UCLA

UCLA Previously Published Works

Title

Procedures from International Guidelines for Assessing Seismic Risk to Flood-Control Levees

Permalink

<https://escholarship.org/uc/item/66g2x2xg>

Journal

Earthquake Spectra, 33(3)

ISSN

8755-2930

Authors

Zimmaro, Paolo
Kwak, Dong Youp
Stewart, Jonathan P
et al.

Publication Date

2017-08-01

DOI

10.1193/072316EQS117EP

Peer reviewed

Procedures from international guidelines for assessing seismic risk to flood control levees

Paolo Zimmaro,^{a)} M.EERI, Dong Youp Kwak,^{b)} M.EERI, Jonathan P. Stewart,^{a)} M.EERI, Scott J. Brandenburg,^{a)} M.EERI, Ariya Balakrishnan,^{c)} Ruben Jongejan,^{d)} Ernesto Ausilio,^{e)} Giovanni Dente,^{f)} Junju Xie,^{g)} and Atsushi Mikami^{h)}

We synthesize engineering procedures for estimating the seismic performance of major flood control levees as given in guidelines documents and design codes from Canada (British Columbia), China, Italy, Japan, New Zealand, and the United States of America (USA). Some guidelines carry the weight of law whereas compliance is optional for others. Most procedures combine a probabilistic ground motion characterization with deterministic assessments of levee performance (uncoupled approach). Ground motions are typically described using peak accelerations for reference site conditions at return periods typically ranging from 100 to 2475 years. Those motions are deterministically modified for soil conditions using numerical simulations or ergodic site factors. Accompanying hydrological conditions are either not specified or are taken at a frequently encountered water level (ranging from mean annual to a 4-month flood event). These demands are used in combination with various soil properties to assess the potential for liquefaction, flow failure, and permanent shear deformations. Drawing upon best practices identified from this review, we recommend procedures for levee risk assessment at the section-level and for levee systems.

^{a)} Department of Civil and Environmental Engineering, University of California Los Angeles, CA, USA

^{b)} RMS, Inc., Newark, CA, USA

^{c)} Department of Water Resources, Sacramento, CA, United States

^{d)} Jongejan RMC, Delft, The Netherlands

^{e)} Department of Civil Engineering, University of Calabria, Italy

^{f)} Department of Environmental and Chemical Engineering, University of Calabria, Italy

^{g)} Institute of Geophysics, China Earthquake Administration, Beijing, China

^{h)} Department of Civil Engineering, Tokai University, Japan

INTRODUCTION

A levee is generally defined as a natural or constructed embankment along the margins of a water body, with the primary purpose of providing protection against flood events. In this paper *levee* refers to both flood-control structures that do not routinely retain water, and embankments that retain water continuously. Levee systems are essential lifelines for population centers and agricultural or industrial areas located adjacent to water bodies. As such, the performance of levees when subjected to storm surge, other flood events, and earthquakes is essential for the resilience of surrounding communities. Despite their critical function, many levees were not properly engineered at the time of their construction and are often founded on soft and weak soils. Not surprisingly, levees are frequently found to have been damaged following major earthquakes (Miller and Roycroft, 2004; Sasaki, 2009; Sasaki et al., 2012; Green et al., 2011; Kwak et al., 2016a).

We have two broad objectives with this paper. The first is to document standard procedures used to assess seismic risk to levees in guidelines documents and design codes from seismically active regions globally. Static stability and risk associated with high-water events (flooding) are topics beyond the scope of this paper, although we recognize commonalities between the respective analyses (e.g., similar site characterization needs). We propose a comprehensive framework comprised of six steps that encompass these procedures as a whole, although frequently one or more steps may be missing from a particular set of guidelines. Having reviewed and evaluated these procedures, our second objective is to provide recommendations for seismic levee guidelines, based on the proposed six-step framework, that attempts to optimize accuracy in the assessment of levee risk while maintaining simplicity.

This work was enabled by a substantial collaborative effort among public agencies, research centers, design professionals, and university researchers, which facilitated access to the key documents used in engineering practice. We have considered guidelines specifically developed for levees and embankments and currently in effect for British Columbia (Canada) (MFLNRO, 2014), China (Ministry of Water Resources, 2005, 2008, 2013), Japan (MLIT, 2007, 2012), and the USA (USACE, 2000, 2014; CA-DWR, 2012, 2015). We also consider approaches from building codes for Italy (Ministry of the Infrastructures, 2008), unenforced state-of-practice guidelines for New Zealand (NZSOLD, 2015) originally written for dams but used in practice for critical levees, as well as the international levee handbook (CIRIA, 2013; hereafter termed global guidelines).

Our six-step framework is useful both to compare approaches from the selected documents and to organize our recommendations. The steps in the assessment process, as detailed in Table 1, are as follows:

- i. Preliminary screening to identify sites not requiring detailed analysis, based on hydrology, geomorphology, depositional environment, and/or level of shaking;
- ii. Site characterization from geological, geotechnical, and hydrological studies;
- iii. Hazard characterization, including ground motion hazard analysis for a reference site condition (typically at selected earthquake return periods, T_E), assessment of site and levee response in consideration of local soil conditions, and co-seismic flood hazard characterization (i.e., water level assumed to be present during design seismic event; the associated return period, T_{WS} , is short relative to that for non-seismic flood event, having return period T_W);
- iv. Assessment of potential for ground failure including soil strength loss associated with liquefaction of granular soils or cyclic softening of clayey soils;
- v. Analysis of post-earthquake instability (including flow failure) as well as seismic and post-seismic deformations. A critical consideration in this step is freeboard loss and its implications for levee breach potential under the co-seismic flood level; and
- vi. Risk mitigation planning and post-earthquake response.

We describe commonalities present in multiple guidelines documents, as well deviations from the general procedures. We illustrate key aspects of steps (iii) – (v) using a hypothetical levee section in an active crustal region.

The six-step procedure is directed toward risk assessment for individual levee segments, which must then be extended to assess risk for a levee system comprised of many segments. Germane to that process is estimation of spatial distribution and correlations of the demands applied to levees and the internal resistances (or capacities) of levee segments against those demands. Methods for characterizing these correlations and evaluating system fragility are considered in the Netherlands (VNK, 2015) and the USA (DRMS, 2008; USACE, 2011), and neglected elsewhere. We consider this subject essential to levee risk assessment, and describe previously applied approaches to this problem by Delta Risk Management Strategy (DRMS, 2008), USACE (2011), a model applied for national flood risk analysis in the Netherlands (VNK, 2015; Jongejan and Maaskant, 2015), and Kwak et al. (2016b).

We conclude by describing the level of enforcement associated with the levee guidelines documents - are they suggested guidelines without enforcement, or do they carry the weight of law as with building codes?

LEEVE RISK ASSESSMENT PROCEDURES

PRELIMINARY SCREENING

Procedures for Preliminary Screening from Guidelines

Screening has the objective of identifying sites for which detailed seismic analysis is not required based on a combination of low seismic hazard, competent soil conditions, low co-seismic flood levels, and/or low consequences of failure. This step is included in the Japan, China and USACE documents (Table 1) and is not included elsewhere. The MLIT (2012) guidelines for Japan describe a sequential two-step screening procedure. First, the levee is screened out (i.e., exempted from further consideration) if the high river water level (HWL) (defined as the river water elevation having a return period of $T_{WS}=0.37$ years, JICE, 2007) is lower than the elevation at one-quarter of the levee height. If this criterion is not met, the levee is still screened out if it is founded on stiff geologic materials classified on national geomorphic maps (Wakamatsu and Matsuoka, 2013) as alluvial fan, mountain, or cliff. The Chinese guidelines also consider the surface geology of foundation soils, with levees on lower Pleistocene materials (or older) screened out. Screening guidelines by USACE (2014) are based on peak ground acceleration (PGA); levees are screened out for design-basis $PGA < 0.1$ g or < 0.05 g, with the latter applying only for frequently hydraulically-loaded levees (defined as river water elevation 0.3 m or higher than the levee landside toe elevation at least once a day for more than 36 days per year on average) and loose foundation soil conditions. The CA-DWR (2015) guidelines require seismic analysis only for urban levees, defined as levees located in a developed area in which there are 10,000 residents or more; non-urban levees are exempted.

Recommendations for Preliminary Screening

We consider each of the screening criteria to be reasonable for the respective application regions. We recommend the use of screening as part of levee risk assessment and consider the most effective bases for excluding levee sections from further consideration to be low co-seismic water levels (relative to the lowest levee base elevation), competent soil conditions in the foundation and levee fill, low seismic demands, and low failure consequences. What comprises “low” and “competent” should be selected based on local practices and experience.

SITE CHARACTERIZATION

Procedures for Site Characterization from Guidelines

As used here, the term *site characterization* refers to subsurface exploration and laboratory testing having the purpose of evaluating site stratigraphy, ground water elevation, and geotechnical engineering properties of levee and foundation materials. All of the guidelines documents listed in Table 1 require site characterization, five of which are prescriptive regarding the spacing of boreholes or cone penetration test (CPT) soundings in the transverse direction (across the cross-section) and/or longitudinal direction (parallel to river flow). As illustrated in Figure 1, prescriptive guidelines generally specify the number of boreholes or CPTs per section, with three being typical (water-side slope, land-side slope, and crest). Where specified, longitudinal spacing L between investigated sections is in the range of 100 to 500 m, being < 300 m by USACE (2000) and the British Columbia (MFLNRO, 2014) guidelines, and 100-500 m in the Chinese guidelines. Where specified, exploration depths Z are given as 1.5 to 2 times the levee height (h) in the Chinese guidelines, 10 to 30 m or practical refusal (British Columbia), or penetration into the foundation layer by a depth of at least h and not less than 3 m (USACE, 2000). Other guidelines documents are not prescriptive regarding lateral spacing or depth of subsurface investigations.

