences, the seismic hazard level considered, and how we represent the hazard (conventionally peak ground acceleration and magnitude) are necessary for the analyses. The widely adopted practice has been to use the empirical stress-based approach ("simplified approach"), using peak ground acceleration (PGA) and magnitude (M_W) to represent the seismic demand at a given site, corresponding to a particular seismic hazard level (probabilistic approach) or under a given scenario earthquake. The ultimate goal is the evaluation of liquefaction triggering, and then the consequences should liquefaction occur (i.e., settlements, lateral spreading, etc.).

However, even with this generally uniform approach to evaluating liquefaction, current codes and other agency approaches or guidelines can vary, particularly with the seismic hazard level desired for evaluation, and how those parameters are obtained. The values of PGA and M_W at a given site can vary depending upon the return period in a probabilistic approach or the associated expected level of performance. Evaluation of liquefaction triggering may significantly change depending on the level of the ground motions selected.

A recent report by the National Academies of Science, Engineering and Mathematics (NASEM) entitled Stat of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences (NASEM 2016) states that "analysis of liquefaction and its consequences remains one of the more active areas of research and development in geotechnical engineering." Yet despite our advances in understanding this topic based on recent research and observations made in post-earthquake reconnaissance, quite often in practice, the overall assessments of liquefaction and its consequences have not significantly changed in the past decades.

A discussion of the current approaches and guidelines (including ASCE 7, NAVFAC, Caltrans, ASCE 61, among others) dealing with the level of ground motions, and liquefaction triggering evaluation is presented herein. Some international codes are also incorporated for comparison.

Furthermore, although limitations with the use of PGA and M_W to represent the seismic demand have long been known, due to the relative ease in which PGA and M_W can be estimated for a site, those limitations have often been overlooked. In recent years, however, several new studies have begun moving in a different direction, looking toward different intensity measures (IMs) to represent the seismic demand, such as Arias Intensity and Cumulative Average Velocity. A discussion of research involving these IMs and their conclusions is included, along with implications for evaluation in liquefaction triggering in practice.

2 CURRENT APPROACHES AND GUIDELINES

The evaluation of liquefaction triggering has most commonly been performed in practice using the empirical stress-based approach, also called the "simplified approach" which was originally developed in the 1970's. Although there have been some updates to the procedure over the past 40 years by various researchers, the overall approach has generally remained the same. A Cyclic Resistance Ratio (CRR) is developed based on the soil profile, and an earthquake-induced Cyclic Stress Ratio (CSR) is developed to represent seismic demand based on PGA (maximum amplitude of earthquake acceleration) and magnitude (a proxy to duration and energy). The biggest question surrounding the seismic demand then is what combination of PGA and M_W to use at what hazard level. Depending on the type of project and agency jurisdiction, how the PGA and magnitude are selected can vary. Although the guidelines and codes generally specify the design ground motions that should be used in the evaluation, some defer the details to the geotechnical engineer performing analyses, while others are more prescriptive in their requirements.

2.1 *Guidelines in California*

2.1.1 *California Building Code*

The current 2016 California Building Code adopts the recommendations in ASCE 7-10 for seismic ground motions and seismic hazard levels for design. The most severe earthquake considered in this standard is the Maximum Considered Earthquake (MCE). For geotechnical-related issues, the Maximum Considered Earthquake Geometric Mean (MCE_G) PGA is used. ASCE 7-10 specifically defines the MCE_G PGA adjusted for site effects (PGA_M) to be used "for evaluation of liquefaction, lateral spreading, seismic settlements, and other soil related issues." The MCE may be associated with probabilistic return period of 2475 years. The more recently released ASCE 7-16 (which will be adopted into the 2018 IBC, and the 2019 California Building Code) considers the same seismic ground motion levels as ASCE 7-10.

Regarding liquefaction hazard evaluation, ASCE 7-10 states that "the potential for liquefaction and soil strength loss evaluated for site peak ground acceleration, earthquake magnitude, and source characteristics consistent with the MCE_G PGA." While the PGA is clearly defined, the appropriate M_W for use in the liquefaction triggering evaluation may be subject to interpretation. Commonly, deaggregation of the seismic hazard is performed, and either a mean or modal magnitude is selected for design. Alternatively, some practicing engineers select the characteristic magnitude from the controlling fault.

