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SUMMARY OF SPT BASED FIELD CASE HISTORY DATA OF CETIN (2016) DATABASE

by

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1. INTRODUCTION

This report provides documentation of the Cetin et al. (2016) field performance case histories, probabilistic maximum likelihood assessment and the sources of differences between the liquefaction triggering resistance estimations (CRR values) of the widely used liquefaction triggering relationships of Seed et al. (1985), Cetin et al. (2004, 2016) and Boulanger and Idriss (2012).

Cetin et al. (2016-a) presented a concise summary of the improved database and the updated triggering relationships. For the sake of completeness, as part of Chapter 2 of this report, Cetin et al. (2016-a) manuscript is re-presented, as edited to include the electronic supplements, and a broader and more detailed documentation of all of case histories. This broader and more detailed documentation along with the interpretations of other researchers is presented in Appendix A. Similarly, site response analyses details and results from Cetin (2000) is re-presented in Appendix B. To enable the readers to quickly refer to Cetin and Seed (2004), it is also included in Appendix C.

The sources of differences between the liquefaction triggering resistance estimations (CRR values) of the widely used liquefaction triggering relationships of Seed et al. (1985), Cetin et al. (2004) and Boulanger and Idriss (2012) were discussed in Cetin et al. (2016-b). Again, for the sake of completeness, as part of Chapter 3 of this report, Cetin et al. (2016-b) manuscript is re-presented, as edited to the include of electronic supplements, and a detailed response to issues addressed by Idriss and Boulanger (2012). Also, the technical response letter (Cetin, 2014) prepared at the request of the members of the NRC Committee on the State of the Art and Practice in Earthquake Induced Soil Liquefaction Assessment, is given in Appendix D, which is helpful to follow the technical discussions.

This report is intended only as a concise summary of a vast amount of data. The interpretations presented are those of the research team. A more detailed description of some of the details of the methods and procedures used to evaluate and analyze these field performance case histories is also presented in Cetin (2000) and Cetin et al. (2000) though the final evaluations presented in this report are the most recent interpretations undertaken under these current studies.

2. UPDATED CETIN ET AL. (2016) SPT-BASED PROBABILISTIC AND DETERMINISTIC SEISMIC SOIL LIQUEFACTION TRIGGERING HAZARD

2.1. INTRODUCTION

The use of in-situ "index" testing continues to be the dominant approach in engineering practice for the assessment of seismic soil liquefaction triggering hazard. Four in-situ test methods have reached a level of sufficient maturity as to represent viable tools for this purpose: (1) the Standard Penetration Test (SPT), (2) the Cone Penetration Test (CPT), (3) measurement of in-situ Shear Wave Velocity (Vs), and (4) the Becker Penetration Test (BPT). Additional methods are under development. Among these four, the oldest and still the most widely used is the SPT.

As will be discussed in Chapter 3, for a given fines-, overburden-, equipment-, and procedure-corrected SPT blowcount ($N_{1,60,CS}$), the corresponding cyclic resistance ratio (CRR) at which liquefaction may be triggered can vary considerably among the relationships of Seed et al. (1985), Cetin et al. (2004, 2016-a), and Boulanger and Idriss (2012). These four triggering relationships will be referred to hereafter as SEA_1985, CEA_2004, CEA_2016 and BI_2012, respectively. The relative differences between the estimated CRR values can be as high as 50% to 80% in the critical low-resistance region of $N_{1,60,CS} < 15$ blows/ft, and these differences can be further increased by "secondary" relationships (e.g. K_σ, etc.) associated with each of these triggering relationships.

For the purpose of identifying reasons underlying the differences between the SEA_1985, CEA_2016 and BI_2012 triggering relationships, each database was carefully reviewed and re-assessed. The authors used this opportunity to update the Cetin et al. (2004) database, and then also to update the regressed liquefaction triggering correlation. This chapter will address the revisions and updating of the Cetin et al. (2004) field performance case history database, and the development of the resulting updated triggering correlation. Chapter 3 will then address remaining differences between the three triggering relationships of SEA_1985, BI_2012 and CEA_2016.

2.2. UPDATED CETIN ET AL. (2016) DATABASE

Inspired by the ongoing technical discussions, the Cetin et al. (2004) case histories were re-visited, and the database was re-processed taking advantage of updates in the current state of knowledge, as well as comments and suggestions made by others. Changes made here include the following:

(1) A typographical error at the third decimal point in the spreadsheet execution of the r_d formula in the original work of Cetin et al. (2004) was identified. This error biased r_d values towards the low side. The

error was zero at the ground surface, and it increased with increasing depth. The overall average resulting difference in r_d values for the 200 case histories after the error correction was found to be an average increase of 6.1%. Because most of the field performance case histories were for soils at shallow depths, the overall average impact on the resulting back-calculated cyclic stress ratio (CSR) values in the data set was only moderate. In addition, r_d values for 50 of the 200 field performance case histories had been directly back-calculated by means of site-specific and event-specific seismic site response analyses using the one-dimensional equivalent linear approach to model nonlinear site response effects (Cetin 2000, Cetin et al., 2000). Hence, the r_d values for those 50 cases had no error and they remain unchanged. For every case history, rd values were re-calculated with the corrected spreadsheet version of the Cetin and Seed (2004) r_d relationship, except for the 50 case histories for which site-specific seismic site response analyses were performed to directly calculate CSR's (and r_d values). When doing so, shear wave velocity $V_{s,12m}$ values were also re-evaluated and updated. As a check of the more detailed evaluations from Cetin et al. (2004), Equations (2-1) and (2-2) from Design Specification for Highway Bridges, Japan Road Association (2002) were also used here to estimate shear wave velocities at shallow depths.

$V_S = 80N^{1/3}$ m/s (for sand) **Eqn. (2-1)**

$V_S = 100N^{1/3}$ m/s (for clay) **Eqn. (2-2)**

As shown in Figure 2.1 (g), this did not resulted in no major systematic changes in any individual $V_{s,12\,m}$ values for the 150 cases back-analyzed using the simplified r_d approach. As will be later shown in Table 2-8, the overall average value of $V_{s,12m}$ for the case history database increased by 6.6%, and much of this was due to minor increases in unit weights, and thus also σ_v', as will be discussed below.

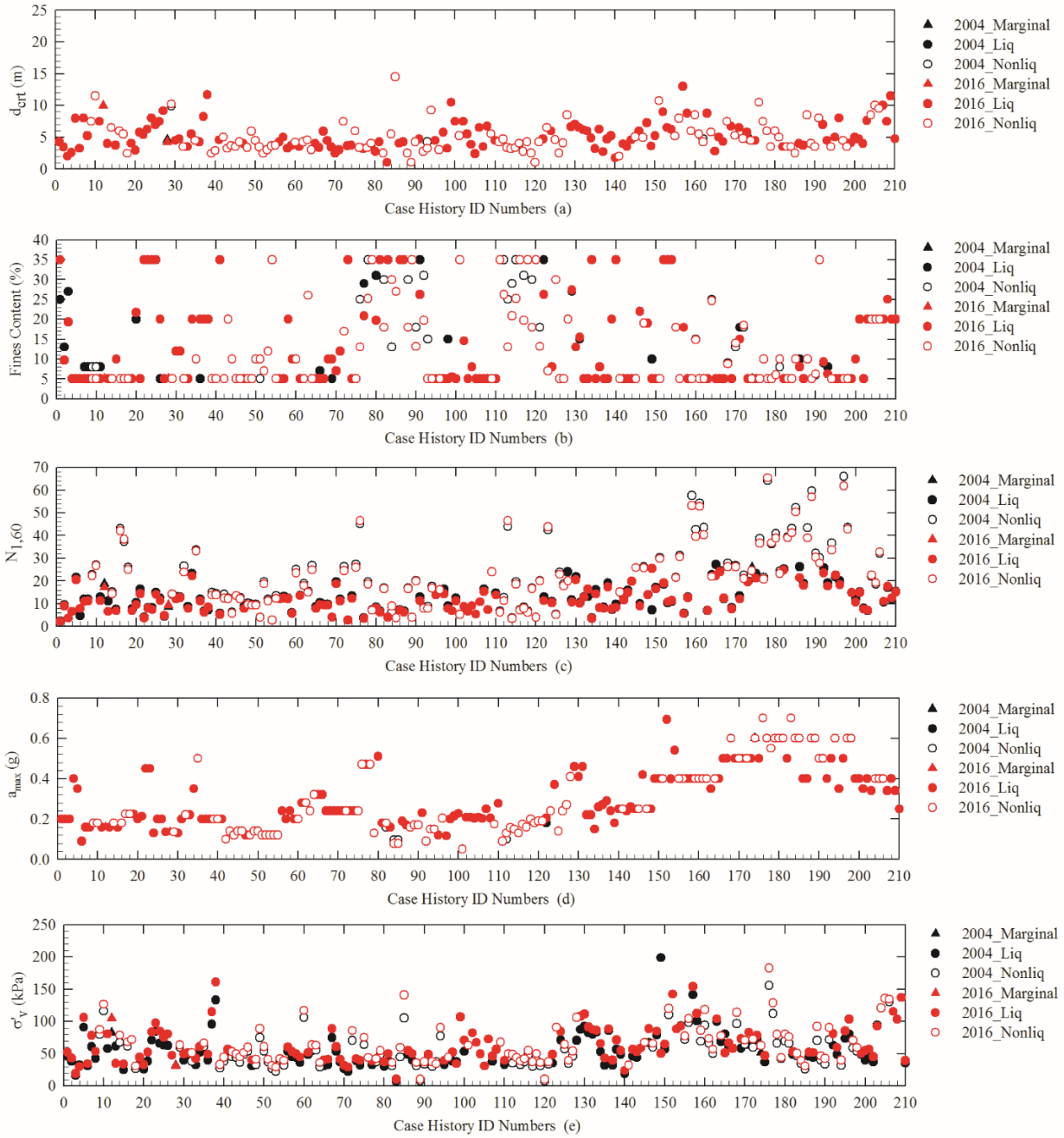


Fig. 2.1. A comparative summary of back-analyzed case history model input parameters

(Case histories are numbered as listed in Table 2-7)

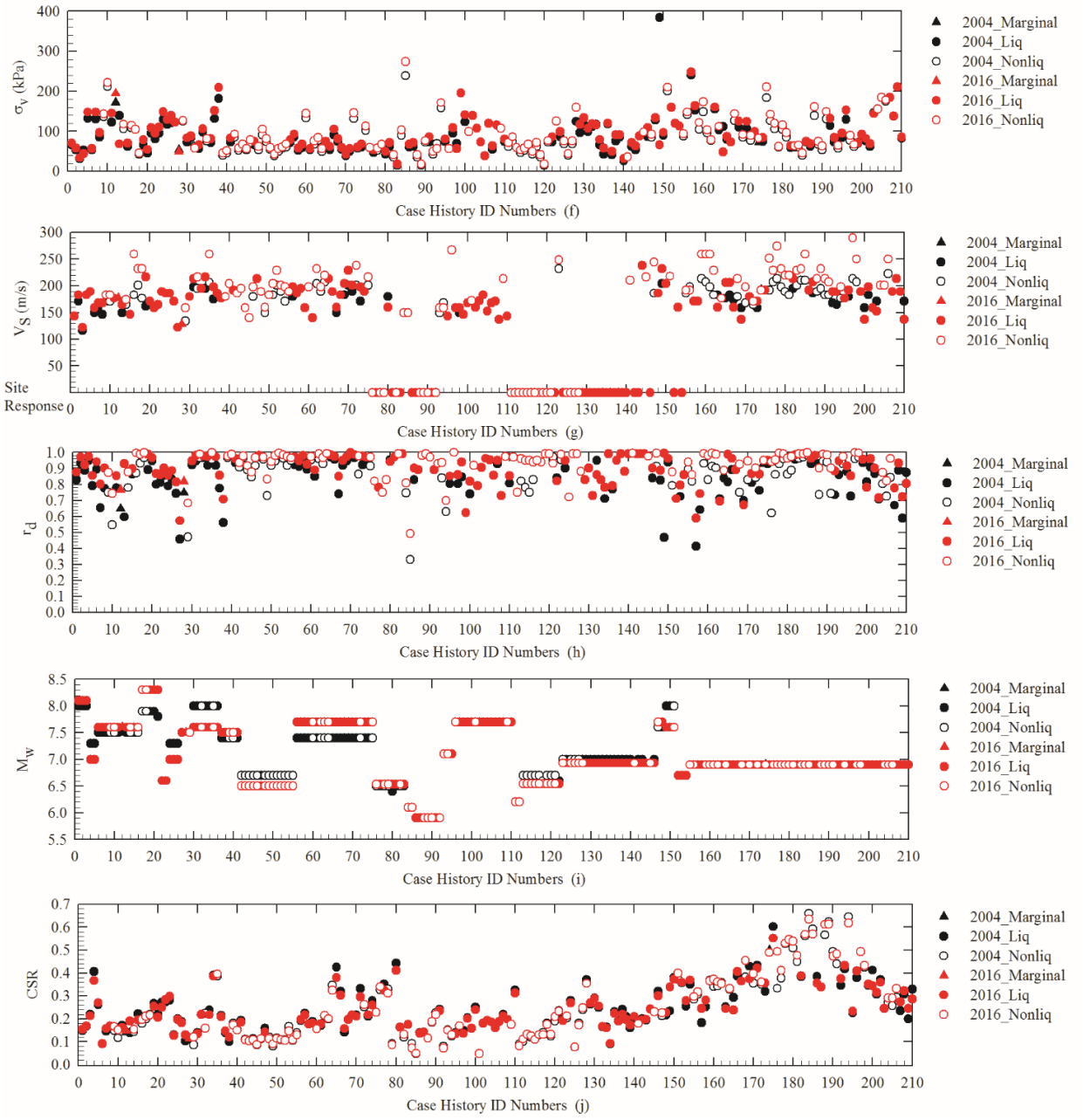


Fig. 2.1. (Cont'd.) A comparative summary of back-analyzed case history model input parameters

(Case histories are numbered as listed in Table 2-7)

(2) Similarly, unit weights for each case history site were updated consistent with the information summarized in Table 2-1, unless case-specific information was available indicating otherwise. Individual unit weights for each case, and each stratum, typically vary slightly based on information available. Details for each case history are presented in Appendix A. Idriss and Boulanger (2010, 2012) had suggested that unit weights were biased low in the original work of Cetin et al. (2004), and significantly lower than the

average unit weights they had employed. The updated unit weights serve to provide systematic overall values of total and effective stresses for the compiled case histories that are, on average, very close to (and slightly lower than) the average values of the case histories common to both databases as modeled by Idriss and Boulanger (2010).

Table 2-1. Typical approximate assumed unit weights as used in CEA_2016 for back analyses of field performance case histories, unless case-specific values are available

(a) Coarse-grained soil layers				
SPT-N_{60}	γ_{moist}		γ_{sat}	
(blows/ft)	(lb/ft^3)	(kN/m^3)	(lb/ft^3)-(kN/m^3)	
0 - 4	100	16.0	110	17.6
5 - 10	110	17.6	120	19.2
11 - 30	120	19.2	125	20.0
30 - 50	125	20.0	135	21.6
(b) Fine-grained soil layers				
0 - 4	100	16.0	110	17.6
5 - 8	110	17.6	120	19.2
9 - 16	115	18.4	125	20.0

(3) Atmospheric pressure (P_a) conversion was more precisely executed as 1 atm.=101.3 kPa = 2116.2 lbs/ft^2 in the C_N and K_σ corrections.

These were all the adjustments and updates common to every case history. Additional modifications were then made to selected individual case histories as described below. These include five of the "8 of 11 key case histories" discussed and contested by Idriss and Boulanger (2012). The original work of Cetin et al. (2004) included the application of systematic screening procedures for evaluation of whether or not a case history would be included in the database. They are presented and explained in Cetin et al. (2004), and will be briefly discussed again later in this report. Those criteria, revolving around the reliability with which the field performance case history can be interpreted and suitably back-analyzed, continue to be used in these current efforts. Additional changes here included the following:

(1) The Cetin et al. (2016) database contains all the cases used by Cetin et al (2004), except for three cases, which were excluded from the updated database. These excluded cases are (1) 1975 Haicheng Earthquake (M_s=7.3) Shung Tai Zi River, (2) 1994 Northridge Earthquake (M_w=6.7) Malden Street Unit D., and (3) 1979 Imperial Valley Earthquake (M_L=6.6) Wildlife Unit B. The reasons for exclusion of these cases are summarized in Table 2-2.

Table 2-2. Reasons for exclusion of the three case histories.

Case History ID	Comments
1975 Haicheng Earthquake (M_s=7.3), Shuang Tai Zi River	The 1975 Haicheng Earthquake, Shuang Tai Zi River case history site was excluded due to lack of documentation of fines content and consistency limits in the original reference. This information is needed to judge if the suspect fine grained soil layer is non-plastic or not. It is noted that Seed et al. (1984) described the critical layer as "silt" with no further details.
1994 Northridge Earthquake (M_w=6.7), Malden Street Unit D	The 1994 Northridge Earthquake, Malden Street Unit D case history site soils are comprised of 3 distinct soil layers: Unit A is a compacted fill, located mostly above the water table, and is judged to be non-liquefiable; Unit B is a fine grained soil layer with N≈2 to 3 blows/ft., and FC>70 %, with average PI = 18%, and average clay content of 31 % (Holzer et al. 1999). Clayey soils with PI=18 %, were categorically judged to be potentially "non-liquefiable" back when the Cetin (2000) and Cetin et al. (2004) databases were compiled. Unit D is Pleistocene silty sand; hence it was judged to be the suspect stratum for liquefaction-induced ground deformations. Unit D was, thus, initially identified as the critical layer. Borings BH 3 and BH 5 were used to characterize the SPT N-values in this critical layer. BH 3 is located within the “permanent ground deformation” zone. However, after having revisited this case history with the improved state of knowledge today, and based on exchanges with the original field investigators, it has been determined that the suspect for observed ground deformations is a "very soft" to "soft" fine-grained (and plastic) Unit B soil layer. This, therefore, represents a case of liquefaction-like softening of cohesive soils of medium plasticity and significant clay content that are not amenable to the type of triggering correlations developed and discussed herein; so Malden Street Unit D case is now excluded from the Cetin et al. (2016) database. It is noted that Malden Street Unit D is classified as a non-liquefied site in the Idriss and Boulanger (2010) database. Contrary to this choice, it is our view that Unit D may not be used in the updated liquefaction case history database as a non-liquefied soil layer due to the potential bias introduced in CSR estimations after cyclic softening of the upper soft lean clay layer (Unit B). The presence of these overlying soft soil layers, potentially susceptible to cyclic softening, can significantly reduce the induced cyclic shear stresses due to their significantly reduced inertial mass participation effects. One of the firm criteria for inclusion of a field performance case history in the overall database (see Cetin et al., 2004) continues to be that only one critical stratum can be included at any location because it is not possible to reliably assess CSR in other strata (both above and below) the liquefied or softened stratum as shear stress transfer across the softened stratum becomes difficult to back-analyze.
1979 Imperial Valley Earthquake (M_L=6.6), Wildlife Unit B	1979 Imperial Valley Earthquake Wildlife Unit B case history site was excluded due to lack of a confirmed no-liquefaction performance by field investigators. This was recently stated by Youd (2013) as follows: "the no assigned to the Wildlife site for the 1979 Imperial Valley earthquake (M=6.5) was not confirmed by investigator observation."

(2) Three case history sites (1989 Loma Prieta Earthquake Miller Farm CMF-10, and 1995 Hyogoken-Nanbu Earthquake Kobe # 6 and #16) are now modeled in these current studies as non-liquefied. The reasons for the modifications of these cases are summarized in Table 2-3.

Table 2-3. Reasons for the modification of three case histories.

Case History ID	Comments
1989 Loma Prieta Earthquake (M_w=6.93), Miller Farm CMF-10	At the time of Cetin 2000 and Cetin et al. 2004 studies, the Miller Farm CMF-10 case history site was classified as a liquefied site (Wayne et al., 1998). The CMF-10 borehole is located in a transition zone in between permanent ground deformation and no ground deformation zones. However, it is now understood that the field investigation team specifically intended CMF-10 to be located in the non-liquefied zone. Given the relative continuity from boreholes CMF-5 to CMF-10, this was difficult to infer. It is our policy to accept the field judgments of well-qualified field investigation teams; hence the CMF-10 case history is now classified as non-liquefied.
1995 Hyogoken-Nanbu Earthquake (M_w=6.9), Kobe # 6	This site was originally shown to be a liquefied site on the overall summary map provided by Prof. Tokimatsu, but it was also later found to be inconsistently listed as a non-liquefied site on subsequent summary table provided by Prof. Tokimatsu through personal communication (Kobe City Office, 1999). When compiling the original Cetin (2000) and Cetin et al. (2004) databases, the site was listed as "liquefied" based on the initial data (the map), and the subsequent change in the summary table was not caught. This is now resolved, and the site is updated as a "non-liquefied" site in the database of Cetin et al. (2016).
1995 Hyogoken-Nanbu Earthquake (M_w=6.9), Kobe # 16	Due to close proximity of the Kobe #15 (Liquefied) and #16 (Non-Liquefied) sites, and similarity in the soil profiles, these two case histories were combined and assessed together as a marginally liquefied (Yes/No) site in the Cetin et al. (2004) database. For consistency, these cases are now treated as two separate case histories, and Kobe #16 is classified as non-liquefied in the Cetin et al. (2016) database.

(3) Based on improved understanding and recent developments in strong ground motion seismology over the past 14 years, and including the findings of the Next Generation of ground-motion Attenuation (NGA) models project, some of the historical earthquake magnitudes (and peak ground acceleration levels) were updated. The resulting new moment magnitudes are summarized in Table 2-4, and they are compared with the moment magnitudes used previously in Cetin et al. (2004). Changes in moment magnitudes are relatively minor, and were made for consistency. These changes had negligible impact on the overall triggering correlations developed.

Table 2-4. Comparison between the moment magnitudes (M_w) used in Cetin et al. (2004) and the updated values used in Cetin et al. (2016)

Earthquake	Cetin et al. (2004)	Cetin et al. (2016)	Reference
1944 Tohnankai	8.00	8.10	
1948 Fukui	7.30	7.00	
1968 Tokachioki	7.90	8.30	
1975 Haicheng	7.30	7.00	USGS Centennial Earthquake Catalog (Engdahl and Villasenor, 2002)
1976 Tangshan	8.00	7.60	
1977 Argentina	7.40	7.50	
1978 Miyagiken-Oki Feb. 20	6.70	6.50	
1978 Miyagiken-Oki June 12	7.40	7.70	
1990 Luzon	7.60	7.70	
1979 Imperial Valley	6.50	6.53	
1987 Superstition Hills	6.70	6.54	NGA Flatfiles (2015)
1989 Loma Prieta	7.00	6.93	
1964 Niigata	7.50	7.60	Incorporated Research Institutions for Seismology (IRIS) Seismo Archives, (2004)
1993 Kushiro-Oki	8.00	7.60	Ide and Takeo (1996)

(4) The 1993 Kushiro-Oki Earthquake Kushiro Port strong motion recording station case history site soils are comprised of 3 distinct soil layers: fine/coarse sand, gravelly sand, and silt. Back when the Cetin (2000) and Cetin et al. (2004) databases were compiled, the silty layer at the depth range of 20 to 22 meters with SPT-N values of 6 to 10 blows/ft was judged to be the suspect stratum for the phase change (due to liquefiaction triggering) in the recorded response after about 30 seconds. However, after having revisited this case history with the extended state of knowledge today, it has been determined that the suspect for the observed response could also be the "medium dense" coarse sand soil layer with SPT-N values of 16 to 18 blows/ft at the depth range of 2.8 to 5.2 meters. Due to presence of multiple suspects (a deeper, high-fines, but relatively lower SPT-N value silt layer vs. shallower, low-fines, but relatively higher SPT-N value coarse sand layer) for a critical layer, this case history is judged to warrant further in depth investigation, and it fails to meet the admissibility criteria. Hence it is excluded from the new (updated) database.

(5) All other cases were also reviewed, and for some cases, details such as the elevation of the phreatic surface, a_{max}, average fines content, critical depth, average SPT-N values, C_R and/or C_B corrections

were re-assessed and adjusted. As shown in Figure 2-1, resulting modifications here were typically minor, and had no significant impact on the resulting liquefaction triggering correlations developed.

(6) The standard deviation (uncertainty) for each case history input parameter was also re-assessed. For the purpose of more systematic treatment of case history variance, a global variance scaling coefficient of 0.39 (i.e.: θ_7) is regressed within the maximum likelihood assessment framework, and this is discussed later in this report when maximum likelihood assessment details are presented.

Idriss and Boulanger (2004) had accepted the screening process employed by Cetin et al. (2004) and had adopted in their database. The Idriss and Boulanger (2010) field performance case history database has 230 case histories. Most of these are from the database screened and compiled by Cetin et al. (2004), but there are also 33 new cases added by Idriss and Boulanger (2010). Twenty seven of these are cases from Iai et al. (1989), compiled after the 1983 Nihonkai-Chubu M=7.7 earthquake. There are also three case histories from the 1989 Loma Prieta earthquake, one from the 1964 Niigata earthquake, one from the 1968 Hyūga-nada earthquake, and one from the 1982 Urakawa-Oki earthquake. Later, Boulanger and Idriss (2014) compiled additional 24 cases from the 1999 Kocaeli and 1999 Chi-Chi earthquakes.

Cetin et al. (2016) re-evaluated these 57 (=33+24) new cases, and accepted 13 of them. The same screening criteria for admission of these 57 new case histories to the Cetin et al. (2004) database were again employed here. Thirteen of these new cases (10 of the Nihonkai-Chubu case histories, and 3 of the Loma Prieta earthquake case histories) satisfied the screening criteria and were added to the updated Cetin et al. (2016) database. These 13 new cases added are listed in Table 2-5.

Table 2-5. List of 13 additional field performance case histories from the Idriss and Boulanger (2010) database that were added to the updated Cetin et al. (2016) database

1983 Nihonkai-Chubu M=7.7 Earthquake	1989 Loma Prieta M_w=6.93 Earthquake
1. Akita Station	1. General Fish
2. Gaiko 1&2	2. Marina Laboratory_F1-F7
3. Hakodate	3. MBARI NO.4-B4B5EB2EB3
4. Nakajima No. 1(5)	
5. Nakajima No. 2(1)	
6. Nakajima No. 2(2)	
7. Nakajima No. 3(3)	
8. Nakajima No. 3(4)	
9. Ohama No. 2(2)	
10. Ohama No. Rvt. (1)	

The reasons for non-inclusion of the remaining 44 additional case histories in the database are mainly due to failure to meet the established acceptability screening criteria for one or more of the following reasons: (1) soil profiles (e.g.: boring logs) are not fully documented and adequately transparently accessible, (2) consistency limits for the critical soil layer are not available to judge if fine-grained soils are either (a) non-plastic or of very low plasticity, or (b) of higher plasticity and thus not suitable for this triggering relationship, (3) the sites are not free field sites, due either to non-level ground or close proximity of a structure and thus potential soil-structure interaction effects, and (4) the use of percussion drilling technique for SPT borings. Further explanations for the non-inclusion of each of these cases are presented in Table 2-6, and in Chapter 3.

Table 2-6. Field performance case histories from the Idriss and Boulanger (2010) and Boulanger and Idriss (2014) databases, which were studied but not included in the updated Cetin et al. (2016) database

Earthquake	Site	Liq.?	Explanation
1964 Niigata M=7.5	Kawagishi-cho	Yes	Borehole located within the vicinity of buildings. No borehole available at free field liquefied site. SPT procedures including sampler and hammer type, and energy efficiency not documented. Fines Content (FC) data not available in the reference document.
1968 Hyūga-nada M=7.5	Hososhima	No	Soil profile not documented but SPT-N values and FC data available. The extent of clayey layer not clearly defined. FC given as 36% but no Atterberg limits. Difficult to judge if the critical layer "clay-like" or "sand-like".

1982 Urakawa-Oki M=6.9	Tokachi	No	The location of the strong motion observation station not identified; hence its proximity to the ocean not verified. Ambiguity if the site free field or not. Soil profile not documented. SPT-N values available but no FC data.
1983 Nihonkai-Chubu M=7.7	Akita station (1)	No	The two sites combined. Otherwise statistical independency assumption not satisfied.
1983 Nihonkai-Chubu M=7.7	Akita station (2)		
1983 Nihonkai-Chubu M=7.7	Aomori Port	No	Soil profile not documented but SPT-N values available. FC of another site with similar characteristics available. Site response analysis needed, however no soil profile. Ambiguity if the site free field or not. In the vicinity of Aomori Port quay walls.
1983 Nihonkai-Chubu M=7.7	Gaiko 1	Yes	The two sites combined. Otherwise statistical independency assumption not satisfied.
1983 Nihonkai-Chubu M=7.7	Gaiko 2		
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(1)	No	Soil profile not documented but SPT-N values and FC data available. Ambiguity if the site free field or not. In the vicinity of Akita Port Channel, which is 10 m deep. The site judged potentially not to be a free field site.
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(2)	No	Soil profile not documented but SPT-N values and FC data available. FC corresponding to the first and critical N values missing. Site response by Iai et.al. (1989) exist, however soil profile not available.The site 0 to 5 m from the nearby 10 m deep Akita Port Channel. The site judged likely not to be a free field site.
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(3)	No	Soil profile not documented but SPT-N values and FC data available. Difficult to identify the critical layer. The site judged likely not to be a free field site.
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(4)	No	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify.
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(5)	No	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify.
1983 Nihonkai-Chubu M=7.7	Ohama No. 1(58-22)	No	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify.
1983 Nihonkai-Chubu M=7.7	Ohama No. 3 (1)	Yes	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify. The site also judged likely not to be a free field site.
1983 Nihonkai-Chubu M=7.7	Ohama No. 3 (3)	Yes	Soil profile not documented but SPT-N values and FC data available. The site not a free field site. In the vicinity of quay wall. Ohama No 3(3) & 3(4) at the same location. Two case histories from one site violate the statistically independency assumption.
1983 Nihonkai-Chubu M=7.7	Ohama No. 3 (4)	Yes	
1983 Nihonkai-Chubu M=7.7	Ohama No. Rvt (2)	No	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify.
1983 Nihonkai-Chubu M=7.7	Ohama No. Rvt (3)	No	Soil profile not documented but SPT-N values and FC data available. Critical layer difficult to identify. Site response by Iai et.al. (1989) exist. FC for the critical layer not available.
1984 Hyuga-nada-oki M=6.9	Hososhima	No	Soil profile not documented but SPT-N values and FC data available. The extent of clayey layer not clearly defined. Critical depth and SPT depths not consistent. FC>35% and Atterberg limits not available. Difficult to judge if the layer "clayey like" or "sand-like"
1990 Loma Prieta Mw=7	Sandholdt UC-B10	No	The site excluded since in Cetin 2004 database the same site used as a "liquefied" case. Two critical layers (one liquefied and other non-liquefied) from one borehole not permitted due to difficulties in estimating CSR with simplified procedure and also against statistical independency assumption.
1993 Loma Prieta Mw=7	MBARI Technology	No	MBARI Technology Building at the site interfere with free field response.CPT available (RC-9,EB9,RC-8) but SPT borehole data not available. Back calculation from results of Figure 8-9 by Boulanger et al. 1995 could not be performed nor checked, since the needed subsurface soil conditions and information could not be traced.
1979 Imperial Valley M_L=6.6	Wildlife B	No	The site not visited after 1979 Imperial Valley earthquake. Hence not confirmed that Wildlife B (1979 Imperial Valley) a nonliqufied site. (Youd, 2013)

1999 M=7.5 Kocaeli-Aug 17	Building C1&C2	Yes	Within 5m of a ~20x20m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Building C3	No	Within 5m of a ~20x20m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site B, Building B1	Yes	Within 5m of a ~20x20m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site D, Building D1	Yes	Within 3m of a ~10x12m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site E, Building E1&E2:sand	Yes	Within 2m of a ~12x17m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site E, Building E1&E2:silt	Yes	Within 2m of a ~12x17m building hence the site judged not to be a free field site. Additionally, adopting two different liquefied case history data points from one borehole (Site E, Building E1&E2:sand - Site E, Building E1&E2:silt) violates statistically independent assumption of MLE assessment.
1999 M=7.5 Kocaeli-Aug 17	Site F, Building F1	Yes	Within 2m of a ~8x12m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site G, Building G2&G3	Yes	Within 2m ~10x20m buildings hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site H, Building H1	Yes	Within 6m of a ~10x15m building hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Site I, Building I2&I2	Yes	Within 4m of ~15x15m buildings hence the site judged not to be a free field site. Additionally, not clear which penetration test data used when processing Building I2&I2 site. "Borehole" I2 a CPT sounding and borehole I1 a SPT log.
1999 M=7.5 Kocaeli-Aug 17	Site J, Building J2&J2	Yes	Within 3m of a ~20x24m building hence the site judged likely not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Değirmenedere DN-1	Yes	A sloping site (1V:5H), hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Değirmenedere DN-2	No	A sloping site (1V:5H), hence the site judged not to be a free field site.
1999 M=7.5 Kocaeli-Aug 17	Yalova Harbor	Yes	Next to the site, there exist a quay wall and wave breaker; hence the site judged not to be a free field site.
1999 M=7.6 Chi-Chi Sep. 21	Wufeng-Site A1	No	The upper soil layer consist of a soft clay unit with SPT N=2. Cyclic mobility of the clay layer likely. CSR estimation difficult. In the immidiate vicinity of the borehole, there exist 1-2 storey buildings. Drilling method reported to be percussion.
1999 M=7.6 Chi-Chi Sep. 21	Wufeng-Site C (WCS-1)	Yes	The upper soil layer consist of soft clay unit with SPT N=4. Cyclic mobility of the clay layer likely. CSR estimation difficult. Drilling method reported to be percussion.
1999 M=7.6 Chi-Chi Sep. 21	Wufeng-Site C (WCS-2)	Yes	The upper soil layer consist of soft clay unit with SPT N=3. Cyclic mobility of the clay layer likely. CSR estimation difficult. Next to the Wufeng-Site C (WCS-2) site, there exist a relatively high retaining wall; hence the site judged not to be a free field site. Drilling method reported to be percussion.
1999 M=7.6 Chi-Chi Sep. 21	Wufeng-Site B (WBS-1)	Yes	Drilling method reported to be percussion.
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 40	Yes	Ground failure in Yuanlin town observed in well-developed areas. (Chu et al., 2004). In PEER database the borehole location shown on a map, however the map resolution not high enough to identify the coordinates of the borehole. Likely that these sites located in the immidiate vicinity of residential structures; hence a free field response not guaranteed. Drilling method reported to be percussion.
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 28	Yes	
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 30	Yes	
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 31	No	
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 10	No	
1999 M=7.6 Chi-Chi Sep. 21	Yuanlin BH 27	No	

2.2.1. Updated Field Performance Case History Database Summary

As shown in Figure 2-2 and in Table 2-7, the updated Cetin et al. (2016) database is composed of 197 cases from the Cetin et al. (2004) database, and 13 additional cases screened and added after Idriss and Boulanger (2010) database, for a total of 210 case histories. In Figure 2-2, the data points are plotted without fines, K_σ and K_{Mw} corrections.

For clarity, cases are plotted as "data points" showing the mean values of CSR_{σ_v',M_w} and $N_{1,60}$ for each case history. In Figure 2-2, the three "source database" groupings are related to timing and provenance of the arrival of the cases into the database. "Seed et al." represents cases from the older database of Seed et al. (1985) that successfully passed screening criteria and were entered into the current database. 1995 Hyogoken-Nanbu (Kobe) earthquake cases are from the Cetin et al. (2000, 2004) database, but are separated out with a different symbol because the Kobe earthquake case histories are eventually treated slightly differently in performing regressions and developing triggering relationships. "Cetin et al." represents cases screened and added to the overall database, subsequent to the work of Seed et al. (1985), but excluding the 1995 Hyoguken-Nanbu (Kobe) earthquakes cases. Some of these are older cases, from various sources, as discussed in previous sections of this report.

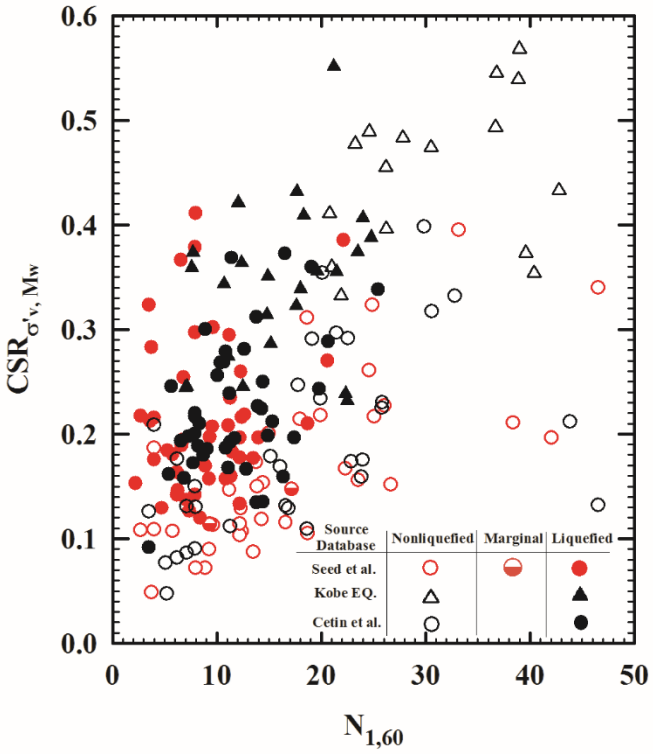


Fig. 2-2. Summary of the Updated Cetin et al. (2016) Field Performance Case History Data.

Table 2-7. Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
1	1944 Tohnankai	Ienaga	Yes	4.3	2.4	70	52	0.20	143	0.87	0.15	8.10	0.84	1.25	0.15	72.5	1.50	1.00	0.90	1.00	1.17	1.40	2.2	5.4	Kishida (1969)
2	1944 Tohnankai	Komei	Yes	3.5	2.0	58	43	0.20	183	0.97	0.17	8.10	0.84	1.33	0.15	9.7	5.75	1.00	0.87	1.00	1.17	1.53	8.9	9.9	Kishida (1969)
3	1944 Tohnankai	Meiko	Yes	2.0	0.5	34	19	0.20	122	0.92	0.21	8.10	0.84	1.75	0.14	19.3	1.95	1.00	0.80	1.00	1.17	2.00	3.6	5.5	Kishida (1969)
4	1948 Fukui	Shonenji Temple	Yes	2.6	1.2	44	30	0.40	183	0.97	0.37	7.00	1.17	1.50	0.21	0.0	3.70	1.00	0.83	1.00	1.17	1.83	6.5	7.0	Kishida (1969)
5	1948 Fukui	Takaya 45	Yes	8.0	3.7	148	106	0.35	189	0.85	0.27	7.00	1.17	0.98	0.23	3.3	16.20	1.00	1.00	1.00	1.30	0.98	20.6	21.2	Kishida (1969)
6	1964 Niigata	Arayamotomachi	Yes	3.3	1.0	56	34	0.09	158	0.94	0.09	7.60	0.97	1.44	0.06	5.0	4.40	1.00	0.86	1.00	1.22	1.72	7.9	8.4	Yasuda and Tohno (1988)
7	1964 Niigata	Cc17-1	Yes	8.0	0.9	148	78	0.16	168	0.80	0.16	7.60	0.97	1.09	0.15	2.0	8.75	1.00	1.00	1.00	1.09	1.14	10.9	11.4	Kishida (1966)
8	1964 Niigata	Cc17-2	Yes	5.3	0.9	96	54	0.16	168	0.90	0.17	7.60	0.97	1.24	0.14	2.0	7.95	1.00	0.93	1.00	1.09	1.37	11.1	11.6	Kishida (1966)
9	1964 Niigata	Old Town -1	No	7.5	1.8	143	87	0.18	183	0.87	0.17	7.60	0.97	1.05	0.16	2.0	17.35	1.00	0.99	1.00	1.21	1.08	22.3	23.0	Koizumi (1964)
10	1964 Niigata	Old Town -2	No	11.5	1.8	222	127	0.18	183	0.74	0.15	7.60	0.97	0.93	0.17	2.0	24.70	1.00	1.00	1.00	1.21	0.89	26.7	27.4	Koizumi (1964)
11	1964 Niigata	Rail Road-1	Yes	7.5	0.9	145	81	0.16	177	0.85	0.16	7.60	0.97	1.08	0.15	2.0	9.35	1.00	0.99	1.00	1.09	1.12	11.3	11.9	Koizumi (1964)
12	1964 Niigata	Rail Road-2	No/Yes	10.0	0.9	194	105	0.16	177	0.77	0.15	7.60	0.97	0.99	0.15	2.0	16.00	1.00	1.00	1.00	1.09	0.98	17.1	17.7	Koizumi (1964)
13	1964 Niigata	River Site	Yes	4.0	0.6	68	35	0.16	165	0.93	0.19	7.60	0.97	1.43	0.14	0.0	4.00	1.00	0.89	1.00	1.09	1.71	6.6	7.1	Ishihara et. al. (1979)
14	1964 Niigata	Road Site	No	6.5	2.5	118	79	0.18	174	0.88	0.15	7.60	0.97	1.09	0.15	0.0	12.05	1.00	0.96	1.00	1.09	1.14	14.4	15.0	Ishihara et. al. (1979)
15	1964 Niigata	Showa Br 2	Yes	3.7	0.0	70	34	0.16	146	0.90	0.19	7.60	0.97	1.45	0.14	10.0	4.10	1.00	0.88	1.00	1.09	1.73	6.8	7.8	Ishihara et. al. (1979)
16	1964 Niigata	Showa Br 4	No	6.0	1.2	115	68	0.18	259	0.99	0.20	7.60	0.97	1.14	0.18	0.0	30.00	1.00	0.95	1.00	1.21	1.22	42.0	42.8	Takada et al (1965)
17	1968 Tokachioki	Hachinohe - 2	No	5.5	2.1	104	72	0.23	232	0.99	0.21	8.30	0.79	1.12	0.24	5.0	28.50	1.00	0.94	1.00	1.21	1.19	38.4	39.1	Ohsaki (1970)
18	1968 Tokachioki	Hachinohe - 4	No	2.4	0.9	46	31	0.23	232	1.00	0.22	8.30	0.79	1.49	0.18	5.0	13.95	1.00	0.82	1.00	1.21	1.81	25.1	25.7	Ohsaki (1970)
19	1968 Tokachioki	Hachinohe-6	Yes	4.1	0.6	78	44	0.23	216	0.99	0.25	8.30	0.79	1.32	0.24	5.0	4.65	1.00	0.89	1.00	1.09	1.51	6.8	7.3	Ohsaki (1970)
20	1968 Tokachioki	Nanaehama1-2-3	Yes	3.0	0.8	55	33	0.20	171	0.97	0.21	8.30	0.79	1.46	0.18	21.7	5.55	1.00	0.84	1.00	1.17	1.75	9.6	11.9	Kishida (1970)
21	1968 Tokachioki	Aomori Station	Yes	5.8	0.0	108	52	0.21	158	0.87	0.25	8.30	0.79	1.25	0.25	3.0	9.00	1.00	0.94	1.00	1.22	1.40	14.4	15.0	Yasuda and Tohno (1988)
22	1971 San Fernando	Juvenile Hall	Yes	5.4	4.3	94	84	0.45	165	0.86	0.28	6.60	1.35	1.07	0.20	65.3	3.30	1.00	0.90	1.00	1.13	1.10	3.7	7.1	Bennett (1989)
23	1971 San Fernando	Van Norman	Yes	6.2	5.0	110	97	0.45	189	0.90	0.30	6.60	1.35	1.01	0.22	59.3	7.35	1.00	0.93	1.00	1.13	1.02	7.9	11.5	Bennett (1989)
24	1975 Haicheng	Panjin Ch. F. P.	Yes	8.0	1.5	148	85	0.13	186	0.86	0.13	7.00	1.17	1.06	0.10	67.0	8.10	1.00	1.00	1.00	0.83	1.09	7.4	10.9	Shengcong et al. (1983)
25	1975 Haicheng	Ying Kou G. F. P.	Yes	7.0	1.5	130	76	0.20	186	0.89	0.20	7.00	1.17	1.10	0.15	48.0	12.40	1.00	0.98	1.00	1.00	1.16	14.0	17.9	Shengcong et al. (1983)
26	1975 Haicheng	Ying Kou P. P.	Yes	7.5	1.5	139	80	0.20	171	0.81	0.18	7.00	1.17	1.08	0.14	20.0	10.30	1.00	0.99	1.00	1.00	1.12	11.4	13.6	Shengcong et al. (1983)
27	1976 Guatemala	Amatitlan B-1	Yes	9.1	1.5	122	47	0.14	122	0.57	0.13	7.50	1.00	1.29	0.10	3.0	4.30	1.00	1.00	1.00	0.75	1.46	4.7	5.2	Seed et al.(1979)
28	1976 Guatemala	Amatitlan B-2	No/Yes	4.3	2.4	49	31	0.14	128	0.82	0.11	7.50	1.00	1.49	0.08	3.0	7.95	1.00	0.86	1.00	0.75	1.81	9.3	9.8	Seed et al. (1979)
29	1976 Guatemala	Amatitlan B-3&4	No	10.2	3.8	126	64	0.14	158	0.68	0.12	7.50	1.00	1.17	0.10	3.0	15.10	1.00	1.00	1.00	0.75	1.26	14.3	14.9	Seed et al. (1979)
30	1976 Tangshan	Coastal Region	Yes	4.5	1.1	83	50	0.13	180	0.95	0.13	7.60	0.97	1.27	0.11	12.0	9.45	1.00	0.90	1.00	1.00	1.43	12.2	13.5	Shengcong et al. (1983)
31	1976 Tangshan	Le Ting L8-14	Yes	4.8	1.0	88	51	0.20	213	0.98	0.22	7.60	0.97	1.26	0.18	12.0	9.85	1.00	0.91	1.00	1.00	1.41	12.7	14.0	Shengcong et al. (1983)
32	1976 Tangshan	Luan Nan-L1	No	3.5	2.9	58	51	0.22	216	0.99	0.16	7.60	0.97	1.26	0.13	5.0	19.55	1.00	0.87	1.00	1.00	1.41	23.9	24.5	Shengcong et al. (1983)
33	1976 Tangshan	Luan Nan-L2	Yes	3.5	1.1	66	43	0.22	195	0.98	0.22	7.60	0.97	1.34	0.17	3.0	5.90	1.00	0.87	1.00	1.00	1.54	7.9	8.4	Shengcong et al. (1983)
34	1976 Tangshan	Qing Jia Ying	Yes	5.5	0.9	106	61	0.35	216	0.97	0.39	7.60	0.97	1.19	0.33	20.0	18.30	1.00	0.94	1.00	1.00	1.29	22.1	24.6	Shengcong et al. (1983)
35	1976 Tangshan	Tangshan City	No	4.5	3.0	81	67	0.50	259	1.00	0.40	7.60	0.97	1.15	0.35	10.0	29.70	1.00	0.90	1.00	1.00	1.23	33.1	34.6	Shengcong et al. (1983)
36	1976 Tangshan	Yao Yuan Village	Yes	4.3	1.0	81	49	0.20	198	0.97	0.21	7.60	0.97	1.27	0.17	20.0	8.65	1.00	0.90	1.00	1.00	1.43	11.1	13.3	Shengcong et al. (1983)
37	1977 Argentina	San Juan B-1	Yes	8.2	4.6	151	115	0.20	186	0.86	0.15	7.50	1.00	0.96	0.15	20.0	9.00	1.00	0.98	1.00	0.75	0.94	6.2	8.2	Idriss et al. (1979)
38	1977 Argentina	San Juan B-3	Yes	11.7	6.7	209	161	0.20	177	0.71	0.12	7.50	1.00	0.86	0.14	20.0	14.10	1.00	1.00	1.00	0.75	0.79	8.4	10.5	Idriss et al. (1979)
39	1977 Argentina	San Juan B-4	No	2.4	1.2	45	33	0.20	180	0.98	0.17	7.50	1.00	1.46	0.12	4.0	13.60	1.00	0.77	1.00	0.75	1.75	13.8	14.3	Idriss et al. (1979)
40	1977 Argentina	San Juan B-5	No	2.9	2.1	52	44	0.20	204	0.99	0.15	7.50	1.00	1.32	0.11	3.0	15.30	1.00	0.80	1.00	0.75	1.51	13.8	14.4	Idriss et al. (1979)
41	1977 Argentina	San Juan B-6	Yes	4.6	1.8	83	56	0.20	192	0.96	0.18	7.50	1.00	1.22	0.15	50.0	6.00	1.00	0.87	1.00	0.75	1.34	5.3	8.7	Idriss et al. (1979)
42	1978 Miyagiken-Oki M=6.7	Arahama	No	5.0	0.9	93	53	0.10	186	0.94	0.11	6.50	1.39	1.25	0.06	0.0	8.90	1.00	0.92	1.00	1.09	1.39	12.4	12.9	Tohno et al. (1981) & Fear et al. (1995)
43	1978 Miyagiken-Oki M=6.7	Hiyori-18	No	3.3	2.4	57	49	0.14	195	0.98	0.10	6.50	1.39	1.27	0.06	20.0	9.10	1.00	0.86	1.00	1.09	1.43	12.2	14.4	Tsuchida et al. (1979, 1980)
44	1978 Miyagiken-Oki M=6.7	Ishinomaki-2	No	3.7	1.4	68	45	0.12	158	0.92	0.11	6.50	1.39	1.31	0.06	10.0	4.00	1.00	0.88	1.00	1.09	1.50	5.7	6.7	Ishihara et al. (1980)
45	1978 Miyagiken-Oki M=6.7	Kitawabuchi-2	No	3.5	3.0	57	52	0.14	140	0.88	0.09	6.50	1.39	1.25	0.05	5.0	11.10	1.00	0.87	1.00	1.00	1.40	13.5	14.0	Iwasaki et al. (1978)
46	1978 Miyagiken-Oki M=6.7	Nakajima-18	No	4.3	2.4	78	60	0.14	198	0.97	0.11	6.50	1.39	1.19	0.07	3.0	9.60	1.00	0.90	1.00	1.09	1.30	12.2	12.7	Tsuchida et al. (1979, 1980)
47	1978 Miyagiken-Oki M=6.7	Nakamura 4	Yes	4.0	0.5	75	40	0.12	213	0.98	0.14	6.50	1.39	1.36	0.07	5.0	5.60	1.00	0.89	1.00	1.00	1.59	7.9	8.4	Iwasaki et al. (1978)

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 (Cont'd.) Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
48	1978 Miyagiken-Oki M=6.7	Nakamura 5	No	3.4	1.3	62	41	0.12	189	0.97	0.11	6.50	1.39	1.35	0.06	4.0	7.10	1.00	0.86	1.00	1.00	1.57	9.6	10.1	Iwasaki et al. (1978)
49	1978 Miyagiken-Oki M=6.7	Oiiri-1	No	5.9	4.3	105	89	0.14	160	0.83	0.09	6.50	1.39	1.04	0.06	5.0	9.15	1.00	0.95	1.00	1.00	1.07	9.3	9.8	Iwasaki et al. (1978)
50	1978 Miyagiken-Oki M=6.7	Shiomi-6	No	4.5	2.4	81	61	0.14	183	0.94	0.11	6.50	1.39	1.19	0.07	10.0	7.30	1.00	0.90	1.00	1.09	1.29	9.3	10.4	Tsuchida et al. (1979, 1980)
51	1978 Miyagiken-Oki M=6.7	Yuriage Br-1	No	3.5	1.7	59	41	0.12	204	0.98	0.11	6.50	1.39	1.35	0.06	10.0	2.95	1.00	0.87	1.00	1.00	1.56	4.0	5.0	Iwasaki et al. (1978)
52	1978 Miyagiken-Oki M=6.7	Yuriage Br-2	No	2.4	1.3	43	32	0.12	229	1.00	0.11	6.50	1.39	1.48	0.05	7.0	11.40	1.00	0.82	1.00	1.12	1.79	18.7	19.5	Iwasaki et al. (1978)
53	1978 Miyagiken-Oki M=6.7	Yuriage Br-3	No	3.0	0.3	56	29	0.12	201	0.98	0.15	6.50	1.39	1.52	0.07	12.0	7.10	1.00	0.85	1.00	1.00	1.86	11.2	12.5	Iwasaki et al. (1978)
54	1978 Miyagiken-Oki M=6.7	Yuriagekami-1	No	3.7	1.8	60	42	0.12	198	0.97	0.11	6.50	1.39	1.34	0.06	60.0	2.00	1.00	0.87	1.00	1.00	1.55	2.7	6.0	Iwasaki et al. (1978)
55	1978 Miyagiken-Oki M=6.7	Yuriagekami-2	No	3.8	0.9	68	40	0.12	189	0.96	0.13	6.50	1.39	1.37	0.07	0.0	8.75	1.00	0.88	1.00	1.00	1.60	12.3	12.8	Iwasaki et al. (1978)
56	1978 Miyagiken-Oki M=7.4	Nakajima-18	Yes	4.3	2.4	78	60	0.24	198	0.97	0.20	7.70	0.94	1.19	0.18	3.0	9.60	1.00	0.90	1.00	1.09	1.30	12.2	12.7	Tsuchida et al. (1979, 1980)
57	1978 Miyagiken-Oki M=7.4	Arahama	Yes	5.0	0.9	93	53	0.20	186	0.94	0.22	7.70	0.94	1.25	0.18	0.0	8.90	1.00	0.92	1.00	1.09	1.39	12.4	12.9	Tohno et al. (1981), Fear et al. (1995)
58	1978 Miyagiken-Oki M=7.4	Hiyori-18	Yes	3.3	2.4	57	49	0.24	195	0.98	0.18	7.70	0.94	1.27	0.15	20.0	9.10	1.00	0.86	1.00	1.09	1.43	12.2	14.4	Tsuchida et al. (1979, 1980)
59	1978 Miyagiken-Oki M=7.4	Ishinomaki-2	Yes	3.7	1.4	68	45	0.20	158	0.93	0.18	7.70	0.94	1.31	0.15	10.0	4.00	1.00	0.88	1.00	1.09	1.50	5.7	6.7	Ishihara et al. (1980)
60	1978 Miyagiken-Oki M=7.4	Ishinomaki-4	No	4.2	1.4	144	117	0.20	198	0.97	0.16	7.70	0.94	0.95	0.17	10.0	23.40	1.00	0.89	1.00	1.21	0.93	23.5	24.8	Ishihara et al. (1980)
61	1978 Miyagiken-Oki M=7.4	Kitawabuchi-2	Yes	3.5	3.0	57	52	0.28	140	0.89	0.18	7.70	0.94	1.25	0.15	5.0	11.10	1.00	0.87	1.00	1.00	1.40	13.5	14.0	Iwasaki et al. (1978)
62	1978 Miyagiken-Oki M=7.4	Kitawabuchi-3	No	4.3	3.0	76	64	0.28	232	0.99	0.21	7.70	0.94	1.17	0.20	0.0	13.15	1.00	0.90	1.00	1.21	1.26	18.0	18.6	Iwasaki et al. (1978)
63	1978 Miyagiken-Oki M=7.4	Nakajima-2	No	4.6	2.4	84	63	0.24	198	0.97	0.20	7.70	0.94	1.17	0.18	26.0	11.90	1.00	0.91	1.00	1.09	1.27	14.9	17.9	Tsuchida et al. (1979, 1980)
64	1978 Miyagiken-Oki M=7.4	Nakamura 1	No	3.0	0.9	57	36	0.32	219	0.99	0.32	7.70	0.94	1.41	0.24	4.0	15.75	1.00	0.85	1.00	1.12	1.67	24.9	25.6	Iwasaki et al. (1978)
65	1978 Miyagiken-Oki M=7.4	Nakamura 4	Yes	4.0	0.5	75	40	0.32	213	0.98	0.38	7.70	0.94	1.36	0.30	5.0	5.60	1.00	0.89	1.00	1.00	1.59	7.9	8.4	Iwasaki et al. (1978)
66	1978 Miyagiken-Oki M=7.4	Nakamura 5	Yes	3.4	1.3	62	41	0.32	189	0.97	0.30	7.70	0.94	1.35	0.24	4.0	7.10	1.00	0.86	1.00	1.00	1.57	9.6	10.1	Iwasaki et al. (1978)
67	1978 Miyagiken-Oki M=7.4	Oiiri-1	Yes	5.9	4.3	105	89	0.24	160	0.85	0.16	7.70	0.94	1.04	0.16	5.0	9.15	1.00	0.95	1.00	1.00	1.07	9.3	9.8	Iwasaki et al. (1978)
68	1978 Miyagiken-Oki M=7.4	Shiomi-6	Yes	4.5	2.4	81	61	0.24	183	0.95	0.20	7.70	0.94	1.19	0.18	10.0	7.30	1.00	0.90	1.00	1.09	1.29	9.3	10.4	Tsuchida et al. (1979, 1980)
69	1978 Miyagiken-Oki M=7.4	Yuriage Br-1	Yes	3.5	1.7	59	41	0.24	204	0.97	0.22	7.70	0.94	1.35	0.17	10.0	2.95	1.00	0.87	1.00	1.00	1.56	4.0	5.0	Iwasaki et al. (1978)
70	1978 Miyagiken-Oki M=7.4	Yuriage Br-2	Yes	2.4	1.3	43	32	0.24	229	1.00	0.21	7.70	0.94	1.48	0.15	7.0	11.40	1.00	0.82	1.00	1.12	1.79	18.7	19.5	Iwasaki et al. (1978)
71	1978 Miyagiken-Oki M=7.4	Yuriage Br-3	Yes	3.0	0.3	56	29	0.24	201	0.99	0.29	7.70	0.94	1.52	0.21	12.0	7.10	1.00	0.85	1.00	1.00	1.86	11.2	12.5	Iwasaki et al. (1978)
72	1978 Miyagiken-Oki M=7.4	Yuriage Br-5	No	7.5	1.3	146	85	0.24	238	0.98	0.26	7.70	0.94	1.06	0.26	17.0	20.45	1.00	0.99	1.00	1.12	1.09	24.6	26.8	Iwasaki et al. (1978)
73	1978 Miyagiken-Oki M=7.4	Yuriagekami-1	Yes	3.7	1.8	60	42	0.24	198	0.98	0.22	7.70	0.94	1.34	0.17	60.0	2.00	1.00	0.87	1.00	1.00	1.55	2.7	6.0	Iwasaki et al. (1978)
74	1978 Miyagiken-Oki M=7.4	Yuriagekami-2	Yes	3.8	0.9	68	40	0.24	189	0.97	0.26	7.70	0.94	1.37	0.20	0.0	8.75	1.00	0.88	1.00	1.00	1.60	12.3	12.8	Iwasaki et al. (1978)
75	1978 Miyagiken-Oki M=7.4	Yuriagekami-3	No	6.0	2.2	113	75	0.24	216	0.97	0.23	7.70	0.94	1.11	0.22	0.0	21.10	1.00	0.95	1.00	1.12	1.16	26.0	26.7	Iwasaki et al. (1978)
76	1979 Imperial Valley	Heber Road A1	No	3.4	1.8	60	44	0.47	*	0.82	0.34	6.53	1.38	1.32	0.19	13.0	32.85	1.00	0.82	1.00	1.13	1.52	46.5	48.7	Youd and Bennett (1983), Benett et. al. (1979)
77	1979 Imperial Valley	Heber Road A2	Yes	3.2	1.8	53	39	0.47	*	0.78	0.32	6.53	1.38	1.38	0.17	20.9	2.35	1.00	0.81	1.00	1.13	1.61	3.5	5.5	Youd and Bennett (1983), Benett et. al. (1979)
78	1979 Imperial Valley	Heber Road A3	No	3.4	1.8	57	42	0.47	*	0.75	0.31	6.53	1.38	1.34	0.17	25.3	12.95	1.00	0.82	1.00	1.13	1.55	18.6	21.7	Youd and Bennett (1983), Benett et. al. (1979)
79	1979 Imperial Valley	Kornbloom B	No	4.0	2.7	68	55	0.13	*	0.83	0.09	6.53	1.38	1.23	0.05	83.0	5.40	1.00	0.85	1.00	1.13	1.36	7.1	10.6	Bennett et al. (1984)
80	1979 Imperial Valley	McKim Ranch A	Yes	2.7	1.5	49	37	0.51	160	0.94	0.41	6.53	1.38	1.40	0.21	19.8	5.40	1.00	0.79	1.00	1.13	1.65	7.9	10.0	Bennett et al. (1984)
81	1979 Imperial Valley	Radio Tower B1	Yes	4.3	2.0	72	50	0.18	*	0.97	0.16	6.53	1.38	1.27	0.09	43.5	4.45	1.00	0.86	1.00	1.13	1.42	6.2	9.7	Bennett et al. (1984)
82	1979 Imperial Valley	Radio Tower B2	No	2.5	2.0	41	36	0.18	*	0.99	0.13	6.53	1.38	1.42	0.07	18.0	11.25	1.00	0.77	1.00	1.13	1.68	16.6	18.7	Bennett et al.(1984)
83	1979 Imperial Valley	River Park A	Yes	1.1	0.3	18	10	0.16	*	0.99	0.18	6.53	1.38	2.15	0.06	91.0	2.65	1.00	0.66	1.00	1.13	2.00	4.0	7.3	Youd et al. (1983)
84	1980 Mid-Chiba	Owi-1	No	5.5	1.0	104	60	0.08	149	0.81	0.07	6.10	1.62	1.20	0.04	30.0	5.80	1.00	0.94	1.15	1.09	1.30	8.9	12.0	Ishihara et. al. (1981)
85	1980 Mid-Chiba	Owi-2	No	14.5	1.0	273	141	0.08	149	0.49	0.05	6.10	1.62	0.89	0.03	27.0	3.50	1.00	1.00	1.15	1.09	0.85	3.7	6.3	Ishihara et. al. (1981)
86	1981 WestMorland	Kornbloom B	Yes	4.0	2.7	68	55	0.19	*	0.90	0.14	5.90	1.75	1.23	0.06	83.0	5.40	1.00	0.85	1.00	1.13	1.36	7.1	10.6	Bennett et al. (1984)
87	1981 WestMorland	Radio Tower B1	Yes	4.3	2.0	72	50	0.17	*	0.89	0.14	5.90	1.75	1.27	0.06	43.5	4.45	1.00	0.86	1.00	1.13	1.42	6.2	9.7	Bennett et al. (1984)
88	1981 WestMorland	Radio Tower B2	No	2.5	2.0	41	36	0.16	*	0.98	0.12	5.90	1.75	1.42	0.05	18.0	11.25	1.00	0.77	1.00	1.13	1.68	16.6	18.7	Bennett et al.(1984)
89	1981 WestMorland	River Park A	No	1.1	0.3	18	10	0.17	*	0.99	0.19	5.90	1.75	2.15	0.05	91.0	2.65	1.00	0.66	1.00	1.13	2.00	4.0	7.3	Youd et al. (1983)
90	1981 WestMorland	River Park C	No	4.3	0.3	76	38	0.17	*	0.97	0.22	5.90	1.75	1.40	0.09	13.2	12.40	1.00	0.86	1.00	1.13	1.64	19.9	21.5	Youd et al. (1983)
91	1981 WestMorland	Wildlife B	Yes	4.7	0.9	86	49	0.23	*	0.89	0.23	5.90	1.75	1.28	0.11	26.2	7.85	1.00	0.88	1.00	1.13	1.44	11.3	14.1	Bennett et al. (1984)