None of the guidelines in Table 1 provide specific recommendations regarding laboratory testing, although the need for laboratory testing to measure soil properties is mentioned in all of the guidelines. The guidelines do not discuss interpretation of variable soil properties within a layer for geotechnical analysis.

Figure 1. Levee sketch illustrating configuration of boreholes and/or CPT soundings.

Recommendations for Site Characterization

We consider three boreholes or CPTs per section to be good practice, because of the lateral heterogeneity normal to the direction of river flow that is often encountered. We refer to manuals and guidelines available in the literature (e.g. Sabatini et al., 2002; Robertson and Cabal, 2015) regarding good practice in the combined use of CPT soundings and borings with sampling (including SPT). We recommend down-river section separation distances and exploration depths be left to the discretion of the field engineer or geologist. However, maximum distances of 500 m would appear to be prudent, with closer section spacing as levees encounter changing surface geologic conditions. Existing borings should be considered in developing site exploration plans – they are often useful for defining stratigraphy, but may have unreliable SPT penetration resistance values due to unknown energy levels associated with the hammer type and lift/drop mechanism.

For soils containing fines, laboratory testing of index properties, consolidation, and shear strength can be useful for assessing liquefaction susceptibility or cyclic softening potential. Vane shear tests are useful for undrained strength characterization in fine-grained soils, particularly soft clays or peats. Whenever practical, variability in soil properties should be considered in geotechnical analysis so as to understand the sensitivity of analysis results to inherent soil property variability. This issue is discussed in detail elsewhere (e.g. Phoon and Kulhawy, 1999; Gordon and Griffiths, 2008).

Table 1. Overview of levee risk assessment guidelines documents and their attributes.

Document	Preliminary screening	Site characterization	Hazard characterization (PGA)			Liquefaction			Post-earthquake instability: Flow failure (FF); deformation analysis (DA)
			Ground motion hazard; ref. site cond. ¹	Corresp. flood T_{WS}^3	Site-levee response ⁴	Susceptibility ⁵	Triggering ⁶	S_{u-liq} ⁷	
Japan: MLIT (2007, 2012)	Based on hydrol. and geomorph.	3 borings per section, laboratory tests (not specified)	Determ.; V_{S30}^{ref} = unspecified	$T_{WS}=0.37$ years	H-E-G (site class based on site period)	Susc. if: $D_w > -10\text{m}$; and $FC < 35\%$ or $PI < 15$, and $D50 < 10\text{mm}$ and $D10 < 1\text{mm}$	Lab-based resistance model (JRA, 2002 and Tamura, 2014)	Empirical and lab based, JRA (2002) and Tamura (2014)	FF: fluid dynamics (Towhata et al., 1999); DA: Pseudo-static (Yasuda et al., 1999), fluid dynamics, or numerical analysis (Iai, 1988)
Italy: Ministry of the infrastructures, (2008)	Not required	Field and laboratory tests (not specified)	Prob. ($T_E^2=30, 50, 475, 2475$ years); $V_{S30}^{ref} = 800$ m/s	Not specified	H-E-G,T (site class from V_{S30})	Not susc. if: $D_w < -15\text{m}$; or $q_{c1N} > 180$, or $(N_1)_{60} > 30$, or grain size distribution-based criterion	Semi-empirical, and/or numerical analysis	Strength reduction suggested, no procedure given	FF: N/A DA: Pseudo-static, Newmark-type, or numerical analysis
New Zealand: NZSOLD (2015)	Not required	Field and laboratory tests (not specified)	Determ. or prob. ($T_E=500, 2500$ years); $V_{S30}^{ref} = 760$ m/s	Not specified	H-E-G (site class from V_{S30} and site period)	Not specified	Semi-empirical (Idriss and Boulanger, 2008, 2014), and/or numerical analysis	Strength reduction suggested, no procedure given	FF: static with S_{u-liq} ; DA: Pseudo-static, Newmark-type, or numerical analysis
China: Ministry of Water Resources (2005, 2008, 2013)	Based on depositional environment	3 to 6 borings per section, $L=100\text{m}$ to 500m , $Z=1.5$ to $2h$, laboratory tests	Prob. ($T_E=475\text{yrs}$); $V_{S30}^{ref} = 800$ m/s	Not specified	N/A	Susc. if: $w > 0.9\text{-LL}$, and $LI > 75$, Grain size distribution-based criterion (Wang, 1980) (No D_w criteria)	Lab-based resistance model (Xie, 2011)	Not specified	FF: N/A DA: Pseudo-static or numerical analysis
British Columbia, Canada:	Not required	3 borings per section,	Prob. ($T_E=100, 475, 2475$ years);	Mean annual	H-E-G,T (site class from V_{S30})	Susc. if:	Semi-empirical (Youd et al., 2001)	Empirical, Idriss and	FF: Pseudo-static or Newmark-type with S_{u-liq} ;

MFLNRO (2014)		$L = 300 \text{ m}$, $Z=10 \text{ m}$ to 30 m	$V_{530}^{ref} = 450 \text{ m/s}$	water level ($T_{WS} \approx 0$)		$PI < 7$; (No D_W criteria)		Boulanger (2008)	DA: Pseudo-static, Newmark-type, or numerical analysis
Global: International levee handbook, CIRIA (2013)	Not required	Field and laboratory tests (not specified)	Determ. or prob. (T_E not specified); $V_{530}^{ref} =$ unspecified	$T_{WS}=5 \text{ yrs}$	N/A	Not susc. if: $PI > 7$; (No D_W criteria)	Semi-empirical, and/or numerical analysis	Not specified	FF: N/A DA: Pseudo-static, Newmark-type, or numerical analysis
USA, Federal: USACE (2000, 2014)	$PGA < 0.1$, or $PGA < 0.05g$ for frequently hydraulically loaded levees with loose foundation conditions	$L=60\text{m}$ to 300m , $Z \geq$ greater of $2h$ or $h+3 \text{ m}$	Determi. or prob. (T_E same as the flood return period used for non-seismic applic.); $V_{530}^{ref} = 760 \text{ m/s}$ or P	highest of median annual, typical seasonal water level, and mean high tide elev.	P-E-G (site param. is V_{530}) or H-E,S-G (site class from V_{530})	Susc. if: $PI < 10$ and $LL < 35$; (No D_W criteria)	Semi-empirical (Youd et al., 2001). Stress demand for 2D geometries from Athanasopoulos-Zekkos and Seed, (2013)	Empirical, Seed and Harder (1990), or Olson and Stark (2002)	FF: static with S_{u-liq} ; DA: Semi-empirical lateral spread models if $h < 4.6 \text{ m}$, Newmark-type if $h > 4.6 \text{ m}$, or numerical analysis
USA, CA: DWR (2012, 2015)	Based on levee location (no seismic analysis for non-urban levees)	Field and laboratory tests (not specified)	Probabilistic $T_E=200 \text{ yrs}$; $V_{530}^{ref} = 335 \text{ m/s}$	higher between avg. summer or winter water elev.	H-E,S-G,T (site class from levee geom. & foundation soils)	Not susc. if: $PI > 10$, or $LL > 35$, or $w < 0.8 \cdot LL$ (No D_W criteria)	Semi-empirical (Cetin et al., 2004, Moss et al., 2006)	Empirical, Seed and Harder (1990)	FF: static or Newmark-type with S_{u-liq} ; DA: Pseudo-static, Newmark-type, or numerical analysis

¹ Reference site condition refers to conditions used in PSHA, which are generally rock or firm soil with a time-averaged V_{530} denoted V_{530}^{ref} motion subsequently modified for site effects. Exception is PSHA performed for actual site condition (marked with P).

² T_E = earthquake return period.

³ T_{WS} = co-seismic water level return period.⁴ H=hybrid, P=PSHA with site term in hazard integral; E=ergodic site factor, S=site-specific amplification function; G=site factor based on V_{530} or site classes (applicable to level ground), T=site factor for topographic effects; N/A = no recommendations for site amplification.

⁵ D_W = difference between elevations of groundwater and base of levee (< 0 for phreatic surface in foundation); FC=finest content; PI=plasticity index; D_{50} and D_{10} = sieve sizes retaining 50% and 10% by weight passing, respectively; q_{c1N} = dimensionless and overburden-normalized CPT tip resistance (Robertson and Wride, 1998); $(N_1)_{60}$ = energy- and overburden-corrected SPT blow count; w = water content; LL = liquid limit; LI = liquidity index.

⁶ Triggering. All models use stress-based liquefaction demand and resistance. Lab-based resistance criteria from Japan derived from JRA, 2002 and Tamura, 2014. Semi-empirical approaches with no specific recommended method given in Italian and global guidelines. Italian document indicates no triggering for $PGA < 0.1g$, or $M < 5$. Semi-empirical and lab-based criteria from China from Xie (2011) (stress-based methods with limiting values of shear wave velocity, SPT blow count, or relative density).

⁷ Post-liquefaction undrained shear strength.

HAZARD CHARACTERIZATION

Procedures for Ground Motion Hazard and Site-Levee Response from Guidelines

Seismic hazard characterization for levee sites has three phases: (1) ground motion characterization for reference (relatively firm) site conditions; (2) site response analysis to estimate the change in ground motions from the reference condition to the ground surface in consideration of local site conditions; and (3) evaluation of co-seismic river channel water level elevation assumed to be present during the design seismic event.

