NAM

Smallest Earthquake Magnitude that Can Trigger Liquefaction

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General Introduction

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This report contains a literature study of field observations, to establish an earthquake magnitude threshold below which the possibility of triggering liquefaction can be discounted.

The study concludes that earthquakes as small as moment magnitude 4.5 can trigger liquefaction in extremely susceptible soil deposits. However, these susceptible soil deposits correspond to site conditions where building construction is not viable. Example of such extremely susceptible soil deposits would be a mud-flat area outside the dikes, a river bed, an impoundment area or a tailings pond. For soil profiles that are sufficiently competent to support foundation loads, the minimum earthquake magnitude for the triggering of liquefaction is about 5. The report therefore proposes that in liquefaction hazard assessments for engineering applications, magnitude 5.0 be adopted as the minimum earthquake size considered.

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Smallest Earthquake Magnitude that Can Trigger Liquefaction

by

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Report of a study performed by the Virginia Tech Center for Geotechnical Practice and Research

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ABSTRACT

Assessment of the potential impact of induced earthquakes on the infrastructure focuses attention on smaller magnitudes than those generally considered in hazard and risk assessments for tectonic seismicity. In this context, it is useful to establish if there is a magnitude threshold below which the possibility of triggering liquefaction can be discounted. Such a lower bound for liquefaction triggering is also required for probabilistic liquefaction hazard analyses. Based on field observations and a simple parametric study, we conclude that earthquakes as small as moment magnitude 4.5 can trigger liquefaction in extremely susceptible soil deposits but these correspond to site conditions where building construction is not viable. For soil profiles that are sufficiently competent to support foundation loads, the minimum earthquake magnitude for the triggering of liquefaction is about 5. We therefore propose that in liquefaction hazard assessments for engineering applications, magnitude 5.0 be adopted as the minimum earthquake size considered.

1. INTRODUCTION

Although an almost negligible contributor to earthquake fatalities, liquefaction triggering is an important threat to the built environment and in particular to infrastructure and lifelines (e.g., Bird and Bommer 2004). The question of the smallest earthquake magnitude that can lead to liquefaction triggering in saturated sand deposits arises because of two factors. One is the growing concerns regarding induced earthquakes resulting from human activities such as hydrocarbon production, hydraulic fracturing and wastewater injection (Davies et al. 2013, Mitchell and Green 2017). Since induced earthquakes often occur in regions of low tectonic seismicity and are viewed as an imposed rather than natural hazard, attention to such events is often focused on magnitudes (in the range from 3 to 5) that are smaller than those generally given attention in conventional earthquake engineering. For small-to-moderate magnitude induced and triggered earthquakes, ground shaking will clearly be of concern both as a source of disturbance to the exposed population as well as a potential cause of damage to buildings that may have been constructed without consideration of seismic loading. A comprehensive induced seismic risk assessment might also consider other earthquake hazards although some of these—such as tsunami and probably also surface rupture—could be easily screened out. Collateral geotechnical hazards are likely to require quantitative evaluation unless, at least for liquefaction, a clearly established minimum magnitude threshold existed, in which case it would be possible to discard this hazard from risk estimations.

Another motivation for exploring the lower magnitude limit associated with liquefaction triggering is for probabilistic liquefaction hazard analysis (PLHA) (e.g., Kramer and Mayfield 2007), for which a lower limit is required in the same way as for probabilistic seismic hazard analysis (PSHA). More than three decades ago, in the very first issue of this journal, Atkinson et al. (1984) presented a simplified approach to PLHA, stating that "*since earthquakes of magnitude less than 5 are not of sufficient duration to cause liquefaction, M5 is the lowest magnitude which contributes to the probability of liquefaction*." However, in a subsequent study, Goda et al. (2011) showed that using a lower bound magnitude of 4.5, versus 5.0, does influence the computed return period of liquefaction for sites where the seismic hazard is dominated by lower magnitude events. In this report, we aim to explore the issue of what is an appropriate lower bound magnitude for evaluating the risk from liquefaction triggering. We begin with a brief discussion of the concept of minimum magnitude as it applies in PSHA and its definition in the framework of PLHA. This is followed by a critical review of field reports of liquefaction due to small earthquakes, after which we present a simplified parametric study aimed at estimating the smallest magnitudes capable of triggered liquefaction in two idealized soil profiles, one we characterize as being "extremely susceptible" to liquefaction and the other as "very susceptible" to liquefaction. The report concludes with our interpretation of the field data and parametric study results to propose a lower bound of earthquake magnitude for the assessment of liquefaction hazard to the built environment.

2. THE CONCEPT OF MINIMUM MAGNITUDE AND LIQUEFACTION

The concept of the minimum magnitude, M_{min}, defined for PSHA integrations is a topic of some confusion in the field of engineering seismology, a situation that may have arisen because it is a parameter often viewed through the lens of seismic hazard whereas in fact it is related to seismic risk. Bommer and Crowley (2017) proposed a definition of M_{min} as the lower limit of integration over earthquake magnitudes such that using a smaller value would not alter the estimated risk to the exposure under consideration. The imposition of an M_{min} value will, however, generally modify the estimate of the ground shaking hazard, particularly for spectral accelerations at shorter oscillator periods and at higher annual frequencies of exceedance. The point is that the hazard contributions from smaller magnitude events are associated with ground motions insufficiently energetic to cause damage to the structures for which the hazard is being estimated and therefore the reduction of the hazard by removal of these events has no impact on the estimated risk (apart from rendering its calculation more efficient). Following from this definition, it is immediately clear that M_{min} may vary with different applications of PSHA results: a value of **M** 5 may be appropriate for defining the seismic design loads for a nuclear power plant, whereas a smaller value may be more fitting for the assessment of seismic risk due to induced earthquakes in a region of unreinforced masonry dwellings.

