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ABSTRACT

The paper focuses on loose clean and silty sand fills that liquefied during the 1989 Loma Prieta, California earthquake. Available field case histories of liquefaction that include measured shear wave velocity from the Andrus et al. (2003) database are used. The liquefaction behavior observed in these field case histories is compared with the results of two large scale and six centrifuge shaking tests conducted by the authors. System identification and site response analyses are used to obtain the corresponding cyclic shear stress ratios in the tests. Due consideration is given to the shaking duration and 1D versus 2D shaking in the laboratory and field. The comparison between field and shaking tests is very good, with both case histories and shaking tests validating well the Andrus and Stokoe (2000) liquefaction chart for M_w = 7.0. This agreement also serves to validate the large scale and centrifuge testing techniques presented, as tools that can be used toward improved methods for liquefaction evaluation and mitigation of sandy fills.

INTRODUCTION

Over the last century, liquefaction of saturated sands has consistently been a significant cause of damage in dozens of earthquakes throughout the world. Recent artificial fills and natural soil deposits have been affected, with both uncompacted fills and Holocene loose alluvial and fluvial sites being the most susceptible to liquefaction (Youd and Hoose, 1977; Youd and Perkins, 1978; Seed, 1979; Youd et al., 2001).

The 1989 Loma Prieta, $M_w \approx 7.0$ earthquake, which occurred in the Santa Cruz Mountains about 100 km south of San Francisco, caused extensive liquefaction and associated damage in the San Francisco-Monterey Bay region of Northern California (Fig. 1). This confirmed the high liquefaction susceptibility of many soils in the region, which had already been demonstrated by the 1906 San Francisco earthquake (Lawson, 1908; O'Rourke et al., 1992; Holzer, 1998). The liquefaction phenomenon during the Loma Prieta event has been extensively documented and studied in many articles and reports, with detailed summaries provided by Seed et al. (1990); EERI (1990); O'Rourke and Pease (1992); and Stewart (1997); as well as by two collections of papers on liquefaction during this earthquake published by the U.S. Geological Survey, USGS (1992, 1998).

Both the 1906 and 1989 events showed the extreme vulnerability to liquefaction and permanent ground deformation of the artificial fills placed in the 19th and 20th centuries along the shoreline of the city of San Francisco, and of similar fills in Treasure Island and the East Bay along the Berkeley-Oakland shorelines (Holzer, 1998). This vulnerability to liquefaction of artificial fills in and around San Francisco is a great cause of concern for future earthquakes. A main motivation for the studies of the 1989 liquefaction case histories, has been to calibrate as well as possible the evaluation methods to predict future liquefaction, as well as the mitigation techniques used to improve these dangerous fills (Mitchell and Wentz, 1998).

The authors have conducted in the last few years a series of centrifuge and large scale shaking experiments of saturated loose sand deposits, which in many respects simulate the conditions of recent uncompacted fills such as those that liquefied in 1989 during the Loma Prieta event (Gonzalez, 2008; Thevanayagam et al., 2009; Dobry et al., 2011; Abdoun et al., 2012). The purpose of this paper is to validate the applicability of these experimental techniques to the San Francisco Bay Area fills - as well as to similar fills around the world - by direct comparison with San Francisco Bay Area uncompacted artificial fills that experienced liquefaction in 1989. This is done using as main tool the liquefaction chart based on field measurements of the shear wave velocity of the

Table 1: Summary Information for V_s-Based Liquefaction and Non-Liquefaction Case Histories of Loose Clean and Silty Sand Fills from the 1989 Loma Prieta Earthquake (modified after Andrus et al., 2003).

(1)	(2)	(3)	(4)	(5)	(6)	(7)
Case History No.	**Site**	**$(a_{max})_{avg}$ (g)**	**Liquefaction? Y=1; N=0**	**Nonplastic Fines Content, FC (%)**	**$V_{S1}^{(1)}$ (m/sec)**	***CSR***
134	Bay Farm Island, Loop	0.27	1	< 12	142	0.18
135	Bay Farm Island, Loop S-R1	0.27	1	< 12	107	0.19
136	Bay Farm Island, Loop R1-R2	0.27	1	< 12	124	0.19
142	Marina District, No. 2	0.15	1	~ 8	129	0.12
143	Marina District, No. 3	0.15	1	~ 12	113	0.12
144	Marina District, No. 4	0.15	1	< 5	137	0.11
146	Marina District, school	0.15	1	2	130	0.11
147	Port of Oakland, POO7-1	0.24	1	<5	152	0.21
148	Port of Oakland, POO7-2	0.24	1	<5	165	0.21
149	Port of Oakland, POO7-2	0.24	1	<5	155	0.21
150	Port of Oakland, POO7-2, S-R1	0.24	1	<5	152	0.21
151	Port of Oakland, POO7-2, R1-R2	0.24	1	< 5	186	0.21
152	Port of Oakland, POO7-3	0.24	1	10	185	0.21
178	Tl Fire Station, Redpath	0.13	1	24	142	0.14
179	Tl Fire Station, Gibbs et al.	0.13	1	24	150	0.14
180	Tl Fire Station, 1992	0.13	1	24	149	0.14
181	Tl Fire Station	0.13	1	24	152	0.13
182	Tl Fire Station, B1-B4	0.13	1	24	155	0.14
183	Tl Fire Station, B2-B3	0.13	1	24	146	0.14
184	Tl Fire Station, B2-B4	0.13	1	24	146	0.14
185	Tl Fire Station, B4-B5	0.13	1	24	148	0.14
186	Tl Fire Station, Portable	0.13	1	24	145	0.14
188	Tl Perimeter, UM05	0.14	1	5	169	0.12
189	Tl Perimeter, UM06	0.14	1	5	157	0.12
190	Tl Perimeter, UM09	0.14	1	14	160	0.11
194	Bay Bridge Toll Plaza, S-R1	0.24	1	~ 9	150	0.21
195	Bay Bridge Toll Plaza, R1-R2	0.24	1	~ 9	151	0.21
196	Bay Bridge Toll Plaza, SFOBB-1	0.24	1	~ 9	155	0.21
197	Bay Bridge Toll Plaza, SFOBB-2	0.24	1	~ 13	151	0.22