Reference site seismic hazard is most often characterized by means of probabilistic seismic hazard analysis (PSHA; McGuire, 2004), which considers all credible seismic sources; magnitudes and earthquake locations on those sources along with their relative likelihoods of occurring; rate of earthquakes on each source; and ground motion levels, combined with their relative likelihoods of occurring, for each of the possible magnitude-location combinations. PSHA results are given by the level of a ground motion intensity measure (e.g., peak ground acceleration or velocity, PGA or PGV) and the mean return period (T_E) with which that intensity measure is likely be exceeded. The value of T_E represents the seismic hazard level considered in the analysis, and is set according to the perceived importance of the levee system. As shown in Table 1, T_E ranges from 30 years (lowest hazard level considered in the Italian building code) to ~2500 years (New Zealand guidelines; highest hazard levels considered by Italian building code and in British Columbia). The intensity measure considered in seismic guidelines is always peak ground acceleration (PGA), which is provided on maps (e.g. Petersen et al., 2015 for the USA, Stucchi et al., 2011 for Italy, Halchuk et al., 2014 for Canada, and General Administration of Quality Supervision, Inspection and Quarantine of China, 2015 for China) for a reference site condition specified in the form of time-averaged shear wave velocities in the upper 30 m (V_{s30}^{ref}) ranging from 335 to 800 m/s (Table 1). In Japanese, New Zealand, and USACE guidelines documents (MLIT 2007; NZSOLD, 2015; USACE, 2014), deterministic specifications of ground motions are used in lieu of, or as an alternative to, PSHA. The USACE (2014) guidelines do not provide specific recommendations on source attributes or the percentile level of ground motion. The New Zealand guidelines allow deterministic hazard assessment using the most recent New Zealand national seismic hazard source model (Stirling et al., 2012) and either median or 84th percentile ground motions, depending on the importance of the levee. The MLIT (2007) procedures are unspecified with respect to source

and ground motion attributes, and consist of uniform PGA maps within three broad zones encompassing the breadth of the land mass.

In the majority of guidelines, site effects on ground motion are accounted for in a deterministic manner (referred to as a *hybrid approach* by Cramer, 2003) by multiplying the probabilistic reference site ground motion by an ergodic site factor (Italy, Japan, New Zealand, British Columbia, and CA-DWR guidelines), which in some cases is supplemented with a topographic amplification factor (British Columbia, Italy). In this context, ergodic indicates that the site factors are derived from ground motion data for diverse geographic regions, and despite their conditioning on V_{S30} or other site parameters, they are not specific to the site of interest.

Figure 2 shows ergodic site factors recommended for hybrid application in the levee guidelines, which exhibit considerable variability (up to approximately a factor of two). The USACE and CA-DWR guidelines refer to site factors published by the National Earthquake Hazards Reduction Program (NEHRP) (BSSC, 2015), which are shown in Figure 2. All considered site factors are category-based, meaning that site amplification is taken as constant within-categories as indicated by the steps in Figure 2. Category limits are either defined directly on the basis of V_{S30} (British Columbia, Italy, New Zealand, USACE, CA-DWR) or, in the case of Japan, can be readily associated with V_{S30} ranges based on site period- V_{S30} correlations (e.g. Zhao et al., 2006) (no site factors are provided in the China or global guidelines). The NEHRP, British Columbia, and Italy site factors are nonlinear, meaning that they can vary with the strength of the reference site ground motion, while the Japan and New Zealand site factors are linear. The nonlinear site factors shown in Figure 2 apply for a reference site peak acceleration of 0.3 g.

Figure 2. V_{S30} -scaling recommended in the levee guidelines. Site factors are shown for a reference site PGA of 0.3 g. The NEHRP factors shown here are used by the USACE and CA-DWR documents.

As an alternative to ergodic site factors, site-specific 1-D or 2-D numerical simulation of wave propagation through the soil deposit can be used to compute site amplification (USACE, 2014; CA-DWR, 2015) (indicated as S in Table 1), which is combined with the reference site hazard in a hybrid manner.

The hybrid approach of deterministically combining site amplification models with a reference site hazard curve (ubiquitous across guidelines) does not preserve the target hazard level in the modified ground motion. Prior work (e.g., Goulet and Stewart, 2009) has shown this approach to under-predict ground motions relative to procedures in which site amplification effects are included within the hazard calculation. Only USACE (2014) guidelines allow for PSHA performed using site-specific V_{S30} coupled with the ergodic site terms embedded within ground motion models (indicated as P in Table 1). This approach overcomes the hybrid bias, but remains ergodic (the site response is not truly site-specific).

Procedures for Water Level Definition from Guidelines

The river water level that should be used in combination with the seismic hazard can be defined in a probabilistic manner having a so-called *co-seismic water level return period* (T_{WS}) corresponding to the mean time interval between which that river water level would be expected to be exceeded. The value of T_{WS} used in this context should not be confused with the

flood return period used for non-seismic applications (T_W), which is quite long (decades or centuries). As shown in Table 1, there are wide variations in T_{WS} , ranging from effectively null (mean annual water level; British Columbia) to 5 years (global). The US guidelines cannot be readily translated to T_{WS} , but would correspond to very low values, being given by USACE (2014) as the higher of the *median annual, typical seasonal water level, and mean high tide elevation* and by CA-DWR (2015) as *the higher of the average summer or winter seasonal water elevations*.

Recommendations for Hazard Definition

Our recommendation is to describe ground motion and water level hazards probabilistically, as is already suggested in many of the guidelines documents, but with disaggregation of the seismic hazard to define controlling earthquake events. We understand that return periods will vary according to the tolerable risk thresholds considered to be acceptable by local populations and the local significance of flood hazards that would be associated with levee breach, so a single universal value should not be prescribed. Our recommendation to use probabilistic, as opposed to deterministic hazard characterization is motivated by the clear meaning of return periods (produced by PSHA) vs. the arbitrary pre-selection of earthquake events and ground motion percentiles inherent to deterministic analysis (the details of which are often obfuscated).

We agree with the presently typical practice of combining long T_E values with relatively short T_{WS} values. For critical levee systems, site-specific PSHA that incorporates non-ergodic site response (within-PSHA implementation of site-specific amplification, P-S-G; Stewart et al., 2017) and appropriate levels of spatial variability of seismic demands (Jayaram and Baker, 2009) is preferred to the present practice of hybrid analysis of site response effects and independent PSHA for all levee locations. Hybrid procedures underestimate hazard (illustrated below), while lack of consideration of spatial variations in ground motion overestimate the hazard on a system-wide basis. These sources of bias, while offsetting, are undesirable because they result in ground motion hazard levels that are unknown, and likely biased relative to target values.

GROUND FAILURE AND STRENGTH LOSS

Levee deformations can be produced by several seismically-induced ground failure mechanisms: (1) liquefaction of saturated cohesionless materials (e.g. gravels, sands, low-plasticity silts) (e.g., Idriss and Boulanger, 2008), (2) seismic slope instability (e.g., Blake et

al., 2002), (3) accelerated volume change of peat from secondary compression re-set (Shafiee et al., 2015), (4) seismic compression of partially saturated levee fill (Duku et al., 2008; Yee et al., 2014), and (5) strength loss from cyclic softening in clay-like fine-grained soils (Boulanger and Idriss, 2007). These ground failure mechanisms tend to act in tandem, for example liquefaction and cyclic softening are associated with undrained strength reductions that may produce slope instability. Because levees are usually located in deltaic, fluvial, and alluvial areas, foundation soils are poorly consolidated and frequently susceptible to liquefaction or cyclic softening if strongly shaken. As a result, ground failure evaluation is universal among the guidelines documents in Table 1. However, these guidelines consider only liquefaction and seismic slope instability, and do not address other mechanisms (exceptions are British Columbia and New Zealand guidelines, which mention cyclic softening, without providing recommendations). The remainder of this section describes liquefaction guidelines; seismic slope instability is covered next.

Procedures for Liquefaction Potential from Guidelines

Liquefaction criteria in the guidelines documents focus principally on issues of susceptibility (pertaining to soil type, and in some cases, groundwater elevation) and liquefaction triggering for susceptible materials. Criteria related to these issues are given in Table 1 for each guideline. Five of the eight guidelines also make reference to post-liquefaction strength reduction, with four providing specific recommendations for evaluating liquefied shear strength (Table 1). Guidelines are not provided for other liquefaction effects, such as surface manifestation, post-liquefaction settlement, or lateral spreading (exception is the USACE guidelines, which provide recommendations for modeling lateral spread displacements).

Susceptibility guidelines provide both exclusive (conditions for which soil is not susceptible) and inclusive (susceptible conditions) criteria, as shown in Table 1. All guidelines consider soil type in some manner, generally involving interpretation of soil plasticity tests and in some cases gradation tests or water content. Only two of the guidelines (Japan and Italy) provide criteria for ground water elevation relative to levee base (D_w , negative for water table in foundation, positive in levee fill). Other documents mention saturation as a necessary condition for liquefaction, but provide no specific depth criteria.

Recommendations for triggering analysis are given by each guideline. All guidelines use stress-based approaches for evaluating seismic demands and liquefaction triggering resistance,

but there is little consistency in the details of these approaches. Japanese and Chinese guidelines evaluate liquefaction resistance from lab tests (JRA, 2002 and Tamura, 2014; Xie, 2011 respectively). All other methods follow classical stress-based approaches that are semi-empirical in origin. Effects of magnitude and effective stress are considered in all guidelines. Stress-based approaches recommended in the guidelines include Youd et al. (2001) SPT- and CPT-based methods (USACE and British Columbia guidelines), Cetin et al. (2004) and Moss et al. (2006) SPT and CPT-based methods, respectively (CA-DWR), and Idriss and Boulanger (2008, 2014) (New Zealand). These methods include procedures for assessing liquefaction resistance (based on penetration resistance and fines content) and stress demands, the latter using stress reduction factors (r_d) derived from 1D ground response simulations. USACE (2014) also allow stress demands to be evaluated from distilled results of numerical simulations for typical 2D levee geometries (Athanasopoulos-Zekkos and Seed, 2013). The use of advanced numerical analysis is allowed as an option to stress-based models in the Italian, New Zealand, and global guidelines.

