Bull. Min. Res. Exp. (2020) 163: 99-114

Bulletin of the Mineral Research and Exploration

http://bulletin.mta.gov.tr

An approach for the application of energy-based liquefaction procedure using field case history data

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Research Article

Keywords:
Soil liquefaction, Energy-based liquefaction method, Field case histories, Earthquake.

Received Date: 27.09.2019
Accepted Date: 09.01.2020

ABSTRACT

This paper presents an overview to the applicability of the "energy-based liquefaction approach" with regards to the new developments in the subject. The method involves comparing the strain energy for the soil liquefaction (capacity) with the strain energy imparted to the soil layer during an earthquake (demand). The performance of the method was evaluated by using a large database of SPT-based liquefaction case history. The energy-based method and the more commonly used stress-based method were compared in their capability to assess liquefaction potential under the same damaging historic earthquakes and geotechnical site conditions. In the procedure, the predictive strain energy equations were used to estimate the capacity energy values. These empirical equations have been developed based on the initial effective soil parameters. As for the energy of any given strong ground motion, it was computed from a velocity-time history of the ground motion and the unit mass of soil through utilization of kinetic energy concepts. The proposed energy-based method has effective way in evaluating the liquefaction potential based on the seismological parameters, contrary to the stress-based approach, where only peak ground acceleration (PGA) is considered.

1. Introduction

The soil liquefaction during a strong ground motion is a significant and ever-present phenomenon that threatens to damage or collapse buildings, bridges, highways, embankments, and other civil engineering structures. Catastrophic events such as Niigata (Japan) in 1964, Loma Prieta (California) in 1989, Kobe (Japan) in 1995, Kocaeli (Turkey) in 1999 indicated that the most striking failures on the ground are due to the soil liquefaction (e.g. sand boils/settlement-type ground deformations, lateral spreading and natural slope failures or flows).

The evaluation of the soil liquefaction is a complex problem in earthquake engineering, due to having numerous factors controlling the mechanism of the liquefaction (e.g. the magnitude, intensity, path effects, attenuation characteristics, types of soils, confining pressure, the distance from the source and other site-specific conditions) (Law et al., 1990). Numerous laboratory techniques and model tests, in-situ techniques and numerical approachs have been performed for assessment of the liquefaction potential (e.g. Finn et al., 1971; Seed and Idriss, 1971; Martin, 1975; DeAlba et al., 1976; Ladd et al., 1989; Elgamal et al., 1989; Tokimatsu et al., 1991; Oka et al., 1994; Youd et al., 2001; Zhang, 2001; Moss et al., 2006; Boulanger and Idriss, 2012). At the same time, several field procedures have been highlighted for more accurate assessment of liquefaction potential. The

Citation info: Kayabalı, K., Selçuk, L., Beyaz, T. 2020. An approach for the application of energy-based liquefaction procedure using field case history data. Bulletin of the Mineral Research and Exploration 163, 99-114. https://doi.org/10.19111/bulletinofmre.677626.

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available evaluation procedures for assessment of soil liquefaction include: 1) the Stress-based approach, 2) the strain-based approach, and 3) the energy-based approach (Green, 2001; Zhang et al., 2015).

The stress-based procedure for evaluation of liquefaction potential started with a basic approach defined by Seed and Idriss (1967), which has since been upgraded with many studies that have contributed the method. The contents of these studies, mostly quoted from Shahien (2007) can be summed up in to the following; a) update of the field case history data, b) deterministic or probabilistic treatments, and c) modification of some components of the liquefaction procedure (e.g. Seed et al., 1975; Seed, 1979,Tokimatsu and Yoshimi, 1983; Seed et al., 1984; Jamiolkowski et al., 1985; NRC, 1985; Ambraseys 1988; Lio et al., 1988; Hendron, 1990; Castro, 1995; Fear and McRoberts, 1995; NCEER,1997; Youd et al., 2001; Seed et al., 2001; Çetin et al., 2000 and 2004; Idris and Boulanger, 2006).

In the generalized framework, the cyclic stress ratio, CSR is compared with the cyclic resistance ratio, CRR of the soil. This procedure, however, involves some uncertainties. In the laboratory applications, the time-dependent irregular variation of shear stress should be converted to equivalent sequences of uniform shear cycles. As for the comparison of the earthquake-induced stress with the harmonic loading conditions, Seed et al. (1975) assumed the equivalent stress as “65% of the maximum shear stress. Ishihara and Yasuda (1975) concluded it’s to be 57% (Zhang et al., 2015). Site response analysis is another approach for determination of cyclic resistance ratio. However, both a site response analysis and the procedure of Seed and Idriss (1967) require the determination of the a_{max} on the ground level of a project site as well. Determination of the a_{max} in a project site also brings with it some uncertainties such as the magnitude scale, site-to-source distance and the attenuation model itself in computing the a_{max}. Although the stress-based procedure has been re-evaluated with adequate studies and also updated with case histories, the limitation relating random loading still continue. (Baziar and Jafarian, 2007; Zhang et al., 2015).

Just like the stress-based liquefaction procedure, strain based approach has some similar limitations. The amplitude of the earthquake-induced cyclic shear strain (γ) is estimated from the cyclic stress (τ) and the shear modulus (G). The other variables are the similar components of cyclic stress as described by Seed and Idriss (1971). In the procedure introduced by Dobry et al. (1982), the cyclic threshold shear strain plays a significant role for pore-water pressure produced by cyclic loading. Upon a series of strain-controlled undrained cyclic tests on saturated sand specimens, Dobry et al. (1982) showed that the threshold shear strain for liquefaction to initiate is approximately 0.11%. Silver and Seed (1971) reported that this value has the range of approximately 0.020% - 0.030% for clean sands. Ladd et al. (1989) found this value to be roughly 0.011%. Vucetic (1994) and Hsu and Vucetic (2004) revealed that the cyclic threshold shear strain value of clayey soils is greater than those of sands. These researchers confirmed that this value was increased or decreased by soil characteristics (Kusumawardani et al., 2015).

As for the energy-based approach, the concept of strain energy and its applications for evaluation liquefaction potential has been described by the researchers (e.g. Davis and Berrill, 1982; Law et al., 1990; Figueroa et al., 1994; Liang 1995; Ostadan et al., 1996; Davis and Berrill 2001; Green, 2001; Baziar and Jafarian, 2007). Davis and Berrill (1982) found out that excess pore water pressure is quite relevant with the amount of strain energy. Thus, the strain energy has been compared with the strain energy imparted to liquefiable soil layer by an earthquake in order to predict the liquefaction. These affords has led to develop the concept of the energy-based liquefaction (Alavi and Gandomi, 2012).There has been a great deal of studies focusing on the strain energy and initial soil parameters in the form of empirical relationships. Figueroa et al. (1994) proposed a relationship relating initial soil properties to dissipated energy. Similarly, Baziar and Jafarian (2007) utilized from the artificial neural network model to suggest a statistical model relating soil parameters to strain energy. They also used a data recorded during earthquakes in addition to the centrifuge tests available to validate their model. They found a reasonable consistency between energy capacity and field observations. The energy-based method offers the following advantages;

1) Energy is related to both shear stress and shear strain;
2) Energy is a scalar quantity that is attributable to the characteristic main earthquake parameters

(e.g. the source-site distance, the earthquake magnitude), all the while considering the entire spectrum of ground motions, contrary to the stress-based approach, where only peak ground acceleration is considered;

3) It has accounting capabilities for the effects of a complex stress–strain history on pore water pressure (Zhang et al., 2015).