Notes:
$^{(1)}V_{S1}=V_S(100/\sigma'_{vo})^{0.25}$

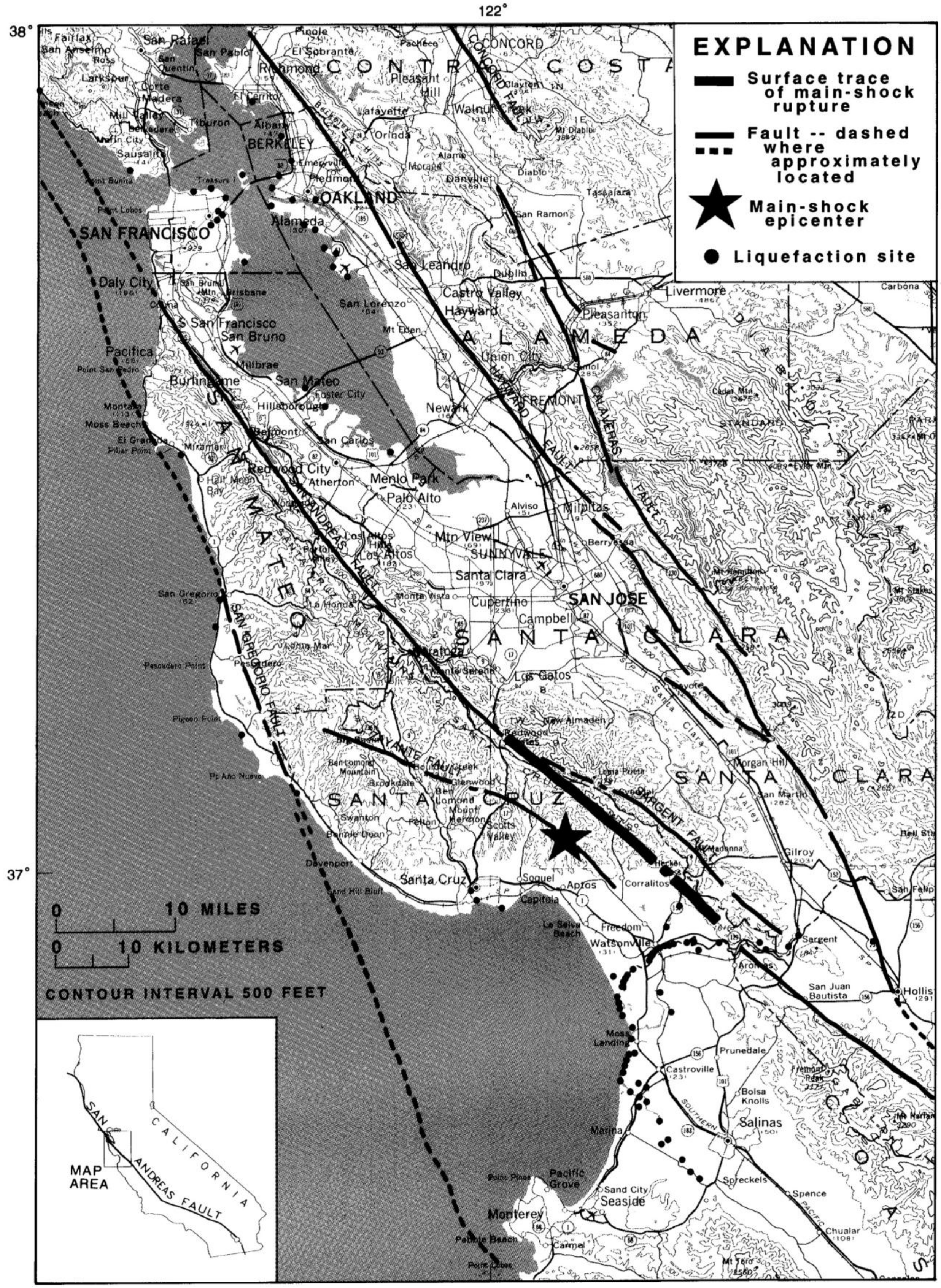

Fig. 1. San Francisco-Monterey Bay region, showing locations of sand boils, lateral spreads, and significant ground settlements associated with liquefaction caused by 1989 Loma Prieta Earthquake of $M_W = 7.0$ (Holzer, 1998).

liquefiable sand proposed by Andrus and Stokoe (2000) on the basis of the Seed and Idriss (1971) Simplified Procedure (see Fig. 2). This chart was originally calibrated by 225 case histories of liquefaction and no liquefaction of sands, silts and gravels around the world (Andrus et al., 2003), which included 29 artificial fills that liquefied in the Loma Prieta earthquake (Table 1).

This paper focuses on the comparison between the centrifuge/large scale tests conducted by the authors and listed in Tables 2-3, and the Loma Prieta case histories of Table 1, using as main tool the Andrus-Stokoe chart of Fig. 2. It is expected that this validation of the centrifuge and large scale testing techniques may be helpful in future uses of similar testing toward improved methods of evaluation and mitigation, for both the fills in San Francisco as well as other similar sandy fills around the world.

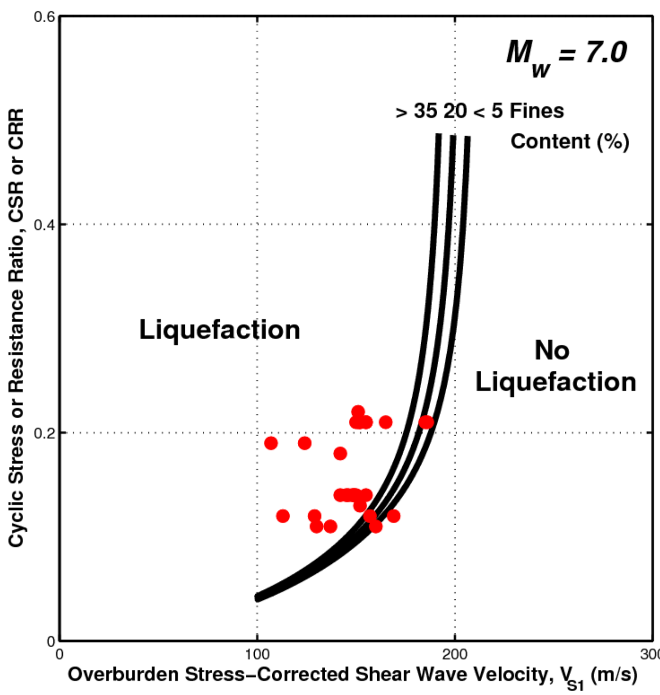

Fig. 2. Andrus and Stokoe (2000) liquefaction chart (curves), and case histories of loose clean and silty sand fills in the 1989 Loma Prieta earthquake from Table 1 (data points).