The definition of M_{min} in PSHA can be directly translated to PLHA by analogy, if the 'risk' is now considered to be the severity or damage potential of soil liquefaction rather than structural damage, which would be a common measure in seismic risk estimation. The role of fragility functions in seismic risk analysis is now replaced by the susceptibility of the soil profiles to liquefaction. For the purpose of addressing the question "What is the smallest magnitude earthquake that can trigger liquefaction?", the 'fragility' (liquefaction susceptibility) of a site needs to be defined. As is discussed in the following sections of the report, the question can be posed in two ways, the first being what is the smallest magnitude of earthquake that can trigger liquefaction in any soil profile? A second, and more pertinent, question from an engineering perspective is: what is the smallest magnitude of earthquake that can trigger liquefaction in a soil profile that is sufficiently competent to support infrastructure? The answer to the latter question is of greater importance.

Before closing this brief discussion of M_{min} in the context of liquefaction hazard analysis, one might ask whether it would not be more appropriate to define the minimum level of ground shaking that might trigger liquefaction. In the context of PSHA, the question is very pertinent and is the reason why alternative approaches to the use of sharp cut-off on magnitudes, regardless of source-to-site distance and other considerations, are employed [e.g., screening criteria based on levels of Cumulative Absolute Velocity, CAV (EPRI 2006)]. However, for the case of liquefaction triggering, this is controlled by both the amplitude (most usually peak ground acceleration, PGA) and the duration (or number of cycles of motion) simultaneously. Therefore, for earthquakes occurring at short distances from the site of interest, the magnitude is potentially a good indicator of the capacity of the motion to trigger liquefaction since both PGA and duration depend on magnitude—and display inverse dependence on distance (Lasley et al. 2017). Moreover, residuals of PGA and duration with respect to median predictions from GMPEs are found to be negatively correlated (Bradley 2011). Studies that have focused on thresholds of PGA for liquefaction triggering have normalized the peak acceleration values to a common reference magnitude precisely to account for the influence of duration (Santucci de Magistris et al. 2013). Absolute minimum PGA thresholds for liquefaction could be defined on the basis of lower amplitudes of motion being incapable of inducing sufficient strain generate excess pore water pressure in the soil, which is requisite for liquefaction triggering (Dobry et al. 1982, Rodriguez-Arriaga and Green 2018) but to use such an approach for screening of liquefaction hazard would require estimation of PGA values, with the attendant difficulties of extrapolating empirical ground-motion prediction equations (GMPEs) to smaller magnitudes (e.g., Bommer et al. 2007). For PLHA, there may be benefits of defining a lower bound for hazard contributions based on a ground-motion parameter, or vector of parameters, but CAV may not be the most suitable metric for this purpose—indeed, its relevance to structural damage has been questioned for some building types (Campbell and Bozorgnia 2012). As a starting point, however, minimum magnitude is potentially an effective lower bound for PLHA and it is clearly a convenient criterion for determining whether liquefaction hazard requires consideration when assessing the impact of induced earthquakes.

Regarding established threshold ground motion and magnitude criteria below which liquefaction evaluations are not required, it is worth briefly summarizing some of the criteria specified in United States (US) design codes ASCE 7-16 (ASCE 2017) and AASHTO (2014). ASCE 7-16 does not require liquefaction, or other potential geologic and seismic hazards, to be evaluated for Seismic Design Categories (SDC) A and B structures, where SDC is a function of both the design ground motions and the Risk Category of the structure. In general, for Risk Category II structures, which encompasses "typical" structures (i.e., non-essential facilities that neither pose a "low" nor a "substantial" risk to human life in the event of their failure), SDC A and B classification is based on the amplitudes of the spectral accelerations of the design ground motions for both 0.2-s and 1.0-s oscillator periods (i.e., S_{DS} and S_{D1}, respectively). These dual criteria inherently encompass both the PGA and duration of the design motions that influence the triggering of liquefaction because, in general, S_{DS} strongly correlates with PGA and S_{D1} strongly correlates with magnitude, which in turn strongly correlates with ground-motion duration. This is illustrated in Figure 1 which shows PGA and the spectral accelerations for 0.2-s and 1.0-s oscillator periods (S_s and S_1, respectively) as a function of earthquake magnitude for motions recorded by the Guerrero accelerograph array in Mexico in 1985 and 1986. All the stations were on hard rock and all the events have epicenters about 25 km from the station (Anderson and Quass 1988). As may be observed from this figure, both PGA and S_s have similar correlations with magnitude (as indicated by similarity in the slopes of lines formed by the data points) and their correlation with magnitude is not as strong as the correlation between S_1 and magnitude. Similar to ASCE 7-16, AASHTO (2014) does not require liquefaction evaluations to be performed for highway bridge sites that are categorized as SDC A or B (Marsh et al. 2014). However, the SDC is solely based on S_{D1} in AASHTO (2014), which again has a relatively strong correlation with earthquake magnitude and ground-motion duration.