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 (Cont'd.) Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
92	1981 WestMorland	McKim Ranch A	No	2.7	1.5	49	37	0.09	*	0.93	0.07	5.90	1.75	1.40	0.03	19.8	5.40	1.00	0.79	1.00	1.13	1.65	7.9	10.0	Bennett et al. (1984)
93	1983 Nihonkai-Chubu M=7.1	Arayamotomachi	No	3.3	1.0	56	34	0.15	158	0.93	0.15	7.10	1.14	1.44	0.09	5.0	4.40	1.00	0.86	1.00	1.22	1.72	7.9	8.4	Yasuda and Tohno (1988)
94	1983 Nihonkai-Chubu M=7.1	Arayamotomachi Coarse Sand	No	9.3	1.0	171	90	0.15	158	0.70	0.13	7.10	1.14	1.04	0.11	0.0	13.10	1.00	1.00	1.00	1.22	1.06	16.9	17.5	Yasuda and Tohno (1988)
95	1983 Nihonkai-Chubu M=7.1	Takeda Ele.Sch.	Yes	4.5	0.4	80	40	0.12	143	0.85	0.13	7.10	1.14	1.37	0.09	0.0	7.85	1.00	0.90	1.00	1.22	1.60	13.8	14.3	Yasuda and Tohno (1988)
96	1983 Nihonkai-Chubu M=7.7	Akita Station	No	3.0	1.8	57	44	0.20	267	1.00	0.17	7.70	0.94	1.32	0.14	2.7	10.30	1.00	0.85	1.00	1.22	1.51	16.0	16.6	Iai. et. al. (1989)
97	1983 Nihonkai-Chubu M=7.7	Aomori Station	Yes	5.8	0.0	108	52	0.12	158	0.86	0.14	7.70	0.94	1.25	0.11	3.0	9.00	1.00	0.94	1.00	1.22	1.40	14.4	15.0	Yasuda and Tohno (1988)
98	1983 Nihonkai-Chubu M=7.7	Arayamotomachi	Yes	3.3	1.0	56	34	0.20	158	0.94	0.20	7.70	0.94	1.44	0.15	5.0	4.40	1.00	0.86	1.00	1.22	1.72	7.9	8.4	Yasuda and Tohno (1988)
99	1983 Nihonkai-Chubu M=7.7	Gaiko 1&2	Yes	10.5	1.5	196	107	0.21	146	0.62	0.16	7.70	0.94	0.98	0.17	5.5	5.80	1.00	1.00	1.00	1.22	0.97	6.9	7.4	Iai. et. al. (1989)
100	1983 Nihonkai-Chubu M=7.7	Gaiko Wharf B-2	Yes	7.5	0.4	141	71	0.23	168	0.82	0.24	7.70	0.94	1.13	0.23	1.0	7.80	1.00	0.99	1.00	1.22	1.19	11.2	11.7	Hamada (1992)
101	1983 Nihonkai-Chubu M=7.7	Hakodate	No	5.3	1.6	99	63	0.05	172	0.92	0.05	7.70	0.94	1.17	0.04	66.0	3.65	1.00	0.93	1.00	1.22	1.27	5.2	8.6	Iai. et. al. (1989)
102	1983 Nihonkai-Chubu M=7.7	Nakajima No. 1 (5)	Yes	7.5	1.8	139	82	0.21	160	0.79	0.18	7.70	0.94	1.07	0.18	14.6	6.50	1.00	0.99	1.00	1.22	1.11	8.7	10.2	Iai. et. al. (1989)
103	1983 Nihonkai-Chubu M=7.7	Nakajima No. 2 (1)	Yes	5.5	1.5	107	67	0.21	172	0.90	0.19	7.70	0.94	1.15	0.18	2.0	4.70	1.00	0.94	1.00	1.22	1.23	6.6	7.1	Iai. et. al. (1989)
104	1983 Nihonkai-Chubu M=7.7	Nakajima No. 2 (2)	Yes	3.9	1.5	73	50	0.20	183	0.96	0.19	7.70	0.94	1.27	0.16	8.0	5.90	1.00	0.88	1.00	1.22	1.43	9.0	9.9	Iai. et. al. (1989)
105	1983 Nihonkai-Chubu M=7.7	Nakajima No. 3 (3)	Yes	2.4	1.6	39	31	0.21	152	0.95	0.16	7.70	0.94	1.49	0.12	3.0	3.00	1.00	0.82	1.00	1.22	1.81	5.4	5.9	Iai. et. al. (1989)
106	1983 Nihonkai-Chubu M=7.7	Nakajima No. 3 (4)	Yes	6.5	1.7	120	72	0.20	168	0.86	0.19	7.70	0.94	1.12	0.18	2.0	7.80	1.00	0.96	1.00	1.22	1.18	10.8	11.4	Iai. et. al. (1989)
107	1983 Nihonkai-Chubu M=7.7	Noshiro Section N-7	Yes	3.5	1.8	63	46	0.25	171	0.95	0.21	7.70	0.94	1.30	0.17	1.0	9.80	1.00	0.87	1.00	1.22	1.48	15.3	15.9	Hamada (1992)
108	1983 Nihonkai-Chubu M=7.7	Ohama No. 2 (2)	Yes	6.8	0.7	116	56	0.20	137	0.73	0.20	7.70	0.94	1.22	0.17	1.1	4.60	1.00	0.97	1.00	1.22	1.34	7.3	7.8	Iai. et. al. (1989)
109	1983 Nihonkai-Chubu M=7.7	Ohama No. Rvt. (1)	No	5.5	1.5	108	68	0.17	213	0.97	0.18	7.70	0.94	1.15	0.16	3.0	17.15	1.00	0.94	1.00	1.22	1.22	24.0	24.6	Iai. et. al. (1989)
110	1983 Nihonkai-Chubu M=7.7	Takeda Elementary Sch.	Yes	4.5	0.4	80	40	0.28	143	0.85	0.31	7.70	0.94	1.37	0.24	0.0	7.85	1.00	0.90	1.00	1.22	1.60	13.8	14.3	Yasuda and Tohno (1988)
111	1987 Elmore Ranch	Radio Tower B1	No	4.3	2.0	72	50	0.09	*	0.97	0.08	6.20	1.56	1.27	0.04	43.5	4.45	1.00	0.86	1.00	1.13	1.42	6.2	9.7	Bennett et al. (1984)
112	1987 Elmore Ranch	Wildlife B	No	4.7	0.9	86	49	0.13	*	0.75	0.11	6.20	1.56	1.28	0.06	26.2	7.85	1.00	0.88	1.00	1.13	1.44	11.3	14.1	Bennett et al. (1984)
113	1987 Superstition Hills	Heber Road A1	No	3.4	1.8	60	44	0.16	*	0.96	0.13	6.54	1.37	1.32	0.07	13.0	32.85	1.00	0.82	1.00	1.13	1.52	46.5	48.7	Youd and Bennett (1983), Benett et. al. (1979)
114	1987 Superstition Hills	Heber Road A2	No	3.2	1.8	53	39	0.15	*	0.95	0.13	6.54	1.37	1.38	0.07	20.9	2.35	1.00	0.81	1.00	1.13	1.61	3.5	5.5	Youd and Bennett (1983), Benett et. al. (1979)
115	1987 Superstition Hills	Heber Road A3	No	3.4	1.8	57	42	0.13	*	0.95	0.11	6.54	1.37	1.34	0.06	25.3	12.95	1.00	0.82	1.00	1.13	1.55	18.6	21.7	Youd and Bennett (1983), Benett et. al. (1979)
116	1987 Superstition Hills	Kornbloom B	No	4.0	2.7	68	55	0.17	*	0.94	0.13	6.54	1.37	1.23	0.08	83.0	5.40	1.00	0.85	1.00	1.13	1.36	7.1	10.6	Bennett et al. (1984)
117	1987 Superstition Hills	McKim Ranch A	No	2.7	1.5	49	37	0.16	*	0.95	0.13	6.54	1.37	1.40	0.07	19.8	5.40	1.00	0.79	1.00	1.13	1.65	7.9	10.0	Bennett et al. (1984
118	1987 Superstition Hills	Radio Tower B1	No	4.3	2.0	72	50	0.20	*	0.94	0.18	6.54	1.37	1.27	0.10	43.5	4.45	1.00	0.86	1.00	1.13	1.42	6.2	9.7	Bennett et al. (1984)
119	1987 Superstition Hills	Radio Tower B2	No	2.5	2.0	41	36	0.18	*	0.99	0.13	6.54	1.37	1.42	0.07	18.0	11.25	1.00	0.77	1.00	1.13	1.68	16.6	18.7	Bennett et al.(1984)
120	1987 Superstition Hills	River Park A	No	1.1	0.3	18	10	0.19	*	0.99	0.21	6.54	1.37	2.15	0.07	91.0	2.65	1.00	0.66	1.00	1.13	2.00	4.0	7.3	Youd et al. (1983)
121	1987 Superstition Hills	River Park C	No	4.3	0.3	76	38	0.19	*	0.93	0.23	6.54	1.37	1.40	0.12	13.2	12.40	1.00	0.86	1.00	1.13	1.64	19.9	21.5	Youd et al. (1983)
122	1987 Superstition Hills	Wildlife B	Yes	4.7	0.9	86	49	0.20	*	0.82	0.19	6.54	1.37	1.28	0.11	26.2	7.85	1.00	0.88	1.00	1.13	1.44	11.3	14.1	Bennett et al. (1984)
123	1989 Loma Prieta	Alameda BF Dike	No	6.5	3.0	125	91	0.24	248	0.99	0.21	6.93	1.20	1.04	0.17	7.0	37.10	1.30	0.94	1.00	0.92	1.06	43.8	45.0	Mitchell et al. (1994)
124	1989 Loma Prieta	Farris Farm	Yes	6.0	4.5	99	84	0.37	*	0.95	0.27	6.93	1.20	1.06	0.21	8.0	9.00	1.00	0.92	1.00	1.13	1.10	10.3	11.2	Holzer et al. (1994)
125	1989 Loma Prieta	Hall Avenue	No	4.6	3.5	75	64	0.14	*	0.72	0.08	6.93	1.20	1.17	0.05	30.0	4.60	1.10	0.88	1.00	0.92	1.26	5.1	8.0	Mitchell et al. (1994)
126	1989 Loma Prieta	MBARI NO:3 EB-1	No	2.5	2.0	44	39	0.24	*	0.99	0.17	6.93	1.20	1.37	0.11	1.0	18.45	1.00	0.77	1.00	1.00	1.60	22.9	23.5	Boulanger et al (1997)
127	1989 Loma Prieta	MBARI NO:3 EB-5	No	4.1	1.8	76	54	0.27	*	0.99	0.25	6.93	1.20	1.24	0.17	1.0	15.15	1.00	0.86	1.00	1.00	1.37	17.8	18.4	Boulanger et al. (1997)
128	1989 Loma Prieta	Miller Farm CMF10	No	8.5	3.0	160	106	0.41	*	0.88	0.35	6.93	1.20	0.99	0.30	20.0	18.30	1.00	0.99	1.00	1.13	0.98	20.1	22.6	Bennett and Tinsley (1995)
129	1989 Loma Prieta	Miller Farm CMF3	Yes	6.6	5.7	117	108	0.46	*	0.83	0.27	6.93	1.20	0.98	0.23	27.3	10.30	1.00	0.94	1.00	1.13	0.97	10.7	13.6	Bennett and Tinsley (1995)
130	1989 Loma Prieta	Miller Farm CMF5	Yes	7.0	4.7	134	111	0.41	*	0.90	0.29	6.93	1.20	0.97	0.25	13.0	20.00	1.00	0.95	1.00	1.13	0.95	20.6	22.2	Bennett and Tinsley (1995)
131	1989 Loma Prieta	Miller Farm CMF8	Yes	6.5	4.9	107	91	0.46	*	0.73	0.26	6.93	1.20	1.04	0.21	15.5	8.95	1.00	0.94	1.00	1.13	1.05	10.0	11.7	Bennett and Tinsley (1995)
132	1989 Loma Prieta	POO7-2	Yes	6.2	3.0	118	88	0.22	*	0.86	0.17	6.93	1.20	1.05	0.13	3.0	12.70	1.10	0.93	1.00	0.92	1.08	12.8	13.3	Mitchell et al. (1994)
133	1989 Loma Prieta	POO7-3	Yes	6.0	3.0	115	86	0.22	*	0.83	0.16	6.93	1.20	1.06	0.13	5.0	16.20	1.10	0.92	1.00	0.92	1.09	16.4	17.0	Mitchell et al. (1994)
134	1989 Loma Prieta	POR-2&3&4	Yes	4.9	3.5	79	65	0.15	*	0.78	0.09	6.93	1.20	1.16	0.07	50.0	3.15	1.10	0.89	1.00	0.92	1.24	3.5	6.8	Mitchell et al. (1994)

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 (Cont'd.) Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
135	1989 Loma Prieta	Sandholdt UC-B10	Yes	3.2	1.7	58	43	0.26	*	0.99	0.22	6.93	1.20	1.33	0.14	2.0	9.15	1.00	0.81	1.00	1.25	1.53	14.3	14.8	Boulanger et al. (1997)
136	1989 Loma Prieta	SFOBB-1&2	Yes	6.3	3.0	120	89	0.27	*	0.79	0.19	6.93	1.20	1.05	0.15	8.0	7.50	1.20	0.93	1.00	0.92	1.07	8.2	9.0	Mitchell et al. (1994)
137	1989 Loma Prieta	State Beach UC-B1	Yes	2.7	1.8	49	40	0.29	*	0.95	0.22	6.93	1.20	1.36	0.13	1.7	5.05	1.00	0.79	1.00	1.25	1.59	7.9	8.4	Boulanger et al. (1997)
138	1989 Loma Prieta	State Beach UC-B2	Yes	4.7	2.7	91	71	0.24	*	0.99	0.20	6.93	1.20	1.13	0.15	1.0	13.25	1.00	0.88	1.00	1.25	1.19	17.4	18.0	Boulanger et al. (1997)
139	1989 Loma Prieta	Treasure Island	Yes	5.3	1.5	91	55	0.18	*	0.88	0.17	6.93	1.20	1.23	0.12	20.0	5.05	1.10	0.90	1.00	1.13	1.36	7.7	9.8	De Alba et al. (1994)
140	1989 Loma Prieta	WoodMarine UC-B4	Yes	1.8	1.0	31	24	0.25	*	0.99	0.21	6.93	1.20	1.62	0.11	35.0	5.75	1.00	0.72	1.00	1.00	2.00	8.3	11.9	Boulanger et al. (1997)
141	1989 Loma Prieta	General Fish	No	2.0	1.7	36	32	0.25	210	0.99	0.18	6.93	1.20	1.47	0.10	5.0	11.50	1.00	0.74	1.00	1.00	1.77	15.1	15.7	Boulanger et al (1995,1997)
142	1989 Loma Prieta	Marine Laboratory UC-B1	Yes	4.0	2.4	72	57	0.24	*	0.99	0.20	6.93	1.20	1.22	0.13	3.0	10.30	1.00	0.85	1.00	1.00	1.34	11.7	12.2	Boulager et al. (1997)
143	1989 Loma Prieta	Marine Laboratory UC-B2	Yes	3.5	2.5	63	53	0.26	*	0.99	0.20	6.93	1.20	1.24	0.13	3.0	13.00	1.00	0.83	1.00	1.00	1.38	14.9	15.4	Boulanger et al. (1997)
144	1989 Loma Prieta	Marine Laboratory_F1-F7	Yes	4.6	1.5	89	59	0.25	238	0.99	0.24	6.93	1.20	1.20	0.17	3.0	13.80	1.00	0.87	1.00	1.25	1.31	19.8	20.4	Boulanger et al (1997)
145	1989 Loma Prieta	MBARI N0.4-B4B5EB2EB3	No	5.1	2.0	98	67	0.25	216	0.98	0.23	6.93	1.20	1.15	0.17	5.0	23.60	1.00	0.89	1.00	1.00	1.23	25.8	26.5	Boulanger et al (1997)
146	1989 Loma Prieta	Miller Farm	Yes	6.0	4.0	108	89	0.42	*	0.90	0.30	6.93	1.20	1.05	0.24	22.0	7.95	1.00	0.92	1.00	1.13	1.07	8.9	11.2	Holzer et al. (1994)
147	1990 Luzon	Cereenan St. B-12	No	5.0	2.3	92	66	0.25	244	0.99	0.23	7.70	0.94	1.16	0.21	19.0	34.80	1.00	0.92	1.00	0.65	1.24	25.8	28.3	Wakamatsu (1992)
148	1990 Luzon	Perez Blv. B-11	Yes	7.3	2.3	133	85	0.25	186	0.89	0.23	7.70	0.94	1.06	0.23	19.0	19.90	1.00	0.98	1.00	0.65	1.09	13.9	16.0	Wakamatsu (1992)
149	1993 Kushiro-Oki	Kushiro Port Seismo St.	Yes	3.6	2.0	66	50	0.40	232	0.99	0.34	7.60	0.97	1.27	0.28	5.0	16.90	1.00	0.87	1.00	1.22	1.42	25.4	26.1	Iai et al. (1994)
150	1993 Kushiro-Oki	Kushiro Port Site A	Yes	5.3	2.0	97	65	0.40	204	0.96	0.37	7.60	0.97	1.16	0.33	2.0	11.70	1.00	0.93	1.00	1.22	1.25	16.5	17.1	Iai et al. (1994)
151	1993 Kushiro-Oki	Kushiro Port Site D	No	10.8	1.6	210	120	0.40	218	0.88	0.40	7.60	0.97	0.94	0.43	0.0	26.70	1.00	1.00	1.00	1.22	0.92	29.8	30.5	Iai et al. (1994)
152	1994 Northridge	Balboa Blv. Unit C	Yes	9.0	7.2	160	142	0.69	*	0.71	0.36	6.70	1.30	0.89	0.31	48.0	19.90	1.00	1.00	1.00	1.13	0.84	19.0	23.3	Bennett et al.(1998)
153	1994 Northridge	Potrero Canyon C1	Yes	6.5	3.3	120	88	0.40	160	0.79	0.28	6.70	1.30	1.05	0.20	44.5	9.50	1.00	0.94	1.00	1.13	1.07	10.8	14.6	Bennett et al. (1998)
154	1994 Northridge	Wynne Ave. Unit C1	Yes	6.3	4.2	113	92	0.54	*	0.86	0.37	6.70	1.30	1.03	0.27	42.4	10.30	1.00	0.93	1.00	1.13	1.05	11.4	15.2	Bennett et al. (1998)
155	1995 Hyogoken-Nanbu	Ashiyama A (Mountain Sand 1)	No	5.2	3.5	94	77	0.40	192	0.94	0.30	6.90	1.21	1.10	0.22	18.0	16.60	1.00	0.93	1.00	1.22	1.15	21.5	23.7	Shibata et al. (1996)
156	1995 Hyogoken-Nanbu	Ashiyama A (Marine Sand)	No	8.0	3.5	148	104	0.40	192	0.86	0.32	6.90	1.21	0.99	0.26	2.0	25.50	1.00	1.00	1.00	1.22	0.99	30.6	31.3	Shibata et al. (1996)
157	1995 Hyogoken-Nanbu	Ashiyama C-D-E (Mountain Sand 2)	Yes	13.0	3.5	247	154	0.40	171	0.59	0.25	6.90	1.21	0.87	0.23	18.0	5.70	1.00	1.00	1.00	1.22	0.81	5.6	7.4	Shibata et al. (1996)
158	1995 Hyogoken-Nanbu	Ashiyama C-D-E (Marine Sand)	Yes	8.8	3.5	164	112	0.40	171	0.74	0.28	6.90	1.21	0.97	0.24	2.0	10.90	1.00	1.00	1.00	1.22	0.95	12.6	13.2	Shibata et al. (1996)
159	1995 Hyogoken-Nanbu	Kobe No 1	No	6.0	2.4	122	86	0.40	259	0.99	0.37	6.90	1.21	1.06	0.28	3.5	42.35	1.00	0.95	1.00	1.22	1.09	53.2	54.1	Tokimatsu (2000), Kobe City City Office (1999)
160	1995 Hyogoken-Nanbu	Kobe No 2	No	8.5	2.9	173	118	0.40	259	0.98	0.37	6.90	1.21	0.95	0.32	14.8	35.20	1.00	1.00	1.00	1.22	0.92	39.6	41.9	Tokimatsu (2000), Kobe City City Office (1999)
161	1995 Hyogoken-Nanbu	Kobe No 3	No	5.5	2.5	103	73	0.40	259	0.99	0.36	6.90	1.21	1.11	0.27	3.3	39.45	1.00	0.94	1.00	1.22	1.17	52.8	53.7	Tokimatsu (2000), Kobe City City Office (1999)
162	1995 Hyogoken-Nanbu	Kobe No 4	No	4.3	2.1	79	57	0.40	229	0.99	0.35	6.90	1.21	1.21	0.24	1.3	27.80	1.00	0.90	1.00	1.22	1.33	40.4	41.2	Tokimatsu (2000), Kobe City City Office (1999)
163	1995 Hyogoken-Nanbu	Kobe No 5	Yes	8.8	3.0	160	104	0.35	160	0.70	0.24	6.90	1.21	0.99	0.20	1.3	5.80	1.00	1.00	1.00	1.22	0.99	7.0	7.5	Tokimatsu (2000), Kobe City City Office (1999)
164	1995 Hyogoken-Nanbu	Kobe No 6	No	5.8	2.3	112	78	0.40	177	0.89	0.33	6.90	1.21	1.09	0.25	24.7	16.70	1.00	0.95	1.00	1.22	1.14	21.9	25.0	Tokimatsu (2000), Kobe City City Office (1999)
165	1995 Hyogoken-Nanbu	Kobe No 7	Yes	2.8	3.2	48	51	0.40	206	0.99	0.24	6.90	1.21	1.26	0.16	0.0	15.60	1.00	0.84	1.00	1.22	1.40	22.3	23.0	Tokimatsu (2000), Kobe City City Office (1999)
166	1995 Hyogoken-Nanbu	Kobe No 8	Yes	5.0	3.0	87	66	0.50	206	0.96	0.41	6.90	1.21	1.15	0.29	0.0	17.35	1.00	0.92	1.00	1.22	1.23	24.0	24.6	Tokimatsu (2000), Tokimatsu et al. (1997)
167	1995 Hyogoken-Nanbu	Kobe No 9	Yes	4.3	2.8	73	58	0.50	160	0.89	0.36	6.90	1.21	1.20	0.25	2.3	8.60	1.00	0.90	1.00	1.22	1.32	12.4	12.9	Tokimatsu (2000), Kobe City City Office (1999))

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 (Cont'd.) Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
168	1995 Hyogoken-Nanbu	Kobe No 10	No	7.5	4.5	144	114	0.60	213	0.92	0.45	6.90	1.21	0.96	0.39	8.8	23.10	1.00	0.99	1.00	1.22	0.94	26.2	27.4	Tokimatsu (2000), Tokimatsu et al. (1997)
169	1995 Hyogoken-Nanbu	Kobe No 11	Yes	6.8	1.5	125	73	0.50	137	0.67	0.37	6.90	1.21	1.12	0.28	5.0	5.55	1.00	0.97	1.00	1.22	1.18	7.7	8.2	Tokimatsu (2000), Tokimatsu et al. (1997)
170	1995 Hyogoken-Nanbu	Kobe No 12	No	5.3	3.2	92	71	0.50	198	0.94	0.40	6.90	1.21	1.13	0.29	14.0	19.60	1.00	0.92	1.00	1.22	1.20	26.2	28.1	Tokimatsu (2000), Kobe City City Office (1999)
171	1995 Hyogoken-Nanbu	Kobe No 13	Yes	6.5	2.3	124	83	0.50	180	0.87	0.42	6.90	1.21	1.07	0.32	15.0	9.30	1.00	0.96	1.00	1.22	1.10	12.0	13.7	Tokimatsu (2000), Tokimatsu et al. (1997)
172	1995 Hyogoken-Nanbu	Kobe No 14	No	4.8	3.1	89	73	0.50	171	0.90	0.36	6.90	1.21	1.12	0.27	18.5	16.00	1.00	0.91	1.00	1.22	1.18	21.0	23.3	Tokimatsu (2000), Kobe City City Office (1999)
173	1995 Hyogoken-Nanbu	Kobe No 15	Yes	5.8	3.7	99	78	0.50	171	0.86	0.36	6.90	1.21	1.09	0.27	4.7	15.00	1.00	0.94	1.00	1.22	1.14	19.6	20.2	Tokimatsu (2000), Kobe City City Office (1999)
174	1995 Hyogoken-Nanbu	Kobe No 16	No	4.5	2.5	83	62	0.60	192	0.95	0.49	6.90	1.21	1.18	0.34	5.0	17.55	1.00	0.90	1.00	1.22	1.27	24.6	25.2	Tokimatsu (2000), Tokimatsu et City al. (1997)
175	1995 Hyogoken-Nanbu	Kobe No 17	Yes	4.5	0.8	84	47	0.50	192	0.95	0.55	6.90	1.21	1.30	0.35	5.0	13.10	1.00	0.90	1.00	1.22	1.47	21.2	21.8	Tokimatsu (2000), Tokimatsu et al. (1997)
176	1995 Hyogoken-Nanbu	Kobe No 18	No	10.5	7.7	211	183	0.70	251	0.94	0.49	6.90	1.21	0.82	0.50	0.0	40.50	1.00	1.00	1.00	1.22	0.74	36.7	37.4	Tokimatsu (2000), Tokimatsu et al. (1997)
177	1995 Hyogoken-Nanbu	Kobe No 19	No	7.5	6.1	142	129	0.60	229	0.95	0.41	6.90	1.21	0.92	0.37	10.0	19.50	1.00	0.99	1.00	1.22	0.89	20.8	22.1	Tokimatsu (2000), Tokimatsu et al. (1997)
178	1995 Hyogoken-Nanbu	Kobe No 20	No	6.0	2.0	119	80	0.55	274	0.99	0.53	6.90	1.21	1.08	0.40	0.0	50.35	1.00	0.95	1.00	1.22	1.12	65.5	66.5	Tokimatsu (2000), Tokimatsu et al. (1997)
179	1995 Hyogoken-Nanbu	Kobe No 21	No	3.5	1.7	62	44	0.60	232	0.99	0.55	6.90	1.21	1.32	0.34	0.0	23.00	1.00	0.87	1.00	1.22	1.52	36.8	37.5	Tokimatsu (2000), Tokimatsu et al. (1997)
180	1995 Hyogoken-Nanbu	Kobe No 22	No	6.0	2.4	116	81	0.60	219	0.96	0.54	6.90	1.21	1.08	0.41	6.0	30.00	1.00	0.95	1.00	1.22	1.12	38.9	39.8	Tokimatsu (2000), Tokimatsu et al. (1997)
181	1995 Hyogoken-Nanbu	Kobe No 23	No	5.0	3.0	96	76	0.60	219	0.97	0.48	6.90	1.21	1.10	0.36	10.0	18.00	1.00	0.92	1.00	1.22	1.15	23.3	24.5	Tokimatsu (2000), Tokimatsu et al. (1997)
182	1995 Hyogoken-Nanbu	Kobe No 24	Yes	3.5	2.4	64	53	0.50	213	0.98	0.39	6.90	1.21	1.24	0.26	0.0	16.95	1.00	0.87	1.00	1.22	1.38	24.8	25.4	Tokimatsu (2000), Tokimatsu et al. (1997)
183	1995 Hyogoken-Nanbu	Kobe No 25	No	3.5	2.2	64	50	0.70	229	0.99	0.57	6.90	1.21	1.26	0.37	2.5	26.05	1.00	0.87	1.00	1.22	1.42	39.0	39.8	Tokimatsu (2000), Tokimatsu et al. (1997)
184	1995 Hyogoken-Nanbu	Kobe No 26	No	3.5	0.9	65	40	0.60	232	0.99	0.63	6.90	1.21	1.37	0.38	0.0	24.45	1.00	0.87	1.00	1.22	1.60	41.2	42.0	Tokimatsu (2000), Tokimatsu et al. (1997)
185	1995 Hyogoken-Nanbu	Kobe No 27	No	2.5	1.1	45	31	0.60	259	1.00	0.57	6.90	1.21	1.49	0.31	10.0	27.70	1.00	0.82	1.00	1.22	1.82	50.4	52.1	Tokimatsu (2000), Tokimatsu et al. (1997)
186	1995 Hyogoken-Nanbu	Kobe No 28	Yes	4.0	1.8	74	52	0.40	192	0.96	0.36	6.90	1.21	1.25	0.23	8.0	14.30	1.00	0.89	1.00	1.22	1.39	21.5	22.5	Tokimatsu (2000), Tokimatsu et al. (1997)
187	1995 Hyogoken-Nanbu	Kobe No 29	Yes	3.8	2.0	69	52	0.40	207	0.98	0.34	6.90	1.21	1.25	0.22	0.0	12.05	1.00	0.88	1.00	1.22	1.40	18.0	18.6	Tokimatsu (2000), Tokimatsu et al. (1997)
188	1995 Hyogoken-Nanbu	Kobe No 30	No	8.5	1.5	161	92	0.60	213	0.90	0.61	6.90	1.21	1.03	0.49	10.0	30.60	1.00	1.00	1.00	1.22	1.05	39.0	40.5	Tokimatsu (2000), Tokimatsu et al. (1997)
189	1995 Hyogoken-Nanbu	Kobe No 31	No	4.0	1.2	74	46	0.60	232	0.99	0.61	6.90	1.21	1.30	0.39	0.0	35.75	1.00	0.89	1.00	1.22	1.48	57.0	57.9	Tokimatsu (2000), Tokimatsu et al. (1997)
190	1995 Hyogoken-Nanbu	Kobe No 32	No	3.5	1.4	63	43	0.50	213	0.98	0.47	6.90	1.21	1.34	0.29	6.3	18.75	1.00	0.87	1.00	1.22	1.54	30.5	31.4	Tokimatsu (2000), Tokimatsu et al. (1997)
191	1995 Hyogoken-Nanbu	Kobe No 33	No	8.0	2.0	149	90	0.50	207	0.90	0.48	6.90	1.21	1.04	0.38	50.0	21.60	1.00	1.00	1.00	1.22	1.06	27.8	32.6	Tokimatsu (2000), Tokimatsu et al. (1997)
192	1995 Hyogoken-Nanbu	Kobe No 34	Yes	7.0	1.8	132	81	0.40	189	0.88	0.37	6.90	1.21	1.08	0.29	9.3	17.65	1.00	0.98	1.00	1.22	1.12	23.5	24.7	Tokimatsu (2000), Tokimatsu et al. (1997)
193	1995 Hyogoken-Nanbu	Kobe No 35	Yes	4.5	2.1	84	60	0.50	189	0.95	0.43	6.90	1.21	1.20	0.30	6.3	12.30	1.00	0.90	1.00	1.22	1.30	17.7	18.4	Tokimatsu (2000), Tokimatsu et al. (1997))
194	1995 Hyogoken-Nanbu	Kobe No 36	No	3.5	0.9	65	40	0.60	198	0.97	0.62	6.90	1.21	1.37	0.37	2.5	20.05	1.00	0.87	1.00	1.22	1.59	33.7	34.4	Tokimatsu (2000), Tokimatsu et al. (1997)
195	1995 Hyogoken-Nanbu	Kobe No 37	Yes	5.0	4.0	95	85	0.35	177	0.91	0.23	6.90	1.21	1.06	0.18	0.0	18.40	1.00	0.92	1.00	1.22	1.09	22.5	23.1	Tokimatsu (2000), Tokimatsu et al. (1997))
196	1995 Hyogoken-Nanbu	Kobe No 38	Yes	8.0	3.0	152	103	0.50	192	0.85	0.41	6.90	1.21	0.99	0.34	5.0	15.20	1.00	1.00	1.00	1.22	0.99	18.3	18.9	Tokimatsu (2000), Tokimatsu et al. (1997)
197	1995 Hyogoken-Nanbu	Kobe No 39	No	4.5	2.6	89	71	0.60	290	1.00	0.49	6.90	1.21	1.13	0.36	0.0	47.00	1.00	0.90	1.00	1.22	1.20	61.9	62.9	Tokimatsu (2000), Tokimatsu et al. (1997)
198	1995 Hyogoken-Nanbu	Kobe No 40	No	3.5	2.8	67	60	0.60	250	1.00	0.43	6.90	1.21	1.19	0.30	0.0	31.10	1.00	0.87	1.00	1.22	1.30	42.8	43.6	Tokimatsu (2000), Tokimatsu et al. (1997)
199	1995 Hyogoken-Nanbu	Kobe No 41	Yes	4.1	2.0	71	51	0.40	189	0.96	0.35	6.90	1.21	1.26	0.23	0.0	9.70	1.00	0.89	1.00	1.22	1.41	14.9	15.4	Tokimatsu (2000), Tokimatsu et al. (1997)
200	1995 Hyogoken-Nanbu	Kobe No 42	Yes	5.0	1.2	92	55	0.40	137	0.78	0.34	6.90	1.21	1.23	0.23	10.0	7.00	1.00	0.92	1.00	1.22	1.36	10.7	11.7	Tokimatsu (2000), Tokimatsu et al. (1997))
201	1995 Hyogoken-Nanbu	Kobe No 43	Yes	4.7	2.2	82	57	0.35	198	0.96	0.31	6.90	1.21	1.21	0.21	20.0	10.00	1.00	0.91	1.00	1.22	1.34	14.8	17.1	Tokimatsu (2000), Tokimatsu et al. (1997)
202	1995 Hyogoken-Nanbu	Kobe No 44	Yes	4.0	1.6	69	45	0.40	158	0.90	0.36	6.90	1.21	1.31	0.23	5.0	4.70	1.00	0.89	1.00	1.22	1.50	7.6	8.1	Tokimatsu (2000), Tokimatsu et al. (1997)

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 (Cont'd.) Summary of Updated Cetin et al. (2016) Field Performance Case History Parameters

Case #	Earthquake	Site	Liq.?	$d_{crt.}$ (m)	GWT (m)	σ_v (kPa)	σ_v' (kPa)	a_{max} (g)	$V^*_{s,12m}$ (m/s)	r_d	$CSR_{\sigma'v,Mw}$	M_w	K_{Mw}	K_σ	$CSR_{\sigma'v=1atm, Mw=7.5}$	FC %	N (mean)	C_S	C_R	C_B	C_E	C_N	$(N_1)_{60}$	$N_{1,60,CS}$	Reference
203	1995 Hyogoken-Nanbu	Port Island Borehole Array Station	Yes	7.6	2.4	144	93	0.34	152	0.72	0.25	6.90	1.21	1.03	0.20	20.0	5.65	1.00	0.99	1.00	1.22	1.05	7.1	9.1	Shibata et al. (1996)
204	1995 Hyogoken-Nanbu	Port Island Improved Site (Ikegaya)	No	8.5	5.0	155	121	0.40	201	0.87	0.29	6.90	1.21	0.94	0.25	20.0	20.25	1.00	1.00	1.00	1.22	0.92	22.6	25.1	Yasuda et al. (1996)
205	1995 Hyogoken-Nanbu	Port Island Improved Site (Tanahashi)	No	10.0	5.0	185	136	0.40	201	0.82	0.29	6.90	1.21	0.91	0.26	20.0	18.20	1.00	1.00	1.00	1.22	0.86	19.1	21.6	Yasuda et al. (1996)
206	1995 Hyogoken-Nanbu	Port Island Improved Site (Watanabe)	No	9.5	5.0	178	134	0.40	250	0.96	0.33	6.90	1.21	0.91	0.30	20.0	31.00	1.00	1.00	1.00	1.22	0.87	32.8	35.6	Yasuda et al. (1996)
207	1995 Hyogoken-Nanbu	Port Island Site I	Yes	10.0	3.0	184	115	0.34	189	0.78	0.27	6.90	1.21	0.96	0.24	20.0	9.70	1.00	1.00	1.00	1.22	0.94	11.1	13.2	Tokimatsu et al. (1996)
208	1995 Hyogoken-Nanbu	Rokko Island Building D	Yes	7.5	4.0	138	104	0.40	213	0.93	0.32	6.90	1.21	0.99	0.27	25.0	14.80	1.00	0.99	1.00	1.22	0.99	17.6	20.6	Tokimatsu et al. (1996)
209	1995 Hyogoken-Nanbu	Rokko Island Site G	Yes	11.5	4.0	210	137	0.34	189	0.72	0.25	6.90	1.21	0.90	0.22	20.0	11.95	1.00	1.00	1.00	1.22	0.86	12.5	14.7	Tokimatsu et al. (1996)
210	1995 Hyogoken-Nanbu	Torishima Dike	Yes	4.8	0.0	86	39	0.25	137	0.81	0.29	6.90	1.21	1.38	0.17	20.0	8.50	1.00	0.91	1.00	1.22	1.61	15.2	17.5	Matsuo (1996)

***CSR and r_d values are estimated by site response analyses; hence a $V_{s,12m}$ value is not estimated and reported.**

Table 2-7 presents case by case summaries of the back-analyzed input parameters for each case history employed in this report. A more complete presentation of the case history database background documentation, and of specific engineering judgments and other important case history back-analysis and parametric evaluation details, are available in Appendix A.

Figure 2-1 presents a visual comparison between the back-analyzed field performance case history input parameters of the Cetin et al. (2004) and Cetin et al. (2016) databases. In this figure, the black and red symbols represent the values from Cetin et al. (2004) and Cetin et al. (2016) databases, respectively. Fines content values listed are capped in between 5 % and 35 %, consistent with the Cetin et al. (2004, 2016) case history probabilistic assessment methodology employed in subsequent development of triggering correlations.

Table 2-8 presents a comparative summary of key database statistics. While calculating these composite statistics for all four of these databases (1) liquefied, (2) nonliquefied, and (3) Hyogoken-Nanbu (Kobe) earthquake case histories were each weighted as will be discussed later in this report. So the cross-comparisons here are based on identical weighting within each database. As a result, the parameter ensemble averages shown in Table 2-8 are not biased by different weighting schemes, and direct cross-comparisons are optimally meaningful here.

Table 2-8. Summary and Comparison of Overall Case History Weighted Average Input Parameters

Parameter	Seed et al. (1984) 125 case history		Cetin et al. (2004) 200 case history		Cetin et al. (2016) 210 case history		Idriss and Boulanger (2010) 230 case history	
	Mean	Mean Std. Dev.	Mean	Mean Std. Dev.	Mean	Mean Std. Dev.	Mean	Mean Std. Dev.
γ_{above_GWT} (kN/m^3)	-	-	15.2	0.67	16.77	0.47	-	-
$\gamma_{below\text{-}GWT}$ (kN/m^3)	-	-	16.91	0.69	18.91	0.47	-	-
Critical Depth: d_{cr} (m)	5.80	-	5.08	0.53	4.93	0.55	5.02	-
$N_{1,60}$	13.51	-	15.83	3.15	15.13	3.09	15.02	-
FC (%)	14.72	-	18.89	3.02	16.57	4.16	16.24	-
$\Delta N_{1,60}$	-	-	1.61	*	1.52	*	1.94	-
$N_{1,60,CS}$	-	-	17.44	*	16.65	*	16.96	-
a_{max} (g)	0.22	-	0.25	0.04	0.24	0.05	0.25	-
σ_v (kPa)	105.58	-	83.87	9.52	89.70	10.60	91.78	-
σ'_v (kPa)	67.45	-	53.48	5.83	60.62	5.62	61.29	-
$V_{s,12m}$ (m/s)	-	-	178.92	-	190.87	-	-	-
r_d	0.953	-	0.859	0.058	0.911	0.057	0.949	-
CSR_{σ_v',M_w}	0.210	-	0.211	0.04	0.208	0.05	0.225	-
M_w	7.12	-	7.06	-	7.09	-	7.13	-
K_{Mw}	-	-	1.17	-	1.17	-	1.12	-
K_σ	-	-	1.23	-	1.25	-	1.06	-
$CSR_{\sigma_v'=1atm.,M_w=7.5}$	0.197	-	0.158	-	0.156	-	0.196	-

* Functions of regressed likelihood model coefficients, as presented later in this report and (2016-a)

It can be seen from Table 2-7, Table 2-8, and Figure 2-1, that the average changes in the input parameters from the Cetin et al. (2004) database to the updated Cetin et al. (2016) database are generally minor and usually non-systematic, except for (1) r_d which increased on average by about 6.1% due to elimination of the typographical error in the original (2004) spreadsheet execution of the r_d formulation, and (2) total vertical stress and (3) effective vertical stress terms, which increased on average by 6.9 % and 13.3%, respectively, due to the modifications of unit weights.

Direct comparisons between the unit weights above and below the phreatic surface cannot be achieved, because the details of the processing and back-analyses of a significant number of the case

histories in the Idriss and Boulanger (2010) database are not adequately transparently presented. So the best cross-comparison available is between the overall averages of σ_v and of σ'_v of the datasets of Cetin et al. (2016) and Idriss and Boulanger (2010). This is imperfect, as the two databases do not have exactly the same full sets of case histories, but overlap between the two sets of case history suites is nearly perfect. The Cetin et al. (2016) database has only 5 case histories which are not included in the Idriss and Boulanger (2010) database. 205 of the same cases occur in each database, and this is as good an approximation as can be achieved without more transparent presentation of the Idriss and Boulanger database case histories. These ratios are:

$\frac{\sigma_{v,IB_2010}}{\sigma_{v,CEA_2016}} = \frac{91.78}{89.70} = 1.023$ and $\frac{\sigma'_{v,IB_2010}}{\sigma'_{v,CEA_2016}} = \frac{61.29}{60.62} = 1.011$ which suggest that the unit weights employed are, on average, compatible between the two databases within approximately ± 2% or better.

It should be noted that the slight overall average increase in r_d values in the new database of Cetin et al. (2016) relative to Cetin et al. (2004) is largely offset by the effects of the increases in unit weights for purposes of calculation of cyclic stress ratios (CSR), as discussed previously, because CSR is a composite parameter as given later in Equation 2-5.

Increasing both the total and effective unit weights tends to reduce the ratio of $\frac{\sigma_v}{\sigma'_v}$ (in Equation 2-5) for cases with critical strata at shallow to moderate depths (which comprise the entirety of both databases), and this in turn reduces CSR. This reduction in CSR due to slight increases in soil unit weights partially offsets the direct increases in CSR due to increases in r_d resulting from correction of the earlier spreadsheet typographical error for r_d in 150 of the cases. Accordingly, these two sets of modifications largely offset each other, and as shown in Table 2-8 the ratio of weighted average (mean) cyclic stress ratio ($CSR_{\sigma'_v,M_w}$) from the Cetin et al. (2004) database to the current Cetin et al. (2016) database is only $\frac{CSR_{\sigma'_v,M_w,2004}}{CSR_{\sigma'_v,M_w,2016}} = \frac{0.211}{0.208}$; a change of approximately 1.4%. Similarly, in the final plot of Figure 2-1(j), it can be seen that the updated $CSR_{\sigma'_v,M_w}$ values actually decreased slightly in nearly as many cases as those for which slight increases in $CSR_{\sigma'_v,M_w}$ occurred.

2.2.2. Conventions Used in Back-Analyzing Liquefaction Field Performance Case Histories

The processing and analysis of the field case histories is one of the most difficult, sensitive and time-consuming elements of the studies involved in developing these types of databases, and the resulting triggering correlations based upon them. It is also of critical importance, and often requires considerable judgment. Differences between the correlations proposed by different teams of researchers are often due, in no small part, to differences in the interpretations and analyses of the contributing individual field case histories. This section presents a concise summary of some of the key elements of the conventions and procedures employed in evaluation of the field performance case histories for these current studies; which are essentially the same as those employed in Cetin et al. (2000, 2004). Additional descriptions and details are provided by Cetin (2000), and some of these are discussed in Chapter 3. More complete summaries of the details of the processing of the liquefaction triggering field case histories employed in these current studies, and citations of source references for these, are presented in Appnedix A.

2.2.2.1. Case History Screening

All case histories used in these studies are free-field and level ground cases. The intent is to select and back-analyze only field case histories with no initial (static) driving shear stresses acting on horizontal planes. Cases with ground slopes of more than about 3%, or cases with a free face (e.g. trench, stream cut, shoreline etc.) in close enough proximity as to produce a non-zero initial shear stress on horizontal planes in the soil, were eliminated. Cases in which soil/structure interaction might have significantly influenced cyclic shear stresses, or resulting liquefaction performance, were also eliminated. Any individual case history (and site) can produce only one performance outcome; sites either liquefied, or did not liquefy. Exceptions were made for two case histories, in which the performance was characterized as “marginal”. There was only one single “most critical stratum” at any borehole, and it was not allowed that both “liquefied” and “non-liquefied” data were drawn from the same borehole (or the same site, if multiple borings characterize the same soil unit), as the onset of liquefaction in one stratum reduces cyclic shear stresses in both overlying and underlying strata, and this is not amenable to sufficiently reliable back-analyses. Accordingly, it is not possible to derive unbiased “non-liquefied” data from other strata at a site where one or more strata have liquefied. In addition, compiling multiple case histories from a single borehole violates the statistical independency assumption of the maximum likelihood formulation.

2.2.2.2. Identification of the Critical Layer and Depth

At each case history site, the critical stratum was identified as the non-plastic, submerged soil stratum most susceptible to triggering of liquefaction. When possible, collected surface boil ejecta materials were also considered, but problems associated with both mixing and segregation during transport and ejection, and recognition that liquefaction of underlying strata can result in transport of overlying soils to the surface through boils, limited the usefulness of some of these data. Experienced post-earthquake field investigation teams are of great value here, as the judgments regarding which strata liquefied can be very challenging. Similarly, for sites where the most potentially critical layer has a relatively high penetration resistance ($N_{1,60,CS} \geq 25$ or so), determination in the field as to whether liquefaction was triggered, or not, can also be challenging. After having identified the critical layer, the mean depth of the critical layer was then taken as the representative critical depth. Soil properties, and penetration resistances, however, were assessed based on data from within the full thickness of the critical layer, when available.

2.2.2.3. Estimation of the CSR

In-situ cyclic stress ratio ($CSR_{\sigma'_v,M_w}$) is taken as the "equivalent uniform CSR" equal to 65% of the single peak cyclic stress ratio (CSR_{peak,σ'_v,M_w}) as shown in Equation 2-3.

$$CSR_{\sigma'_v,M_w} = 0.65 \cdot CSR_{peak,\sigma'_v,M_w} \quad \textbf{Eqn. (2-3)}$$

Peak ground surface acceleration (a_{max}) values for each case history were estimated using all available information, including source mechanism and geometry, local and regional recorded strong motion data, and suites of available ground motion prediction equations. Values of a_{max}, were taken as the geometric mean of the two orthogonal horizontal components of motion. Source mechanism, near-field effects, and local site response effects were all accounted for as fully as possible. In most cases, multiple applicable ground motion prediction equations were locally calibrated for event-specific and azimuth-specific variations using nearby strong ground motion records, and the results were then considered jointly to develop best estimates of a_{max}. Adjustments for site effects were made either based on judgment and experience, or on event-specific data, or were based on performance of full site-specific seismic response analyses. Peak ground surface acceleration (a_{max}) was considered to be log-normally distributed. Uncertainty (or variance) in a_{max} was evaluated for each case, and is directly reflective of the level and quality of data and information available for each case history.

The values of a_{max} were targeted as being representative of "geometric mean" peak horizontal ground surface accelerations, and adjustments should therefore be made for other forms of characterization of accelerations for forward analyses of engineering projects.

In-situ CSR_{σ_v',M_w} was evaluated based on performance of site-specific and event-specific seismic site response analyses (using SHAKE 91; Idriss and Sun, 1992), for 48 of the 210 liquefaction field performance case histories, where (1) sufficient sub-surface data was available, and (2) where suitable "input" motions could be developed from nearby strong ground motion records. For these cases where direct, site-specific and event-specific site response analyses were performed, the equivalent uniform cyclic stress ratio at any depth was calculated directly from the peak cyclic shear stress acting on a horizontal plane ($\tau_{\text{hv,cyclic,peak}}$) from the seismic response analyses as given in Equation 2-4.

$$CSR_{\sigma_v',M_w} = 0.65 \cdot \frac{\tau_{\text{hv, cyclic, peak}}}{\sigma_v'} \qquad \textbf{Eqn. (2-4)}$$

For the remaining 162 cases, wherein full seismic site response analyses were not performed, CSR_{σ_v',M_w} was evaluated using the estimated a_{max} and Equation 2-5, with r_d-values estimated by using the probabilistic, site-dependent and motion-dependent predictive r_d relationship developed by Cetin et al. (2000, 2004).

$$CSR_{\sigma_v',\alpha,M_w} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \qquad \textbf{Eqn. (2-5)}$$

In Equation 2-5, as defined earlier, a_{max} is the peak horizontal ground surface acceleration; g is acceleration of gravity, σ_v and σ'_v are total and effective vertical stresses, respectively, and r_d is the stress reduction or nonlinear shear mass participation factor as estimated by Equation 2-6.

d < 20 m (~65 ft); 5.5 < M_w < 8.2; 0.05 g < a_{max} < 0.55 g; 90 m/s < $V_{s,12\,m}$ < 230 m/s:

$$r_d(d,M_w,a_{\max},V_{s,12m}^*) = \frac{\left[1+\dfrac{-23.013-2.949\cdot a_{\max}+0.999\cdot M_w+0.0525\cdot V_{s,12m}^*}{16.258+0.201\cdot e^{0.341\cdot(-d+0.0785\cdot V_{s,12m}^*+7.586)}}\right]}{\left[1+\dfrac{-23.013-2.949\cdot a_{\max}+0.999\cdot M_w+0.0525\cdot V_{s,12m}^*}{16.258+0.201\cdot e^{0.341\cdot(0.0785\cdot V_{s,12m}^*+7.586)}}\right]} \pm \sigma_\varepsilon r_d \qquad \textbf{Eqn. (2-6)}$$

d ≥ 20 m (~65 ft) ; 5.5 < M_w < 8.2; 0.05 g < a_{max} < 0.55 g; 90 m/s < $V_{s,12\,m}$ < 230 m/s:

$$r_d(d, M_w, a_{\max}, V^*_{s,12m}) = \frac{\left[1 + \frac{-23.013 - 2.949 \cdot a_{\max} + 0.999 \cdot M_w + 0.0525 \cdot V^*_{s,12m}}{16.258 + 0.201 \cdot e^{0.341 \cdot (-20 + 0.0785 \cdot V^*_{s,12m} + 7.586)}}\right]}{\left[1 + \frac{-23.013 - 2.949 \cdot a_{\max} + 0.999 \cdot M_w + 0.0525 \cdot V^*_{s,12m}}{16.258 + 0.201 \cdot e^{0.341 \cdot (0.0785 \cdot V^*_{s,12m} + 7.586)}}\right]} - 0.0046 \cdot (d - 20) \pm \sigma_{\varepsilon r_d}$$

d < 12 m (~40 ft):

$$\sigma_{\varepsilon r_d}(d) = d^{0.850} \cdot 0.0198$$

d ≥ 12 m (~40 ft):

$$\sigma_{\varepsilon r_d}(d) = 12^{0.850} \cdot 0.0198$$

In Equation 2-6, "d" is in meters, a_{max} is in gravitational acceleration (g's), $V^*_{s,12\ m}$ is the average shear wave velocity over the top 12 meters in m/sec, and "e" is the exponential symbol.

This is a critical difference between these current studies and the correlations developed by (1) Seed et al. (1985) and (2) Boulanger and Idriss (2012). A full explanation of the development of the probabilistic r_d relationship is presented in Cetin and Seed (2000, 2004) and in Appendix C. A large suite of 50 actual soil profiles from among the 200 actual case history sites in the liquefaction field performance case history database of Cetin et al. (2004) were analyzed using SHAKE91 (Idriss and Sun, 1992) to implement the equivalent linear method to approximate nonlinear seismic site response. These 50 soil profiles were carefully selected to suitably represent the ranges of site conditions present within the larger suite of case history sites. A suite of 42 strong motion time histories, selected to provide input motions for magnitudes ranging from M_w = 5.7 to 8.1, were each applied as input motions to each of the 50 representative soil profiles, and the time histories for these input motions were carefully selected so that near-field, mid-field, and far-field motions were suitably represented within each range of magnitudes. This produced 50 x 42 = 2,100 seismic site response analyses for a broad range of site conditions for actual sites in the field case history database. In addition, 53 case histories had been subjected to site-specific and event-specific site response analyses, producing 53 additional sets of site response analysis results. In the end 2,153 one-dimensional seismic site response analyses were performed.

Probabilistic regressions were then performed to investigate the principal factors that affected the relationship between peak horizontal cyclic shear stress ($\tau_{hv,cyclic}$) at any depth and the peak

horizontal ground surface acceleration (a_{max}), and then to develop the predictive relationship of Equation 2-6 in order to develop a predictive relationship for r_d that (1) would be specifically appropriate to the large range of site conditions in the field performance case history database, and (2) would adaptively account for variations in site conditions, and in input motions, to provide more reliable r_d values than had previously been available in other "simplified" r_d relationships. This will be discussed further in Chapter 3.

The r_d relationship developed by Cetin et al. (2004) differs from previous efforts not only because the large suites of site conditions were specifically representative of actual site conditions within the liquefaction performance case history database. They also differ because of the very large number of site response analyses performed for these sites, and the deliberately chosen broad range of "input" motions employed. Figure 2-3 shows the r_d curves for all of the 2,153 site response analyses performed.

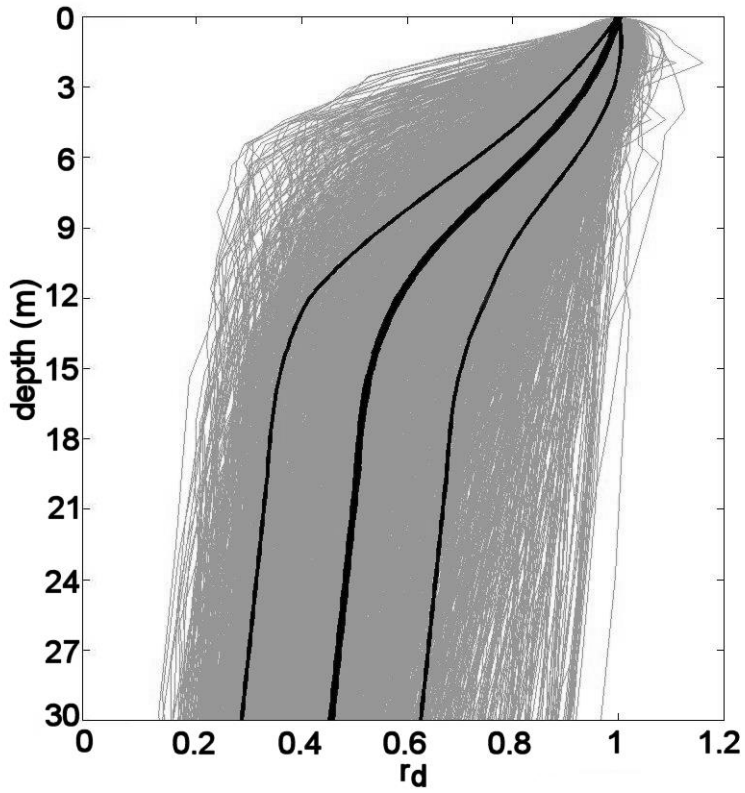
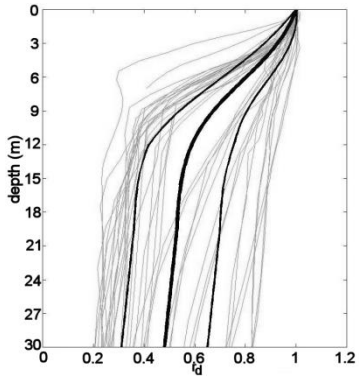


Fig. 2-3. Plots of r_d for all 2,153 site response analyses for all combinations of sites and input motions superimposed with the predictions based on group mean values of V_s, M_w, and a_{max} (Heavy black lines show regressed mean and $\pm$ 1 standard deviations)

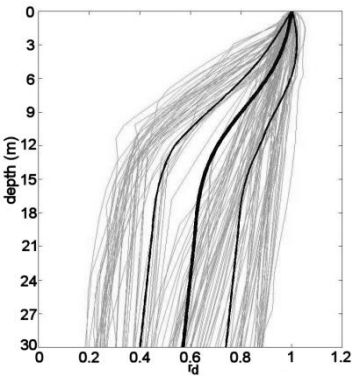
Probabilistic regressions were performed on the very large body of resulting site response analyses, and it was determined that the principal factors affecting the variation of r_d vs. depth were

(1) intensity of shaking, (2) stiffness of the soil column, (3) variability of stiffness ("layering") within the soil column and (4) magnitude of the causative event. Intensity of shaking interacts with soil column stiffness; the harder a column is shaken, and the softer the column is, the greater the degree of nonlinearity of response that results (and r_d values decrease more rapidly with increasing depth). Increased heterogeneity or distinct "layering" of the site conditions tends to break up coherence, and also tends to cause r_d values to decrease more rapidly with increasing depth. Increased magnitude serves as a proxy for duration, and is the least significant of the four factors. Regressions were performed on the results from the 2,153 site response analyses to develop the relationship presented in Equation 2-6, wherein probabilistic estimates of r_d can be made as a function of (1) depth, (2) a_{max}, (3) $V_{S,12m}$ and (4) M_w. Figure 2-4 shows how this regressed relationship compares with regard to prediction of the results of the large suite of site response analyses performed. In this figure, the 2,153 seismic site response analysis results are "binned" into 12 bins. In each figure, the heavy solid (center) line represents the median (or 50th percentile) predicted value of r_d from Equation 2-6, and the other two lines show the plus and minus one standard deviation predictions, also based on Equation 2-6, and based on the mean values of a_{max}, M_w, and $V^*_{s,12m}$ for each bin.

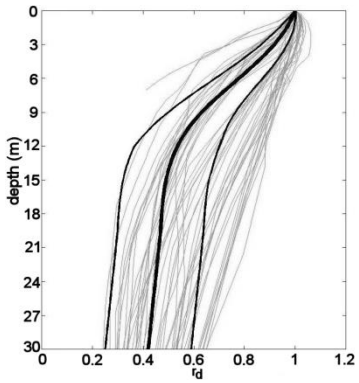
As shown in Figure 2-4, predictive "fit" is good over a large range of site and ground shaking conditions. It should be noted that Figure 2-4 sub-divides the large suite of back-analyses into 12 "bins", but that the predictive relationship of Equation 2-6 is not "binned", and instead transitions continuously and smoothly across the ranges of a_{max}, $V_{S,12m}$ and M_w analyzed.



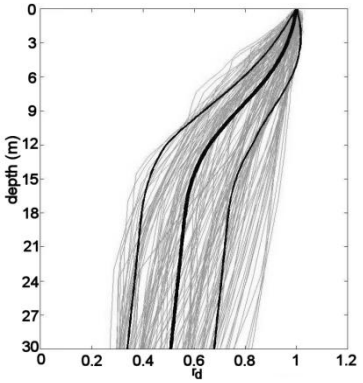
(a) $M_w \geq 6.8$, $a_{max} \leq 0.12g$, $V^*_{s,12m} \leq 160$ m/s



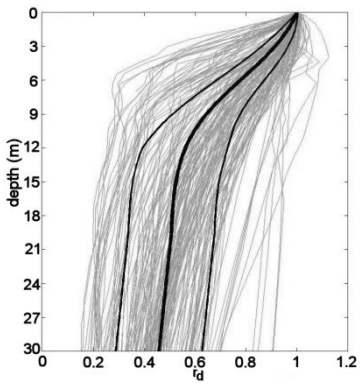
(b) $M_w \geq 6.8$, $a_{max} \leq 0.12g$, $V^*_{s,12m} > 160$ m/s



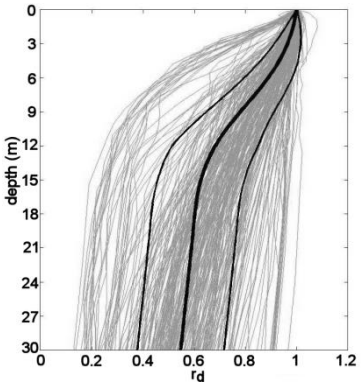
(c) $M_w < 6.8$, $a_{max} \leq 0.12g$, $V^*_{s,12m} \leq 160$ m/s



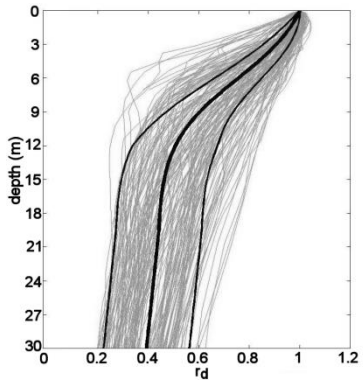
(d) $M_w < 6.8$, $a_{max} \leq 0.12g$, $V^*_{s,12m} > 160$ m/s



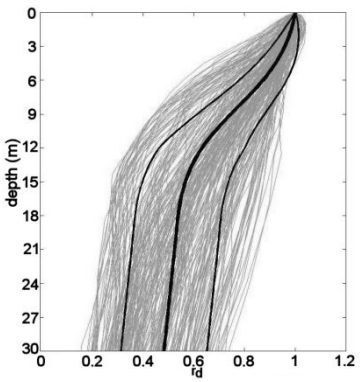
(e) $M_w \geq 6.8$, $0.12g < a_{max} \leq 0.23g$, $V^*_{s,12m} \leq 160$ m/s



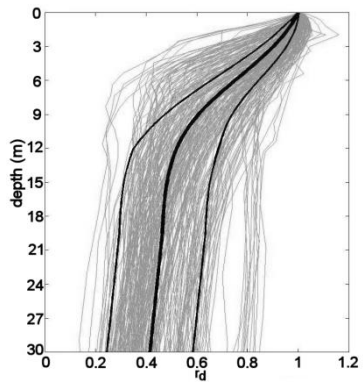
(f) $M_w \geq 6.8$, $0.12g < a_{max} \leq 0.23g$, $V^*_{s,12m} > 160$ m/s



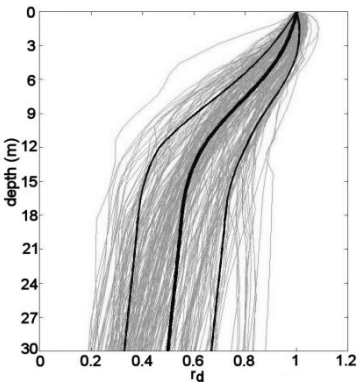
(g) M_w<6.8, 0.12g< a_{max} ≤0.23g, $V^*_{s,12m}$ ≤160 m/s



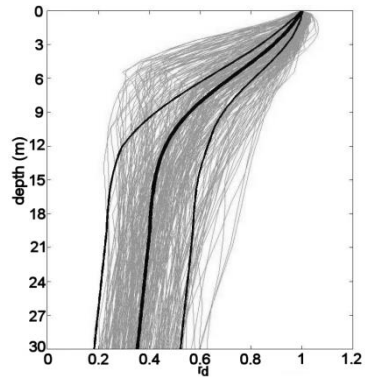
(h) M_w<6.8, 0.12g< a_{max} ≤0.23g, $V^*_{s,12m}$ >160 m/s



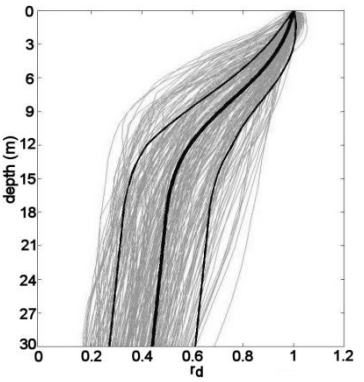
(i) M_w≥6.8, 0.23g< a_{max}, $V^*_{s,12m}$ ≤160 m/s



(j) M_w≥6.8, 0.23g< a_{max}, $V^*_{s,12m}$ >160 m/s



(k) M_w<6.8, 0.23g< a_{max}, $V^*_{s,12m}$ ≤160 m/s



(l) M_w<6.8, 0.23g< a_{max}, $V^*_{s,12m}$ >160 m/s

Fig. 2-4. r_d plots for 12 "bins" (based on selected sub-ranges of a_{max}, V_s and M_w) superimposed with the predictions based on each bin's mean values of V_s, M_w, and a_{max} (with + 1 standard deviation also shown).

It should also be noted that the predictive relationship of Equation 2-6 is potentially limited in applicability for some sets of conditions due to the limitations of even the very broad ranges of (50) actual site conditions analyzed for its development. The relationship of Equation 2-6 appears to produce "overly stiff" (nearly rigid body) r_d-type response predictions for very stiff sites. Accordingly, it is recommended that an upper bound of $V_{s,12m} \leq 230$ m/s be employed in forward analyses. Sites with higher values of $V_{s,12m}$ are "very stiff", and tend not to be of concern with regard to potential liquefaction. As an alternative to use of Equation 2-6 with this upper limit, site-specific seismic site response analyses could also be performed. Similarly, for unusually soft sites (e.g. sites with $V_{s,12m} \leq 90$ m/s) Equation 2-6 can be employed, but these are often challenging conditions that may warrant site-specific seismic response analyses, and thus direct response analyses are recommended here.

The correct way to estimate $V_{s,12m}$ is to calculate apparent travel time through each sub-layer, down to a depth of 12 meters, and then to divide the total travel time by the distance travelled. This can be done as given in Equation 2-7.

$$V_{s,12m} = \frac{12m}{\sum_{i=1,\ldots n}^{n} \frac{H_i}{V_{s,i}}}, \quad \text{n=\# of soil layers} \qquad \textbf{Eqn. (2-7)}$$

In Equation 2-7, H_i is the thickness of each sublayer down to 12 meters depth, and $V_{s,i}$ is the shear wave velocity of each of these sublayers. The un-scaled (non-normalized) value of $CSR_{\sigma'_v,\alpha,M_w}$ was evaluated for each case history, based on either (1) direct, site-specific and event-specific seismic site response analyses, or (2) based on the "simplified" approach of Equation 2-5 employing the probabilistic r_d estimates produced by Equation 2-6 (Cetin et al., 2000, 2004).

Finally, it should be noted that these approaches were employed to develop best-estimates of r_d, and of variance in r_d, for purposes of development of eventual liquefaction triggering relationships. These approaches were developed to be as unbiased as possible, relative to the actual field case history database employed, making them unique in this regard among the three sets of liquefaction triggering relationships considered in this report. As a result, the resulting liquefaction triggering relationships developed do not need to be employed in forward engineering analyses only in conjunction with Equation 2-6 (although that can be a suitable approach); they can also be employed in conjunction with project-specific seismic site response analyses to directly evaluate CSR's, and with project-specific seismic site response and soil-structure-interaction analyses as well.

2.2.2.4. Estimation of Variance in the CSR

Factors contributing to overall variance in estimation of the equivalent uniform cyclic shear stress ratio (CSR_{σ_v',M_w}) were summed within a structural reliability framework. The main contributors to this variance were (1) uncertainty in a_{max}, and (2) uncertainty in nonlinear shear mass participation factor (or r_d). Uncertainty in a_{max} was largest when instrumental strong motion recordings in the vicinity (and region) of the case history site were sparse or unavailable, so that available ground motion attenuation relationships could not be locally well-calibrated on an event-specific and azimuth-specific basis. Additional variables, which also contributed (though usually to lesser degree) to overall variance in estimates of CSR_{σ_v',M_w} were (3) the vertical limits (top and base) of the critical soil stratum, (4) uncertainty in soil unit weights, and (5) uncertainty regarding the location of the phreatic surface (or "water table depth") at the time of the earthquake.

2.2.2.5. Estimation of Representative SPT-N Values

A limited number of case histories had to be characterized based on only a single SPT boring, while at other sites multiple SPT borings, each with multiple SPT, are available to characterize a single critical soil stratum. These studies did not assign a "case history" to each valid SPT boring. Instead, if available, multiple borings at the same site are generally grouped together to jointly define a stratum, and they are then considered en masse as a single case history. Cases with more SPT data for the critical stratum generally have resulting lower uncertainty or variance in SPT N-values. As a result, when probabilistic regressions are subsequently performed to develop triggering relationships, cases in which more extensive characterization data are available for a critical stratum intrinsically carry somewhat more weight (exert more influence) than less well-characterized cases with more limited in situ data (and higher uncertainty or variance).

The $N_{1,60}$-values employed were "truncated mean values" within the critical stratum, and these were developed as follows: Measured N-values (from one or more locations) within a critical stratum were corrected for overburden, energy, equipment, and procedural effects to develop $N_{1,60}$ values. These were then plotted vs. elevation. In many cases, a given soil stratum would be found to contain an identifiable sub-stratum based on a group of localized low $N_{1,60}$-values that was significantly more critical than the rest of the stratum. In such cases, the sub-stratum was usually taken as the "critical stratum". Occasional high values, not apparently representative of the general characteristics of the critical stratum, were considered "non-representative", and were eliminated in a number of the cases. Spurious high values can be the result of gravel interference, or other non-representative issues. Engineering judgment is required to define "spurious". Similarly, though less often, very low $N_{1,60}$

values (very much lower than the apparent main body of the stratum, and often associated with locally high fines content) were sometimes eliminated if the soils had significant fines and if it appeared likely that the fines might have been somewhat plastic (clayey) in nature. The remaining, corrected $N_{1,60}$ values were then used to evaluate both the mean of $N_{1,60}$ within the critical stratum, and the variance in $N_{1,60}$. If consistent (if they appear to represent the same sub-stratum), N values measured slightly above ground water table are also used to characterize the density state of the critical layer. However, the critical depth range is correctly defined starting from the ground water table depth.

2.2.2.6. Execution of SPT Corrections

All N-values were corrected by multiplying with C_N to account for overburden effects (to the hypothetical value, N_1 that "would" have been measured if the effective overburden stress (σ_v') at the depth of the SPT had been 1 atm.) based on Equation 2-8 (after Liao and Whitman, 1986).

$N_1 = N \cdot C_N$ where $C_N = \left(\frac{\sigma_v'}{P_a}\right)^{0.5} \leq 2.0$ **Eqn. (2-8)**

A maximum (limiting) value of $C_N \leq 2.0$ was employed, which only affected 5 of the 210 field performance case history sites (i.e.: the Meiko, River Park A sites, and WoodMarine UC-B4). The resulting N_1 values were then further corrected for energy, equipment, and procedural effects to fully standardized $N_{1,60}$ values as

$N_{1,60} = N_1 \cdot C_R \cdot C_S \cdot C_B \cdot C_E$ **Eqn. (2-9)**

where C_R, C_S, C_B, and C_E are corrections for "short" rod length, non-standardized sampler configuration, borehole diameter, and hammer energy efficiency. The corrections for C_R, C_S, C_B and C_E employed correspond largely to those recommended by the NCEER Working Group (NCEER 1997, and Youd et al., 2001), and these are summarized in Table 2-9.

Table 2-9. Recommended corrections for SPT equipment, energy and procedures

<table>
<tr><td>C_R</td><td>$C_R = 0.48 + 0.225 \cdot \ln(d)$; $d \le 10m$; (Eq. T-1)
$C_R = 1$; $10m < d < 30m$

where d = rod length (or “stick-up) from the top of the SPT sampler to the striking point at the top of the rod.</td></tr>
<tr><td>C_S</td><td>For samplers with an indented space for interior liners, but with liners omitted during sampling,
$C_S = 1 + \frac{N_{1,60}}{100}$ (Eq. T-2)
With limits of $1.10 \le C_S \le 1.30$</td></tr>
<tr><td>C_B</td><td><table><tr><th>Borehole diameter</th><th>Correction (C_B)</th></tr><tr><td>65 to 115 mm</td><td>1.00</td></tr><tr><td>150 mm</td><td>1.05</td></tr><tr><td>200 mm</td><td>1.15</td></tr></table></td></tr>
<tr><td>C_E</td><td>$C_E = \frac{ER}{60\%}$ (Eq. T-3)
where ER (energy efficiency ratio) is the fraction or percentage of the theoretical SPT impact hammer energy actually transmitted to the sampler, expressed as %

• The best approach is to directly measure the impact energy transmitted with each blow with instrumented rod. When available, direct energy measurements were employed.
• The next best approach is to use a hammer and automatic (mechanical) trip hammer release system that has been demonstrated to deliver repeatable energy, and which has been calibrated based on direct (instrumented) energy measurements.
• Otherwise, ER must be estimated. For good field procedures, equipment and monitoring, the following approximate guidelines for SPT performed with rope and cathead are suggested:

<table><tr><th>Equipment</th><th>Approximate ER (see Note 3)</th><th>C_E (see Note 3)</th></tr><tr><td>-Safety Hammer[1]</td><td>0.4 to 0.75</td><td>0.7 to 1.2</td></tr><tr><td>-Donut Hammer[1]</td><td>0.3 to 0.6</td><td>0.5 to 1.0</td></tr><tr><td>-Donut Hammer[2]</td><td>0.7 to 0.85</td><td>1.1 to 1.4</td></tr><tr><td>-Automatic-Trip Hammer (Donut or Safety Type)</td><td>0.5 to 0.8</td><td>0.8 to 1.4</td></tr></table>• For lesser quality fieldwork (e.g.: irregular hammer drop distance, excessive sliding friction of hammer on rods, wet or worn rope on cathead, etc.) further judgmental adjustments are needed.</td></tr>
</table>

Notes: (1) Based on rope and cathead system, two turns of rope around cathead, “normal” release (not the Japanese “throw”), and rope not wet or excessively worn.