Although the energy-based liquefaction procedure offers great advantage, as mentioned here, its application is limited until now due to the fact that the basic principles and extensions of energy-based approach have not been discussed in detail with corresponding applications of stress-based liquefaction procedure. In several researches, the results obtained from the energy-based approach were compared to those of the stress-based approach under the same seismic motions, and some applications are available for actual liquefaction case histories (Kokusho, 2013; 2017; Kokusho and Mimori, 2015; Kokusho et al., 2015). Kokusho et al., (2015) investigated a liquefaction case by a far-field (M: 8.0) earthquake in order to compare the evaluations by stress-based method and energy-based method. It demonstrated a better applicability of energy-based method than stress-based method. Because the maximum acceleration in that case was only about 0.05 g, while its seismic wave energy enough to liquefy. Kokusho and Mimori (2015) pointed out that the energy-based method gives similar results as the stress-based method. Kokusho (2017) report that it is still necessary to apply energy-based method to more case histories to demonstrate its reliability in much more practical conditions. The aim of this study is to demonstrate a simplified procedure of the energy-based approach in accordance with further improvements. To demonstrate the consistency and reliability of the procedure, a large liquefaction database of past events compiled by several researchers (Seed et al., 1984; Idriss and Boulanger, 2004, 2008 and 2010; Çetin et al., 2000, 2004 and 2016) were used as a verification data in the proposed procedure.

2. Energy-Based Liquefaction Approach and Predictive Strain Energy Equations

There has been a great deal of studies relating the energy-based procedures. These procedures involve the different energy measurements in the terms of basic parameters to the demand and the capacity (Green, 2001). In the procedure, the amount of total strain energy for initial liquefaction is obtained from the laboratory testing (cyclic shear or cyclic triaxial testing) or field recorded data. The stress- strain time histories are recorded, and strain energy is given by the area inside the hysteresis loop generated from the stress and strain time histories (Figure 1). This area shows the dissipated energy per unit volume (Ostadan et al., 1996; Green, 2001; Alavi and Gandomi, 2012).

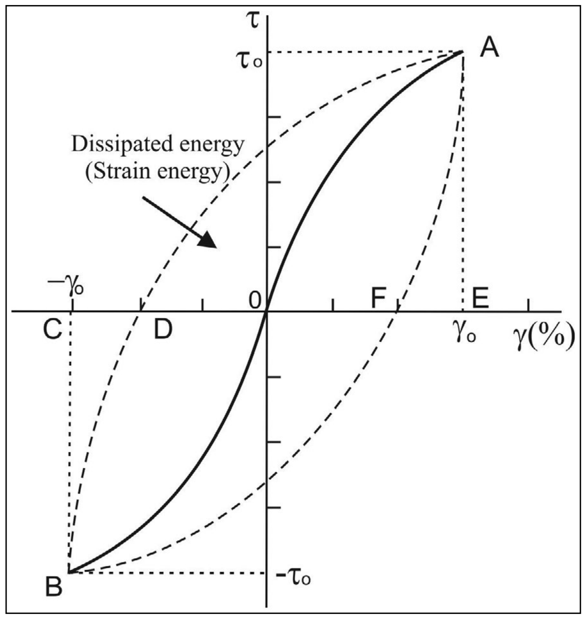

Figure 1- A typical shear stress–strain hysteresis loop.

The total energy (δW) gained by the soil specimen until the onset of liquefaction is computed as follows (Figueroa et al., 1994; Liang et al., 1995):

$$\delta W = \sum_{i=1}^{n-1} 0.5(\tau_i + \tau_{i+1})(\gamma_{i+1} - \gamma_i) \quad (1)$$

Where, τ is shear stress, γ and n are shear strain and cycle numbers, respectively. The total amount of energy is considered as a measurement of the soil capacity against the initial liquefaction. The energy-based liquefaction approach is validated through laboratory testing or recorded field data. Numerous tests were performed to develop the energy-based models relating the energy capacity, confining pressure, strain amplitudes and soil initial parameters. Figueroa et al. (1994) conducted a series of tests on sands using a hollow-cylinder torsional shear device. They utilized from initial soil properties in order to establish a relationship relating the strain energy. Some energy-based formulations developed for liquefaction assessment were given in table 1. These statistical models were mostly generated by the multiple linear regression analysis. In recent years, different statistical methods such as ANNs, artificial neural networks and SVM, support vector machines have been considered in order to provide more reliable results. In this context, Chen et al. (2005) proposed an energy-based method by using back-propagation neural networks to assess the soil liquefaction. These statistical methods perform sufficiently well in the evaluation liquefaction probability due to their prediction performance. However, they have some limitations. A major restriction of artificial neural network is that it is not satisfactory for generating practical predictive equations. Besides, the network model is not variable and identified in advance (Alavi et al., 2011).

3. The Approach in the Proposed Method

Calculation steps of soil capacity and demand were summarized fallowing subsections. Both two parameters is required for assessment of liquefaction potential.

3.1. Evaluation of the Seismic Demand

The total energy (E, Joule) resulted from a quake is given by the equation of Gutenberg and Richter (1956):

$$E = 10^{4,8+1,5M} \tag{2}$$

Only a part of energy propagates along the site-source distance. It's some part will be scattered by inelastic attenuation and energy attenuation is possible due to geometric spreading. (Law et al., 1990). The energy (W) imparted by an earthquake on a unit of mass of matter (e.g., soil) is computed as follows;

$$W = \frac{1}{2} m v^2 \tag{3}$$

Where, m is mass of liquefiable soil layer and v is velocity. As mentioned above, the dissipated energy is expressed in the unit volume of the soil mass. The unit soil mass is numerical value of the saturated density since the volume is 1 unit. To determine the total amount of the energy imparted to liquefiable soil layer by an earthquake, strong motion acceleration-time history of any event needs to be obtained from accelerograms. Thus, Eq.3 is performed to obtain the cumulative energy versus the time.

The general procedure for the evaluation of the soil liquefaction is to compare two parameters; 1) the seismic demand and 2) the soil capacity to induce liquefaction. In the proposed method, the strain energy equations were performed to compute the capacity of the soil, as expressed by following section.

3.2. Evaluation of the Soil Capacity

The predictive energy equations in table 1 require the calculation of the initial soil parameters. An exact determination of σ'_{mean} in-situ is very difficult and the initial effective overburden stress (σ'_v) is commonly preferred rather than the initial mean effective stress, σ'_{mean} or P'_o which could be interchangeably related as shown below (Seed et al., 1986). Thus, σ'_{mean} is

Table 1- Empirical strain energy equations between the dissipated energy and soil parameters.