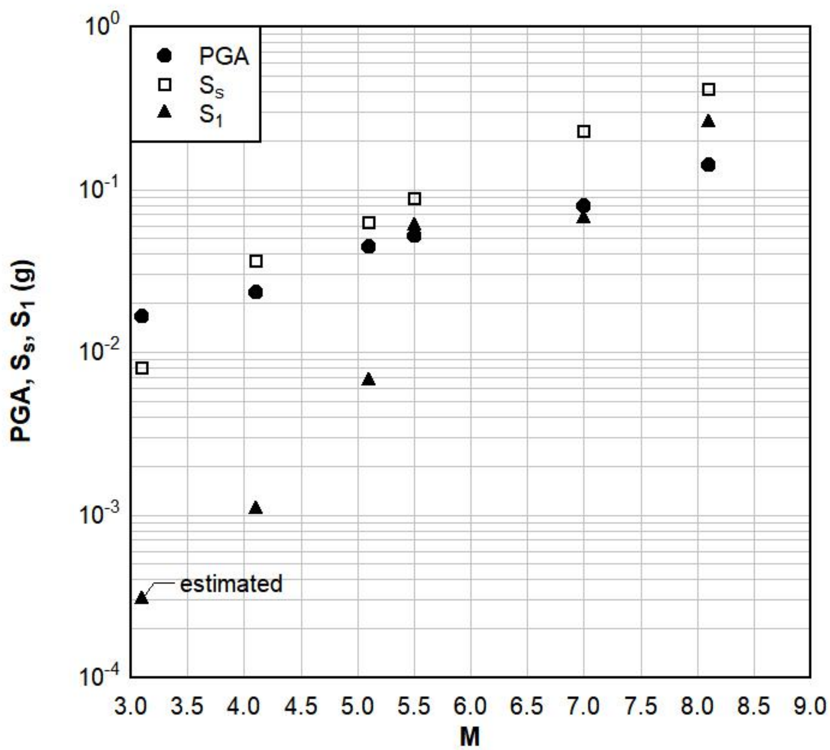


Figure 1. Peak ground acceleration and spectral accelerations for 0.2-s and 1.0-s period oscillators vs. earthquake magnitude for motions recorded by the Guerrero accelerograph array in Mexico in 1985 and 1986. All the stations were on hard rock and all the events have epicenters about 25 km from the station.

Both the ASCE 7-16 and AASHTO (2014) criteria are readily implementable because seismic hazard maps accompany the codes and liquefaction evaluations are performed in a pseudo-probabilistic manner, rather than in a probabilistic manner (i.e., liquefaction is evaluated for a ground motion having a given return period, rather than the return period of liquefaction being evaluated). Also, it is worthy of note that the seismic hazard maps that accompany both ASCE 7-16 and AASHTO (2014) are based on PSHA that uses a lower bound magnitude of 5.0.

3. FIELD OBSERVATIONS OF LIQUEFACTION IN SMALL EARTHQUAKES

Assessment of field reports of liquefaction triggering by small-to-moderate earthquakes needs to infer lower bound magnitude thresholds requires evaluation of three factors. One of these, as indicated above, is the susceptibility of the soil deposits reported to have liquefied. The other two factors are the reliability of the earthquake source parameters (particularly the magnitude value) and the confidence that can be placed in the observed effects being both genuinely associated with liquefaction and unambiguously the result of the earthquake in question. Below we briefly review the available case histories, but not in chronological order for reasons that will be apparent.

Before reviewing individual case histories of liquefaction in smaller earthquakes, we note that in databases compiled for the derivation of models used in various types of liquefaction related hazard assessments we find no case of events smaller than magnitude 5. There is a single case of **M** 5, for example, in the database of Ambraseys (1988) with the next smallest being **M** 5.2. In the database of Japanese liquefaction cases of Kuribayashi and Tatsuoka (1975) and that for Greek cases by Papadoplous and Lefkoplous (1993) all events are larger than magnitude 5. Similarly, all cases of lateral spreading reported by Keefer (1984) and Rodriguez et al. (1999) are larger than magnitude 5. Based on these compendia of case histories, observations of liquefaction effects in events of less than **M** 5 would be seem to be exceptional.

3.1. PAWNEE, OKLAHOMA, 2016

This earthquake, likely triggered by wastewater injection, had a moment magnitude **M** 5.8 and caused small sand boils and cracks due to lateral spreading at three locations along the

Arkansas River (Clayton et al. 2016, Kolawole et al. 2017). The magnitude is well constrained and there is clear photographic evidence of the liquefaction effects that manifest in the clearly susceptible environment of a river bank. There is, in fact, nothing remarkable about this particular case history, but it is included in this overview because it is the only report that we are aware of liquefaction due to an induced or triggered earthquake. This is significant because inferences about minimum magnitude thresholds for liquefaction made from field observations are inevitably subject to the claim that absence of evidence cannot be taken as evidence of absence. However, counter to this view is the fact that induced and triggered seismic events have tended to attract great scrutiny in recent years and yet no triggered or induced event smaller than the Pawnee earthquake has been reported to trigger liquefaction.

3.2. ROERMOND, THE NETHERLANDS, 1992

This earthquake in the south of the Netherlands on the border with Germany triggered liquefaction effects similar to those observed in the Pawnee earthquake, for which there is clear photographic evidence. Grain size gradation curves obtained from laboratory tests on samples from one of the liquefied sites indicated a poorly graded sand that would be highly susceptible to liquefaction (Nieuwenhuis 1994). The earthquake was assigned a moment magnitude of **M** 5.3 by the US Geological Survey.

3.3. OLANCHA, CALIFORNIA, 2009

There were observations of extensive liquefaction of susceptible sand deposits in an area where the water table was close to the ground surface following this earthquake of magnitude **M** 5.2. Holzer et al. (2010) published a paper presenting these observations as noteworthy precisely because "*liquefaction….is common in earthquakes with moment magnitudes (**M**) greater than 6 and frequently causes damage, but it is rarely associated with earthquakes of **M**≤5.2*." Holzer et al. (2010) attribute the occurrence of liquefaction in this earthquake to the susceptibility of the soils (an active alluvial fan depositing coarser grains over fine-grained lake deposits) rather than exceptional ground motions.