(2) Rope and cathead with special Japanese “throw” release. (See also Note 4.)

(3) For the ranges shown, values roughly central to the mid-third of the range are more common than outlying values, but ER and CE can be even more highly variable than the ranges shown if equipment and/or monitoring and procedures are not good.

(4) Common Japanese SPT practice requires additional corrections for borehole diameter and for frequency of SPT hammer blows. For “typical” Japanese practice with rope and cathead, donut hammer, and the Japanese “throw” release, the overall product of CB x CE is typically in the range of 1.0 to 1.3.

The correction for "short" rod length between the driving hammer and the penetrating sampler was taken as a nonlinear "curve", rather than the step-wise incremental values of the NCEER Workshop recommendations, but the two agree well at all NCEER mid-increments of length. Except for cases where rod "stick-up" (protrusion) above the top of the borehole was recorded, rod protrusion of ~ 1.2 m (~ 4 ft) above the top of the borehole was assumed for donut hammers and for the USGS safety hammers, and rod protrusion of ~ 2.1 m (~7 ft) was assumed for all other safety hammers. When direct SPT energy measurements are available, energy and short rod corrections are combined, and are based on instrumentally measured energies. It is rare to perform an SPT with a distance between the top of the sampler and the point of impact at the top of the rod being less than about 2 meters. Data is sparse here, and it is recommended that in performing "forward" engineering analyses of actual projects rod length should be limited to d ≥ 1.5 to 2 meters in using Equation T-1 in Table 2-9.

C_S was applied in cases where a "nonstandard" SPT sampler was used in which the sampler had an internal space for sample liner rings, but the rings were not used. This results in an "indented" interior liner annulus of enlarged diameter, while the diameter of the cutting shoe is slightly less. This in turn reduces friction between the sample and the interior of the sampler, resulting in reduced overall penetration resistance (Seed et al., 1984 and 1985). The reduction in penetration resistance is on the order of ~10 % in loose soils (N_1<10 blows/ft.), and ~30 % in very dense soils (N_1>30 blows/ft.), so C_S varies from 1.1 to 1.3 over this range.

Borehole diameter corrections (C_B) were employed as recommended in the NCEER Workshop Proceedings.

Corrections for hammer energy (C_E), which were often significant, were applied as recommended by the NCEER Working Group, except in those cases where better hammer-specific/system-specific information (based on direct energy measurements) was available. Cases where better information was available included cases where either direct (instrumented) energy measurements were made during driving of the SPT sampler, or where the hammer and the raising/dropping system had been pre-calibrated by means of direct (instrumented) driving energy measurements, and had been demonstrated to deliver blows with repeatable energy. It is noted again that when direct hammer energy measurement is available, short rod and hammer energy corrections are executed in a combined manner. For the cases compiled from the Seed et al. (1985) database, C_E is consistently estimated, unless new evidence suggesting otherwise exists. For Hyogoken-Nanbu (Kobe, Japan) earthquake case histories, average SPT hammer energy ratio (ER) is taken as ER=73%.

2.2.2.7. Variance in SPT Blowcounts

Based on assessment of the distributions of $N_{1,60}$ in all of the field cases wherein more than 5 values of $N_{1,60}$ were available to characterize a given stratum, the characteristic distribution of $N_{1,60}$ appeared to be between normally distributed and log-normally distributed. A normal distribution was selected to represent $N_{1,60}$ within each stratum. For the relatively few cases wherein the critical stratum had only one single useful $N_{1,60}$-value, the standard deviation was taken as 2 blows/ft; a value typical of the larger variances among the cases with multiple $N_{1,60}$ values within the critical stratum (reflecting the increased uncertainty due to lack of data when only a single value was available).

2.3. DEVELOPMENT OF PROBABILISTIC LIQUEFACTION TRIGGERING CURVES

A detailed presentation of the probabilist framework employed in the development of liquefaction triggering will be given in Chapter 4. However, a brief summary is also presented as follows. Using the field case history data as assessed and presented in Table 2-7, liquefaction triggering correlations were developed based on probabilistic "regressions" performed employing the maximum likelihood estimation (MLE) method. The specific formulations and approaches employed are described in detail by Cetin (2000), Cetin et al., (2002), and Cetin et al. (2016), and in Chapter 4 of this report. The MLE serves essentially the same purpose as multi-dimensional probabilistic "regression" analyses, but (1) allows for separate and more appropriate treatment of various contributing sources of aleatory uncertainty, and (2) facilitates treatment of more descriptive variables/parameters than have been attempted in most previous, similar studies while also permitting monitoring of model parameter interactions and co-variances. Within the MLE analyses, all data were modeled not as "points", but rather as random variables with variances in descriptive input variables (i.e.: CSR, $N_{1,60}$, σ'_v, FC, M_w). That means, for example, that the "dots" in figures like Figure 2-5 are not really "points"; instead they are centroidal representations of what are actually distributions (much like bumps) with varying uncertainties in both the lateral and vertical directions depending upon the quality and quantity of data available for each case history.

A special challenge for development of unbiased soil liquefaction triggering correlations is post-earthquake field investigation sampling disparity. Researchers who receive funding and support for post-earthquake liquefaction field investigations tend to proceed mainly to sites where liquefaction was observed. As a result, there are a relatively large number of "liquefied" sites in the available databases. It is then more difficult to find and investigate sites (and strata) where soils of potentially liquefiable types exist, but did not liquefy; because by not liquefying they did not so obviously reveal themselves. As a result, there is a relative paucity of useful "nonliquefied" field performance case

histories. This is not due to a lack of potentially liquefiable but nonliquefied soils; instead it is the result of natural sampling bias as researchers are preferentially drawn to liquefied sites.

This sampling disparity results in a biased data set in which nonliquefied sites are under-represented. Development of a method for accounting for this sampling disparity has long proven elusive. Cetin et al. (2004) developed such a method, and it is explained in Cetin (2000), Cetin et al. (2002), and Cetin et al. (2004). For the data set of the Cetin et al. (2004) database, it was determined that sampling disparity would be suitably mitigated by (1) up-weighting the value of nonliquefied sites by a weighting factor of 1.2, and (2) by down-weighting the value of liquefied sites by a factor of 0.8 in performing the probabilistic regressions. This is approximately equivalent to simply up-weighting the non-liquefied cases by a factor of 1.5, but partitioning the weighting factors among both liquefied and non-liquefied cases allows improved treatment of overall model uncertainty.

The updated 2016 database has nearly the same overall characteristics (and almost the same cases) as the 2004 database, and the same weighting factors (1.2 and 0.8) were again applied to liquefied and nonliquefied case histories. An additional weighting factor was needed to address the large number of case histories from the 1995 Hyogoken-Nanbu (Kobe, Japan) earthquake. The database has 56 case histories from this single event; and that very large number results in an unbalance of the overall data set wherein that single event is over-represented. This was addressed in the regressions performed for development of the Cetin et al. (2016) database by down-weighting the Hyogoken-Nanbu (Kobe) earthquake case histories by a weighting factor of 0.25. Additional details of the regressions performed are presented in Chapter 3 & 4.

It is noted that Boulanger and Idriss (2012) adopted the same weighting factors of 0.8 and 1.2 for liquefied and nonliquefied cases, respectively, based on the work of Cetin (2000) and Cetin et al. (2004). Boulanger and Idriss did not, however, apply any weighting factor to the Hyoguken-Nanbu (Kobe) earthquake case histories.

The maximum likelihood approach for developing a limit state model for the initiation (or "triggering") of soil liquefaction begins with the selection of a mathematical model. The model for the limit-state function has the general form $g = g(\mathbf{x},\boldsymbol{\Theta})$, where $\mathbf{x} = (N_{1,60}, CSR_{\sigma'_v,\alpha,M_w}, M_w, FC, \sigma'_v)$ is a set of descriptive variables, and $\boldsymbol{\Theta}$ is the set of unknown model coefficients. Consistent with the usual definition in structural reliability, liquefaction is assumed to have occurred when $g(\mathbf{x},\boldsymbol{\Theta})$ takes on a negative value. The limit-state surface $g(\mathbf{x},\boldsymbol{\Theta}) = 0$ denotes the initiation of liquefaction (or marginal liquefaction). The following model from Cetin et al. (2002 and 2004) is used for the limit state function:

$$\hat{g}(x,\boldsymbol{\theta}) = N_{1,60}(1+\theta_1 FC) - \theta_2 \ln(M_w) - \theta_3 \ln(\sigma'_v / \mathrm{P_a}) + \theta_4 FC + \theta_5 - \theta_6 \ln(CSR_{\sigma'_v,\alpha,M_w}) \quad \textbf{Eqn. (2-10)}$$

where $\boldsymbol{\theta} = (\theta_1, \ldots, \theta_6)$ is the set of unknown model coefficients. An additional model coefficient θ_7 will be introduced later to calibrate the case history variance terms.

The limit state function given in Equation 2-10 has the advantage of assessing K_σ, K_{Mw} and fines corrections within a unified framework. Hence, as part of the maximum likelihood assessment, the overall liquefaction triggering relationship will be assessed jointly and all at once with these corrections or adjustment factors included. Equation 2-10 assumes that the liquefaction triggering hazard can be completely explained by a set of five descriptive variables $\mathbf{x} = (N_{1,60}, CSR_{\sigma'_v,\alpha,M_w}, M_w, FC, \sigma'_v)$. But other variables exist, which may influence the initiation of liquefaction. Even if the selected descriptive variables were able to fully explain the liquefaction triggering phenomenon, the adopted mathematical expression may not have the ideal form. Hence, Equation 2-10 is by definition an imperfect model of the limit-state function. This is signified by use of a superposed "hat" on $\hat{g}$. To account for the influences of the missing variables and the possible incorrect model form, a random model error term, ε, was introduced as given in Eq.2-11.

$$g(x,\boldsymbol{\theta}) = N_{1,60}(1+\theta_1 FC) - \theta_2 \ln(M_w) - \theta_3 \ln(\sigma'_v / \mathrm{P_a}) + \theta_4 FC + \theta_5 - \theta_6 \ln(CSR_{\sigma'_v,\alpha,M_w}) + \varepsilon \quad \textbf{Eqn. (2-11)}$$

With the aim of producing an unbiased model (i.e., one that, on the average, makes the correct prediction), the mean of ε is set to zero, and for convenience is assumed to be normally distributed. The standard deviation of ε, denoted σ_ε, however is unknown and must be estimated as part of the regression. The set of unknown coefficients of the model, therefore, is $\boldsymbol{\Theta} = (\boldsymbol{\theta}, \sigma_\varepsilon)$.

Let $\mathbf{x}_i = (N_{1,60,i}, CSR_{\sigma'_v,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i})$ be the values of $N_{1,60}$, *CSR*, M_w, *FC* and σ'_v for the i^{th} case history, respectively, and ε_i be the corresponding realization of the model correction term. If the i^{th} case history field performance is liquefied, then $g(x_i, \varepsilon_i, \boldsymbol{\theta}) \le 0$. On the other hand, if the i^{th} case history field performance is nonliquefied, then $g(x_i, \varepsilon_i, \boldsymbol{\theta}) > 0$. When $g(x_i, \varepsilon_i, \boldsymbol{\theta}) \cong 0$ then the case history is defined as marginally liquefied (or marginally nonliquefied). Assuming that the Cetin et al. (2016) database is compiled from statistically independent liquefaction field performance case histories, the likelihood function can be written as the product of the probabilities of the observations given in Equation 2-12.

$$L(\boldsymbol{\theta}, \sigma_\varepsilon) = \prod_{l=1}^{\text{Num. of liquefied sites}} P\left[g(x_l, \varepsilon_l, \boldsymbol{\theta}) \le 0\right] \cdot \prod_{j=1}^{\text{Num. of non-liquefied sites}} P\left[g(x_j, \varepsilon_j, \boldsymbol{\theta}) > 0\right] \cdot \prod_{k=1}^{\text{Num. of marginally-liquefied sites}} P\left[g(x_k, \varepsilon_k, \boldsymbol{\theta}) \cong 0\right] \quad \textbf{Eqn. (2-12)}$$

Suppose that $\mathbf{x}_i = (N_{1,60,i}, CSR_{\sigma'_v,\alpha,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i})$ for each liquefaction field performance case history is exact, i.e., no measurement or estimation uncertainty is present. Then, noting that Equation 2-11 has the normal distribution with mean $\hat{g}(x_i, \boldsymbol{\theta})$ and standard deviation σ_ε, the likelihood function (Equation 2-12) can be written as given in Equation 2-13.

$$L(\boldsymbol{\theta}, \sigma_\varepsilon) = \prod_{\text{liquefiedsites}} \Phi\left[-\frac{\hat{g}(x_l, \boldsymbol{\theta})}{\sigma_\varepsilon}\right]^{w_{liq.}} \cdot \prod_{\text{non-liquefiedsites}} \Phi\left[\frac{\hat{g}(x_j, \boldsymbol{\theta})}{\sigma_\varepsilon}\right]^{w_{nonliq.}} \cdot \prod_{\text{marginally-liquefiedsites}} \varphi\left[\frac{\hat{g}(x_k, \boldsymbol{\theta})}{\sigma_\varepsilon}\right] \quad \textbf{Eqn. (2-13)}$$

In Equation 2-13, $\Phi[\cdot]$ and $\varphi[\cdot]$ are the standard normal cumulative function and the standard normal probability density functions. Also note that w_{liq} and w_{nonliq} are the sampling bias correction factors 0.8 and 1.2, respectively.

If $\mathbf{x}_i = (N_{1,60,i}, CSR_{\sigma'_v,\alpha,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i})$ is inexact (uncertain), then this additional uncertainty should be included in the formulation of the likelihood function as given in Equation 2-14.

$$L(\boldsymbol{\theta}, \sigma_\varepsilon) = \prod_{\text{liquefiedsites}} \Phi\left[-\frac{\hat{g}(x_i, \boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right]^{w_{liq.}} \cdot \prod_{\text{non-liquefiedsites}} \Phi\left[\frac{\hat{g}(x_i, \boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right]^{w_{nonliq.}} \cdot \prod_{\text{marginally-liquefiedsites}} \varphi\left[\frac{\hat{g}(x_i, \boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right] \quad \textbf{Eqn. (2-14)}$$

Assuming that the mean values of the case history descriptive input parameters $N_{1,60,i}, CSR_{\sigma'_v,\alpha,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i}$ are estimated in an unbiased manner (i.e., mean choices, rather than conservative or unconservative choices, are made in case history processing) then the error terms of $N_{1,60,i}, \ln\left(CSR_{\sigma'_v,\alpha,M_w,i}\right), \ln\left(M_{w,i}\right), FC_i, \ln\left(\sigma'_{v,i}\right)$ can be considered as normally distributed random variables with zero means and standard deviations $\sigma_{N_{1,60},i}$, $\sigma_{\ln(CSR_{\sigma'_v,\alpha,M_w}),i}$, $\sigma_{\ln(M_w),i}$, $\sigma_{FC,i}$ and $\sigma_{\ln(\sigma'_v),i}$, respectively. Then the overall variance of each liquefaction field performance case history (σ^2_{tot}) is estimated as the sum of the variance of case history input parameters ($\sigma^2_{case\text{-}history}$) and model error ($\sigma^2_\varepsilon$) as given in Equation 2-15.

$$\sigma^2_{tot,i} = \left(\theta_7 \cdot \sigma_{case-history,i}\right)^2 + \sigma^2_\varepsilon \quad \textbf{Eqn. (2-15)}$$

In Equation 2-15, $\sigma_{case-history,i}$ represents the consolidated uncertainty of individual case history input parameters, and as discussed earlier a scaling factor of θ_7 is systematically applied to them. This factor (θ_7) is one of the regressed parameters of the overall triggering relationship. If the higher-order terms are eliminated, then $\sigma_{case-history,i}$ can be estimated as given Equation 2-16

$$\sigma^2_{case-history,i} = (\theta_6)^2 \cdot \left[\text{cov}(CSR_{\sigma'_v,\alpha,M_w,i})\right]^2 + \sigma^2{}_{N_{1,60,I}} \cdot (1+\theta_1 \cdot FC_i)^2 + \sigma^2{}_{FC_i} \cdot (\theta_1 \cdot N_{1,60,i} + \theta_4)^2 + (\theta_3)^2 \cdot \left[\text{cov}(\sigma'_{v,i})\right]^2 \quad \textbf{Eqn. (2-16)}$$

Finally, the eight model coefficients (i.e.: θ_1 through θ_7, and σ_ε) are assessed all at once in a single overall regression in order to maximize the likelihood function presented in Equation 2-14. The resulting new model coefficients are listed in Table 2-10. This table also presents the previous Cetin et al. (2004) model coefficients for comparison. The resulting (new) recommended seismic soil liquefaction triggering relationships are presented in Equations 2-17 and 2-18.

The resulting new probabilistic boundary curves, based on the updated field performance case history database with 210 liquefaction triggering field case histories are shown in Figure 2-5. Figure 2-6 presents a direct comparison between the new triggering relationship, and the previous relationship of Cetin et al. (2004).

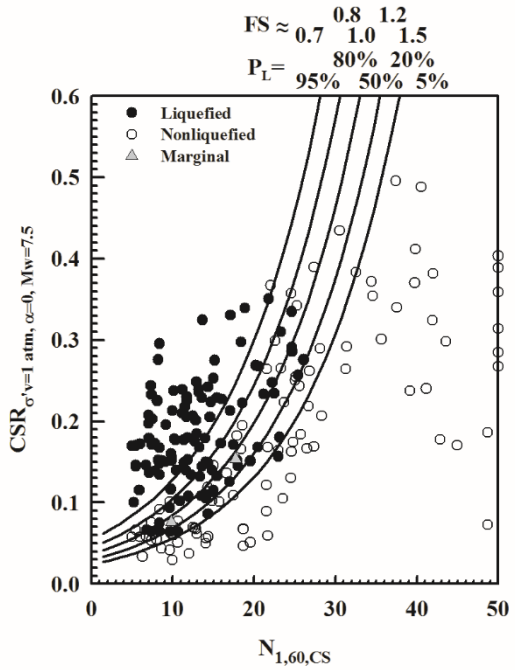


Fig. 2-5. New probabilistic seismic soil liquefaction triggering curves

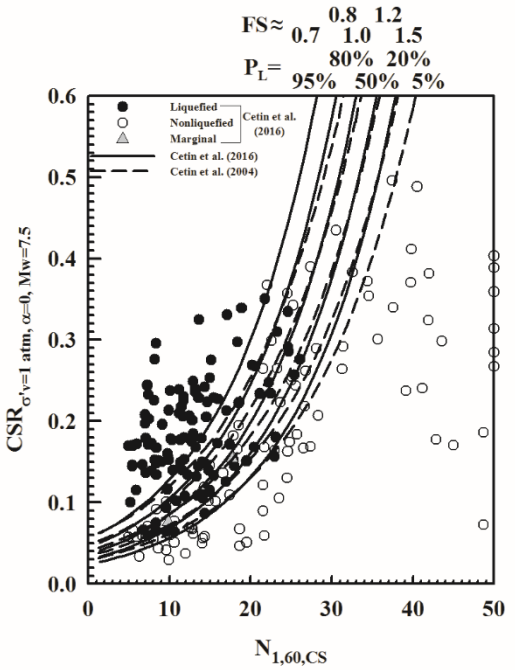


Fig. 2-6. Comparison between the triggering boundary curves of Cetin et al. (2004) and these current studies.

Table 2-10. A comparative summary of limit state model coefficients

Model Coefficients	Cetin et al. (2004)	Cetin et al. (2016)
θ_1	0.004	0.00167
θ_2	29.530	27.352
θ_3	3.700	3.958
θ_4	0.050	0.089
θ_5	16.850	16.084
θ_6	13.320	11.771
θ_7	-	0.392
σ_ϵ	2.70	2.95

$$P_L(N_{1,60}, CSR_{\sigma'_v,\alpha=0,M_w}, M_w, \sigma'_v, FC) = \Phi\left(-\frac{\left(\begin{array}{l} N_{1,60}\cdot(1+\theta_1\cdot FC)-\theta_6\cdot\ln\left(CSR_{\sigma'_v,\alpha=0,M_w}\right)- \\ \theta_2\cdot\ln(M_w)-\theta_3\cdot\ln\left(\frac{\sigma'_v}{P_a}\right) \\ +\theta_4\cdot FC+\theta_5 \end{array}\right)}{\sigma_\varepsilon}\right) \quad \textbf{Eqn. (2-17)}$$

$$CRR(N_{1,60}, M_w, \sigma'_v, FC, P_L) = \exp\left[\frac{\left(\begin{array}{l} N_{1,60}\cdot(1+\theta_1\cdot FC)-\theta_2\cdot\ln(M_w) \\ -\theta_3\cdot\ln\left(\frac{\sigma'_v}{P_a}\right)+\theta_4\cdot FC+\theta_5+\sigma_\varepsilon\cdot\Phi^{-1}(P_L) \end{array}\right)}{\theta_6}\right] \quad \textbf{Eqn. (2-18)}$$

In Eq. 2-17, P_L is the probability of liquefaction in decimals (i.e. P_L = 30% is input as 0.30), $CSR_{\sigma_v',\alpha=0,M_w}$ is not "adjusted" for vertical overburden stress or magnitude/duration effects (corrections are executed within the Equation itself), FC is percent fines content (by dry weight) expressed as an integer (e.g.: 12% fines is input as FC = 12) with the limit of $5 \le FC \le 35$, P_a is atmospheric pressure (= 1 atm.=101.3 kPa=2116.2 psf) in the same units as the in-situ vertical effective stress (σ'_v), and Φ is the standard cumulative normal distribution. The cyclic resistance ratio, CRR, for a given

probability of liquefaction can be expressed as given in Equation 2-18, where $\Phi^{-1}(P_L)$ is the inverse of the standard cumulative normal distribution (i.e. mean = 0, and standard deviation = 1). For spreadsheet construction purposes, the command in Microsoft Excel for this specific function is "NORMINV(P_L,0,1)". In Figure 2-5, factor of safety (FS) values corresponding to probability of liquefaction 5, 20, 50, 80, 95 % are also shown. Approximate factors of safety, FS values can be estimated by Equation 2-19 based on the assumption that P_L value of 50 % corresponds to a best-estimate factor of safety value equal to 1.0 and the resulting FS values for the five sets of P_L contours are also shown in Figure 2-5.

$$FS = \frac{CRR(P_L = 50\%)}{CRR(P_L)} = \exp\left[\frac{\left(-\sigma_\varepsilon \cdot \Phi^{-1}(P_L)\right)}{\theta_6}\right] = \exp\left[\frac{\left(-2.95 \cdot \Phi^{-1}(P_L)\right)}{11.771}\right] = \exp\left[-0.251 \cdot \Phi^{-1}(P_L)\right] \quad \textbf{Eqn. (2-19)}$$

For the purpose of assessing the effects of sampling disparity (and weighting), the same exercise was then repeated, but this time without applying weights on non-liquefied and liquefied case history data points. Figure 2-7 presents the relative positions of equi-probability (5%, 20%, 50%, 80% and 95%) liquefaction triggering contours with (solid lines) and without the weighting factors (dashed lines). Since it would be potentially over-conservative to leave the sampling disparity problem unaddressed, the model developed with weighting factors of $w_{\text{non-liquefied}} = 1.2$ and $w_{\text{liquefied}} = 0.8$ is recommended for forward engineering assessments.

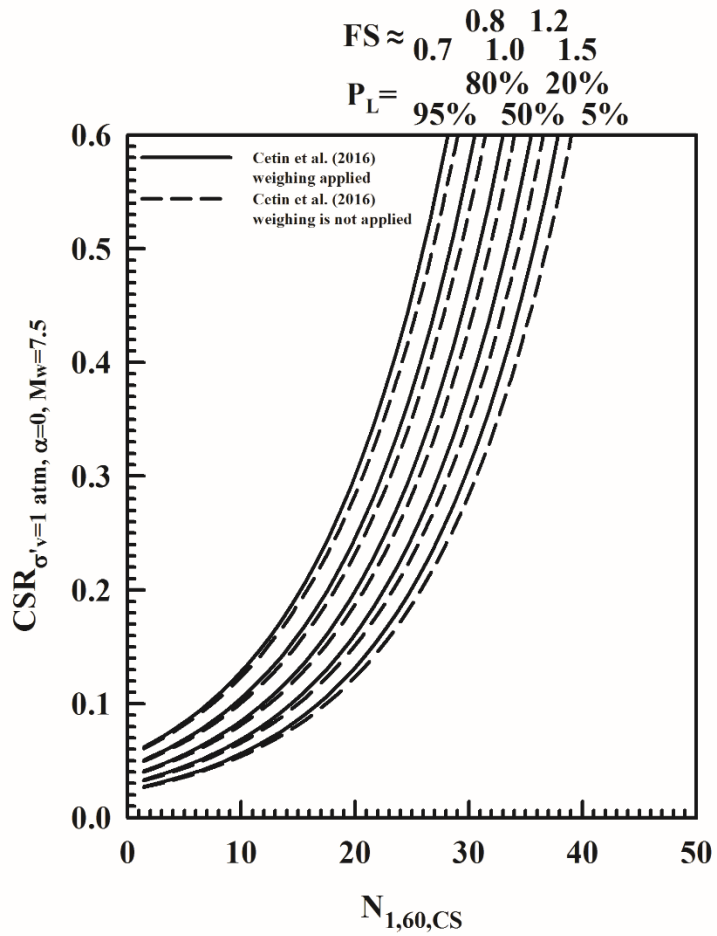


Fig. 2-7. Effects of implementing compensation for sampling disparity (solid lines) vs. leaving sampling disparity unaddressed.

2.4. RECOMMENDED USE OF THE NEW CORRELATIONS

The proposed new probabilistic correlations can be used in either of two ways. They can be used directly, all at once, as summarized in Equation 2-17 and 18. Alternatively, they can be used "in parts" as has been conventional for most previous, similar methods. To do this, measured SPT N-values in the critical layer must be corrected to $N_{1,60}$-values, using Equation 2-9. The resulting $N_{1,60}$-values must then be further corrected for fines content $N_{1,60,cs}$-values, using either Equations 20 and 21 or Figure 5 as.

$$N_{1,60,CS} = N_{1,60} + \Delta N_{1,60} \qquad \textbf{Eqn. (2-20)}$$

where

$$\Delta N_{1,60} = FC \cdot (\theta_1 \cdot N_{1,60} + \theta_4) = FC \cdot (0.00167 \cdot N_{1,60} + 0.089) \quad \text{lim: } 5\% \le FC \le 35\% \qquad \textbf{Eqn. (2-21)}$$

Figure 2-8 shows the regressed fines corrections for these current studies, and also for the previous triggering relationships of Cetin et al. (2004). The overall average fines corrections for the

two relationships are closely similar, but the new fines correction relationship has a lesser dependence upon $N_{1,60}$.

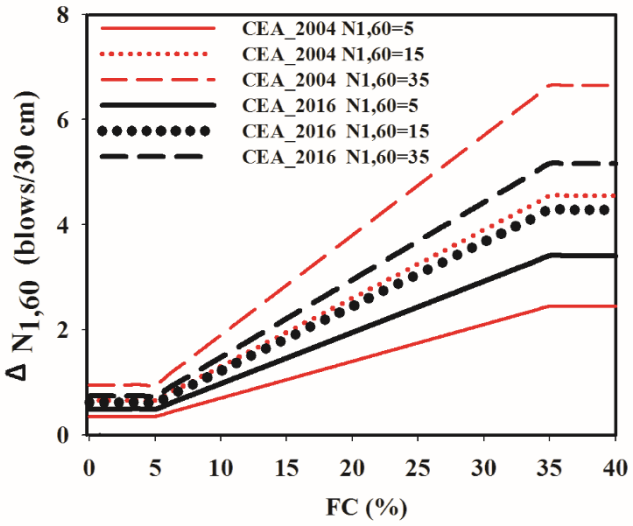


Fig. 2-8 Proposed $N_{1,60}$ dependent fines correction

Similarly, in-situ equivalent uniform $CSR_{\sigma_v',\alpha,M_w}$ must be evaluated either based on (1) direct seismic site response analyses, or (2) direct seismic site response and soil-structure-interaction analyses, or (3) using the "simplified" approach employing Equation 2-5, and the Cetin et al. (2004) r_d relationships as given in Equation 2-6.

$CSR_{\sigma_v',\alpha,M_w}$ must then be adjusted by the K_σ, K_α, and K_{Mw} correction factors to convert overburden and static shear stresses, and duration (magnitude) to the references states of σ'_v=1 atm., α=0 and M_w = 7.5, as given in Equations 22 through 24. Note that for level sites (which includes all field performance case histories in the database) the value of K_α = 0.

$$CSR_{\sigma_v'=1atm.,\alpha=0,M_w=7.5} = CSR_{\sigma_v',\alpha,M_w} \cdot \frac{1}{K_\sigma} \cdot \frac{1}{K_{Mw}} \cdot \frac{1}{K_\alpha} \text{; lim: } CSR_{\sigma_v'=1atm.,\alpha=0,M_w=7.5} \leq 0.6 \qquad \textbf{Eqn. (2-22)}$$

$$K_\sigma = \left(\frac{\sigma_v'}{P_a}\right)^{-\theta_3/\theta_6} = \left(\frac{\sigma_v'}{P_a}\right)^{-3.958/11.771} = \left(\frac{\sigma_v'}{P_a}\right)^{-0.336} \text{; lim: } 0.8 \leq K_\sigma \leq 1.6 \qquad \textbf{Eqn. (2-23)}$$

$$K_{Mw} = \left(\frac{M_w}{7.5}\right)^{-\theta_2/\theta_6} = \left(\frac{M_w}{7.5}\right)^{-27.352/11.771} = \left(\frac{M_w}{7.5}\right)^{-2.324} \text{; lim: } 5.5 \leq M_w \leq 8.4 \qquad \textbf{Eqn. (2-24)}$$

As an alternative to Equations 2-23 and 2-24, K_σ and K_{Mw} corrections can be employed by using Figures 2-9 and 2-10.

Figure 2-9 shows the new K_σ curve regressed from the liquefaction performance field case history database, and the lower portion of Figure 2-9 shows a histogram of the distribution of case

histories with different ranges of σ'$_v$. This field case history-based K_σ relationship is valid over the range of approximately 0.25 atm ≤ σ'$_v$ ≤ 1.8 atm. Extrapolation to higher values of σ'$_v$ for forward engineering analyses is discussed in Chapter 3.

The new K_σ curve is observed to be largely consistent with the Cetin et al. (2004) recommendations, which were also based on regressions of field performance case histories, and at least approximately also fits laboratory cyclic simple shear test results of Cetin and Bilge (2013), and is similar at least in form to a dataset of cyclic triaxial and cyclic simple shear data collected and compiled by Montogomery et al. (2014). A further discussion of the relative merits of laboratory-based K_σ relationships vs. field performance-based K_σ relationships, and of extrapolation to higher effective vertical stresses than 1.8 atm., is presented in Chapter 3.

Figure 2-9 presents the new K_{Mw} curve, which was also developed based on the overall regression of the liquefaction triggering field performance case history database. It agrees well with the relationship of Cetin et al. (2004), which was also based on regression of field case histories, and it is approximately centered between the K_{Mw} relationships of Idriss (1995) and Boulanger and Idriss (2012), both of which are based on laboratory undrained cyclic test data (and do not employ the field case history data). Agreement between these four sets of K_{Mw} relationships, developed based on two very different sets of approaches, is very good.

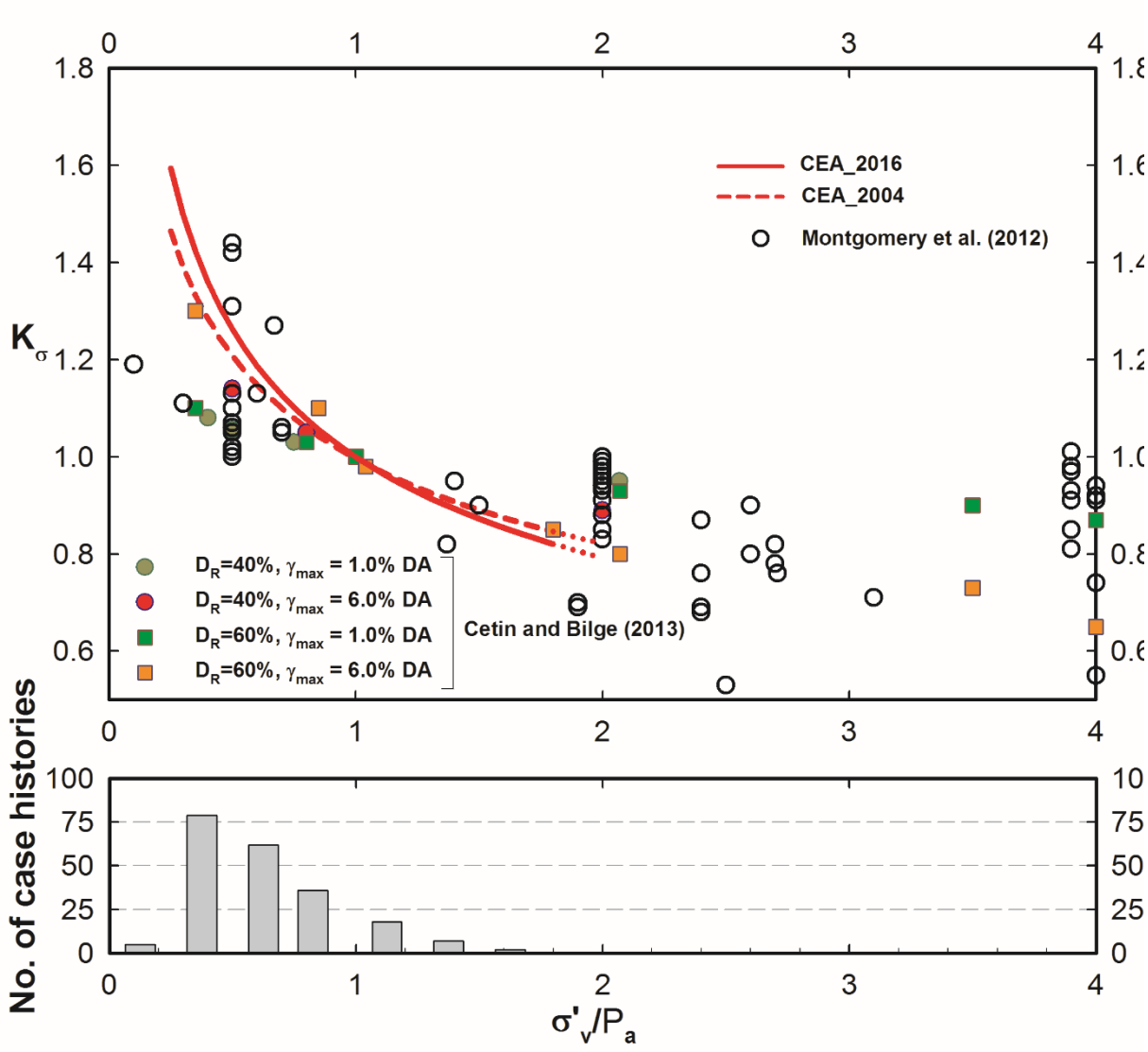


Fig. 2-9 Newly regressed K_σ corrections compared with available cyclic laboratory test data

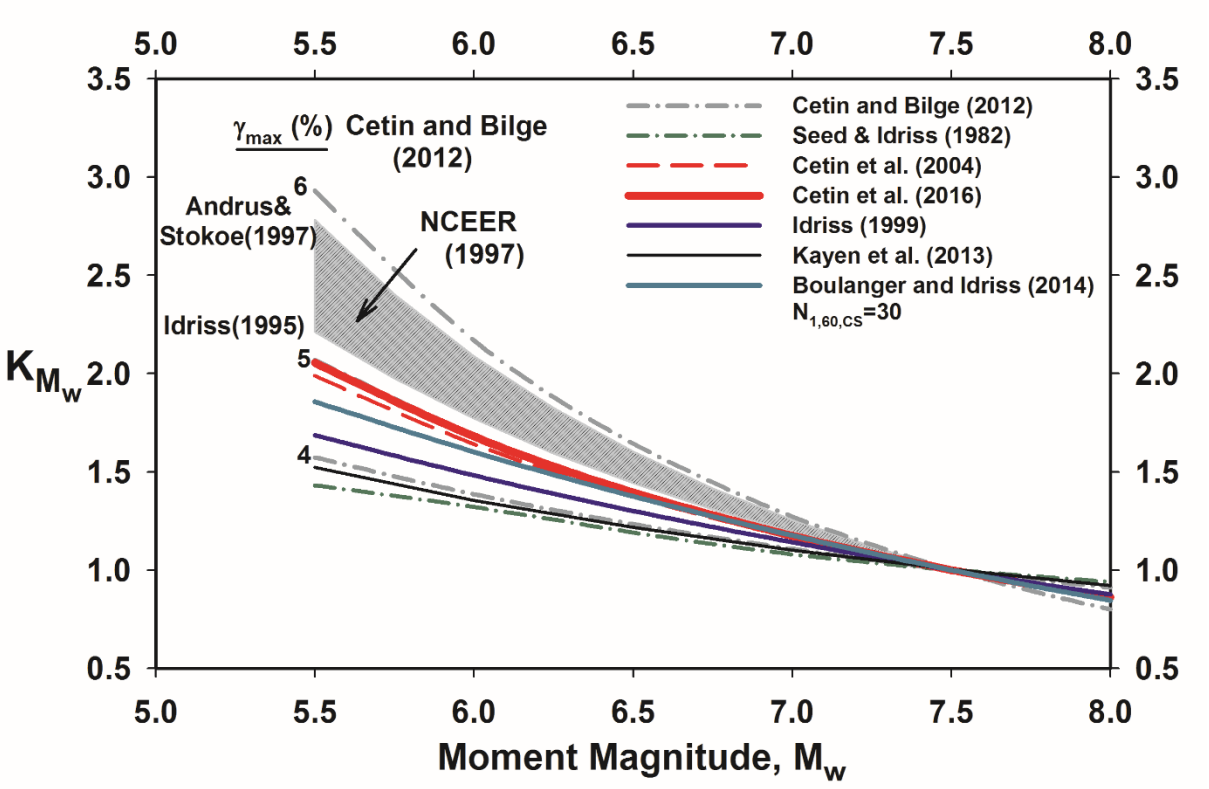


Fig. 2-10 Newly regressed K_{Mw} corrections compared with available literature

The resulting, fully adjusted and normalized values of $N_{1,60,cs}$ and $CSR_{\sigma_v'=1atm.,M_w=7.5,\alpha=0}$ can then be used, with Figure 2-5, to assess probability of initiation of liquefaction. For "deterministic" evaluation of liquefaction resistance, either Equation 2-18 or the P_L = 50% (≈FS=1.0) boundary curve of Figure 2-5 can be used to approximate factors of safety.

2.5. CHAPTER CONCLUSIONS

New probabilistic and deterministic liquefaction triggering curves are recommended based on the most recent case history database of Cetin et al. (2016). Maximum likelihood models for assessment of seismic soil liquefaction initiation are developed. In the course of developing the proposed likelihood function, the relevant uncertainties including (a) measurement/estimation uncertainties of input parameters, (b) limit state model imperfection, (c) statistical uncertainty, and (d) uncertainties arising from inherent variability were addressed.

The use of site-specific and event-specific seismic site response-based calculations of CSR, and of compatible (unbiased) probabilistic r_d relationships based on suitably representative suites of actual site conditions (drawn from 50 actual liquefaction field performance case history database sites) in "simplified" assessment of in-situ cyclic shear stress ratio (CSR) for the remaining 150 field case histories, results in triggering relationships that are unbiased with respect to use in conjunction with

either (1) direct seismic site response analyses (or site response and soil-structure interaction analyses) for evaluation of in-situ CSR, or (2) improved "simplified" assessment of in-situ CSR in forward engineering analyses. This is important, as these studies also show that other widely used correlations are unconservatively biased when used in conjunction with direct site response analyses (or soil-structure interaction analyses) for assessment of CSR, as a result of bias in other "simplified" r_d recommendations currently available.

The new models provide an improved basis for engineering assessment of the likelihood of liquefaction initiation. The proposed models deal explicitly with the issues of (1) fines content (FC), (2) magnitude (duration) effects (i.e.: K_{Mw} correction), and (3) effective overburden stress effects (i.e.: K_s correction), and these "secondary" correction factor relationships are also all developed based on regression of the full field case history database. As a result, the overall triggering relationships provide both (1) an unbiased basis for evaluation of liquefaction initiation hazard, and (2) a basis for assessment of overall model uncertainty. Overall uncertainty in application of these new correlations to field problems is driven strongly by the difficulties/uncertainties associated with project-specific engineering assessment of the necessary "loading" and "resistance" variables. In the estimation of these loading and resistance terms, the need to use a series of corrections and normalizations continues to be involved at the heart of the triggering assessment. This encourages devotion of attention and discussions to improved understanding of these secondary corrections and normalization relationships (e.g.: fines adjustments, K_σ and K_{Mw}).

The intent of this development of an updated set of triggering relationships is to provide an unbiased set of liquefaction triggering relationships, correctly normalized (or "centered") at σ'_v = 1 atm., M_W = 7.5 and α = 0. These relationships can be used for either probabilistic or deterministic forward engineering project evaluations of liquefaction triggering potential.

3. EXAMINATION OF DIFFERENCES BETWEEN THREE SPT-BASED SOIL LIQUEFACTION TRIGGERING RELATIONSHIPS

3.1. INTRODUCTION

Chapter 2 presented the updating of the seismic soil liquefaction triggering relationship of Cetin et al. (2004), and compared the updated relationship with its earlier version. In this chapter, a detailed cross-comparison is made between three triggering relationships: (1) Seed et al. (1985) as slightly updated by the NCEER Working Group (Youd et al., 2001), (2) Boulanger and Idriss (2012), and (3) Cetin et al. (2016). These three triggering relationships will be referred to hereafter as SEA_1985, BI_2012 and CEA_2016, respectively. Differences between these three triggering relationships, and the apparent causes of these differences are examined. Also examined are the impacts of these differences on levels of conservatism with regard to evaluation of likelihood of triggering of liquefaction.

Figure 3-1 shows the established soil liquefaction triggering "boundary curves" associated with each of these relationships. All three relationships have been re-plotted at the same scales to make cross-comparisons easier and more direct. The liquefaction triggering field case history data points plotted in each figure are those of the original authors, and all data points (as well as the boundary curves) are normalized to a fines-corrected "clean sand" reference condition of $N_{1,60,CS}$ rather than $N_{1,60}$.

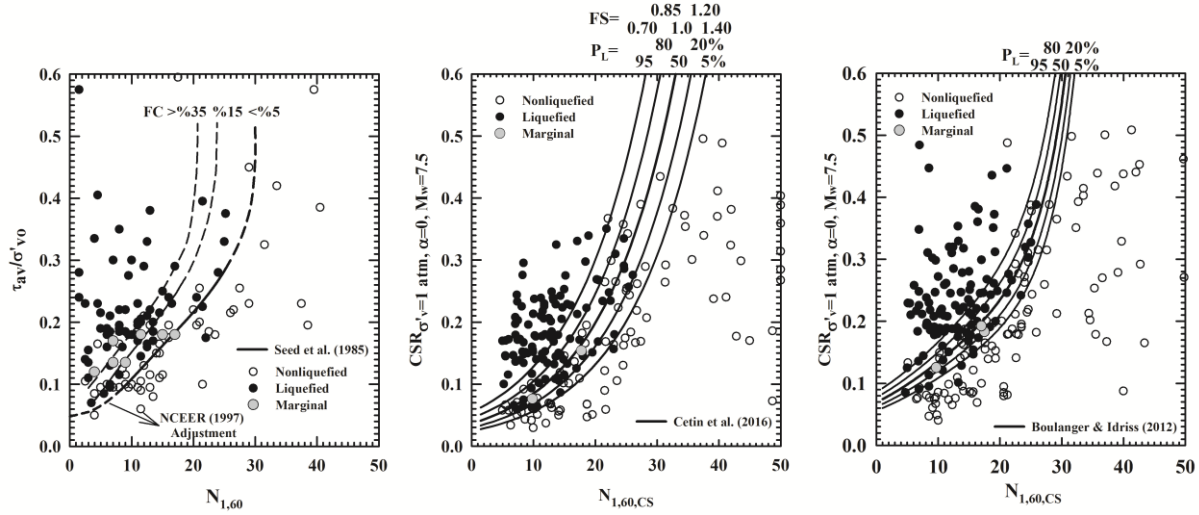


Fig. 3-1. Liquefaction triggering relationships as proposed by (a) Seed et al. (1985) as modified slightly by Youd et al.(2001), (b) Cetin et al. (2016), and (c) Boulanger and Idriss (2012) (CSR values are plotted after correcting for typograhical errors described in Boulanger and Idriss, 2014).

Plotting all three relationships to the same scale is helpful with regard to making cross-comparisons. But it can be difficult to see in detail some of the differences between the boundary curves of these three relationships. Accordingly, Figure 3-2(a) shows all three relationships, with the BI_2012 and CEA_2016 relationships represented by contours of P_L = 50%, and Figure 2(b) repeats Figure 2(a) but with these two probabilistic relationships represented by contours of P_L = 20%. The SEA_1985 relationship had no probabilistic basis, so the clean sand boundary curve for that relationship remains in the same position in both figures, and serves as a useful visual point of reference. All of the boundary curves shown in Figure 3-2 are presented on a "clean sand" basis (fines content ≤ 5%).

As shown in Figure 3-2, there are significant differences between the triggering boundary curves at these two important levels of hazard or probability of liquefaction. At low SPT penetration resistance ($N_{1,60,CS}$ ≤ 20 blows/ft) it can be seen that (1) the probabilistic curves of BI_2012 are the least conservative, (2) the probabilistic curves of CEA_2016 are the most conservative, and (3) the deterministic curve of SEA_1985 is intermediate and transitions between the other two. It is interesting to note that Seed et al. (1985) had targeted approximately a 10 % to 20 % probability of liquefaction, though with their relatively sparse available field performance data set and the lack of a formal probabilistic regression, there was considerable uncertainty as to the degree to which they had met that target (Seed, Personal Communication, 1988).

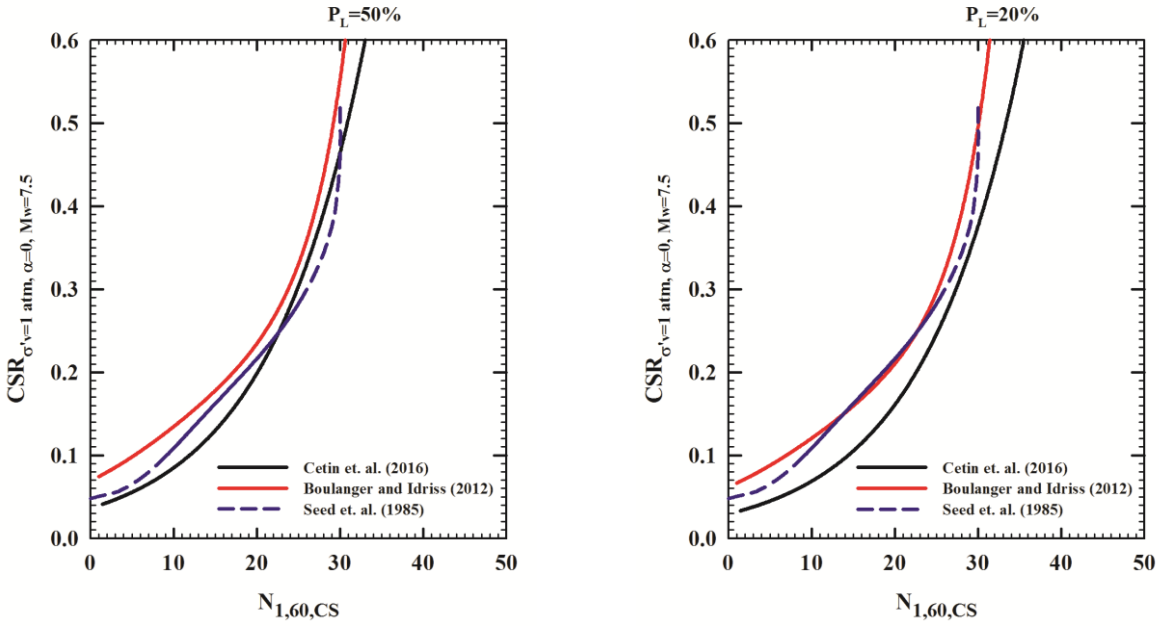


Fig. 3-2. Clean sand boundary curves ($N_{1,60,CS}$) for all three relationships co-plotted on the same two figures; (a) the deterministic curve of SEA_1985 (as modified slightly by Youd et al., 2001) and the P_L = 50% contours of BI_2012 and CEA_2016, and (b) the deterministic curve of SEA_1985 and the P_L = 20% contours of BI_2012 and CEA_2016.

It must also be noted that examination of the boundary curves alone does not fully characterize overall levels of hazard or conservatism. Each of the three sets of boundary curves are developed to act in conjunction with a number of prescribed or recommended engineering protocols in terms of parameter assessment (e.g. evaluation of earthquake-induced cyclic stress ratio (CSR), $N_{1,60}$, etc.), and with a number of additional ("secondary") relationships that result in further adjustments for effective overburden stress (σ'_v), causative earthquake magnitude (M or M_w), and fines adjustments ($\Delta N_{1,60}$ as a function of fines content). These "secondary" relationships can also have potentially significant impacts on forward assessments of liquefaction hazard for engineering projects. They can either compound or partially offset levels of conservatism or unconservatism in the baseline boundary curves shown in Figures 3-1 and 3-2, and their impacts differ over varying ranges of parameters. Accordingly, it is necessary to jointly examine both (1) the proposed sets of boundary curves, as well as (2) the secondary relationships, and (3) the associated recommended engineering protocols for forward analyses of projects, in evaluating differences between the three triggering relationships.

Figures 3-1 and 3-2 also show that differences between the three triggering relationships are less pronounced at the "upper" portions of the boundary curves ($N_{1,60,CS} \geq 20$ blows/ft). Differences here follow the same pattern, with the BI_2012 curves being least conservative, and the CEA_2016 curves being the most conservative. It is important to note, however that (1) the ratios of the differences here (in terms of CSR) are lesser in magnitude than at the lower portions of the curves applicable to lower penetration resistances, and (2) differences at higher $N_{1,60,CS}$ values generally have lesser engineering implications because soils with higher corrected SPT $N_{1,60,CS}$ values have higher post-liquefaction strengths and tend to have more limited cyclic deformation potential as well.

For most engineered projects, it is soils with lower corrected SPT penetration resistance ($N_{1,60,CS} \leq 20$ blows/ft) that are of principal concern. Unfortunately, it is in this lower range of $N_{1,60,CS}$ values, where the potential consequences can often be high, that the greatest differences between the three triggering relationships occur.

3.2. ARE THE EIGHT "KEY CASES" CRITICAL TO THE DIFFERENCES IN THE LIQUEFACTION TRIGGERING CURVES?

Idriss and Boulanger (2012) proposed that *"the lower position of the Cetin et al. (2004) liquefaction triggering correlation is found to be primarily caused by the interpretations and usage for 8 of the 11 key case histories that had controlled the position of the liquefaction triggering correlation. When properly interpreted, these eight case histories no longer support the position of the published Cetin et al. liquefaction triggering curve."* Technical comments of the co-authors regarding Idriss and Boulanger's hypothesis was shared with the engineering community through an open public technical letter (report) submitted to NRC Committee on the State of the Art and Practice in Earthquake Induced

Soil Liquefaction Assessment (2014) (Appendix D). A brief summary of the issues raised by Idriss and Boulanger (2012), and the responses are also presented in Table 3-1.

Table 3-1. Concerns of Idriss and Boulanger (2012) regarding the eight case histories in Cetin et al. (2004) database and the authors' responses.

No	Case history	Idriss and Boulanger (2012)	Cetin et al. (2016) Database	Notes
1	Miller Farm CMF-10	Recommended to be used as a non-liquefied case history site as opposed to liquefied.	Accept as non-liquefied	CMF-10 borehole is located in a transition zone. At the time of our initial work, we classified this case as a liquefied site (Wayne et al., 1998). However, now, it is understood that the field investigation team specifically intended CMF 10 to be located in the non-liquefied zone. Given the relative continuity from CMF 5 to CMF 10, this was difficult to infer from their reference report. The field investigation team's expert judgment on this case history is accepted, and the CMF 10 case history is updated to a non-liquefied one.
2	Malden Street Unit D		Case history removed from analysis	The 1994 Northridge Earthquake, Malden Street Unit D case history site (Holzer et al. 1999) is composed of 3 distinct soil layers: Unit A is a compacted fill, located mostly above the water table, and is judged to be non-liquefiable; Unit B is a fine grained soil layer with N≈2-3 FC>70 %, average PI = 18%, and average clay content of 31 % (Holzer et al. 1999; page 5). Clayey soils with PI=18 %, were categorically judged to be potentially non-liquefiable back when Cetin (2000) studies were performed. Unit D is Pleistocene silty sand; hence it was concluded to be the suspect for liquefaction-induced ground deformations. Unit D was identified as the critical unit. BH 3 and BH 5 were used to characterize the SPT N values in this critical layer. Also note that BH 3 is located within the “permanent ground deformation” zone. However, after having revisited this case history with the extended state of knowledge today, it has been decided that the suspect for liquefaction-induced ground deformations is "very soft" to "soft" fine-grained Unit B layer; hence Malden Street Unit D case is decided to be excluded from Cetin et al (2016) database. We note that Malden Street Unit D was classified as a non-liquefied site in Idriss and Boulanger (2010) database. Contrary to this choice, we believe that Unit D may not be used in the updated liquefaction case history database as a non-liquefied soil layer due to the potential bias introduced in CSR estimations after cyclic softening of the upper soft lean clay layer (Unit B). The presence of these overlying soft soil layers, potentially susceptible to cyclic softening, can significantly reduce the induced cyclic shear stresses due to their significantly reduced inertial mass participation effects. This makes the estimation of r_d and CSR values a difficult task, which is beyond fully reliable limits of either simplified procedures or total-stress based site response analyses.

3	Kobe No 6	Recommended to be used as a non-liquefied case history site as opposed to liquefied.	Accept as non-liquefied	Kobe # 6 site was shown to be a liquefied site on the overall summary map, as originally provided by Prof. Tokimatsu. However, conflicting with the map legend, the same site was also listed by Prof. Tokimatsu as a non-liquefied site on the accompanying summary table. When compiling the original Cetin (2000) and Cetin et al. (2004) databases, the site was listed as "Liquefied". After personal communication with Prof. Tokimatsu, the site was updated as a "Non-liquefied" site in the database of Cetin et al. (2016).
4	Kobe No 7	Critical depth to be taken as 3.3 m and representative N value as 8 blows/ft	Unchanged	On the basis of the summary table and the borehole log data provided by Prof. Tokimatsu, the N values corresponding to depth range of 4.3 to 8.3 m was judged to be critical. The SPT blow count at 6.3 m depth was excluded in the averaging scheme by Prof. Tokimatsu, due to its anomalously high N value, which suggested potential gravel interference. However, for this site, Idriss and Boulanger (2010) adopted 3.3 m depth as their critical depth and used the single N value of 8 blows/ft. Note that 3.3 m depth is very close to the reported ground water table depth of 3.16 m. We respectfully disagree with the interpretation of Idriss and Boulanger (2010) on this case. N-values are locally variable, sometimes due to localized variability in soil conditions, and sometimes due to SPT procedures (e.g. variable hammer energy, etc.). Hence when characterizing the critical layer, it is preferred to use the "median", as opposed to the lowest N-value, in the critical layer.
5	Railroad 2	Unit weight of the soil layer is selected very low.	Accept revised unit weights	The soil unit weights for the Rail Road # 2 Site are updated as the same unit weights proposed by Idriss and Boulanger (2010).
6	Panjin Chem. Fertilizer Plant	r_d value is inconsistent with the equation given in Cetin et al. (2004) (Common to cases 3 through 7)	r_d corrected	A typographical error at the third decimal point in one of the model parameters (in Excel execution of the r_d formula), and it is corrected. (Common to cases #3 through #7, in this table)
7	San Juan B-3			
8	Shuang Tai Zi River	Recommended to be used as a non-liquefied case history site as opposed to liquefied.	Case history removed from analysis	The 1975 Haicheng Earthquake, Shuang Tai Zi River Site was correctly classified as a non-liquefaction case history site in the original work of Cetin et al. (please refer to page 319, Appendix 2 of Cetin (2000); “Nature of Failure: Non-liquefied site”), Hence, it was also processed and assessed correctly as a non-liquefied site. Because this site was originally processed and modeled as non-liquefied site, as part of the Cetin (2000) and Cetin et al. (2004) studies, there is no controversy, and no change to be made here. Unfortunately, in the final summary tables, it was listed as “liquefied” due to a typographical error. However, during the compilation of Cetin et al. (2016) database the site is decided to be excluded due to lack of documentation of fines content and Atterberg limit parameters in the original reference. Note that in the Seed et al. (1984) database, the critical layer is described as "silt".

In the initial analysis to test the Idriss and Boulanger (2012) statement regarding these 8 case histories, the authors set aside differences in the interpretation, as summarized in Table 3-1, and modified these cases exactly as proposed. The modified Cetin et al. (2004) database was used to update the liquefaction triggering boundary curves. Maximum likelihood assessment was performed on the intercept term (θ_6 of Cetin, 2000 likelihood function, or θ_5 in Table 2.10 in Chapter 2), which enabled translation (shifting) of the original boundary curves. The updated parameter was estimated as 17.10 as compared to its earlier version of 16.85, and the resulting updated curve is comparatively shown in Figure 3-3 along with the position of these 8 case histories.

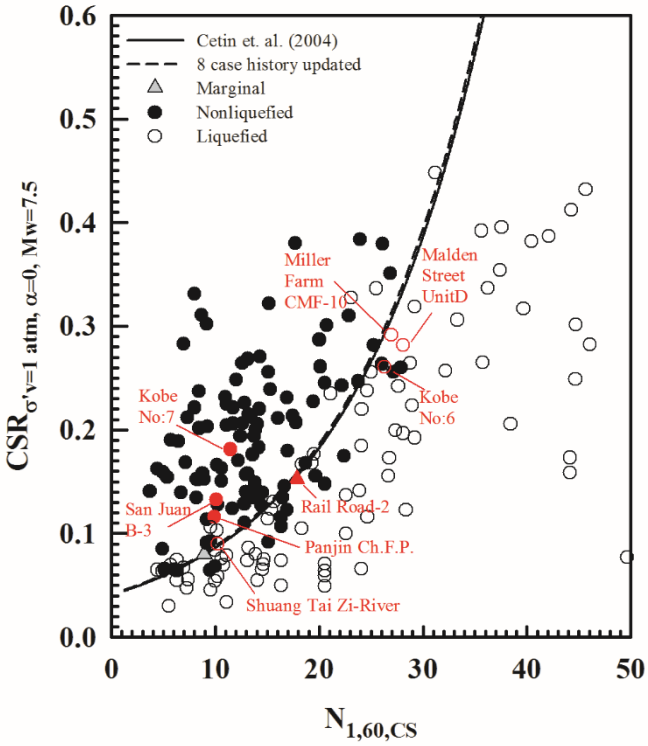


Fig. 3-3. Updated (after modification of 8 case histories exactly as recommended by Idriss and Boulanger, 2012) and original (Cetin et al., 2004) P_L= 50 % CRR boundaries.

The solid curve is the 50 % probability of liquefaction curve of Cetin et al. (2004) and the dashed curve represents the liquefaction triggering curve developed after modifying the eight case histories exactly as recommended by Idriss and Boulanger (2012). As can be seen in Figure 3-3, the hypothesis of Idriss and Boulanger (2012) that "8 of the key case histories control the position of the liquefaction triggering correlation" is not verified. These "8 key case histories" do not control the location of the boundary curves, and even when the authors, for the sake of the analyses, completely accept the Idriss and Boulanger (2012) interpretation of these eight case histories, "the position of the published Cetin et al. liquefaction triggering curve" is essentially unchanged.

This conclusion is not surprising to the authors. Cetin (2000), and Cetin et al. (2002) and Cetin et al (2016) studied the effects of sampling bias on the liquefaction triggering boundary curves through bias-weighting of the case history data. The weighting scheme involved amplifying the number of non-liquefied sites by 20 % ("adding" ~19 (≈ 20 % of 95) nonliquefied case histories) and decreasing the number of liquefied sites also by 20 % ("removing" ~23 (≈ 20 % of 113) liquefied case histories), leading to an overall weighting factor of 1.5. As shown in Figure 3-4, "adding" 19 and "deleting" 23 case history sites through the outlined weighting scheme produced only a modest shift in the liquefaction triggering curves. Hence, it is not surprising that incorporating all of the recommendations of Idriss and Boulanger (2010 and 2012), which only involve eight case histories, resulted in almost no shift in the liquefaction boundary curves. Nevertheless, authors appreciate the suggestions and efforts aimed to improve case history data quality.

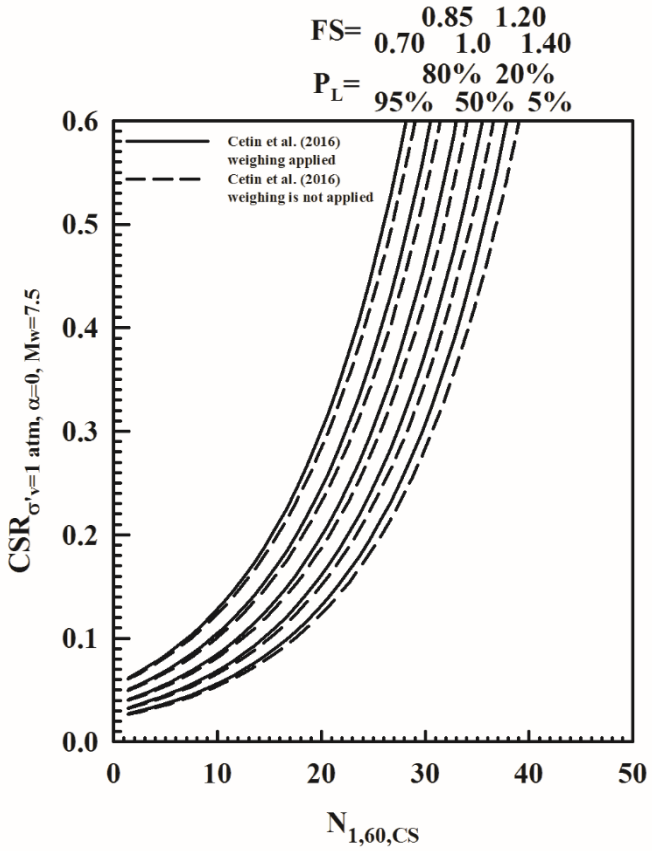


Fig. 3-4. Effects of implementing compensation for sampling disparity (solid lines) vs. leaving sampling disparity unaddressed.

Testing the Idriss and Boulanger (2010 and 2012) hypothesis revealed that the differences in the resulting liquefaction triggering boundary curves are not explained by the differences in the interpretation of a small number of cases. Given 200 case histories of Cetin et al. (2004), disagreements on only eight case history data interpretations should be judged as a major agreement rather than

disagreement. The authors believe that the liquefaction case history database has reached a sufficient level of maturity and quality, that modifications of a limited number of case histories due to different expert opinions will not produce more than minor shifts in liquefaction triggering boundary curves. Therefore, other reasons underlie the differences observed among liquefaction triggering curves, which is discussed next.

3.3. EXAMINATION OF THE SIX MAIN DIFFERENCES BETWEEN THE THREE TRIGGERING RELATIONSHIPS

An abbreviated summary of the six main differences between the three triggering relationships is presented in Table 3-2, and a discussion of these follows. This supplemental table is a useful summary guide to the discussion that follows. Similarly, Figure 3-5 presents a visual cross-comparison of input parameters of case histories present in common in both the Idriss and Boulanger (2010) and Cetin et al. (2016) databases. Table 2-8 of Chapter 2 presents a summary overview of average (mean) values of key parameters and indices, for each of the three databases of Seed et al. (1985), Boulanger and Idriss (2012) and Cetin et al. (2016), and this will also be useful in the discussions that follow.

Table 3-2. Summary of the Six Main Differences between the Three Liquefaction Triggering Relationships Considered

Issue or Input Parameter	Seed et al. (1985)	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comments
1. r_d Relationships and Evaluation of CSR in Back-Analyses of Liquefaction Field Performance Case Histories	CSR based on "simplified" back-analyses using r_d of Seed and Idriss (1971), which was based on one-dimensional site response analyses of sites not representative of the large field case history database. ↑ ○ Bias: Unconservative, because the r_d relationship overestimates CSR for a majority of the field case histories.	CSR based on "simplified" back-analyses using r_d of Idriss (1999), which was based on one-dimensional site response analyses of sites that were again not representative of the large field case history database. ↑ ○ Bias: Unconservative, because the r_d relationship overestimates CSR for a majority of the field case histories.	48 case histories were back-analyzed using site specific and event-specific seismic site response analyses. The remaining 162 case histories were back-analyzed using the r_d relationship of Cetin and Seed (2004) which was developed using 50 of the actual liquefaction field performance case history sites, and which adaptively deals with the ranges of conditions in that case history database. ○ Bias: Unbiased	-
2. K_σ Relationships	Field case history data were not corrected for K_σ effects. ↑ ○ Bias: Unconservative, because a majority of the field case histories occur at shallow depths (σ'_v<1 atm.), and the normalization for K_σ effects would, on average, have caused the normalized field data to plot lower in terms of normalized CSR_{1atm}.	Field case history data were normalized using the K_σ relationship of Idriss and Boulanger (2004, 2008), which was developed based on laboratory cyclic test data. ↑ ○ Bias: Difficult to fully assess. Somewhat conservatively biased, relative to Cetin et al. (2016), but the laboratory-based approach is defensible[1].	Field case history data were normalized using a K_σ relationship that was developed based on probabilistic regression of the full field case history database. ○ Bias: Likely unbiased[1].	[1]The two different approaches taken by (1) Idriss and Boulanger, and (2) Cetin et al. (2016) are both defensible approaches. Cetin et al. (2016) prefer the K_σ relationship regressed from the field performance data, rather than laboratory data because it is actual field performance that engineers must analyze and predict.