THE USGS (1998) REPORT

This United States Geological Survey (USGS) report on liquefaction during the 1989 Loma Prieta earthquake, was issued almost a decade after the event. It consists of a compilation of papers by experts that contain in-depth follow-up studies, hence supplementing the initial reconnaissance reports that focused mostly on documenting the case histories. About half of the papers in the report deal with liquefaction of uncompacted fills, while the rest focus on natural sites and improved fills. Most relevant to this paper herein, are the comments on the significance of artificial fill liquefaction in the Bay Area, contained in the introductory summary provided by Holzer (1998), who coordinated the effort:

"The investigation of the great 1906 San Francisco earthquake (Lawson, 1908) documented extensive permanent ground deformation in fills that had been placed along the shoreline of the city of San Francisco and in the loose, sandy deposits of the major streams of the San Francisco–Monterey Bay region… Despite these observations, much of the fill that was placed into San Francisco Bay after 1906 was sited without concern to its seismic stability.… The most significant damage from liquefaction occurred along an arc approximately 98 km from the epicenter or 84 km from the end of the seismic source zone, in hydraulic fills placed into San Francisco Bay: in the Marina District, on Treasure Island, and along the Oakland-Berkeley shoreline… the bad news is that liquefaction damage caused by the 1989 Loma Prieta earthquake is one more reminder of the seismic hazard posed by the many loose sandy fills around the margins of San Francisco Bay and the natural deposits that underlie the stream valleys."

"Seven of the case histories in this paper address the stability of the tens of millions of cubic meters of loose fills that have been placed into San Francisco Bay since 1845 to reclaim more than 40 km^2 of tidal and submerged land. These case histories describe liquefaction and subsurface investigations of loose, sandy parts of these fills in the bay near and in San Francisco and Oakland.…These investigators confirm that areas of liquefaction in the 1989 Loma Prieta earthquake were underlain predominantly by loose sandy fills, much of which were hydraulically placed."

"An ominous concern expressed by most of the authors… is the continuing vulnerability of these liquefiable deposits to future earthquakes. These investigators are particularly concerned about the hydraulic fills that have been placed into San Francisco Bay on which industrial or residential development has occurred."

LARGE SCALE AND CENTRIFUGE SHAKING TESTS

The results of the recent series of eight large scale and centrifuge shaking tests on clean and silty sand deposits reported by Gonzalez (2008) and Abdoun et al. (2012), offers the possibility of providing additional validation to the liquefaction chart for loose clean and silty sand fills of Fig. 2. They also allow validation of the large scale and centrifuge testing techniques used, as appropriate tools to investigate recently deposited sandy fills in the field. The eight tests are listed in Table 2; they include two large scale experiments conducted at the University at Buffalo (UB), and six centrifuge experiments performed at Rensselaer Polytechic Institute (RPI). Figure 3 is a sketch of the large laminar box at UB used in the two large scale tests. All eight experiments simulated a 5-6 m submerged sand deposit which is either horizontal or mildly sloping. Clean fine Ottawa sand with essentially no fines, and deposited by hydraulic filling, was used for the two large scale tests. Either the same Ottawa sand or a silty sand with 21% nonplastic fines - labeled Scaled Sand in Table 2 – were used in the centrifuge tests, with the sand deposited by dry pluviation in all six centrifuge experiments. The use of a clean sand and a silty sand with 21% fines content, as well as the use of hydraulic filling and dry pluviation methods of deposition in these experiments, are all reasonably consistent with the features of the Loma Prieta field case histories included in Table 1 and plotted in Fig. 2. The prototype permeability of the soil in all eight tests was about that of a fine sand; this was accomplished by using a viscous pore fluid in the centrifuge experiments with Ottawa sand, and water as pore fluid in the Scaled Sand centrifuge experiments (Abdoun et al., 2012).

In the first phase of the shaking, lasting 5 seconds, all eight large scale and centrifuge model deposits were subjected to about ten cycles of lateral base uniform shaking with a peak input acceleration ranging from 0.014g to 0.176g (Table 2). At the end of this 10-cyle phase, in six of the tests, the recorded excess maximum pore water ratios, $(r_u)_{max}$, ranged between 0 (no excess pore pressures) and 0.7 (significant pore pressure generation short of full liquefaction, defined as $(r_u)_{max} \approx 1.0$), see Table 2. On the other hand, in centrifuge Tests PF-V1 and PF-P1, the soil was liquefied at the end of the 10 cycles, with $(r_u)_{max} = 1.0$ in both cases. Profiles of shear wave velocity versus depth were obtained in the tests by Abdoun et al. (2012), using System Identification (SI), which allowed determination of the normalized shear wave velocity, V_{s1}, for each of the experiments. The corresponding values of V_{s1} are reproduced in Table 2; V_{s1} varies from 119 to 174 m/s, covering approximately the same range of V_{s1} for liquefiable loose sandy fills in the field exhibited by Table 1 and Fig. 2.

The input acceleration records for three of the eight tests are presented in Fig. 4. Figure 4 also reproduces from the original Gonzalez (2008) and Abdoun et al. (2012) publications, the time histories of cyclic shear stress obtained using SI for locations close to mid-depth in the same tests. These cyclic stress histories (as well as the profiles of V_s versus depth mentioned before), were extracted from the measured lateral accelerations using the SI method developed by Elgamal et al. (1995, 1996) and Zeghal et al. (1995). In Tests LG-0 and FF-V3, where the sand deposit did not liquefy, the shear stress histories of Fig. 4 consist of about ten cycles of similar amplitude. On the other hand, in Test PF-P1, the shear stresses in Fig. 4 drop almost to zero after about 2 seconds due to the liquefaction of the soil.

Table 2. Soil Testing Parameters and Pore Pressure Response In Large Scale And Centrifuge Tests at 25g Simulating a 5-6m Thick Saturated Sand Deposit (Gonzalez, 2008; Abdoun et al., 2012).