Additionally, for public schools, hospitals, and other essential services buildings, the requirements of California Geological Survey Note 48 must also be met. In general, the recommendations are consistent with ASCE 7-10 (evaluation for MCE_G, using PGA_M). CGS Note 48 also requires that the modal magnitude obtained from deaggregation of the MCE seismic hazard is used in the analyses.

2.1.2 *City of Los Angeles/County of Los Angeles*

In 2014, the City of Los Angeles added supplemental requirements for evaluation of liquefaction triggering by adding a second screening level – in addition to the evaluation of

liquefaction triggering for PGA_M, liquefaction triggering for 2/3 of PGA_M is also performed. This approach is similar to the use of the "Design Earthquake" in ASCE 7-10 for structural engineers that is 2/3 of the MCE_R ground motions. The magnitude (either mean or modal) is obtained from deaggregation of the seismic hazard, corresponding to the 2475-year return period for the MCE_G level, and the 475-year return period for the evaluation at 2/3 MCE_G. The concept proposed by the City of Los Angeles is interesting in that it attempts to replicate what the structural engineers evaluate, a "Design" level and a "No Collapse" level.

The County of Los Angeles also developed specific requirements for the evaluation of liquefaction and lateral spreading hazards (County of Los Angeles 2013, 2014). In their 2013 *Manual for Preparation of Geotechnical Reports*, the County states that liquefaction should be evaluated at either a probabilistic seismic hazard level corresponding to 10% probability of exceedance in 50 years (475-year return period), or a deterministic seismic hazard analysis using magnitude and PGA corresponding to the maximum earthquake event for that fault. However, in 2014, the requirement was revised to be more consistent with ASCE 7-10, and a 2% probability of exceedance in 50 years is now the minimum hazard level.

2.1.3 *Naval Facilities Engineering Command (NAVFAC)*

NAVFAC generally uses two earthquake levels in the evaluation of liquefaction, although the seismic hazard levels can vary depending on type of structure (such as ordinary buildings, piers and wharves, etc.). For typical buildings and for piers and wharves, the Level 1 earthquake has a 50% probability of exceedance in 50 years (72-year return period), and the Level 2 earthquake has a 10% probability of exceedance in 50 years (475-year return period).

For "essential" buildings, the Level 1 earthquake remains the same, but the Level 2 earthquake increases to a 10% in 100 years (975-year return period).

2.1.4 *Piers and wharves/Port of Los Angeles (POLA)*

Piers and wharves follow the seismic design recommendations of ASCE 61-14. Depending on the design classification (High, Moderate, or Low), there are up to three seismic hazard levels that are considered, the Operating Level Earthquake (OLE), the Contingency Level Earthquake (CLE) and Design Earthquake (DE). For projects with a "High" design classification, the OLE and CLE are ground motions correspond to a probability of exceedance of 50% in 50 years (72-year return period), and 10% in 50 years (475-year return period), and the DE is the Design Earthquake in accordance with ASCE 7-05. For projects with a "Moderate" design classification, only two levels are considered, CLE (20% in 50 years, a 224-year return period), and the DE. For projects with a "Low" design classification, only the DE seismic hazard level is evaluated.

2.1.5 *Caltrans*

The California Department of Transportation (Caltrans), develops ground motions through a comparison of probabilistic seismic hazard analyses and deterministic seismic hazard analyses. The controlling earthquake is defined by a geometric mean acceleration response spectrum determined based on the larger of a deterministic seismic hazard analysis or a probabilistic seismic hazard analysis (5% probability of exceedance in 50 years/975-year return period). For liquefaction hazard analyses, the design PGA is used along with the appropriate magnitude (either the maximum deterministic magnitude or the mean or modal M_W if PGA is probabilistic).