3.4. LOMA PRIETA AFTERSHOCK, CALIFORNIA, 1991

Holzer et al. (2010) note the report by Sims and Gavin (1995) of liquefaction caused by a magnitude **M** 4.6 aftershock of the October 1991 Loma Prieta earthquake. Clear evidence of liquefaction was documented for this earthquake, which is significant since it establishes that the lower bound magnitude is at least as small as 4.6. However, it is noteworthy that the liquefaction was observed to occur in the extremely susceptible deposits of the dry Soda Lake, which was formerly a man-made settling basin. Moreover, Sims and Gavin (1995) noted that the "*sandblows developed during the March 1991 aftershock erupted only through pre-existing vents*." As a result, it is very possible that surficial evidence of liquefaction having triggered at depth would not have manifested if liquefaction dikes from the source stratum to the ground surface had not formed at the site during the **M** 6.9 Loma Prieta main shock.

3.5. RANDOLPH, UTAH, 2010

This case history is also noted by Holzer et al. (2010) and it supports the lower bound implied by the previous case with the moment magnitude of this event estimated at 4.5-4.6. Sand boils were observed on the banks of a river (Fig. 2) that appears to be almost swampy ground. DuRoss (2011) reported that "*We attribute the occurrence of liquefaction to highly susceptible sediments very near the epicenter*."

3.6. CHRISTCHURCH, NEW ZEALAND, 2010-2011

Widespread liquefaction was triggered throughout Christchurch and surrounding areas during the 2010-2011 Canterbury, New Zealand, earthquake sequence (CES), with the largest event in the sequence being the 2010 **M** 7.1 Darfield earthquake (e.g., Cubrinovski and Green 2010, Cubrinovski et al. 2011, Green et al. 2011, Green et al. 2014, Wood et al. 2017). As many as ten distinct episodes of liquefaction triggering occurred in regions of Christchurch during the CES (Quigley et al. 2013). These observations are significant because on-site inspections were made following many of the felt episodes of shaking and hence this is a case for which there is actual evidence for absence—at least of any effects observable at the surface. Figure 3 shows the key findings from that study, from which it can be appreciated that the smallest event to have triggered liquefaction had a moment magnitude of **M** 5.0. The red squares in the figure, which correspond to events for which no surface ejecta were observed, range from **M** 4.5 to 5.4.

This is an important series of case histories because it is one of the only instances of liquefaction being observed in the vicinity of buildings (i.e., low-rise residential structures), implying that the ground was essentially competent under static conditions, albeit a site that is very susceptible to liquefaction. Whereas there have been observations of liquefaction due to smaller events on extremely susceptible ground—on which it is hard to imagine that it would have been possible to build—these results point to a threshold of **M** 5 for the triggering of liquefaction in soils capable of supporting construction.

Figure 2. Liquefaction effects observed following the **M** 4.6 Randolph earthquake in Utah (Pankow et al. 2015).

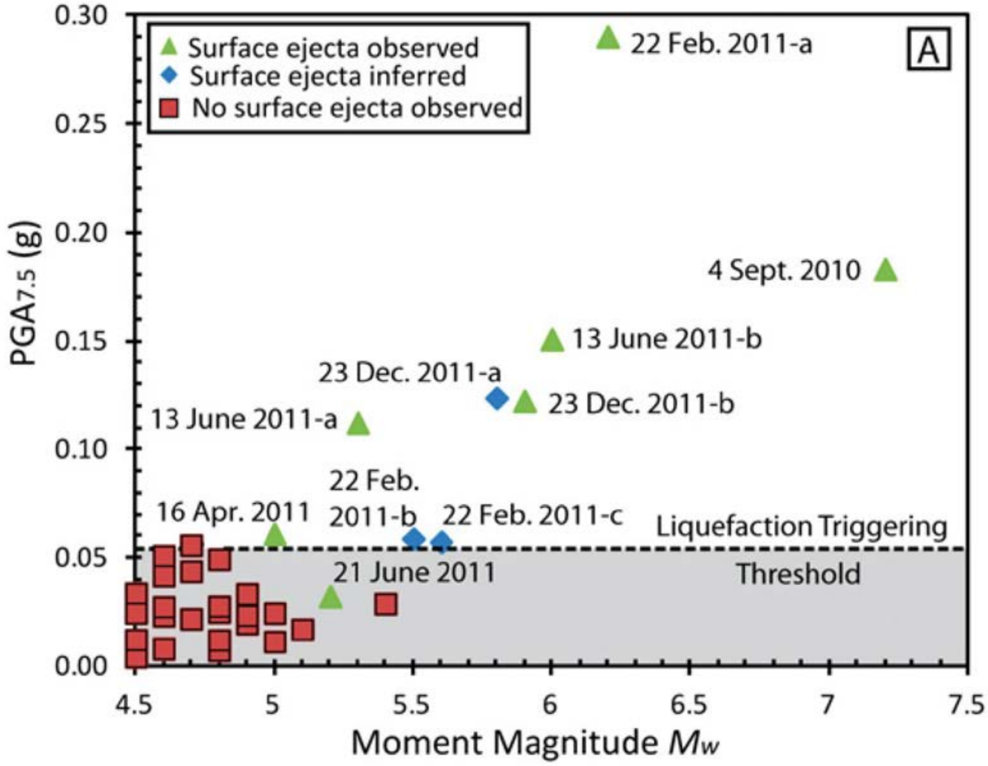


Figure 3. Observations of liquefaction as a function of magnitude and magnitude-normalized PGA ($PGA_{7.5}$) for a site in Avonside, an eastern suburb of Christchurch, New Zealand, during the 2010-2011 CES (Quigley et al 2013).