Test	*Test Type* (1)	*Prototype Deposit Thickness (m)*	*Test Inclination* (2)	*Sand*	*Fines Content, FC (%)*	*Relative Density, D_r (%)*	*Deposition Method*(3)	*Normalized Shear Wave Velocity, V_{s1} (m/s)*(4)	*Input a_{max} (g)*	*Maximum Pore Pressure Ratio, $(r_u)_{max}$ after 5 seconds*	*Maximum Pore Pressure Ratio, $(r_u)_{max}$ after 2.5 seconds*
SG-1	LS	5.6	SG	Ottawa	0.1	40	HF	119	0.017	0.7	0.18
FF-P1	C	6.0	SG	Scaled Sand	21	50	DP	140	0.014	0	0
FF-P2	C	6.0	SG	Scaled Sand	21	35	DP	132	0.019	0.38	0.19
FF-V1	C	6.0	SG	Ottawa	0.1	40	DP	174	0.015	0	0
FF-V3	C	6.0	SG	Ottawa	0.1	40	DP	174	0.041	0.27	0.08
LG-0	LS	4.85	LG	Ottawa	0.1	40	HF	124	0.020	0.25	0.17
PF-V1	C	6.0	SG	Ottawa	0.1	40	DP	174	0.120	1.0	0.93
PF-P1	C	6.0	SG	Scaled Sand	21	50	DP	140	0.176	1.0	1.0

Notes:

(1) LS = large scale test; C = Centrifuge test
(2) SG = sloping ground (2^0 test angle); LG = level ground
(3) HF = Hydraulic Fill; DP = Dry Pluviation
(4) V_{s1} = shear wave velocity, V_s, at a vertical effective overburden pressure, σ'_{v0} = 1 atmosphere = 101.33 kPa

Table 3. Cyclic Shear Stresses and Strains at Mid-Depth from System Identification (SI) in Large Scale and Centrifuge Tests (Abdoun et. al., 2012), Supplemented by Site Response Analyses Using Program SHAKE (this work).

				From System Identification		*From SHAKE Analysis*	
Test	*Normalized Shear Wave Velocity, V_{SI} (m/s)*	*Liquefaction? Y = 1; N = 0*	*Depth, z (m)*	*Cyclic Shear Stress, $\tau_c = \tau_{10}$ (kPa)*	*CSR = τ_c/σ'_{v0}*	*Cyclic Shear Stress, $\tau_c = \tau_{10}$ (kPa)*	*CSR = τ_c/σ'_{v0}*
SG-1	119	0	2.85	1.05	0.039	1.26	0.047
FF-P1	140	0	2.5	1.09	0.044	1.23	0.050
FF-P2	132	0	2.4	1.68	0.075	1.11	0.050
FF-V1	174	0	2.4	0.90	0.040	0.88	0.039
FF-V3	174	0	2.5	2.47	0.104	2.72	0.115
LG-0	124	0	2.3	1.16	0.053	1.13	0.052
PF-V1	174	1	2.25	-	-	6.77	0.317
PF-P1	140	1	2.25	-	-	14.93	0.678

(1) CSR reduced by 10% to account for 2D shaking in the field.
(2) $(CSR)_{2D}$ corresponding to 10 cycles of shaking and an approximate earthquake magnitude, M_w=7.0

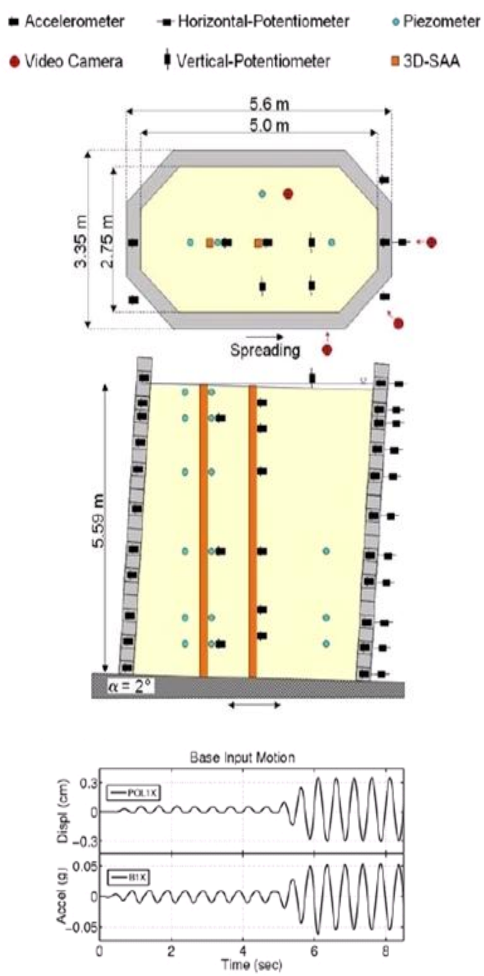

Fig. 3. Laminar box at the University at Buffalo (UB) used for the large scale tests (Thevanayagam et al., 2009).

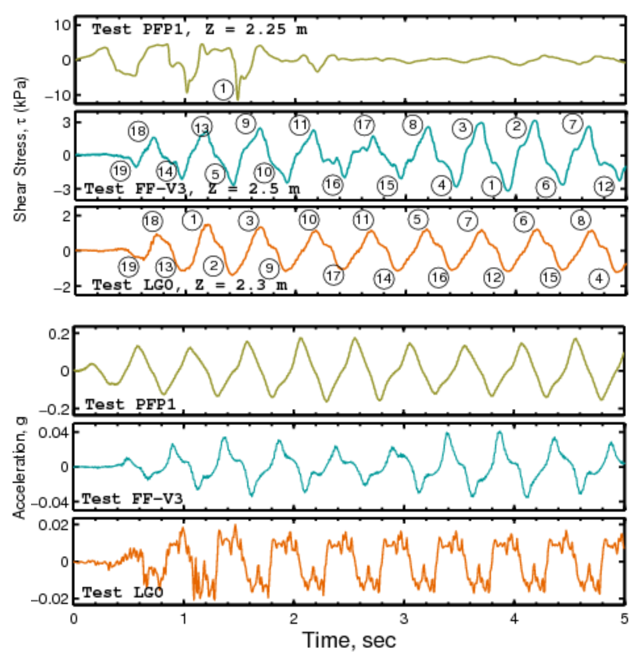

Fig. 4. Input acceleration and cyclic shear stress time histories recorded in large scale test LG-0 and centrifuge tests FF-V3 and PF-P1 (modified after Abdoun et al., 2012).