Most triggering procedures represent seismic demand in terms of both PGA (converted to cyclic stress ratio) and earthquake magnitude (**M**). PGA is taken from the site-adjusted hazard as described previously. Magnitude is also included to approximately capture the effects of shaking duration (or number of cycles). The selection of an appropriate **M** to pair with PGA is variable in the reviewed guidelines documents, as follows:

- **M** values are used in procedures that include magnitude scaling factors, but the guidelines provide no recommendations on its selection (Italy and global).
- Specific **M** values are recommended for the region to which the guidelines apply (e.g., **M** 6.5 in the CA Delta region) (British Columbia, CA-DWR, New Zealand). A variation on this approach is to not specify an **M** value, but instead to specify a resistance modification factor that is **M**-sensitive (Japan and China). The Japanese guidelines provide these modification factors for two regions, which are broadly controlled by different earthquake types (subduction and shallow crustal), while the Chinese modification factors are provided for various mapped seismic districts.
- Disaggregation of PSHA results is used to select one or more **M** values representative of earthquake contributions from faults of different sizes (USACE). When multiple sources having distinct magnitudes contribute significantly to the hazard, USACE (2014) allows for averaging magnitudes to form a single event or repeating analyses

with each respective magnitude and combining the results in a manner that is not specified.

Recommendations for Ground Failure and Strength Loss

We recommend that ground failure assessment for levees consider all viable mechanisms, including liquefaction (details below), cyclic softening, seismic compression, permanent shear deformations/landsliding (next section), and post-cyclic accelerated volume change in peat.

With regard to liquefaction risk assessment for application to levees, we recommend using procedures with distinct stages of analysis related to susceptibility, triggering, and effects. Susceptibility guidelines should consider both soil type and hydrogeological conditions. The objective of a soil type assessment should be to differentiate granular from clay-like soils. While this differentiation can be made on a crude basis using index tests such as PI and gradation, such approaches carry large epistemic uncertainty. For critical levee sections, we recommend that soil behavior be evaluated using laboratory tests of mechanical behavior (e.g., cyclic stress-strain response, and/or consolidation tests combined with undrained monotonic shear strength tests to evaluate possible strength normalization), which more directly differentiates sand-like from clay-like soil behavior (e.g., Idriss and Boulanger, 2008).

The key consideration with regard to the impact of hydrogeological conditions on susceptibility is groundwater elevation relative to levee base (D_w). Experience has shown that when D_w is < -1 m (ground water elevation more than 1 m below levee base elevation), levees are unlikely to be vulnerable to the instabilities caused by liquefaction (Kwak et al. 2016a).

For granular soils with shallow groundwater, the potential for liquefaction triggering should be based on suitable semi-empirical models that account for the effects of $\mathbf{M}$, effective stress, and static shear stress. The most critical liquefaction effects for levees will typically be strength loss and post-liquefaction settlement. We recommend both be considered, especially when freeboard loss is potentially critical (frequently loaded levees). Research on both liquefaction triggering and its effects is rapidly evolving; for this reason, we recommend that guidelines not recommend specific methods, but instead encourage use of contemporary procedures that are reasonably well accepted by the informed technical community.

For granular soils where liquefaction triggering is predicted, shear strength can be taken as the post-liquefaction undrained strength (S_{u-liq}); we recommend the use of empirical models that consider the effect of effective overburden pressure on the normalized strength S_{u-liq}/σ'_{v0} .

(where σ_{v0}' is pre-earthquake vertical effective stress) (e.g., Kramer and Wang, 2015). When liquefaction is not triggered, some strength reduction is still possible due to pore pressure generation that does not reach σ_{v0}' . This can be approximately accounted for using models for estimation of peak shear strain γ_{max} (Cetin et al., 2009) combined with models for pore pressure ratio given γ_{max} (Cetin and Bilge, 2012). For clayey soils where cyclic softening is predicted, the monotonic undrained strength is usually reduced by about 20% (Boulanger and Idriss, 2007).

POST-EARTHQUAKE INSTABILITY AND DEFORMATION ANALYSIS

Procedures for Post-Earthquake Instability and Deformation Analysis from Guidelines

Flow failure represents a condition whereby static shear stresses in a levee or its foundation exceed the post-liquefaction shear strengths, rendering the structure statically unstable. The USACE, CA-DWR, and New Zealand guidelines require analysis of flow slide potential for sites where liquefaction is triggered. A post-seismic static limit equilibrium analysis is performed using undrained shear strengths (S_{u-liq}) in layers where liquefaction is triggered. Flow failure is predicted if the factor of safety falls below specified values of 1.0 (USACE, CA-DWR) or 1.2 (New Zealand). A conceptually similar fluid dynamics approach is used in Japan whereby liquefied soil is represented by a fluid viscosity. The Chinese, Italian, and global guidelines do not address flow failure.

If a site is not subject to flow failure, the potential for slope deformations caused by slide mass inertia that develops during earthquake shaking is evaluated using four broad methods of analysis:

1. Most guidelines recommend pseudo-static methods in which the de-stabilizing effects of inertial forces above a slip surface are represented by a seismic coefficient k that is taken as $f_{eq} \times \text{PGA}/g$, where f_{eq} is a dimensionless factor that is either unspecified (China, New Zealand, USACE) or varies between 0.2 to 1.0 (most typical value is ~ 0.5). Pseudo-static methods are not used when liquefaction is predicted with the exception of British Columbia and CA-DWR, which recommend to use S_{u-liq} in liquefied soils for such analysis. When liquefaction is not triggered but pore pressures are generated, the Italy, USACE, CA-DWR, and British Columbia guidelines recommend to reduce shear strengths for seismic stability evaluations. Although this method does not compute slope displacements,

deformations are assumed as small if the analysis returns a sufficiently high factor of safety. These limiting factors of safety are not always specified (China, Italy, Japan, global), but where provided range from 1.0 to 1.3.

2. For sites where liquefaction is expected and levee heights are less than 4.6 m (15 ft), empirical or semi-empirical lateral spreading models are used to estimate lateral spread displacements (Youd et al., 2002; Zhang et al., 2004; Faris et al., 2006; Kramer, 2008; Bardet et al., 2002; Rauch, 1997; Rauch and Martin, 2000) in the USACE (2014) guidelines.
3. Six of the eight guidelines recommend the use of Newmark sliding block procedures either through integration or simplified chart-based solutions. CA-DWR require such analysis when the post-seismic safety factor is between 1.0-1.3, and the pseudo-static safety factor (Method 1) ≤ 1 . Yield accelerations are evaluated using limit equilibrium procedures and shear strengths similar to those for pseudo-static. The effects of flexibility of the slide mass on the inertial seismic demand are not addressed in the guidelines when the analyses are applied as formal integrations. Chart-based solutions to account for effects of slide mass flexibility are provided in the global, CA-DWR, and USACE documents. Limiting (maximum tolerable) displacements are provided in the CA-DWR and British Columbia documents, but otherwise are not provided. The maximum tolerable displacement for the CA-DWR guidelines (for frequently loaded levees) is 1 m. The British Columbia document recommends limiting displacement ranging from 0.03-0.9 m for $T_E = 100$ -475 years, respectively.
4. All guidelines allow the use of numerical analysis procedures to assess levee response and deformations. No specific constitutive models or numerical platforms are recommended (exception is the Japan guidelines, which recommends the Iai, 1988 model, or the fluid dynamics model; Towhata et al., 1999). In general, no requirements for validation of numerical procedures is provided in the documents.

Recommendations for Post-Earthquake Instability and Deformation Analysis

When severe strength loss from liquefaction is predicted in levee or foundation soils, we recommend that guidelines initially direct engineers to a static limit equilibrium analysis using S_{u-liq} in soils where liquefaction is anticipated. Significant freeboard loss can be anticipated if flow failure were to occur.

When flow failure is not anticipated, we recommend that slope stability and deformation calculations be sequenced such that the engineer begins with pseudo-static methods for screening purposes. Conservative values of f_{eq} should be provided that are calibrated for the magnitude of earthquakes expected in the region and a tolerable level of displacement (details in Stewart et al., 2003, as applied in slope stability guidelines by Blake et al., 2002). If application of the resulting seismic coefficients exceeds a threshold safety factor (we recommend unity), then no further analysis is required. In these analyses, tension cracks should be taken into account for cohesive levee fills, and depth of cracking should be computed as $z_{cr} = 2c'/\gamma\sqrt{K_a}$ for c- ϕ material, and as $z_{cr} = 2S_u/\gamma$ for undrained conditions (e.g., Duncan et al., 2014). Heavy rainfall events are unlikely to happen during an earthquake. As a result, we do not recommend including the destabilizing effects of water in tension cracks.

When a site fails the screening analysis, we recommend that the next analysis be of a Newmark type, either through integration or through the use of chart-based solutions. Other documents provide detailed recommendations in this regard (USACE, 2014; CA-DWR, 2015; CIRIA, 2013; Blake et al., 2002). Because levees are two-dimensional structures, response-displacement models based on a 1-D geometry (e.g., Bray and Travarasrou, 2007) may not be reliable. Site-specific analyses of seismic demands within the 2-D slide mass (e.g., using Quad4m or similar, Hudson et al., 1994) may be more reliable for such cases. More advanced numerical methods can also be performed, but only constitutive models and numerical codes for which the results are well validated in the open literature should be used. Guidance on the selection of suitable ground motions for use in such analyses are provided by Athanasopoulos-Zekkos and Saadi (2012), Stewart et al. (2014), and Athanasopoulos-Zekkos et al. (2016).