3.7. FALCON STATE, VENEZUELA, 1989

Audemard and de Santis (1991) report sand boils occurring in the delta of the Tocuyo River in Venezuela as a result of an earthquake swarm in 1989. The coastal sand deposits where the sand boils were observed were clearly susceptible to liquefaction and in all cases seemed to have found their way to surface through crab burrows and existing fractures. This case history is noteworthy, however, as an illustration of the importance of establishing reliable source parameters for the earthquakes and clear association of the liquefaction phenomena with the seismic event.

Audemard and de Santis (1991) identify two earthquakes, with body-wave magnitudes m_b of 5.7 and 5.0, both more than 15 km offshore, as being the cause of the liquefaction. The two earthquakes occurred six days apart but it is unclear from the paper if the field studies were conducted in such a way as to separate and distinguish their effects. The source parameters for the earthquakes were obtained from the national seismological service in Venezuela. The catalog of the International Seismological Centre lists three earthquakes on the same dates and in the same area. The first two (m_b 4.7 and m_b 4.6, respectively) occurred on April 30 at least 30 km from the coastline; the third occurred on May 4, much closer to the shore and is assigned m_b 5.4 and surface-wave magnitude M_S of 5.2. Notwithstanding that these locations may also be offset from the true epicenters, it seems reasonable to conclude that the modest manifestations of liquefaction in this highly susceptible environment were the result of the final earthquake of magnitude greater than 5.

3.8. BARROW-IN-FURNESS, UK, 1865

The previous case history highlights an instance of uncertainty regarding the source characteristics of the earthquake to which observations of liquefaction have been attributed. In the case of the liquefaction effects claimed to have occurred near the coastal village of Barrow-in-Furness in NW England in 1865, there is doubt regarding both the earthquake source parameters and the actual liquefaction effects as well. This case history warrants careful consideration because if the claims of Musson (1998) were verified, this is a game changer: an earthquake with a magnitude "*perhaps most likely between 2½ and 3½*" triggering the following liquefaction phenomena according to a contemporary account: "*We saw at a distance from us, a great mass of sand, water and stone thrown up into the air higher than a man's head….when we got to the place there were two or three holes in the sand, large enough to bury a horse and cart, and in several places near them, the sand was so soft and puddly that they would have mired any one if he had gone on to them*." Musson (1998) also reports structural damage caused by the shaking consistent with Modified Mercalli Intensity (MMI) of VIII. A very shallow focal depth is offered as an explanation for such intense motions from such a small earthquake, but very superficial earthquakes would also be expected to have rather low stress drops.

The reported sand boils and volcanoes occurred on tidal flats that are clearly extremely susceptible to liquefaction. Musson (1989) states that "*further evidence of the easily liquefiable nature of the sands is provided by the fact that Morecombe Bay* [10 km to NE] *is notorious for quicksand just in normal conditions*." However, regardless of how susceptible the sands may have been to liquefaction, it is still difficult to reconcile the small magnitude assigned to the earthquake with the very dramatic liquefaction effects with ejecta being projected more than two meters into the air. Musson (1989) notes a report attributing the reported effects to the escape of a large body of gas but discounts this alternative explanation. One reason given to discredit the gas explanation is that the observers would have reported smelling gas, despite the fact that they observed the rising material from some distance on a presumably wind-swept tidal plain and the fact that naturally-occurring methane is odorless. Another reason put forward by Musson (1989) to discount an escape of gas as the explanation is "*the description of quicksand-like effects is more in line with classical liquefaction than with a gas burst*;" to us, sand, rocks and water being thrown into the air do not seem consistent with quicksand and are much more suggestive of an explosive phenomenon. How a micro-to-small magnitude earthquake could generate sufficient cycles of motion to trigger severe liquefaction and associated excess pore pressures required to eject rocks 2 m off the ground is hard to explain.

We are unable to state conclusively whether the magnitude assigned to the earthquake is grossly underestimated or if the reported liquefaction effects were actually due to another cause, but we believe that it is easy to establish reasonable doubt with regard to the story related by Musson (1989). Taking the reported magnitude of 2.5-3.5 at face value, globally there are a little more than 400,000 earthquakes of this size annually; therefore, in the 150 years since the Barrow-in-Furness earthquakes, there have been more than 60 million events of similar size. Even discounting all offshore and sub-crustal earthquakes and all events occurring in remote areas away from human habitation, the complete lack of any comparable observations from such small events would at the very least suggest that what is reported to have happened in 19th Century England was an event with a probability of less than 10^{-7}. More likely, it suggests that the claim of liquefaction triggering by an earthquakes of M ~3 is unfounded and hence this case history can be dismissed from inferences regarding lower bounds of magnitude for liquefaction triggering.

3.9. CHRISTCHURCH, NEW ZEALAND, 1869 AND 1870

Two significant earthquakes that impacted Christchurch, New Zealand, between the start of organized European settlement in 1850 and the September 2010 **M** 7.1 Darfield earthquake occurred in 1869 and 1870 (Downes and Yetton 2012). The macroseismic epicenter of the 1869 Christchurch earthquake is estimated to be approximately 3.5 km SW of the center of the city's most densely populated region at the time, which is currently the city's Central Business District (CBD). Damage to chimneys, unreinforced masonry structures, and internal contents of both residences and businesses were reported in the CBD and nearby suburbs, with the damage intensity in the CDB being assessed as MMI VII. Based on an MMI isoseismal map for the event and New Zealand-specific ground motion prediction equations (Dowrick and Rhoades 1998), Downes and Yetton (2012) estimate the event had a shallow focal depth and a moment magnitude of 4.7 to 4.9. No liquefaction or ground damage was reported for this event, despite several regions that experienced the most intense shaking and structural damage having deposits that are very susceptible to liquefaction (e.g., Avonside, Fig. 3).