Equation	Researcher	Expression	r
(I)	Figueroa et al. (1994)	$\log(W) = 2.002 + 0.00477\,\sigma'_{mean} + 0.0116D_r$	0.97
(II)	Liang (1995)	$\log(W) = 2.062 + 0.0039\,\sigma'_{mean} + 0.0124D_r$	0.96
(III)	Dief and Figueroa (2001)	$\log(W) = 1.164 + 0.0124\,\sigma'_{mean} + 0.0209D_r$	0.97
(IV)	Baziar and Jafarian (2007)	$\log(W) = 2.1028 + 0.00456\,\sigma'_{mean} + 0.005685D_r + 0.001821FC - 0.02868C_u + 2.0214D_{50}$	0.80
(V)	Jafarian et al. (2012)	$W = 0.1363P'_o\,(D_r^{4.925}) + 5.375\,(10^{-3}P'_o)$	0.80

W: measured strain energy density required for triggering liquefaction (J/m³), P_o' and σ'_{mean}: initial effective mean confining pressure and initial mean stress (in kPa), D_r: initial relative density (%), FC : percentage of fines content, C_u : coefficient of uniformity and D_{50} = mean grain size (mm) expressed by effective overburden stress (σ'_v) and coefficient of lateral earth pressure (K_o);

$$\sigma'_{ort} = \left(\frac{1+2K_0}{3}\right)\sigma'_v \quad (4)$$

Where, K_o and ϕ' are obtained using the following expressions (Eqs 5 and 6), ϕ' is the effective angle of internal friction, which is expressed in term of $(N_1)_{60}$ (Hatanaka and Uchida, 1996).

$$K_o = 1 - \sin(\phi') \quad (5)$$

$$\phi' = (20N_{1,60})^{0,5} + 20 \quad (6)$$

On the other hand, the relative density, (D_r) is defined for natural soils as follows (Skempton, 1986):

$$D_r = \left(\frac{N_{1,60}}{55}\right)^{0.5} \quad (7)$$

Eq.7 is employed to compute the relative density for clean sands. Herein $(N_1)_{60}$ is derived for clean sands and it should be modified to take into account fines content (FC) to obtain an equivalent clean sand value, $(N_1)_{60cs}$, as follows (Youd et al., 2001):

$$(N_1)_{60cs} = (N_1)_{60} + exp\left[1{,}63 + \frac{9{,}7}{FC+0{,}1} - \left(\frac{15{,}7}{FC+0{,}1}\right)^2\right] \quad (8)$$

Eq.8 is employed to find the “equivalent” relative density for fine-grained soils. Once σ'_{mean} and Dr are computed with the expressions defined above, the strain energy (i.e. capacity) is calculated by using any of the Equations I-V in table 1.To evaluate the soil liquefaction, the proposed method considers only a comparison between demand and capacity.

4. Examination of the Procedure for the Assessment of Soil Liquefaction

4.1. Database for Past Earthquakes

The SPT-based database was compiled and updated by several researchers (Seed et al., 1984; Idriss and Boulanger, 2004, 2008 and 2010; Çetin et al., 2000, 2004 and 2016) for liquefaction correlation of cohesionless soils. The first database was presented by Seed et al. (1984) which contained only 125 cases. Then, the first large database was presented by Idriss and Boulanger (2004). It includes both the compiled and updated data of Seed et al. (1984) and -Çetin et al. (2000, 2004). The total number of evidence is 230 and surface evidences of liquefaction were observed in only 115 case histories. Additionally, some of these cases were not approved by Çetin et al. (2016). In the end, the data set updated by Çetin et al. (2016) contains 210 cases with consistent screening standards enforced throughout. The values of earthquake and soil parameters such as magnitude (M), maximum ground acceleration (a_{max}), depths to the relevant layer and ground water table, the total (σ_v) and effective vertical stress (σ'_v), SPT-blow counts (N), correction factors (e.g. C_E, C_R, C_B and C_S), $(N_1)_{60}$, fines content (FC) and $(N_1)_{60cs}$were presented in the updated data set of Çetin et al. (2016) for 20 major earthquakes.

This updated dataset was used to verify the proposed energy-based method in this investigation. For the calculation of the seismic demand on a soil layer, it is necessary to obtain the acceleration and velocity time histories of significant earthquakes. However, strong-motion data for many earthquakes prior to 1979 were not available. Therefore, only 115 cases with 9 major earthquakes in the data set of Çetin et al. (2016) were evaluated by using the existing acceleration records. The acceleration records are obtained from the seismic stations nearest to the sites of liquefaction/no liquefaction cases. Some information about seismic stations is given in table 2. The distances between the seismic stations and the sites of liquefaction/non-liquefaction cases are also given in table 3. Vertical (Up) acceleration component records of the ground motions are used to compute the cumulative energy of the earthquakes, because they can reach very high values at the surface close to the fault and compressive structural damage can occasionally be observed in the near field (Kunnath et al., 2008; Papazoglou and Elnashai, 1996; Riches, 2015; Tsaparli et al., 2016). High values was recorded during past earthquakes (e.g Northridge in 1994 and Kobe in 1995) where soil liquefaction events occurred (Shibata et al., 1996; Trifunac and Todorovska, 1996; Yasuda, 1996; Tsaparli et al., 2016). More recently, Canterbury earthquakes (New Zealand in 2010-11) are an important example for high records of vertical acceleration values and also soil liquefaction events (Bradley, 2012; Tsaparli et al., 2016).

Table 2- Strong motion stations and peak ground acceleration data for past major earthquakes.

Earthquake	Station			
	Code/ID	Lat/Long	Closest dist to epicenter (km)	PGA (cm/s^2)
1995 Hyogoken-Nambu (Kobe)	KJM (JMA, Japan Meteorological Agency)	34.6833/135.1800	1.5	336.13
1994 Northridge	Arleta Nordhoff Ave Fire Sta CGS - CSMIP Station 24087	34.2358/118.4398	9.5	539.39
1993 Kushiro-Oki	Kushiro Local Meteorological Observatory, JMA (KSR), Hokkaido	42.9786/144.3880	7.0	356.00
1989 Loma Prieta	CGS-47459 CGS-58483 CGS-58505 CGS-58117	36.9091/121.7575 37.7988/122.2582 37.9355/122.3434 37.8253/122.3739	17.0 89.0 105.0 97.0	647.00 42.00 29.00 20.00
1987 Superstition and Elmore Ranch	CGS-01336 CGS-11369	32.7735/115.4481 33.0370/115.6235	48.0 22.0	225.00 187.00
1981 Westmorland	CGS-11369	33.0370/115.6235	7.0	627.57
1979 Imperial Valley	CGS-01335	32.7933/115.5625	28.0	231.00
1971 San Fernando	C&GS241	34.2211/118.4711	22.0	167.00

4.2 Comparison of the Demand to the Capacity

The results of the strain energy imparted by strong ground motion (demand, W_{quake}) and the capacity of the soil to induce liquefaction (W_{liq}) are presented in table 3. The partial data in table 3, namely a_{max}, depths to layer of interest and water table, the total and effective vertical stresses on the layer, $(N_1)_{60}$, fines content (FC) and $(N_1)_{60cs}$ were taken from the updated database of Çetin et al. (2016). M_{sat} and M_t were deduced from the borehole logs and imported data in the database of Çetin et al. (2016). σ_{mean}, K_o, ϕ' and D_r were computed using Equations of 4-7. The capacities (W_{liq}) were calculated for 4 different empirical relationships (Equations I, II, III and V in Table 1) by employing the appropriate mean effective stresses and the relative densities. The ground motion records nearest to the site of interest were selected for the computation of demands.