RISK MITIGATION PLANNING AND POST-EARTHQUAKE RESPONSE

Procedures for Risk Mitigation Planning and Post-Earthquake Response from Guidelines

The results of calculations performed in the preceding sections will fall into two general categories for a levee section. A favorable outcome is that the crest deformation for the target hazard level (return period T_E) is sufficiently small that the levee provides protection against flooding given the target water level (for return period T_{WS}). An unfavorable outcome is that this level of flood protection is not provided due to excessive freeboard loss. The question addressed here is what guidance or requirements accompany outcomes of these types? We then discuss related questions of post-earthquake response regarding levee damage and repair.

While some guidelines documents do not provide suggestions for the interpretation of assessed risk for existing levees (Japan, China, and global), others provide two general options:

1. No pre-event remediation is required, but measures should be taken to facilitate rapid post-earthquake repair. This may include establishing contracts with earthwork contractors and stockpiling needed materials. This approach is used for intermittently-loaded levees by USACE and CA-DWR.
2. Timely mitigation of the ground deformation hazard is required so as to achieve target seismic risk levels. This approach is used by British Columbia, New Zealand, Italy, and by USACE and CA-DWR for continuously loaded levees.

Post-earthquake response guidelines (USACE, CA-DWR, global, New Zealand, British Columbia) begin with careful levee inspection, looking for loss of freeboard, landsliding, and ground cracking. Damaged levee sections are subject to interim repair, which may be followed by an engineering analysis and design process, and concluded with further levee modification and repair. In the USACE, CA-DWR, and global documents, the interim repair should occur within 6-8 weeks and restore a 10-year flood protection level. The longer-term modification should occur within one year and restore a 200-year level of flood protection. The New Zealand and British Columbia guidelines are not specific regarding the timelines for repairs and the target levels of flood protection. Japan does not have a single specific guideline addressing post-earthquake response. Each MLIT branch has an internal, unpublished guideline specifically developed for the area of application.

Recommendations for Risk Mitigation Planning and Post-Earthquake Response

Both options 1 and 2 above for responding to levee risk assessments are sensible, and reflect the importance of whether levees are intermittently or continuously loaded. We concur with the general approach of USACE and CA-DWR in this regard. We also agree with post-earthquake response guidelines in those documents, with the provision that repair times should be selected locally based on local risk tolerance.

EXAMPLE LEVEE SECTION ANALYSIS

In this section, we illustrate steps (iii) – (v) (hazard characterization to deformation analysis) of the proposed assessment process using a hypothetical levee section in an active crustal region. As shown in Figure 3a, the target levee site is adjacent to two active faults with the attributes shown. Ground motions are analyzed using the Boore et al. (2014) model.

Step (iii): Hazard Analysis and Site Response

Step (iii) comprises ground motion hazard analysis for a reference site condition and modification for site response. Figure 3b shows the PGA hazard curve for a reference site condition of $V_{S30} = 760$ m/s. For return periods of $T_E = 100$ -2475 years considered in the various guidelines in Table 1, reference site PGA ranges from 0.17-0.56 g. The deterministic median and 84th percentile ground motions (for full rupture of the smaller nearby fault) are 0.29 g and 0.53 g, respectively.

The site condition is shown in Figure 4a ($V_{S30} = 215$ m/s). Figure 4b shows PGA amplification from an ergodic, V_{S30} -based, model (Seyhan and Stewart, 2014) and from site-specific 1-D ground response analyses performed in Deepsoil Ver. 6.1 (Hashash et al., 2016) and interpreted according to the protocols of Stewart et al. (2014).

Figure 3. (a) Location of hypothetical levee site adjacent to two active faults; (b) PGA hazard curves for the example site for reference site conditions ($V_{S30}^{ref} = 760$ m/s) and for foundation soil conditions ($V_{S30} = 215$ m/s). Site amplification computed using ergodic model and implemented in hybrid manner (H-E-G) and from site-specific analysis as implemented in the PSHA (P-S-G). Notation defined in footnote (4) from Table 1.

Figure 4. (a) Soil stratigraphy and corresponding shear wave velocity (V_s) profile for example site; (b) Site amplification from ergodic, V_{S30} -based model, and from site-specific amplification model derived from 1-D ground response analysis

Figure 3b shows the impact of implementing the site-specific amplification (from Figure 4b) within the hazard calculations (preserving the hazard level) and in a hybrid manner. These calculations were performed in OpenQuake (Pagani et al., 2014). The probabilistic and non-ergodic result (within-PSHA implementation of site specific amplification; P-S-G) is beyond current practice but is shown here to illustrate the bias associated with currently utilized approximate procedures. As shown in Figure 3b, the difference between the hybrid (H-E-G) and probabilistic (P-S-G) ground motions is approximately 50% at the 475 and 2475-year return periods. The rapid drop off of the hybrid hazard beyond 0.4 g is a characteristic feature driven mainly by too-strong nonlinear site response in the hybrid hazard calculation (Goulet and Stewart 2009).

PGA disaggregation results for the example site at the 475-year return period (Figure 5) illustrate distinct contributions of the two sources (shown in Figure 3) in magnitude-distance-epsilon space. The weighted mean magnitude and distance are $\bar{M}=6.2$ and $\bar{R}_{JB}=15$ km, which do not correspond to an actual contributing event in this case. One approach for combining events for liquefaction analysis (Step iv) is to sample the full $\mathbf{M}-R_{JB}$ joint distribution (i.e., all bars in the disaggregation) along with their relative contributions to liquefaction triggering (Kramer, 2008). This approach has been referred to as *multiple-scenario* by Kramer (2008). An approximation of this approach, inspired by Bazzurro and Cornell (1999), is to represent

the $\mathbf{M}$ - R_{JB} joint distribution with a small number of modes representing distinct faults (two in the example from Figure 5), compute liquefaction effects for each mode, and combine the effects in a weighted manner (referred to here as *modal weighted average*). The bivariate modal $\mathbf{M}$ - R_{JB} pairs (i.e., peaks of the $\mathbf{M}$ - R_{JB} joint distributions) in Figure 5 are $\mathbf{M}_A^*=5.75$ and $R_A^*=4.5$ km (Fault A) and $\mathbf{M}_B^*=7.75$ and $R_B^*=33.5$ km (Fault B). The cumulative weight of each source to reference site PGA hazard is 65% and 35% for Faults A and B, respectively.

Figure 5. Disaggregation of PGA hazard at the $T_E = 475$ -year return period for example site in Figure 3. Distance is closest distance to surface projection (R_{JB}), epsilon indicates the number of standard deviations above the natural log mean for the ground motion contribution.

Step (iv): Liquefaction, Cyclic Softening, and Strength Loss

Figure 6 shows the geotechnical conditions of the example case being analyzed, where $(N_1)_{60}$ indicates energy- and over-burden corrected standard penetration test blow counts; S_u indicates undrained shear strengths corresponding to typical rates of shear in laboratory tests. The S_u values (Figure 6c) used for the site are taken from normalized relationships by Ladd and Foote (1974) and reflect over consolidation ratios (OCRs) ranging from 1.5-4 in the fill and 2.4 in the natural clay.

The clayey soils are taken as non-susceptible to liquefaction, but are evaluated for cyclic softening (as shown below). The sandy soils below 7 m depth are taken as liquefaction susceptible. We apply the Boulanger and Idriss (2012) procedures for liquefaction triggering, with results shown in Figure 6d for alternate $\mathbf{M}$ specifications (weighted mean, Fault A and B modal scenarios, multiple-scenario, and modal weighted average). These results are obtained

using the deterministic version (i.e. 15th percentile curve) of the Boulanger and Idriss (2012) SPT-based procedure. Liquefaction is clearly triggered in the sand over the depth interval 7 to 17 m for the Fault B scenario and is for practical purposes not triggered for the Fault A scenario. The two methods for combining events produce intermediate results that are similar to each other and barely trigger liquefaction. Post-liquefaction undrained shear strengths (S_{u-liq}) are shown in Figure 6c using procedures from the current guidelines (Table 1).

Cyclic softening analysis of the natural clay layer (using procedures in Boulanger and Idriss, 2007) provides factors of safety less than one, indicating that strength loss of about 20% relative to values corresponding to typical rates of shear in monotonic laboratory tests (S_u) can be expected (reduced strength values are indicated as S_u (softened) in Figure 6c). This analysis considers the horizontal static shear stress in the clay layer, which has an average value over the levee half width corresponding to $\alpha = \tau_{hv} / \sigma'_{v0} \approx 0.05$.

Figure 6. (a) Levee geometry and stratigraphy; (b) Energy- and overburden-corrected SPT blow-count profile, $(N_1)_{60}$, (c) Undrained shear strength (S_u) profiles with and without softening for the fine-grained materials and portions of sand layer potentially subject to liquefaction at the example ground motion level (S_{u-liq}) (IB08 = Idriss and Boulanger, 2008; OS02 = Olson and Stark, 2002, and (d) Depth variation of factor of safety (FS) against liquefaction triggering from disaggregated mean event, individual fault scenarios, and various combinations of scenarios.

Step (v): Post-Earthquake Instability and Deformation Analysis

Liquefaction of the foundation sand occurs for the Fault B scenario, but not for Fault A. Given these different outcomes, we separately evaluate levee stability and deformations for both scenarios. For the Fault B scenario (liquefied foundation), the USACE, CA-DWR, and New Zealand guidelines require analysis of flow slide potential (i.e. post-seismic static limit equilibrium analysis), which is done using the S_{u-liq} values from Figure 6c. We also consider softened shear strengths in the clay layers. For these conditions, the factor of safety is computed

as 1.1, with the failure surface geometry shown in Figure 7, indicating marginal stability against flow slide. Under such conditions, it is possible that the levee would lose functionality.