The macroseismic epicenter of the 1870 earthquake that impacted Christchurch is estimated be under Lake Ellesmere, which is approximately 25 km S-SW of the CBD and is a former mouth of the Waimakariri River, which currently empties into Pegasus Bay north of Christchurch. Lake Ellesmere is shallow and is better described as a lagoon/estuary than a lake. Shaking from this event was felt over much of the central portion of the South Island of New Zealand and the estimated magnitude of this event is **M** 5.6-5.8 (Downes and Yetton 2012). Damage in the CBD and nearby suburbs from this event was similar to, but slightly less (MMI VI vs. MMI VII) than, that experienced during the 1869 Christchurch earthquake. No liquefaction was reported per se, but muddying of a creek near Lake Ellesmere was reported. This may indicate that liquefaction occurred in the creek bed, which was pervasive in the Avon and Heathcote River beds and in the Avon-Heathcote Estuary during the 2010-2011 CES (e.g., Fig. 4). The Avon-Heathcote Estuary is also a former mouth of the Waimakariri River and, thus, likely has similar deposits as Lake Ellesmere (Green et al. 2018); these deposits may also be similar to the tidal flats which are purported to have severely liquefied during the 1865 Barrow-in-Furness earthquake (Musson 1989).

Although there is some uncertainty about the earthquake source parameters and liquefaction response of the deposits for these two events, clearly deposits susceptible to liquefaction were subjected to shaking during these events (e.g., river and lake bed deposits and Avonside). The significance of these events is that the observed liquefaction responses are in direct accord with those made during the 2010-2011 CES and completely independent (i.e., the **M** ~4.7-4.9 event did not trigger liquefaction and the **M** ~5.6-5.8 event likely triggered liquefaction).

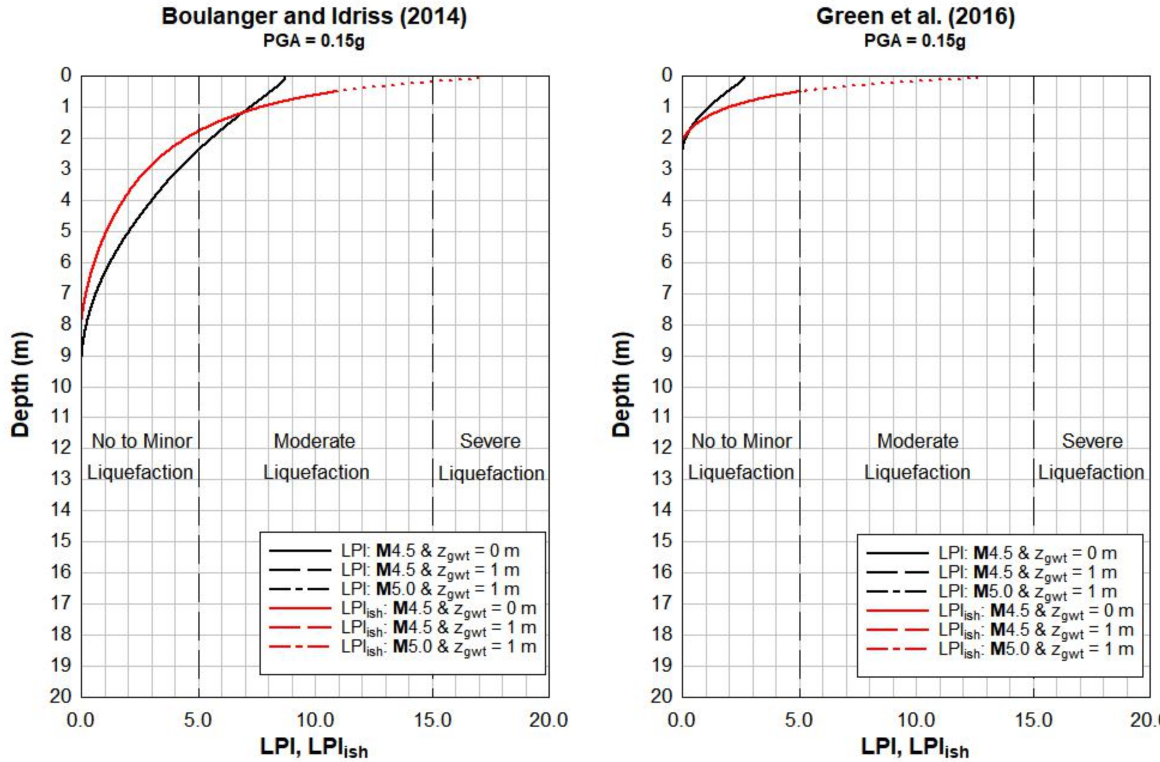
Figure 4. Severe surficial liquefaction manifestations in the Avon-Heathcote Estuary that formed during the 2011, **M** 6.2 Christchurch earthquake, where the PGA at this site estimated to be greater than 0.6 g (Photo courtesy of Greg DePascale, 22 Feb. 2011).

4. A SIMPLE PARAMETRIC STUDY

In order to explore the lower limit on earthquake magnitude for triggering liquefaction, a simple parametric study is performed using two idealized profiles, one that we refer to as being "very susceptible" to liquefaction and the other as being "extremely susceptible" to liquefaction. As shown subsequently, the distinction between the two profiles manifests in their ability to support construction, as well as their resistance to liquefaction triggering. Both idealized profiles are comprised of thick deposits of loose, clean fine sand with shallow ground water tables. In the engineering analyses, we model the profile that we refer to as being extremely susceptible to liquefaction as having the ground water table at the ground surface (i.e., z_{gwt} = 0 m) and having a constant normalized cone penetration test (CPT) tip resistance (q_{c1Ncs}) equal to 84 atm down to a depth of 20 m. Using the correlation proposed by Robertson (2014), the sand in this deposit has a relative density (Dr) of approximately 20%, which is very loose and about the loosest state found in nature (e.g., in very young, estuary deposits). The profile that is very susceptible to liquefaction is identical to the one that is extremely susceptible to liquefaction, except that it has a 1-m thick dense crust (i.e., q_{c1Ncs} = 180 atm) and the depth to the ground water table corresponds to the base of the dense crust (i.e., z_{gwt} = 1 m), which by all accounts is still very shallow.