The authors focused on the shear stress time histories recorded in the tests at mid-depth, such as those plotted in Fig. 4, to define for the deposit the representative cyclic shear stress, τ_c, and associated cyclic stress ratio, CSR = τ_c/σ'_{v0} (where σ'_{v0} is the effective overburden vertical pressure before the shaking). Calculation of the value of CSR is a critical step in the use of the Seed and Idriss Simplified Procedure (Fig. 2, see also Youd et al., 2001). The uniform input acceleration and shear stress histories in Fig. 4, are different from typical earthquake ground acceleration records, which are much more irregular; it is due to this irregularity that typically the value of the cyclic stress ratio, CSR = τ_c/σ'_{v0} is taken to be 65% of the maximum stress ratio associated with the maximum acceleration. Because the stress history in Fig. 3 consists of cycles of similar amplitude, the factor 0.65 is not applicable, and all that is needed is to define the value of τ_c that best characterizes the time history. Abdoun et al. (2012) did this by ranking the 20 or so half-cycles of each time history from the largest peak (No. 1) to the smallest (No. 19 in the two histories of deposits that did not liquefy in Fig. 4). Then the median value, corresponding to the tenth peak, was selected as τ_c. This numbering and definition of the median, $\tau_c = \tau_{10}$, discussed in detail by Abdoun et al. (2012), is illustrated for Tests LG-0 and FF-V3 in Fig. 4. The corresponding values of τ_c at mid-depth for the six tests that did not liquefy are listed in Table 3, which also includes the associated cyclic stress ratios, CSR = τ_c/σ'_{v0}. Figure 5 presents the profiles of CSR versus depth obtained by Abdoun et al. for the same six tests using this SI procedure.

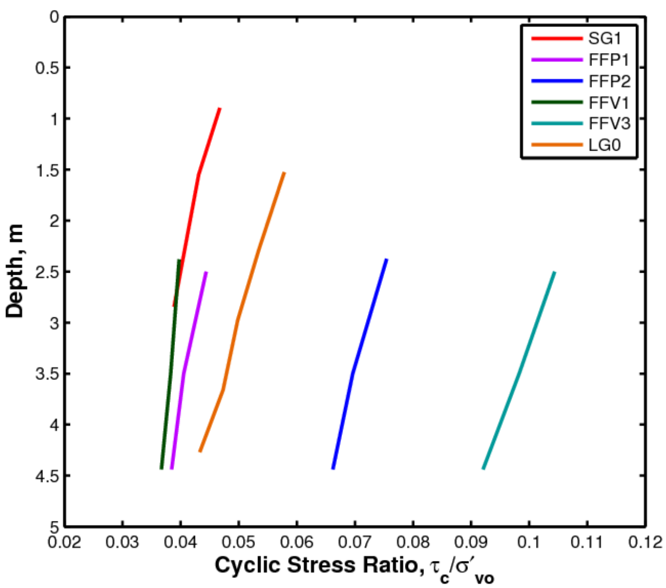

Fig. 5. Profiles of cyclic shear stress ratio experienced by the soil in the six large scale and centrifuge tests that did not liquefy (Abdoun et al., 2012).

EVALUATION OF CENTRIFUGE TESTS PF-V1 AND PF-P1 USING SHAKE ANALYSES

As clearly shown by the shear stress time history of Test PF-P1 in Fig. 4, soil strain-softening due to high excess pore pressures and liquefaction radically decreases the accelerations and shear stresses experienced by the deposit. This effect has been observed many times in the field and laboratory. Therefore, the values of CSR at mid-depth in the two experiments that caused liquefaction (PF-V1 and PF-P1), cannot be obtained with the SI method used in the previous section for the six tests that did not liquefy. In fact, due to this complicating effect of liquefaction on CSR, the application of the Seed and Idriss Simplified Procedure necessarily neglects any influence of pore pressure and liquefaction in softening the soil when calculating CSR (Youd et al. 2001; Dobry and Abdoun, 2011).

The authors conducted site response analyses which did not consider any effect of strain-softening, in order to evaluate the corresponding values of CSR at mid-depth for Tests PF-V1 and PF-P1. Computer Program SHAKE was used for these analyses (Schnabel et al., 1972). Clearly the results of such analyses are not representative at all of the actual response of the deposit in a case like Test PF-P1, for which cyclic shear stresses and accelerations essentially disappear after a few seconds (Fig. 4). However, neglecting this soil softening as done in the SHAKE analyses is necessary for consistency with charts using the Simplified Procedure such as Fig. 2, which is what matters for the comparison implemented later in this paper.

In order to establish the credibility and consistency of the two methods used in this paper to evaluate CSR for the large scale and centrifuge tests (SI for the six tests that did not liquefy, and SHAKE runs for the two tests that did liquefy), the authors conducted SHAKE analyses for all eight experiments. The runs utilized the standard average modulus reduction and damping curves available in the program for sandy soils, as well as the actual saturated unit weight of the sand. The inertia of the laminar box rings was incorporated in the calculation of τ_c and CSR = τ_c/σ'_{v0}. The profile of shear wave velocity, V_s, versus depth, z, was assumed to follow the parabolic law: $V_s = V_{s1} (\sigma'_{v0} /P_a)^{0.25}$, where P_a = 1 atmosphere = 101.33 kPa, and with the value V_{s1} for each deposit obtained from Table 2. Figure 6 sketches the model of the soil and input base acceleration used for the SHAKE analysis of large scale Test LG-0. Once the shear stress time histories were calculated by SHAKE at the various depths, these time histories were subjected to the same procedure described in the previous section to obtain τ_c and CSR. That is, τ_c was defined as the median, τ_{10}, of the stress history calculated at mid-depth.

The results of these eight SHAKE site response analyses are summarized in the last two columns of Table 3, and are plotted in Figs. 7-9.