For the Fault A scenario (no liquefaction), we first apply the pseudo-static method with the 475-year PGA of 0.34g. Shear strengths in the clay layers are taken from the softened S_u values in Figure 6c. For sands, a drained friction angle of 35-40° is used, but effective stresses are assumed to be reduced in the upper 10 m of the sand layer by pore pressure generation (we apply a pore pressure ratio that ranges from $r_u \approx 0.5$ to zero over the upper 10 m, following the approach suggested by Cetin et al., 2009, based on estimated maximum shear strains of 2.5-0.75%; Cetin and Bilge, 2012). With these strength values, the pseudo-static safety factors range from 1.2 ($f_{eq} = 0.2$) to 0.9 ($f_{eq} = 0.5$). Given these results, the CA-DWR procedures then require Newmark analysis (the British Columbia, New Zealand, Italian, USACE and global guidelines do not link Newmark to pseudo-static results). The yield acceleration is $k_y = 0.13$ for a failure surface similar that for static analysis (Figure 7). The peak demand within the slide mass (i.e., seismic coefficient k_{max}) ranges from $0.3-0.8 \times \text{PGA/g}$ using the chart based solutions from the global, CA-DWR, and USACE guidelines. The resulting displacements (also from chart-based solutions) range from 0-0.2 m, and the associated freeboard loss is likely in the range of 0-0.1 m based on the geometry of the failure surface.

It is clear from the two sets of results that the Fault B scenario is more damaging to the levee than is the Fault A scenario, despite the similar ground motion levels. This example highlights the importance of linking hazard results to realistic scenario earthquakes in consideration of hazard disaggregation.

Figure 7. Example levee geometry showing critical failure surface geometry and tension crack depth.

LEVEE SYSTEM ANALYSIS

Levee system risk analysis involves calculation of the probability of failure relative to levees' flood control function and assessment of the consequences. Consideration of the distributions of demand and capacity, and the spatial correlation of demand and of capacity are crucial aspects of system analysis. Terminology applied to levee systems is often somewhat vague, and we herein suggest definitions that have specific meaning in the context of system risk assessment and that enable a clear presentation of existing recommendations. First, a *levee system* provides flood control for a specific region, and often consists of many kilometers of levee. Because levee systems are long, they are often divided into *reaches* that may be a few hundred meters to a few kilometers in length. A *reach* (or *section*, VNK, 2015; Jongejan and Maaskant, 2015) is defined herein as a continuous length of levee of homogeneous construction, cross-section geometry, statistical distribution of geotechnical properties, hydrological demand, consequences of failure, and potentially other parameters relevant to risk assessment. Although *reaches* exhibit uniform statistical distributions of geotechnical properties, the properties themselves may vary spatially within the reach. A *segment* is defined herein as a short length of levee for which the capacity and demand can essentially be considered uniform.

None of the guidelines documents from Table 1 address the analysis of system risk, although the subject is considered in the Delta Risk Management Strategy (DRMS) document in California (URS Corporation, Jack R. Benjamin & Associates Inc.; DRMS, 2008), a USACE guidelines document for I-walls (structural walls that extend the height of levees) (USACE, 2011), and in guidelines for flood risk in the Netherlands (currently under development, based on methods used in Veiligheid Nederland in Kaart; VNK, 2015). System analysis at its most fundamental level is performed on segments, for which the capacity and demand between segments are both spatially correlated. The analysis procedures presented in previous sections of this manuscript apply at the segment level, and the system analysis begins with the application of these procedures to evaluate the performance of individual segments within the system in the form of safety factors (for pseudo-static methods) or displacements. An important detail in these calculations is that the seismic demands should be assessed for scenario events that are compatible with the probabilistic hazard (details in Jarayam and Baker, 2009) rather than for the uniform hazard spectrum. Scenario events are required because a single earthquake is unlikely to produce ground motions throughout the system that are consistent with uniform

hazard spectrum values. This segment-level performance must then be converted to failure probabilities, which is subject to epistemic uncertainty for the present problem of levee geometries due to lack of prior work on the subject, but which has been investigated in a more general sense for static problems (e.g. Christian et al., 1994, Baecher and Christian, 2003), pseudo-static slope stability analyses (e.g. Luzi et al., 2000), and Newmark-type displacements as applied to landslide on natural slopes (Jibson et al., 2000).

The system-level analysis then aggregates the segment failure probabilities into a system failure probability with due consideration given to the spatial correlation of capacity and of demand. Kwak et al. (2016b) presented a framework for performing such analyses using Monte Carlo simulation involving randomly generated spatially correlated random variables. Generating a large number of correlated random variables can be computationally expensive. An alternative is to compute the probability of failure within a reach, and to subsequently aggregate the system failure probability based on the failure probabilities of the reaches composing the system. Such an approach was adopted for analysis of levee system failure due to flood risk in the Netherlands (VNK, 2015; Jongejan and Maaskant, 2015; Vrouwenvelder, 2006). The probability of failure of each reach is computed based on consideration of the distribution of all capacity and demand variables and their autocorrelation functions within the reach. Furthermore, capacity and demand variables may be spatially correlated among reaches, which must be taken into consideration in the calculation of system reliability (Zimmaro et al., 2017). DRMS (2008) adopted a *reach*-based reliability approach in which fragility functions were assigned to reaches (rather than segments) based on a combination of engineering analysis and judgment. The reaches were then assumed to be statistically independent in the system analysis. This approach is less rigorous than the Dutch approach because it fails to account for spatial correlation of demand and capacity within each reach in the fragility analysis, and among the reaches in the system analysis.

Another approach to computing the probability of system failure is to divide the levee system into *characteristic lengths* (USACE, 2011). A characteristic length is defined herein as a specific length of levee for which the probability of system failure computed by assuming successive lengths are statistically independent is identical to the probability of system failure computed in a more robust manner involving explicit consideration of correlation. The system analysis amounts simply to the calculation of the complement of the product sum of probability of survival for each characteristic length, and is therefore simpler than the approaches that

explicitly consider correlation. A problem with this approach is that the characteristic length can only be strictly defined by first performing a more robust system analysis to compute the probability of failure, and subsequently solving for the characteristic length. In practice, however, a specific value for characteristic length is often assumed from the outset to facilitate a rapid system analysis (e.g., USACE recommends a maximum of 300 m for flood risk analysis). Note that the characteristic length is strongly dependent on the spatial distribution and correlation of capacity and demand, and therefore may vary for different hazards. For example, earthquake ground motion tends to be more spatially variable than water level elevations following weather events, which would result in shorter characteristic lengths for seismic hazard than for flood hazard.

ENFORCEMENT

The guidelines documents in Table 1 have varying levels of enforcement. We have found that regulation and enforcement levels are generally compatible with one of the following:

Rigorous: The guideline carries the weight of law and applies both to existing (levee construction precedes guidelines publication) and new (proposed) levees.

Intermediate: The guideline carries the weight of law but applies only to new levees or levees proposed for significant modification (changes of side slopes or height). Existing levees not scheduled for modification are unaffected.

Optional: The guideline is not a legal document. Rather, it is an optional resource that can be considered by engineers at their discretion.

The rigorous enforcement category is only applied under specific conditions in California (details below). Guidelines falling into the intermediate category include Japan, Italy, China, British Columbia, and USACE. The global and New Zealand guidelines are optional.

Local considerations can significantly impact the application of the guidelines. For example, in California, the CA-DWR guidelines are applicable state-wide, but can be superseded by local regulations outside of the Sacramento-San Joaquin Valley. Within that region, their use is mandatory for new levees and existing levees that provide flood protection in flood zones proposed for development. The USACE guidelines apply only to USACE-owned existing levees. While the New Zealand guidelines are optional, upgrades to existing levees or design of new levees in recent years typically follow the NZSOLD Dam Safety Guidelines (2015) document.

In addition to the selected documents, we gathered information for other countries. Chile lacks regulation for seismic levee design. The Netherlands has particularly strong regulations for levees that have been designated primary flood defenses (Ministry of transport works and water management – Water act, 2009), but those regulations do not provide seismic provisions as of now.

CONCLUSIONS

In this paper, we present the results of an extensive review of seismic levee guidelines from six countries with critical levee infrastructure located in regions of high seismicity. Many of these guidelines are in grey literature; accessing and interpreting the documents required the collaborative effort of the large author team and many other individuals (listed in acknowledgements). Technical recommendations within the guidelines take many forms, but following extensive deliberation and discussion with expert panels, we have suggested the structure followed in this paper, with the principle elements consisting of (i) procedures for screening out levees with low risk not requiring detailed analysis; (ii) site exploration methods and spacing; (iii) seismic and accompanying co-seismic water level hazard characterization; (iv) assessment of potential for strength loss in levee or foundation soils; (v) evaluation of flow slide and/or deformation potential; and (vi) risk mitigation planning and post-earthquake response.

Some elements of this procedure are broadly similar across the eight considered guidelines documents, such as the use of relatively low return periods for water level hazard to accompany much longer return periods for seismic hazard. Others are quite variable, such as the consideration (or not) of flow slide potential when liquefaction triggering is predicted. Our motivation for this work was in part to identify these similarities and differences, so as to facilitate identification of best practices. With that mind, we have provided recommendations for the six elements in the systematic procedure, which should be useful in the development of future guidelines for levees and other infrastructure comprised of earth structures.