The parametric study performed using the two profiles entailed predicting the severity of surficial liquefaction manifestations using the Boulanger and Idriss (2014) (BI14) and Green et al. (2016) (Gea16) CPT-based simplified liquefaction evaluation procedures, operating within the Liquefaction Potential Index (LPI: Iwasaki et al. 1978) and Ishihara-inspired LPI (LPI_{ish}: Maurer et al. 2015) liquefaction damage potential frameworks. Two earthquake magnitudes were considered, **M** 4.5 and **M** 5.0, and a peak ground acceleration (PGA or a_{max}) of 0.15 g is assumed, which is reasonable, although likely having an epsilon greater than zero, for the epicentral region of shallow earthquakes in this magnitude range.

Figure 5 shows the results of the liquefaction parametric study, and as shown, the computed LPI and LPI_{ish} values at the ground surface are only greater than zero for **M** 4.5 and z_{gwt} = 0 m (i.e., extremely susceptible profile). For these analyses, it is assumed that an LPI and LPI_{ish} value of 5 is the threshold that separates no-to-minor and moderate surficial liquefaction manifestations, and an LPI and LPI_{ish} value of 15 is the threshold that separates moderate and severe surficial liquefaction manifestations (Maurer et al. 2014, Maurer et al. 2015). Based on these thresholds, the BI14 procedure operating within the LPI and LPI_{ish} frameworks predicts moderate/severe surficial liquefaction manifestations for the scenario **M** 4.5 and z_{gwt} = 0 m, while the Gea16 procedure predicts no-to-minor/moderate surficial liquefaction manifestations for this scenario. Note that the dotted portion of the LPI_{ish} curves at shallow depths in Figure 5 is because the framework was never calibrated for z_{gwt} = 0 m conditions, and the computed values above 0.5 m for this conditions is an extrapolation of the procedure beyond its recommended range of use.



(a) (b)

Figure 5. Results from the liquefaction parametric study on "very susceptible" (i.e., q_{c1Ncs} = 84 atm and z_{gwt} = 1 m) and "extremely susceptible" (i.e., q_{c1Ncs} = 84 atm and z_{gwt} = 0 m) profiles: (a) Computed LPI and LPI_{ish} profiles using the BI14 CPT-based simplified procedure; and (b) computed LPI and LPI_{ish} profiles using the Gea16 CPT-based simplified procedure.

Although the severity of the predicted surficial liquefaction manifestation varies depending on which simplified procedure and liquefaction damage potential framework are used, the trends are consistent. Namely, surficial liquefaction manifestations are only predicted for an **M** 4.5 event for the profile that is extremely susceptible to liquefaction (i.e., z_{gwt} = 0 m), and earthquakes having magnitudes 5.1 and 5.8 (assuming PGA = 0.15 g) or greater are required for surficial liquefaction manifestations to be predicted for the profile that is very susceptible to liquefaction using the BI14 and Gea16 procedures, respectively. This finding is consistent with AASHTO (2014) which justifies not requiring liquefaction evaluations to be performed for SDC A and B highway bridge sites because "*For Seismic Design Categories A and B, the potential for liquefaction is generally low, as peak ground accelerations are likely to be less than 0.14g and earthquake magnitudes are likely to be less than 6.0.*" (Marsh et al. 2014).

The distinction between the profiles that are very versus extremely susceptible to liquefaction are put into context by their abilities to support construction. Towards this end, the settlement and factor of safety against bearing capacity failure (FS_{bc}) for a 1-story, conventional light-framed building (e.g., wood-framed, ranch style house) are computed for the two profiles. For residential structures, tolerable settlement is generally limited to 0.0254 m (1 inch) and the minimum acceptable FS_{bc} is 3 (NAVFAC 1986). Based on the "presumptive" allowable bearing pressures and corresponding minimum footing widths specified in the International Residential Code (IRC 2007), the design loads imposed on a strip footing from the superstructure of a 1-story, conventional light-framed building is back-calculated to be approximately 18.7 kN/m (1300 lb/ft).

The CPT-based procedures proposed by Meyerhof (1974) and Meyerhof (1956) are used to evaluate settlement and bearing capacity, respectively. Both of these procedures are based on the uncorrected CPT tip resistance (q_c) averaged over a depth equal to the footing width (B) below the base of the footing. Accordingly, the procedure proposed by BI14 is used to back-calculate q_c from the assumed q_{c1Ncs} values for the two profiles; note that Gea16 adopted the BI14 procedure that relates uncorrected and corrected CPT tip resistances. Figure 6 shows the back-calculated q_c profiles, and as may be observed, there is a slight difference in the q_c values for the two profiles below the depth of the dense crust, which is due to differences in z_{gwt} for the two profiles and its influence on vertical effective stress.