2.1.6 *California High Speed Rail*

The California High Speed Rail (CAHSR 2012) considers two levels of earthquake ground motions in the evaluation of seismic hazards, including liquefaction. The Maximum Considered Earthquake (MCE) and the Operating Basis Earthquake (OBE). The MCE is the greater of a probabilistic spectrum based upon a 10% probability of exceedance in 100 years (950-year return period) and a deterministic spectrum (corresponding to the maximum rupture, M_{max}) of any fault near the project site. The OBE ground motions correspond to a probabilistic spectrum based on an 86% probability of exceedance in 100 years (50-year return period).

2.2 *International guidelines*

Several codes or standards from international agencies and organizations are presented for a comparison to the practice in the United States. Recommendations from New Zealand, Japan, and Europe are included in the following sections.

2.2.1 *New Zealand*

Seismic ground motions for evaluation of liquefaction are considered for two seismic hazard levels, the ultimate limit state (ULS) and the serviceability limit state (SLS) (NZGS/MBIE, 2016). For typical buildings (normal use and occupancy), the ULS corresponds to an earthquake with a return period of 500 years, and the SLS corresponds to an earthquake with a return period of 25 years. However, depending on the structure, other return periods may also be considered for the ULS and SLS, in accordance with the Standards New Zealand (1992). Seismic demand parameters include a_{max} (PGA) and M_w. The seismic hazard factors (PGA coefficients corresponding to a 1000-year return period) are then modified by a return period factor and a site response factor (for rock, shallow soil, or deep soil) to develop the appropriate a_{max} for use in the liquefaction hazard evaluation. The effective earthquake magnitude is selected from the maps for the appropriate return period.

Note that based on recent earthquakes in New Zealand (the Canterbury Earthquake Sequence), the mapped values presented in the NZTA Bridge Manual (2014) are considered out of date for the Canterbury Earthquake Region. Although the seismic hazard model for New Zealand was revised in 2010, those updates have not yet been incorporated into any standards or codes. In the interim, if liquefaction is evaluated within the Canterbury Earthquake Region, a specified procedure must be followed instead of the one previously described. For the SLS, two scenarios are analyzed (a_{max}=0.13g, M_w=7.5 and a_{max}=0.19g, M_w=6), and the combination that results in the largest total volumetric strains is adopted. For the ULS, an a_{max} = 0.35g and M_w=7.5 is to be used.

2.2.2 *Japan*

Two earthquakes are considered for evaluation of liquefaction, Level 1 and Level 2. The return period for the Level 1 earthquake is approximately the life span of a structure (i.e., 100 years or less), and is commonly performed for return periods between 50 to 70 years. The Level 2 earthquake is related to the maximum credible earthquake and has a return period of at least 1000 years. The PGA for Level 1 is prescribed as 150~200cm/s^2 (0.15 – 0.2g) has a magnitude of 6.5. For Level 2 a minimum PGA of 350 cm/s^2 (0.36g) or larger is used, with a magnitude of 8 or higher (Abe 2018).

2.2.3 *Europe*

In Europe, seismic design is controlled by Eurocode 8 (European Committee for Standardization 2004). There are two seismic hazard levels for design of structures – the "No-collapse requirement" (NCR) and the "Damage limitation requirement" (DLR). The recommended values for the associated probability of exceedance for these two seismic hazard levels are 10% in 50 years (475-year return period) and 10% in 10 years (95-year return period), for the NCR and DLR, respectively, although alternate values may be adopted by an individual country. The seismic ground motion is described by the reference peak ground acceleration on ground type A (a_{gR}), where ground type A is rock or rock-like formation with a $V_{S,30}$ >800 m/s. For design purposes (and the evaluation of liquefaction), this reference ground acceleration is obtained from zonation maps, and then modified by an importance factor and a soil factor. Magnitude selection is not clearly specified; however, it is stated that seismic design should be based on the hazard assessment and "consideration should be given to the magnitude of earthquakes that contribute most to the seismic hazard" (European Committee for Standardization 2004).

It is interesting to note that the Eurocode 8 specifically mentions that if the controlling earthquakes that affect the site are generated by sources that are very different, it is suggested that the possibility of using more than one shape of spectra to represent the different scenarios should be considered. Under this evaluation, different values of peak ground acceleration would be required for the different earthquake and corresponding spectra, and subsequently, multiple liquefaction triggering evaluations.