Table 3-2. (Contn'd) Summary of the Six Main Differences between the Three Liquefaction Triggering Relationships Considered

Issue or Input Parameter	Seed et al. (1985)	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
3. Truncation of K_σ	The lack of correction of the field case history data for K_σ effects, as discussed in (2.) above, represented an inadvertent truncation of K_σ to not greater than 1.0, nor less than 1.0. ↑○ Bias: As discussed in (2.) above, this produced an unconservative bias.	K_σ was truncated to $K_\sigma \leq 1.1$. This affected K_σ values for 47 of the 230 field performance case histories. This systematic truncation produced a result which was similar to the inadvertent truncation of Seed et al. (1985), but with reduced severity. The resulting bias was still significant. ↑○ Bias: Appears to be unconservative, because the truncation of K_σ increased the normalized CSR_{1atm} values for 47 of the field performance case histories. The most affected cases have $N_{1,60,CS}$ values greater than 20 blows/ft. Hence, if truncation was eliminated, boundary curves are expected to slightly shift down and rotate clockwise.	K_σ was not truncated in back-analyses and development of the triggering relationships. Truncation to $K_\sigma \leq 1.6$ is recommended for forward engineering analyses. Only 5 out of 210 field case histories have $K_\sigma > 1.6$, and if a truncation to $K_\sigma \leq 1.6$ had been employed in triggering correlation development, the resulting overall impact on the regressed triggering correlation would have been negligible. ○ Bias: Unbiased	-
4. Probabilistic Treatment (Triggering Boundary Curves)	N/A	Employed a partial maximum likelihood approach, but did not perform a fully independent probabilistic regression. The regression performed employed only one (single) degree of freedom. As a result, the regressed boundary curves were not well able to adapt themselves to fit the data. There were inconsistencies in handling of probability and uncertainty. ↑○ and ←○ Bias: Unconservative. The problems above led to overestimation of CRR at low values of $N_{1,60,CS} \leq 15$, and to underestimation of variance at all locations, especially at $CSR \geq 0.25$.	The triggering relationship was developed within a Bayesian framework, using the maximum likelihood method. The model regressed had eight degrees of freedom, and was flexibly able to adapt itself to the case history data set. Both model uncertainty and input parameter uncertainties, were modeled and incorporated in these probabilistic regressions. ○ Bias: Unbiased	-

Table 3-2. (Contn'd) Summary of the Six Main Differences between the Three Liquefaction Triggering Relationships Considered

Issue or Input Parameter	Seed et al. (1985)	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
5. Probabilistic Treatment of K_σ, K_{Mw}, and Fines Adjustments	N/A	K_σ, K_{Mw}, and fines adjustments ($\Delta N_{1,60}$) were not formally regressed. Liquefaction field performance case history data were not used in the development of K_σ, and K_{Mw}; instead laboratory test data and engineering judgment were employed. Field performance data and engineering judgment were used in the development of $\Delta N_{1,60}$, but no formal regression was performed.  and Bias: Appears to be unconservative, but difficult to fully quantify the degree based on the documentation (and data) currently available.	Relationships for K_σ, K_{Mw}, and fines adjustments ($\Delta N_{1,60}$) were all developed based on regressions of the overall liquefaction triggering field performance case history database. ○ Bias: Unbiased	Cetin et al. (2016) prefer the relationships based on (1) formal and probabilistic regressions rather than visual judgments, and (2) actual liquefaction field performance case history data rather than laboratory data and inferences from strong motion recordings. The approaches taken by the other two relationships are acceptable methods, but they are less likely to reliably predict actual field behaviors.
6. Transparency	This was the best and the most transparently documented relationship up to that point in time. It was not, however, sufficiently well documented as to provide for ease of fully detailed checking of significant details such as engineering choices and judgments made in individual case history back-analyses, etc. ○ (?) Bias: Lack of transparency likely contributed little to overall bias in the relationship. But it did serve to obscure some of the origins of the relationship, and as a result some of the errors made appear to have been transferred forward (and repeated) in the development of the later relationships.	The triggering relationship has been the subject of multiple research reports and papers providing background documentation. Despite this, 101 of the 230 cases continue to be incompletely documented, and cannot be fully examined and checked. The remainder of the necessary background documentation regarding correlation development has also not been fully transparently provided to date. ○ (?) Bias: Difficult to evaluate, due to incomplete documentation.	The database, and calculations for correlation development were well documented in a transparent manner. As a result, they have been well examined and reviewed in unprecedented detail. It, thus appears that the database, and the analyses, have been well examined and that potentially significant errors, especially systematic errors, have likely been addressed. ○ Bias: Unbiased.	Full and proper transparency should be a requisite for all future triggering correlations, and for other critically important engineering analysis and design methodologies that may potentially affect public safety on a broad scale.

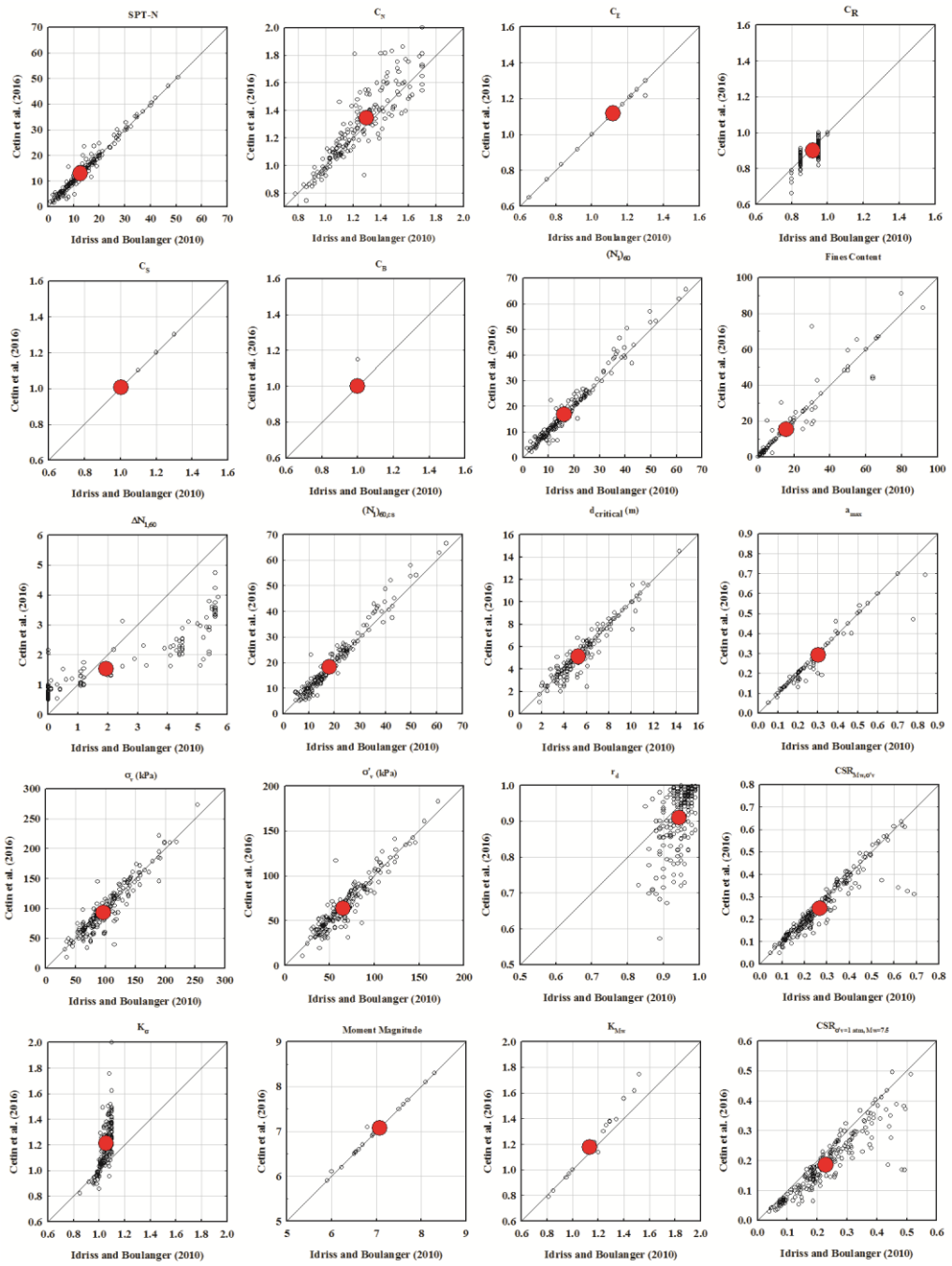


Fig. 3-5. Visual comparison of back-analyzed case history parameters and input parameters for (1) Idriss and Boulanger (2010) and (2) Cetin et al. (2016). Red dots show the mean of the case history input parameters.

3.3.1. Differences in r_d Relationships (1)

Differences in the stress reduction (shear mass participation) factor, or r_d relationships, is the first of the issues listed in Table 3-2. The earlier work of Seed et al. (1984, 1985) employed the r_d relationship developed by Seed and Idriss (1971) in the "simplified" framework, in which CSR in back-

analyses of case histories was evaluated using Equation 2-5 . To implement this simplified approach, it was necessary to develop the first "r_d curves". In early 1970's, the field of seismic site response analysis was in its early stages, and the r_d curves were developed by analyzing a limited number of site profiles. Those profiles consisted of 100 feet (33 m.) of sand, and were not representative of the broad natural ranges of site conditions, and the variable and often layered stratigraphy, present at many of the liquefaction field performance case history sites employed in the development the triggering relationship of Seed et al. (1985).

The resulting non-representativeness of the limited number of sand-only sites was then further compounded because at that early juncture, "input" ground motions were typically of lower intensity than are commonly used today. As also discussed in Chapter 2, and in more detail in Cetin and Seed (2004), applying stronger levels of input motions employed in these analyses to softer sites (e.g. layered sites with softer clays and silts rather than just sands) results in more nonlinearity and tends to reduce r_d values more rapidly with increasing depth. Natural variability in layering or stratigraphy tends to break up coherence of overall site response, and this also tends to reduce r_d values more rapidly with increasing depth.

The r_d curves of Seed and Idriss (1971) are non-representative of the field case history sites and the shaking levels of many of the case histories in the SPT-based soil liquefaction triggering catalog. These r_d values are biased unconservatively with regard to back-analyses of the performance case history databases because they neglect these effects, and therefore they tend to systematically overestimate CSR. That, in turn, causes the field case history data points to be plotted "too high" (vertically) on plots like Figure 3-1 (a), which then unconservatively biases (i.e.: shifts liquefaction triggering boundary curves up) the resulting triggering relationship.

The late Prof. H. Bolton Seed was engaged in a research effort to study this at the time of his unfortunate death, and his last Ph.D. student, Dr. Ramin Golesorkhi, performed one-dimensional site response analyses using both the equivalent linear approach and also fully nonlinear models, to evaluate these r_d effects and also to evaluate the ranges over which the equivalent linear analyses would be an adequately reliable proxy for nonlinear response behaviors for a variety of other applications as well. Dr. Golesorkhi applied his r_d-related response analyses mainly to monolithic sites again comprised entirely of sand strata. Figure 3-6 (a) shows the r_d curves developed by Golesorkhi (1989) for his analyses of sand-only sites, re-scaled to the same vertical and horizontal axes as the other site response analysis results shown in Figures 3-6 (b), (c) and (d) for direct comparisons.

In the early 1980's, a group of Japanese researchers had also advanced the assessment of r_d, and they performed a total of 143 one-dimensional site response analyses by using 5 different input motions (with maximum horizontal accelerations varying in the range of 0.052 to 0.233 g), for a suite of layered soil sites (with actual stratigraphy) to investigate this issue (Imai et al., 1981). Based on

multiple reflection theory, they performed one-dimensional seismic site response analyses for these sites, with real stratigraphy, and their results are presented in Figure 3-6 (b), again scaled to matching axes.

Cetin and Seed (2004) performed a significantly larger number of site response analyses, using the one-dimensional dynamic response software SHAKE91 (Idriss and Sun, 1993) to approximate nonlinear response effects, and they applied these analyses to real sites with real stratigraphy as also described in the Chapter 2. The results of these 2,153 site response analyses are presented in Figure 3-6 (c). These results differed from the other three sets of r_d curves presented in that (1) the 50 sites analyzed were all actual sites from the larger liquefaction field performance case history database, so that they had variable site conditions and stratigraphy and were "representative" of the field case history database, and (2) a suite of 42 input motions was applied to 50 of these sites, and these input motions were carefully selected to range from low to high intensities of shaking, for low to high magnitude causative events, and in each magnitude range the actual strong motions recordings employed were selected to include motions representative of near-field, mid-field and far-field events. In addition, for 53 liquefaction field case history sites where a nearby strong motion recording was available that could be scaled for use as an "input" motion, actual site-specific and earthquake event-specific site response analyses were performed. Regressions were then performed to develop r_d relationships based on suitably representative site conditions wherein the r_d values are a function of: (1) site stiffness and stratigraphy, (2) levels and intensity of shaking, (3) causative earthquake magnitude (as a proxy for duration effects), and (4) depth. This was described in a bit more detail in Chapter 2.

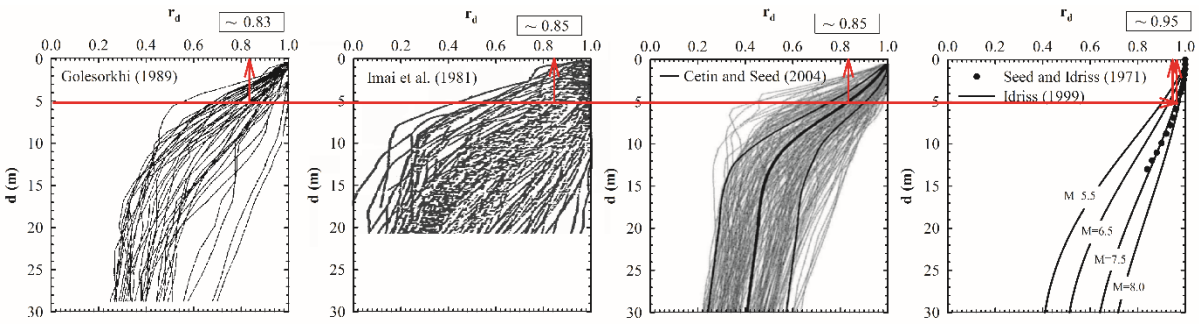


Fig. 3-6. Plots of r_d values calculated based on seismic site response analyses by (a) Golesorkhi (1989), (b) Imai et al. (1981), (c) Cetin and Seed (2004), and (d) Seed and Idriss (1971) and Idriss (1999). The red line and red arrows show the r_d values estimated at the approximate median critical depth of the databases of and Boulanger and Idriss (2012) and of Cetin et al. (2016).

The dark dots in Figure 3-6 (d) show the middle values of the early recommended r_d curve of Seed and Idriss (1971), which was not magnitude dependent. This early curve was developed from stiff monolithic (sand) sites lacking layering or stratigraphy, and lower levels of shaking, and the associated range is narrow and does not span as broadly as the other three suites of analysis results. The differences are significant at the relatively shallow depths of principal interest for back-analyses of the liquefaction field performance case histories. As shown in Table 2-8, the mean depth of critical strata in the large liquefaction field performance case history databases tends to be on the order of approximately 5 meters.

The suite of four solid lines in Figure 3-6 (d) show the r_d curves developed and recommended by Idriss (1999). In early presentations, Prof. Idriss stated that these had been based on Golesorkhi (1989), with some additional analyses. Later, a U.C. Davis research report (Idriss and Boulanger, 2010) presented the development of these r_d curves attributed to Idriss (1999) based on performing one-dimensional equivalent linear seismic site response analyses, in which suites of input motions of three differing magnitudes (M_w = 5.5, 6.5 and 7.5) and varying intensities were applied as input motions to the bases of six soil profiles, to produce a suite of 512 seismic site response analysis results. Five of these six sites were again monolithically comprised of 100 feet (33 m.) of sand, underlain by “rock”. The sixth site was a modification of the USGS strong motion recording site at La Cienega in Los Angeles, California. This is a site with actual stratigraphy, but the V_s profile at this site was "modified". The five sand-only sites are non-representative of the liquefaction case history database sites, and they again produce “overly stiff” r_d behaviors. The modified sixth site produces even “stiffer” r_d behavior (r_d values farther to the right). Additionally, the “representative” r_d curves developed based on these analyses of six sites were not selected at the mean or median values, but rather at the 65th-percentile values; further biasing the resulting r_d curves farther to the right and not complying with the use of mean input parameters within the maximum likelihood framework employed to develop a liquefaction triggering relationship. Using higher r_d values results in shifting case history data points vertically upwards in the $N_{1,60}$ vs CSR domain, and hence produces unconservatively biased liquefaction triggering curves contrary to the original intent of being conservative.

The differences between the various r_d relationships shown in Figure 3-6 are significant at the relatively shallow depths of principal interest for back-analyses of the liquefaction field performance case histories. The mean depth of critical strata in the large liquefaction field performance case history databases tends to be on the order of approximately 5 meters.

As shown in Figure 3-6, these r_d curves of Idriss (1999) are not in good overall agreement with the r_d curves developed by the other three research groups, and they produce r_d values that are biased to the high side at all depths.

An important pair of postulates affecting this and other elements of back-analyses of liquefaction performance field case histories for purposes of developing liquefaction triggering relationships are:

Postulate 1: It is often true that decisions and approximations that would be "conservative" in forward engineering analyses of actual projects (e.g. over-estimation of r_d, and of resulting CSR values), are instead un-conservative when applied to the back-analyses of field performance case histories for purposes of developing triggering relationships.

As an example here, the over-estimation of r_d (and of CSR) that resulted from using r_d curves developed for the non-representative and overly "stiff" site conditions would have been conservative in a forward analysis for an actual engineering project. But in back-analyses of liquefaction performance field case histories, the overestimation of r_d (and of resulting CSR's) serves instead to displace the plotted case histories vertically upwards on the eventual "boundary curve" plots (e.g. Figures 3-1(a), (b) and (c)), producing an unconservative bias in the triggering relationships developed based on these data. This same principal applies to multiple other parameters and relationships, not just r_d and CSR, and so it will be discussed further throughout this report.

Postulate 2: Engineers are often taught that it is important to "repeat the same mistakes going forward (executing engineering analyses of real projects) as were made going backwards (e.g.: back-analyzing field case histories for development of triggering relationships)". But this does not guarantee that errors or bias in the back-analyses will be suitably mitigated in forward engineering analyses for a specific site or engineering project.

The first postulate is clear and self-explanatory. The second one is a bit more subtle. Unconservative bias in back-analyses is not necessarily mitigated (or counter-balanced) by performing forward analyses using identical protocols. The overall bias from inappropriate back-analyses is largely "averaged" into the relationships developed, but forward analyses will be applied to a specific project site and conditions rather than to an "average" site with average conditions. It is true that some counter-balancing occurs if the "simplified" (e.g. the r_d-based) approach is used to evaluate CSR's in situ for forward analyses of an actual project, and if the same r_d curves are employed as were used for the back-analyses upon which the triggering curves are based. But the degree of counter-balancing is variable, and dependent upon the juxtaposition of actual site conditions vs. the "average" conditions in the liquefaction field performance database. More importantly, for sites and projects of sufficient importance, or sufficient challenge and complexity, that site-specific seismic site response analyses (or even site-specific site and soil-structure interaction seismic response analyses) are warranted, in-situ CSR's are then directly (and correctly) calculated in these more detailed site-specific and project-specific forward analyses (employing higher-order seismic response analyses to directly calculate CSR at each point of interest, rather than employing the "simplified" r_d curves approach), and thus no

counter-balancing (or compensating) error occurs in the forward analyses. This, in turn, means that the unconservative bias associated with the use of non-representative r_d curves in back-analyses and formulation of the triggering relationship is then not at all compensated for in the types of large, challenging, or complicated projects that can often be among those of greatest importance (e.g. large dams, major buildings, significant infrastructure, challenging site conditions, etc.)

3.3.2. Differences in K_σ Relationships (2)

The second issue addressed in Table 3-2 is the development (or selection) of K_σ relationships for normalization of CSR's at arbitrary effective overburden stresses to "equivalent" CSR's normalized to the equivalent CSR that would just trigger liquefaction at an effective vertical stress of σ'_v = 1 atm. This is accomplished using Equation 3-1, where K_σ is a function of effective vertical stress, and sometimes also a function of relative density (e.g. $N_{1,60,CS}$).

$$CSR_{\sigma'_v=1atm.,\alpha,M_w} = \frac{CSR_{\sigma'_v,\alpha,M_w}}{K_\sigma} = \frac{\tau_{hv,cyclic,average}}{\sigma'_v}\frac{1}{K_\sigma} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_v}{\sigma'_v} \cdot r_d \cdot \frac{1}{K_\sigma} \qquad \textbf{Eqn. (3-1)}$$

The early triggering relationships (e.g. Seed, 1976, etc.) were not normalized for vertical effective overburden stress, because it was tacitly recognized that they were based on liquefaction field performance case histories for "shallow" soil strata; as discussed previously. The relationship of Seed et al. (1985) was the latest of these early relationships, and it was also not normalized for K_σ effects. But K_σ effects had suddenly become important in the mid-1970's and the early 1980's because triggering relationships were increasingly being applied to analyses of major earth dams (with potentially liquefiable strata under high effective overburden stresses). Laboratory research and principles of critical state soil mechanics show that soils of a given density or normalized SPT penetration resistance would be more likely to liquefy at the same CSR if they were at higher initial effective overburden stress, because the higher effective stress would suppress cyclic dilation and increase cyclic compressions on shear stress reversal during cyclic loading.

Accordingly, research efforts were undertaken to develop K_σ relationships based on laboratory undrained cyclic testing. By the mid-1980's, (1) a body of laboratory test data were available, and K_σ relationships were being proposed by different sets of experts, and (2) it was becoming increasingly common practice to assume that the "shallow" liquefaction triggering relationships developed based on field performance case history data were suitably representative for conditions conforming to $\sigma'_v \leq$ 1 atm., and then K_σ relationships would be applied to these triggering "curves" to extrapolate in order to develop evaluations of liquefaction resistance (e.g. CRR) at higher initial effective stresses (greater depths) where $\sigma'_v >$ 1 atm.

The NCEER Working Group (Youd et al. 2001) proposed a K_σ relationship of this type for application to the triggering relationship of Seed et al. (1985). This K_σ relationship was based on cyclic undrained laboratory test data, and K_σ was a function of both σ'_v and also $N_{1,60,CS}$. That was a suitable approach based on the state of knowledge at that time. This K_σ relationship is shown in Figure 3-7.

Boulanger (2003), Boulanger and Idriss (2004) and Idriss and Boulanger (2004, 2008, 2010) produced less conservative K_σ relationships for extrapolating triggering curves that had been developed based on shallow data to higher overburden stresses. These K_σ relationships were again based on interpretations of available laboratory cyclic test data. These relationships provide for a lesser rate of decrease in liquefaction triggering resistance (CRR) with increasing σ'_v. This is also a valid approach, and their K_σ relationship is a function of both σ'_v and also $N_{1,60,CS}$.

Cetin et al. (2004, and 2016-a) recognized the difficulties and uncertainties involved in extrapolating the results of (1) undrained cyclic triaxial tests, and (2) of undrained cyclic simple shear tests with uni-directional horizontal loading, both with uniform cycles, to evaluation of the fully-multi-directional (in a horizontal plane) and non-uniform cyclic shear loadings of principal interest for prediction of field behavior in real earthquakes. Accordingly, they preferred to develop a K_σ relationship as part of the overall probabilistic regressions of the full liquefaction triggering field performance case history database, which intrinsically embodies these three-dimensional and fully multi-directional and irregular real world seismic cyclic loadings. This had not previously been accomplished, due to the difficulties involved in extracting multiple model fitting parameters from a very large and highly complex field performance case history data set while simultaneously dealing with multiple sources of both model and input parameter uncertainties.

The resulting K_σ relationship of Cetin et al. (2016) is a function of σ'_v only, and it is shown by the solid red line in Figure 3-7. An additional variable was regressed, as part of the overall regression of the field case history data, to investigate whether an effect of $N_{1,60,CS}$ could be discerned from the large field case history database. It was found that a very slight influence of $N_{1,60,CS}$ on K_σ could be discerned, but it was in a direction inverse to the effect of $N_{1,60}$ apparent in the laboratory test data sets employed by Youd et al. (2001) and by Boulanger and Idriss (2014). That raised two possibilities: either the field relationship was subtle enough that it was partially masked by the overall variances in the complex field case history data set, or something else was going on.

Cetin and Bilge (2013) performed a large number of undrained cyclic simple shear tests to investigate K_σ, and discovered that K_σ is strongly a function of the cyclic shear strain level at which "triggering" of liquefaction or cyclic mobility is deemed to have occurred. At conventional levels of laboratory cyclic shear strain, commonly employed as "triggering" criteria, and if the same levels of "triggering strain" are employed for all tests (regardless of density or $N_{1,60,CS}$), then the K_σ curves of Youd et al. (2001), Boulanger and Idriss (2012) and Cetin et al. (2016), appear generally reasonable in

form. But the field case history data likely represent differing levels of cyclic strains, as denser strata (with higher $N_{1,60,CS}$ values) typically exhibit more limited cyclic shear strain potential. It could then be inferred, based on the laboratory data set of Cetin and Bilge (2013) that a slightly "inverted" relationship between K_σ and $N_{1,60,CS}$ might be expected; with the effect that increasing $N_{1,60,CS}$ values affects K_σ in a manner slightly inverse to the trends posited by Youd et al. (2001) and others, as was actually observed in the regression of the liquefaction triggering field case history database. There would be significant uncertainty associated with this, however, and so pending further additional investigation by other researchers, it was decided in developing the triggering relationship of Cetin et al. (2016) to take a middle position and employ a K_σ relationship (1) that is a function of σ'_v only, (2) that is regressed from the overall field performance case history database, and (3) that is not necessarily recommended to be used outside the vertical effective stress range of the field performance case history database. As discussed in Chapter 2, the K_σ relationship of Cetin et al. (2016) is employed internally within the development of the triggering relationship, and it is used mainly to correctly "center" (or normalize) the relationship to a reference effective overburden stress of σ'_v = 1 atm. Extrapolation of the resulting normalized triggering relationship can then be accomplished by means of any of a number of other proposed K_σ relationships. This will be discussed further.

Figure 3-7 shows the K_σ relationships of (1) the NCEER Working Group (Youd et al., 2001) as applied after the fact to the triggering relationship developed by Seed et al. (1985). The histogram shown in the lower part of Figure 3-7 shows the distribution of effective vertical stresses in the field case history database of Cetin et al. (2016), which is for these purposes very similar to that of Idriss and Boulanger (2010, 2012). The Cetin et al. (2004 and 2016) regressed K_σ relationships based on the observed field case history performance agree reasonably well with the relationship proposed by Youd et al. (2001) and with available cyclic laboratory test results. The K_σ relationship proposed by Boulanger and Idriss (2012) shown in Figure 4 does not employ field case history data. Instead, it is based on the assessment of undrained cyclic laboratory test data and engineering judgment. For the purpose of enabling a direct visual comparison, available cyclic laboratory test results as compiled and interpreted by Montgomery et al. (2012), which served as the principal basis of the Boulanger and Idriss (2012) K_σ relationship, are also shown in Figure 3-7, along with additional data developed by Cetin and Bilge (2013) based on undrained cyclic simple shear testing. The data set gathered and compiled by Montgomery et al. was based on both cyclic triaxial and cyclic simple shear testing, with a majority of the data developed by cyclic triaxial testing.

There are significant uncertainties with regard to interpretation of cyclic triaxial test data for purposes of development of liquefaction triggering K_σ relationships for forward application to field conditions that will involve both non-uniform and three-dimensionally, randomly directionally varying in plan view (3-D) cyclic simple shear loadings. There are lesser levels of uncertainty in employing the

more limited one-directional uniform cyclic simple shear test data that make up part of the data set compiled by Montgomery and all of the data developed by Cetin and Bilge, but these uncertainties are not negligible. Accordingly, it is posited that the best K_σ relationship for use in development of the types of liquefactions triggering curves considered here are K_σ relationships regressed from the actual field performance case history databases.

These would only be demonstrably superior, however, over the ranges of the field case history data upon which they are based. Accordingly, it is posited that the K_σ relationship of Cetin et al. (2016), as presented in Chapter 2, would be applicable to a range of initial effective vertical overburden stresses of approximately 0.25 atm.$\leq \sigma'_v \leq$ 1.8 atm.; which is the principal range of the field case history database. There is at present no way to be certain to what degree this K_σ relationship will then usefully continue on with the same equational form, to significantly higher levels of σ'_v. As a result, good arguments can then be made that after a given liquefaction triggering relationship has been largely correctly "centered" (normalized) to σ'_v = 1 atm., any of a number of proposed K_σ relationships might then be used for extrapolation of the triggering relationships to significantly higher effective overburden stresses.

Examining the data summarized in Figure 3-7, it can be noted that on the basis of both field case histories and cyclic laboratory test data, the Boulanger and Idriss (2012) K_σ relationship is the least conservative of the relationships shown in Figure 4 for extrapolation to high effective stresses (i) when extrapolating to greater depths (higher σ'_v) for forward assessments, and (ii) when applied to back-analyses of field performance case histories at shallow depths (which occur mainly at low σ'_v) for purposes of developing a liquefaction triggering relationship, based on the inverse rule of Postulates No's. 1 and 2, as previously discussed. That does not mean that there is no support for their K_σ relationship in the laboratory data presented. Based on laboratory data only, there is some support for the K_σ relationship of Boulanger and Idriss (2012), but as shown in Figure 3-7 there are also laboratory data that suggest that this relationship is unconservatively biased. The K_σ relationship of Boulanger and Idriss is unconservatively biased relative to the overall laboratory database compiled and presented by Montgomery et al. (2012), and it is further biased if one then adds the multiple sets of cyclic simple shear test data by Cetin and Bilge (2013) interpreted based on a liquefaction "triggering" criterion of 6% double amplitude cyclic shear strain (+/- 3 % cyclic shear strain), which also fall well below the Boulanger and Idriss (2012) K_σ curves. The data from Cetin and Bilge (2013) agree much more closely (1) with the less aggressive (more conservative) K_σ curves of Youd et al. (2001), and (2) with the regression of the field case history data by Cetin et al. (2016) as well.

Because the triggering relationship of Cetin et al. (2016) was normalized (or "centered") in an unbiased manner to represent conditions at σ'_v = 1 atmosphere, based on K_σ regressed specifically for the field case history database, in principal any K_σ curve or relationship could then be employed in

conjunction with this relationship for forward engineering project analyses. The triggering curves of Seed et al. (1985) are not normalized, and so there is an initial (and unconservative) bias there. The triggering relationship of Boulanger and Idriss (2012) was normalized with a K_σ relationship shown in Figure 3-7, which is clearly unconservative for back-analyses of case histories with $\sigma'_v < 1$ atm. (and for development of triggering relationships based on those back-analyses) relative to both the laboratory data presented and also (and more importantly) the two regressesed K_σ relationship based on the large body of field case hstory data. For forward engineering analyses of situations with $\sigma'_v > 1$ atmosphere, this initial unconservative bias of the triggering relationship would appear to be further exacerbated due to apparent unconservative bias of their proposed K_σ relationship, relative both the overall laboratory data and also the K_σ relationships regressed based on the large set of field case history data.

In principal, it can be argued that there is a significant degree of leeway in judgments that can be made for extrapolating K_σ relationships to values of σ'_v significantly greater than 1 atm., but there are two caveats: (1) extrapolations should not be conclusively "ruled out" by the field case history data-based K_σ relationships, as the large field data set is arguably the best and most pertinent data available here, and (2) extrapolations should also not be unconservative relative to the overall body of laboratory data shown in Figure 3-7. The K_σ relationship of Boulanger and Idriss (2012) appears to fail to satisfy both of these constraints.

The K_σ curves of Youd et al. (2001) were developed by a large group of many of the world's top liquefaction experts, and it is suggested that engineers might consider this overall situation (and Figure 3-7) and then might judge that the Youd et al. (2001) curves appear to continue to be a reasonable and defensible basis for forward analyses for very high σ'_v. It is the view of the authors of this report that more research is needed here, and that in the face of current uncertainty it would be good to avoid potentially serious unconservatism; especially at very high effective stresses, as those high stress levels often occur in conjunction with major dams or other critical and/or potentially high risk structures.

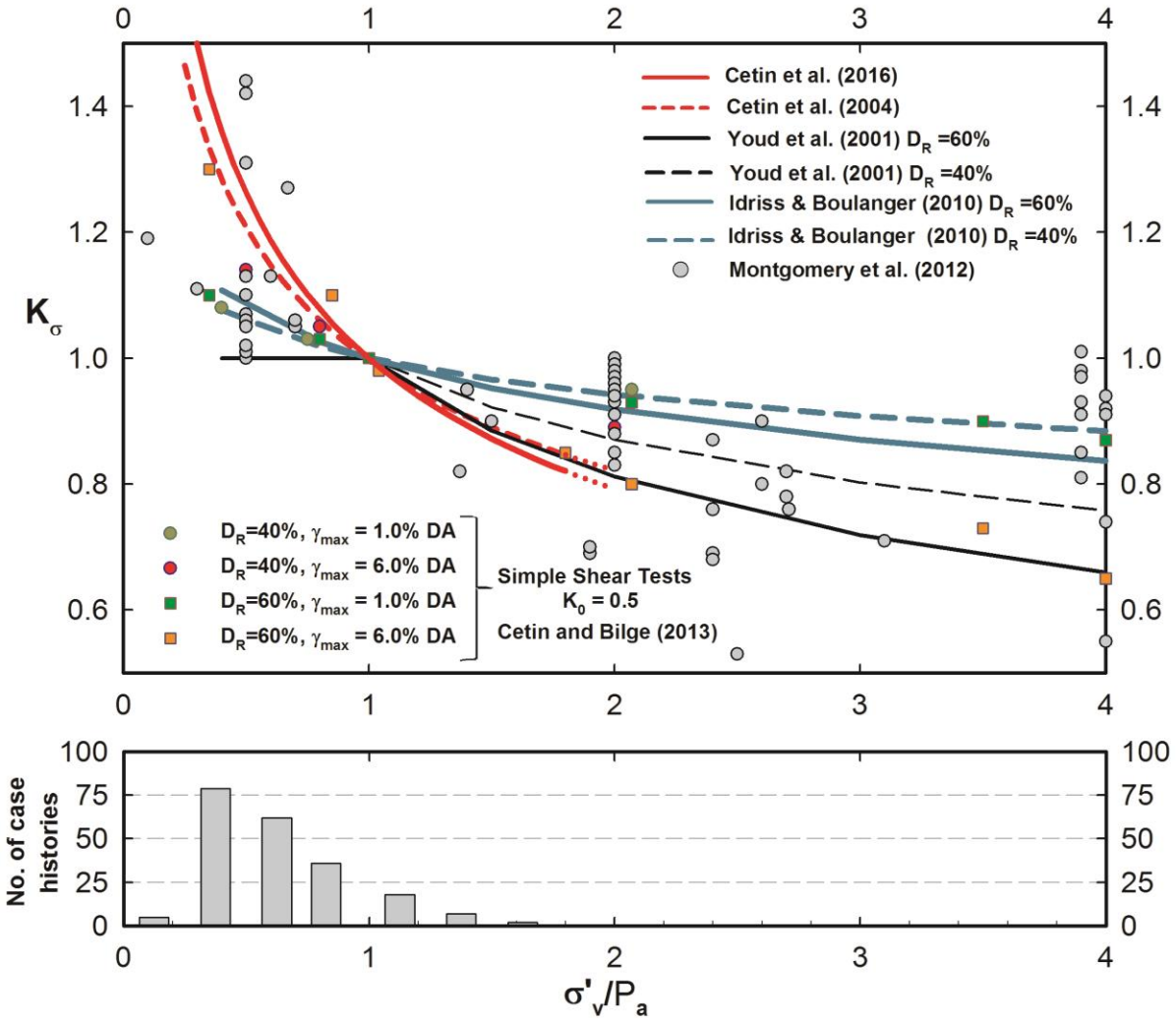


Fig. 3-7. The recommended K_σ relationships of (1) Youd et al. (2001) as appended to the triggering relationship of Seed et al. (1985), (2) Boulanger and Idriss (2012), and (3) Cetin et al. (2016).

3.3.3. Differences in Truncations of K_σ Relationships (3)

Truncation of K_σ is the third issue addressed in Table 3-2. As discussed earlier, it had often generally been assumed that the earlier "shallow" liquefaction triggering relationships (e.g. Seed et al., 1985) were appropriate to σ'_v = 1 atm., and as a result it had become somewhat standard practice to assume that those triggering curves were representative for $\sigma'_v \leq 1$ atm. But the field case history database of Seed et al. (1985) was actually more closely correlated with an overall average field case history stress level of σ'_v = 0.67 atm. Assuming that it was 1 atm., and then appending a K_σ relationship from 1 atm. to progressively higher values of σ'_v had approximately the equivalent effect of "truncating" K_σ to $K_\sigma \leq 1.0$, as illustrated in Figure 3-7 for the relationship recommended by Youd et al. (2001). Based again on Postulates 1 and 2, the truncation of K_σ would be conservative for forward analyses of actual projects with low effective stresses, but for back-analyses of liquefaction field case

histories and for subsequent development of liquefaction triggering relationships, this type of truncation creates a significant unconservative bias in the resulting triggering relationships produced. Truncating at $K_\sigma \leq 1.0$ has the effect of increasing the overall average "normalized" CSR values back-calculated from the case histories, because (1) a majority of those cases had $\sigma'_v < 1$ atm., and thus $K_\sigma > 1.0$, and (2) in back-analyses the back-calculated CSR is multiplied by $1/K_\sigma$ before plotting the results in the triggering boundary curve plots of Figure 3-1. That, in turn, means that truncation of K_σ biases the triggering boundary curves unconservatively by pushing CSR values vertically upwards in plots like those of Figures 3-1 (a) through (c).

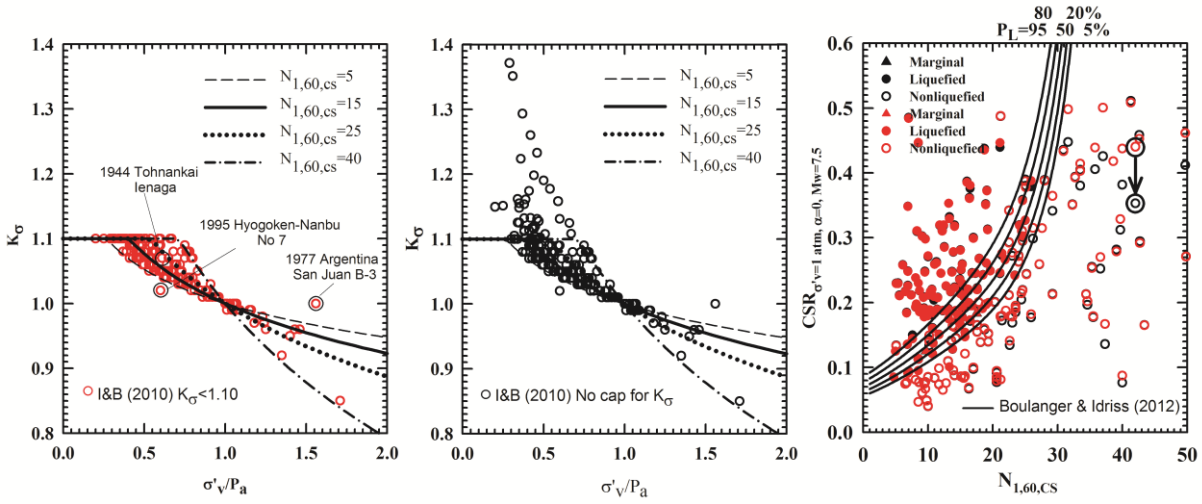
Cetin et al. (2004) addressed this, and to mitigate this truncation error, K_σ values used for back analyses of case histories and subsequent development of their triggering relationship were not truncated. However, a limit of $K_\sigma \leq 1.5$ was then recommended at very shallow depths for forward engineering assessments. If a $K_\sigma \leq 1.5$ limit had been employed in the processing of the the case histories, only 7 of the 200 field performance case histories would have been affected, and the overall impact on the triggering relationship would have been negligible. Similarly, there would have been little to no significant effect on the triggering relationship developed by Seed et al (1985) if truncation to a maximum value of 1.5 had been employed.

Cetin et al. (2016) specifically did not apply a truncation of K_σ in processing the case history back analyses. However, for forward (design) assessments, it is recommended that K_σ to be limited (truncated) to $K_\sigma \leq 1.6$. This affects only soils at very shallow depths, and very low effective overburden stresses ($\sigma'_v < 0.25$ atm.). Only 5 of the 210 field performance case histories back-analyzed by Cetin et al. (2016) would have been affected by this $K_\sigma \leq 1.6$ truncation if it had been employed in the development of the triggering relationship, but it was not. Because so few cases would have been affected, and to relatively minor degrees, overall bias of the resulting triggering relationship would have been negligible. Based again on Postulates 1 and 2, for forward engineering evaluations truncation to $K_\sigma \leq 1.6$ would then be conservative for some forward engineering assessments (not for back analyses of case histories) at very shallow depths. Such shallow depths (low effective overburden stresses) are seldom of significant engineering concern, and some minor conservatism here (1) is unlikely to significantly affect engineering projects, and (2) is warranted by the uncertainties associated with K_σ relationships at very low effective stresses.

Idriss and Boulanger (2006) elected to truncate their K_σ relationship at $K_\sigma \leq 1.0$, and later Idriss and Boulanger (2008) truncated at $K_\sigma \leq 1.1$. It was stated that truncation affected only a few cases, and that it had little impact on their overall correlations developed. Figure 3-8 (a) shows the K_σ values of the Idriss and Boulanger (2010) database, estimated by their K_σ relationship, and truncated at $K_\sigma \leq 1.1$ for use in development of their resulting triggering relationship. The lines in the background show their K_σ relationship, which is a function of $N_{1,60,CS}$. Figure 3-8 (b) again shows K_σ values calculated based on

their K_σ relationship, but this time without applying the truncation at $K_\sigma \leq 1.1$. As shown, 47 of their 230 cases are affected (and unconservatively biased) due to this truncation. Without truncation these 47 case history points would have produced normalized CSR values that would have plotted lower on the CSR vs. $N_{1,60,CS}$ triggering curve plots as shown in Figure 5(c), which illustrates the re-location of just one of the case histories (vertically downwards) by not truncating K_σ. Examining their overall field case history database, it is observed that K_σ truncation effects appear to be more pronounced for $N_{1,60,CS}$ > 20 blows/ft, and the resulting triggering curves would thus likely be somewhat more affected (unconservatively biased) at this larger range of $N_{1,60,CS}$ values due to this truncation.

There is no physical reason or basis for truncation of K_σ in back-analyses and development of triggering relationships, and significant truncation is simply a straightforward imposed constraint (or bias) that produces unconservative bias in the resulting triggering relationships. Both Postulates 1 and 2 apply here, and either (1) not correcting for K_σ effects (failing to "center" or normalize the triggering relationship correctly at σ'_v = 1 atm.) as is commonly done when extrapolating the "shallow" relationship of Seed et al. (1985) to higher levels of effective overburden stress, or (2) truncating at either $K_\sigma \leq 1.0$ or $K_\sigma \leq 1.1$, as was done in the development of the triggering relationships of Idriss and Boulanger (2006) and of Boulanger and Idriss (2012), respectively, are sources of unconservative bias in the liquefaction triggering relationships of Seed et al. (1985) and Boulanger and Idriss (2012).



(a) **(b)** **(c)**

Fig. 3-8. The K_σ relationship recommended by Boulanger and Idriss (2012), with their field case history data points plotted showing the effects of (a) truncation to $K_\sigma \leq 1.1$ (Idriss and Boulanger 2008), (b) no truncation of K_σ, and (c) an example of the the shift in CSR for one case history in the eventual liquefaction triggering relationship of Boulanger and Idriss (2102) if K_σ is not truncated.

3.3.4. Differences in Probabilistic Treatment Used in the Development of the Triggering Relationships (4)

Probabilistic treatment in the development of the triggering relationships is the fourth main difference between the three relationships addressed in Table 3-2. Seed et al. (1985) did not perform formal probabilistic regressions. Instead they plotted the data in several "bins", separating these based on fines contents, and then hand-drew three sets of boundary curves for data with (a) FC ≤ 5%, (b) 5% < FC < 35%, and (c) FC ≥ 35% based on engineering judgment. As discussed previously, Prof. H. Bolton Seed had targeted these three boundary curves at approximately a 10% to 20% probability of liquefaction, but he recognized that (1) the sparseness of the available field performance case history data, and (2) the lack of a formal probabilistic regression, raised questions as to the degree to which this target was actually met.

Boulanger and Idriss (2012) state that their liquefaction triggering boundary curves are based on regressions performed by the maximum likelihood method. The formal application of the maximum likelihood method would require at least the following two attributes to be incorporated in the required assessments and analyses: (1) the regressions would have to be performed using functional shapes (or equational forms) providing both (a) suitable general characteristics and also (b) sufficient (regressable) degrees of freedom so that the overall relationship could suitably adapt its shape and position to conform to the dictates of the data, and (2) the regressions would have to employ correct and appropriate treatment of both (a) model error or uncertainty, and (b) input parameter uncertainty.

As shown in Figure 3-1 (c), the functional shape employed for the development of their "boundary curves" is an equation that has a "bent" shape much like an Australian boomerang, with somewhat more curvature near the middle and less at the two ends. This selected functional shape (equational form) has only one single regressable degree of freedom (model coefficient) C_0. As a result, based on regression, this shape can only translate in one single direction (vertically), but it cannot independently translate in two orthogonal directions (e.g. laterally and vertically), and it cannot independently rotate or adjust its bend or curvature. This lack of sufficient degrees of freedom represents a "stiff" regression, in which the resulting regressed relationship cannot well adapt itself to conform to the dictates of the database. This violates the first of the two requirements of a maximum likelihood regression as set forth above.

A second issue is that the input parameter uncertainty of each case history needs to be consistently incorporated, and then both input parameter uncertainties and overall model uncertainty need to be suitably handled. The performance of a maximum likelihood regression is governed by the need to correctly model both (a) model error, ϵ, and (b) input parameter variance or uncertainty of

individual case histories (i.e.: uncertainty in $N_{1,60,i}, CSR_{\sigma'_v,\alpha,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i}$ of the i[th] case history, represented by standard deviations designated as $\sigma_{N_{1,60},i}$, $\sigma_{\ln(CSR_{\sigma'_v,\alpha,M_w}),i}$, $\sigma_{\ln(M_w),i}$, $\sigma_{FC,i}$ and $\sigma_{\ln(\sigma'_v),i}$ etc.). Neither the model error nor the variance or uncertainty of individual case history input parameters were fully and systematically modeled within the maximum likelihood regressions performed by Boulanger and Idriss (2012).

Boulanger and Idriss (2012) did not develop fully independent estimates of either model variance or of input parameter uncertainty. Instead, the mean estimates of (i) model error or variance, and (ii) individual input parameter variances as estimated by Cetin et al. (2004) along with (iii) additional (not formally quantified) judgment based primarily on the variance terms of selected seismological strong ground motion prediction equations' variance terms, were used to assume the standard deviation of model error value of 0.13 that was employed in Boulanger and Idriss (2012), despite (a) differences between the field performance case histories selected for inclusion in the two databases, and (b) multiple differences in the processing and back-analyses of the case histories between the two investigation teams. As presented and discussed in Cetin (2000), the model coefficients of liquefaction triggering relationships are also inter-correlated; hence assuming a value for one model coefficient (e.g.: assuming standard deviation of model error, σ_ϵ as 0.13) inevitably constrains the other model coefficient (e.g.: C_0 in Boulanger and Idriss), even if an attempt is made to independently (but actually conditionally) regress them due to intercorrelation of these coefficients. Assuming a value for the standard deviation of model error rather than probabilistically assessing it is essentially equivalent to judgmentally (deterministically) deciding about the relative positions of equi-probability liquefaction triggering curves relative to the P_L=50 % curve, rather than properly regressing them, and this is not acceptable within a probabilistic assessment framework.

Cetin et al. (2016), in contrast, repeated the same level of effort that had previously been devoted to the estimation of individual input parameter error (or uncertainties) in processing and back-analyses of the field performance case histories by Cetin et al. (2004). The results differed slightly from the 2004 estimates due primarily to (1) differences in the case histories included in the database, and (2) differences in some of the details involved the back-analyses of the case histories. As discussed in Chapter 2 & 4, it was then necessary to appropriately distribute the overall uncertainty between input parameter uncertainty and model error. An element of the solution here was to extend the regression (still by the maximum likelihood method) to include determination of the most appropriate (maximum likely best fit) distribution between input parameter uncertainty and model error. This was a challenging analytical effort. In the end, a total of seven model fitting coefficients (θ_1 to θ_6 and σ_ϵ) were regressed in a manner similar to that employed by Cetin et al. (2004), and an eighth model

coefficient (θ_7), addressing the relative distributions of input parameter uncertainty and model error was also (simultaneously) regressed in the same overall maximum likelihood regression as discussed in the Chapter 2 & 4.

The failure to independently assess, and to suitably account for, the uncertainties associated with (i) model error or variance, and (ii) individual input parameter variances, and the failure to employ sufficient regressible degrees of freedom in the maximum likelihood regression, biased the regressions performed by Boulanger and Idriss (2012). This appears to be further exacerbated by equational constraints on the distribution of model error, which is represented graphically by the "spread" between the probability of liquefaction contours shown in Figure 3-1 (b).

As shown in Figures 3-1 (b) and 3-1 (c) the resulting shapes of the median (P_L = 50%) boundary curves for the two regressed relationships differ significantly, and so do the associated contours of P_L = 5%, 20%, 50%, 80% and 95% for the two relationships. The "spread" of these probabilistic contours is a representation of the "uncertainty" (or model errors) of these two triggering relationships.

Figure 3-9 is an enlarged view of Figure 3-1 (c), and it is annotated with a vertical dashed line at $N_{1,60,CS}$ = 20 blows/ft, so that the impacts of the three principal shortcomings in the regression performed by Boulanger and Idriss (2012) can be more clearly examined.

The first of these is the failure of the mean (PL = 50%) triggering boundary curve to suitably "fit" the field case history data at low values of $N_{1,60,CS} \leq 20$ due primarily to the overly "rigid" equational shape selected and the lack of sufficient degrees of regressible model fitting parameters. In this low $N_{1,60,CS}$ range, there are 15 Liquefied field cases "transgressing" below the PL = 50% line, and 7 Liquefied cases transgressing below the PL = 15% line. Reversing this assessment, in the same low $N_{1,60,CS} \leq 20$ range, there are only 7 Non-Liquefied cases transgressing above the PL = 50% line, and only 1 Non-Liquefied case transgressing above the PL = 85% line. All of the 3 "Marginal" liquefaction case histories in the $N_{1,60,CS} \leq 20$ range (indicated in the figure by triangles) also plot below the PL = 50% line. So upon close inspection, even when weighting factors of 0.8 and 1.2 are applied to the poorly classified "Liquefied" and "Non-Liquefied" cases, respectively, the uneven (unconservatively biased) positioning of the boundary curves in the range of $N_{1,60,CS} \leq 20$, relative to the data as developed and plotted by Boulanger and Idriss (2012), can be clearly seen.

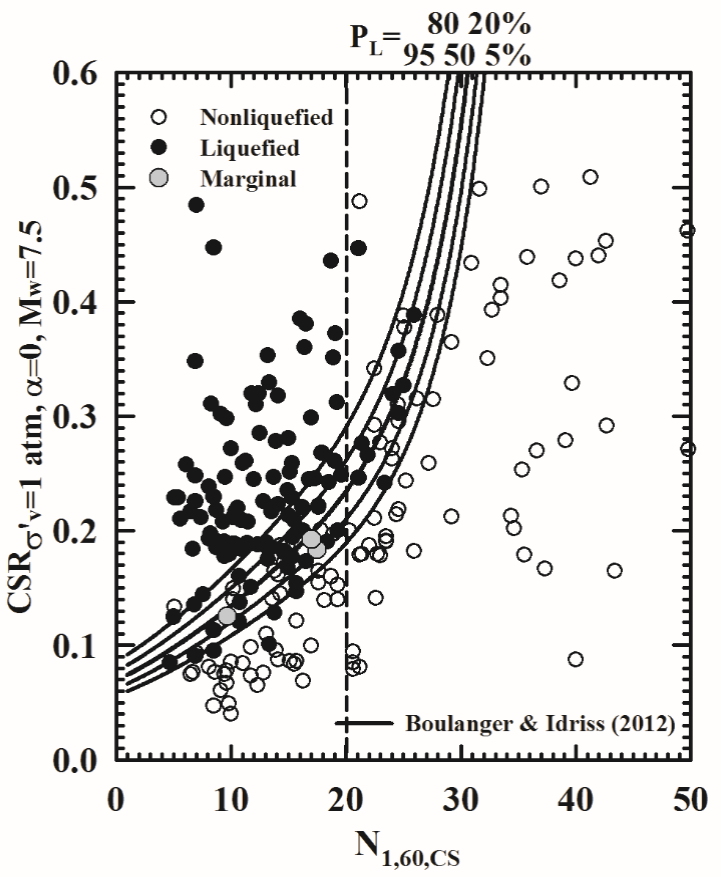


Fig. 3-9. The probabilistic triggering relationship boundary curves of Boulanger and Idriss (2012), and their field case history data points.

Re-examining Figures 3-1(b) and 3-1(c), it can be noted that both probabilistic relationships have PL-based boundary curves that are relatively closely spaced (indicating relatively low levels of uncertainty) in their bottom left corners. As the curves begin to rise, the uncertainty (and thus also the "spread" between the PL-based curves) begins to increase. In the Cetin et al. (2016) relationship, this spread continues to further increase as the curves rise to the top right extent of the figure. But in the Boulanger and Idriss (2012) relationship, the spread between the PL-based curves initially increases, and then begins to decrease again and the curves draw closer together in the top right corner.

Additional investigators have performed similar probabilistic regressions for liquefaction triggering relationships, employing a variety of different field liquefaction performance case history data sets. Significant examples include: Liao et al. (1988), Youd and Noble (1997), Toprak et al. (1999) and Juang et al. (2002). The overall levels of uncertainty vary between these different efforts, depending on (1) the number and quality of the field case histories in their databases, (2) case history back-analysis details, and (3) the probabilistic approaches employed in performing the regressions. But there are two common attributes of all of these regressions: (1) all of them produce regressed triggering relationships with significantly larger model error (or uncertainty) than the relationship of

Boulanger and Idriss (2012), and (2) all of them produce relationships in which the spreading of the contours is the narrowest in the bottom left corner and then increases to a maximum spreading (or uncertainty) at the top right corner of the "boundary curve" plots.

Figures 3-10 (a) and 3-10 (b) show two of these four additional relationships. Figure 3-10 (a) shows the relationship developed by Liao et al. (1988), which was a noteworthy early effort of this type, by early experts in the field of geotechnical probability and reliability. Figure 3-10 (b) shows the more recent relationship developed by Juang et al. (2002) by using the Bayesian mapping approach. All four of these previous relationships developed suites of probabilistic boundary curves with (1) significantly higher overall uncertainty than the curves of Boulanger and Idriss (2012), and (2) all four of them produced boundary curves with the lowest uncertainty in the lower left corner (at low penetration resistances) and with then progressively increasing model uncertainty towards the upper right corner, where the highest uncertainty occurs at the highest penetration resistances.

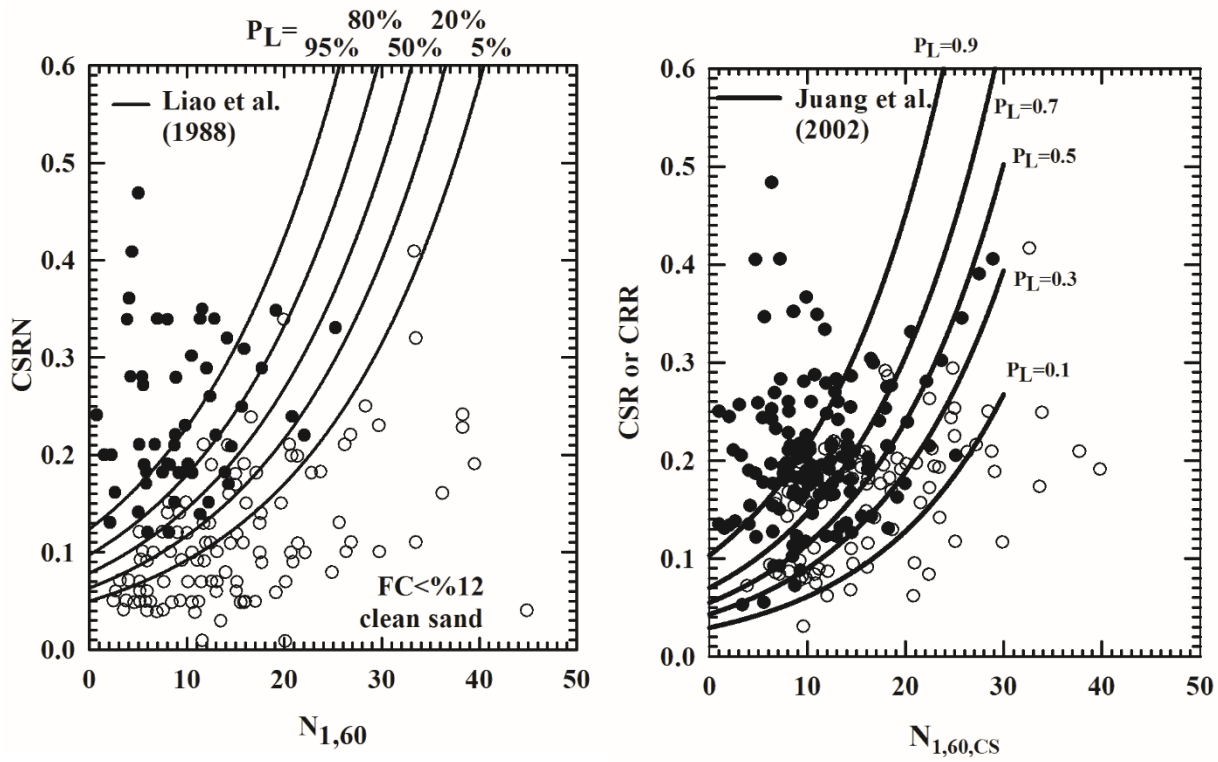


Fig. 3-10. The probabilistic triggering relationship boundary curves of (a) Liao et al. (1988) and (b) Juang (2002), and their field case history data points.

The authors of this current report have, as an ensemble, been personally involved in developing a number of liquefaction triggering field the case histories in the databases discussed. An important lesson from those field investigations is the relatively high level of difficulty often involved in determining whether or not a site "triggered" with regard to liquefaction when penetration

resistances are high ($N_{1,60,CS}$ ≥ 20 to 25 or so), because (1) these higher blowcount soils have limited cyclic strain potential, and (2) they undergo lesser levels of post-earthquake volumetric reconsolidation and so exude lesser levels of boil fluids and associated soil ejecta. As a result, uncertainty is intrinsically higher in this upper blowcount range as a result of uncertainty in characterizing observed field performance with regard to triggering (or non-triggering).

Considering the previous probabilistic studies, and this field experience-based observation, the spreading of the probability contours of Boulanger and Idriss (2012) is judged to be inconsistent with expected trends. Overall, model uncertainty appears to be underestimated at all locations for the probabilistic triggering relationship of Boulanger and Idriss (2012), and this appears to be most pronounced at higher $N_{1,60,CS}$ values. Shortcomings in the treatment of both parameter uncertainty and overall model uncertainty appear to invalidate the probabilistic elements of this triggering relationship.

3.3.5. Differences in Probabilistic Treatment in the Development of K_σ, K_{Mw}, and Fines Adjustments ($\Delta N_{1,60}$) Relationships (5)

Probabilistic treatment in the development of "secondary" relationships dealing with (1) effective overburden effects (K_σ), (2) causative magnitude (duration) scaling effects (K_{Mw}), and (3) fines adjustments ($\Delta N_{1,60}$), is the fifth of the major issues addressed in Table 3-2. Seed et al. (1985) employed no formal probabilistic approaches in the development of their main triggering curves, and they also used no formal probabilistic approaches in the development of their relationships for these three additional issues.

Boulanger and Idriss (2012) preferred not to employ probabilistic regressions to develop, or assist in the development of, "secondary" relationships dealing with (1) effective overburden effects (K_σ), (2) causative magnitude scaling effects (K_{Mw}), and (3) fines adjustments ($\Delta N_{1,60}$). A combination of non-probabilistic regressions and engineering judgments were instead used to develop these three secondary relationships, and the resulting relationships are discussed in Table 3-2. No regressions have been presented to explain the fines correction, and this appears to be a matter of engineering judgment. No use was made of the large liquefaction field performance case history database in development of relationships for K_σ and for K_{Mw}. The lack of probabilistic bases for these relationships, each of which affect the back-analyses of all of the field performance case histories, and the failure to attempt to analytically or judgmentally infer uncertainties associated with these "secondary" relationships, further affect the resulting probabilistic triggering relationship developed.

Cetin et al. (2016) followed a significantly different set of approaches here. All three relationships (K_σ, K_{Mw}, and $\Delta N_{1,60}$) were developed as part of a combined overall regression of the full field case history database along with the development of other triggering relationship elements (e.g.

the probabilistic triggering boundary curves, etc.). As a result, (1) the large field performance case history database was a dominant contributor to the defining of these "secondary" relationships, and (2) the resulting "secondary" relationships were intrinsically compatible with the overall probabilistic regressions performed, and the resulting probabilistically-based triggering relationships. Two of the resulting case history-based relationships (K_σ , and K_{Mw}) were then checked and were confirmed to agree suitably well with fundamentally different and independenty developed laboratory testing-based relationships and data, as discussed previously.

3.3.6. Differences in Transparency of Case History Processing and Documentation (6)

The sixth issue addressed in Table 3-2 is "Transparency", and with it the corollary issue of appropriate technical review. Transparency refers to the adequacy and transparency of documentation of (1) the background source data, (2) the selection, processing and analyses of those data, and (3) the many details and judgments made at each stage along the way in developing these types of complicated triggering relationships. Better, or worse, background documentation and transparency does not directly affect the likely bias of a given triggering relationship. But it makes it easier for other engineers and researchers to understand, and to back-check and review, the development of these types of important relationships.

Cetin et al. (2004) were comprehensively transparent in the presentation of the background development of their triggering relationship. As a result, it was studied by other engineers and investigators, and issues and errors were identified and challenged. As a consequence, Cetin et al (2016-a) were able to revise their derivations, employing a field case history database that was arguably unusually well-checked and vetted by other experts. That is an ideal outcome; and the authors of this report would suggest that full and transparent documentation should be a fundamental requirement for all similar efforts to develop engineering analysis tools for important problems with broad ramifications for public safety.

Seed et al. (1985) had also provided what was, at that time, an open and transparent documentation of their data and analyses. Their case history database was significantly smaller than the more recent efforts, and the relationship developed was "simpler" and lacked a formal probabilistic basis. The level of documentation transparency was not as extensive as that of Cetin et al. (2004), and hence full and complete independent examination of all details was not possible for a number of their case histories. But most cases could be suitably checked and examined, and the steps then taken in development of the triggering relationship were clearly explained.

The work of Idriss and Boulanger (2006) could not be properly checked or technically reviewed due to lack of documentation. Idriss and Boulanger (2008) published their triggering correlation in an EERI Monograph, but the documentation was incomplete and it could not be determined at that time which dots and circles on the triggering boundary curve plots corresponded to which case histories. Details of individual case history processing could also not be properly checked. The missing background documentation was later presented as a U.C. Davis research report by Idriss and Boulanger (2010). With the release of this 2010 document, it was possible to know which case histories were included in the development of their liquefaction triggering relationships. Relatively complete background details were presented, and so could then be traced and checked for 101 of the 230 case histories used in their relationships. The remaining 129 case histories continue to be incompletely documented, and thus it continues to be impossible to fully evaluate and back-analyze them.

3.4. ADDITIONAL DIFFERENCES BETWEEN THE THREE TRIGGERING RELATIONSHIPS

Table 3-3 lists 11 additional issues and factors that result in differences between the three triggering relationships. These are not all of the remaining factors. Instead they are selected either because they can make a potentially non-negligible difference in certain ranges of applications, or because engineers tend to ask about these and their effects. The issues discussed in Table 3-3 are generally less significant than those of Table 3-2.

Table 3-3. A Summary List of Additional Differences between the Three Liquefaction Triggering Relationships Considered

Input Parameter	Seed et al. (1985)*	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
7. Selection and 8. Screening of Field Performance Case Histories	Seed et al. (1985) had access to a significantly more limited set of potential candidate field performance case histories than are currently available. As a result, they were not able to set their screening criteria at as high a set of standards as the more recent studies of Boulanger and Idriss (2012) and Cetin et al. (2016). ○ (?) Bias: Bias is non-systematic, and difficult to evaluate, due to incomplete documentation.	Boulanger and Idriss accepted the screening criteria of Cetin et al. (2004), and largely adopted their data set. Accordingly 36 lower reliability cases from the Seed et al. (1985) database were eliminated. Then 33 new cases were added, without applying the same screening criteria. 20 of these cases were subsequently rejected when these cases were screened by Cetin et al. (2016). Cetin et al. (2016) re-evaluated these new cases, and accepted 13 of them. Later, additional 24 cases from the 1999 Kocaeli and 1999 Chi-Chi earthquakes were also compiled as discussed in Boulanger and Idriss (2014). All 24 of the new cases from Boulanger and Idriss (2014) were rejected by Cetin et al. (2016), due to incomplete soil characterization or documentation, or due to structural interference to free field response. ○ (?) Bias: Bias is non-systematic, and difficult to evaluate, due to incomplete documentation.	Cetin et al. (2004) employed a well-defined and high set of standards with regard to data completeness, documentation, and reliability, as well as the absence of K_α effects, and the absence of soil-structure interaction effects. These same standards were again employed in screening and then admitting 13 additional case histories to the field performance database of Cetin et al. (2016). ○ Bias: Unbiased.	It is difficult to evaluate the impacts in terms of overall bias, and also the impacts in terms of overall model uncertainty (or dispersion). In principal, it is the position of Cetin et al. (2016) that better and more consistent screening of field case histories is to be preferred.
9. Size of Field Performance Database	The available candidate performance case histories were limited in 1985. The database of 125cases was the best that had been assembled up to that time, but it suffered from (1) lower numbers of cases, and (2) lower screening standards for acceptance of case histories, than more recent efforts. Case histories were especially sparse for CSR ≥ 0.25 and $20 \le N_{1,60,CS} \le 35$, so that the upper end of the triggering boundary curves were poorly constrained. ○ (?) Bias: Bias is non-systematic, and difficult to evaluate.	As discussed in (7) and (8) above, Boulanger and Idriss assembled a larger database, with 230 cases, but at the cost of sacrificing screening standards for some of the case histories. ○ (?) Bias: Bias is non-systematic, and it is difficult to evaluate.	Cetin et al. (2016) assembled a slightly smaller database than that of Boulanger and Idriss (2012), with 210 cases. The difference was due to deliberative application of higher (and consistent) screening standards for acceptance of case histories. ○ Bias: Unbiased.	-

Table 3-3. (Contn'd) A Summary List of Additional Differences between the Three Liquefaction Triggering Relationships Considered

Input Parameter	Seed et al. (1985)*	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
10. Selection of Critical Depth	For most of the field case histories the critical depth was selected as either (a) the depth of the lowest SPT blowcount, or (b) the mean of the depth range of the apparently most critical stratum. For about 20 cases, however, despite the presence of more critical shallower suspects, the critical layer was selected as the deeper stratum. This usually caused an overestimation of CSR for those cases. ○(?) Bias: Overall bias is difficult to evaluate, due to incomplete documentation. Bias due to selection of the base of the critical stratum is systematic, and serves to bias the back-estimated CSR values (vertically upwards) for those cases.	For most of the field case histories, the critical depth was selected as the mean of the depth range of the apparently most critical stratum. For about 20 cases, however, despite the presence of more critical shallower suspects, the critical layer was selected as the deeper stratum consistent with Seed et al. (1985). This again usually caused an overestimation of CSR for those cases. ○(?) Bias: Overall bias is difficult to evaluate, due to incomplete documentation. Bias due to selection of the base of the critical stratum is systematic, and serves to bias the back-estimated CSR values (vertically upwards) for those cases.	Mean (mid-depth) of the critical layer's submerged depth range was taken as the representative critical depth. ○ Bias: Unbiased.	Adopting mean depth minimizes the error in input parameter estimations and is most consistent with the maximum likelihood regressions performed by Boulanger and Idriss (2012) and Cetin et al. (2016).
11. Selection and Processing of Representative $N_{1,60}$-Values	For most cases, averaged $N_{1,60}$-values within the critical layer appear to have been employed. ○ Bias: Unbiased.	For most cases, averaged $N_{1,60}$-values within the critical layer were employed. For a few cases, the depth of the lowest $N_{1,60}$-value was employed (e.g.: 1979 Imperial Valley, 1981 West Morland, 1987 Elmore Ranch, 1987 Superstition Hills, Radio Tower B1 case history site; 1995 Hyogoken Nanbu No 7; 1978 Miyagiken-Oki, Hiyori-18). Given the incomplete documentation, it is difficult to be certain how often this occurred, but it seems to have been fairly rare. $N_{1,60}$-values measured just above the phreatic surface were excluded. Variances in $N_{1,60}$ values for each case history were not evaluated. ○ Bias: Unbiased or minor.	Selected as the averaged (median) $N_{1,60}$-value in the critical layer. If a partially submerged soil layer exists, N-values just above ground water table are also sometimes used in the averaging scheme to characterize the critical layer; if they appear to be consistent. This mainly affects characterization of uncertainty or variance. Variance in $N_{1,60}$ is characterized for each case history. ○ Bias: Unbiased.	N-values are locally variable, sometimes due to localized variability in soil conditions, and sometimes due to variability of SPT procedures (e.g. variable hammer energy, etc.). When characterizing the critical layer, using the average $N_{1,60}$-value is the preferred approach.