ACKNOWLEDGEMENTS

This work was supported by California Department of Water and Resource (CA-DWR) under contract number 4600008849. This funding source is gratefully acknowledged. Any opinions, findings, and conclusions or recommendations expressed in this material are those of the authors and do not necessarily reflect those of the CA-DWR and FOREM. We thank

Grant Lovell at Tonkin and Taylor for providing useful information on New Zealand guidelines. We also thank Derek Wilshire, Secretary of the New Zealand society of large dams (NZSOLD), for providing access to the New Zealand dam safety guidelines (2015). We thank Ramon Verdugo at Universidad de Chile, and Victor A. Contreras Luarte at UCLA for providing information on Chile procedures. We also thank Steve Mahnke at CA-DWR, Leslie F. Harder, Jr. at HDR Inc., and Vlad G. Perlea and George Hu at the U.S. Army Corps of Engineers (USACE) for their useful suggestions as part of an expert review panel for this work.

REFERENCES

- Athanasopoulos-Zekkos, A., Pence, H., and Lobbetael, A., 2016. Ground Motion Selection for Seismic Slope Displacement Evaluation of Earthen Levees, *Earthq. Spectra*, **32**, 217-237.
- Athanasopoulos-Zekkos, A. and Seed, R.B., 2013. Simplified Methodology for Consideration of 2D Dynamic Response of Levees in Liquefaction Triggering Evaluation, *J. Geotech. Geoenviron. Eng.*, **139**, 1911-1922.
- Athanasopoulos-Zekkos and M. Saadi, 2012. Ground Motion Selection for Liquefaction Evaluation Analysis of Earthen Levees, *Earthq. Spectra*, **28**, 1331-1351.
- Baecher, G. B., and Christian, J. T., 2003. *Reliability and statistics in geotechnical engineering*, John Wiley, Chichester, England.
- Bardet, J. P., Tobita, T., Mace, N., and Hu, J. 2002. Regional modeling of liquefaction-induced ground deformation, *Earthq. Spectra*, **18**, 19-46.
- Bazzurro, P., and Cornell, C. A. 1999. Disaggregation of seismic hazard, *Bull Seismol Soc Am*, **89**, 501-520.
- Blake, T. F., Hollingsworth, R. A., and Stewart, J. P., editors, 2002. Recommended procedures for implementation of DMG Special Publication 117 Guidelines for analyzing and mitigating landslide hazards in California, *Southern California Earthquake Center*, Los Angeles, CA.
- Boore, D. M., Stewart, J. P., Seyhan, E., and Atkinson, G. M., 2014. NGA-West 2 equations for predicting PGA, PGV, and 5%-damped PSA for shallow crustal earthquakes, *Earthq. Spectra*, **30**, 1057-1085.
- Boulanger, R. W., and Idriss, I. M., 2007. Evaluation of cyclic softening in silts and clays, *J. Geotech. Geoenviron. Eng.*, **133**, 641-652.
- Boulanger, R. W., and Idriss, I. M., 2012. Probabilistic SPT-based liquefaction triggering procedure, *J. Geotech. Geoenviron. Eng.*, **138**, 1185-1195.

- Bray, J. D., and Travasarou, T., 2007. Simplified procedure for estimating earthquake-induced deviatoric slope displacements, *J. Geotech. Geoenviron. Eng.*, **133**, 381-392.
- Building Seismic Safety Council (BSSC), 2015. *NEHRP Recommended Seismic Provisions for New Buildings and Other Structures*, Federal Emergency Management Agency, Washington D.C.
- Cetin, K. O., and Bilge, H. T., 2012. Cyclic large strain and induced pore pressure models for saturated clean sands, *J. Geotech. Geoenviron. Eng.*, **138**, 309-323.
- Cetin, K. O., Bilge, H. T., Wu, J., Kammerer, A. M., and Seed, R. B., 2009. Probabilistic models for cyclic straining of saturated clean sands, *J. Geotech. Geoenviron. Eng.*, **135**, 371-386.
- Cetin, K. O., Seed, R. B., Der Kiureghian, A., Tokimatsu, K., Harder, L. F., Jr., Kayen, R. E., and Moss, R. E., 2004. Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential, *J. Geotech. Geoenviron. Eng.*, **130**, 1314-1340.
- Christian, J. T., Ladd, C. C., and Baecher, G. B., 1994. Reliability applied to slope stability analysis. *J. Geotech. Eng.*, **120**, 2180-2207.
- CIRIA, 2013. *The International Levee Handbook*, CIRIA, US Army Corp of Engineers, French Committee for Dams and Hydraulic Works, CIRIA C731.
- Cramer, C. H., 2003. Site-specific seismic-hazard analysis that is completely probabilistic, *Bull. Seismol. Soc. Am.*, **93**, 1841-1846.
- Delta Risk Management Strategy (DRMS), 2008. Levee vulnerability, Tech. Memo. Delta Risk Manag. Strateg. Final Phase 1, Prepared by URS Corporation, Jack R. Benjamin & Associates Inc., Oakland.
- Department of water resources of the State of California (CA-DWR), 2012. *Urban Levee Design Criteria*, May 2012.
- Department of water resources of the State of California (CA-DWR), 2015. *Guidance document for geotechnical analyses*. Prepared for by URS Corporation California Dept. of Water Resources, Sacramento, CA.
- Duku, P. M., Stewart, J. P., Whang, D. H., and Yee, E., 2008. Volumetric strains of clean sands subject to cyclic loads, *J. Geotech. Geoenviron. Eng.*, **134**, 1073-1085.
- Duncan, J.M., Wright, S.G., and Brandon, T.L., 2014. *Soil strength and slope stability*, 2nd Edition, Hoboken, New Jersey, John Wiley & Sons.
- Faris, A. T., Seed, R. B., Kayen, R. E., and Wu J., 2006. A semi-empirical model for the estimation of maximum horizontal displacement due to liquefaction-induced lateral spreading. *Proc. 8th US Nat. Conf. Earthq. Eng.*, April 18-22, San Francisco, CA.

- General Administration of Quality Supervision, Inspection and Quarantine of China, 2015. *Seismic ground motion parameters zonation map of China (GB18306-2015)*. China Standards Press. (in Chinese).
- Gordon, A. F., and Griffiths, D. V., 2008. *Risk assessment in geotechnical engineering*. Wiley.
- Goulet, C. A., and Stewart, J. P., 2009. Pitfalls of deterministic application of nonlinear site factors in probabilistic assessment of ground motions, *Earthq. Spectra*, **25**, 541–555.
- Green, R. A., Allen, J., Wotherspoon, L., Cubrinovski, M., Bradley, B., Bradshaw, A., Cox, B., and Algie, T., 2011. Performance of Levees (Stopbanks) during the 4 September 2010 M_w 7.1 Darfield and 22 February 2011 M_w 6.2 Christchurch, New Zealand, Earthquakes, *Seismol. Res. Lett.*, **82**, 939-949.
- Halchuk, S., Allen, T. I., Adams, J., and Rogers G. C., 2014. Fifth Generation Seismic Hazard Model Input Files as Proposed to Produce Values for the 2015 National Building Code of Canada, *Open File 7576*, Geological Survey of Canada.
- Hashash, Y. M. A., Musgrove M. I., Harmon, J. A., Groholski, D. R., Phillips C. A., and Park, D., 2016. *DEEPSOIL 6.1, User Manual*, University of Illinois at Urbana-Champaign, Urbana, IL.
- Hudson, M. B., Beikae, M., and Idriss, I. M., 1994. QUAD4M, a computer program to evaluate the seismic response of soil structures using finite element procedures and incorporating a compliant base, Center for Geotechnical Modeling, Dept. Civil and Environ. Eng., UC Davis.
- Iai, S., 1988. A model that does not break out in the two-dimensional effective stress analysis for liquefaction, *Proc. 43rd Annual Conf. of JSCE* (in Japanese).
- Idriss, I. M., and Boulanger, R. W., 2008. *Soil liquefaction during earthquakes*, Earthquake Engineering Research Institute (EERI), MNO-12. Oakland, CA.
- Idriss, I. M., and Boulanger, R. W., 2014. CPT and SPT based liquefaction triggering procedures, *Report No. UCD/CGM-14/01*, Dept. Civil & Environ. Eng., UC Davis.
- Japan Institute of Country-ology and Engineering (JICE), 2007. *Guidance of maximum water level calculation at normal time for inspection of seismic performance for river structures*, JICE (in Japanese).
- Japan Road Association (JRA), 2002. *Specifications for highway bridges, Part V: Seismic design*, JRA.
- Jayaram, N., and Baker, J. W., 2009. Correlation model for spatially distributed ground-motion intensities, *Earthq. Eng. Struct. Dyn.*, **38**, 1687–1708.
- Jibson, R. W., Harp, E. L., and Michael, J. A., 2000. A method for producing digital probabilistic seismic landslide hazard maps. *Eng. Geology*, **58**, 271–289.