Table 1. Seismic Ground Motions used for Liquefaction Triggering Evaluation in California

Standard or Regulation	Ground Motion
ASCE 7-10 (2016 CBC)	PGA_M and M_w from MCE_G
ASCE 7-16 (2018 IBC; 2019 CBC)	PGA_M and M_w from MCE_G
CGS Note 48	PGA_M and modal M_w from MCE_G
City of Los Angeles (2014)	Level 1: PGA=2/3 PGA_M; modal or mean M_w from 475-yr
	Level 2: PGA=PGA_M; modal or mean M_w from 2475-yr
Caltrans	PGA larger from deterministic or 975-yr probabilistic;
	M_w either deterministic or larger or mean or modal from 975-yr
CAHSR	MCE = PGA greater of 950-year probabilistic or deterministic
	OBE = PGA from 50-year probabilistic
NAVFAC (ordinary buildings)	Level 1: PGA and M_w from 72-yr probabilistic
	Level 2: PGA and M_w from 475-yr probabilistic
ASCE 61-14 ("High")	OLE: PGA and M_w from 72-yr probabilistic
	CLE: PGA and M_w from 475-yr probabilistic
	DE: PGA=PGA_M; modal or mean M_w from 2475-yr

Table 2. Seismic Ground Motions used for Liquefaction Triggering Evaluation Internationally

Standard or Regulation	Ground Motion
Standards New Zealand	ULS: PGA and M_W from 500-year probabilistic
	SLS: PGA and M_W from 25-year probabilistic
Japan	Level 1: PGA=150~200 cm/s^2 and associated Mw
	Level 2: PGA=350 cm/s^2 and associated Mw
Eurocode 8	NCR: PGA and M_W from 475-year probabilistic
	DLR: PGA and M_W from 95-year probabilistic

2.3 *Discussion*

The following table summarizes the ground motions used for the evaluation of liquefaction triggering from the various approaches and standards discussed above.

When the simplified procedures for evaluating liquefaction triggering were originally developed in the 1970's, the analysis was generally performed using a deterministic approach, with a consistent magnitude and associated PGA from a particular seismic source. However, although deterministic evaluation may still be performed, the trend in recent years has become to use probabilistic ground motions to represent the PGA and M_W instead. With this practice though, it is possible that an inconsistent pair of PGA and M_W may be used in the liquefaction hazard evaluation. Depending on the seismic environment and the deaggregation results at the desired hazard level, use of the larger of mean or modal magnitude may not be compatible with the PGA obtained from the hazard calculation.

Note that Eurocode addresses the need for potentially evaluating more than one seismic source if analyses show that the overall seismic hazard is controlled by very different sources. This is similar to the interim approach for evaluating liquefaction in the Canterbury Earthquake Region of New Zealand, where two scenarios are evaluated – a lower PGA with a higher magnitude and a higher PGA associated with a lower magnitude.

Additionally, review of Tables 1 and 2 indicates that aside from ASCE 7 and Caltrans, nearly all other approaches consider at least 2 seismic hazard levels – one associated with "design" ground motions at a longer return period (typically on the order of 500 years or greater) and one associated with more frequent shaking (on the order of 25 to 100 years).

3 PERFORMANCE-BASED DESIGN AND PROBABILISTIC LIQUEFACTION HAZARD EVALUATION

The National Academies of Science, Engineering and Mathematics (NASEM) released a report in 2016 entitled *State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences* (NASEM, 2016). In this report, NASEM does not make

recommendations on the hazard levels that should be used or seismic ground motions that practitioners should use in performing analyses. Instead, they make a general recommendation to "refine, develop, and implement performance-based approaches to evaluating liquefaction, including triggering, the geotechnical consequence of triggering, structural damage, and economic loss models to facilitate performance-based evaluation and design." Within the discussion of liquefaction analyses for performance-based design, the report suggests that probabilistic liquefaction hazard analyses (PLHA) are a useful tool to better understand uncertainties and risk associated with liquefaction. In PLHA, all levels of shaking and all contributing magnitudes are considered in the liquefaction analyses, and not just ground motions associated with a single return period as is conventionally required by codes and standards and performed in practice.