Figure 7 includes the shear stress time history at mid-depth calculated by SHAKE for large scale Test LG-0, as well as the time history for the same test and depth obtained before using SI, reproduced from Fig. 4. The comparison in Fig. 7 indicates very good agreement between the two procedures, with the values of τ_c from SI and SHAKE being, respectively, 1.16 and 1.13 kPa (see Table 3). The comparison of values of τ_c and CSR in Table 3 between SI and SHAKE for the six tests where the two procedures were used, is reasonably good in all cases, with differences not exceeding about 20-30%. A visual comparison between the CSR profiles of Fig. 8a (SHAKE) with those in Fig. 5 (SI), confirms this reasonably good agreement for a range of depths. Figure 8b includes the corresponding CSR profiles calculated by SHAKE for the two tests that liquefied the soil, with the corresponding values of CSR at mid-depth listed in Table 3.

Figure 9 includes a comparison of *maximum shear stress ratios*, τ_{max}/σ'_{v0}, at mid-depth, obtained using both SI and SHAKE for all eight tests, including both experiments where the soil did and did not liquefy. As it was impossible to extract CSR = τ_c/σ'_{v0} using SI and the median, $\tau_c = \tau_{10}$, for the two tests that liquefied, the authors decided to use τ_{max}/σ'_{v0} for the comparison. The agreement in Fig. 9 is again reasonably good for the tests that did not liquefy, as expected. On the other hand, the values of τ_{max}/σ'_{v0} for Tests PF-V1 and PF-P1 in which the soil liquefied, are much smaller for the SI than for SHAKE, as expected, due to the strain softening effect after high pore pressures and liquefaction developed in the sand deposit. Figure 9, in conjunction with the results and comparisons discussed before using Table 3 and Figs. 5 and 7-8, clearly show that for the cases of no liquefaction, the SI and SHAKE procedures can be relied to provide similar values of CSR. Therefore, the authors confirmed their decision to use the CSR values obtained with SI in Table 3, to plot the corresponding six large scale and centrifuge tests as case histories in the field liquefaction chart of Fig. 2. On the other hand, the values of CSR obtained with SHAKE in Table 3, were used to plot the two centrifuge tests which induced liquefaction, also as case histories in the field liquefaction chart of Fig. 2.

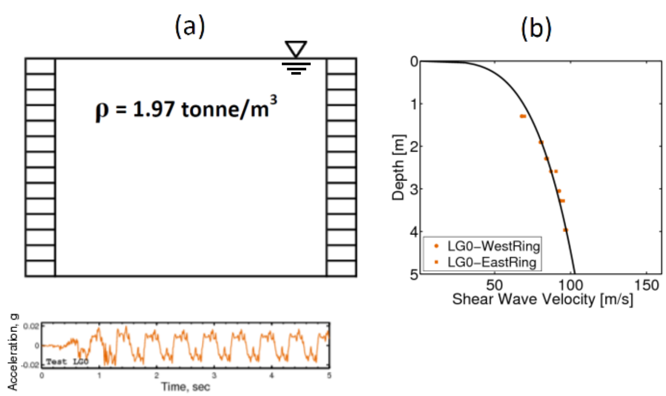

Fig. 6. SHAKE site response analysis of large scale Test LG-0: a) soil model and input shaking; and b) shear wave velocity profile.

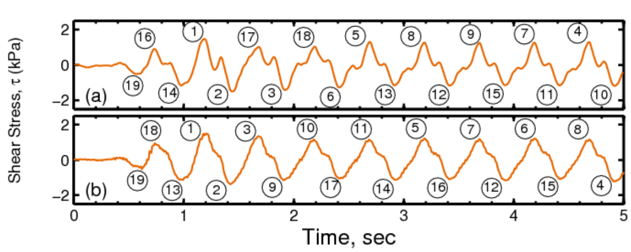

Fig. 7. Comparison between cyclic shear stress time histories at mid-depth for large scale Test LG-0: a) computed using Program SHAKE; and b) obtained using System Identification (SI) from the recorded accelerograms.

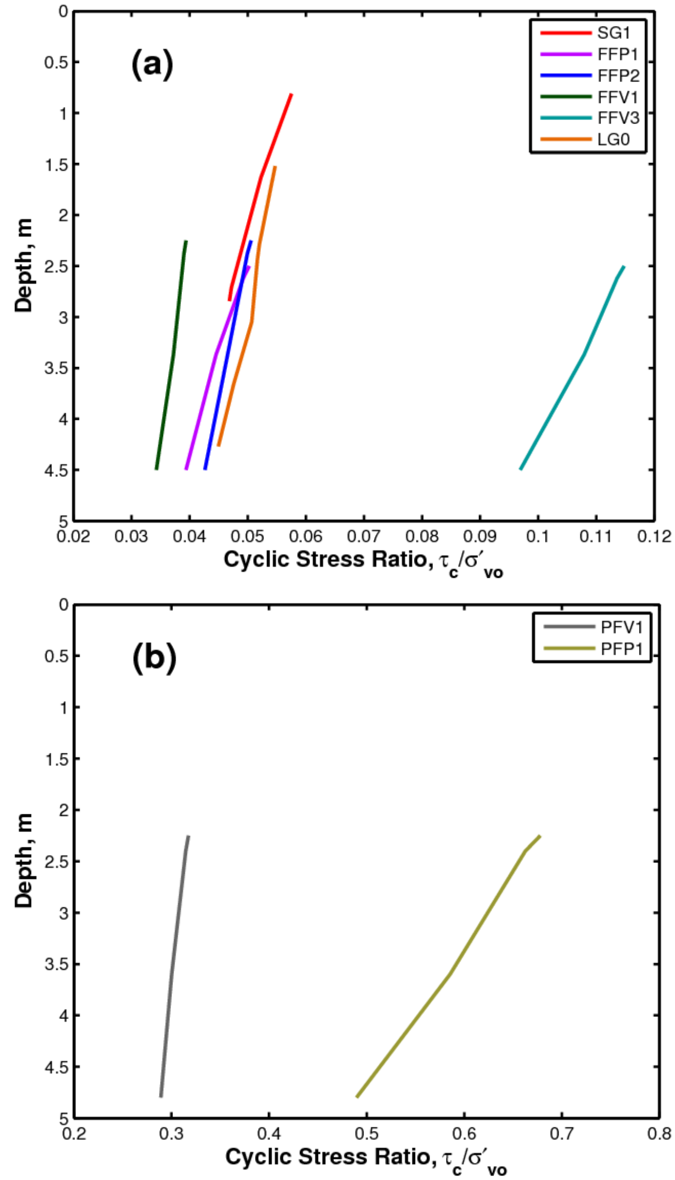

Fig. 8. Profiles of cyclic shear stress ratio calculated using Program SHAKE in the eight large scale and centrifuge tests: (a) tests that did not liquefy the soil; and (b) tests that caused liquefaction.