The depth of embedment (i.e., depth to the bottom of the footing) is required to be a minimum of 0.3048 m (1 ft) below the undisturbed ground surface and all exterior footings need to be embedded down to the frost line depth (IRC 2007), which will vary based on regional temperatures. In the contiguous US the frost line depth ranges from 0 to 2.54 m (0 to 100 inches), with about half the land mass of the contiguous US having a frost line depth of 0.6096 m (24 inches) or less. Accordingly, for the calculations presented herein, the frost line depth is assumed to be 0.6096 m (24 inches). Using this depth of embedment and assuming B = 0.3048 (1 ft), which is standard for 1-story conventional light-framed construction in the US, q_c averaged over a depth B below the base of the footing (i.e., q_{c_avg}) for the profiles that are very and extremely susceptible to liquefaction are 86.1 and 20.6 atm, respectively. Based on these values, the settlement computed using the Meyerhof (1974) procedure is negligible for both profiles [i.e., well below the tolerable limit of 0.0254 m (1 inch)]. However, the FS_{bc} for the very and extremely susceptible profiles are 10.5 and 2.3, respectively; these FS_{bs} are above and below, respectively, the minimum acceptable FS_{bc} of 3.0. Although the FS_{bc} = 2.3 is not that much below 3.0 and that a wider footing width could be used to increase FS_{bs}, this factor of safety is for the in-place structure. If one considers the construction process and that the tire pressure for construction equipment with pneumatic tires is approximately 103.4 kPa (15 psi), on the low end, the upper bound FS_{bc} for the construction equipment under static conditions is 0.2 for the profile that is extremely susceptible to liquefaction. This FS_{bc} is comparable to that of an average-sized human trying to walk across the site shown in Figure 2, the difficulty of which is easily imaginable.

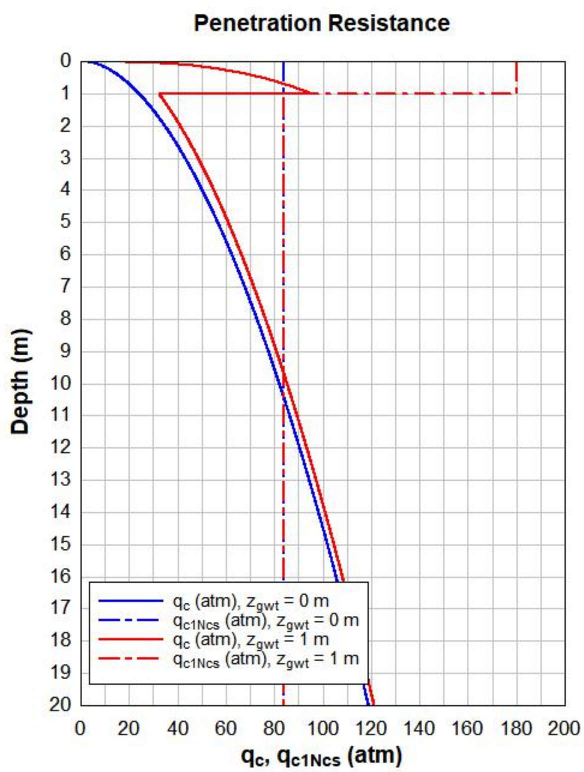


Figure 6. Corrected and uncorrected CPT tip resistances (q_{c1Ncs} and q_c, respectively) for the very susceptible (z_{gwt} = 0 m) and extremely susceptible (z_{gwt} = 1 m) profiles.

5. DISCUSSION AND CONCLUSIONS

There is clear and reliable field evidence that surface manifestations of liquefaction triggering have occurred due to earthquakes of magnitude as small as **M** 4.5, but these invariably correspond to exceptional cases of extremely susceptible ground that could not support even the lightest construction. Evidence for both the occurrence and absence of liquefaction triggering in susceptible ground supports the conclusion that **M** 5.0 is the lower bound for liquefaction triggering of relevance to the built environment. The conclusions inferred from field observations are supported by the results of analyses using engineering models. We also emphasize that the threshold magnitude is a necessary but not sufficient condition: the occurrence of an earthquake of **M** 5 close to susceptible soil deposits will not automatically result in liquefaction triggering.

One clear consequence of this conclusion is that when assessing the risk associated with induced seismicity, unless earthquakes in excess of magnitude 5 are expected, liquefaction can be disregarded as a hazard. The other logical consequence of our findings is that in PLHA, earthquakes of magnitude smaller than **M** 5 do not need to be considered in the hazard integrations. This applies equally to genuine PLHA and pseudo-probabilistic liquefaction hazard assessment: in the latter, contributions to the PGA hazard from earthquakes smaller than magnitude 5 should be eliminated.

Our conclusion regarding the minimum magnitude to be considered in PLHA actually confirms and substantiates the lower limit proposed by Atkinson et al. (1984). However, as noted in the Introduction, Goda et al. (2011) find non-negligible contributions to liquefaction hazard in Canadian cities—and particularly in Montreal (their Fig. 6)—from earthquakes in the range of magnitude 4.5 to 5. This apparent disparity with the conclusion drawn herein is likely due to the stress reduction factor (r_d) relationship that Goda et al. (2011) used in conjunction with the small strain shear wave velocity (Vs) based liquefaction evaluation procedure proposed by Andrus and Stokeo (2000) (AS00). Aside from general shortcomings of Vs-based liquefaction evaluation procedures, Goda et al. (2011) used the r_d relationship proposed by Liao et al. (1988), which is independent of earthquake magnitude. Subsequent r_d relationships proposed by Idriss (1999), Cetin et al. (2004), and Lasley et al. (2016), among others, show that r_d tends to decrease as magnitude decreases (i.e., the soil column responds less rigidly as magnitude decreases). This is mainly due to the decrease in the energy of long period motions in smaller magnitude events, as illustrated by the trend in S_1 as a function of **M** shown in Figure 1. This implies that the cyclic stresses imposed in a stratum at depth in a soil profile are less demanding as the magnitude of the earthquake decreases, separate from the additional magnitude dependencies of PGA and duration of shaking. Accounting for this phenomenon would likely reconcile the disparity of the influence of events less than **M** 5 on the computed liquefaction hazard shown in Goda et al. (2011) with that shown herein.

6. ACKNOWLEDGMENTS

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