PLHA was introduced by Kramer and Mayfield (2007), and while it may be performed for larger or more complex projects where understanding the risks associated with liquefaction may be critical, PLHA is not performed in routine practice due to the complexity and effort required, which can be not only challenging to execute but often cost prohibitive.

To accommodate the need for an approach to PLHA that can be easily implemented in engineering practice, in recent years researchers (Mayfield et al. 2010, Franke et al. 2014, and Ulmer and Franke 2016) have developed simplified probabilistic procedures to evaluate liquefaction triggering. Using a similar methodology as the USGS National Seismic Hazard Mapping program, reference parameter maps have been developed by Ulmer and Franke (2016) corresponding to a reference soil profile, which can be corrected for site-specific conditions and to estimate factor of safety against liquefaction with depth, corresponding to the hazard level of interest. Unlike the conventional liquefaction hazard analyses performed in practice, which are all based on a single scenario event based on ground motions corresponding to a single seismic hazard level (such as PGA_M and M_W for a 2475-year return period), these liquefaction reference parameter maps incorporate a range of possible ground motion scenarios, for multiple seismic hazard levels.

Franke et al. (2016) notes that the conventional approach for selecting ground motions for liquefaction hazard analyses often results in practice with the false assumption that the probability of liquefaction triggering is the same as that of the ground motions themselves (i.e., since the 2475-year PGA was used, the corresponding liquefaction hazard has the same return period). Use of these ground motions is referred to as the "pseudo-probabilistic" procedure, since it is not a true probabilistic analysis of liquefaction. The reference parameter maps, on the other hand, by considering varied ground motions and their likelihood, are developed specifically to evaluate the return period associated with liquefaction directly.

Despite the advantages of PLHA, one key disadvantage is that for conventional projects, performance of PLHA may not meet code requirements. In performance-based design, project-specific seismic design criteria are developed and agreed upon by the reviewing agency. Exceptions or modifications to the code may be performed. But for a conventional structure (such as a building) that follows the more prescriptive code requirements, use of PLHA may not be compatible with the code.

4 ALTERNATIVE INTENSITY MEASURES FOR REPRESENTING SEISMIC DEMAND

Although PGA has been conventionally used to represent the ground motion intensity for liquefaction triggering procedures and evaluating the consequences of liquefaction, the parameter has its limitations. One of the most significant limitations is that PGA is a scalar value; it represents the largest transient acceleration in an earthquake record. However, it does not capture the overall behavior of the earthquake record very well and may not accurately capture the duration or energy associated with shaking.

Alternative intensity measures (IMs) have been proposed for use in liquefaction evaluations by various researchers. There are generally two types of IMs – peak transient IMs and evolutionary IMs. Peak transient IMs, such as PGA or spectral acceleration at a particular period (Sa (T)), represent the largest amplitude in an earthquake record but may not accurately represent duration or frequency content. Evolutionary IMs, such as Arias Intensity (I_A) or cumulative average velocity (CAV), accumulate throughout earthquake shaking. In recent studies, the evolutionary IMs promise to be better predictors of liquefaction triggering and consequences.

One of the early studies in this area was by Kramer and Mitchell (2006), who evaluated the efficiency and sufficiency of three IMs, PGA, Ia, and cumulative average velocity after a 5 cm/s^2 threshold acceleration (CAV_5), for evaluation of liquefaction. They ran numerous liquefaction simulations using the 1-D nonlinear effective stress site response analyses using the program WAVE by scaling the input ground motions to the same target IM value, then evaluating the pore pressure generation under the scaled ground motions. They found that the ground motions scaled to CAV_5 produced the most uniform pore pressure generation, even with randomly selected ground motions. Their conclusion was that using an IM that results in more accurate estimates can help reduce the conservatism in interpretation of results.