Table 3-3. (Contn'd) A Summary List of Additional Differences between the Three Liquefaction Triggering Relationships Considered

Input Parameter	Seed et al. (1985)*	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
12. Selection and Processing of Representative FC Values	It is difficult to back-track the evaluations/selections of representative fines contents for many of the case histories due to incomplete documentation. Agreement with FC evaluations of the other two triggering correlations' databases is generally good, but some of the FC values are difficult to reproduce. ○ (?) Bias: Bias is non-systematic, and difficult to evaluate, due to incomplete documentation.	Mean or available FC value based on best available information was employed here. ○ Bias: Bias is non-systematic, and appears not to be significant.	Mean or available FC value based on best available information was employed here. ○ Bias: Bias is non-systematic, and appears not to be significant.	The overall average over the full databases shows that $(FC_{I\&B})_{avg} \leq (FC_{Cetin})_{avg}$, but the difference between these averages is only 2.5% for the case histories common to both databases, representing excellent overall agreement, especially considering differences in cases selected for inclusion in the two data sets.
13. Fines Adjustment ($\Delta N_{1,60,CS}$)	The fines adjustment relationship was not developed by formal regression, but was instead based on hand-drawn boundary curves for three "bins" of data with three ranges of fines contents. Engineering judgment was then used to infer approximate fines adjustments. The fines adjustment of Seed et al. (1985) is a function of fines content and CSR. Soils with FC ≤ 5% have null adjustment, and fines adjustments reach a limiting maximum value at FC ≥ 35%. Data were sparse, and as a result there was significant uncertainty associated with this pioneering assessment of apparent fines corrections for liquefaction triggering. In later years, Prof. Seed came to feel that this initial fines adjustment somewhat overstated the actual fines correction warranted. ○ (?) Bias: Bias is difficult to evaluate.	The basis of Boulanger and Idriss fines correction was not fully documented. The "binning" and hand-drawn boundary curves exercise employed by Seed et al (1985) is likely to be used to develop their fines correction. Fines adjustments are a function of FC only. Once again, soils with FC ≤ 5% have nearly null adjustment, and fines adjustments reach an asymptotic maximum value at about FC ≥ 35%. ○ (?) Bias: Bias is difficult to evaluate, and appears likely to be small.	Fines content adjustment relationships were formally regressed within the overall probabilistic maximum likelihood regressions performed. The resulting fines adjustments are a function of both FC and $N_{1,60}$. ○ Bias: As this fines adjustment relationship was based on the overall triggering relationship regressions, it appears likely to be essentially unbiased.	Both Boulanger and Idriss (2012) and Cetin et al. (2016) developed fines correction relationships producing lesser overall fines adjustments than Seed et al. (1985). The two newer sets of fines adjustment relationships differ somewhat, with the overall average fines adjustments for each of the two databases expressed as $(\Delta N_{1,60,Cetin})_{avg}$ and $(\Delta N_{1,60,I\&B})_{avg}$ differing by only 0.40 blows/30cm.

Table 3-3. (Contn'd) A Summary List of Additional Differences between the Three Liquefaction Triggering Relationships Considered

Input Parameter	Seed et al. (1985)*	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
14. C_N	Seed et. al. (1985) employed overburden normalization relationship, which was developed based on large-scale laboratory test data. There is no official equational form, and the relationship is expressed as a figure. ○ (?) Bias: Bias is difficult to evaluate.	C_N is a function of effective vertical stress and also $N_{1,60,CS}$. The maximum value of C_N at low effective overburden stress is "capped" at $C_N \leq 1.7$. This cap affects 16 of the case histories in the IB-2010 database. ○ Bias: This is a valid approach, and the results should likely be judged to be unbiased.	C_N is a function of effective vertical stress, based on the relationship proposed by Whitman and Liao (1986) and recommended by the NCEER Working Group (Youd et al, 2001). The maximum value of C_N at low effective overburden stress is "capped" at $C_N \leq 2.0$. This cap affects 5 of the case histories ○ Bias: This is a valid approach, and the results should likely be judged to be unbiased.	For mean $N_{1,60} \approx 15$, the three sets of C_N values are largely comparable. For effective overburden stresses of $\sigma'_v < 1\ atm$: (1) if $N_{1,60}<15$, then $C_{N,I\&B} > C_{N,Cetin}$, and (2) if $N_{1,60}>15$, then $C_{N,Cetin} > C_{N,I\&B}$. --- In the two datasets, the overall average value of $N_{1,I\&B}$ is slightly lower than the overall average of $N_{1,Cetin}$, as shown in Table 2-8 of the Chapter 2
15. Selection and processing of a_{max}	It is difficult to track the approaches employed to estimate a_{max} for many of the cases. Attenuation relationships were used, and these were locally calibrated by strong motion records when such records were available. It appears that the larger of the two horizontal components recorded was targeted for a number of the cases. But given the early state of knowledge with regard to attenuation relationships, and the relative paucity of strong motion recordings to locally calibrate these for specific case histories, it is difficult to assess (1) the exact type of "a_{max}" targeted for each case, and (2) potential bias. ○ (?) Bias: Bias is difficult to evaluate.	Maximum of the two horizontal components recorded, if available (e.g.: 1980 Mid-Chiba Owi 1&2; 1964 Niigata Kawagishi-Cho., 1983 Nihonkai-Chubu Gaiko1, Gaiko 2, Akita station, Aomori Port, etc.) ○ Bias: This is a valid approach, and the results should likely be judged to be unbiased.	Geometric mean of the two horizontal components, if available. Site response analyses, if possible. ○ Bias: This is a valid approach, and the results should likely be judged to be unbiased.	Targeting the maximum of the two horizontal components, rather than the geometric mean, causes I&B a_{max} values to be slightly higher than Cetin et al. a_{max} values. This can be recovered by employing the maximum or the geometric mean, respectively, in forward engineering analyses. a_{max} values on the average are comparable

Table 3-3. (Contn'd) A Summary List of Additional Differences between the Three Liquefaction Triggering Relationships Considered

Input Parameter	Seed et al. (1985)*	Boulanger and Idriss (2012)	Cetin et al. (2016)	Comment
16. K_{Mw}	Seed et al. (1985) developed a new K_{Mw} scaling relationship based on (1) laboratory undrained cylic test data, and (2) processing of earthquake strong ground motion recordings. This was a valid approach, but it made no use of the field performance case history database. (?) Bias: Bias is difficult to evaluate.	Boulanger and Idriss (2012) used the K_{Mw} scaling relationship developed by Idriss (1999), again based on (1) laboratory undrained cyclic test data, and (2) processing of earthquake storng ground motion recordings. This was a valid approach, but it again made no use of the field performance case history database. (?) Bias: Bias is difficult to evaluate.	A new K_{Mw} scaling relationship was developed as part of the overall maximum likelihood regressions performed. The resulting relationship for K_{Mw} was thus based on regression of the full liquefaction field triggering performance case history database. ○ Bias: Unbiased.	All three approaches should be considered valid. Given the uncertainties involved in correlating cyclic laboratory test data (cyclic triaxial, simple shear or torsional shear) with the multi-directional and irregular cyclic field loadings from real earthquakes, it is suggested here that regression of the field performance database is a preferred approach. Fortunately, agreement among the three sets of K_{Mw} relationships is reasonably good, except for very low magnitude causative events.
17. Weighting of Field Case Histories to Account for Sampling Disparity	No weighting factors were employed. In a probabilistic regression, that would have produced likely bias due to field sampling disparity. But as the boundary curves were instead developed based on judgment that may not be the case. ○ (?) Bias: Bias is difficult to evaluate.	The same weighting factors developed by Cetin et al. (2004) were applied to mitigate the bias otherwise introduced by field sampling disparity. These were: (1) Liquefied Cases $W_{liq.} = 0.8$, and (2) Nonliquefied Cases $W_{nonliq.} = 1.2$. ○ (?) Bias: This suitably mitigates field sampling bias with regard to liquefied and nonliquefied sites, but it may leave case histories from the 1995 Hyogoken Nanbu (Kobe) earthquake over-represented.	The weighting factors developed by Cetin et al. (2004) were applied to mitigate the bias otherwise introduced by field sampling disparity. An additional factor was applied to down-weight the Kobe EQ cases. Factors applied were: Liquefied Cases: $W_{liq.} = 0.8$ Nonliquefied Cases: $W_{nonliq.} = 1.2$ Kobe EQ Cases: $W_F = 0.25$ ○ Bias: Unbiased.	-

The additional factors discussed in Table 3-3 are numbered 7 through 17 and will be discussed next.

3.4.1. Selection (7) and Screening (8) of Field Case Histories, and Size (9) of Resulting Databases

The evaluation and assessment of field case histories as to their suitability for inclusion in the eventual databases upon which the triggering relationships are developed, is a critical step, because the overall quality and reliability of the databases strongly affects the quality of the resulting triggering relationships developed. Seed et al. (1985) had a more limited number of potential candidate case histories, and so were not able to set as high a standard with regard to screening of cases prior to adoption into the data set.

Cetin et al. (2004) had access to a significantly larger suite of candidate case histories, and was able to establish systematic standards with regard to availability, adequacy and documentation of site characterization data. Seed et al. (1985) had set standards that included non-use of data from sites with potential K_α effects, and potential soil-structure interaction effects. Cetin et al. set similar standards here as well. The use of more than one critical stratum at any given location was rejected, because when one stratum begins to soften or liquefy that changes the overall site response in ways that are difficult to reliably back-analyze for purposes of back-calculation of representative in situ cyclic stress ratios (CSR's). Cetin et al. (2004) re-evaluated the case histories employed by Seed et al. (1985), and deleted 36 cases. Cetin et al. (2004) then evaluated more than 200 additional candidate cases, and added only 110 of these to the database.

Idriss and Boulanger (2006) accepted the screening and judgments of Cetin et al. (2004), and largely adopted their database. They then added 33 additional cases, but did not apply the same screening standards and protocols to these new cases. Later, additional 24 cases from the 1999 Kocaeli and 1999 Chi-Chi earthquakes were also compiled as discussed in Boulanger and Idriss (2014). However, a new regression was not performed with the extended database. Cetin et al. (2016) re-evaluated these new cases, and accepted 13 of them. 20 and 24 of the new cases from Idriss and Boulanger (2010), and Boulanger and Idriss (2014) were rejected respectively, due to incomplete soil characterization or documentation, or due to structural interference to free field response, respectively. Table 3-1 summarizes the underlying reasons for non-conformance with Cetin et al. screening standards and protocols.

Overall, the early data set of Seed et al. (1985) was the smallest (125 cases), and it was necessary to include some cases of lower quality even to achieve those numbers. The data set of Boulanger and Idriss (2012) was the largest (230 cases), but 20 of those cases failed to meet and pass

the screening criteria of Cetin et al. (2016). The data set of Cetin et al. (2016) contains 210 cases, and is nearly as large as the data set of Boulanger and Idriss, but with consistent screening standards enforced throughout. It appears likely that the sparseness of the data available to Seed et al. (1985) adversely affected their triggering relationship, especially with regard to corrections for fines content. The data sets of Boulanger and Idriss (2012) and of Cetin et al. (2016) differed a bit, but this does not appear to have been a major contributor to differences between the two relationships. Instead, those differences appear to have more to do with differences in the use made of the field performance case history data to develop the two triggering relationships.

3.4.2. Selection of Critical Depth (10)

An important pair of choices to be made in back-analysis of each of the field performance case histories are (1) the location of the most critical stratum (which is the soil stratum that will be back-analyzed), which must be characterized with a top and a base elevation, and (2) the "critical depth" that will be taken as most representative of this critical stratum. Seed et al. (1985) tended largely to employ the observations and judgments of post-earthquake field investigators with regard to identification of the critical stratum, augmented and cross-checked with use of the available in situ data (e.g.: SPT blow counts, fines contents and fines plasticity data, etc.) to select the critical stratum, and then they typically selected either (1) the mid-point (mid-depth) of that stratum, or (2) the depth of the lowest measured penetration resistance, as the critical depth. For a number of case histories (e.g.:1944 Tohnankai Earthquake, Komei, 1948 Fukui Earthquake, Shonenji Temple, etc.) however, they employed a deeper critical depth than the mid-depth of the critical stratum. That served to bias estimates of CSR for those cases. Adopting these cases from Seed et al. database without modifications, Idriss and Boulanger (2010) accepted their critical depth choices. The selection of the bases of the critical strata for these cases was not well documented by Seed et al. (1985) and Idriss and Boulanger (2010). The approach employed by Boulanger and Idriss (2012) otherwise was to define the critical depth as the mid-depth of the critical stratum. Cetin et al. (2004, 2016) also employed both field observations and the available soil data to determine the apparent extent of the most critical stratum, and then selected the mid-depth of the critical stratum as the representative (mean) critical depth.

3.4.3. Selection and Processing of Representative SPT $N_{1,60}$ Values (11)

Seed et al. (1985) most commonly adopted the averaged $N_{1,60}$ values as the representative value. For most cases, averaged $N_{1,60}$-values within the critical layer were employed by Boulanger and Idriss (2012). However, valid for a few cases, the depth of the lowest $N_{1,60}$-value was employed (e.g.: 1979 Imperial Valley, 1981 West Morland, 1987 Elmore Ranch, 1987 Superstition Hills, Radio Tower B1

case history site; 1995 Hyogoken Nanbu No 7; 1978 Miyagiken-Oki, Hiyori-18). Given the incomplete documentation, it is difficult to be certain how often this occurred, but it seems to have been fairly rare. Variances in $N_{1,60}$ values for each case history were not evaluated. Cetin et al. selected the averaged (median) $N_{1,60}$-value in the critical layer as the representative value. If a partially submerged soil layer exists, N-values just above ground water table are also sometimes used in the averaging scheme to characterize the critical layer; if they appear to be consistent. This mainly affects characterization of uncertainty or variance. Variance in $N_{1,60}$ is characterized for each case history.

3.4.4. Selection and Processing of Representative Fines Content (FC) Value (12)

Seed et al. (1985) most commonly adopted the FC value corresponding to the critical depth. However, occasionally the available FC value (e.g.: 1944 Tohnankai-Chubu, Meiko case history) was also used as the representative value. Boulanger and Idriss (2012) and Cetin et al. (2016) both averaged the available FC values in the critical layer. On the average, FC values by Boulanger and Idriss (2012) are smaller than that of Cetin et al. (2016). This difference is mostly due to differences in the selection of critical depths and hence corresponding FC values.

3.4.5. Fines Adjustment, ($\Delta N_{1,60,CS}$) (13)

Each of the three sets of triggering relationships employs a correction (or adjustment) for effects of fines content. The fines corrections developed and adopted by Seed et al. (1985), Boulanger and Idriss (2012), and Cetin et al. (2004 and 2016), and are presented and compared in Figure 3-2.

There were significant differences in the procedures employed to develop these fines corrections, and in the case history data sets upon which they were based. Seed et al. had sparse field case history data, and they manually plotted the data (in CSR vs. $N_{1,60}$ space) with the data "binned" into three sets of cases with (1) FC < 5 %, (2) 5% ≤ FC ≤ 35% (assumed to represent approximately FC = 15%), and (3) FC > 35%. They then drew lines, by hand, based on engineering judgement, to develop the three triggering boundary curves shown in Figure 3-1 (a). No formal regressions were performed. The late Prof. H. Bolton Seed later became concerned that subsequent accumulating data suggested that the cresulting (inferred) fines adjustments were a bit too large, and he employed a slightly lower set of fines adjustments ($\Delta N_{1,60}$) in his final paper (Seed, 1987).

The basis of Boulanger and Idriss fines correction scheme cannot be fully traced. The presentation in Idriss and Boulanger (2008) indicates that they initially repeated this same "binning" process, employing the plotting of three sets of binned data and then the hand-drawing of several sets of boundary curves based on engineering judgment, as a basis for inferring their new fines corrections.

This was again based on engineering judgment, and no formal regressions were performed. The resulting fines adjustments have been further modified in Boulanger and Idriss (2012), but the precise approach taken has not been fully transparently explained and there have still been no formal regressions presented.

Cetin et al. (2004, and again in 2016) developed fines corrections based on the overall (formal) probabilistic regressions of the large field performance case history database, so that these fines corrections are the only set of formally regressed fines corrections among the three sets of triggering relationships examined in this report. The purpose of performing formal regressions is to develop keener, and more reliable, insights than can be obtained based on visual judgments.

The fines adjustments ($\Delta N_{1,60}$) of Seed et al. (1985), shown in Figure 3-11, are a function of $N_{1,60}$ (or CSR) and they are the largest of the four fines adjustment relationships shown. The fines adjustment of Boulanger and Idriss (2012) does not vary as a function of CSR, nor as a function of $N_{1,60}$, and it is of intermediate size (generally lower than that of Seed et al. (1985) and generally a bit higher than that of Cetin et al., 2016, except at very high values of $N_{1,60}$. The fines adjustments of Cetin et al. were the only ones formally regressed based on the field case history database, and based on comparison with these it would appear that the fines adjustments of Seed et al. (1985) were indeed a bit on the large side, and that there is somewhat better (but imperfect) agreement between the fines adjustments of Boulanger and Idriss (2012) and those of Cetin et al. (2016).

As shown in Table 2-8 and Figure 3-11, the corresponding difference is only approximately 2.4 % in the resulting overall "median" fines corrected $N_{1,60,CS}$ values for the two data sets of Boulanger and Idriss (2012) and Cetin et al. (2016). These are relatively modest, but non-zero, differences and they would be expected to have relatively modest effects on the overall triggering relationships developed. It is important to employ fines adjustments compatible with the triggering relationship selected.

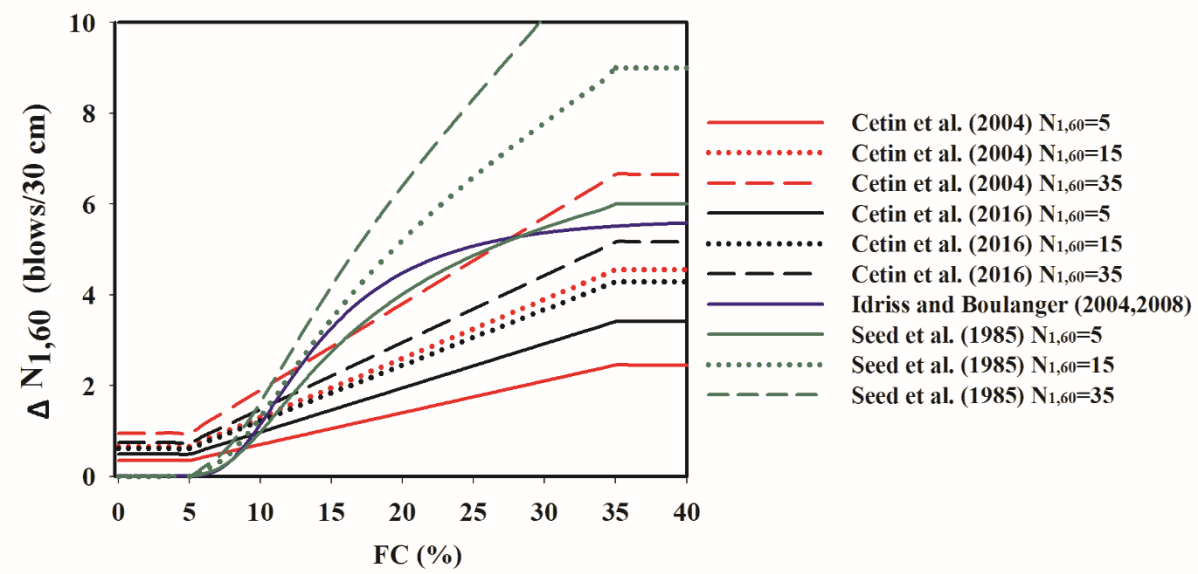


Fig. 3-11. Comparative illustration of fines correction

3.4.6. Selection and execution of overburden (C_N) and rod length (C_R) Corrections (14)

In the building of these data catalogs, Seed et al. (1985), Cetin et al. (2004) and Idriss and Boulanger (2008, 2010), truncated C_N at some chosen maximum or limiting values. Idriss and Boulanger (2010) adopted an upper limit of 1.7 on overburden correction term of C_N. The upper limit on CN was selected as 2.0 by Seed et al, and Cetin et al., also consistent with the recommendations of NCEER (1997). The choice of a "cap" had relatively minor overall impact on differences between the three relationships, as the upper cap of $C_N \leq 1.7$ used by Idriss and Boulanger (2008, 2010) affected 16 out of 230 cases. Note that Idriss and Boulanger (2010) C_N correction is relative density (or $N_{1,60,CS}$) dependent, and the denser the layer, the smaller is the C_N correction, whereas the Cetin et al. (2004 and 2016) C_N correction is only a function of vertical effective stress .

The effect of the different C_N relationships by different research teams on the overall liquefaction triggering curves is more difficult to quantitatively assess, and like the other factors discussed it is a bit a matter of chance as to which data points are affected, and to what degree, and what their locations (and impacts of the boundary curves developed) are. Nonetheless, it is noted that the approximate "median" C_N curves used by all three research teams are largely similar to each other, especially in the depth (and overburden stress) ranges where most of the case histories are found. The parameter C_N likely has a relatively minor, effect on the differences among these models.

By Cetin et al., the correction for "short" rod length between the driving hammer and the penetrating sampler was taken as a nonlinear "curve", rather than the step-wise incremental values of the NCEER Workshop recommendations, but the two agree well at all NCEER mid-increments of length.

Idriss and Boulanger (2010) also used C_R values consistent with NCEER Workshop recommendations. However, the major difference is in the assessment of rod "stick-up" (protrusion). Except for cases where rod “stick-up” (protrusion) above the top of the borehole was recorded, rod protrusion of ~ 1.2 m (~4 ft) above the top of the borehole was assumed for donut hammers and for the USGS safety hammers, and rod protrusion of ~ 2.1 m (~ 7 ft) was assumed for all other safety hammers by Cetin et al. When direct SPT energy measurements are available, energy and short rod corrections are combined. Rod stick up was not incorporated into Idriss and Boulanger, and Seed et al. (1985) C_R estimations.

3.4.7. Selection of peak ground acceleration, a_{max} (15)

Due to ground motion prediction equations used back then, Seed et al. maximum ground acceleration values are mostly based on maximum of the two horizontal acceleration components. Even though it is applicable to a limited number of cases and of minor importance, Idriss and Boulanger preferred to use the maximum of two horizontal acceleration components (e.g.: 1980 Mid-Chiba Owi 1&2; 1964 Niigata Kawagishi-Cho., 1983 Nihonkai-Chubu Gaiko1, Gaiko 2, Akita station, Aomori Port, etc.) as opposed to Cetin et al's choice of the geometric mean, if and when available. For those cases, these choices cause Seed et al. and Idriss and Boulanger case history data points shift up on the CSR axis.

3.4.8. Magnitude (Duration), K_{Mw} correction (16)

Scaling of the triggering relationships for numbers of equivalent uniform cycles of seismic loading (or duration of shaking) is another correction that affects liquefaction triggering assessments, particularly for small magnitude cases. Duration, or numbers of cycles, are correlated with causative earthquake magnitude, so the parameter employed here is K_{Mw}. As shown Figure 3-12, there exist significant differences among K_{Mw} recommendations proposed by various research teams for very low magnitudes (e.g. $M_w = 5.5$), and differences can also be significant at very high magnitudes (e.g. $M_w = 8.0$ and greater).

The K_{Mw} relationships of (1) Idriss (1995) which defines the lower bound of the range recommended by the NCEER Working Group (Youd et al., 2001) for application to the triggering relationship of Seed et al. (1985) and (2) Boulanger and Idriss (2012) were both developed based on (1) laboratory undrained cyclic test data and (2) processing of large numbers of strong ground motion recordings, and no use was made of the liquefaction triggering field case histories. The K_{Mw} relationships of Cetin et al. (2004, and 2016) are based on regressions of the large liquefaction

triggering field case history databases, and make no use of laboratory data. As a result, these two sets of relationships are based on fully independent sets of data, and types of data.

The relationships of Cetin et al. are located between the relationships of Idriss (1995) and Boulanger (2012), as shown in Figure 3-12, and there is relatively good agreement among these despite the very different approaches, and fully independent data sets, upon which they are based. In addition, all of the large field case history data sets discussed here have median values of magnitude approximately equal to 7.1. As a result, the impacts of K_{Mw} on differences between the triggering relationships is small.

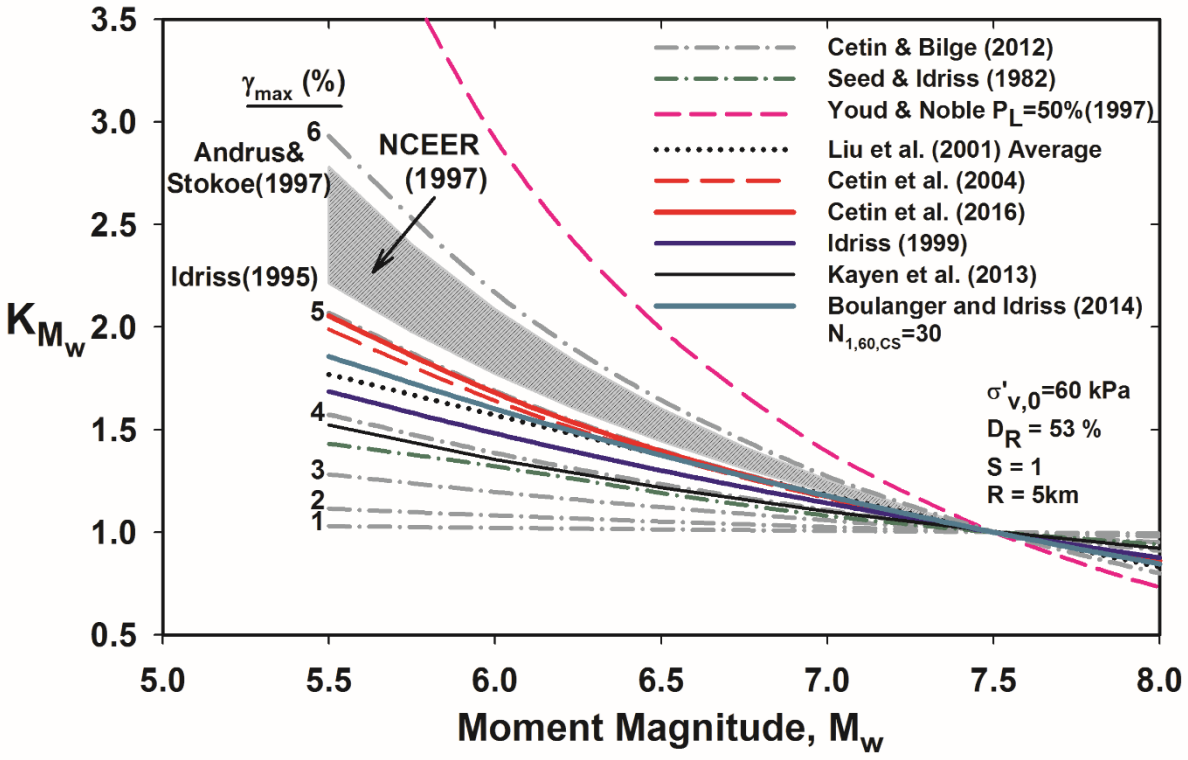


Fig. 3-12. Magnitude (duration) scaling factors

3.4.9. Application of data weighting (17)

Seed et al. (1985) did not apply a formal weighting scheme to balance the number of liquefied & nonliquefied cases. Idriss and Boulanger (2010), and Cetin et al. (2004, 2016) applied a weighting of 0.8 and 1.2 to liquefied and nonliquefied case histories, respectively. In addition to these weights, unlike Idriss and Boulanger (2010); Cetin et al. (2016) down scaled 56 Hyogoken-Nanbu case histories by applying a weighting factor of 0.25, which are mostly compiled from Masado fill sites. This controlled the over-representation (dominance of them) in the development of resulting liquefaction triggering curves.

3.5. CHAPTER SUMMARY AND CONCLUSIONS

For a given $N_{1,60,CS}$ value, the corresponding cyclic resistance or CSR values (i.e.: CRR) associated with any targeted level of likelihood of liquefaction triggering based on the triggering relationships of (1) Seed et al. (1985), (2) Boulanger and Idriss (2012) and (3) Cetin et al. (2016) are observed to be significantly different. These differences occur at essentially all locations on the triggering curves, but they are most pronounced at low $N_{1,60,CS}$ values. For conditions corresponding to σ'_v = 1 atm., the differences between the estimated CRR values reach as high as 50 to 80 % in the critical region of $N_{1,60,cs}$ < 20 blows/ft, a region within which the consequences of triggering of liquefaction can be especially significant due to low post-liquefaction strengths and high cyclic shear strain potential.

This chapter has examined the principal sources and causes of differences between the three triggering relationships, addressing each source of differences in turn. It is now useful to summarize by instead addressing each of the three overall triggering relationships in turn.

3.5.1. Seed et al. (1985)

The triggering relationship of Seed et al. (1985) was the last of the "first generation" of empirical triggering relationships based on field case histories, and it was the first essentially "complete" relationship of this type as it was the first comprehensive effort to address both (1) fines corrections, and (2) adjustments of measured SPT penetration resistances to account for variations in SPT equipment and procedures. Key attributes, and issues, associated with this early relationship include the following:

1. The field case history data set was sparse, due to the limited number of available cases from which to construct this database, and Seed et al. had to accept and use some cases of lower quality, reliability, or level of documentation than the other two triggering relationships to develop a database even this large. A total of 125 liquefaction field case histories were utilized in developing this triggering relationship.

2. The early "r_d" curves of Seed and Idriss (1971) were used for back-analyses of the field case histories. These r_d curves had been developed based on one-dimensional site response analyses of simplified, monolithic, sand-only site conditions that were not representative of the more layered and more complex site conditions present at many of the field case history sites. The acceleration levels applied to these sites were not sufficiently high as to encompass the levels of shaking of some of the case histories. Use of these non-representative r_d curves produced estimates of back-calculated CSR that were systematically biased to the high side, tending to push the resulting CSR values vertically upwards on the triggering plots, and producing a source of unconservative bias in the resulting triggering relationship.

3. No normalization for effective overburden stress was performed (no K_σ corrections were applied) because this triggering relationship was recognized to be generally applicable to "shallow" site conditions corresponding to the shallow conditions of most of the liquefaction triggering field case histories. Because the most "representative" vertical overburden stress of the overall case history database, and thus the triggering relationship as well, was more nearly $\sigma'_v = 0.67$ atm. than $\sigma'_v = 1$ atm., not correcting for K_σ effects was largely equivalent to "truncation" of K_σ to $K_\sigma \leq 1.0$. This, unconservatively biased the resulting plotted CSR values for a majority of the case histories, moving the CSR's for cases with $\sigma'_v < 1$ atm. (a majority of the cases) vertically upwards on the triggering plots, and this introduced a significant additional source of unconservative bias in the triggering relationship developed.

4. The early fines adjustments of this relationship were not developed based on regression, but were instead estimated based on manual plotting of the field data and engineering judgment. The sparse available field data led to an initial level of fines adjustments that the late Prof. Seed later came to view as somewhat unconservative (too large) as additional liquefaction triggering field data continued to become available. That was later confirmed by the subsequent relationships developed by both Boulanger and Idriss (2102) and by Cetin et al. (2004, 2016), who both developed smaller fines corrections based on the significantly larger field case history databases available to them.

5. There was no formal probabilistic basis for this triggering relationship, and so it is not known with any accuracy what level of probability of liquefaction triggering is represented by the "deterministic" triggering curve developed. Accordingly, this relationship cannot be used directly in performing probabilistic assessments of liquefaction triggering hazard, nor in associated/resulting risk evaluations.

6. This early triggering relationship was presented and documented in a manner that was open and transparent for its time, but the level of transparency was incomplete and so (1) it was difficult to fully back-check some elements of the work, and (2) some of the "secondary" inconsistencies in back-analyses of the case histories were carried forward into the triggering relationship of Boulanger and Idriss (2012).

3.5.2. Boulanger and Idriss (2012)

This relationship began as the Idriss and Boulanger (2004, 2006) triggering relationship, and it has subsequently been modified in several iterations to produce the current relationship of Boulanger and Idriss (2012). Changes have been incremental, and the character of the overall relationship remains largely intact. Key attributes, and issues, associated with this relationship include the following:

1. Boulanger and Idriss largely accepted and adopted the significantly enlarged case history database of 197 field performance cases developed by Cetin et al. (2004), and then added 33 additional cases to develop a case history database of 230 cases. This was considerably larger than the case history database employed by Seed et al. (1985), and with the least reliable of the earlier case histories employed by Seed et al. (1985) having been screened out and deleted by Cetin et al. (2004). The result was a significantly larger database of generally higher overall quality. Some of the 33 cases added by Boulanger and Idriss subsequently failed to meet the screening criteria employed by Cetin et al. (2016) and were deleted, but overall the two large databases of (1) Boulanger and Idriss (2012) and (2) Cetin et al. (2016) were largely similar, and quality was generally good in both.

2. Boulanger and Idriss (2016) repeated the short-coming #2 above of Seed et al. (1985), and again employed r_d curves that were developed based on analyses of non-representative, simplified, and overly "stiff" site conditions. These were newly developed r_d curves, but they had strikingly similar attributes to those previously employed by Seed et al. (1985), and the use of these non-representative r_d curves again produced estimates of back-calculated CSR that were systematically biased to the high side, tending to push the resulting CSR values vertically upwards on the triggering plots. This, again, introduced a source of unconservative bias in the resulting triggering relationship developed.

3. Boulanger and Idriss were aware of the need for normalization to for effective overburden stress effects, and so K_σ correction relationships were developed and applied. Unfortunately, it was decided to truncate K_σ to a value of $K_\sigma \leq 1.1$ in processing their case history data. This truncation affected 47 of the 230 case histories, and in a similar manner as the inadvertent truncation of K_σ implicit in the use of the triggering relationship of Seed et al. (1985). As a result, this truncation of K_σ is again a source of unconservative overall bias in this triggering relationship. Boulanger and Idriss developed their own K_σ relationship, and this relationship was also less conservative than previous relationships when applied to back-analyses of field case histories, the majority of which occurred at relatively shallow depths, and thus at effective overburden stress of less that $\sigma'_v = 1$ atm. This exacerbated the truncation error, and added further unconservative bias.

4. The precise approach taken in developing the fines correction recommendations for this triggering relationship has not been fully transparently explained. The initial fines corrections of Idriss and Boulanger (2006) were again based on plotting binned sub-sets of the field case history data and engineering judgment, as Seed et al. (1985) had done. These have since been updated by Boulanger and Idriss (2012), but thei basis is unexplained and no regressions have yet been presented as a basis for developing the fines correction recommendations for this 2012 triggering relationship. The resulting fines adjustments are significantly smaller (more conservative) than those of Seed et al.

(1985), and they are on average somewhat larger than those of Cetin et al. (2016). These fines adjustments are not a function of $N_{1,60}$, as is the case for the fines adjustments of both Seed et al. (1985) and Cetin et al. (2016), and this appears to result in locally unconservative (over-sized) fines adjustments at very low $N_{1,60}$ values. This a localized issue, and it does not appear to significantly adversely affect the overall triggering relationship at higher $N_{1,60}$ values.

5. Formal probabilistic regressions were performed to develop this overall triggering relationship, but due to short-comings in these regressions (1) the resulting triggering boundary curves do not provide a reasonable and unbiased "fit" of the back-analyzed field case history data, especially at values of $N_{1,60,CS} \leq 20$, and (2) overall model uncertainty is underestimated. These shortcomings would appear to invalidate this relationship with regard to use in conjunction with formal probabilistic or risk-based forward engineering applications. In addition, the "stiffly" regressed model provides an unconservative"fit" for the field data as plotted, and this is especially pronounced at lower penetration resistances of $N_{1,60,CS} \leq 20$. Model uncertainty is underestimated at all locations, and this is particularly pronounced at higher values of $N_{1,60,CS} \geq 20$.

6. The K_σ relationship proposed by Boulanger and Idriss (2012) was based on laboratory undrained cyclic test data, rather than regressions of the field case history database, and it differs from the K_σ relationship developed by Cetin et al. (2012). It is less conservative than other K_σ relationships for use in back-analyses of field case histories and development of triggering relationships, and this is then multiplicatlively compounded by it also being less conservative for forward engineering analyses for cases with very high effective overburden stresses.

7. Documentation and transparency was lacking when the initial relationship of Idriss and Boulanger (2006, 2008) was first published. As a result, their work could not be properly evaluated and fully checked. Improved documentation was eventually provided by Idriss and Boulanger (2010), but only 101 of the 230 case histories were suitably documented with regard to assumptions and details involved in their back-analyses. Independent checking of the processing and back-analyses of the remaining 129 field case histories continues to be difficult to impossible.

3.5.3. Cetin et al. (2016)

1. Cetin et al. (2004) developed systematic screening criteria to evaluate the suitability and reliability of potential candidate case histories. They applied those to the case history database of Seed et al. (1985), and this eliminated 35 of the 125 cases. They next examined more than 200 potential new candidate field case histories, and based on the same screening criteria they adopted 110 of those. Cetin et al. (2016) deleted 3 cases, and then screened 33 additional cases added by Boulanger and

Idriss (2012) and found 13 of them to meet the screening criteria. These cases were added, producing a final database with 210 field performance case histories, all of them systematically screened for quality and reliability.

2. Cetin et al. (2016) employed r_d curves that had been developed based on 2,153 site response analyses of 50 actual sites from the case history database, and based on regressions of those analyses to develop r_d relationships that varied as a function of site conditions (layering and stiffness), as well as intensity and duration of shaking, to develop back-analysis estimates of CSR for 48 of the case histories. These probabilistic r_d curves were employed for assessment of CSR in 162 of the field case histories. The remaining 48 case histories were back-analyzed by means of site-specific and event-specific one-dimensional site response analyses using (1) available nearby ground motion records from the actual earthquakes (scaled to transpose them to the local site), and (2) actual site stratigraphy and soil properties. The resulting back-calculated values of CSR for all 210 case histories were thus specifically unbiased best-estimates, with case-specific uncertainties also evaluated, and they are compatible for use in forward engineering analyses employing either (1) "simplified" (rd-based) evaluations of CSR's, or (2) CSR's developed directly by means of and event-specific seismic site response (or site response and soil-structure interaction) analyses. This is the only one of the three triggering relationships that can be used (without bias) for forward engineering analyses employing direct calculations of CSR's based on direct seismic site response and/or soil-structure interaction analyses.

3. No truncation of K_σ was employed. Back-calculated CSR values for all case histories were therefore correctly normalized for effective overburden stress effects, and so were the overall database and the resulting triggering curves developed. The K_σ relationship employed in development of the overall triggering curves was based on regression of the large field case history database, and so was specifically appropriate over the ranges of vertical effective stress well represented in this database ($0.25 \text{ atm.} \leq \sigma'_v \leq 1.8 \text{ atm.}$). This K_σ relationship was used to "center" or normalize the overall triggering curves to a condition representative of $\sigma'_v = 1$ atm. Having accomplished that in an unbiased manner, the resulting triggering curves can then be extrapolated to much higher effective vertical stresses employing suitable K_σ relationships of the engineer's choice.

4. Fines corrections were developed based on formal probabilistic regressions of the large liquefaction field performance case history database.

5. Formal probabilistic regressions of the field case history database were performed to develop the resulting probabilistic liquefaction triggering relationship. Suitable degrees of freedom were available in the regression so that the triggering curves could conform themselves to the dictates of the large

field database, and the very difficult and time-consuming task of evaluating and treating both individual parameter uncertainties as well as overall model uncertainty were suitably performed. As a result, this triggering relationship provides an unbiased framework for application to probabilistic liquefaction triggering and overall risk evaluations.

6. Because the initial work of Cetin et al. (2004) was transparently well-documented, the back-analyses, assumptions, etc. involved in those studies were well examined by other researchers. Issues, questions, and challenges resulting from the examinations and reviews were thus able to be implemented to develop (1) a resulting database that is arguably the best reviewed field case history data set developed to date for these purposes, and (2) regressed updated triggering relationships that benefitted from both challenges and discussions of details of the previous work. A similar level of transparency and documentation is provided for the updated work of Cetin et al. (2016).

7. The K_σ relationship employed within the development of the triggering relationship was developed based on regression of the large field case history database. As a result, it provides an unbiased basis for the specific purpose of "normalizing" the triggering relationship to a representative effective vertical overburden stress of σ'_v = 1 atm. A discussion of the merits of various K_σ relationships for extrapolation to higher effective vertical stresses is presented in this report, in conjunction with Figure 3-7.

8. Documentation of the data and analyses involved in the development of the new triggering relationship of Cetin et al. (2016) is again presented in a complete and transparent manner, so that other engineers and researchers can thoroughly examine and check all details.

Re-examining the trigging relationship plots of Figures 3-1 (a), (b) and (c) more closely, with the benefit of the discussions above, it can now be clearly seen that the case history data tend to plot higher on the plots in Figures 1(a) and (b), than in Figure 1(c). It can also be noticed that the triggering curves plot higher (especially for $N_{1,60,CS}$ < 20). The main reasons for this are now reasonably well understood.

The triggering relationships of both Seed et al. (1985) and Boulanger and Idriss (2012) can be demonstrated to be unconservatively biased, especially at $N_{1,60,CS}$ < 20. This unconservatism appears to be exacerbated when the K_σ relationship of Boulanger and Idriss (2012) is then employed to extrapolate to values of σ'_v significantly greater than 1 atmosphere.

This unconservatism can be expected to be most significant (1) for engineering projects where critical strata have representative values of $N_{1,60,CS}$ < 20, (2) for projects where direct, site-specific seismic site response (or site response and soil-structure interaction) analyses are performed to

directly calculate CSR values, rather than using the "simplified (r_d-based) approach, and/or (3) for engineering projects where potential liquefaction of soils with σ'_v significantly greater than 1 atm. is of potential concern. It should be further noted that (4) the lack of validity of the probabilistic elements of the triggering relationship of Boulanger and Idriss (2012) obviates its use in probabilistic or risk-based engineering analyses and assessments. The triggering relationship of Seed et al. (1985) has no probabilistic content, and so is only ever properly applied in a deterministic manner.

Engineers who have employed these relationships can use the four sets of characteristics above as a partial basis to determine which projects may potentially warrant review or re-evaluation.

As a concluding remark, one of the most significant underlying messages of this manuscript is likely the importance of fully transparent documentation of the background details and data involved in the types of back-analyses and regressions, etc. employed in the development of these complex types of engineering analysis tools. This foments the ability of other engineers and researchers (and oversight agencies) to fully review and understand the work, which is important for engineering analysis tools with significant ramifications with regard to public safety.

That lesson is already being implemented. A multi-year Next Generation Liquefaction (NGL) program is now underway, coordinated through the Pacific Earthquake Engineering Research Center (PEER), and involving an unusually large number of researchers, in an effort designed to develop improved liquefaction triggering relationships (and for SPT, CPT, V_S, and other approaches) employing programs, structures and protocols largely paralleling those of the highly successful PEER NGA program which has addressed Next Generation (ground motion) Attenuation relationships. Intrinsic to both of these efforts (NGA and NGL) are stringent requirements with regard to full and open transparency and documentation, as well as discussion and review throughout the process among the engineers and researchers involved. The NGL effort is currently targeted at producing new triggering relationships over the next five years, or so, and experience from the NGA program suggests that (1) there may be some delays, and (2) the eventual results are likely to be well-reviewed and well-checked new engineering analysis tools, with good communal support within both the research and practice communities.

In the meantime, engineers will continue to have to sort through the thicket of confusion surrounding the existing liquefaction triggering relationships. It is the hope of the authors that the materials presented in this Chapter will be helpful in that regard.

4. Probabilistic Models for Seismic Soil Liquefaction Assessment

In this Chapter, an attempt is made to clarify the essentials of maximum likelihood analysis and the methodology followed to develop a reliability-based mathematical model for seismically-induced soil liquefaction initiation problem. The philosophical issues as well as the theoretical details regarding the maximum likelihood assessment paradigm itself are beyond the scope of this section. In the following sections, particular attention is given to the discussion of the mathematical models for seismic soil liquefaction initiation hazard, with emphasis on formulation of likelihood functions for exact and inexact observation cases. Also the mathematical representations of the uncertainties regarding the measurable or observable variables, such as $N_{1,60}$ and $CSR_{\sigma v',Mw}$, are discussed.

4.1. Baye's Theorem and Likelihood Function

Since its publication in 1763, Bayes' theorem as a basis for statistical inference has become one of the widely used theorems for updating the present state of knowledge with more recent observations. Bayes' theorem states that

$$f_{\theta}(\underline{\theta}|\underline{x}) = c \cdot L(\underline{x}|\underline{\theta}) \cdot f'_{\theta}(\underline{\theta}) \qquad \textbf{Eqn. (4-1)}$$

where $\underline{\theta}$ is a vector of unknown parameters, $f'_{\theta}(\underline{\theta})$ is the prior distribution, $L(\underline{x}|\underline{\theta})$ is the likelihood function, $c = \left[\int_{\Theta} L(\underline{x}|\underline{\theta}) \cdot f'_{\theta}(\underline{\theta}) \cdot d\underline{\theta}\right]^{-1}$ is a normalization factor, and $f_{\theta}(\underline{\theta}|\underline{x})$ is the updated posterior distribution of model parameter set $\underline{\theta}$. The likelihood function is proportional to the conditional probability of making the observations, given the value of parameter set $\underline{\theta}$.

$$L(\underline{\theta}) \propto P(observation|\underline{\theta}) \qquad \textbf{Eqn. (4-2)}$$

This is where the available new information enters the updating formula. The prior distribution on $\underline{\theta}$, $f'_{\theta}(\underline{\theta})$, reflects our knowledge on the model parameters, prior to the new information becoming available, which may incorporate subjective information such as engineering judgement, constraints on parameters, or a prior identification of the model (Der Kiureghian, 1990).

The posterior distribution incorporates the prior (possibly subjective information), as well as the objective information through the likelihood function.

Bayesian parameter estimation based on observed or measured data requires:

1. A mathematical model that will link the current observations to the probabilities of making those observations, which will be used in formulating the likelihood function.

2. Prior knowledge or guesses about what the likely behavior could be (implies initial guesses or constraints about the prior distribution of model parameters). If such information is not available, non-informative priors can be used.

Mathematical liquefaction models, which will be further implemented in formulating the likelihood function for various cases of uncertainty assumptions, as well as the selection of appropriate prior distributions reflecting our state of knowledge before the observations become available, will be discussed next.

4.2. Maximum Likelihood Assessment

The MLE serves essentially the same purpose as multi-dimensional probabilistic "regression" analyses, but (1) allows for separate and more appropriate treatment of various contributing sources of aleatory uncertainty, and (2) facilitates treatment of more descriptive variables/parameters than have been attempted in most previous, similar studies while also permitting monitoring of model parameter interactions and co-variances. Within the MLE analyses, all data can be modeled not as "points", but rather as random variables with variances in descriptive input variables (i.e.: CSR, $N_{1,60}$, σ'_v, FC, M_w). That means, for example, that the "dots" in figures like Figure 2-5 are not really "points"; instead they are centroidal representations of what are actually distributions (much like bumps) with varying uncertainties in both the lateral and vertical directions depending upon the quality and quantity of data available for each case history.

As part of maximum likelihood assessment, consistent with the name of the assessment, the model coefficients $\underline{\theta}$ are estimated to maximize the likelihood function, $L(\underline{\theta})$. When formulating the likelihood function different sources of uncertainties can be incorporated in the likelihood formulation, which will be discussed next.

4.2.1. Sources of Uncertainty

Proper modeling of uncertainties within a probabilistic framework requires understanding the sources of errors for a model. Before discussing the details of the mathematical relationship that will link current observations to future predictions, sources of uncertainty, specifically for seismic soil liquefaction model, need to be clarified.

4.2.1.1. Measurement / Estimation Errors

In the process of assessing a model, measurements of variables under laboratory or field conditions are necessary. Such measurements are prone to error and this naturally introduces

additional uncertainty into the derived model. Specifically, in the soil liquefaction case, this kind of uncertainty arises from the aforementioned variability in SPT blow count estimations. Also predicting or measuring the level of shaking at the site of interest can be prone to variability. Significant effort has been spent on reviewing the available case histories and compile them considering the uncertainties related to each component of $N_{1,60}$ and $CSR_{\sigma v',Mw}$, including but not limited to critical depth (depth to the stratum that actually liquefied, or the most vulnerable stratum that did not liquefy), peak ground acceleration, depth to phreatic surface, unit weight of soil, etc. Figure 4.1 shows median ± one standard deviation range of Cetin et al. (2016) data set. Figures 4.1(a), (b) show case histories compiled from Seed et al. (1985) database and Hyogoken-Nanbu (Kobe) earthquake, respectively.

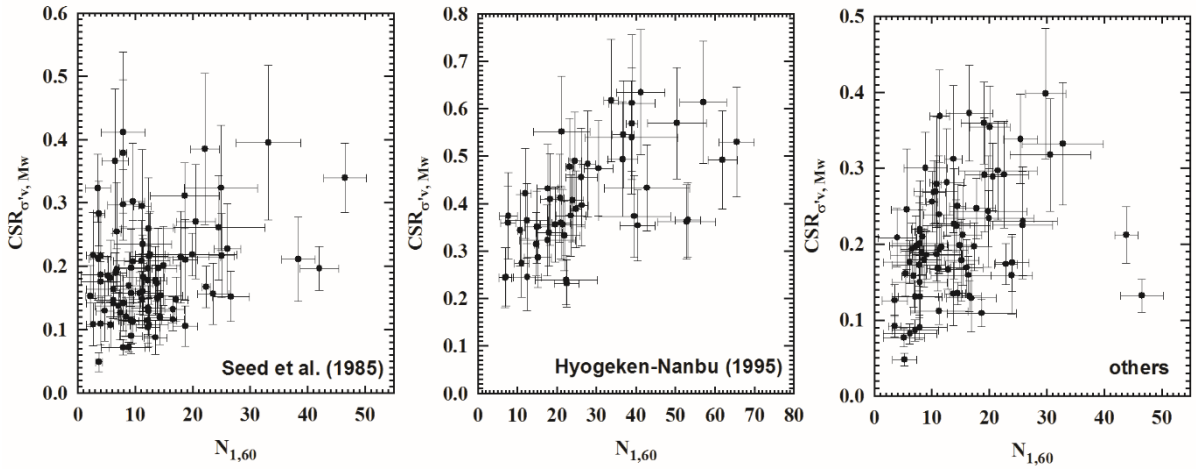


Figure 4-1. Median and ± one standard deviation range of Cetin et al. (2016) database

4.2.1.2. Errors of Simplification (Model Uncertainty)

The nature of modeling implies selection of some relatively more important aspects of a phenomenon vs. the ones, which are less important. For the sake of simplicity, and due to lack of data, some of the variables that actually influence the resistance of soil to liquefaction such as soil gradation, plasticity index, permeabilities of overlying and underlying layers, etc. are not incorporated into the proposed soil liquefaction initiation model. Those missing variables may be sources of errors for seismic soil liquefaction predictions. Similarly, incorrect formulation of the variable interactions may cause some errors in the estimated behavior. The influence of the missing variables and the "imperfect" functional form of the selected model can be represented by a random model correction term, ϵ.

4.2.1.3. Statistical Errors (Model Parameter Uncertainties)

The accuracy of the estimated parameters of a model depends on the size of the available data. When the sample of observations has finite size, which is often the case in engineering problems, additional uncertainty is introduced into the model due to statistical error in estimating the parameters.

4.2.2. Mathematical Liquefaction Models

The maximum likelihood approach for developing a limit state model for the initiation (or "triggering") of soil liquefaction begins with the selection of a mathematical model. The model for the limit-state function has the general form $g = g(\mathbf{x}, \boldsymbol{\Theta})$, where $\mathbf{x} = (N_{1,60}, CSR_{\sigma'_v,\alpha,M_w}, M_w, FC, \sigma'_v)$ is a set of descriptive variables, and $\boldsymbol{\Theta}$ is the set of unknown model coefficients. Consistent with the usual definition in structural reliability, liquefaction is assumed to have occurred when $g(\mathbf{x}, \boldsymbol{\Theta})$ takes on a negative value. The limit-state surface $g(\mathbf{x}, \boldsymbol{\Theta}) = 0$ denotes the initiation of liquefaction (or marginal liquefaction).

For modeling the likelihood of initiation of liquefaction, Seed et al. (1971) presented a relationship, widely known as "simplified procedures" where the observable/measurable variables were selected as overburden and energy corrected standard penetration test blow-counts ($N_{1,60}$) and the cyclic stress ratio normalized for K_σ, and K_{Mw} effects (i.e.: $CSR_{\sigma v',Mw}$).

Liao et al. (1988), following a method of binary regression using the same descriptive variables, presented a probabilistically-based liquefaction initiation relationship in the form of:

$$P_L = \frac{1}{1 + e^{-(\beta_0 + \beta_1 \cdot \ln(CSR_{\sigma'_v=1atm,\alpha=0,M_w=7.5}) + \beta_2 \cdot N_{1,60})}} \qquad \textbf{Eqn. (4-3)}$$

where P_L is the probability of liquefaction. The model parameters, β_0, β_1, and β_2, were estimated through maximization of the likelihood function.

Motivated by this and others, the following model from Cetin et al. (2002 and 2004) is used for the limit state function:

$$\hat{g}(x, \boldsymbol{\theta}) = N_{1,60}(1 + \theta_1 FC) - \theta_2 \ln(M_w) - \theta_3 \ln(\sigma'_v / \mathrm{P_a}) + \theta_4 FC + \theta_5 - \theta_6 \ln(CSR_{\sigma'_v,\alpha,M_w}) \qquad \textbf{Eqn. (4-4)}$$

where $\boldsymbol{\theta} = (\theta_1, \ldots, \theta_6)$ is the set of unknown model coefficients. An additional model coefficient θ_7 will be introduced later to calibrate the case history variance terms.

The limit state function given in Equation 4-4 has the advantage of assessing K_σ, K_{Mw} and fines corrections within a unified framework. Hence, as part of the maximum likelihood assessment, the overall liquefaction triggering relationship will be assessed jointly and all at once with these corrections or adjustment factors included.

Equation 4-4 assumes that the liquefaction triggering hazard can be completely explained by a set of five descriptive variables $\mathbf{x} = (N_{1,60}, CSR_{\sigma'_v,\alpha,M_w}, M_w, FC, \sigma'_v)$. But other variables exist, which may influence the initiation of liquefaction. Even if the selected descriptive variables were able to fully explain the liquefaction triggering phenomenon, the adopted mathematical expression may not have the ideal form. Hence, Equation 2-10 is by definition an imperfect model of the limit-state function. This is signified by use of a superposed "hat" on $\hat{g}$. To account for the influences of the missing variables and the possible incorrect model form, a random model error term, ε, was introduced as given in Equation 4-5.

$$g(x,\boldsymbol{\theta}) = N_{1,60}(1+\theta_1 FC) - \theta_2 \ln(M_w) - \theta_3 \ln(\sigma'_v / \mathrm{P_a}) + \theta_4 FC + \theta_5 - \theta_6 \ln(CSR_{\sigma'_v,\alpha,M_w}) + \varepsilon \qquad \textbf{Eqn. (4-5)}$$

4.2.3. Likelihood Functions

4.2.3.1. Statistically Independent Exact Observations

With the aim of producing an unbiased model (i.e., one that, on the average, makes the correct prediction), the mean of ε is set to zero, and for convenience is assumed to be normally distributed. The standard deviation of ε, denoted σ_ε, however is unknown and must be estimated as part of the regression. The set of unknown coefficients of the model, therefore, is $\boldsymbol{\Theta} = (\boldsymbol{\theta}, \sigma_\varepsilon)$.

Let $\mathbf{x}_i = (N_{1,60,i}, CSR_{\sigma'_v,M_w,i}, M_{w,i}, FC_i, \sigma'_{v,i})$ be the values of $N_{1,60}$, *CSR*, M_w, *FC* and σ'_v for the i^{th} case history, respectively, and ε_i be the corresponding realization of the model correction term. If the i^{th} case history field performance is liquefied, then $g(x_i, \varepsilon_i, \boldsymbol{\theta}) \le 0$. On the other hand, if the i^{th} case history field performance is nonliquefied, then $g(x_i, \varepsilon_i, \boldsymbol{\theta}) > 0$. When $g(x_i, \varepsilon_i, \boldsymbol{\theta}) \cong 0$ then the case history is defined as marginally liquefied (or marginally nonliquefied). Assuming that the Cetin et al. (2016) database is compiled from statistically independent liquefaction field performance case histories, the likelihood function can be written as the product of the probabilities of the observations given in Equation 4-6.

$$L(\boldsymbol{\theta},\sigma_\varepsilon)=\prod_{l=1}^{\text{Num. of liquefied sites}} P\left[g(x_l,\varepsilon_l,\boldsymbol{\theta})\le 0\right]\cdot\prod_{j=1}^{\text{Num. of non-liquefied sites}} P\left[g(x_j,\varepsilon_j,\boldsymbol{\theta})>0\right]\cdot\prod_{k=1}^{\text{Num. of marginally-liquefied sites}} P\left[g(x_k,\varepsilon_k,\boldsymbol{\theta})\cong 0\right] \quad \textbf{Eqn. (4-6)}$$

Suppose that $\mathbf{x}_i=(N_{1,60,i},CSR_{\sigma'_v,\alpha,M_w,i},M_{w,i},FC_i,\sigma'_{v,i})$ for each liquefaction field performance case history is exact, i.e., no measurement or estimation uncertainty is present. Then, noting that Equation 4-5 has the normal distribution with mean $\hat{g}(x_i,\boldsymbol{\theta})$ and standard deviation σ_ε, the likelihood function (Equation 4-6) can be written as given in Equation 4-7.

$$L(\boldsymbol{\theta},\sigma_\varepsilon)=\prod_{\text{liquefiedsites}}\Phi\left[-\frac{\hat{g}(x_l,\boldsymbol{\theta})}{\sigma_\varepsilon}\right]^{w_{liq.}}\cdot\prod_{\text{non-liquefiedsites}}\Phi\left[\frac{\hat{g}(x_j,\boldsymbol{\theta})}{\sigma_\varepsilon}\right]^{w_{nonliq.}}\cdot\prod_{\text{marginally-liquefiedsites}}\varphi\left[\frac{\hat{g}(x_k,\boldsymbol{\theta})}{\sigma_\varepsilon}\right] \quad \textbf{Eqn. (4-7)}$$

In Equation 2-13, $\Phi[\cdot]$ and $\varphi[\cdot]$ are the standard normal cumulative function and the standard normal probability density functions. Also note that w_{liq} and w_{nonliq} are the choice-based sampling bias correction factors, which will be discussed in Section 4.2.3.3

4.2.3.2. Statistically Independent Inexact Observations

If $\mathbf{x}_i=(N_{1,60,i},CSR_{\sigma'_v,\alpha,M_w,i},M_{w,i},FC_i,\sigma'_{v,i})$ is inexact (uncertain), then this additional uncertainty should be included in the formulation of the likelihood function as given in Equation 4-8.

$$L(\boldsymbol{\theta},\sigma_\varepsilon)=\prod_{\text{liquefiedsites}}\Phi\left[-\frac{\hat{g}(x_i,\boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right]^{w_{liq.}}\cdot\prod_{\text{non-liquefiedsites}}\Phi\left[\frac{\hat{g}(x_i,\boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right]^{w_{nonliq.}}\cdot\prod_{\text{marginally-liquefiedsites}}\varphi\left[\frac{\hat{g}(x_i,\boldsymbol{\theta})}{\sigma_{\varepsilon,tot,i}}\right] \quad \textbf{Eqn. (4-8)}$$

Assuming that the mean values of the case history descriptive input parameters $N_{1,60,i},CSR_{\sigma'_v,\alpha,M_w,i},M_{w,i},FC_i,\sigma'_{v,i}$ are estimated in an unbiased manner (i.e., mean choices, rather than conservative or unconservative choices, are made in case history processing) then the error terms of $N_{1,60,i},\ln\left(CSR_{\sigma'_v,\alpha,M_w,i}\right),\ln\left(M_{w,i}\right),FC_i,\ln\left(\sigma'_{v,i}\right)$ can be considered as normally distributed random variables with zero means and standard deviations $\sigma_{N_{1,60},i}$, $\sigma_{\ln(CSR_{\sigma'_v,\alpha,M_w}),i}$, $\sigma_{\ln(M_w),i}$, $\sigma_{FC,i}$ and $\sigma_{\ln(\sigma'_v),i}$, respectively. Then the overall variance of each liquefaction field performance case history (σ^2_{tot}) is estimated as the sum of the variance of case history input parameters ($\sigma^2_{case\text{-}history}$) and model error ($\sigma^2_\epsilon$) as given in Equation 4-9.

$$\sigma^2_{tot,i}=\left(\theta_7\cdot\sigma_{case-history,i}\right)^2+\sigma^2_\varepsilon \quad \textbf{Eqn. (4-9)}$$

In Equation 4-9, $\sigma_{case-history,i}$ represents the consolidated uncertainty of individual case history input parameters, and as discussed earlier a scaling factor of θ_7 is systematically applied to them. This

factor (θ_7) is one of the regressed parameters of the overall triggering relationship. If the higher-order terms are eliminated, then $\sigma_{case-history,i}$ can be estimated as given Equation 4-10.

$$\sigma^2_{case-history,i} = (\theta_6)^2 \cdot \left[\text{cov}(CSR_{\sigma'_v,\alpha,M_w,i})\right]^2 + \sigma^2{}_{N_{1,60,I}} \cdot (1+\theta_1 \cdot FC_i)^2 + \sigma^2{}_{FC_i} \cdot (\theta_1 \cdot N_{1,60,i} + \theta_4)^2 + (\theta_3)^2 \cdot \left[\text{cov}(\sigma'_{v,i})\right]^2 \quad \textbf{Eqn. (4-10)}$$

Finally, the eight model coefficients (i.e.: θ_1 through θ_7, and σ_ε) are assessed all at once in a single overall regression in order to maximize the likelihood function presented in Equation 4-8. The resulting new model coefficients are listed in Table 4-1. This table also presents the previous Cetin et al. (2004) model coefficients for comparison. The resulting (new) recommended seismic soil liquefaction triggering relationships are presented in Equations 4-39 and 4-40.

The resulting new probabilistic boundary curves, based on the updated field performance case history database with 210 liquefaction triggering field case histories are shown in Figure 2-5 (a). Figure 2-6 (b) presents a direct comparison between the new triggering relationship, and the previous relationship of Cetin et al. (2004).

4.2.3.3. Choice-based Sampling Bias

A special challenge for development of unbiased soil liquefaction triggering correlations is post-earthquake field investigation sampling disparity. Researchers who receive funding and support for post-earthquake liquefaction field investigations tend to proceed mainly to sites where liquefaction was observed. As a result, there are a relatively large number of “liquefied” sites in the available databases. It is then more difficult to find and investigate sites (and strata) where soils of potentially liquefiable types exist, but did not liquefy; because by not liquefying they did not so obviously reveal themselves. As a result, there is a relative paucity of useful “nonliquefied” field performance case histories. This is not due to a lack of potentially liquefiable but nonliquefied soils; instead it is the result of natural sampling bias as researchers are preferentially drawn to liquefied sites.

The final data set contained roughly twice as many liquefied as non-liquefied informative data, and most large data sets assembled by prior researchers were found to have similar ratios (e.g.: Liao et al., 1988; Youd and Noble, 1997, etc.). The problem is that this represents a sampling disparity problem, and is not an unbiased reflection of actual field occurrences. Simply put, post-earthquake field investigators are more inclined to perform borings and tests at liquefied sites, than at sites where no apparent liquefaction occurred. Given finite research budgets, it would be asking too much to expect researchers to randomly space their SPT borings at non-liquefied sites in the hope of encountering non-liquefied soils of potentially liquefiable type.

Many researchers (e.g.: Manski and Lerman, 1977; Hsieh et al., 1985) addressed the uneven sampling problem as a result of choice-based sampling process. In their studies, Manski and Lerman (1977) defined both exogenous and choice- based sampling processes as follows:

"Existing estimation methods presuppose an exogenous sampling process, that is one in which a sequence of decision makers are drawn and their choice behaviors observed. In contrast, in choice-based sampling processes, a sequence of chosen alternatives are drawn and the characteristics of the decision makers selecting those alternatives are observed."

In the same studies, they have recommended a correction to the likelihood function, which can be stated specifically for liquefaction models as follows:

$$L(\underline{\theta},\sigma_{\varepsilon}|\underline{x}_i) \propto \prod_{i=1}^{k}\left[P(g(N_{1,60,i},CSR_i,M_{w,i},FC_i,\sigma'_{v,i},\varepsilon_i,\underline{\theta})<0)\right]^{w_{liq.}} \prod_{i=k+1}^{j}\left[P(g(N_{1,60,i},CSR_i,M_{w,i},FC_i,\sigma'_{v,i},\varepsilon_i,\underline{\theta}))>0)\right]^{w_{nonliq.}} \prod_{i=j+1}^{l}\left[P(g(N_{1,60,i},CSR_i,M_{w,i},FC_i,\sigma'_{v,i},\varepsilon_i,\underline{\theta}))\approx 0)\right] \quad \textbf{Eqn. (4-11)}$$

The weighting factors $w_{liq.}$ and $w_{nonliq.}$ are defined as:

$$w_{liq} = \frac{Q_p}{Q_s} \text{ and } w_{nonliq} = \frac{1-Q_p}{1-Q_s} \quad \textbf{Eqn. (4-12)}$$

where Q_p is the true (population) proportion of occurrences of liquefaction and Q_s is the corresponding sample proportion.

A major difficulty in correcting the likelihood functions for choice-based sampling is the fact that the true proportion of liquefaction cases in nature, Q_p, is unknown. Two approaches were invoked to address this problem. The first was to consult with experts, and to attempt to develop expert consensus regarding the true proportion of liquefaction cases. All experts consulted agreed that the bias was real, and that the corrective weighting factor ($w_{nonliq.}$) for the non-liquefied data should be greater than 1.0. The most common range for the ratio of $w_{nonliq.}/w_{liq.}$ was on the order of 1.5 to 2.

The second approach was to treat the weighting factors, w_1 and w_2, as variables in the maximum likelihood analyses performed, and to assess the weighting factors that provided equal (even) likelihood of occurrences for both liquefied and non-liquefied subsets. This requires an iterative procedure. The procedure followed can be summarized as follows:

1. Pick a pair of values for w_1 and w_2 that are subject to the constraints: (a) $w_{nonliq} + w_{liq} = 2$ (i.e.: overall information should stay approximately constant, and should not be artificially changed), and (b) w_1>1.0 (non-liquefied cases are underrepresented).
2. Estimate the posteriors (mean and standard deviation) for model parameters based on weighted likelihood function given in Equation 2-14 by using weighting factors defined in Step 1.
3. Based on estimated posteriors (Step 2), calculate the overall likelihood values separately for the liquefied and non-liquefied case history data subsets. If the values are comparable, stop the iteration and use the w_1 and w_2 pairs defined in Step 1, if not go to Step 1 and update weighting factors until convergence is achieved.