- Jongejan, R. B., and Maaskant, B., 2015. Quantifying flood risks in the Netherlands, *Risk Analysis*, **35**, 252-264.
- Kramer, S. L., 2008. Evaluation of liquefaction hazards in Washington State, *Final Research Report, Agreement T2695, Task 66 Liquefaction Phase III*. Prepared for Washington State Transportation Commission, Dept. of Transportation, in cooperation with U.S. Dept. Transportation, Federal Highway Administration.
- Kramer, S. L. and Wang, C.-H., 2015. Empirical model for estimation of the residual strength of liquefied soil, *J. Geotech. Geoenviron. Eng.*, **141**, 4015038.
- Kwak, D. Y., Stewart, J. P., Brandenburg, S. J., and Mikami, A., 2016a. Characterization of seismic levee fragility using field performance data, *Earthq. Spectra*, **32**, 193-215.
- Kwak, D. Y., Stewart, J. P., Brandenburg, S. J., and Mikami, A., 2016b. Seismic levee system fragility considering spatial correlation of demands and component fragilities, *Earthq. Spectra*, **32**, 2207-2228.
- Ladd, C. and Foote, R., 1974. A new design procedure for stability of soft clays. *J. Geotech. Eng.*, **100**, 763-786.
- Luzi, L., Pergalani, F., and Terlien M. T. J., 2000. Slope vulnerability to earthquakes at subregional scale, using probabilistic techniques and geographic information systems, *Eng. Geology*, **58**, 313–336.
- McGuire, R. K., 2004. *Seismic hazard and risk analysis*, Earthquake Engineering Research Institute (EERI), MNO-10, Oakland, California.
- Miller, E. A., and Roycroft, G. A., 2004. Seismic performance and deformation of levees: four case studies, *J. Geotech. Geoenviron. Eng.*, **130**, 344–354.
- Ministry of Forests, Lands, and Natural Resource Operations (MFLNRO) - British Columbia, 2014. *Seismic Design Guidelines for Dikes*, Flood Safety Division, 2nd Edition, February 2014.
- Ministry of Land, Infrastructure, Transport and Tourism (MLIT) - Japan, 2012. *Inspection manual of seismic levee performance regarding Level 2 earthquake* (in Japanese).
- Ministry of Land, Infrastructure, Transport and Tourism (MLIT) - Japan, 2007. *Inspection guideline for seismic performance of river structures* (in Japanese).
- Ministry of the Infrastructures - Italy, 2008. *Norme tecniche per le costruzioni*, Decree of the Minister of the Infrastructures, 14 January 2008, Gazzetta Ufficiale della Repubblica Italiana No. 29, Rome (in Italian).
- Ministry of Transport, Public Works and Water Management – The Netherlands, 2009. Water Act.

- Ministry of Water Resources - China, 2005. *Code of geological investigation of levee project (SL188-2005)*. China water power press, Beijing (in Chinese).
- Ministry of Water Resources - China, 2008. *Code for engineering geological investigation of water resources and hydropower (GB50487-2008)*. China planning press, Beijing (in Chinese).
- Ministry of Water Resources - China, 2013. *Code for design of levee project (GB50286-2013)*. China planning press, Beijing (in Chinese).
- Moss, R. E., Seed, R. B., Kayen, R. E., Stewart, J. P., Der Kiureghian, A., and Cetin, K. O., 2006. CPT-based probabilistic and deterministic assessment of in situ seismic soil liquefaction potential, *J. Geotech. Geoenviron. Eng.*, **132**, 1032-1051.
- New Zealand Society on Large Dams (NZSOLD), 2015. *New Zealand dam safety guidelines*.
- Olson, S. M., and Stark, T. D., 2002. Liquefied strength ratio from liquefaction flow failure case histories, *Can. Geotech. J.*, **39**, 629-647.
- Pagani, M., Monelli, D., Weatherill, G. A., Danciu, L., Crowley, H., Silva, V., et al., 2014. OpenQuake engine: an open hazard (and risk) software for the Global earthquake model. *Seismol Res Lett*, **85**, 692-702.
- Petersen, M. D., Moschetti, M. P., Powers, P. M., Mueller, C. S., Haller, K. M., Frankel, A. D., Zeng, Y., Rezaeian, S., Harmsen, S. C., Boyd, O. S., and Field, N., 2015. The 2014 United States national seismic hazard model. *Earthq. Spectra*, **31**, S1-S30.
- Phoon, K. K., and Kulhawy, F. H., 1999. Characterization of geotechnical variability. *Can. Geotech. J.*, **36**, 612-624.
- Rauch, A. F., 1997. EPOLLS: An empirical method for predicting surface displacements due to liquefaction-induced lateral spreading in earthquakes. PhD thesis, Virginia Tech, Blacksburg, VA.
- Rauch, A. F., and Martin, J. R., II, 2000. EPOLLS model for predicting average displacements on lateral spreads. *J. Geotech. Geoenviron. Eng.*, **126**, 360-371.
- Robertson, P. K., and Cabal, K.L., 2015. *Guide to cone penetration testing 6th edition*. Gregg Drilling and Testing Inc., USA.
- Robertson, P. K., and Wride, C. E., 1998. Evaluating cyclic liquefaction potential using the cone penetration test, *Can. Geotech. J.*, **35**, 442-459.
- Sabatini, P. J., Bachus, R. C., Mayne, P. W., Schneider, J. A., and Zettler, T. E., 2002. Geotechnical engineering circular no. 5: evaluation of soil and rock properties (No. FHWA-IF-02-034).

- Sasaki, Y., 2009. River dike failures during the 1993 Kushiro-oki Earthquake and the 2003 Tokachioki Earthquake, *Earthquake Geotechnical Case Histories for Performance-Based Design*, T. Kokusho (editor), 131–157.
- Sasaki, Y., Towhata, I., Miyamoto, K., Shirato M., Narita, A., Sasaki, T., and Sako S., 2012. Reconnaissance report on damage in and around river levees caused by the 2011 off the Pacific coast of Tohoku earthquake, *Soils and Foundations*, **52**, 1016-1032.
- Seed, R. B., and Harder, L. F., Jr., 1990. SPT-based analysis of cyclic pore pressure generation and undrained residual strength, *Proc. H.B. Seed Mem. Sym., Bi-Tech Publishing Ltd.*, Vol. 2, 351-376.
- Seyhan, E., and Stewart, J. P., 2014. Semi-empirical nonlinear site amplification from NGA-West 2 data and simulations, *Earthq. Spectra*, **30**, 1241-1256.
- Shafiee, A., Stewart, J. P., and Brandenberg, S. J., 2015. Reset of secondary compression clock for peat by cyclic straining, *J. Geotech. Geoenviron. Eng.*, **141**, 02815001.
- Stewart, J. P., Afshari, K., and Hashash, Y. M. A., 2014. Guidelines for Performing Hazard-Consistent One-Dimensional Ground Response Analysis for Ground Motion Prediction, *PEER Report No. 2014/16*, Pacific Earthquake Engineering Research Center, UC Berkeley, CA.
- Stewart, J. P., Blake, T. M., and Hollingsworth, R. A., 2003. A screen analysis procedure for seismic slope stability, *Earthq. Spectra*, **19**, 697-712.
- Stewart, J. P., Afshari, K., and Goulet C. A., 2017. Non-ergodic site response in seismic hazard analysis, *Earthq. Spectra*, in press.
- Stirling, M. W., McVerry, G. H., Gerstenberger, M. C., Litchfield, N. J., Van Dissen, R. J., Berryman, K. R., et al., 2012. National seismic hazard model for New Zealand: 2010 update, *Bull. Seism. Soc. Am.*, **102**, 1514-1542.
- Stucchi, M., Meletti, C., Montaldo, V., Crowley, H., Calvi, G. M., and Boschi, E., 2011. Seismic hazard assessment (2003–2009) for the Italian building code, *Bull. Seismol. Soc. Am.*, **101**, 1885–1911.
- Tamura, K., 2014. Seismic design of highway bridge foundations with the effects of liquefaction since the 1995 Kobe earthquake, *Soils and Foundations*, **54**, 874-882.
- U.S. Army Corps of Engineers (USACE), 2000. *Design and construction of levees*, EM 1110-2-1913, USACE Washington, DC.
- U.S. Army Corps of Engineers (USACE), 2011. *Design of I-walls*, Appendix I: Length effects in levee system reliability, Washington DC.
- U.S. Army Corps of Engineers (USACE), 2014. Draft document, *Guidelines for seismic evaluation of levees*, ETL 1110-2-580, USACE Washington, DC.

- Veiligheid Nederland in Kaart 2 (VNK) Project Office, 2008. The national flood risk analysis for the Netherlands, final report. *VNK Project Office Rijkswaterstaat*.
- Wakamatsu, K., and Matsuoka, M., 2013. Nationwide 7.5-Arc -Second Japan Engineering Geomorphologic Classification Map and V_{s30} Zoning, *J. Disaster Research*, **8**, 904-911.
- Wang, W., 1980. Some findings in soil liquefaction, *Chinese J. Geotech. Eng.*, **2**, 55–63. (in Chinese).
- Xie, D., 2011. *Soil Dynamics*, Higher education press, Beijing (in Chinese).
- Yee, E., Duku, P. M., and Stewart, J. P., 2014. Cyclic volumetric strain behavior of sands with fines of low plasticity, *J. Geotech. Geoenviron. Eng.*, **140**, 04013042.
- Youd, T. L., Hansen, C. M., and Bartlett, S. F., 2002. Revised multilinear regression equations for prediction of lateral spread displacement, *J. Geotech. Geoenviron. Eng.*, **128**, 1007–1017.
- Youd, T. L., Idriss, I. M., Andrus, R. D., Arango, I., Castro, G., Christian, J. T., et al., 2001. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils, *J. Geotech. Geoenviron. Eng.*, **127**, 817-833.
- Zhang, G., Robertson, P. K., and Brachman, R. W. I., 2004. Estimating liquefaction-induced lateral displacements using the standard penetration test or cone penetration test, *J. Geotech. Geoenviron. Eng.*, **130**, 861-871.
- Zhao, J. X., Zhang, J., Asano, A., Ohno, Y., Oouchi, T., Takahashi, T., et al., 2006. Attenuation relations of strong ground motion in Japan using site classification based on predominant period. *Bull. Seismol. Soc. Am.*, **96**, 898-913.
- Zimmaro, P., Stewart, J. P., Brandenberg, S. J., Kwak, D. Y., Jongejan, R., 2017. System reliability of flood control levees, PBD-III: 3rd International conference on performance-based design in earthquake geotechnical engineering, Vancouver, Canada, in review.