Greenfield and Kramer (2018) also recently evaluated three IMs (PGA_M, I_a, and CAV) and their relationship to liquefaction triggering and post-triggering deformation, particularly for long-duration ground motions. Their investigation found that I_a and CAV correlate very well with duration, however, PGA_M does not.

Recent research by both Bray and Macedo (2018) and Bullock et al. (2018) has considered alternate intensity measures in the development of new simplified models to estimate settlements of shallow foundations on liquefiable sites. Bray and Macedo (2018) concluded that CAV_{dp} (standardized CAV) and the 5%-damped one-second spectral acceleration (S_a1) were the optimal IMs. Bullock et al. (2018) concluded that CAV was the optimum predictor for foundation settlement, and CAV, PGV, and V_{gi} (peak incremental ground velocity) were the optimum predictors for building tilt.

Observations of liquefied and non-liquefied sites after recent large Chilean subduction zone earthquakes were used as case studies by Montalva et al. (2017) to evaluate the widely used liquefaction triggering procedures. Using the most frequently used current shear wave velocity-based methods and SPT-based methods (where data was available), they found large errors in the prediction of liquefaction triggering. In this dataset, the error was greater than 35%, all with false positives (methods predicted liquefaction, but none was observed). The authors suggest that the higher error for the Chilean earthquakes may be related to the long duration of ground shaking in comparison to the events that generally control the existing dataset used in the development of triggering methodologies, and that PGA is not the best IM to represent the seismic demand of large subduction zone earthquakes.

5 DISCUSSION AND CONCLUSIONS

Seismic ground motion for evaluating liquefaction varies depending on the structure type (building, bridges, piers, etc.) and seismic hazard level of interest, and is generally represented by PGA and M_W based on deterministic or probabilistic seismic hazard analyses. Even though currently ASCE 7 uses only a single hazard level to evaluate liquefaction, most other codes, including all of the international codes we reviewed, consider a minimum of two seismic hazard levels. When probabilistic ground motions are used, care should be taken to adopt a consistent magnitude and PGA pairing. In our opinion, the use of two seismic hazard levels (such as a Maximum Considered Earthquake level, and a more frequent earthquake within the lifetime of the structure, such as 50-100 years) may be a better approach to understanding liquefaction risks for a given site.

PLHA has several advantages against the conventional practice using the "simplified procedure", PLHA is consistent with a Performance-Based Design approach, it can prevent the erroneous pairing of PGA obtained from PSHA with an inconsistent magnitude, and since a wide variety of ground motions can contribute to the evaluation, the risk of liquefaction at a particular seismic hazard level can be evaluated directly. In our opinion, simplified PLHA may be a path forward to improving our ability to better understand the liquefaction hazard.

With regard to the more recent research surrounding evolutionary IMs to represent ground motions, what is interesting is that none of the approaches have used either the same software, models, or methods, and yet their results tend to converge – evolutionary intensity measures, particularly CAV, are better at representing liquefaction and its effects. In areas where the "simplified procedure" may not be as effective (such as megathrust earthquakes), representing the seismic ground motions by an evolutionary IM instead of PGA may be a good alternative.

REFERENCES

Abe, A. 2018. Personal Communication, Tokyo Soil Research Co. Ltd., Tokyo, Japan.

ASCE 2010. *ASCE Standard ASCE/SEI 7-10, Minimum Design Loads for Buildings and Other Structures*. Reston, Virginia: American Society of Civil Engineers.

ASCE 2014. *ASCE/COPRI 61-14, Seismic Design of Piers and Wharves*. Reston, Virginia: American Society of Civil Engineers.

ASCE 2017. *ASCE Standard ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures*. Reston, Virginia: American Society of Civil Engineers.

Bray, J.D. & Macedo, J. 2018. Evaluating liquefaction-induced building settlements. *Proceedings of the 11th National Conference in Earthquake Engineering*, Los Angeles, CA: Earthquake Engineering Research Institute, Los Angeles, CA.

Bullock, Z., Dashti, S., Liel, A., Porter, K. & Karimi, Z. 2018. Efficiency, Sufficiency, and Predictability of Intensity Measures for Predicting Liquefaction Consequences. *Proceedings of the 11th National Conference on Earthquake Engineering*, Los Angeles, CA: EERI

California Building Standards Commission 2016. *2016 California Building Code, California Code of Regulations, Title 24*. California Building Standards Commission: Sacramento, CA.