Happily both the expert opinions and the procedure defined above lead to the same scaling factor: $\frac{w_{nonliq}}{w_{liq}} \approx 1.5$ (i.e.: $w_{nonliq.}$=1.2, $w_{liq.}$=0.8). Finally a sensitivity study was performed, and it was found that a scaling factor of 1.5 produced only a modest shift in the correlations, as illustrated in Figure 4.2, and that it would be potentially over-conservative to leave the sampling disparity problem unaddressed. Accordingly, all non-liquefied and liquefied data were scaled by weighting factors of $w_{nonliq.}$=1.2 and $w_{liq.}$=0.8, respectively as stated in Equation 4-11.

For the data set of the Cetin et al. (2004) database, it was determined that sampling disparity would be suitably mitigated by (1) up-weighting the value of nonliquefied sites by a weighting factor of 1.2, and (2) by down-weighting the value of liquefied sites by a factor of 0.8 in performing the probabilistic regressions. This is approximately equivalent to simply up-weighting the non-liquefied cases by a factor of 1.5, but partitioning the weighting factors among both liquefied and non-liquefied cases allows improved treatment of overall model uncertainty.

The updated 2016 database has nearly the same overall characteristics (and almost the same cases) as the 2004 database, and the same weighting factors (1.2 and 0.8) were again applied to liquefied and nonliquefied case histories. An additional weighting factor was needed to address the large number of case histories from the 1995 Hyogoken-Nanbu (Kobe, Japan) earthquake. The database has 56 case histories from this single event; and that very large number results in an unbalance of the overall data set wherein that single event is over-represented. This was addressed in the regressions performed for development of the Cetin et al. (2016) database by down-weighting the Hyogoken-Nanbu (Kobe) earthquake case histories by a weighting factor of 0.25.

4.2.3.4. Estimating the Error Terms in Observations

Taylor's Expansion

For a set of multi-variate functions $Y_k = g_k(X_1,...,X_n)$ of random variables X_i, i =1,…,n, the Taylor series expansion around the mean point $\underline{M} = (M_1,...,M_n)$ is given by (Der Kiureghian, 1998)

$$Y_k = g_{k0} + \sum_{i=1}^{n}(X_i - \mu_i)g'_{ki} + \frac{1}{2}\sum_{i=1}^{n}\sum_{j=1}^{n}(X_i - \mu_i)\cdot(X_j - \mu_j)\cdot g''_{kij} + ... \qquad \textbf{Eqn. (4-13)}$$

where;

$$g_{k0} = g_k(x_1,...,x_n)\big|_{x=\underline{M}}$$
$$g'_{ki} = \frac{\partial g_k(x_1,...,x_n)}{\partial x_i}\Big|_{x=\underline{M}} \qquad \textbf{Eqn. (4-14)}$$
$$g''_{kij} = \frac{\partial^2 g_k(x_1,...,x_n)}{\partial x_i \partial x_j}\Big|_{x=\underline{M}}$$

If only the first two terms of the expansion presented above are retained, the first-order approximations of the mean and variance of Y_k are given by

$$\mu_{Y_k} \cong g_{k0} \qquad \textbf{Eqn. (4-15)}$$

$$\sigma^2_{Y_k} \cong \sum_{i=1}^{n}\sum_{j=1}^{n} g'_{ki}g'_{kj}\rho_{ij}\sigma_i\sigma_j$$
$$\cong \sum_{i=1}^{n} g'^2_{ki}\sigma_i^2 + 2\sum_{i=1}^{n-1}\sum_{j=i+1}^{n} g'_{ki}g'_{kj}\rho_{ij}\sigma_i\sigma_j \qquad \textbf{Eqn. (4-16)}$$

Also, the first-order approximations of the covariance between Y_k and Y_l, $k \neq l$ is obtained from

$$\mathrm{Cov}[Y_k, Y_l] \approx \sum_{i=1}^{n}\sum_{j=1}^{n} g'_{ki}g'_{lj}\rho_{ij}\sigma_i\sigma_j \qquad \textbf{Eqn. (4-17)}$$

For the special case of multivariate nonlinear functions $Y = a_0 X_1^{a_1} X_2^{a_2} ... X_n^{a_n}$, the derivatives evaluated at the mean point are estimated by

where $\mu_i \neq 0$ is assumed. The first-order approximation of the mean is

$$\mu_Y = a_o \mu_1^{a_1}\mu_2^{a_2}...\mu_n^{a_n} \qquad \textbf{Eqn. (4-18)}$$

The first-order approximation of variance is given by:

$$\sigma_Y^2 = \sum_{i=1}^{n}\sum_{j=1}^{n} a_0^2 a_i a_j \frac{\mu_Y^2}{\mu_i \mu_j}\rho_{ij}\sigma_i\sigma_j \qquad \textbf{Eqn. (4-19)}$$

After dividing Equation 4-19 by μ_Y^2 and introducing the coefficients of variation $\delta_i = \frac{\sigma_i}{\mu_i}$ and $\delta_Y = \frac{\sigma_Y}{\mu_Y}$, one obtains the following approximate expression for the coefficient of variation of Y.

$$\delta_Y^2 \approx \sum_{i=1}^{n} a_i^2 \delta_i^2 + 2\sum_{i=1}^{n-1}\sum_{j=1+1}^{n} a_i a_j \rho_{ij}\delta_i\delta_j \qquad \textbf{Eqn. (4-20)}$$

where use of the symmetry of the covariance matrix of **X** has been made.

Estimating the Uncertainty in N1,60

The in-situ SPT blow-counts (N), are corrected for the effects of effective overburden stresses C_N, energy C_E, borehole diameter C_B, short rod C_R, sampler C_S.

$$N_{1,60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \qquad \textbf{Eqn. (4-21)}$$

By using Equations 4-20 and 4-21, for the case of uncorrelated correction factors and SPT blow-counts, the square of the coefficient of variation (c.o.v.) δ of $N_{1,60}$ can be written as the sum of the squared c.o.v. of the individual variables, i.e.:

$$\delta_{(N_1)60}^2 \cong \delta_N^2 + \delta_{C_N}^2 + \delta_{C_E}^2 + \delta_{C_B}^2 + \delta_{C_R}^2 + \delta_{C_S}^2 \qquad \textbf{Eqn. (4-22)}$$

The results of the laboratory based studies of the correction terms revealed that the uncertainties regarding these terms are relatively small. For the case of exact correction terms, the mean and c.o.v. for $N_{1,60}$ are reduced to a more convenient form.

$$\mu_{(N_1)60} \cong \mu_N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S \qquad \textbf{Eqn. (4-23)}$$

$$\delta_{(N_1)60} \cong \delta_N \qquad \textbf{Eqn. (4-24)}$$

Estimating the Uncertainty in $CSR_{\sigma v',Mw}$ and S = ln($CSR_{\sigma v',Mw}$)

Similarly the uncertainty in $CSR_{\sigma v',Mw}$, where only the total stress and effective stress terms are assumed to be correlated, can be estimated as follows:

$$CSR_{\sigma_v',M_w} = \frac{0.65 \cdot a_{\max} \cdot \sigma_v \cdot r_d}{g \cdot \sigma_v'} \qquad \textbf{Eqn. (4-25)}$$

$$\mu_{CSR_{\sigma_v',M_w}} \cong \frac{0.65 \cdot \mu_{a_{\max}} \cdot \mu_{\sigma_V} \cdot \mu_{r_d}}{g \cdot \mu_{\sigma'_V}} \quad \textbf{Eqn. (4-26)}$$

$$\delta^2_{CSR_{\sigma_v',M_w}} = \delta^2_{a_{\max}} + \delta^2_{r_d} + \delta^2_{\sigma_v} + \sigma^2_{\sigma'_v} - 2 \cdot \rho_{\sigma'_v\sigma_v} \cdot \delta_{\sigma'_v} \cdot \delta_{\sigma_v} \quad \textbf{Eqn. (4-27)}$$

Given the total and effective stress terms, shown in Equations 4-25 and 4-27, the variance and the covariance between them are calculated by utilizing Equations 4-28 and 4-32.

$$\mu_{\sigma_v} \cong \mu_{\gamma_1} \cdot \mu_{h_w} + \mu_{\gamma_2} \cdot (\mu_h - \mu_{h_w}) \quad \textbf{Eqn. (4-28)}$$

$$\mu_{\sigma'_v} \cong \mu_{\gamma_1} \cdot \mu_{h_w} + (\mu_{\gamma_2} - \gamma_w) \cdot (\mu_h - \mu_{h_w}) \quad \textbf{Eqn. (4-29)}$$

$$\sigma^2_{\sigma_v} \cong \mu^2_{h_w} \cdot \sigma^2_{\gamma_1} + (\mu_h - \mu_{h_w})^2 \cdot \sigma^2_{\gamma_2} + \mu^2_{\gamma_2} \cdot \sigma^2_h + (\mu_{\gamma_1} - \mu_{\gamma_2})^2 \cdot \sigma^2_{h_w} \quad \textbf{Eqn. (4-30)}$$

$$\sigma^2_{\sigma'_v} \cong \mu^2_{h_w} \cdot \sigma^2_{\gamma_1} + (\mu_h - \mu_{h_w})^2 \cdot \sigma^2_{\gamma_2} + (\mu_{\gamma_2} - \gamma_w)^2 \cdot \sigma^2_h + (\mu_{\gamma_1} + \gamma_w - \mu_{\gamma_2})^2 \cdot \sigma^2_{h_w} \quad \textbf{Eqn. (4-31)}$$

$$\mathrm{Cov}[\sigma_v, \sigma'_v] \cong (\mu^2_{h_w} \cdot \sigma^2_{\gamma_1}) + (\mu_{\gamma_1} - \mu_{\gamma_2}) \cdot (\mu_{\gamma_1} + \gamma_w - \mu_{\gamma_2}) \cdot \sigma^2_{h_w} + (\mu_h - \mu_{h_w})^2 \cdot \sigma^2_{\gamma_2} + (\mu_{\gamma_2}) \cdot (\mu_{\gamma_2} - \gamma_w) \cdot \sigma^2_h \quad \textbf{Eqn. (4-32)}$$

In Equations 4-28 to 4-32, μ_{γ_1} and μ_{γ_2} are the mean estimates of unit weights of soils above and below the water table respectively, γ_w is the unit weight of water, μ_h is the mean estimate of critical depth, and finally μ_{h_w} is the mean estimate of the depth to phreatic surface. Except the unit weight of water, all the terms are inexact. Due to random nature of soil layering and deposition (most case histories are from old river or flood channel sites), γ_1 and γ_2 can be reasonably assumed to be uncorrelated. Having estimated the variances and the covariances of the total and effective stress terms, the correlation coefficient between them is estimated from:

$$\rho_{\sigma_v\sigma'_v} \cong \frac{\mathrm{Cov}[\sigma_v, \sigma'_v]}{\mathrm{Var}[\sigma_v] \cdot \mathrm{Var}[\sigma'_v]} \quad \textbf{Eqn. (4-33)}$$

Since the term implemented in likelihood function (Equation 4-5) is actually the $\ln(CSR_{\sigma v',Mw})$, it is required to perform the log transformation and estimate the mean and the standard deviation for $\ln(CSR_{\sigma v',Mw})$ rather than the CSR itself. If CSR is assumed to be log normally distributed (e.g.: Abrahamson and Silva, 2008 etc.) with mean μ_{CSR} and standard deviation σ_{CSR} then S = ln(CSR) is normally distributed with mean λ_S and standard deviation, ς_S. They are defined as follows:

$$CSR = LN\left(\mu_{CSR}, \sigma^2_{CSR}\right) \quad \textbf{Eqn. (4-34)}$$

$$S = \ln(CSR) = N\left(\lambda_s, \zeta_s\right) \quad \textbf{Eqn. (4-35)}$$

$$\lambda_S = \ln(\mu_{CSR}) - \frac{1}{2} \cdot \zeta_S^2 \quad \textbf{Eqn. (4-36)}$$

$$\zeta_S = \sqrt{\ln\left(1 + \delta^2_{CSR}\right)} \quad \textbf{Eqn. (4-37)}$$

$$\delta_{CSR} = \frac{\sigma_{CSR}}{\mu_{CSR}} \quad \textbf{Eqn. (4-38)}$$

In the process of data compilation, special attention was given not to include more than one case from sites with similar soil characteristics and shaken by same levels of peak ground acceleration (i.e.: cases from closely located borings). This eliminated the possibility of the presence of potentially correlated $CSR_{\sigma v',Mw}$ data in the database.

4.3. Probability-Based Liquefaction Triggering Boundary Curves

Model coefficients for exact and inexact case history variables are estimated to maximize the likelihood functions and are given in Table 4-1.

Table 4-1. A comparative summary of limit state model coefficients

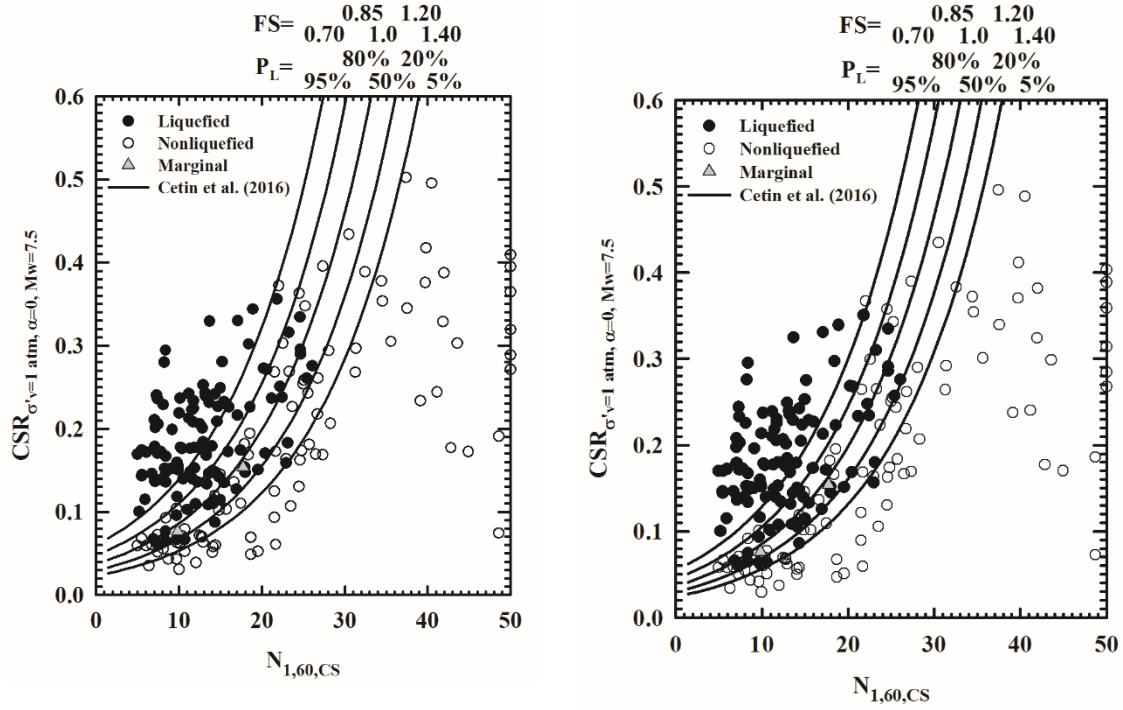
	Cetin et al. (2016)	
Model Coefficients	Case History Uncertainty Not Modeled (Exact)	Case History Uncertainty Modeled (Inexact)
θ_1	0.00145	0.00167
θ_2	25.722	27.352
θ_3	3.995	3.958
θ_4	0.093	0.089
θ_5	12.545	16.084
θ_6	11.940	11.771
θ_7	-	0.392
σ_ε	3.50	2.95

By using the model coefficients listed in Table 4-1 along with the limit state function given in Equation 4-5, Equations 4-39 and 4-40 are recommended to estimate probability of liquefaction triggering and corollary cyclic resistance ratio, respectively.

$$P_L(N_{1,60}, CSR_{\sigma'_v,\alpha=0,M_w}, M_w, \sigma'_v, FC) = \Phi\left(-\frac{\left(\begin{array}{l} N_{1,60}\cdot(1+\theta_1\cdot FC)-\theta_6\cdot\ln\left(CSR_{\sigma'_v,\alpha=0,M_w}\right)- \\ \theta_2\cdot\ln(M_w)-\theta_3\cdot\ln\left(\frac{\sigma'_v}{P_a}\right) \\ +\theta_4\cdot FC+\theta_5 \end{array}\right)}{\sigma_\varepsilon}\right) \quad \textbf{Eqn. (4-39)}$$

$$CRR(N_{1,60}, M_w, \sigma'_v, FC, P_L) = \exp\left[\frac{\left(\begin{array}{l} N_{1,60}\cdot(1+\theta_1\cdot FC)-\theta_2\cdot\ln(M_w) \\ -\theta_3\cdot\ln\left(\frac{\sigma'_v}{P_a}\right)+\theta_4\cdot FC+\theta_5+\sigma_\varepsilon\cdot\Phi^{-1}(P_L) \end{array}\right)}{\theta_6}\right] \quad \textbf{Eqn. (4-40)}$$

For exact and inexact case history input parameters, Figures 4-2 (a) and (b) are generated by using Equation 4-39 along with the model coefficients given in Table 4-1, respectively.



(a) Case history input parameter uncertainty not modeled.

(b) Case history input parameter uncertainty modeled.

Fig. 4-2. New probability-based liquefaction triggering curves

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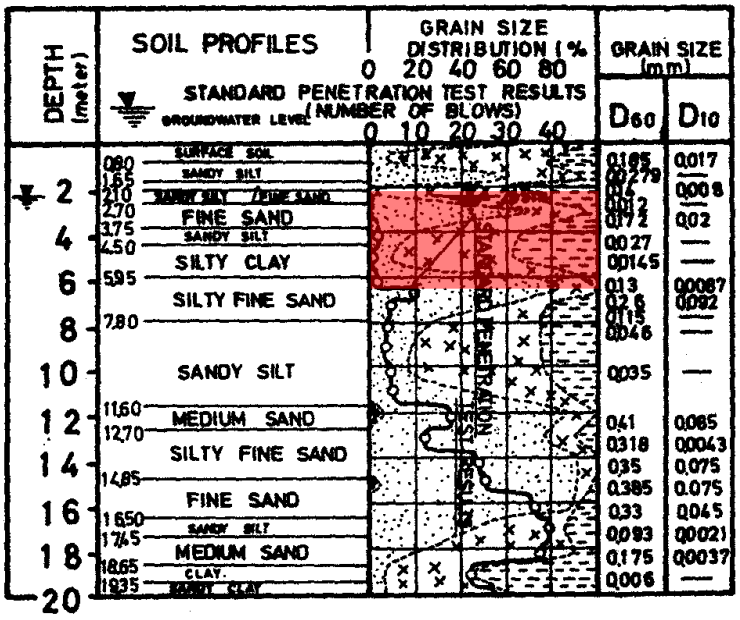
APPENDIX A

SUMMARY OF FIELD CASE HISTORY DATA

Case number:	1
Earthquake:	1944 Tohnankai
Magnitude:	8.3 (M_R)
Location:	Ienaga
References:	Kishida (1969)
Nature of Failure:	…differential settlements occurred in addition to eruption of water.
Comments:	The epicenter of the earthquake is located 165 kms south-southwest of Nagoya city. The seismic intensity of shaking was estimated as JMAIS V-VI. In the source document, the liquefied zone was predicted at depth range of 11.5-12.5 m based on the criterias as follow : Effective overburden pressure<2.0 kg/cm^2 D_r<75% Saturated coarse grained soil with U_c<10 and 0.074mm<D_{50}<2.0 mm

Summary of Data — SPT

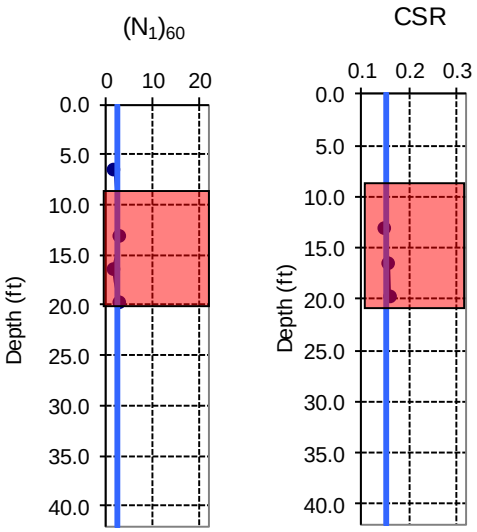
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.150 ± 0.050		0.15
Data Class	B	-		% Fines	72.5 ± 36.9	30.0	30
Critical Depth Range (ft)	8.0 - 20.0	14.1	10.0	% PI			
Depth to GWT (ft)	8.0 ± 0.4	7.9	8.0				
σ_v (psf)	1460.0 ± 222.1	1670.8	1120.0	N	1.5 ± 0.5	2.3	2
σ_v' (psf)	1085.6 ± 101.5	1274.0	1000.0	C_R	0.90	0.95	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.87 ± 0.068	0.99	0.980	C_B	1.00	1.00	
CSR	0.153 ± 0.048	0.144	0.145	C_E	1.17	1.17	1.17
Equivalent Magnitude	8.1	8.1		C_N	1.40	1.32	1.37
MSF		0.85	0.94	$(N_1)_{60}$	2.2 ± 0.8	3.4	3
CSRN		0.159	0.16				



γ_m=100±3 pcf

γ_s=110±3 pcf

(Ⅲ) IENAGA SHINDEN

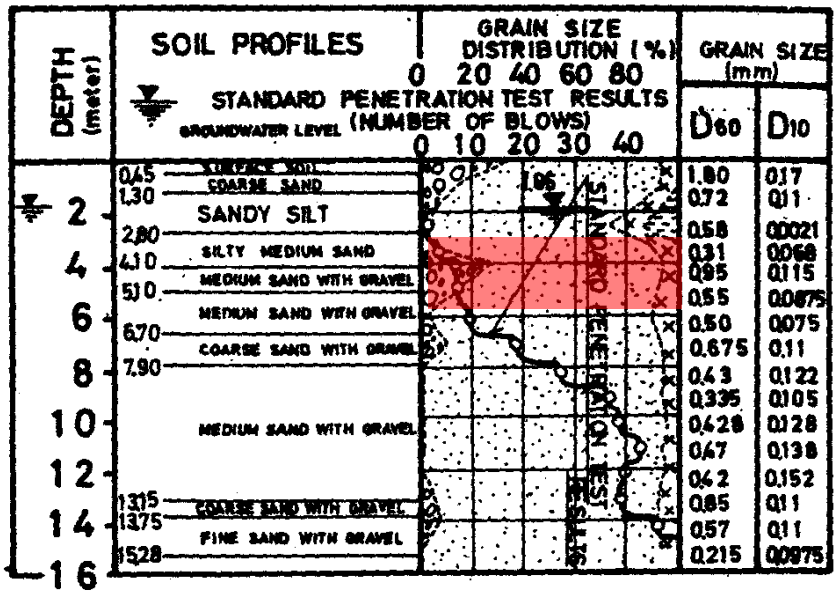


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	clay	silt	Depth (m)	D60	D10
2.0	6.6	NA	1	1.80	0.80	***1.7***	***99.0***	40.0	59.0	2.0	0.140	0.008
4.0	13.1	0.150	2	1.42	0.89	***2.9***	***83.0***	34.0	49.0	2.1	0.012	-
5.0	16.4	0.158	1	1.33	0.92	***1.4***	***90.0***	37.0	53.0	2.7	0.172	0.020
6.0	19.7	0.160	2	1.25	0.95	***2.8***	***18.0***	4.0	14.0	4.0	0.027	-
					Mean:	2.2	72.5			5.0	0.015	-
					St. Dev.	0.8	36.9			6.0	0.130	0.0087

Case number:	2
Earthquake:	1944 Tohnankai
Magnitude:	8.3 (M_R)
Location:	Komei
References:	Kishida (1969)
Nature of Failure:	The subsidence of ground and extensive damage to houses. Eruption of water during the earthquake.
Comments:	The epicenter of the earthquake is located 165 kms south-southwest of Nagoya city. Borelog was obtained from a site where subsidence of the ground and extensive damage to houses occurred as a result of liquefaction of the sandy layer. A budhist temple supported on piles did not show any settlement, but the ground around the temple subsided ~40 cm and water erupted during the earthquake. The seismic intensity of shaking was estimated as JMAIS V-VI. In the source document,the liquefied zone was predicted in the depth range of 2-7 m based on criterias as follow : Effective overburden pressure < 2.0 kg/cm2 D_r<75% Saturated coarse grained soil with U_c<10 and 0.074mm<D_{50}<2.0 mm Tip of the piles were at 8 m below ground surface. SPT energy was estimated as 70 % by Seed et al (1984).

Summary of Data SPT

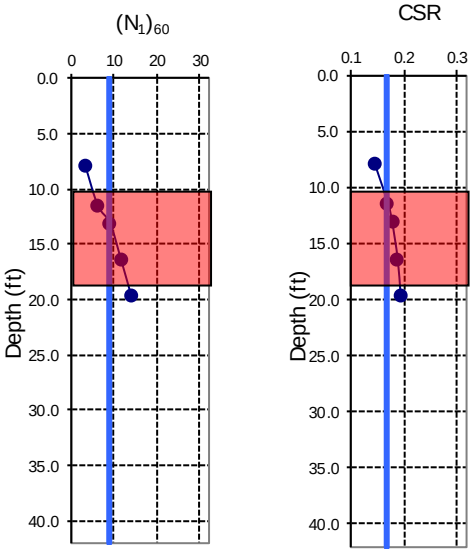
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.4
Data Class	B	-		% Fines	9.7 ± 1.5	10.0	10
Critical Depth Range	6.4 - 16.4	17.1	17.0	% PI			
Depth to GWT (ft)	6.4 ± 0.4	6.9	7.0				
σ_v (psf)	1214.8 ± 193.4	2046.8	2040.0	N	5.8 ± 1.7	5.9	8
σ_v' (psf)	904.6 ± 92.5	1420.2	1420.0	C_R	0.87	0.95	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.97 ± 0.057	0.98	0.960	C_B	1.00	1.00	
CSR	0.169 ± 0.053	0.182	0.180	C_E	1.17	1.17	1.17
Equivalent Magnitude	8.1	8.1		C_N	1.53	1.25	1.17
MSF		0.85	0.92	$(N_1)_{60}$	8.9 ± 2.7	8.2	11
CSRN		0.207	0.20				



(I) KOMEI TOWN

γ_m=100±3 pcf

γ_s=115±3 pcf

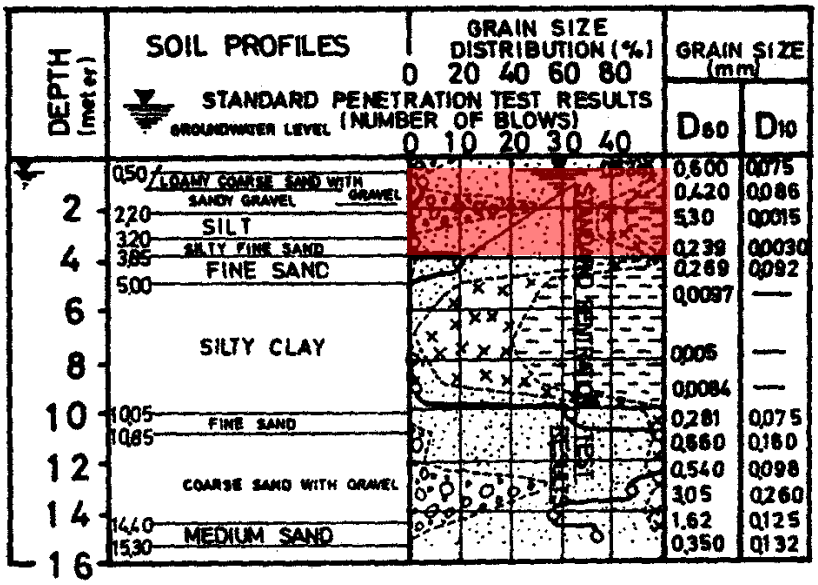


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	clay	silt	D60	D10
2.4	7.9	0.144	2	1.72	0.82	3.3	26.0	13.0	13.0	0.580	0.002
3.5	11.5	0.170	4	1.53	0.87	***6.2***	***10.0***	0.0	10.0	0.310	0.068
4.0	13.1	0.177	6	1.46	0.89	***9.1***	***8.0***	0.0	7.0	0.950	0.115
5.0	16.4	0.188	8	1.35	0.92	***11.6***	***11.0***	0.0	8.0	0.550	0.088
6.0	19.7	0.194	10	1.26	0.95	13.9	0.0	0.0	11.0	0.500	0.075
					Mean:	8.9	9.7				
					St. Dev.	2.7	1.5				

Case number:	3
Earthquake:	1944 Tohnankai
Magnitude:	8.3 (M_R)
Location:	Meiko
References:	Kishida (1969)
Nature of Failure:	" …very fine soil came out of the ground, and houses settled as much as about 1 meter"
Comments:	The epicenter of the earthquake is located 165 kms south-southwest of Nagoya city. The seismic intensity of sahking is estimated as JMAIS V-VI Kishida (1969) predicted the liquefied zone to be from 0.5-5 m based on following criteria : Effective overburden pressure<2.0 kg/cm^2 D_r<75% Saturated coarse grained soil with U_c<10 and 0.074mm<D_{50}<2.0 mm.

Summary of Data SPT

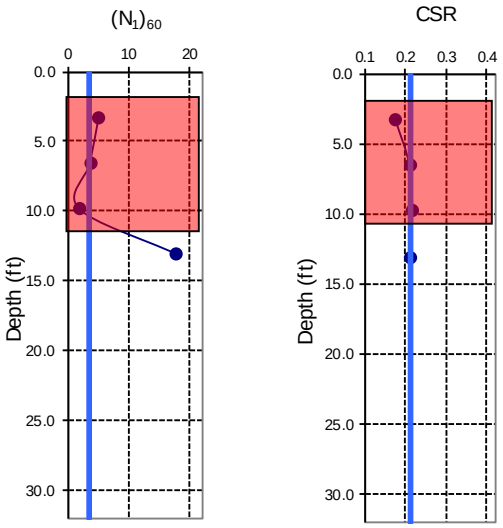
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.200 ± 0.050		0.2
Data Class	B			% Fines	19.3 ± 10.8	27.0	27
Critical Depth Range	1.6 - 11.5	12.1	12.0	% PI			
Depth to GWT (ft)	1.6 ± 0.3	6.9	7.0				
σ_v (psf)	703.2 ± 181.9	1441.1	1440.0	N	2.0 ± 0.9	1.0	1
σ_v' (psf)	397.3 ± 81.5	814.5	820.0	C_R	0.80	0.85	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.92 ± 0.036	0.99	0.980	C_B	1.00	1.00	
CSR	0.212 ± 0.066	0.225	0.225	C_E	1.17	1.17	1.17
Equivalent Magnitude	8.1	8.1		C_N	2.00	1.70	1.49
MSF		0.85	0.94	$(N_1)_{60}$	3.6 ± 1.6	1.7	1.5
CSRN		0.245	0.24				



γ_m=100±3 pcf

γ_s=110±3 pcf

(II) MEIKO STREET

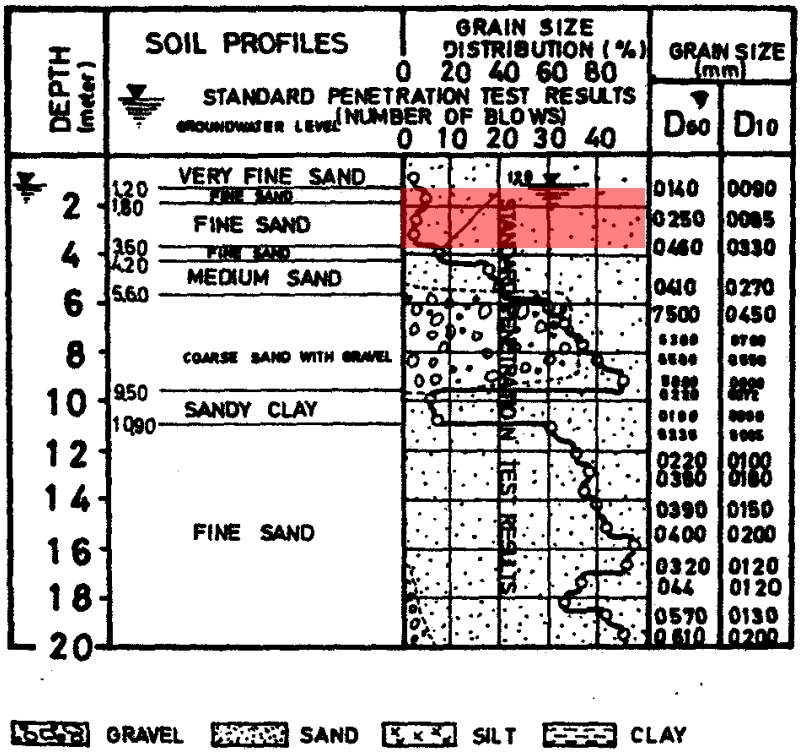


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	clay	silt	D60	D10
1.0	3.3	0.178	3	2.00	0.73	***5.1***	***7.0***	0	7	0.420	0.085
2.0	6.6	0.212	2	2.00	0.80	***3.7***	***27.0***	15	12	5.300	0.002
3.0	9.8	0.218	1	1.95	0.85	***1.9***	***24.0***	10	14	0.238	0.003
4.0	13.1	0.214	10	1.73	0.89	17.9	18.0	6	12	0.269	0.082
					Mean:	3.6	19.3				
					St. Dev.	1.6	10.8				

Case number:	4
Earthquake:	1948 Fukui
Magnitude:	7.0 (Mw) USGS Centennial Earthquake Catalog
Location:	Shonenji Temple
References:	Kishida (1969) Hamada et al (1989)
Nature of Failure:	"… eruption of water and sand volcanoes were quite promonent, and the main building of the temple settled 0.30 m"
Comments:	The epicenter of the earthquake is located 5 kms east of Fukui City. The seismic intensity of shaking was estimated as JMAIS V-VI. A PGA value of 0.4 g was adopted by Seed et al (84) Shonenji Temple and Agricultural Union sites are 500 m apart. Kishida (1969) predicted the critical zone to be from 1-4 m based on: Effective overburden pressure<2.0 kg/cm2 D_r<75% Saturated coarse grained soil with U_c<10 and 0.074mm<D_{50}<2.0 mm SPT energy was estimated as 70 % by Seed et al. (84)

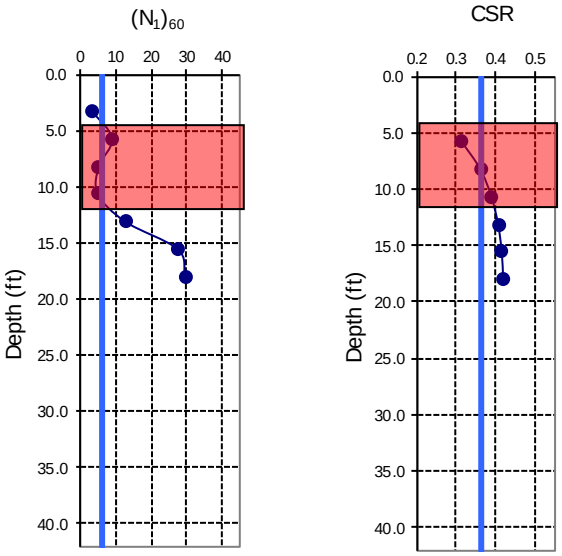
Summary of Data SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.45
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	3.9 - 13.0	13.1	13.0	% PI			
Depth to GWT (ft)	3.9 ± 0.3	3.9	4.0				
σ_v (psf)	912.7 ± 175.4	1566.4	1560.0	N	3.7 ± 1.3	8.0	8
σ_v' (psf)	631.1 ± 83.0	1002.5	1000.0	C_R	0.83	0.85	
a_{max} (g)	0.400 ± 0.120	0.40	0.400	C_S	1.00	1.00	
r_d	0.97 ± 0.044	0.96	0.970	C_B	1.00	1.00	
CSR	0.366 ± 0.114	0.390	0.395	C_E	1.17	1.17	1.17
Equivalent Magnitude	7.0	7.0		C_N	1.83	1.48	1.37
MSF		1.14	1.04	$(N_1)_{60}$	6.5 ± 2.3	11.8	13
CSRN		0.318	0.38				



γ_m=100±3 pcf

γ_s=115±3 pcf

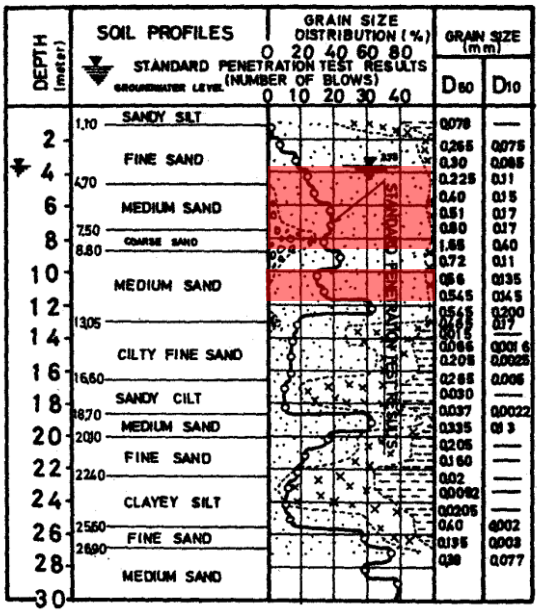


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	D60	D10
1.0	3.3	NA	2	2.00	0.73	3.4	0.14	0.01
1.8	5.7	0.315	5	2.00	0.78	***9.1***	0.23	0.07
2.5	8.2	0.363	3	1.85	0.82	***5.3***	0.46	0.33
3.3	10.7	0.391	3	1.68	0.86	***5.0***	0.41	0.27
4.0	13.1	0.408	8	1.55	0.89	12.9	7.50	0.45
4.8	15.6	0.418	18	1.45	0.91	27.8	-	-
5.5	18.0	0.422	20	1.37	0.94	29.8	-	-
					Mean:	6.5		
					St. Dev.	2.3		

Case number:	5
Earthquake:	1948 Fukui
Magnitude:	7.0 (Mw) USGS Centennial Earthquake Catalog
Location:	Takaya 45
References:	Kishida (1969) Hamada et al (1989)
Nature of Failure:	Sand volcanoes were observed
Comments:	The epicenter of the earthquake is located 5 kms east of Fukui City. The seismic intensity of shaking is estimated as JMAIS V-VI. A PGA value 0.35 g was adopted by Seed et al (84) A paddy place, quickly draining. Takaya 2 and 45 are approximately parallel to Kuzuryu river. Kishida (1969) predicted the critical zone to be from 2-13 m based on: Effective overburden pressure<2.0 kg/cm2 D_r<75% Saturated coarse grained soil with U_c<10 and 0.074mm<D_{50}<2.0 mm SPT energy was estimated as 78 % by Seed et al. (84)

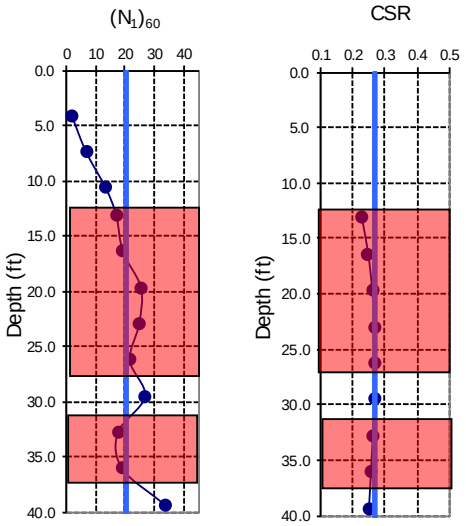
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.500 ± 0.050		0.65
Data Class	B			% Fines	3.3 ± 1.3	4.0	4
Critical Depth Range	12.3 - 40.0	24.6	23.0	% PI			
Depth to GWT (ft)	12.3 ± 0.3	12.1	12.0				
σ_v (psf)	3084.3 ± 579.8	2944.8	2760.0	N	16.2 ± 2.7	17.3	19
σ_v' (psf)	2220.0 ± 294.6	2172.1	2070.0	C_R	1.00	0.95	
a_{max} (g)	0.350 ± 0.105	0.35	0.350	C_S	1.00	1.00	
r_d	0.85 ± 0.115	0.90	0.950	C_B	1.00	1.00	
CSR	0.270 ± 0.090	0.283	0.290	C_E	1.30	1.30	1.30
Equivalent Magnitude	7.0	7.0		C_N	0.98	0.99	0.98
MSF		1.14	1.04	$(N_1)_{60}$	20.6 ± 3.4	21.1	24
CSRN		0.251	0.28				



110±3 pcf

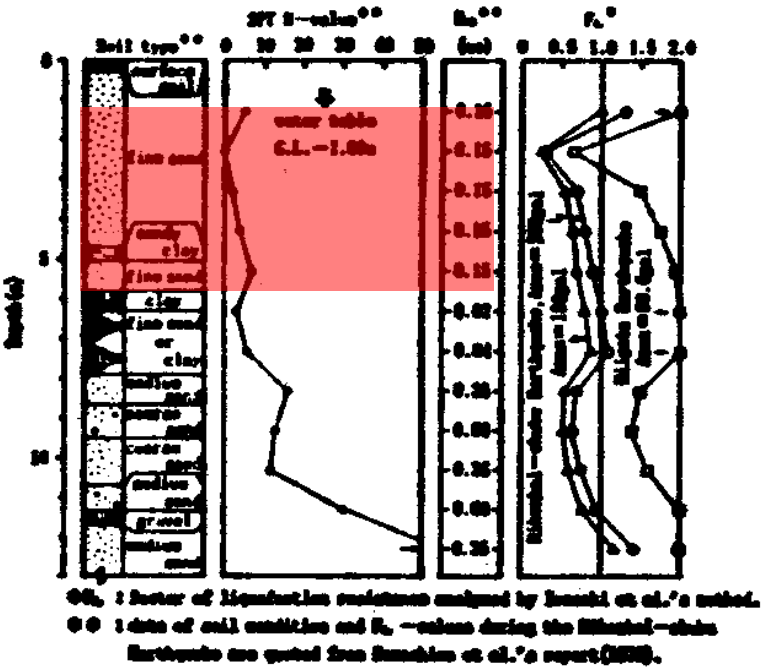
125±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	silt	clay	gravel	D60	D10
1.3	4.1	NA	1	2.00	0.75	2.0	52.0	36	16	0	0.078	-
2.3	7.4	NA	4	1.61	0.81	6.8	8.0	8	0	0	0.265	0.075
3.3	10.7	NA	9	1.34	0.86	13.5	7.0	7	0	0	0.300	0.085
4.0	13.1	0.226	12	1.23	0.89	***17.0***	3.0	3	0	0	0.225	0.510
5.0	16.4	0.248	14	1.15	0.92	***19.2***	3.0	3	0	4	0.400	0.150
6.0	19.7	0.261	19	1.08	0.95	***25.4***	5.0	5	0	6	0.510	0.170
7.0	23.0	0.268	19	1.02	0.98	***24.7***	2.0	2	0	9	0.600	0.170
8.0	26.2	0.270	17	0.98	1.00	***21.5***	2.0	2	0	27	1.650	0.400
9.0	29.5	0.268	22	0.93	1.00	26.7	5.0	5	0	12	0.720	0.310
10.0	32.8	0.263	15	0.90	1.00	***17.5***	5.0	5	0	0	0.560	0.135
11.0	36.1	0.257	17	0.86	1.00	***19.1***	3.0	3	0	0	0.545	0.145
12.0	39.4	0.250	31	0.83	1.00	33.6	2.0	2	0	0	0.545	0.200
					Mean:	20.6	3.3					
					St. Dev.	3.4	1.3					

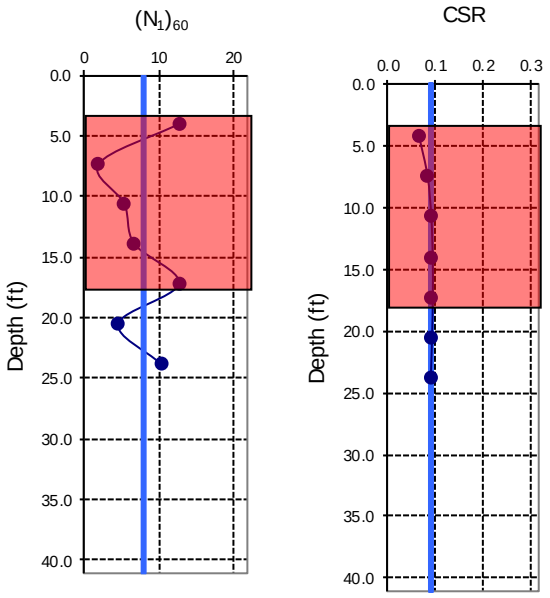
Case number:	6
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Aki City (Arayamotomachi)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils
Comments:	The epicenter of the earthquake is located some 35 miles north of the city. At Arayamotomachi Town in Akita City, liquefaction was caused in a long and narrow area by Niigata Earthquake. The damage was concentrated in a small zone about 100 m in width and 500 m in length, in an area of land recently reclaimed over an old river by filling with sand from adjacent sand dunes. At a point near the center of the liquefied area, a layer of very loose fine sand, with SPT N values of 0-6, forms a deposit extending from the ground surface to about 6 m in depth. The water table is at 1 m depth. The maximum surface acceleration was assumed to be 0.09 g based on the value recorded at the Akita Prefectural Government building 4 km from Arayamotomachi Town.
Summary of Data	SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.150 ± 0.050		
Data Class	A			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	3.3 - 18.0	10.8		% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.3					
σ_v (psf)	1177.0 ± 284.0	1315.8		N	4.4 ± 2.7	2.6	
σ_v' (psf)	716.4 ± 132.4	856.3		C_R	0.86	0.85	
a_{max} (g)	0.090 ± 0.014	0.09		C_S	1.00	1.00	
r_d	0.94 ± 0.054	0.98		C_B	1.00	1.00	
CSR	0.090 ± 0.016	0.089		C_E	1.22	1.22	
Equivalent Magnitude	7.6	7.6		C_N	1.72	1.70	
MSF		0.97		$(N_1)_{60}$	7.9 ± 4.8	4.7	
CSRN		0.086					



γ_m=100±3 pcf

γ_s=115±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.3	4.1	0.065	7.0	2.00	0.75	***12.8***	fine sand
2.3	7.4	0.083	1.0	1.97	0.81	***1.9***	fine sand
3.3	10.7	0.090	3.0	1.72	0.86	***5.4***	fine sand
4.3	13.9	0.093	4.0	1.54	0.90	***6.7***	fine sand
5.3	17.2	0.093	8.0	1.41	0.93	***12.8***	fine sand
6.3	20.5	0.092	3.0	1.31	0.96	4.6	clay seam
7.3	23.8	0.089	7.0	1.23	0.98	10.3	fine sand or clay
					Mean:	7.9	
					St. Dev.	4.8	

Case number:	7
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Niigata Cc17-1
References:	Kishida (1966) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	Along Shinano River many buildings tilted or subsided constructed on reclaimed land. Sand particles at ~ 5-7 m depth came out; sand volcanoes at various places.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has bulit up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depths. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. The epicenter of the earthquake is located 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was in the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. SPT values were taken both before and after the earthquake. N values before the earthquake were used for the analysis. SPT energy was estimated as 65 % by Seed et al. (84)

Summary of Data SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.200 ± 0.050		0.3
Data Class	B			% Fines	2.0 ± 2.0	2.0	2
Critical Depth Range	16.4 - 36.1	23.0	23.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	3089.6 ± 400.0	2756.9	2760.0	N	8.8 ± 2.3	8.0	8
σ_v' (psf)	1639.0 ± 202.0	1503.8	1510.0	C_R	1.00	0.95	
a_{max} (g)	0.160 ± 0.024	0.16	0.160	C_S	1.00	1.00	
r_d	0.80 ± 0.116	0.94	0.950	C_B	1.00	1.00	
CSR	0.157 ± 0.033	0.179	0.180	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	1.14	1.20	1.14
MSF		0.97	1.00	$(N_1)_{60}$	10.9 ± 2.9	9.9	10
CSRN		0.178	0.18				

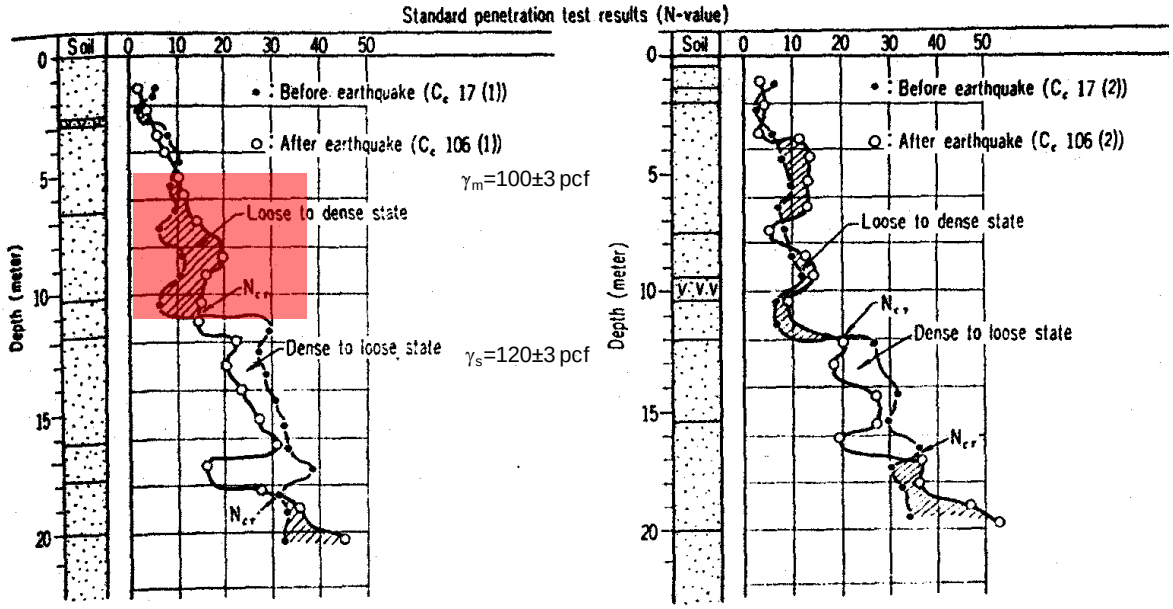
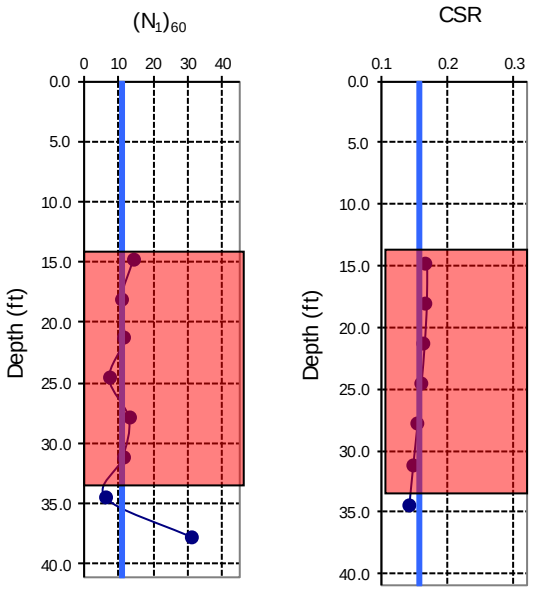


Fig. 5. Typical examples of standard penetration test results before and after Niigata earthquake (Location of boring is shown in Fig. 3)



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4.5	14.8	0.168	10	1.47	0.90	***14.5***
5.5	18.0	0.167	8	1.35	0.94	***11.0***
6.5	21.3	0.164	9	1.25	0.96	***11.8***
7.5	24.6	0.160	6	1.17	0.99	***7.6***
8.5	27.9	0.154	11	1.10	1.00	***13.3***
9.5	31.2	0.148	10	1.05	1.00	***11.5***
10.5	34.4	0.143	6	1.00	1.00	***6.6***
11.5	37.7	0.138	30	0.96	1.00	31.4
					Mean:	10.9
					St. Dev.	2.9

Case number:	8
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Niigata Cc17-2
References:	Kishida (1966) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	In the Shinano River reclaimed land, many buildings tilted or subsided. The sand particles at ~ 5-7 m depth came out; sand volcanoes at various places.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea. The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. The epicenter of the earthquake is located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was in the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. SPT values were taken both before and after the earthquake. N values before the earthquake were used for the analysis. SPT energy was estimated as 65 % by Seed et al. (84)

Summary of Data SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.200 ± 0.050		
Data Class	A			% Fines	2.0 ± 2.0	8.0	
Critical Depth Range	11.5 - 23.0	17.4		% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0					
σ_v (psf)	2006.9 ± 233.8	1775.3		N	8.0 ± 1.4	7.9	
σ_v' (psf)	1119.3 ± 119.2	898.1		C_R	0.93	0.95	
a_{max} (g)	0.160 ± 0.024	0.16		C_S	1.00	1.00	
r_d	0.90 ± 0.081	0.96		C_B	1.00	1.00	
CSR	0.168 ± 0.030	0.199		C_E	1.09	1.09	
Equivalent Magnitude	7.6	7.6		C_N	1.37	1.55	
MSF		0.97		$(N_1)_{60}$	11.1 ± 1.9	12.7	
CSRN		0.188					

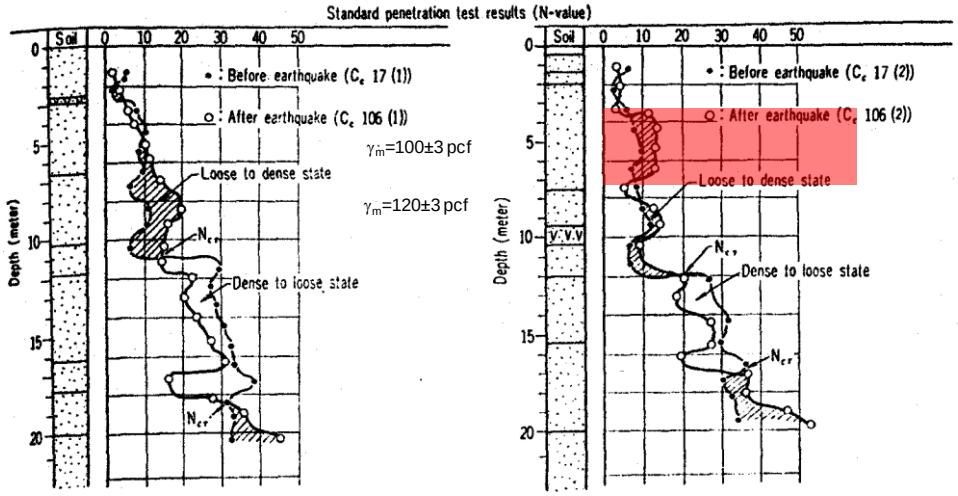
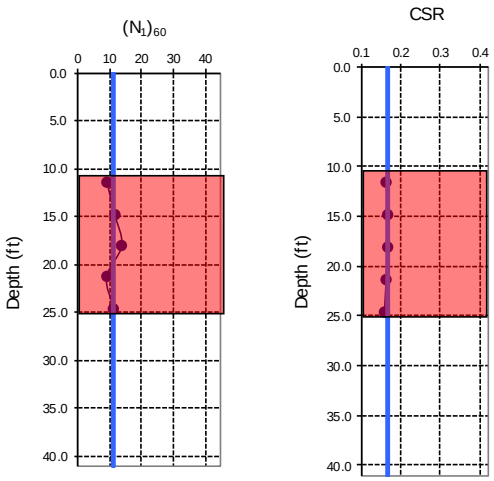


Fig. 5. Typical examples of standard penetration test results before and after Niigata earthquake (Location of boring is shown in Fig. 3)



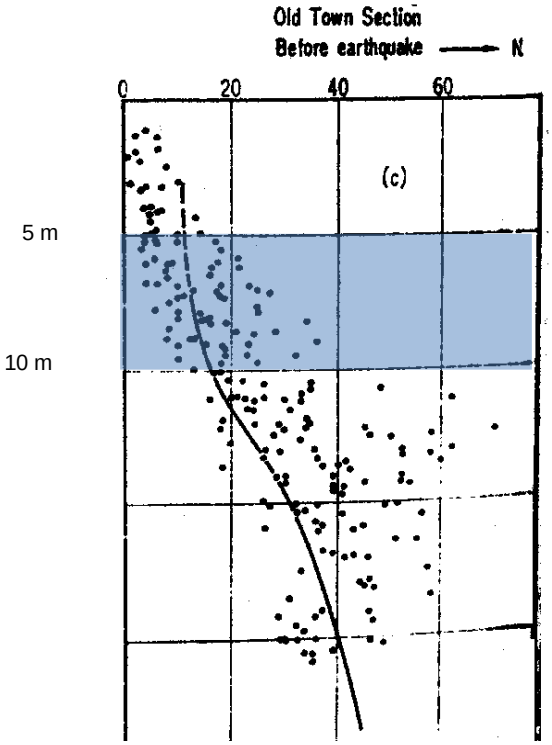
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.5	11.5	0.165	6	1.64	0.87	***9.3***
4.5	14.8	0.168	8	1.47	0.90	***11.6***
5.5	18.0	0.167	10	1.35	0.94	***13.8***
6.5	21.3	0.164	7	1.25	0.96	***9.2***
7.5	24.6	0.160	9	1.17	0.99	***11.4***
					Mean:	11.1
					St. Dev.	1.9

Despite the fact that it is sligthly below the bottom of the critical depth range, due to consistent trend in N values, N=9 is used to characterize the critical layer.

Case number:	9
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Old Town-1
References:	Koizumi (1964) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	No surface evidence of liquefaction has been observed at Old Town section.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was ditted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was in the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. Seed et al (84) estimated PGA not as 0.16 but 0.18 g probably due to the fact the site did not liquefy so no de-amplification. Seed et al (84) adopted a rod energy of 73 % (average Japanese energy value) for non-liquefied sites and 65 % (Japanese lower bound) for liquefied sites. Water table was at 1-2 m except dune sand. SPT values were taken before the earthquake. SPT energy was estimated as 73 % by Seed et al. (84)

Summary of Data SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.200 ± 0.050		0.3
Data Class	A			% Fines	2.0 ± 2.0	2.0	2
Critical Depth Range	16.4 - 32.8	23.0	23.0	% PI			
Depth to GWT (ft)	6.0 ± 0.3	5.9	6.0				
σ_v (psf)	2985.8 ± 346.8	2756.9	2760.0	N	17.4 ± 0.5	18.0	18
σ_v' (psf)	1824.8 ± 181.5	1691.7	1700.0	C_R	0.99	0.95	
a_{max} (g)	0.180 ± 0.027	0.18	0.180	C_S	1.00	1.00	
r_d	0.87 ± 0.110	0.94	0.950	C_B	1.00	1.00	
CSR	0.167 ± 0.033	0.179	0.180	C_E	1.21	1.21	1.21
Equivalent Magnitude	7.6	7.6		C_N	1.08	1.10	1.08
MSF		0.97	1.00	$(N_1)_{60}$	22.3 ± 0.7	22.7	23.5
CSRN		0.178	0.18				

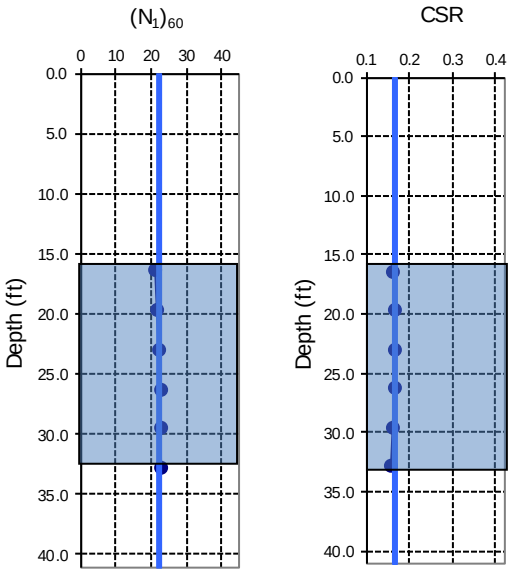


5 m

10 m

γ_m=110±3 pcf

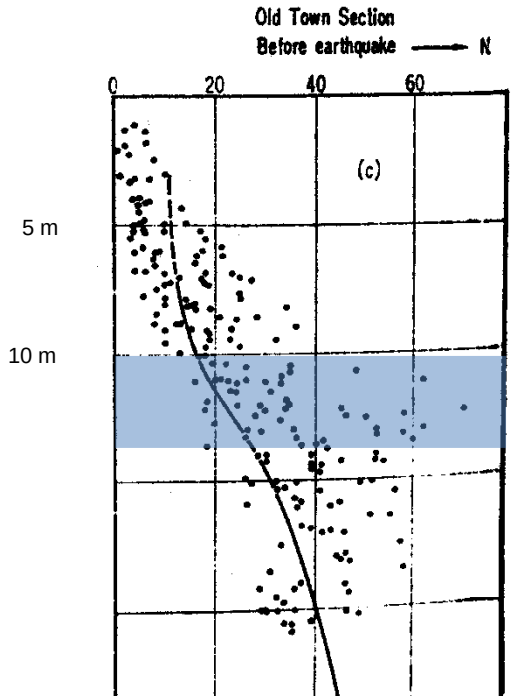
γ_s=125±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.0	16.4	0.164	15	1.27	0.92	***21.2***
6.0	19.7	0.167	16	1.18	0.95	***21.7***
7.0	23.0	0.168	17	1.11	0.98	***22.2***
8.0	26.2	0.166	18	1.05	1.00	***22.8***
9.0	29.5	0.163	19	1.00	1.00	***22.9***
10.0	32.8	0.159	20	0.95	1.00	***23.0***
					Mean:	22.3
					St. Dev.	0.7

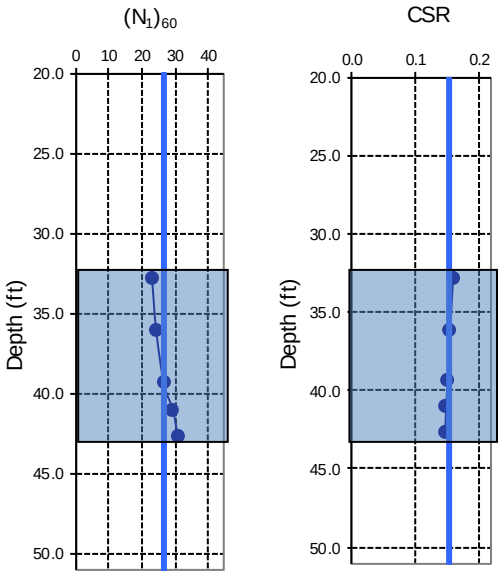
Case number:	10
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Old Town-2
References:	Koizumi (1964) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	No surface evidence of liquefaction has been observed at Old Town section.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was dotted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was in the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. Seed et al (84) estimated PGA not as 0.16 but 0.18 g probably due to the fact the site did not liquefy so no de-amplification. Seed et al (84) adopted a rod energy of 73 % (average Japanese energy value) for non-liquefied sites and 65 % (Japanese lower bound) for liquefied sites. Water table was at 1-2 m except dune sand. SPT values were taken before the earthquake. SPT energy was estimated as 73 % by Seed et al. (84)
Summary of Data	SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.200 ± 0.050		0.3
Data Class	B			% Fines	2.0 ± 2.0	2.0	2
Critical Depth Range	32.8 - 42.7	33.1	33.0	% PI			
Depth to GWT (ft)	6.0 ± 0.3	5.9	6.0				
σ_v (psf)	4626.2 ± 226.8	3968.2	3960.0	N	24.7 ± 3.0	20.0	20
σ_v' (psf)	2646.3 ± 141.9	2276.5	2270.0	C_R	1.00	1.00	
a_{max} (g)	0.180 ± 0.027	0.18	0.180	C_S	1.00	1.00	
r_d	0.74 ± 0.158	0.90	0.910	C_B	1.00	1.00	
CSR	0.152 ± 0.040	0.184	0.185	C_E	1.21	1.21	1.21
Equivalent Magnitude	7.6	7.6		C_N	0.89	0.97	0.94
MSF		0.97	1.00	$(N_1)_{60}$	26.7 ± 3.2	23.5	22.5
CSRN		0.191	0.19				



γ_m=110±3 pcf

γ_s=125±3 pcf

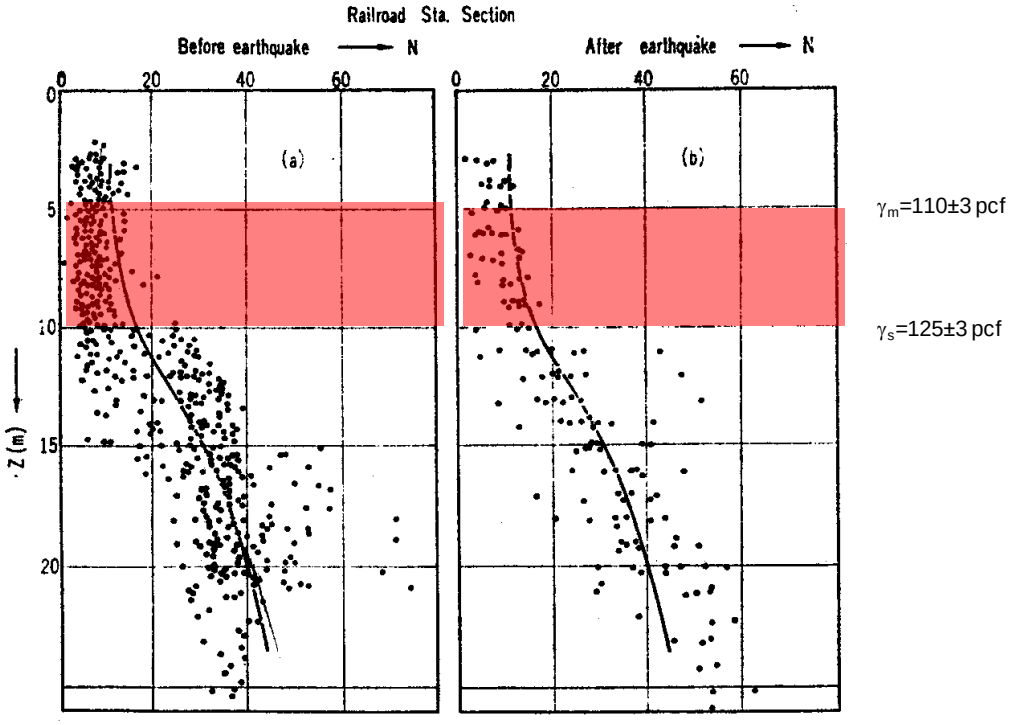


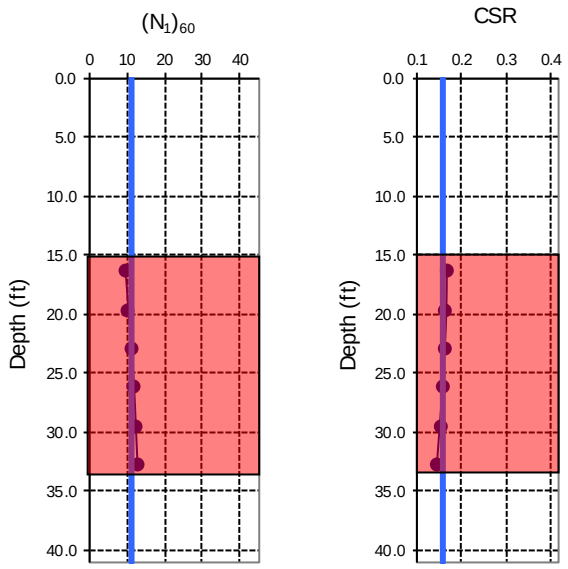
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
10.0	32.8	0.159	20	0.95	1.00	***23.0***
11.0	36.1	0.154	22	0.91	1.00	***24.2***
12.0	39.4	0.150	25	0.88	1.00	***26.5***
12.5	41.0	0.148	28	0.86	1.00	***29.1***
13.0	42.7	0.146	30	0.85	1.00	***30.7***
					Mean:	26.7
					St. Dev.	3.2