COMPARISON OF LARGE SCALE AND CENTRIFUGE SHAKING TESTS WITH FIELD CASE HISTORIES FROM 1989 LOMA PRIETA EARTHQUAKE

The eight large scale and centrifuge shaking experiments processed by Abdoun et al. (2012), and described in Tables 2-3 and Figs. 3-5, constitute additional liquefaction case histories which can be used to augment the twenty-nine field case histories of loose clean and silty sand fills plotted in Fig. 2. This is made possible by the SI processing of six of the tests which allowed obtaining the corresponding values of CSR listed in Table 3, and by the SHAKE evaluation of the two tests that induced liquefaction in the soil, which allowed obtaining the values of CSR listed in the last two rows of Table 3. The corresponding values of CSR selected this way, from either SI or SHAKE, are listed in Table 4. Furthermore, these new eight case histories were obtained under controlled conditions including readings from a number of sensors, allowing better definition of excess pore pressures as well as of the cyclic stress ratios induced in the deposit. As discussed before, the ranges of clean and silty sands, deposition methods, and values of V_{s1} in these eight case histories are generally consistent with the corresponding ranges for the field data points of loose clean and silty sand fills in Fig. 2. Furthermore, the duration of the shaking used in all eight tests (about ten cycles) corresponds approximately to the duration of an earthquake magnitude, M_w = 7 in the field (Idriss and Boulanger, 2008).

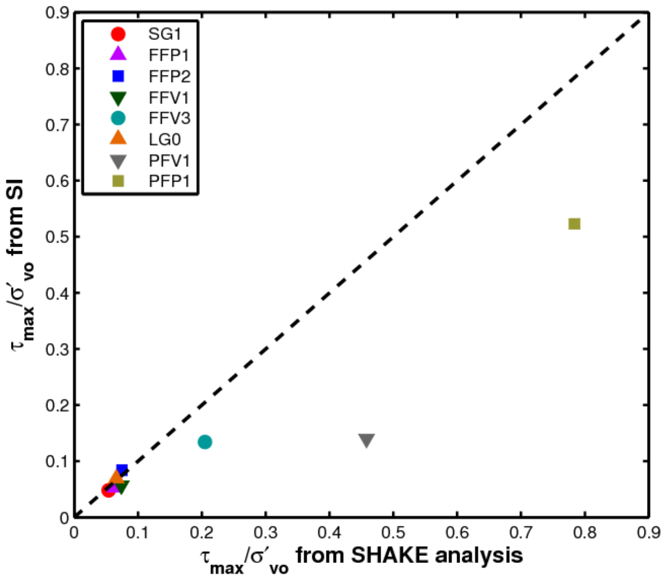

Fig. 9. Comparison between maximum shear stress ratios at mid-depth computed with SHAKE and obtained using System Identification (SI).

Figure 10 includes the data points for these eight case histories using the CSR at mid-depth listed in Table 4, plotted versus the V_{s1} of the deposit, also listed in the same table. Figure 10 also includes the Andrus and Stokoe field liquefaction chart for clean sands and M_w = 7, reproduced from Fig. 2. Before plotting the CSR from Table 4 in Fig. 5, the CSR of each test was decreased by 10%, to account for the fact that the test deposits were subjected to one-directional shaking, compared with 2D shaking in the field (Seed, 1979; Boulanger and Idriss, 2011). That is, $(CSR)_{2D}$ = 0.9CSR is plotted versus V_{s1} for all eight data points in Fig. 10.

The plotting of data points in Fig. 10 takes advantage of the additional information on pore pressure buildup after 5 seconds and ten cycles of shaking, $(r_u)_{max}$, obtained in all tests and listed in Table 2. As indicated in the figure, the eight case histories are divided in three groups, which plot on different zones of the graph, as follows:

- Two tests with $(r_u)_{max}$ = 0, that plot significantly below the curve separating liquefaction from no liquefaction (open dots);
- Four tests with $(r_u)_{max}$ = 025 to 0.70, that is with significant pore pressure buildup short of liquefaction, which plot slightly below or on the curve (half-filled dots); and
- Two tests with $(r_u)_{max}$ = 1.0, indicating that the deposit liquefied, which plot significantly above the curve (full dots).

The location of these three groups is very consistent with the field information reflected in the Andrus and Stokoe curve. This includes both the fact that the curve separates well the two cases of liquefaction from the six tests that did not liquefy, and the fact that the four experiments which had significant pore pressure buildup short of liquefaction, tend to plot slightly below the curve.

Table 4. Cyclic Stress Ratios, CSR, Selected from Table 3 and Used to Plot the Large Scale and Centrifuge Tests in Figs. 10-11 After Correction for 2D Shaking.

Test	*Normalized Shear Wave Velocity, V_{SI} (m/s)*	*Liquefaction? Y = 1; N = 0*	*Depth, z (m)*	*Selected CSR = τ_c/σ'_{v0}*	*$(CSR)_{2D}$ = 0.9CSR(1)*
SG-1	119	0	2.85	0.039	0.035
FF-P1	140	0	2.5	0.044	0.040
FF-P2	132	0	2.4	0.075	0.068
FF-V1	174	0	2.4	0.040	0.036
FF-V3	174	0	2.5	0.104	0.094
LG-0	124	0	2.3	0.053	0.048
PF-V1	174	1	2.25	0.317	0.285
PF-P1	140	1	2.25	0.678	0.610

Notes:

(1) CSR reduced by 10% to account for 2D shaking in the field.