To demonstrate how the proposed method works figure 2 was constructed, which covers field cases of 1995, Kobe/Hanshin (Hyogoken-Nanbu) Earthquake. It shows the acceleration-time history, velocity and the cumulative work for the first 2 cases (location #1 and 2 in table 3). The sites of no liquefaction cases were 15 km away from the epicenter of earthquake. The closest seismic station (KJM) to the site of cases was approximately 2.5 km. For these fields, the unit mass for the soil was taken as 1874 kg, which resulted in a demand of 2061 J/m^3 (Figure 2) when the Eq.3 was used along with the velocity time history. Since the demand for the station record is less than the soil capacities calculated using the 4 predictive equations, the liquefaction at these sites is verified from the perspective of the proposed method. Similarly, the location #5, as given in table 3, is a site of liquefaction in the same section. The unit mass of soil at this site is 1762 kg and thus the cumulative work or the demand is 1938 J/m^3, which is greater than all the capacities calculated using the 4 predictive equations. The comparison of the demand to capacity indicates that soil liquefaction should take place at location #5 as well.

The reliability and accuracy of the proposed method were examined for 115 cases in the data set of Çetin et al. (2016). The energy-based liquefaction method yields similar results with the stress-based liquefaction method for past events. Attempts have been made to provide more reliable seismic demand values by using near station records. However, the near station records for some case histories of past earthquakes are not available, and the use of the relatively far field ground motion records (>15km) resulted in a high seismic demand for a few non-liquefiable sites due to their site-to-source distances. The comparison between the demand values calculated from the nearest station and capacity values calculated for each site are given in figure 3. It shown that the results of the strain energy imparted by strong ground

Table 3- List of liquefiable/non-liquefiable cases used in this study (data source; updated data set of Çetin et al. (2016).

Location Name/Code	M	a_{max} (g)	Liquef.	Depth (m)	GWT (m)	σ_v	σ'_v	M_t (kg)	$(N_1)_{60}$	FC (%)	ϕ' (°)	K_o	σ'_{mean}	$(N_{1,60})_{cs}$	D_r (%)	W_{liq} (J/m³) a	b	c	d	W_{quake} (J/m³)* St.Code	Distance Rcls **(km)
16 Jan. 1995, Kobe/Hanshin (Hyogoken-Nanbu) Earthquake, Mw= 7.2 (Japan)																				KJM	
1	6.9	0.4	No	6	2.4	122	86	1874	53.2	3.5	52.9	0.2	40.3	54.1	110.3	2978	3865	9317	9125	2061	2.5
2	6.9	0.4	No	8.5	2.9	173	118	1922	39.6	14.8	48.9	0.25	58.7	41.9	97.1	2560	3125	8334	7232	2114	2.5
3	6.9	0.4	No	5.5	2.5	103	73	1601	52.8	3.3	52.8	0.2	34.2	53.7	109.9	2757	3619	7692	7622	1761	2.5
4	6.9	0.4	No	4.3	2.1	79	57	1762	40.4	1.3	48.7	0.25	28.4	41.2	96.28	1797	2327	3381	3370	1938	2.5
5	6.9	0.4	Yes	8.8	3	160	104	1762	7	1.3	32.2	0.47	67	7.5	41.08	628	680	714	474	1938	2
6	6.9	0.4	No	5.8	2.3	112	78	1601	21.9	24.7	42.4	0.33	43	25	75	1194	1444	1838	1651	1761	2
7	6.9	0.4	Yes	2.8	3.2	48	51	1762	22.3	0.1	41.4	0.34	28.5	23	71.94	938	1162	1049	920	1938	2
8	6.9	0.5	Yes	5	3	87	66	1601	24	0.1	42.2	0.33	36.5	24.6	74.4	1094	1339	1482	1354	1761	2
9	6.9	0.5	Yes	4.3	2.8	73	58	1681	12.4	2.3	36.1	0.41	35.2	12.9	53.87	624	737	533	418	1849	2
10	6.9	0.6	No	7.5	4.5	144	114	1922	26.2	8.8	43.4	0.31	61.8	27.4	78.52	1612	1890	3724	2891	2114	1
11	6.9	0.5	Yes	6.8	1.5	125	73	1762	7.7	5	32.8	0.46	46.6	8.2	42.95	528	598	436	350	1938	1.5
12	6.9	0.5	yes	5.3	3.2	92	71	1762	26.2	14	43.7	0.31	38.3	28.1	79.51	1280	1575	1998	1894	1938	1
13	6.9	0.5	Yes	6.5	2.3	124	83	1922	12	15	36.6	0.4	50	13.7	55.52	767	882	881	645	2114	1.5
14	6.9	0.5	No	4.8	3.1	89	73	1842	21	18.5	41.6	0.34	40.7	23.3	72.41	1087	1314	1520	1350	2026	1
15	6.9	0.5	Yes	5.8	3.7	99	78	1601	19.6	4.7	40.1	0.36	44.5	20.2	67.42	992	1179	1333	1109	1761	1.5
16	6.9	0.6	No	4.5	2.5	83	62	1762	24.6	5	42.4	0.33	34.1	25.2	75.3	1092	1345	1448	1333	1938	1
17	6.9	0.5	Yes	4.5	0.8	84	47	1762	21.2	5	40.9	0.35	26.5	21.8	70.04	873	1081	904	767	1938	2
18	6.9	0.7	No	11	7.7	211	183	2002	36.7	0.1	47.3	0.26	93.3	37.4	91.73	3243	3658	17289	8813	2202	1
19	6.9	0.6	No	7.5	6.1	142	129	1922	20.8	10	41	0.34	72.6	22.1	70.52	1466	1657	3447	2160	2114	1.5
20	6.9	0.6	No	6	2	119	80	1762	65.5	0.1	56.5	0.17	35.5	66.5	122.3	3895	5217	14496	13258	1938	2
21	6.9	0.6	No	3.5	1.7	62	44	1601	36.8	0.1	47.4	0.26	22.4	37.5	91.86	1494	1943	2300	2131	1761	2
22	6.9	0.6	No	6	2.4	116	81	1922	38.9	6	48.2	0.25	40.7	39.8	94.63	1968	2479	4435	4450	2114	2
23	6.9	0.6	No	5	3	96	76	1601	23.3	10	42.1	0.33	42	24.5	74.25	1158	1401	1724	1547	1761	2.5
24	6.9	0.5	Yes	3.5	2.4	64	53	1842	24.8	0.1	42.5	0.32	29.1	25.4	75.6	1042	1297	1273	1157	2026	2.5
25	6.9	0.7	No	3.5	2.2	64	50	1762	39	2.5	48.2	0.25	25.1	39.8	94.63	1658	2155	2842	2747	1938	2
26	6.9	0.6	No	3.5	0.9	65	40	1601	41.2	0.1	49	0.25	19.9	42	97.21	1677	2213	2768	2464	1761	2.5
27	6.9	0.6	No	2.5	1.1	45	31	1766	50.4	10	52.3	0.21	14.7	52.1	108.3	2127	2895	4060	3033	1943	2
28	6.9	0.4	Yes	4	1.8	74	52	1601	21.5	8	41.2	0.34	29.2	22.5	71.15	926	1143	1030	900	1761	4
29	6.9	0.4	Yes	3.8	2	69	52	1762	18	0.1	39.3	0.37	30	18.6	64.69	787	958	774	641	1938	4
30	6.9	0.6	No	8.5	1.5	161	92	1601	39	10	48.5	0.25	46.1	40.5	95.46	2134	2664	5378	5245	1761	3.5
31	6.9	0.6	No	4	1.2	74	46	1601	57	0.1	54	0.19	21.2	57.9	114.1	2673	3630	6488	5651	1761	2.5
32	6.9	0.5	No	3.5	1.4	63	43	1601	30.5	6.3	45.1	0.29	22.7	31.4	84.05	1217	1559	1593	1438	1761	2.5
33	6.9	0.5	No	8	2	149	90	1601	27.8	50	45.5	0.29	47.2	32.6	85.64	1662	2032	3459	3251	1761	4
34	6.9	0.4	Yes	7	1.8	132	81	1681	23.5	9.3	42.2	0.33	44.7	24.7	74.55	1202	1448	1890	1675	1849	4.5
35	6.9	0.5	Yes	4.5	2.1	84	60	1762	17.7	6.3	39.2	0.37	34.7	18.4	64.34	820	989	870	726	1938	4.5
36	6.9	0.6	No	3.5	0.9	65	40	1601	33.7	2.5	46.2	0.28	20.7	34.4	87.98	1323	1713	1819	1616	1761	4