Case number:	11
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Rail Road-1
References:	Koizumi (1964) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	The SPT measurements before and after the earthquake are used for predicting liquefaction in conjunction with the surface evidence of it.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was dotted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The epicenter of the earthquake is located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. Seed et al (84) adopted a rod energy of 73 % (average Japanese energy value) for non-liquefied sites and 65 % (Japanese lower bound) for liquefied sites. Water table was at 1-2 m except dune sand. SPT values were taken before and after the earthquake. SPT energy % was estimated as 65 % by Seed et al. (84)

Summary of Data SPT

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.200 ± 0.050		0.3
Data Class	A			% Fines	2.0 ± 2.0	2.0	2
Critical Depth Range	16.4 - 32.8	33.1	33.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	3030.8 ± 348.0	3968.2	3960.0	N	9.4 ± 1.1	10.0	10
σ_v' (psf)	1682.6 ± 183.8	2088.5	2090.0	C_R	0.99	1.00	
a_{max} (g)	0.160 ± 0.024	0.16	0.160	C_S	1.00	1.00	
r_d	0.85 ± 0.110	0.90	0.910	C_B	1.00	1.00	
CSR	0.160 ± 0.032	0.178	0.180	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	1.12	1.01	0.98
MSF		0.97	1.00	$(N_1)_{60}$	11.3 ± 1.3	11.0	10.5
CSRN		0.182	0.18				



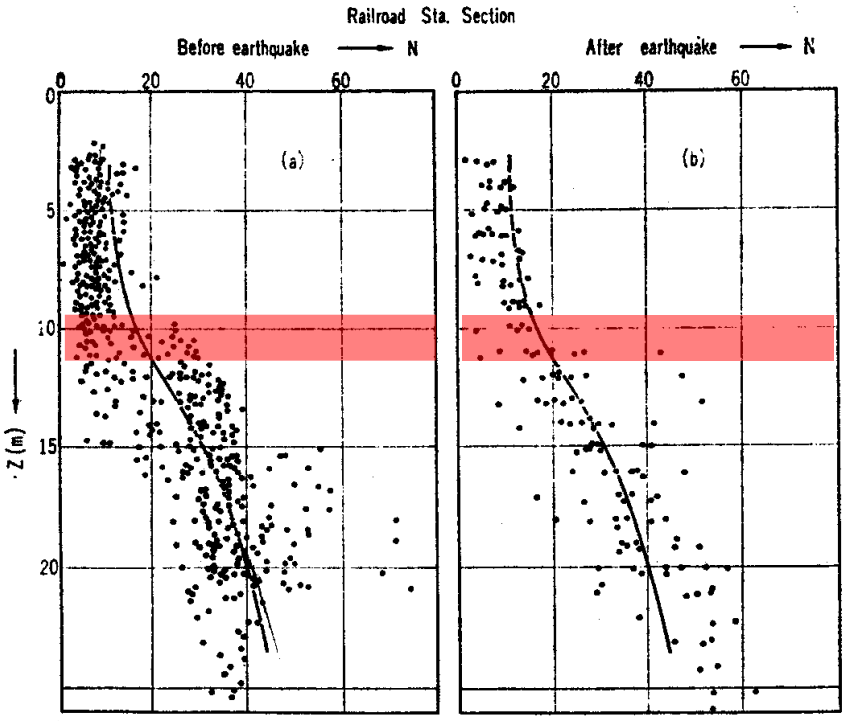


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.0	16.4	0.166	7	1.35	0.92	***9.5***
6.0	19.7	0.165	8	1.24	0.95	***10.3***
7.0	23.0	0.162	9	1.16	0.98	***11.1***
8.0	26.2	0.158	10	1.09	1.00	***11.9***
9.0	29.5	0.153	11	1.03	1.00	***12.4***
10.0	32.8	0.147	12	0.98	1.00	***12.9***
					Mean:	11.3
					St. Dev.	1.3

Case number:	12
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Rail Road-2
References:	Koizumi (1964) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	The SPT measurements before and after the earthquake were used for predicting liquefaction in conjunction with the surface evidence of it.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was ditted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. Seed et al (84) adopted a rod energy of 73 % (average Japanese energy value) for non-liquefied sites and 65 % (Japanese lower bound) for liquefied sites. Water table was at 1-2 m except dune sand. SPT values were taken before and after the earthquake. SPT energy was estimated as 65 % by Seed et al. (84)

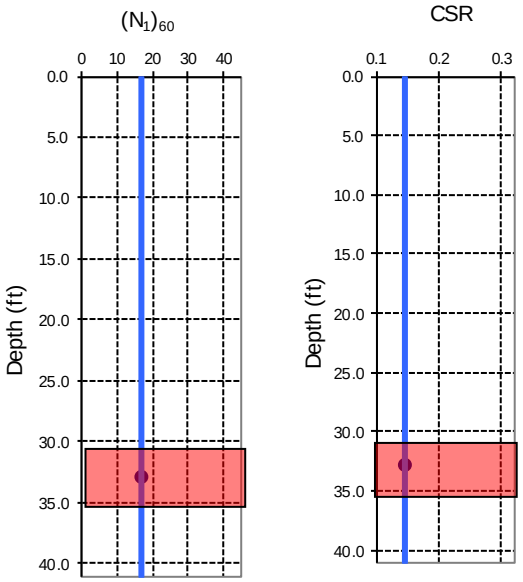
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No/Yes	Marginal	No/Yes	D_{50}	0.200 ± 0.050		0.3
Data Class	B		C	% Fines	2.0 ± 2.0	2.0	2
Critical Depth Range	29.5 - 36.1	33.1	33.0	% PI			
Depth to GWT (ft)	3.0 ± 1.0	3.0	3.0				
σ_v (psf)	4056.0 ± 164.3	3968.2	3960.0	N	16.0 ± 1.9	16.0	16
σ_v' (psf)	2196.0 ± 122.5	2088.5	2090.0	C_R	1.00	1.00	
a_{max} (g)	0.160 ± 0.024	0.16	0.160	C_S	1.00	1.00	
r_d	0.77 ± 0.140	0.90	0.910	C_B	1.00	1.00	
CSR	0.147 ± 0.035	0.178	0.180	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	0.98	1.01	0.98
MSF		0.97	1.00	$(N_1)_{60}$	17.1 ± 2.0	17.5	17
CSRN		0.182	0.18				



γ_m=110±3 pcf

γ_s=125±3 pcf

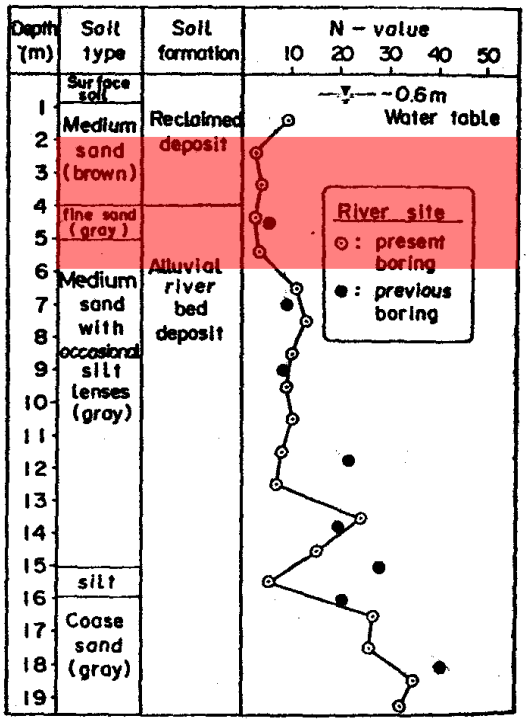


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
10.0	32.8	0.147	16	0.98	1.00	***17.1***
					Mean:	17.1
					St. Dev.	2.0

Case number:	13
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	River Site
References:	Ishihara et. al. (1979) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	Surface cracking and ejection of sand and water; flooding of sand and water over the ground surface.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was ditted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The River Site was located ~ 10 m from the bank of the Shinano river. The site was reclaimed by in 1955 by constructing a dike along the river channel. The top 1 m consists of uncompacted fill dumped in air. Undensified fine sand dumped through water between depth of 1-3m. From 3-5m overconsolidated medium sand deposits. Fluvial river deposits consisting of medium sand with occasional silt lenses. Below 15 m a silt layer underlain by uniform coarse sand. The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. SPT values were taken before and after the earthquake. SPT energy was estimated as 65 % by Seed et al. (84)

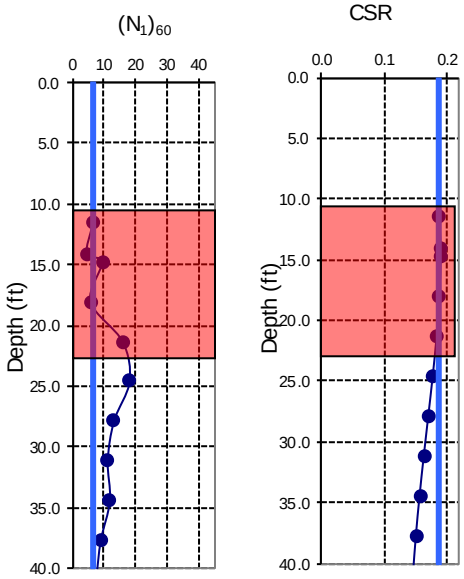
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.4
Data Class	A			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	6.6 - 19.7	15.1	15.0	% PI			
Depth to GWT (ft)	2.0 ± 0.3	2.0	2.0				
σ_v (psf)	1423.9 ± 243.0	1796.1	1800.0	N	4.0 ± 1.2	6.0	6
σ_v' (psf)	727.8 ± 110.6	981.6	990.0	C_R	0.89	0.95	
a_{max} (g)	0.160 ± 0.024	0.16	0.160	C_S	1.00	1.00	
r_d	0.93 ± 0.064	0.97	0.970	C_B	1.00	1.00	
CSR	0.189 ± 0.032	0.183	0.185	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	1.71	1.52	1.38
MSF		0.97	1.00	$(N_1)_{60}$	6.6 ± 1.9	9.4	9
CSRN		0.176	0.19				



γ_m=100±3 pcf

γ_s=110±3 pcf

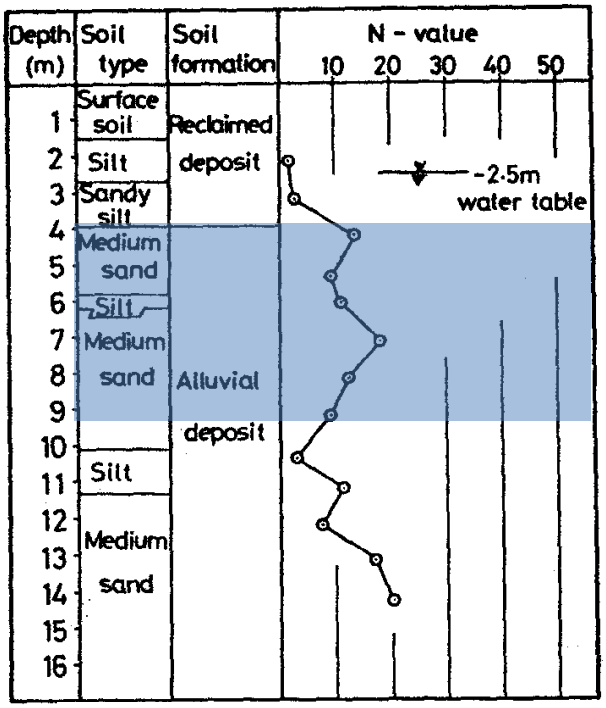


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	0.179	3	2.00	0.82	***5.4***
3.5	11.5	0.188	4	1.80	0.87	***6.8***
4.3	14.1	0.190	3	1.65	0.90	***4.9***
4.5	14.8	0.190	6	1.62	0.90	***9.6***
5.5	18.0	0.188	4	1.48	0.94	***6.1***
6.5	21.3	0.183	11	1.38	0.96	*15.9*
7.5	24.6	0.177	13	1.29	0.99	*18.1*
8.5	27.9	0.171	10	1.22	1.00	*13.3*
9.5	31.2	0.164	9	1.15	1.00	*11.3*
10.5	34.4	0.157	10	1.10	1.00	*12.0*
11.5	37.7	0.152	8	1.06	1.00	*9.2*
12.5	41.0	0.147	7	1.01	1.00	*7.8*
					Mean:	6.6
					St. Dev.	1.9

Case number:	14
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Road Site
References:	Ishihara et. al. (1979) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	No surface evidence of liquefaction
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was ditted with several small islands. The EQ was the first large vibration the recent deposits had experienced. The Road Site was located on level ground adjacent to the road ~ 350 m from the Shinano River bank. The top 1.5 m consists of surface soil which is underlain to a depth of 4 m by silt and sandy silt. Below 4 m the soil profile consists of alternate layers of medium sand and silt to a depth of 16 m. The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. Seed et al (84) estimated PGA for the site a little higher (0.18 g) than the recorded one probably due to no de-amplification at non-liquefied sites. Seed et al (84) adopted a rod energy of 73 % (average Japanese energy value) for non-liquefied sites and 65 % (Japanese lower bound) for liquefied sites. SPT values were taken before the earthquake. SPT energy was estimated as 65 % by Seed et al. (84)

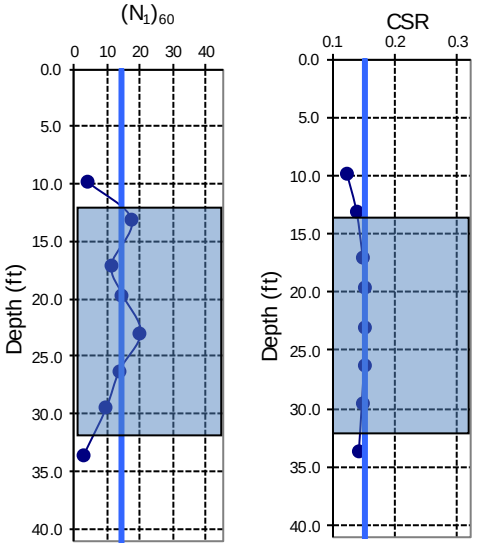
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.360 ± 0.129		0.36
Data Class	A			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	13.1 - 29.5	20.0	20.0	% PI			
Depth to GWT (ft)	8.2 ± 0.4	7.9	8.0				
σ_v (psf)	2460.6 ± 345.0	2401.8	2400.0	N	12.1 ± 3.2	12.0	12
σ_v' (psf)	1641.7 ± 177.8	1649.9	1650.0	C_R	0.96	0.95	
a_{max} (g)	0.180 ± 0.027	0.18	0.180	C_S	1.00	1.00	
r_d	0.88 ± 0.097	0.95	0.960	C_B	1.00	1.00	
CSR	0.154 ± 0.029	0.162	0.165	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	1.14	1.13	1.09
MSF		0.97	1.00	$(N_1)_{60}$	14.4 ± 3.8	14.1	14.5
CSRN		0.162	0.17				



γ_m=100±3 pcf

γ_s=125±3 pcf

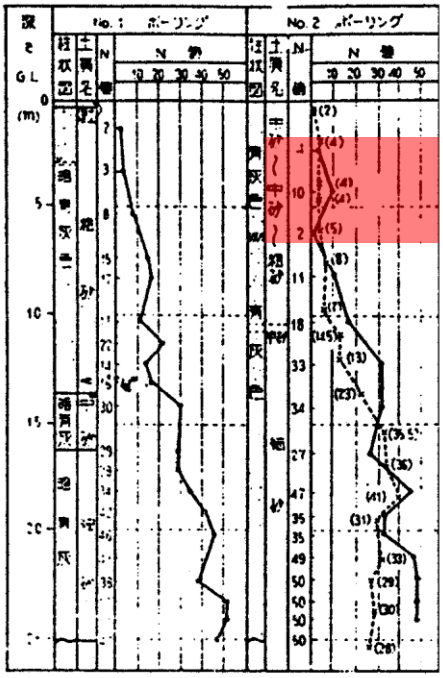


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.0	9.8	0.125	3	1.51	0.85	4.2
4.0	13.1	0.141	13	1.37	0.89	***17.2***
5.2	17.1	0.150	9	1.24	0.93	***11.3***
6.0	19.7	0.153	12	1.17	0.95	***14.6***
7.0	23.0	0.154	17	1.10	0.98	***20.0***
8.0	26.2	0.152	12	1.04	1.00	***13.6***
9.0	29.5	0.149	9	0.99	1.00	***9.7***
10.3	33.6	0.144	3	0.94	1.00	3.1
					Mean:	14.4
					St. Dev.	3.8

Case number:	15
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Showa Br 2
References:	Takada et al (1965) in Japanese Ishihara et. al. (1979) Fear et al. (1995) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	The deck spans G3 to G7 collapsed.The left end of the bridge was separated from the bank; piles in the piers deformed.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was ditted with several small islands. The EQ was the first large vibration the recent deposits had experienced. Showa Bridge experienced damage during Niigata Earthquake. Coarse Sand Layer : the most recent strata; approximately 13 m thick at the left bank, decreasing to ~ 5 m at the right bank; a mixture of silt, loam, small gravel and organic material. Medium Sand Layer: 15 m thick at the right bank (top of strta just more than 5m deep) increasing in depth and decreasing in thickness from the right to left bank; approximately 3 m thick at the left bank; some organic material, coarse sand, fine sand and iron sand, causing variations in N. Fine Sand Layer: Deeper than 16-18 m; dark blue gray color; some parts contain silt and iron sand; N>30 The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. SPT values were taken before and after the earthquake. SPT energy was estimated as 65 % by Seed et al. (84)

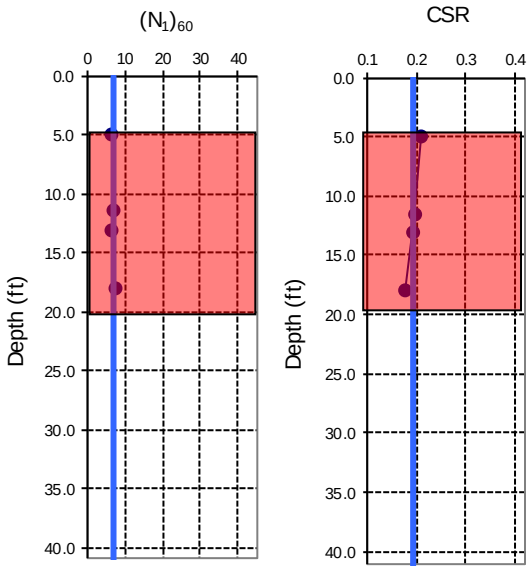
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.4
Data Class	A			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	4.5 - 20.0	14.1	14.0	% PI			
Depth to GWT (ft)	0.0 ± 0.3	0.0	0.0				
σ_v (psf)	1470.0 ± 312.2	1670.8	1680.0	N	4.1 ± 0.2	4.0	4
σ_v' (psf)	705.6 ± 154.4	814.5	810.0	C_R	0.88	0.95	
a_{max} (g)	0.160 ± 0.024	0.16	0.160	C_S	1.00	1.00	
r_d	0.90 ± 0.061	0.97	0.970	C_B	1.00	1.00	
CSR	0.195 ± 0.033	0.210	0.210	C_E	1.09	1.09	1.09
Equivalent Magnitude	7.6	7.6		C_N	1.73	1.70	1.50
MSF		0.97	1.00	$(N_1)_{60}$	6.8 ± 0.3	7.0	6.5
CSRN		0.199	0.21				



γ_m=120±3 pcf

γ_s=120±3 pcf

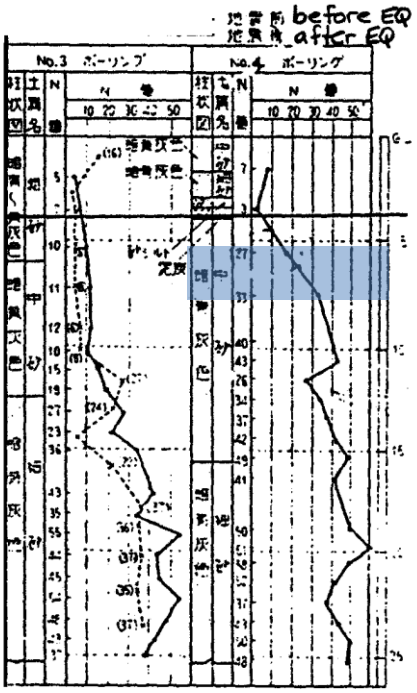


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.5	4.9	0.210	4	2.00	0.77	***6.7***
3.5	11.5	0.196	4	1.79	0.87	***6.8***
4.0	13.1	0.192	4	1.67	0.89	***6.5***
5.5	18.0	0.179	5	1.43	0.94	***7.3***
					Mean:	6.8
					St. Dev.	0.3

Case number:	16
Earthquake:	1964 Niigata
Magnitude:	7.6 (Mw) http://ds.iris.edu/seismo-archives/quakes/1964niigata/
Location:	Showa Br 4
References:	Takada et al (1965) in Japanese Fear et al. (1995) Seed and Idriss (1966), "An analysis of soil liquefaction in the Niigata Eq."
Nature of Failure:	No damage to little damage.
Comments:	Niigata is located on the west coast of Japan where Shinano River enters the sea The river has built up a considerable thickness of alluvial sand deposits, which along the coast have been overlain by deposits of dune sand. In general the sand deposits underlying the city are relatively loose near the ground surface, but gets denser with increasing depth. The sand extents to a depth of ~100' and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively flat. Niigata city was below sea level about thousand years ago and the bay was dotted with several small islands. The EQ was the first large vibration the recent deposits had experienced. Showa Bridge experienced damage during The Niigata Earthquake. Coarse Sand Layer : the most recent strata; approximately 13 m thick at the left bank, decreasing to ~ 5 m at the right bank; a mixture of silt, loam, small gravel and organic material. Medium Sand Layer: 15 m thick at the right bank (top of strta just more than 5m deep) increasing in depth and decreasing in thickness from the right to left bank; approximately 3 m thick at the left bank; some organic material, coarse sand, fine sand and iron sand, causing variations in N. Fine Sand Layer: Deeper than 16-18 m; dark blue gray color; some parts contain silt and iron sand; N>30 The epicenter of the earthquake was located some 35 miles north of the city. From the seismograph, located in the basement of an apartment building it was estimated that the PGA at Niigata was of the order of 0.16 g. The soil below seismograph liquefied during the earthquake. It is interesting to note the marked change in form of record from a predominantly short period motion to a long period motion which occurred after about 8 seconds. SPT values were taken after the earthquake. SPT energy was estimated as 73 % by Seed et al. (84)

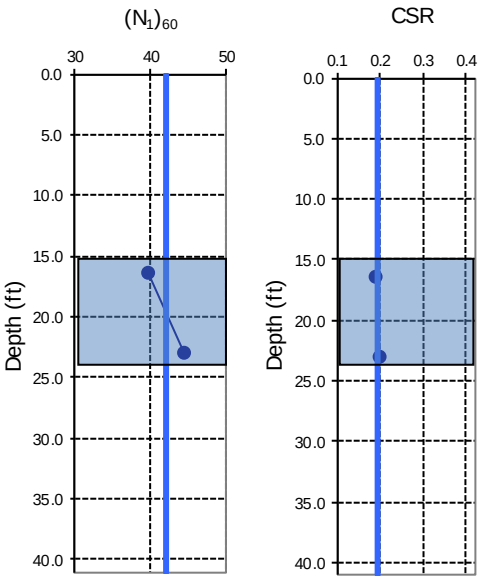
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.300 ± 0.050		0.3
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	16.4 - 23.0	20.0	20.0	% PI			
Depth to GWT (ft)	4.0 ± 0.3	3.9	4.0				
σ_v (psf)	2400.6 ± 145.1	2401.8	2400.0	N	30.0 ± 2.4	27.0	27
σ_v' (psf)	1421.9 ± 85.1	1399.3	1400.0	C_R	0.95	0.95	
a_{max} (g)	0.180 ± 0.027	0.18	0.180	C_S	1.00	1.00	
r_d	0.99 ± 0.091	0.95	0.960	C_B	1.00	1.00	
CSR	0.197 ± 0.035	0.191	0.195	C_E	1.21	1.21	1.21
Equivalent Magnitude	7.6	7.6		C_N	1.22	1.14	1.18
MSF		0.97	1.00	$(N_1)_{60}$	42.0 ± 3.4	35.5	38.5
CSRN		0.178	0.20				



γ_m=110±3 pcf

γ_s=125±3 pcf

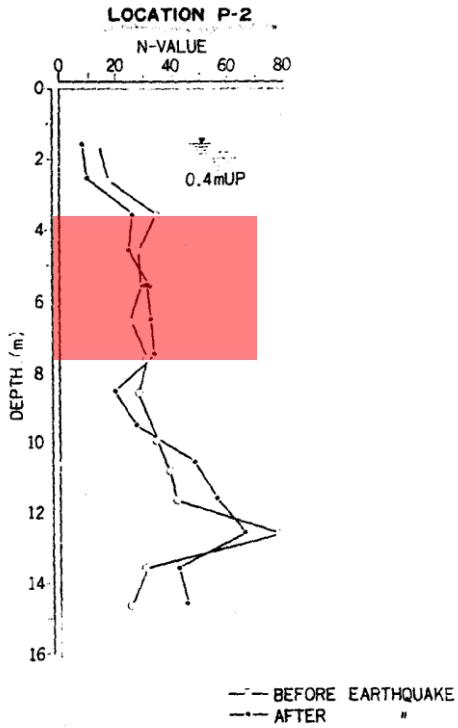


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5	16.4	0.191	27	1.32	0.92	***39.6***
7.0	23.0	0.201	33	1.14	0.98	***44.4***
					Mean:	42.0
					St. Dev.	3.4

Case number:	17
Earthquake:	1968 Tokachi-Oki
Magnitude:	7.8 (MR)
Location:	Hachinohe-2
References:	Ohsaki (1970)
Nature of Failure:	No surface evidence of liquefaction. Moderate densification after earthquake.
Comments:	The sites are located in the city of Hachinohe ~560 kms north of Tokyo. The site is a flat, sandy beach fronting on the Pacific Ocean. The sites consist of sands almost entirely down to a depth of more than 20 m from the ground surface containing large quantity of iron sand. However, the iron sand had been excavated and backfilled with waste sand except at P2 &5 The ground water table is not deeper than 1.5 m everywhere in the site. Site has been improved by vibrofloatation. It seems like average Japanese estimated rod energy 72.5 % has been used by Seed et al (1984). Same values will be adopted The epicenter was located ~180 kms form the city of Hachinohe. The PGA was recorded as 0.225 g at the harbor of Hachinohe.

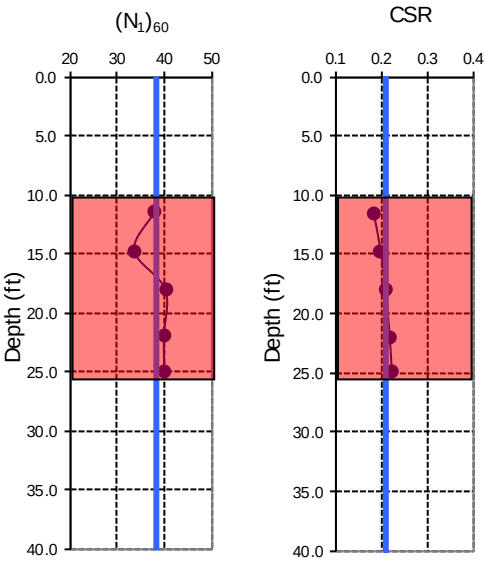
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.250 ± 0.050		0.25
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	10.0 - 26.0	20.0	20.0	% PI			
Depth to GWT (ft)	7.0 ± 0.3	6.9	7.0				
σ_v (psf)	2180.0 ± 335.6	2401.8	2400.0	N	28.5 ± 2.2	28.0	28
σ_v' (psf)	1493.6 ± 172.2	1587.3	1590.0	C_R	0.94	0.95	
a_{max} (g)	0.225 ± 0.068	0.23	0.230	C_S	1.00	1.00	
r_d	0.99 ± 0.084	0.98	0.960	C_B	1.00	1.00	
CSR	0.211 ± 0.066	0.221	0.215	C_E	1.21	1.21	1.21
Equivalent Magnitude	8.3	8.3		C_N	1.19	1.10	1.10
MSF		0.81	0.93	$(N_1)_{60}$	38.4 ± 2.9	35.3	37.5
CSRN		0.254	0.23				



γ_m=115±3 pcf

γ_s=125±3 pcf

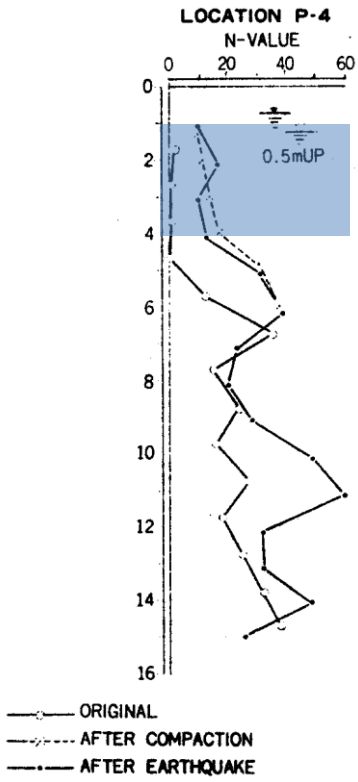


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.5	11.5	0.183	26	1.40	0.87	***38.1***
4.5	14.8	0.200	24	1.28	0.90	***33.6***
5.5	18.0	0.211	30	1.19	0.94	***40.4***
6.7	22.0	0.221	31	1.10	0.97	***40.0***
7.6	24.9	0.225	32	1.05	0.99	***40.2***
					Mean:	38.4
					St. Dev.	2.9

Case number:	18
Earthquake:	1968 Tokachi-Oki
Magnitude:	7.8 (MR)
Location:	Hachinohe-4
References:	Ohsaki (1970)
Nature of Failure:	No surface manifestation of soil liquefaction.
Comments:	The sites are located in the city of Hachinohe ~560 kms north of Tokyo. The site is a flat, sandy beach fronting on the Pacific Ocean. The sites consist of sands almost entirely down to a depth of more than 20 m from the ground surface containing large quantity of iron sand. However, the iron sand had been excavated and backfilled with waste sand except at P2 &5 The ground water table is not deeper than 1.5 m everywhere in the site. Site has been improved by vibrofloatation. It seems like average Japanese estimated rod energy 72.5 % has been used by Seed et al (1984). Same values will be adopted. The epicenter was located ~180 kms form the city of Hachinohe. The PGA was recorded as 0.225 g at the harbor of Hachinohe.

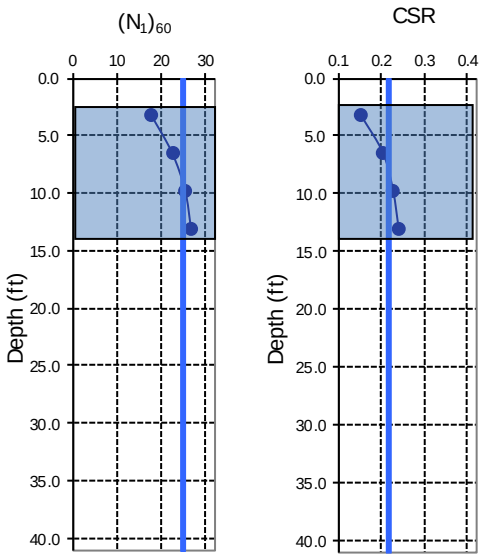
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.250 ± 0.050		0.25
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	3.0 - 13.0	13.1	13.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	955.0 ± 209.1	1566.4	1560.0	N	14.0 ± 1.2	16.0	16
σ_v' (psf)	643.0 ± 106.7	939.8	940.0	C_R	0.82	0.85	
a_{max} (g)	0.225 ± 0.068	0.23	0.230	C_S	1.00	1.00	
r_d	1.00 ± 0.042	0.99	0.970	C_B	1.00	1.00	
CSR	0.217 ± 0.067	0.246	0.240	C_E	1.21	1.21	1.21
Equivalent Magnitude	8.3	8.3		C_N	1.81	1.40	1.41
MSF		0.81	0.94	$(N_1)_{60}$	25.1 ± 2.2	23.0	27.5
CSRN		0.276	0.26				



γ_m=110±3 pcf

γ_s=125±3 pcf

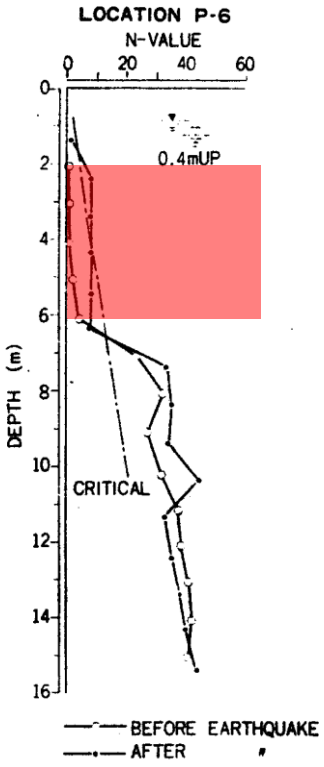


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	0.153	10	2.00	0.73	17.8
2.0	6.6	0.205	12	1.96	0.80	***22.6***
3.0	9.8	0.228	15	1.67	0.85	***25.6***
4.0	13.1	0.241	17	1.48	0.89	***27.0***
					Mean:	25.1
					St. Dev.	2.2

Case number:	19
Earthquake:	1968 Tokachi-Oki
Magnitude:	7.8 (MR)
Location:	Hachinohe-6
References:	Ohsaki (1970)
Nature of Failure:	Eruption of sand and water. Significant densification after earthquake
Comments:	The sites are located in the city of Hachinohe ~560 kms north of Tokyo. The site is a flat, sandy beach fronting on the Pacific Ocean. The sites consist of almost entirely sandy deposits down to a depth of 20 m from the ground surface containing large quantity of iron sand. However, the iron sand had been excavated and backfilled with waste sand except at P2 &5 The ground water table is not deeper than 1.5 m everywhere in the site. Site has been improved by vibrofloatation. Estimated rod energy 65 % has been used by Seed et al. Same value will be adopted for this study. The epicenter was located ~180 kms form the city of Hachinohe. The PGA was recorded as 0.225 g at the harbor of Hachinohe.

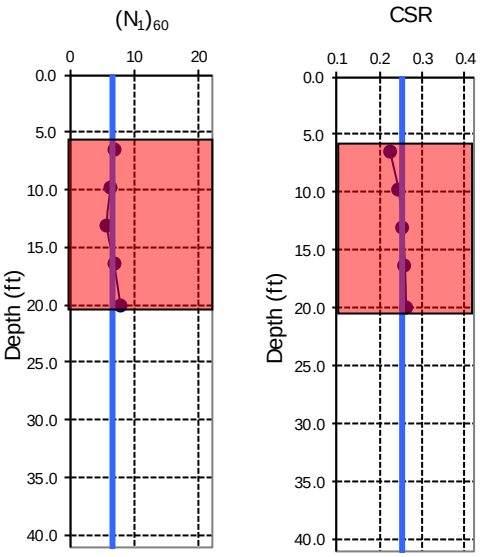
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.250 ± 0.050		0.25
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	6.6 - 20.0	13.1	13.0	% PI			
Depth to GWT (ft)	2.0	2.0	2.0				
σ_v (psf)	1632.5 ± 281.3	1566.4	1560.0	N	4.7 ± 0.5	6.0	6
σ_v' (psf)	927.4 ± 144.7	877.2	870.0	C_R	0.89	0.85	
a_{max} (g)	0.225 ± 0.068	0.23	0.230	C_S	1.00	1.00	
r_d	0.99 ± 0.065	0.99	0.970	C_B	1.00	1.00	
CSR	0.254 ± 0.079	0.265	0.260	C_E	1.09	1.09	1.09
Equivalent Magnitude	8.3	8.3		C_N	1.51	1.63	1.47
MSF		0.81	0.95	$(N_1)_{60}$	6.8 ± 0.7	9.1	9.5
CSRN		0.304	0.28				



γ_m=110±3 pcf

γ_s=125±3 pcf

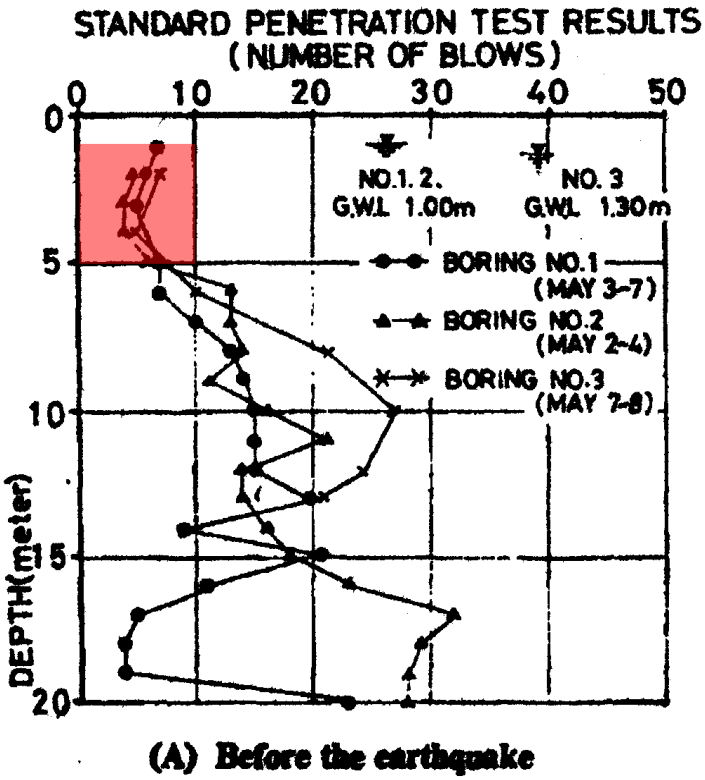


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	0.228	4	2.00	0.80	***7.0***
3.0	9.8	0.245	4	1.73	0.85	***6.4***
4.0	13.1	0.254	4	1.52	0.89	***5.9***
5.0	16.4	0.259	5	1.37	0.92	***6.9***
6.1	20.0	0.261	6	1.25	0.95	***7.8***
					Mean:	6.8
					St. Dev.	0.7

Case number:	20
Earthquake:	1968 Tokachi-Oki
Magnitude:	7.8 (MR)
Location:	Nanaehama
References:	Kishida (1970)
Nature of Failure:	"complete liquefaction of level ground from the ground surface to some depth was observed" "water and soil began to spout out from the ground surface soon after the beginning of the earthquake and continued for about an hour" So soft, hard to walk on even a week after the earthquake
Comments:	The site is located at Nanaehama beach in Kamiiso Town in Hokkaido Island Reclamation was made at the site by hydarulic fill at about three years before the earthquake. The soil was coarse grained sand in accordance with the unified classification system, having the 50% size of less than 2.0 mm and the uniformity coefficient of less than 10. The relative density was less than 75% No fine grained strata above the saturated sand. 3 borings before the earthquake and 3 additional borings after the earthquake SPT energy was estimated as 70 % by Seed et al (1984)

Summary of Data

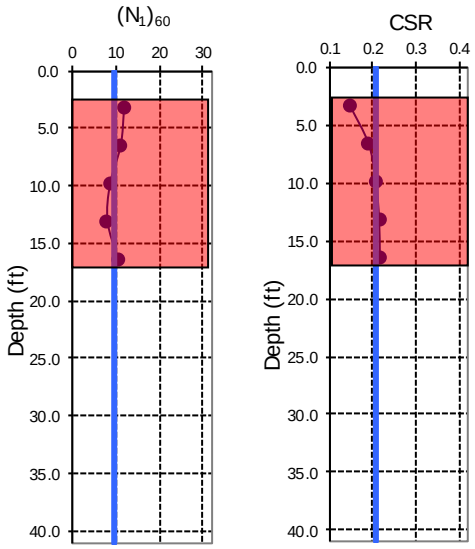
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.121 ± 0.034		0.12
Data Class	B			% Fines	21.7 ± 4.8	20.0	20
Critical Depth Range	3.0 - 16.4	13.1	13.0	% PI			
Depth to GWT (ft)	2.5 ± 0.6	3.0	3.0				
σ_v (psf)	1139.2 ± 269.1	1566.4	1560.0	N	5.6 ± 0.9	5.0	5
σ_v' (psf)	690.1 ± 134.0	939.8	940.0	C_R	0.84	0.85	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.97 ± 0.050	0.99	0.970	C_B	1.00	1.00	
CSR	0.207 ± 0.065	0.213	0.210	C_E	1.17	1.17	1.17
Equivalent Magnitude	8.3	8.3		C_N	1.75	1.52	1.41
MSF		0.81	0.95	$(N_1)_{60}$	9.6 ± 1.5	7.6	8
CSRN		0.244	0.22				



(A) Before the earthquake

γ_m=110±3 pcf

γ_s=120±3 pcf

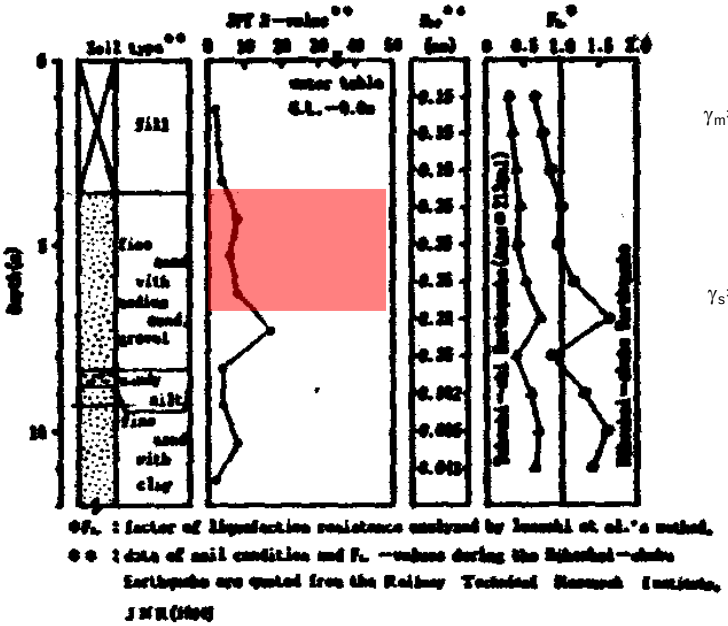


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	0.148	7	2.00	0.73	12.0
2.0	6.6	0.191	6	2.00	0.80	***11.2***
3.0	9.8	0.208	5	1.74	0.85	***8.6***
4.0	13.1	0.215	5	1.54	0.89	***8.0***
5.0	16.4	0.217	7	1.40	0.92	***10.5***
					Mean:	9.6
					St. Dev.	1.5

Case number:	21
Earthquake:	1968 Tokachi-Oki
Magnitude:	7.8 (MR)
Location:	Aomori City (Aomori Station)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils, settled structures.
Comments:	The station faces the sea and the north part of the station is used as a landing place for ferryboats. The north part was built on reclaimed land constructed around 1945, but the sout part was built on natural land. Liquefaction was induced at the north part of the station during the Tokachi-Oki and the main shock of Nihonakai-chubu EQ's. Many sand volcanoes, settlement of rails and buildings and uplift of buried tanks were observed. The maximum surface acceleration at Aomari Port, about 1 km north of Aomori Station was recorded as 0.213 g.

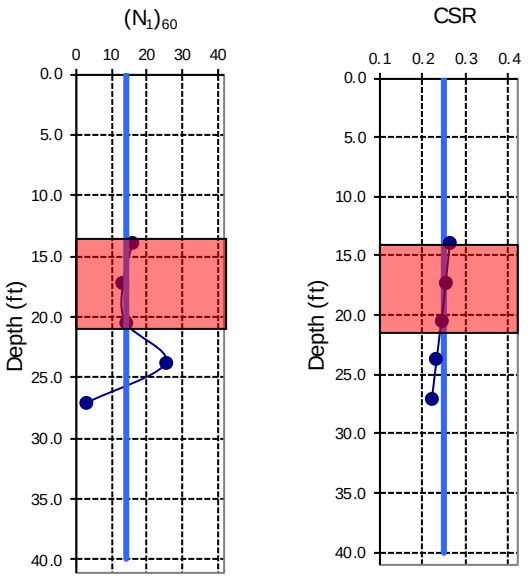
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	No		D_{50}	0.250 ± 0.050		
Data Class	A			% Fines	3.0 ± 2.0	3.0	
Critical Depth Range	13.1 - 24.6	18.7		% PI			
Depth to GWT (ft)	0.0 ± 0.4	0.0					
σ_v (psf)	2263.8 ± 236.6	1984.1		N	9.0 ± 0.9	9.0	
σ_v' (psf)	1086.6 ± 124.8	793.6		C_R	0.94	0.95	
a_{max} (g)	0.213 ± 0.011	0.21		C_S	1.00	1.00	
r_d	0.87 ± 0.087	0.98		C_B	1.00	1.00	
CSR	0.250 ± 0.029	0.335		C_E	1.22	1.22	
Equivalent Magnitude	8.3	8.3		C_N	1.40	1.58	
MSF		0.81		$(N_1)_{60}$	14.4 ± 1.4	16.5	
CSRN		0.376					



γ_m=100±3 pcf

γ_s=120±3 pcf

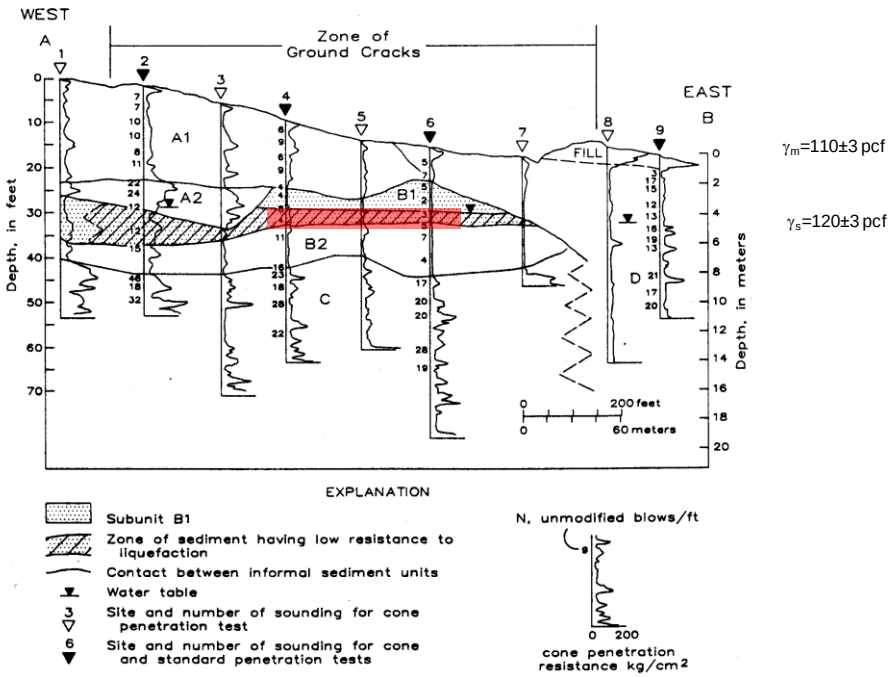


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
4.3	13.9	0.264	9.0	1.62	0.90	***15.9***	
5.3	17.2	0.255	8.0	1.46	0.93	***13.2***	
6.3	20.5	0.245	9.0	1.34	0.96	***14.0***	
7.3	23.8	0.234	17.0	1.24	0.98	25.3	Gravel?
8.3	27.1	0.223	2.0	1.17	1.00	2.8	Clay?
					Mean:	14.4	
					St. Dev.	1.4	

Case number:	22
Earthquake:	1971 San Fernando
Magnitude:	6.4 (ML) (Mw=6.6)
Location:	Juvenile Hall
References:	Bennett (1989)
Nature of Failure:	Lateral Spreading, sand boils
Comments:	On February 9, 1971, a magnitude ML 6.4 earthquake caused major damage in the San Fernando Valley, CA. One facility that received heavy damage was San Fernando Valley Juvenile Hall. The site is located north of the main trace of surface faulting on the upthrust block. About 50 sand boils occurred in the field south of the Juvenile Hall near boring 10. Sand boils also occurred in the northwestern and northeastern parts of Upper Van Norman Lake. The lateral spread lies in the north-central part of San Fernando Valley within a small synclinal basin bounded on the north and south by hills composed of clastic sedimentary rocks. The basin contains several alluvial fans that originate from Welden, Grapevine, and Sombrero canyons. On the basis of sediment properties and penetration data, four units have been defined; From top to bottom the units are A, poorly sorted, loose to medium-dense silty sand; B, very loose to loose poorly sorted silty sand to silt; C, medium-dense to dense, poorly sorted silty sand; and D, stiff clayey silt. Unit B is divided into two subunits, B1 and B2 on the basis of grain size and plasticity. Subunit B1 consists of poorly sorted, loose sandy silt and silty sand, it has an average sand and gravel content of 38 %, an average clay content of 10 %. Subunit B2 consists of very loose, poorly sorted sandy silt and silt. It has an average clay content of 19 %. Unit B2 is probably not susceptible to liquefaction due to large clay fraction.Unit B has a PI of 3-10 %. PGA was estimated at the crest and the abutments of Lower San Fernando Dam as 0.55 and 0.48 g. A value of 0.45 is adopted as used by Seed et al (84) SPT energy was given as 68 % THE BORINGS MAY NOT BE IDENTICAL WITH THE ONE USED BY SEED ET AL. (84).

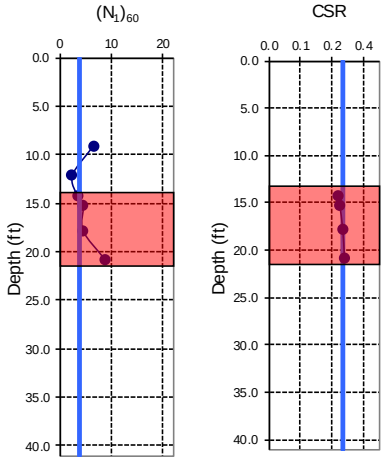
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.047 ± 0.011		Silt
Data Class	A			% Fines	65.3 ± 8.0	55.0	>50
Critical Depth Range	14.4 - 20.7	20.0	20.0	% PI	3-10		
Depth to GWT (ft)	14.0 ± 0.2	15.1	15.0				
σ_v (psf)	1966.3 ± 132.0	2339.2	2330.0	N	3.3 ± 0.9	3.5	2
σ_v' (psf)	1744.6 ± 74.3	2005.0	2000.0	C_R	0.90	0.95	0.75
a_{max} (g)	0.450 ± 0.068	0.45	0.450	C_S	1.00	1.00	
r_d	0.86 ± 0.082	0.92	0.950	C_B	1.00	1.00	
CSR	0.283 ± 0.051	0.312	0.325	C_E	1.13	1.13	1.00
Equivalent Magnitude	6.6	6.6		C_N	1.10	1.03	1.00
MSF		1.26	1.16	$(N_1)_{60}$	3.7 ± 1.0	3.9	1.5
CSRN		0.246	0.28				



γ_m=110±3 pcf

γ_s=120±3 pcf

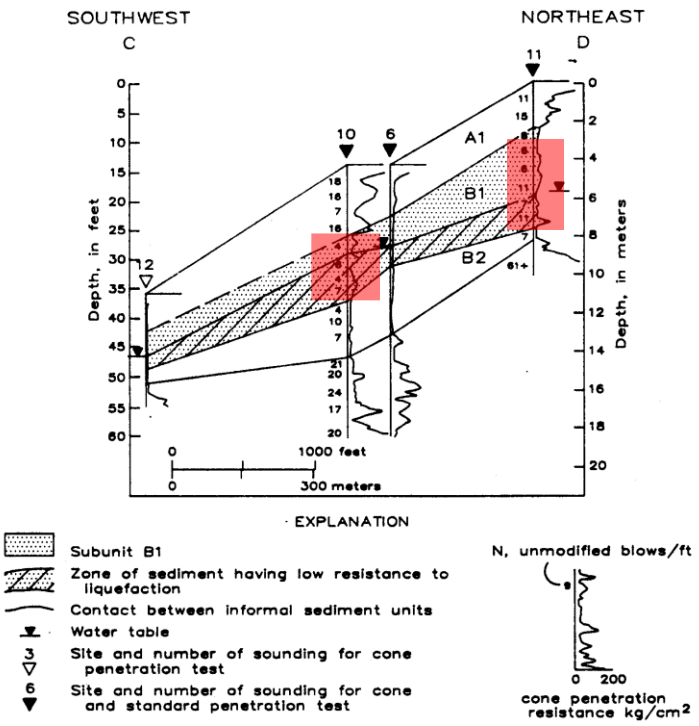


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC (%)	Silt	Clay	D_{50}	
2.8	9.1	NA	5.0	1.45	0.79	6.5	83	72	11	0.042	BH6
3.7	12.1	NA	2.0	1.26	0.84	***2.4***	**64**	48	16	0.037	BH6
4.4	14.3	0.266	3.0	1.17	0.87	***3.4***	**74**	62	12	0.042	BH6
4.6	15.3	0.272	4.0	1.15	0.88	***4.6***	**68**	56	12	0.045	BH4
5.4	17.8	0.284	4.0	1.10	0.91	***4.5***	**55**	46	9	0.062	BH4
6.3	20.8	0.291	8.0	1.05	0.93	8.9	66	58	8	0.052	BH4
					Mean:	3.7	65.3			0.047	
					St. Dev.	1.0	8.0			0.011	

Case number:	23
Earthquake:	1971 San Fernando
Magnitude:	6.4 (M_L) (Mw=6.6)
Location:	Van Norman
References:	Bennett (1989)
Nature of Failure:	Lateral Spreading, sand boils
Comments:	On February 9, 1971, a magnitude M_L 6.4 earthquake caused major damage in the San Fernando Valley, CA. One facility that received heavy damage was the San Fernando Valley Juvenile Hall. The site is located north of the main trace of surface faulting on the upthrust block. About 50 sand boils occurred in the field south of the Juvenile Hall near boring 10. Sand boils also occurred in the northwestern and northeastern parts of Upper Van Norman Lake. The lateral spread lies in the north-central part of San Fernando Valley within a small synclinal basin bounded on the north and south by hills composed of clastic sedimentary rocks. The basin contains several alluvial fans that originate from Welden, Grapevine, and Sombrero canyons. On the basis of sediment properties and penetration data, four informal units have been defined; From top to bottom the units are A, poorly sorted, loose to medium-dense silty sand; B, very loose to loose poorly sorted silty sand to silt; C, medium-dense to dense, poorly sorted silty sand; and D, stiff clayey silt. Unit B is divided into two subunits, B1 and B2 on the basis of grain size and plasticity. Subunit B1 consists of poorly sorted, loose sandy silt and silty sand, it has an average sand and gravel content of 38 %, an average clay content of 10 %. Subunit B2 consists of very loose, poorly sorted sandy silt and silt. It has an average clay content of 19 %. Unit B2 is probably not susceptible to liquefaction due to large clay fraction.Unit B has a PI of 3-10 %. PGA was estimated at the crest and the abutments of Lower San Fernando Dam as 0.55 and 0.48 g. A value of 0.45 is adopted as used by Seed et al (84) SPT energy was given as 68 % THE BORINGS MAY NOT BE IDENTICAL WITH THE ONE USED BY SEED ET AL. (84).

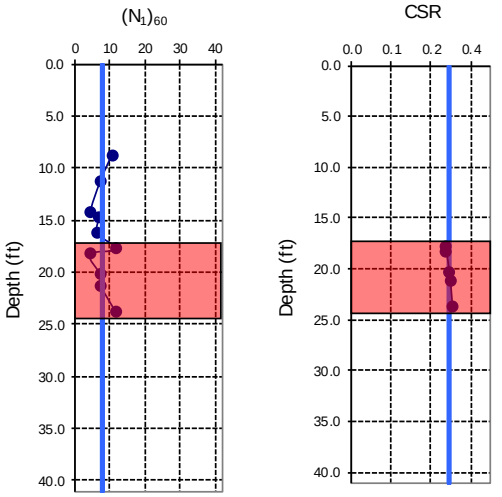
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.067 ± 0.034		0.1
Data Class	A			% Fines	59.3 ± 14.2	50.0	20
Critical Depth Range	17.0 - 24.0	20.0	20.0	% PI	3-10		
Depth to GWT (ft)	16.3 ± 0.9	15.1	10.0				
σ_v (psf)	2296.9 ± 149.1	2339.2	2250.0	N	7.4 ± 2.5	7.3	9
σ_v' (psf)	2035.2 ± 96.2	2005.0	1800.0	C_R	0.93	0.95	0.75
a_{max} (g)	0.450 ± 0.068	0.45	0.450	C_S	1.00	1.00	
r_d	0.90 ± 0.094	0.92	0.950	C_B	1.00	1.00	
CSR	0.297 ± 0.055	0.312	0.345	C_E	1.13	1.13	1.00
Equivalent Magnitude	6.6	6.6		C_N	1.02	1.03	1.05
MSF		1.26	1.15	$(N_1)_{60}$	7.9 ± 2.7	8.1	7
CSRN		0.245	0.30				



γ_m=110±3 pcf

γ_s=120±3 pcf

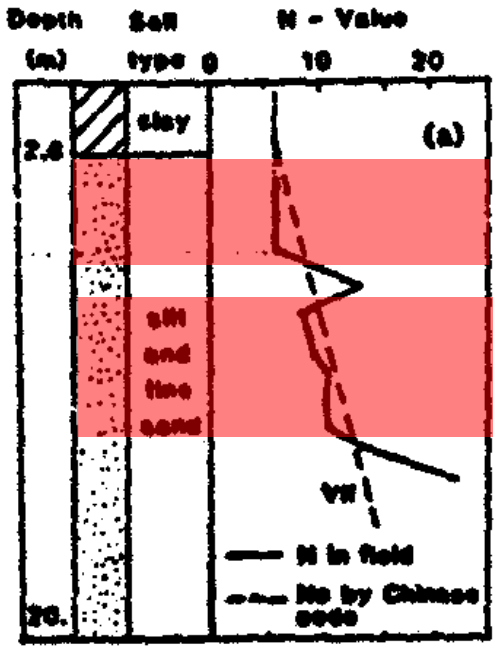


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC (%)	Silt	Clay Content	D50	
2.7	8.8	NA	8.0	1.48	0.78	***10.5***	***56.0***	42	14	0.059	BH11
3.4	11.3	NA	6.0	1.31	0.82	***7.3***	***65.0***	46	19	0.043	BH11
4.3	14.3	NA	4.0	1.16	0.87	***4.6***	***78.0***	58	20	0.031	BH10
4.5	14.8	NA	6.0	1.14	0.87	***6.8***	***52.0***	40	12	0.067	BH11
5.0	16.3	NA	6.0	1.09	0.89	***6.6***	***52.0***	45	7	0.072	BH10
5.4	17.8	0.283	11.0	1.06	0.90	***12.0***	***38.0***	30	8	0.14	BH11
5.6	18.3	0.286	4.0	1.05	0.91	***4.3***	***65.0***	58	7	0.055	BH10
6.2	20.3	0.296	7.0	1.02	0.93	***7.5***	***61.0***	47	14	0.051	BH11
6.5	21.3	0.300	7.0	1.01	0.94	***7.5***	***83.0***	70	13	0.038	BH10
7.2	23.8	0.307	11.0	0.98	0.96	***11.7***	***43.0***	33	10	0.11	BH11
					Mean:	7.9	59.3			0.1	
					St. Dev.	2.7	14.2			0.0	

Case number:	24
Earthquake:	1975 Haicheng
Magnitude:	7.0 (Mw) USGS Centennial Earthquake Catalog
Location:	Panjin Chemical Fertilizer Plant
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied site Extensive sand boils and sand volcanoes were seen in the NW part of the region of 7th grade seismic intensity.
Comments:	This site was located in the NW part of the region and shaken by 7th grade on Chinese intensity scale. (corresponds to JMA scale of 4-5). It is located mainly in flood plain; top clayey layer ~2.6 m thick and underlain by liquefaction susceptible silty sand or fine sand > 2m thick. At the moment of Haicheng EQ, the surface clayey soil layers were frozen at many sites. A 65 m high reinforced concrete tower on 17 m reinforced concrete piles was not damaged- its walls cracked due to differential settlement of the ground. There was almost no liquefaction near the epicenter of he EQ. PGA was estimated as >0.1 g SPT energy was estimated as 50 % by Seed et al. (84)

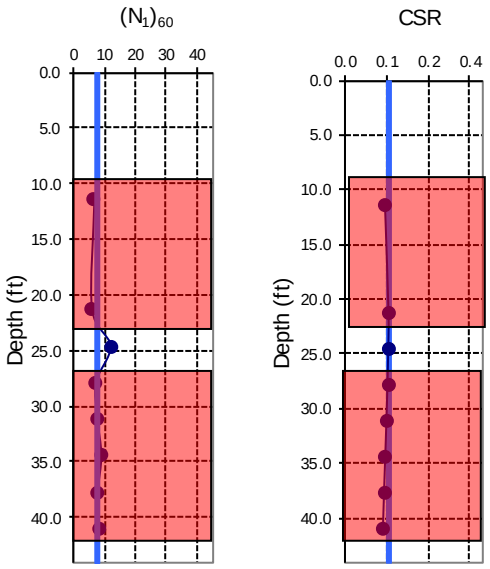
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.064 ± 0.050		0.064
Data Class	B			% Fines	67.0 ± 2.0	67.0	67
Critical Depth Range	11.5 - 41.0	26.9	30.0	% PI			
Depth to GWT (ft)	5.0 ± 0.3	4.9	5.0				
σ_v (psf)	3099.7 ± 594.2	3237.2	3675.0	N	8.1 ± 1.2	9.1	8
σ_v' (psf)	1773.1 ± 291.2	1858.8	2115.0	C_R	1.00	0.95	
a_{max} (g)	0.130 ± 0.039	0.20	0.130	C_S	1.00	1.00	
r_d	0.86 ± 0.116	0.89	0.920	C_B	1.00	1.00	
CSR	0.127 ± 0.042	0.203	0.135	C_E	0.83	0.83	0.83
Equivalent Magnitude	7.0	7.0		C_N	1.09	1.07	0.98
MSF		1.14	1.04	$(N_1)_{60}$	7.4 ± 1.1	7.6	6.5
CSRN		0.175	0.13				



γ_m=110±3 pcf

γ_s=120±3 pcf

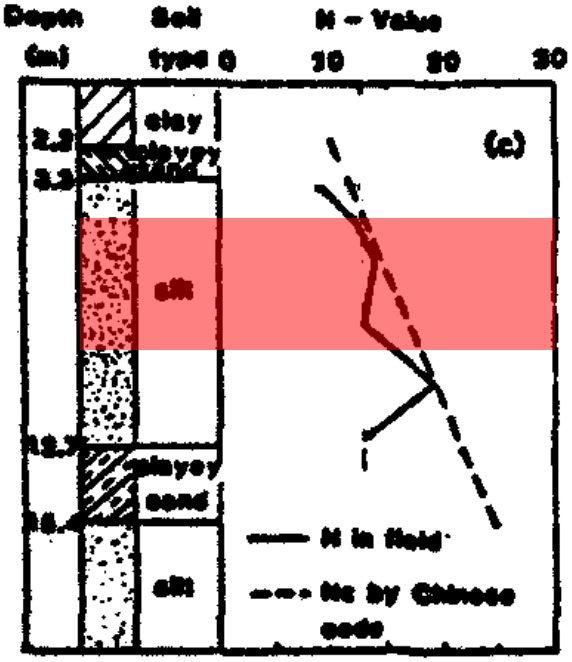


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
3.5	11.5	0.118	6.0	1.51	0.87	***6.6***	
6.5	21.3	0.129	6.0	1.19	0.96	***5.7***	
7.5	24.6	0.128	13.0	1.12	0.99	12.0	gravel?
8.5	27.9	0.125	8.0	1.06	1.00	***7.1***	
9.5	31.2	0.122	9.0	1.01	1.00	***7.6***	
10.5	34.4	0.118	11.0	0.97	1.00	***8.9***	
11.5	37.7	0.115	10.0	0.93	1.00	***7.8***	
12.5	41.0	0.111	11.0	0.90	1.00	***8.2***	
					Mean:	7.4	
					St. Dev.	1.1	

Case number:	25
Earthquake:	1975 Haicheng
Magnitude:	7.0 (Mw) USGS Centennial Earthquake Catalog
Location:	Yingkou Glass Fibre Plant
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied site
Comments:	Yingkou Glass Fibre Plant site is located in the west of epicenter, in the 9th grade on the Chinese intensity scale, in the zone of slight liquefaction. Thin top clayey layers were underlain by greater than 2 m thick potentially liquefiable silty fine sand layers. At the moment of the EQ, the surface clayey soil layers were frozen at many sites. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

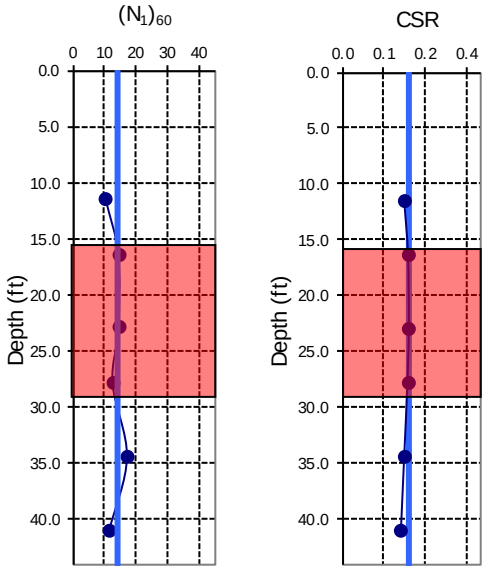
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.078 ± 0.050		0.078
Data Class	B			% Fines	48.0 ± 2.0	48.0	48
Critical Depth Range	16.4 - 29.5	25.6	27.0	% PI			
Depth to GWT (ft)	5.0 ± 0.2	4.9	5.0				
σ_v (psf)	2706.0 ± 268.4	3070.2	3300.0	N	12.4 ± 1.0	13.0	13
σ_v' (psf)	1584.1 ± 138.3	1775.3	1930.0	C_R	0.98	0.95	
a_{max} (g)	0.200 ± 0.060	0.30	0.200	C_S	1.00	1.00	
r_d	0.89 ± 0.103	0.90	0.930	C_B	1.00	1.00	
CSR	0.197 ± 0.063	0.304	0.210	C_E	1.00	1.00	1.00
Equivalent Magnitude	7.0	7.0		C_N	1.16	1.08	1.02
MSF		1.14	1.05	$(N_1)_{60}$	14.0 ± 1.1	13.3	13.5
CSRN		0.260	0.20				



γ_m=110±3 pcf

γ_s=120±3 pcf

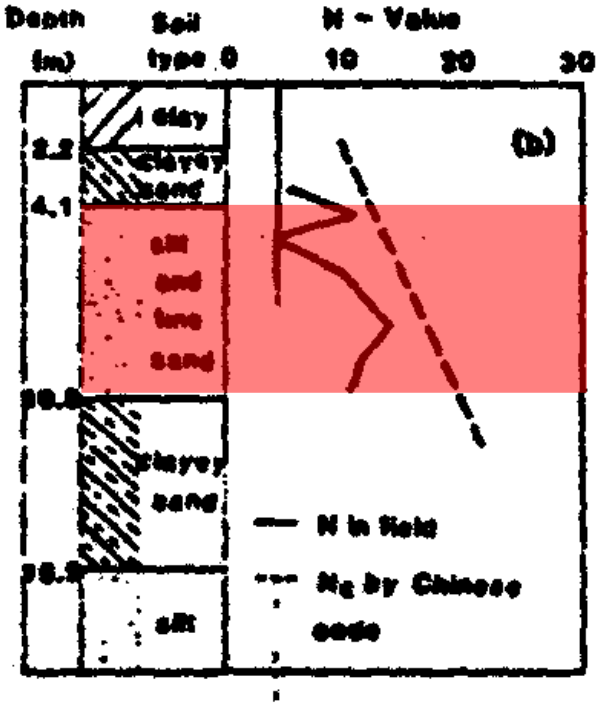


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$N_{1,60}$	
3.5	11.5	0.181	8.0	1.51	0.87	10.5	clayey sand?
5.0	16.4	0.194	12.0	1.32	0.92	***14.6***	
7.0	23.0	0.197	13.0	1.16	0.98	***14.7***	
8.5	27.9	0.192	12.0	1.06	1.00	***12.8***	
10.5	34.4	0.180	18.0	0.97	1.00	17.5	
12.5	41.0	0.169	13.0	0.90	1.00	11.7	
					Mean:	14.0	
					St. Dev.	1.1	