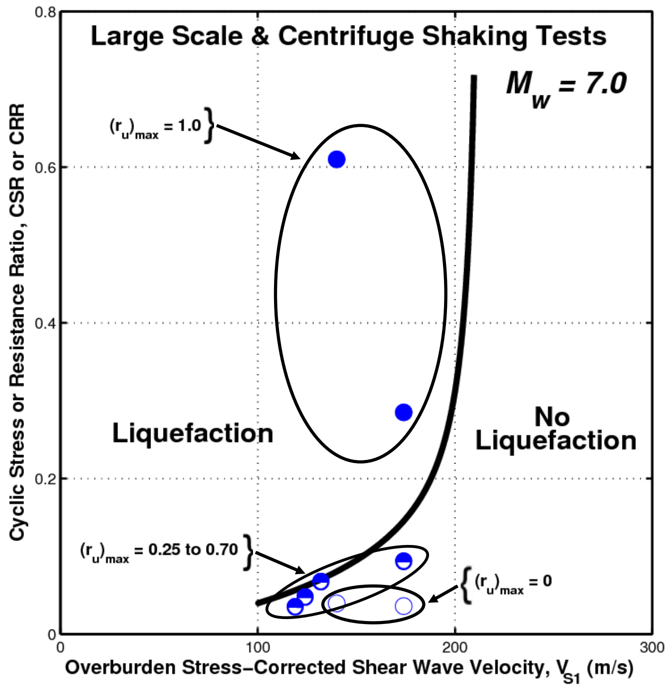

Fig. 10. Large scale and centrifuge shaking test case histories compared with Andrus and Stokoe field liquefaction chart

These eight data points from the large scale and centrifuge tests are plotted again in Fig. 11, together with the same 1989 Loma Prieta field case histories of loose sandy fills from Fig. 2, and again including the Andrus and Stokoe curve for M_w = 7.0. The data points in Fig. 11 are plotted utilizing the usual convention of full data points for liquefaction and open data points for no liquefaction, disregarding the additional information on the value of $(r_u)_{max}$ available for the six centrifuge and large scale tests that did not liquefy.

All data points in Fig. 11 – irrespective of they being field case histories or large scale/centrifuge tests – plot consistently above or below the Andrus and Stokoe curve depending on the corresponding deposits having liquefied or not. That is, Fig. 11 confirms that the eight large scale and centrifuge tests reported in this paper, are representative of the liquefaction response experienced by similar loose clean and silty sandy fills during the 1989 Loma Prieta earthquake. Furthermore, the figure indicates that the clean sand curve proposed by Andrus and Stokoe in their 2000 paper, constitutes a reliable boundary separating liquefaction from no liquefaction for loose recent artificial fills consisting of either clean sands or silty sands with non plastic fines up to about 34%, in both the field and the laboratory. This validates the applicability of the experimental techniques presented in this paper (large scale and centrifuge tests), to the San Francisco Bay Area fills as well as similar recent loose sandy fills around the world.

While the 1989 Loma Prieta earthquake had a moment magnitude close to M = 7.0 (values of both M_w = 6.9 and M_w = 7.0 have been used in liquefaction studies, see Andrus et al., 2003; Moss et al., 2006; Idriss and Boulanger, 2010), the duration of the ground shaking was shorter than that usually associated with M_w = 6.9-7.0 (Seed et al., 1990). To determine the possible influence of this shorter duration on the comparison between field and laboratory results of Fig. 11, the authors went back to the piezometric records for the eight large scale and centrifuge tests listed in Fig. 2, and noted the maximum pore pressure recorded after 2.5 seconds, that is after 5 cycles of shaking, half of the total duration of the shaking considered until now. These values of $(r_u)_{max}$ after 2.5 sec have been listed in Table 2, side-by side with the values of $(r_u)_{max}$ after the full 10 cycles and 5 seconds of shaking. For the six tests that did not liquefy, the values of $(r_u)_{max}$ are decreased, which does not affect the non-liquefaction status of the corresponding data points in Fig. 11. For the two experiments that did liquefy after 5 sec, $(r_u)_{max}$ = 0.93 and 1.0 after 2.5 sec in Table 2, indicating in both cases pore pressures high enough to be considered full liquefaction. Therefore, none of the eight data points in Fig. 11 are affected, and the conclusions above about the validity of the large scale and centrifuge tests to represent the liquefaction response of loose artificial fills during the 1989 Loma Prieta earthquake, are not changed when a shorter duration of shaking is considered.

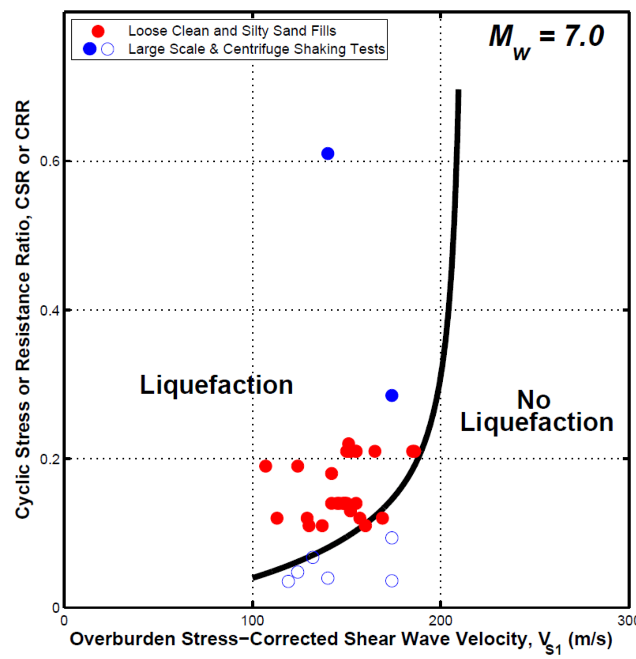

Fig. 11. Andrus and Stokoe field liquefaction chart compared with field case histories of loose sandy fills from the 1989 Loma Prieta earthquake, and with the eight large scale and centrifuge shaking tests.

CONCLUSION

The paper compares results of eight liquefaction case histories obtained from large scale and centrifuge shaking tests, with twenty-nine field case histories of loose, recent clean and silty sand fills that liquefied in the 1989 Loma Prieta earthquake.

As shown by Fig. 11, the comparison is excellent, with both the tests and the field case histories validating each other as well as the Andrus and Stokoe field liquefaction chart for clean sands. Therefore, the large scale and centrifuge testing techniques presented here, can be used with confidence in future studies aimed at improving liquefaction evaluation and mitigation methods of sandy fills during earthquakes.

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