Table 3- (continue).

Location Name/Code	M	a_{max} (g)	Liquef.	Depth (m)	GWT (m)	σ_v	σ'_v	M_t (kg)	$(N_1)_{60}$	FC (%)	ϕ' (°)	K_o	σ'_{mean}	$(N_{1,60})_{cs}$	D_r (%)	W_{liq} (J/m³) a	b	c	d	W_{quake} (J/m³)* St.Code	Distance **Rcls (km)
16 Jan. 1995, Kobe/Hanshin (Hyogoken-Nanbu) Earthquake, Mw= 7.2 (Japan)																				**KJM**	
37	6.9	0.4	Yes	5	4	95	85	1922	22.5	0.1	41.5	0.34	47.5	23.1	72.09	1161	1384	1816	1546	2114	5
38	6.9	0.5	Yes	8	3	152	103	1842	18.3	5	39.4	0.36	59.4	18.9	65.21	1101	1265	1833	1305	2026	5
39	6.9	0.6	No	4.5	2.6	89	71	1922	61.9	0.1	55.5	0.18	32	62.9	119	3425	4592	11149	10432	2114	5
40	6.9	0.6	No	3.5	2.8	67	60	1922	42.8	0.1	49.5	0.24	29.6	43.6	99.05	1959	2544	3988	4003	2114	4.5
41	6.9	0.4	Yes	4.1	2	71	51	1601	14.9	0.1	37.5	0.39	30.3	15.4	58.86	675	813	588	466	1761	5.5
42	6.9	0.4	Yes	5	1.2	92	55	1762	10.7	10	35.3	0.42	33.8	11.7	51.31	573	676	453	354	1938	5.5
43	6.9	0.4	Yes	4.7	2.2	82	57	1601	14.8	20	38.5	0.38	33.3	17.1	62.03	760	915	748	612	1761	6
44	6.9	0.4	Yes	4	1.6	69	45	1762	7.6	5	32.7	0.46	28.8	8.1	42.69	431	505	259	214	1938	6.5
Ashiyama A (Sand 1)	6.9	0.4	yes	5.2	3.5	94	77	1762	21.5	18	41.8	0.33	42.8	23.7	73.02	1131	1363	1663	1470	1938	1.5
Ashiyama A (M. Sand)	6.9	0.4	no	8	3.5	148	104	1762	30.6	2	45	0.29	55	31.3	83.92	1728	2075	3975	3454	1938	1.5
Ashi. C-D-E (Mnt Sand 2)	6.9	0.4	yes	13	3.5	247	154	1762	5.6	18	32.2	0.47	99.3	7.4	40.8	890	902	1773	698	1938	1.5
Ashi. C-D-E (M. Sand)	6.9	0.4	yes	8.8	3.5	164	112	1762	12.6	2	36.2	0.41	67.9	13.2	54.5	907	1006	1394	830	1938	1.5
Port Island BH Array St.	6.9	0.3	Yes	7.6	2.4	144	93	1762	7.1	20	33.5	0.45	58.8	9.1	45.25	642	712	690	477	1938	3.5
Port Is. Imp.St(Ikegaya)	6.9	0.4	No	8.5	5	155	121	1762	22.6	20	42.4	0.33	66.6	25.1	75.15	1554	1793	3635	2581	1938	4.5
Port Imp.St.(Tanahashi)	6.9	0.4	No	10	5	185	136	1762	19.1	20	40.8	0.35	76.8	21.6	69.71	1503	1682	3741	2183	1938	5
Port Imp St(Watanabe)	6.9	0.4	No	9.5	5	178	134	1762	32.8	20	46.7	0.27	69	35.6	89.5	2341	2760	7765	5817	1938	5.5
Port Island Site I	6.9	0.3	Yes	10	3	184	115	1762	11.1	20	36.2	0.41	69.7	13.2	54.5	926	1022	1469	852	1938	5.5
Rokko Island Building D	6.9	0.4	Yes	7.5	4	138	104	1762	17.6	25	40.3	0.35	59.2	20.6	68.08	1186	1371	2091	1532	1938	5.5
Rokko Island Site G	6.9	0.3	Yes	12	4	210	137	1762	12.5	20	37.1	0.4	81.8	14.7	57.51	1147	1243	2404	1172	1938	5.5
Torishima Dike	6.9	0.3	Yes	4.8	0	86	39	1762	15.2	20	38.7	0.37	22.7	17.5	62.75	689	849	572	435	1938	12
18 Jan. 1994, Northridge Earthquake, M=6.7 (USA)																				***ARLETA***	
Balboa Blv. Unit C	6,7	0,84	Yes	9	7,2	160	142	1762	19	48	41.6	0.34	79.2	23.3	72.41	1658	1856	4559	2625	2643	3
Potrero Canyon C1	6.7	0.4	Yes	6.5	3.3	120	88	1762	10.8	44	37.1	0.4	52.6	14.6	57.31	828	950	1034	745	2643	6
Wynne Ave Unit C1	6.7	0.5	Yes	6.3	4.2	112	92	1762	11.4	42.4	37.4	0.39	54.7	15.2	58.48	874	1001	1161	825	2643	11
15 Jan 1993, Kushiro-Oki Earthquake, M=7.6 (Japan)																				**KSR**	
Kushiro Port Quay Wall St. A	7.6	0.4	Yes	5.3	2	97	65	1681	16.5	2	38.5	0.38	38	17.1	62.03	800	954	855	698	2690	15
Kushiro Port Quay Wall Site D	7.6	0.4	No	11	1.6	210	120	1922	29.8	0.1	44.7	0.3	63.7	30.5	82.84	1849	2177	4849	3780	3075	15
Kushiro Port Seismo St.	7.6	0.5	Yes	3.6	2	66	50	1762	25.4	5	42.8	0.32	27.3	26.1	76.63	1050	1315	1272	1151	2819	15
18 Oct. 1989, Loma Prieta Earthquake, M= 6.9 (USA)																				**CGS-58483**	
Alameda Bay Farm Dike	6.9	0.2	No	6.5	3	125	91	2002	43.8	7	50	0.23	44.5	45	100.6	2408	3044	6594	6497	1401	6
POO7-2	6.9	0.3	Yes	6.2	3	181	88	1922	12.8	3	36.3	0.41	53.3	13.3	54.7	777	887	928	658	1345	1.5
POO7-3	6.9	0.3	yes	6	3	115	86	1922	16.4	5	38.4	0.38	50.4	17	61.85	911	1060	1205	915	1345	1.5
Farris Farm	6.9	0.4	Yes	6	4.5	99	84	1601	10.3	8	35	0.43	51.9	11.2	50.2	679	771	719	517	2081	1
SFOBB-1 & 2	6.9	0.3	Yes	6.3	3	120	89	1922	8.2	8	33.4	0.45	56.3	9	45	620	691	635	453	1345	10