California Geological Survey 2013. Note 48, Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings.

California High Speed Rail 2012. California High-Speed Train Project Design Criteria.

Campbell, K. W. & Bozorgnia, Y. 2011. Predictive Equations for the Horizontal Component of Standardized Cumulative Absolute Velocity as Adapted for use in the Shutdown of U.S. Nuclear Power Plants. *Nuclear Engineering and Design* 241(7): 2558–2569.

City of Los Angeles 2014. Letter to Geology and Soil Engineering Firms Practicing in the City of Los Angeles from the Grading Division, Los Angeles Department of Building and Safety, July 16, 2014.

County of Los Angeles 2013. *Manual for Preparation of Geotechnical Reports*. County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division, July 1, 2013.

County of Los Angeles 2014. GS 045.0, Liquefaction and Lateral Spread. *Administrative Manual*, County of Los Angeles, Department of Public Works, Geotechnical and Materials Engineering Division.

European Committee for Standardization (ed.) 2004. *EN 1988–1, Eurocode 8: Design of structures for earthquake resistance – Part 1: General rules, seismic actions and rules for buildings*. Technical Committee CEN/TC 250 "Structural Eurocodes", European Committee for Standardization (CEN).

Franke, K.W., Mayfield, R.T. & Wright, A.D. 2014. Simplified uniform hazard liquefaction analysis for bridges. *Transportation Research Record* 2407(1): 47–55.

Franke, K.W., Ulmer, K.J., Ekstrom, L.T. & Meneses, J.M. 2016. Clarifying the differences between traditional liquefaction hazard maps and probabilistic liquefaction reference parameter maps. *Soil Dynamics and Earthquake Engineering* 90: 240–249.

Greenfield, M.W. & Kramer, S.L. 2018. The Effects of Long-Duration Ground Motion on Soil Liquefaction. *Proceedings of the 11th National Conference in Earthquake Engineering*, Los Angeles, CA: Earthquake Engineering Research Institute.

Kramer, S.L. & Mayfield, R.T. 2007. Return period of soil liquefaction. *Journal of Geotechnical and Geoenvironmental Engineering* 140(1): 110–120.

Kramer, S.L. & Mitchell, R.A. 2006. Ground Motion Intensity Measures for Liquefaction Hazard Evaluation, *Earthquake Spectra* 22(2): 413–438.

Mayfield, R.T., Kramer, S.L. & Huang, Y.M. 2010. Simplified approximation procedure for probabilistic evaluation of liquefaction potential. *Journal of Geotechnical and Geoenvironmental Engineering* 136: 140–150.

Montalva, G.A., Ruz, F., Roncagliolo, J. & Escare, E. 2017. Liquefaction Triggering Analysis in Megathrust Earthquakes, *Proceedings of the 16th World Conference on Earthquake Engineering*, Santiago, Chile, January 9th to 13th, 2017.

National Academies of Science, Engineering and Mathematics 2016. State of the Art and Practice in the Assessment of Earthquake-Induced Soil Liquefaction and Its Consequences, Committee on State of the Art and Practice in Earthquake-Induced Soil Liquefaction Assessment, Board on Earth Sciences and Resources, Division of Earth and Life Studies. Washington DC: The National Academies Press.

NAVFAC 1997. *Seismic Design for Buildings*, Technical Manual P–355.

New Zealand Transport Agency 2014. *Bridge manual (SP/M/022)* Third edition.

New Zealand Geotechnical Society 2016. New Zealand Geotechnical Society (NZGS) and Ministry of Business, Innovation & Employment (MBIE) guidelines for Earthquake Geotechnical Engineering Practice in New Zealand, Module 1: Overview of the Guidelines.

Ulmer, K.J. & Franke, K.W. 2016. Modified performance-based liquefaction triggering procedure using liquefaction loading parameter maps. *Journal of Geotechnical and Geoenvironmental Engineering* 142(3).