Table 3- (continue).

Location Name/Code	M	a_{max} (g)	Liquef.	Depth (m)	GWT (m)	σ_v	σ'_v	M_t (kg)	$(N_1)_{60}$	FC (%)	ϕ' (°)	K_o	σ'_{mean}	$(N_{1,60})_{cs}$	D_r (%)	W_{liq} (J/m³) a	b	c	d	W_{quake} (J/m³)* St.Code	Distance **Rcls (km)
18 Oct. 1989, Loma Prieta Earthquake, M= 6.9 (USA)																				**CGS-58505**	
Hall Avenue	6.9	0.1	No	4.6	3.5	75	64	1601	5.1	30	32.6	0.46	41	8	42.43	489	560	362	302	320	2.5
POR-2 & 3 & 4	6.9	0.2	Yes	4.9	3.5	79	65	1601	3.5	50	31.7	0.48	42.3	6.8	39.12	454	515	320	284	320	2
																				CGS-58117	
Treasure Island	6.9	0.2	Yes	5.3	1.5	91	55	1601	7.7	20	34	0.44	34.5	9.8	46.96	514	601	374	299	190	0.2
																				CGS-47459	
MBARI No.3: EB-1	6.9	0.3	No	2.5	2	44	39	1762	22.9	1	41.7	0.34	21.7	23.5	72.72	889	1118	897	733	2291	10
MBARI No.3: EB-5	6.9	0.3	No	4.1	1.8	76	54	1762	17.8	1	39.2	0.37	31.3	18.4	64.34	790	959	788	654	2291	10
Miller Farm CMF3	6.9	0.4	Yes	6.6	5.7	117	108	1922	10.7	27.3	36.5	0.41	65.2	13.6	55.32	901	1005	1344	831	2499	5
Miller Farm CMF5	6.9	0.4	Yes	7	4.7	134	111	1922	20.6	13	41.1	0.34	62.4	22.2	70.68	1316	1519	2598	1874	2499	5
Miller Farm CMF8	6.9	0.4	Yes	6.5	4.9	107	91	1601	10	15.5	35.3	0.42	55.9	11.7	51.31	731	825	851	586	2081	5
Miller Farm CMF10	6.9	0.4	No	8.4	3	160	106	1762	20.1	20	41.3	0.34	59.4	22.6	71.31	1296	1506	2460	1850	2291	5
Sand holdt UC-B10	6.9	0.3	Yes	3.2	1.7	58	43	1762	14.3	2	37.2	0.4	25.7	14.8	57.71	622	754	488	371	2291	1.8
State Beach UC-B1	6.9	0.3	Yes	2.7	1.8	49	40	1762	7.9	1.7	33	0.46	25.5	8.4	43.47	425	502	245	194	2291	10
State Beach UC-B2	6.9	0.3	Yes	4.7	2.7	91	71	1922	17.4	1	39	0.37	41.2	18	63.64	865	1028	1012	828	2499	10
Wood Marine UC-B4	6.9	0.3	Yes	1.8	1	31	24	1762	8.3	35	35.4	0.42	14.7	11.9	51.74	470	577	268	157	2291	10
Wood Marine UC-B4	6.9	0.3	Yes	1.8	1	31	24	1762	8.3	35	35.4	0.42	14.7	11.9	51.74	470	577	268	157	2291	10
General Fish	6.9	0.3	No	2	1.7	36	32	1762	15.1	5	37.7	0.39	18.9	15.7	59.43	605	746	438	301	2291	12
M Laboratory UC-B1	6.9	0.3	Yes	4	2.4	72	57	1762	11.7	3	35.6	0.42	34.9	12.2	52.39	597	704	491	384	2291	10
M Laboratory UC-B2	6.9	0.3	Yes	3.5	2.5	63	53	1762	14.9	3	37.5	0.39	31.5	15.4	58.86	684	822	609	485	2291	10
M Laboratory F1-F7	6.9	0.3	Yes	4.6	1.5	89	59	1762	19.8	3	40.2	0.35	33.6	20.4	67.75	888	1079	993	854	2291	10
MBARI[B4/B5/EB2/ EB3]	6.9	0.3	No	5.1	2	98	67	1922	25.8	5	43	0.32	36.5	26.5	77.22	1180	1452	1701	1590	2499	10
Miller farm	6.9	0.4	No	6	4	108	89	1762	8.9	22	35	0.43	55	11.2	50.2	703	792	785	547	2291	5
23-24 Nov. 1987, Elmore Ranch and Superstition Hills Earthquakes Mw=6.2 and Mw=6.5 (USA)																				**CGS-11369**	
Radio Tower B1	6.2	0.1	No	4.3	2	72	50	1601	6.2	43.5	33.9	0.44	31.4	9.7	46.72	494	580	339	270	576	9
Wildlife B	6.2	0.1	No	4.7	0.9	86	49	1601	11.3	26.2	36.8	0.4	29.4	14.1	56.32	625	750	508	396	576	14
																				CGS-01336	
Heber Road A1	6.5	0.2	No	3.4	1.8	60	44	1601	46.5	13	51.2	0.22	21.1	48.7	104.7	2075	2770	4110	3722	256	6
Heber Road A2	6.5	0.2	No	3.2	1.8	53	39	1601	3.5	20.9	30.5	0.49	25.8	5.5	35.18	341	397	166	159	256	6
Heber Road A3	6.5	0.1	No	3.4	1.8	57	42	1601	18.6	25.3	40.8	0.35	23.7	21.7	69.87	842	1049	828	680	256	6
Mc Kim Ranch A	6.5	0.2	No	2.7	1.5	49	37	1762	7.9	19.8	34.1	0.44	23.2	10	47.43	460	550	277	205	282	7
																				CGS-11369	
Radio tower B1	6.5	0.2	No	4.3	2	72	50	1601	6.2	43.5	33.9	0.44	31.4	9.7	46.72	494	580	339	270	576	9
Radio tower B2	6.5	0.2	No	2.5	2	41	36	1601	16.6	18	39.3	0.37	20.8	18.7	64.87	714	886	599	448	576	9
Kornbloom B	6.5	0.2	No	4	2.7	68	55	1601	7.1	83	34.6	0.43	34.2	10.6	48.84	539	632	406	320	576	9
River Park A	6.5	0.2	No	1.1	0.3	18	10	1601	4	91	32.1	0.47	6.46	7.3	40.53	318	389	123	45	576	9
River Park C	6.5	0.2	No	4.3	0.3	76	38	1601	19.9	13.2	40.7	0.35	21.5	21.5	69.55	815	1019	765	605	576	9
Wildlife B	6.5	0.2	Yes	4.7	0.9	86	49	1601	11.3	26.2	36.8	0.4	29.4	14.1	56.32	625	750	508	396	576	17

Table 3- (continue).

Location Name/Code	M	a_{max} (g)	Liquef.	Depth (m)	GWT (m)	σ_v	σ'_v	M_t (kg)	$(N_1)_{60}$	FC (%)	ϕ' (°)	K_o	σ'_{mean}	$(N_{1,60})_{cs}$	D_r (%)	W_{liq} (J/m³) a	b	c	d	W_{quake} (J/m³)* St.Code	Distance **Rcls (km)
26 Apr. 1981, Westmorland Earthquakes, M=5.9 (USA)																				**CGS-11369**	
Kornbloom B	5.9	0.3	Yes	4	2.7	68	55	1601	7.1	83	34.6	0.43	34.2	10.7	49.07	542	636	410	323	256	9
Radio Tower B1	5.9	0.2	Yes	4.3	2	72	50	1601	6.2	43.5	33.9	0.44	31.4	9.7	46.72	494	580	339	270	256	9
Radio Tower B2	5.9	0.2	No	2.5	2	41	36	1601	16.6	18	39.3	0.37	20.8	18.7	64.87	714	886	599	448	256	9
River Park A	5.9	0.2	No	1.1	0.3	18	10	1601	4	91	32.1	0.47	6.46	7.3	40.53	318	389	123	45	256	9
River Park C	5.9	0.2	No	4.3	0.3	76	38	1601	19.9	13.2	40.7	0.35	21.5	21.5	69.55	815	1019	765	605	256	9
Wildlife B	5.9	0.3	Yes	4.7	0.9	86	49	1601	11.3	26.2	36.8	0.4	29.4	14.1	56.32	625	750	508	396	256	17
15 Oct. 1979, Imperial Valley Earthquakes, Mw=6.5 (USA)																				**CGS-01335**	
Heber Road A1	6.5	0.8	No	3.4	1.8	60	44	1601	46.5	13	51.2	0.22	21.1	48.7	104.7	2075	2770	4110	3722	8005	15
Heber Road A2	6.5	0.8	Yes	3.2	1.8	53	39	1601	3.5	20.9	30.5	0.49	25.8	5.5	35.18	341	397	166	159	8005	15
Heber Road A3	6.5	0.8	No	3.4	1.8	57	42	1601	18.6	25.3	40.8	0.35	23.7	21.7	69.87	842	1049	828	680	8005	15
Kornbloom B	6.5	0.1	No	4	2.7	68	55	1601	7.1	83	34.6	0.43	34.2	10.6	48.84	539	632	406	320	8005	25
Mc Kim Ranch A	6.5	0.5	Yes	2.7	1.5	49	37	1762	7.9	19.8	34.1	0.44	23.2	10	47.43	460	550	277	205	8810	17
Radio Tower B1	6.5	0.2	Yes	4.3	2	72	50	1601	6.2	43.5	33.9	0.44	31.4	9.7	46.72	494	580	339	270	8005	24
Radio Tower B2	6.5	0.2	No	2.5	2	41	36	1601	16.6	18	39.3	0.37	20.8	18.7	64.87	714	886	599	448	8005	24
River Park A	6.5	0.2	Yes	1.1	0.3	18	10	1601	4	91	32.1	0.47	6.46	7.3	40.53	318	389	123	45	8005	20
09 Feb. 1971, San Fernando Earthquake, Mw= 6.6 (USA)																				***C&GS-241***	
Juvenile Hall	6.6	0.5	Yes	5.4	4.3	94	84	1762	3.7	65.3	31.9	0.47	54.4	7.1	39.97	531	589	472	373	5568	5
Van Norman	6.6	0.5	Yes	6.2	5	110	97	1762	7.9	59.3	35.2	0.42	59.8	11.5	50.87	754	843	929	613	5568	5

Equations for Wlig (J/m³); a) Figueroa et al., 1994, b) Liang, 1995, c) Dief and Figueroa, 2001 and d) Jafarian et al., 2012

*This study [W=0,5+(m*v²)] (Joule/m³) and Station Code (Exm.; KSR Kushiro Local Meteorological Observatory),

** Closest distance to station.

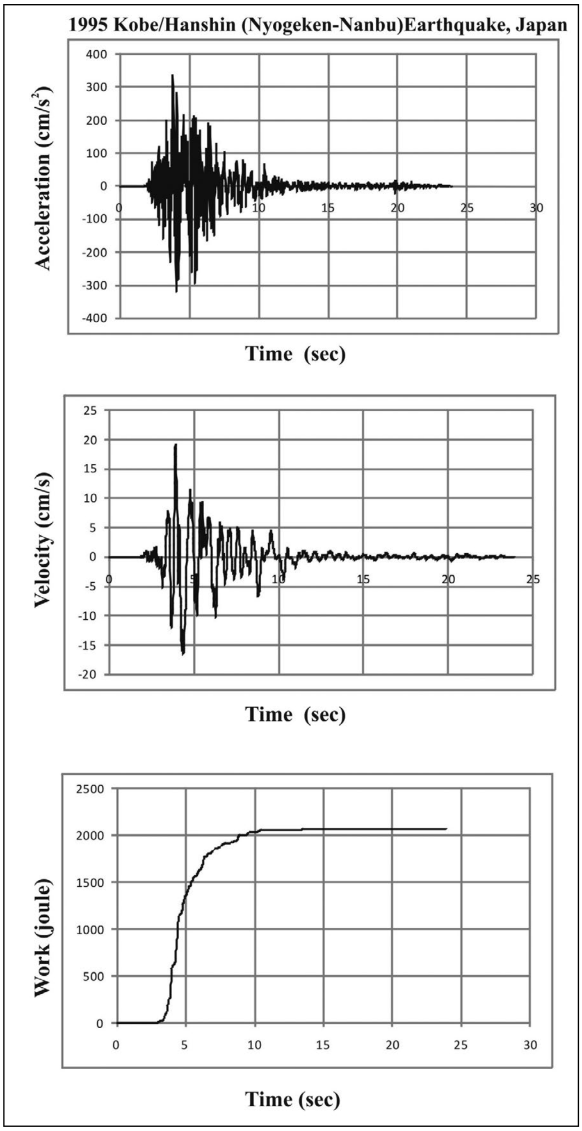

Figure 2- Time histories of acceleration, velocity and work for the 1995, Kobe/Hanshin (Hyogoken-Nanbu) Earthquake (Mw=6.9).

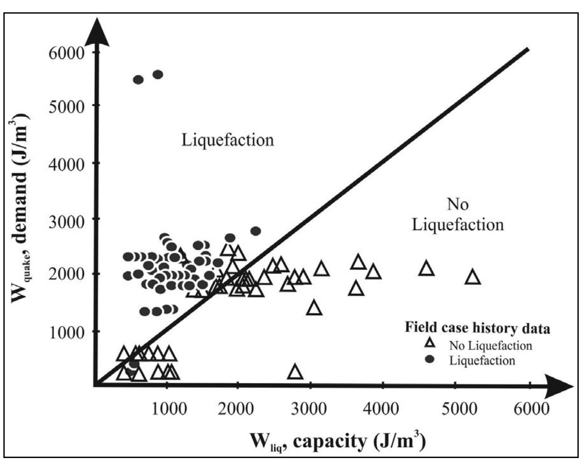

Figure 3- The comparison between the demand and capacity values for liquefiable/non liquefiable cases.

motion (demand, W_{quake}) and the soil capacity (W_{liq}) for liquefaction/no liquefaction case histories are quite consistent within the proposed method. The demand energy is larger than the capacity energy of the soil for liquefaction sites, or vice versa for non-liquefiable sites. Herein, the capacity values were calculated by using the equation of Liang (1995). Other models also provide partially same results. On the other hand, the method provides much more consistent results within the field observations of case histories. For example, the data base of Çetin et al. (2016) indicated that there is no surface evidence of liquefaction in the site of Treasure Island (Case#139) for 1989 Loma Prieta Earthquake. This observation is consistent with the result of the proposed method, that is, the demand for the station record is less than the soil capacities. For the stress-based approach, the liquefaction in this site was expressed by the change in the frequency of the ground motion (Çetin et al., 2016). Treasure Island is about 97 km away from the epicenter of the Loma Prieta Earthquake. Idris and Boulenger (2010) estimated the peak ground acceleration value to be 0.16g, whereas Çetin et al. (2016) suggested this value be 0.180 ± 0.027. These differences are even wider for some other historical cases. Considering that the ground acceleration values in the database compiled by different researchers are not consistent with each other and peak ground acceleration values are estimated using ground motion prediction models despite uncertainties still involved in these approaches, the proposed energy-based method seems to be much more capable for far-field ground motions in evaluating liquefaction potential with regards to the main earthquake parameters, such as magnitude and seismic source distances; contrary to the stress-based approach, where only peak ground acceleration value is considered.

5. Discussion

The reliability of proposed method was examined by utilizing a large database compiled and updated by several researchers. The results between the capacities and demands indicated that the proposed method appears to work in a reasonably good success. However, the seismic demands for some sites within the database were not checked by the near station records due to lack of available data. For these sites, the seismic demand values were calculated from far field station records (>15 km). The use of these records provided partially high seismic demand values, due to their relatively close distance to source.

In this study, vertical acceleration records were used to determine the seismic demand values, because high vertical ground accelerations have been mostly recorded in past earthquake events and liquefaction events were observed at these sites, as given table 2 and 3. Bradley (2012) stated that there may be a relationship between the high vertical components of acceleration and soil liquefaction for the 2010-2011 Canterbury earthquake sequence in New Zealand. Extensive liquefaction and re-liquefaction of sandy deposits were observed at Christchurch and Compressive structural damage was evident due to the high vertical accelerations registered with peak surface amplitudes well exceeding a value of 1g (Riches, 2015; Lee et al., 2013; Tsaparli et al., 2016).

Some limitations of the energy-based method are associated with the computation of the capacity values. The predictive equations are based upon a series of laboratory tests and are within their specific data ranges; these generally allow the calculation of the liquefaction energy by employing basic soil parameters. However, these also ignore many soil criteria controlling the liquefaction probability (e.g. percentage of fines content (FC) and grains shape, D_{50}, C_u, etc.).

The liquefaction/non-liquefaction database indicated that the Dief and Figueroa (2001) and Jafarian et al. (2012) predictive equations result in greater capacities than those of Figueroa et al. (1994) and Liang (1995). Although some relationships relating some of soil initial parameters to energy capacity were developed by researchers, they were not utilized due to the limited parameters in the data set for the past earthquakes. Thus, the failure of the proposed method for past significant earthquakes is attributed to these uncertainties.

Yet still, considering that the stress and strain based approaches all have the requirement of determination of the a_{max} for a given site, the energy-based procedure has a clear advantage over those. Even though it is possible to utilize some attenuation relationships, those require correct employment of magnitude and distance values; which bring along some uncertainties like the type of the distance to be utilized (causative fault, hypocentral, epicentral distances) and the attenuation relationship itself. Furthermore, a_{max} at the depth of bedrock needs to be converted to the a_{max} of the surface level for the response analysis of a given site, even for an educated assumption. However the energy-based approach does not need this transformation as the total amount of energy passing through in a soil will remain the same. Finally, the stress and strain based approaches utilize one more parameter that is open to debate between researchers; the average shear stress value is assumed to be 65% for these methods, but some researchers claim that this value may not be correct. In the energy-based approach, on the other hand, no such coefficient is utilized, not even the depth correction and r_d corrections utilized by the stress and strain methods.

The ground acceleration values in the database compiled by different researchers were found to be inconsistent with each other. Besides, as mentioned before, peak ground acceleration values are mostly estimated from ground motion prediction models, despite uncertainties still involved in these models. The authors believe that these limitations in the assessment of liquefaction cannot be properly addressed without adequate consideration of seismological data and in-situ characterization of liquefiable fields. The proposed method seems to be much more capable for far-field ground motions in assessment of liquefaction, contrary to the stress-based approach, where only peak ground acceleration value is considered. This simplified method can be used for assessing the liquefaction potential at any sites by providing the near station records and in-situ characterization of soils.

6. Conclusion

A simplified energy-based approach for determination of soil liquefaction was presented. The proposed method was evaluated using a large database delineated by the previous researches. As a result, the proposed method was found to have great utility in making quick assessments of the liquefaction potential, using only the in-situ data and seismological records. The observations in liquefaction/non liquefaction sites are mostly consistent with the results of the calculations of the proposed method. The near station ground motion records provide reliable results in order to determine seismic demand values. In case of the use of long distance records for a project site, on the other hand, high seismic demand values for non-liquefiable sites may emerge due to their distances to the source.

The borehole data of sites for liquefaction/non-liquefaction case histories were employed to compute the capacity energy values foreseen by predictive strain energy equations. Results indicate an acceptable performance of the equations to determine the capacity energy values of soils. Comparisons between demand and capacity energies confirmed the hypothesis of the method as well. The demand energy is larger than the capacity energy of the soil for liquefaction sites, or vice versa for non-liquefiable sites.

Different results for capacity values are likely to be obtained from these predictive energy equations, though, since the reliability and the accuracy of the derived equations are high only within their specific data ranges used. Although some other relationships relating soil initial parameters to energy capacity were developed by researchers, they were not utilized due to the limited parameters in the data set for the past earthquakes. The partial failure of the proposed method for those past significant earthquakes is thus attributed to these uncertainties. There is no doubt that the proposed method can be used for pre-design purposes, after checking more actual case histories to demonstrate its accuracy and reliability.

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