Case number:	26
Earthquake:	1975 Haicheng
Magnitude:	7.0 (Mw) USGS Centennial Earthquake Catalog
Location:	Yingkou Paper Plant
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied site
Comments:	Yingkou Paper Plant site is located in the west of epicenter, in the 9th grade on the Chinese intensity scale, in the zone of slight liquefaction. Thin top clayey layers were underlain by greater than 2 m thick potentially liquefiable silty fine sand layers. At the moment of the EQ, the surface clayey soil layers were frozen at many sites. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

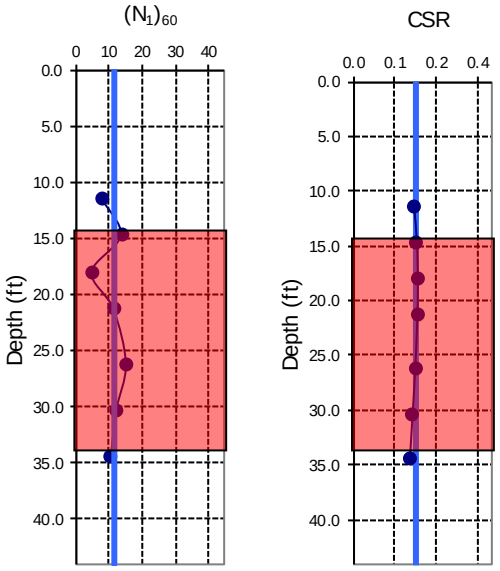
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.100 ± 0.050		silt and sand
Data Class	B			% Fines	20.0 ± 2.0	5.0	silt and sand
Critical Depth Range	14.8 - 34.4	26.9	27.0	% PI			
Depth to GWT (ft)	5.0 ± 0.2	4.9	5.0				
σ_v (psf)	2902.9 ± 398.4	3299.9	3300.0	N	10.3 ± 3.2	11.0	11
σ_v' (psf)	1678.6 ± 198.8	1921.5	1930.0	C_R	0.99	0.95	
a_{max} (g)	0.200 ± 0.060	0.30	0.200	C_S	1.00	1.00	
r_d	0.81 ± 0.110	0.89	0.930	C_B	1.00	1.00	
CSR	0.183 ± 0.060	0.298	0.210	C_E	1.00	1.00	1.00
Equivalent Magnitude	7.0	7.0		C_N	1.12	1.05	1.02
MSF		1.14	1.05	$(N_1)_{60}$	11.4 ± 3.6	11.0	11
CSRN		0.259	0.20				



γ_m=110±3 pcf

γ_s=120±3 pcf

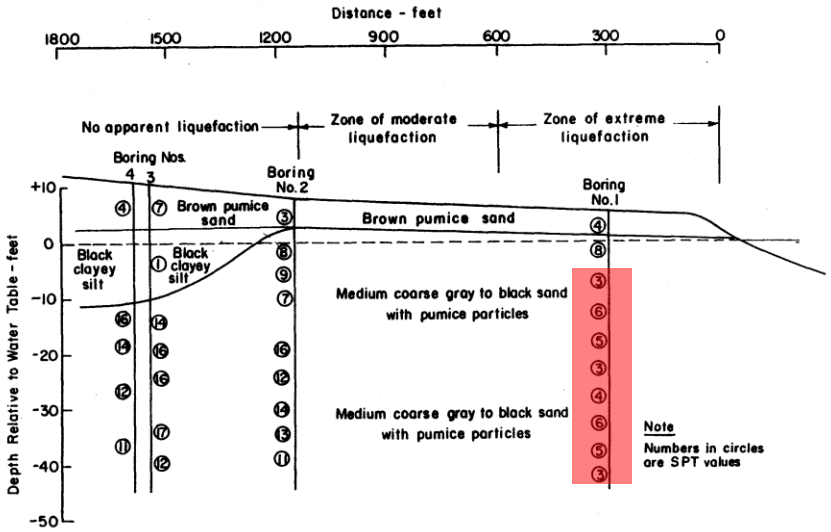


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
3.5	11.5	0.177	6.0	1.51	0.87	7.9	clayey sand?
4.5	14.8	0.185	11.0	1.38	0.90	***13.7***	
5.5	18.0	0.188	4.0	1.28	0.94	***4.8***	
6.5	21.3	0.187	10.0	1.19	0.96	***11.5***	
8.0	26.2	0.180	14.0	1.09	1.00	***15.3***	
9.3	30.3	0.173	12.0	1.03	1.00	***12.3***	
10.5	34.4	0.164	11.0	0.97	1.00	***10.7***	
					Mean:	11.4	
					St. Dev.	3.6	

Case number:	27
Earthquake:	1976 Guatemala
Magnitude:	7.5 (M_L)
Location:	Amatitlan B-1
References:	Seed et al. (1979)
Nature of Failure:	Subsidence and flooding of beach areas, severe ground cracking sizes up to several meters. Severe damage to houses, numerous sand boils and lateral spreading-induced structural damages. Expulsion of occasional pieces of of pumice, ranging from small gravel to cobble sizes together with sand boil.
Comments:	The sites are located on deltaic deposit near the mouth of a small river, Rio Villalobos at the settlement of La Playa on the north-east shore of Lake Amatitlan.The general area behind the shoreline is quite flat with the ground sloping gently up about 2 degs. away from the lake. Borehole is located in the midst of the highly liquefied zone. Two test pits were dug to a depth of ~ 5' on the level ground 40' from the beach These showed the upper 4' of soil to be pumice sand underlain by a thin layer of dense fibrous peats and organic silt. Two field density test in the pumice sand revealed that it had wet and dry densities ranging from 55.5-61.5 and 35.5-39.4 pcf. respectively. Light-weight pumice sand at all boring locations was located above water table and was not itself the layer of liquefaction. The epicenter was located ~170 kms north-east from Lake Amatitlan However, north terminus of the fault is only 40 kms away from the lake. The Modified Mercalli Intensity of shaking at the site is high VI or low VII. Based on correlations, PGA was estimated as 0.12-0.15 g. This estimation was checked by the level of damage at nearby structures. Also attenuation relationships confirmed the range. Based on seismoscope records, USGS predicted the PGA at a site ~ 25 km from the causative fault as 0.25 g. Calibrating the attenuation relationships with this information, PGA at the site is predicted as 0.15 g. Local Magnitude saturates at magnitude 7. Thus, it is hard to estimate the moment magnitude for the earthquake. However based on the recorded motion and attenuation relationships, moment magnitude is back calculated as 7-8.2. Rope and pulley system (2 turns of the rope around the pulley) was used. Estimated rod energy is 45%. SPT values were taken after the earthquake.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.800 ± 0.050		0.8
Data Class	C			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	10.0 - 50.0	34.1	34.0	% PI			
Depth to GWT (ft)	5.0	4.9	5.0				
σ_v (psf)	2550.0 ± 600.6	2903.1	2910.0	N	4.3 ± 1.4	6.0	6
σ_v' (psf)	990.0 ± 186.0	1796.1	1800.0	C_R	1.00	1.00	
a_{max} (g)	0.135 ± 0.041	0.14	0.135	C_S	1.00	1.00	
r_d	0.57 ± 0.117	0.89	0.890	C_B	1.00	1.00	
CSR	0.129 ± 0.047	0.126	0.190	C_E	0.75	0.75	0.75
Equivalent Magnitude	7.5	7.5		C_N	1.46	1.10	1.05
MSF		1.00	1.00	$(N_1)_{60}$	4.7 ± 1.5	5.0	5
CSRN		0.125	0.19				



γ_m=60±1 pcf

γ_s=90±1 pcf

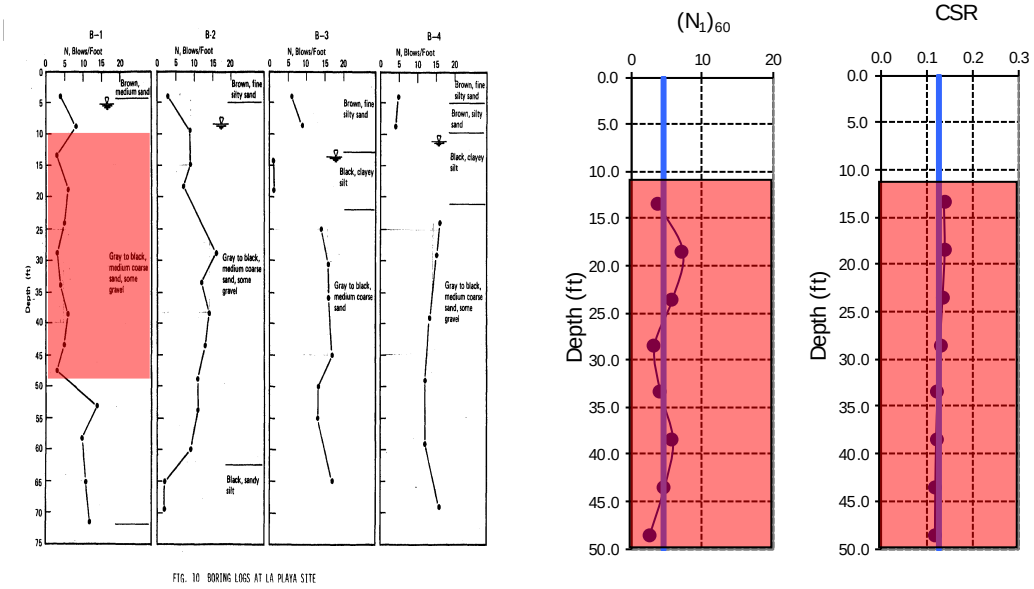


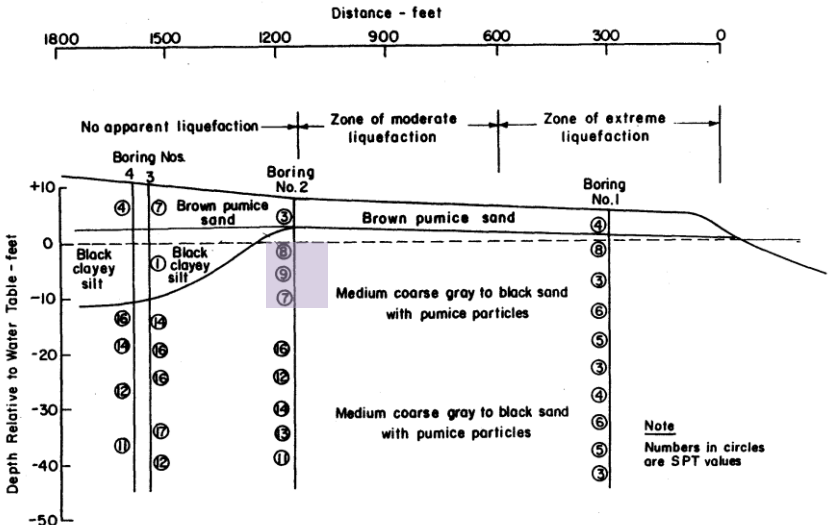
FIG. 10 BORING LOGS AT LA PLAYA SITE

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4.1	13.5	0.141	3	1.99	0.86	***3.8***
5.65	18.5	0.143	6	1.77	0.91	***7.3***
7.2	23.6	0.137	5	1.61	0.96	***5.8***
8.7	28.5	0.131	3	1.49	1.00	***3.3***
10.2	33.5	0.126	4	1.40	1.00	***4.2***
11.75	38.5	0.122	6	1.31	1.00	***5.9***
13.3	43.6	0.120	5	1.24	1.00	***4.7***
14.8	48.6	0.119	3	1.19	1.00	***2.7***
					Mean:	4.7
					St. Dev.	1.5

Case number:	28
Earthquake:	1976 Guatemala
Magnitude:	7.5 (ML)
Location:	Amatitlan B-2
References:	Seed et al. (1979)
Nature of Failure:	Right at the boundary of zone of moderate liquefaction to nonliquefaction
Comments:	The sites are located on deltaic deposits near the mouth of a small river, Rio Villalobos at the settlement of La Playa on the north-east shore of Lake Amatitlan.The general area behind the shoreline is quite flat with the ground sloping gently up about 2 degs away from the lake. Two test pits were dug to a depth of ~ 5' on the level ground 40' from the beach These showed the upper 4' of soil to be pumice sand underlain by a thin layer of dense fibrous peats and organic silt. Two field density test in the pumice sand revealed that it had wet and dry densities ranging from 55.5-61.5 and 35.5-39.4 pcf. respectively. Light-weight pumice sand at all boring locations was located above water table and was not itself the layer of liquefaction. The epicenter was located ~170 kms north-east from Lake Amatitlan However, north terminus of the fault is only 40 kms away from the lake. The Modified Mercalli Intensity of shaking at the site is high VI or low VII. Based on correlations, PGA was estimated as 0.12-0.15 g. This estimation was checked by the level of damage at the nearby structures. Also attenuation relationships confirmed the range. Based on seismoscope records, USGS predicted the PGA at a site ~ 25 km from the causative fault as 0.25 g. Calibrating the attenuation relationships with this information, PGA at the site was predicted as 0.15 g. Local Magnitude saturates at magnitude 7. Thus, it is hard to estimate the moment magnitude for the earthquake. However based on the recorded motion and attenuation relationships, moment magnitude is back calculated as 7-8.2. Rope and pulley system (2 turns of the rope around the pulley) was used. Estimated rod energy is 45%. SPT values were taken after the earthquake.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No/Yes	Marginal	Yes/No	D_{50}	0.800 ± 0.050		0.8
Data Class	B			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	8.0 - 20.0	15.1	15.0	% PI			
Depth to GWT (ft)	8.0	7.9	8.0				
σ_v (psf)	1020.0 ± 180.5	1148.7	1150.0	N	8.0 ± 1.1	8.0	8
σ_v' (psf)	645.6 ± 56.9	710.1	705.0	C_R	0.86	0.95	
a_{max} (g)	0.135 ± 0.041	0.14	0.135	C_S	1.00	1.00	
r_d	0.82 ± 0.061	0.97	0.950	C_B	1.00	1.00	
CSR	0.113 ± 0.037	0.138	0.135	C_E	0.75	0.75	0.75
Equivalent Magnitude	7.5	7.5		C_N	1.81	1.70	1.60
MSF		1.00	1.00	$(N_1)_{60}$	9.3 ± 1.3	9.7	9
CSRN		0.126	0.14				



γ_m=60±1 pcf

γ_s=90±1 pcf

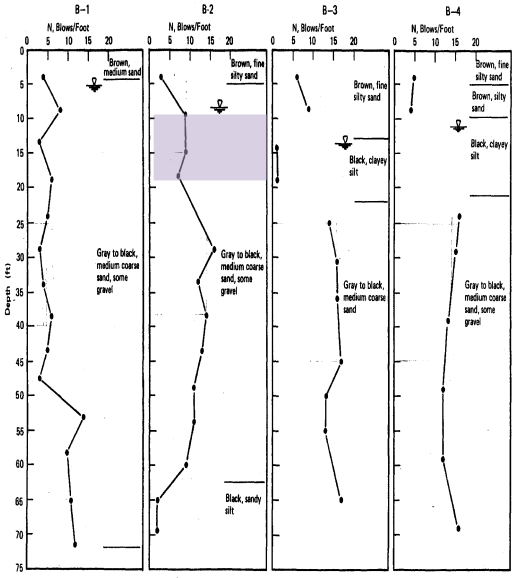
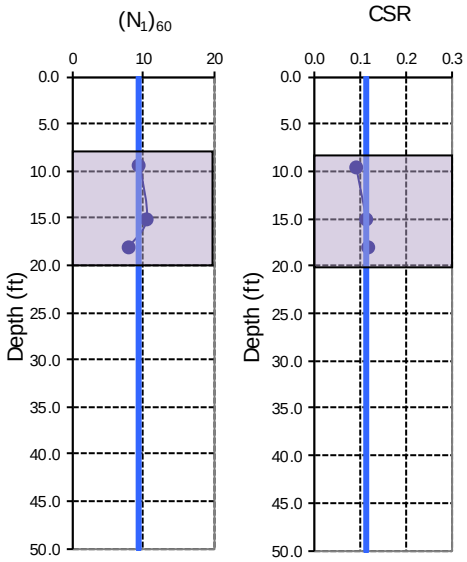


FIG. 10 BORING LOGS AT LA PLAYA SITE

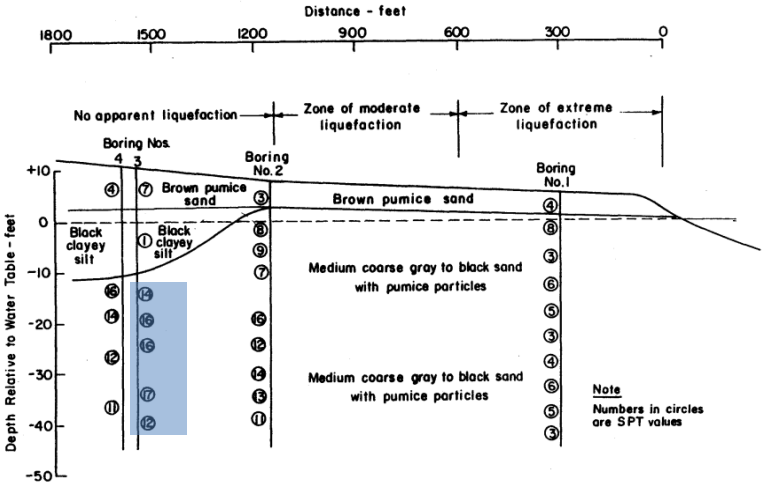


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.9	9.5	0.092	8	2.00	0.80	***9.6***
4.6	15.1	0.116	9	1.77	0.88	***10.5***
5.5	18.0	0.121	7	1.67	0.91	***8.0***
					Mean:	9.3
					St. Dev.	1.3

Case number:	29
Earthquake:	1976 Guatemala
Magnitude:	7.5 (ML)
Location:	Amatitlan B-3&4
References:	Seed et al. (1979)
Nature of Failure:	Located at the "no apparent liquefaction" zone
Comments:	The sites are located on deltaic deposit near the mouth of a small river, Rio Villalobos at the settlement of La Playa on the north-east shore of Lake Amatitlan.The general area behind the shoreline is quite flat with the ground sloping gently up about 2 degs away from the lake. Two test pits were dug to a depth of ~ 5' on the level ground 40' from the beach These showed the upper 4' of soil to be pumice sand underlain by a thin layer of dense fibrous peats and organic silt. Two field density test in the pumice sand showed that it had wet and dry densities ranging from 55.5-61.5 and 35.5-39.4 pcf respectively. Light-weight pumice sand at all boring locations was located above water table and was not itself the layer of liquefaction. The epicenter was located ~170 kms north-east from Lake Amatitlan However, north terminus of the fault is only 40 kms away from the lake. The Modified Mercalli Intensity of shaking at the site is high VI or low VII. Based on the correlations, PGA was estimated as 0.12-0.15 g. This estimation was checked by the level of damage at the nearby structures. Also attenuation relationships confirmed the range. Based on seismoscope records, USGS predicted the PGA at a site ~ 25 km from the causative fault as 0.25 g. Calibrating the attenuation relationships with this information, PGA at the site is predicted as 0.15 g. Local Magnitude saturates at magnitude 7. Thus, it is hard to estimate the moment magnitude for the earthquake. However based on the recorded motion and attenuation relationships, moment magnitude is back calculated as 7-8.2. Rope and pulley system (2 turns of the rope around the pulley) was used. Estimated rod energy is 45%. SPT values were taken after the earthquake. Amatitlan B-3 & 4 are ~40' apart.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.800 ± 0.050		0.8
Data Class	C			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	22.0 - 45.0	35.1	35.0	% PI			
Depth to GWT (ft)	12.5	11.2	11.0				
σ_v (psf)	2640.3 ± 349.2	2861.3	2860.0	N	15.1 ± 1.9	16.0	16
σ_v' (psf)	1329.3 ± 120.5	1482.9	1490.0	C_R	1.00	1.00	
a_{max} (g)	0.135 ± 0.041	0.14	0.135	C_S	1.00	1.00	
r_d	0.68 ± 0.128	0.89	0.890	C_B	1.00	1.00	
CSR	0.119 ± 0.043	0.149	0.150	C_E	0.75	0.75	0.75
Equivalent Magnitude	7.5	7.5		C_N	1.26	1.19	1.14
MSF		1.00	1.00	$(N_1)_{60}$	14.3 ± 1.8	14.3	14
CSRN		0.144	0.15				



γ_m=60±1 pcf

γ_s=90±1 pcf

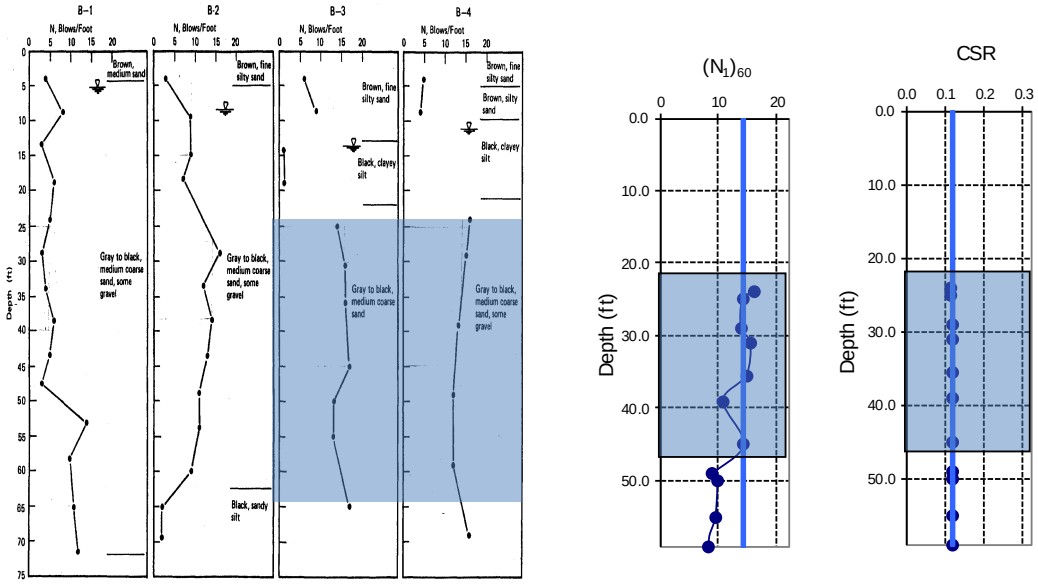


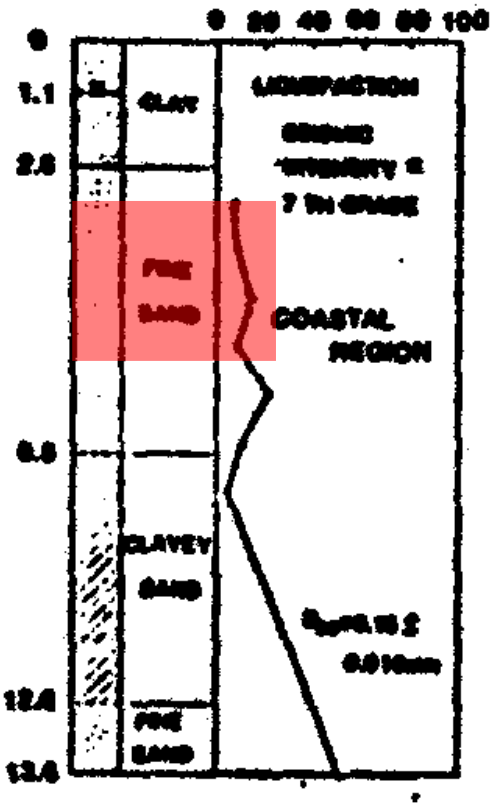
FIG. 10 BORING LOGS AT LA PLAYA SITE

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
7.3	24.0	0.116	16	1.41	0.96	***16.3***	B-4
7.6	25.0	0.117	14	1.39	0.97	***14.2***	B-3
8.8	29.0	0.119	14	1.33	1.00	***13.9***	B-4
9.4	31.0	0.119	16	1.30	1.00	***15.5***	B-3
10.8	35.5	0.119	16	1.24	1.00	***14.8***	B-3
11.9	39.0	0.118	12	1.20	1.00	***10.8***	B-4
13.7	45.0	0.118	17	1.13	1.00	***14.5***	B-3
14.9	49.0	0.118	11	1.10	1.00	9.1	B-4
15.2	50.0	0.118	12	1.09	1.00	9.8	B-3
16.8	55.0	0.120	12	1.05	1.00	9.4	B-3
18.0	59.0	0.121	11	1.02	1.00	8.4	B-4
19.8	65.0	0.123	17	0.98	1.00	12.5	B-3
21.0	69.0	0.124	16	0.96	1.00	11.5	B-4
					Mean:	14.3	
					St. Dev.	1.8	

Case number:	30
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Coastal Region
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied
Comments:	This site is located in the south-east of Tangshan City, south of Jing Shan Railway, in zone of liquefaction (Source document, Fig. 11, Qian Jian Yeng?). The site was identified as in the area of 7th grade on the Chinese intensity scale. Consisted essentially of flood plains formed by flooding from Luan and Dou Rivers. Luan river changed its river course frequently. Therefore, loose sand deposits were very common in the region. The Tangshan Earthquake had a focal depth of 12-16 km and epicenter in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

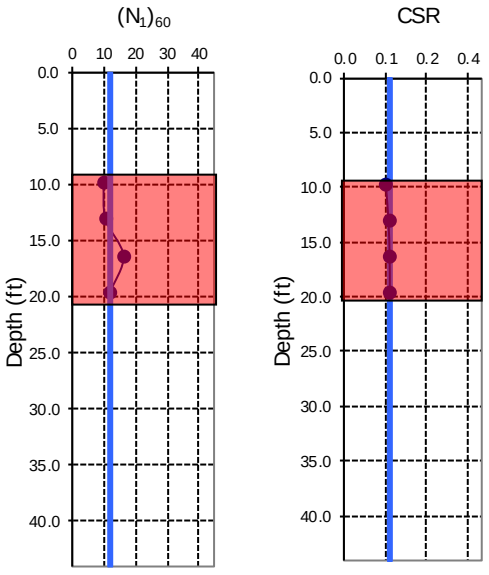
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.140 ± 0.050		0.14
Data Class	B			% Fines	12.0 ± 2.0	12.0	12
Critical Depth Range	9.8 - 19.7	14.8	20.0	% PI			
Depth to GWT (ft)	3.6 ± 0.4	3.6	4.0				
σ_v (psf)	1735.6 ± 200.0	1817.0	2440.0	N	9.5 ± 2.2	9.0	10
σ_v' (psf)	1039.5 ± 103.0	1127.8	1440.0	C_R	0.90	0.95	
a_{max} (g)	0.130 ± 0.039	0.13	0.130	C_S	1.00	1.00	
r_d	0.95 ± 0.064	0.97	0.960	C_B	1.00	1.00	
CSR	0.133 ± 0.041	0.130	0.140	C_E	1.00	1.00	1.00
Equivalent Magnitude	7.6	7.6		C_N	1.43	1.37	1.16
MSF		0.97	0.97	$(N_1)_{60}$	12.2 ± 2.9	11.7	11.5
CSRN		0.125	0.15				



γ_m=110±3 pcf

γ_s=120±3 pcf

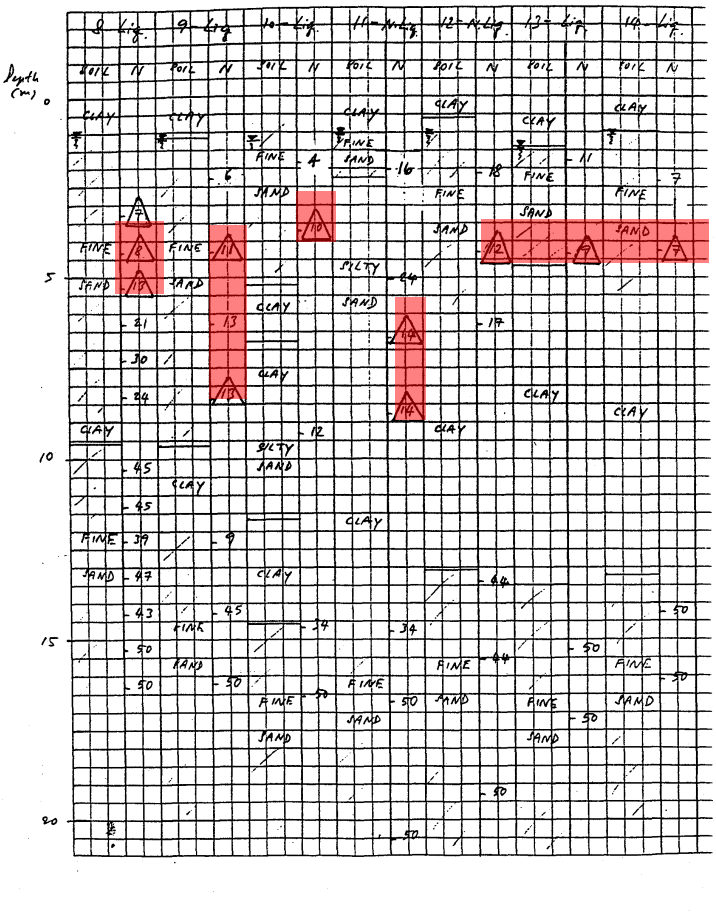


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.0	9.8	0.124	7.0	1.67	0.85	***9.9***
4.0	13.1	0.131	8.0	1.50	0.89	***10.6***
5.0	16.4	0.135	13.0	1.37	0.92	***16.4***
6.0	19.7	0.135	10.0	1.26	0.95	***12.0***
					Mean:	12.2
					St. Dev.	2.9

Case number:	31
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Le Ting L8-14
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied
Comments:	This site is located in the south-east of Tangshan City, south of Jing Shan Railway, in the zone of severe liquefaction (Source doc., Fig. 11, Qian Jian Yeng?). The site was identified as in the area of 7th grade on the Chinese intensity scale. Alluvial (Holocene) deposits. Manifestations of liquefaction were observed in the coastal region, south of Le Ting County. The Tangshan Earthquake had a focal depth of 12-16 km and epicenter in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. New borelogs were obtained in addition to the typical borelog (Fear et al, 1995) PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

Summary of Data

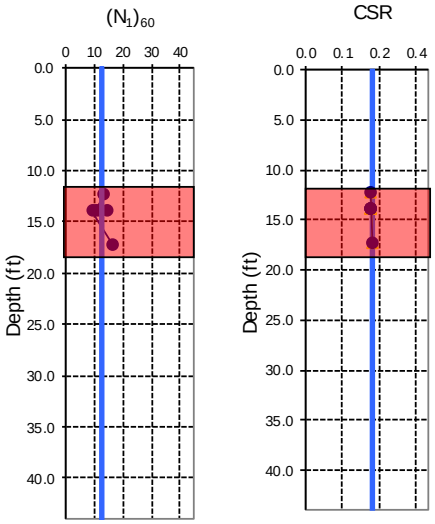
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.185 ± 0.040		0.22
Data Class	B			% Fines	12.0 ± 2.0	12.0	12
Critical Depth Range	11.5 - 19.7	14.4	6.7	% PI			
Depth to GWT (ft)	3.3 ± 0.6	4.9	5.0				
σ_v (psf)	1837.3 ± 168.5	1691.7	760.0	N	9.9 ± 2.0	9.7	10
σ_v' (psf)	1069.3 ± 92.4	1106.9	655.0	C_R	0.91	0.85	0.75
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.98 ± 0.067	0.97	0.990	C_B	1.00	1.00	
CSR	0.219 ± 0.068	0.194	0.155	C_E	1.00	1.00	1.00
Equivalent Magnitude	7.6	7.6		C_N	1.41	1.39	1.65
MSF		0.97	0.97	$(N_1)_{60}$	12.7 ± 2.6	11.5	12.5
CSRN		0.186	0.16				



γ_m=110±3 pcf

γ_s=120±3 pcf

137

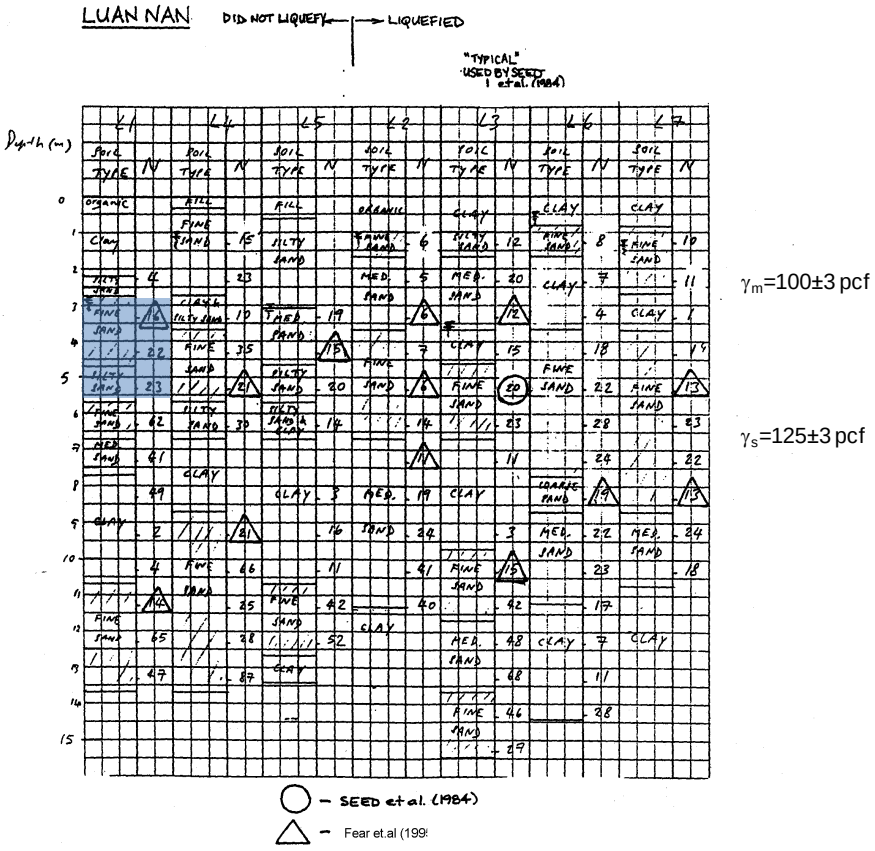


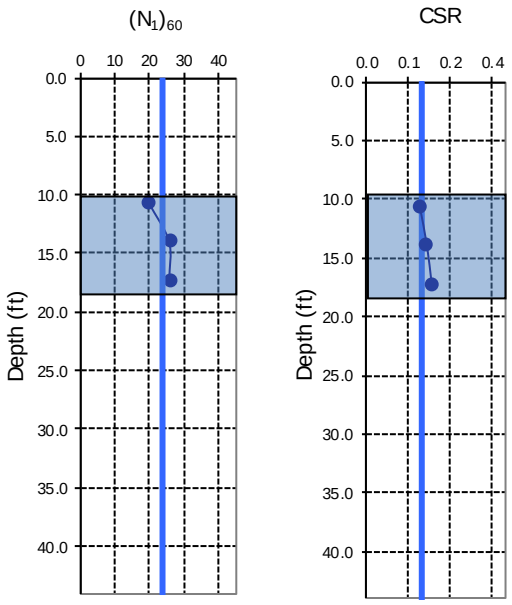
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
3.8	12.3	0.210	10.0	1.55	0.88	***13.6***	L10
4.3	13.9	0.215	8.0	1.47	0.90	***10.6***	L8
4.3	13.9	0.215	11.0	1.47	0.90	***14.5***	L9
4.3	13.9	0.215	9.0	1.47	0.90	***11.9***	L13
4.3	13.9	0.215	7.0	1.47	0.90	***9.2***	L14
5.3	17.2	0.221	13.0	1.35	0.93	***16.3***	L8
					Mean:	12.7	
					St. Dev.	2.6	

Case number:	32
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Luan Nan-L1
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	This site was classified as in the area of 8th grade on the Chinese intensity scale. It is located just south of the Jing Shan Railway, and south-east of Tangshan City, in the zone of liquefaction (Source document, fig.11). The Tangshan Earthquake had a focal depth of 12-16 km, the epicenter was located in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. Additional borelogs for this site were obtained from a report by the Beijing Municipal Bureau of City Planning (1982) (Fear et al., 1995). Based on newly available borelogs, it became apparent that representative N value selection by Seed et al (84) can be further improved. Borelog L2 is thought to be a better representative borelog. Probably the medium-fine sand layer liquefied at L2 liquefied first and excess pore pressure dissipated to the neighbouring layers causing them to liquefy as well. Seed et al (84) used borelog L3 (probably the rest were not available) PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.170 ± 0.050		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	4.9 - 18.0	11.5		% PI			
Depth to GWT (ft)	9.4 ± 0.3	3.6					
σ_v (psf)	1201.6 ± 275.0	1294.9		N	19.6 ± 2.9	19.3	
σ_v' (psf)	1068.5 ± 140.4	793.6		C_R	0.87	0.85	
a_{max} (g)	0.220 ± 0.066	0.22		C_S	1.00	1.00	
r_d	0.99 ± 0.052	0.98		C_B	1.00	1.00	
CSR	0.159 ± 0.051	0.226		C_E	1.00	1.00	
Equivalent Magnitude	7.6	7.6		C_N	1.41	1.49	
MSF		0.97		$(N_1)_{60}$	23.9 ± 3.6	24.4	
CSRN		0.211					



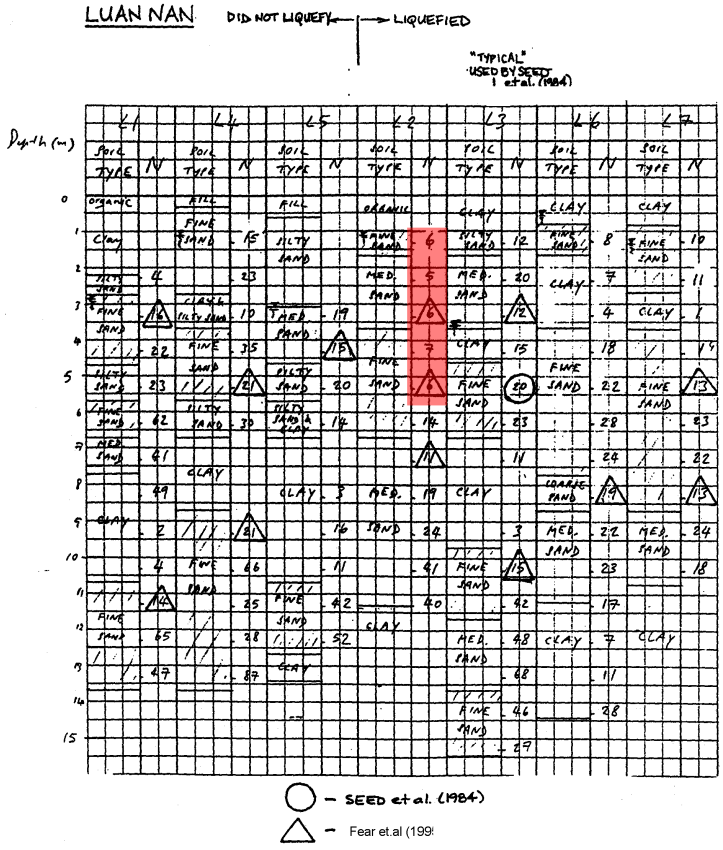


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.7	0.153	16.0	1.44	0.86	***19.8***
4.3	13.9	0.174	22.0	1.32	0.90	***25.9***
5.3	17.2	0.188	23.0	1.22	0.93	***26.0***
					Mean:	23.9
					St. Dev.	3.6

Case number:	33
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Luan Nan-L2
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied
Comments:	This site was classified as in the area of 8th grade on the Chinese intensity scale. It is located just south of the Jing Shan Railway, and south-east of Tangshan City in the zone of liquefaction (Source document, fig.11) The Tangshan Earthquake had a focal depth of 12-16 km and the epicenter was located in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. Additional borelogs for this site were obtained from a report by the Beijing Municipal Bureau of City Planning (1982) (Fear et al., 1995). Based on newly available borelogs, it became apparent that representative N value selection by Seed et al (84) can be further improved. Borelog L2 is thought to be a better representative borelog. Probably the medium-fine sand layer liquefied at L2 liquefied first and excess pore pressure dissipated to the neighbouring layers causing them to liquefy as well. Seed et al (84) used borelog L3 (probably the rest were not available) PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

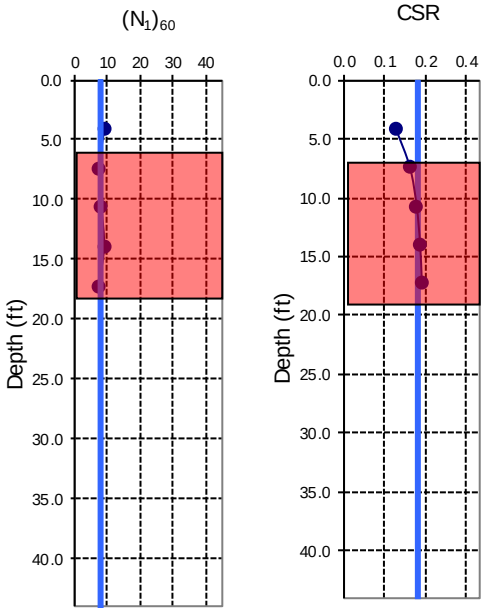
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.170 ± 0.050		
Data Class	B			% Fines	3.0 ± 2.0	3.0	
Critical Depth Range	4.9 - 18.0	11.5		% PI			
Depth to GWT (ft)	3.6 ± 0.3	3.6					
σ_v (psf)	1381.2 ± 274.7	1169.6		N	5.9 ± 0.6	5.9	
σ_v' (psf)	889.9 ± 140.1	668.3		C_R	0.87	0.85	
a_{max} (g)	0.220 ± 0.066	0.22		C_S	1.00	1.00	
r_d	0.98 ± 0.052	0.98		C_B	1.00	1.00	
CSR	0.217 ± 0.067	0.241		C_E	1.00	1.00	
Equivalent Magnitude	7.6	7.6		C_N	1.54	1.70	
MSF		0.97		$(N_1)_{60}$	7.9 ± 0.8	8.5	
CSRN		0.225					



γ_m=100±3 pcf

γ_s=125±3 pcf

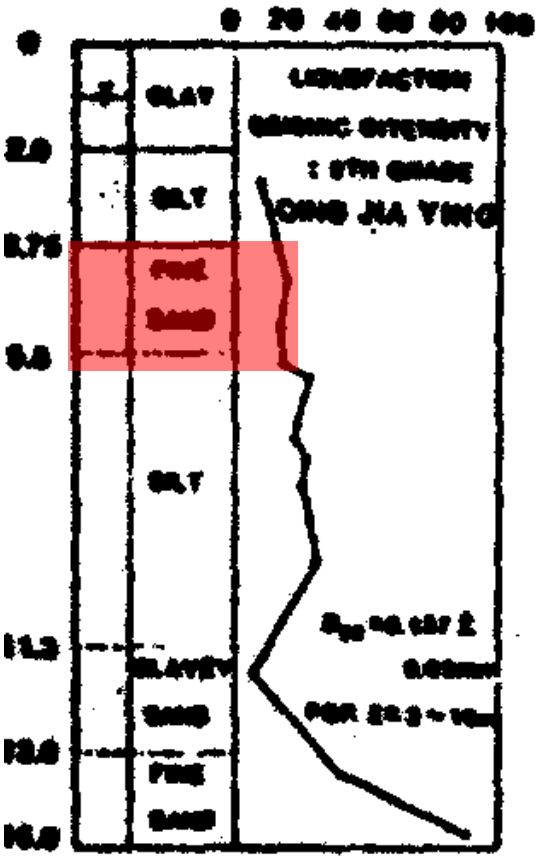


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.3	4.1	0.152	6.0	2.00	0.75	9.0	too close to surface
2.3	7.4	0.194	5.0	1.83	0.81	***7.4***	
3.3	10.7	0.214	6.0	1.59	0.86	***8.2***	
4.3	13.9	0.224	7.0	1.42	0.90	***8.9***	
5.3	17.2	0.229	6.0	1.30	0.93	***7.3***	
					Mean:	7.9	
					St. Dev.	0.8	

Case number:	34
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Qing Jia Ying
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Liquefied
Comments:	This site was classified as in the area of 9th grade on the Chinese intensity scale. It is located in just south of the Jing Shan Railway, and south-east of Tangshan City in the zone of liquefaction (Source document, fig.11., Qian Jia Yeng?) The Tangshan Earthquake had a focal depth of 12-16 km and the epicenter was located in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

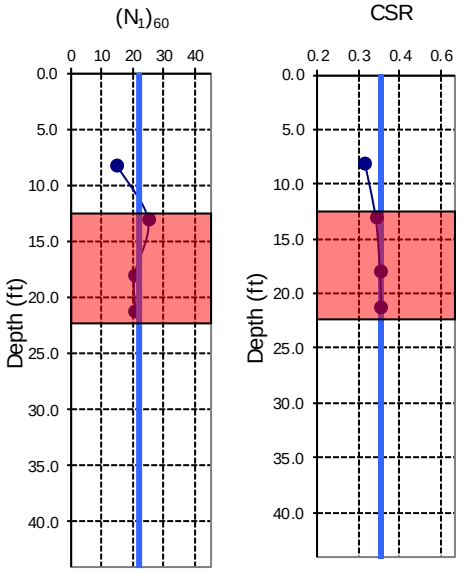
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.137 ± 0.050		0.137
Data Class	B			% Fines	20.0 ± 2.0	20.0	20
Critical Depth Range	14.8 - 21.3	17.4	17.4	% PI			
Depth to GWT (ft)	3.0 ± 0.4	3.0	3.0				
σ_v (psf)	2211.3 ± 144.4	2130.3	2130.0	N	18.3 ± 2.1	17.0	17.0
σ_v' (psf)	1269.6 ± 84.7	1232.2	1230.0	C_R	0.94	0.95	
a_{max} (g)	0.350 ± 0.105	0.35	0.350	C_S	1.00	1.00	
r_d	0.97 ± 0.076	0.96	0.960	C_B	1.00	1.00	
CSR	0.385 ± 0.120	0.378	0.380	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.6	7.6		C_N	1.29	1.24	1.3
MSF		0.97	0.96	$(N_1)_{60}$	22.1 ± 2.5	20.1	21.50
CSRN		0.357	0.40				



γ_m=110±3 pcf

γ_s=125±3 pcf

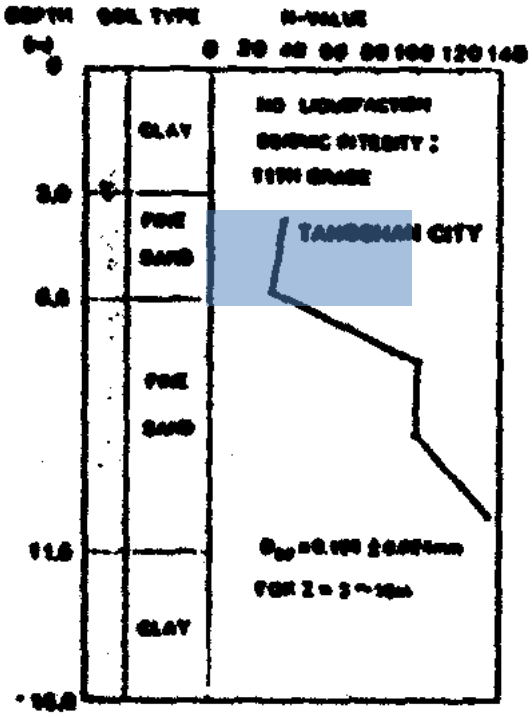


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.5	8.2	0.339	10.0	1.80	0.82	14.8	silt-clay contact
4.0	13.1	0.372	19.0	1.48	0.89	***25.0***	
5.5	18.0	0.385	17.0	1.29	0.94	***20.6***	
6.5	21.3	0.389	18.0	1.20	0.96	***20.8***	
					Mean:	22.1	
					St. Dev.	2.5	

Case number:	35
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Tangshan City
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Non-liquefied site
Comments:	This site was classified as in the area of 11th grade on the Chinese intensity scale. It is located in just north of the Jing Shan Railway, and north-west of the the epicentral area. Sandy layers are generally dense with the relative density values of 80 to 90 % and the ground water level was low, at ~ 5-10 m. below the ground surface. The Tangshan Earthquake had a focal depth of 12-16 km and the epicenter was located in Tangshan City. The area based on seismic intensity may be divided into two regions by the Jing-Shan railway. The north of the railway is mainly of hill areas, and experienced relatively lower seismic intensity. The south of the railway is mainly of flood plains including plains of marine and continental sediments. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

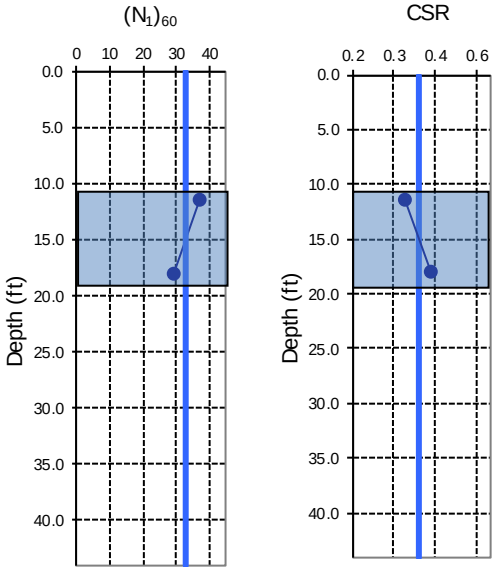
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.196 ± 0.050		0.196
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	11.5 - 18.0	17.4	17.5	% PI			
Depth to GWT (ft)	9.8 ± 0.3	10.2	10.0				
σ_v (psf)	1697.8 ± 140.7	2046.8	2040.0	N	29.7 ± 5.0	30.0	30.0
σ_v' (psf)	1390.7 ± 77.3	1566.4	1570.0	C_R	0.90	0.95	
a_{max} (g)	0.500 ± 0.150	0.50	0.500	C_S	1.00	1.00	
r_d	1.00 ± 0.064	0.96	0.960	C_B	1.00	1.00	
CSR	0.395 ± 0.122	0.405	0.405	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.6	7.6		C_N	1.23	1.11	1.1
MSF		0.97	0.96	$(N_1)_{60}$	33.1 ± 5.6	31.6	33.50
CSRN		0.389	0.42				



γ_m=110±3 pcf

γ_s=125±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.5	11.5	0.352	32.0	1.34	0.87	*37.1*
5.5	18.0	0.427	27.0	1.15	0.94	*29.1*
					Mean:	33.1
					St. Dev.	5.6

Case number:	36
Earthquake:	1976 Tangshan
Magnitude:	7.6 (Mw) USGS Centennial Earthquake Catalog
Location:	Yao Yuan Village
References:	Shengcong et al (1983) Fear et al. (1995)
Nature of Failure:	Sand swept in the canal; depression due to liquefaction.
Comments:	An extraordinary level of damage to one-story house due to soil liquefaction was observed in Yao Yuan Village, near Wang Tan, Le Ting County. A big sand boil into a canal due to soil liquefaction was observed. Correspondingly, a big depression appeared. The length of the depression was around 30m, the width and depth were around 10 m and 4m respectively. Assuming that the groundwater level was the same with the water level in the canal (at 1 m depth from the ground surface), the maximum pore pressure in the liquefied sand layer at depth of 2 m was estimated as ~ 30kN/m2 which was 3 times as large as the water pressure in the canal. It seems that this pressure gradient induced the horizontal migration of pore water, which was easier to occur than the vertical migration due to "impermeable" top clay layer. PGA was estimated as >0.1 g SPT energy was estimated as 60 % by Seed et al. (84)

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.150 ± 0.050		Silt and sand
Data Class	B			% Fines	20.0 ± 2.0	20.0	Silt and sand
Critical Depth Range	11.5 - 16.4	20.0	20.0	% PI			
Depth to GWT (ft)	3.3 ± 0.4	3.0	3.0				
σ_v (psf)	1693.7 ± 108.0	2464.5	2455.0	N	8.7 ± 3.8	9.0	9.0
σ_v' (psf)	1028.4 ± 63.5	1399.3	1395.0	C_R	0.90	0.95	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.97 ± 0.061	0.95	0.960	C_B	1.00	1.00	
CSR	0.208 ± 0.064	0.218	0.220	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.6	7.6		C_N	1.43	1.22	1.2
MSF		0.97	0.96	$(N_1)_{60}$	11.1 ± 4.9	10.5	11.00
CSRN		0.214	0.23				

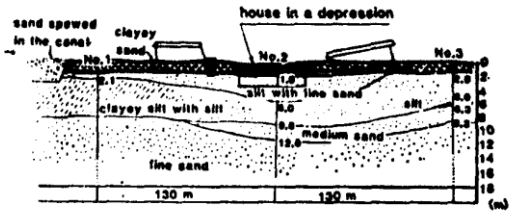


Fig. 19. House sunk into ground, Yao Yuan village, Le Ting county, induced during Tangshan earthquake (reproduced from Fang et al, 1981)

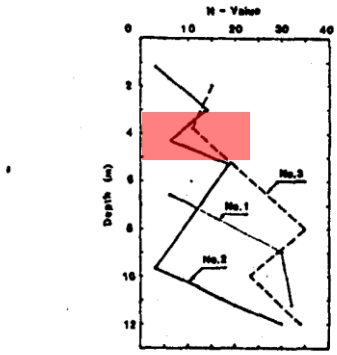
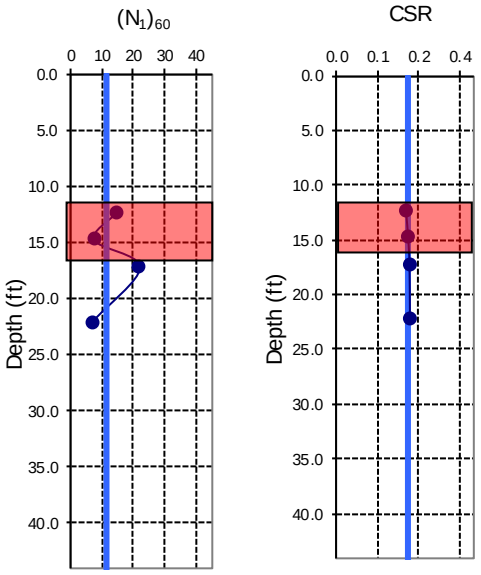


Fig. 20. *N*-values measured at Yao Yuan village, Le Ting county (reproduced from Fang et al, 1981)

γ_m=110±3 pcf

γ_s=125±3 pcf

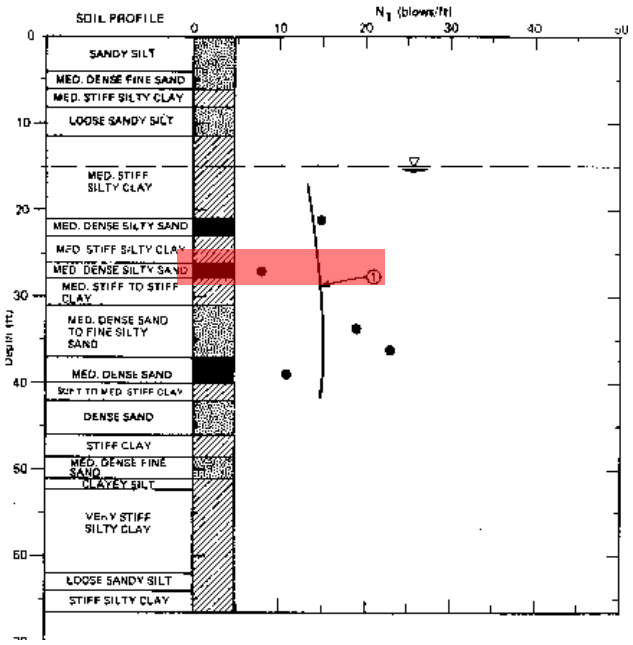


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
3.8	12.3	0.204	11.0	1.51	0.88	***14.6***	No:3 Silt
4.5	14.8	0.209	6.0	1.40	0.90	***7.6***	No:2 Silt with fine sand
5.3	17.2	0.212	18.0	1.31	0.93	21.9	No:2 Silt with fine sand
6.8	22.1	0.213	6.0	1.17	0.97	6.8	No:1 Clayey silt
					Mean:	11.1	
					St. Dev.	4.9	

Case number:	37
Earthquake:	1977 Argentina
Magnitude:	7.5 (Mw) USGS Centennial Earthquake Catalog
Location:	San Juan B-1
References:	Idriss et al. (1979) Fear et al. (1995)
Nature of Failure:	At several locations, the surface manifestation of liquefaction has been observed in association with abandoned fluvial deposits
Comments:	The boreholes were drilled at Dept. of Caucete in the Barrio Justo P. Castro, newly constructed single family stucco homes. The borehole is located northeast of Crevasse. Ground response analyses were performed based on generalized soil profile: 0-700 ft : sand with lenses of silt and clay 700-1400 ft : gravel >1400 ft : limestone, claystone and sand stone Shear moduli values were estimated from SPT values; Borehole1,3&6 and 2,4&5 were grouped for the analyses. The drainage was prevented at the surface by the clay layer. PGA record was made available by INPRES in San Juan and corrected by USGS. A PGA of 0.187 g was recorded at the seismographic station which was located within 10 km of the study area. Average unit weight of soil was estimated as 110 pcf. Dry Unit weight was reported in the range of 86-93 pcf for silty clay (CL) layer in Table B-1. Estimated rod energy was 45%. N values were back calculated from N_1 plots by using above summarized info.

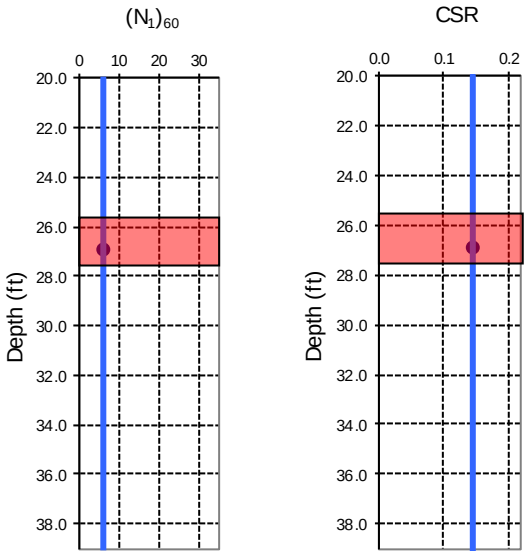
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.140 ± 0.050		0.15
Data Class	B			% Fines	20.0 ± 2.0	20.0	20
Critical Depth Range	26.0 - 28.0	26.9	27.0	% PI			
Depth to GWT (ft)	15.0	15.1	15.0				
σ_v (psf)	3150.0 ± 71.4	2965.7	2970.0	N	9.0 ± 2.9	9.0	9.0
σ_v' (psf)	2401.2 ± 64.2	2213.9	2220.0	C_R	0.98	0.95	↑
a_{max} (g)	0.200 ± 0.030	0.20	0.200	C_S	1.00	1.00	↕
r_d	0.86 ± 0.107	0.92	0.940	C_B	1.00	1.00	↕
CSR	0.146 ± 0.029	0.160	0.165	C_E	0.75	0.75	↓ 0.750
Equivalent Magnitude	7.5	7.5		C_N	0.94	0.97	1.0
MSF		1.00	0.97	$(N_1)_{60}$	6.2 ± 2.0	6.3	6.00
CSRN		0.161	0.16				



γ_m=110±3 pcf

γ_s=125±3 pcf



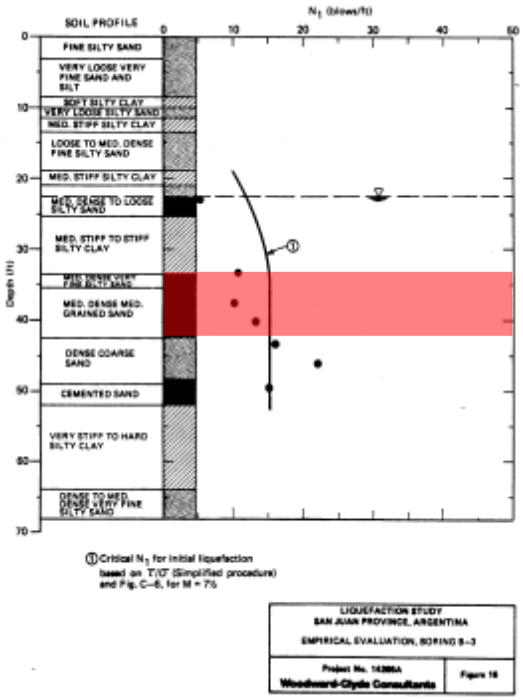
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
8.2	26.9	0.146	9	0.94	0.98	***6.2***
					Mean:	6.2
					St. Dev.	2.0

assigned

Case number:	38
Earthquake:	1977 Argentina
Magnitude:	7.5 (Mw) USGS Centennial Earthquake Catalog
Location:	San Juan B-3
References:	Idriss et al. (1979) Fear et al. (1995)
Nature of Failure:	At several locations, the surface manifestation of liquefaction has been observed in association with abandoned fluvial deposits.
Comments:	The borehole was located in north of downtown Caucete on private property which suffered substantial damage during the earthquake. Ground response analyses were performed based on generalized soil profile: 0-700 ft : sand with lenses of silt and clay 700-1400 ft : gravel >1400 ft : limestone, claystone and sand stone Shear moduli values were estimated from SPT values; Borehole1,3&6 and 2,4&5 were grouped for the analyses. The drainage was prevented at the surface by the clay layer. PGA record was made available by INPRES in San Juan and corrected by USGS. A PGA of 0.187 g was recorded at the subject seismographic station which was located within 10 km of the study area. Average unit weight of soil was estimated as 110 pcf. Dry Unit weight was reported in the range of 86-93 pcf in Table B-1. Estimated rod energy was 45% N values are back calculated from N1 plots by using above summarized info.

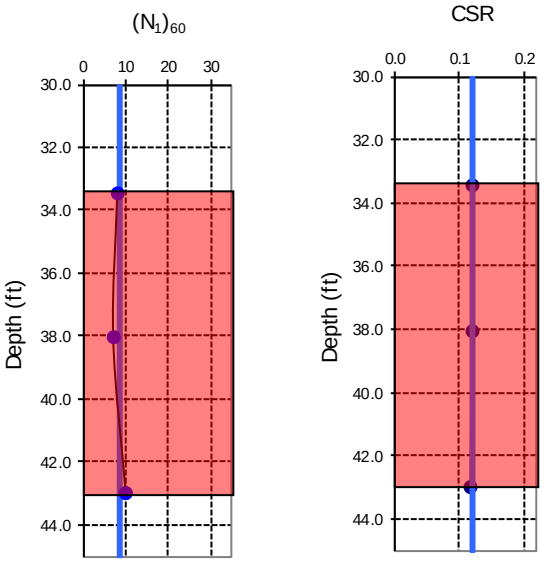
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.140 ± 0.050		SP
Data Class	B			% Fines	20.0 ± 2.0	5.0	
Critical Depth Range	33.5 - 43.0	36.4	39.0	% PI			
Depth to GWT (ft)	22.0	22.0	22.0				
σ_v (psf)	4370.0 ± 207.0	4156.2	3190.0	N	14.1 ± 2.5	13.0	12.0
σ_v' (psf)	3356.0 ± 123.7	3258.1	2130.0	C_R	1.00	1.00	
a_{max} (g)	0.200 ± 0.030	0.20	0.200	C_S	1.00	1.00	
r_d	0.71 ± 0.144	0.87	0.860	C_B	1.00	1.00	
CSR	0.120 ± 0.030	0.169	0.165	C_E	0.75	0.75	0.750
Equivalent Magnitude	7.5	7.5		C_N	0.79	0.78	1.0
MSF		1.00	0.97	$(N_1)_{60}$	8.4 ± 1.5	7.6	8.50
CSRN		0.169	0.16				



γ_m=110±3 pcf

γ_s=120±3 pcf

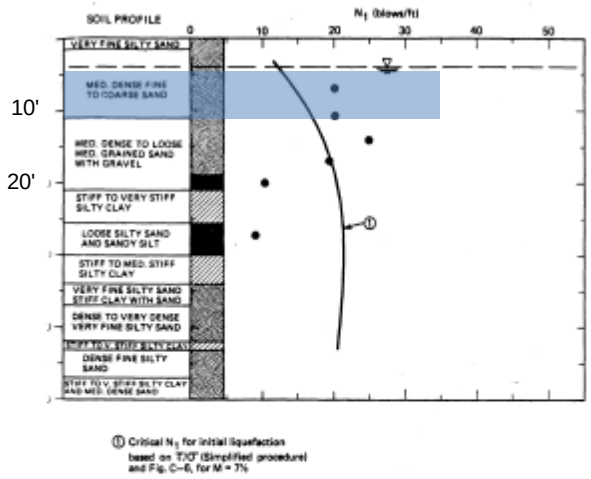


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
10.2	33.5	0.120	13	0.83	1.00	***8.1***
11.6	38.1	0.120	12	0.80	1.00	***7.2***
13.1	43.0	0.119	17.5	0.76	1.00	***10.0***
					Mean:	8.4
					St. Dev.	1.5

Case number:	39
Earthquake:	1977 Argentina
Magnitude:	7.5 (Mw) USGS Centennial Earthquake Catalog
Location:	San Juan B-4
References:	Idriss et al. (1979) Fear et al. (1995)
Nature of Failure:	There is no surface evidence of liquefaction
Comments:	The borehole is located in the Dept. of 9 de Julio in the Finca Delta; It is at the corner of vineyard. Ground response analyses were performed based on generalized soil profile: 0-700 ft : sand with lenses of silt and clay 700-1400 ft : gravel >1400 ft : limestone, claystone and sand stone Shear moduli values were estimated from SPT values; Borehole1,3&6 and 2,4&5 were grouped for the analyses. PGA record was made available by INPRES in San Juan and corrected by USGS. A PGA of 0.187 g was recorded at the seismographic station which was located within 10 km of the study area. Average unit weight of soil was estimated as 110 pcf. Dry Unit weight was reported in the range of 86-93 pcf in Table B-1. Estimated rod energy was 45% N values were back calculated from N_1 plots by using above summarized info.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.290 ± 0.050		0.29
Data Class	B			% Fines	4.0 ± 2.0	4.0	4
Critical Depth Range	4.0 - 12.0	12.1	12.0	% PI			
Depth to GWT (ft)	4.0	3.9	4.0				
σ_v (psf)	940.0 ± 167.6	1315.8	1320.0	N	13.6 ± 0.7	14.0	14.0
σ_v' (psf)	690.4 ± 86.4	814.5	820.0	C_R	0.77	0.85	
a_{max} (g)	0.200 ± 0.030	0.20	0.200	C_S	1.00	1.00	
r_d	0.98 ± 0.038	0.98	0.970	C_B	1.00	1.00	
CSR	0.173 ± 0.029	0.204	0.200	C_E	0.75	0.75	0.750
Equivalent Magnitude	7.5	7.5		C_N	1.75	1.60	1.5
MSF		1.00	0.98	$(N_1)_{60}$	13.8 ± 0.7	14.3	16.00
CSRN		0.186	0.20				

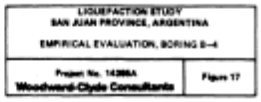


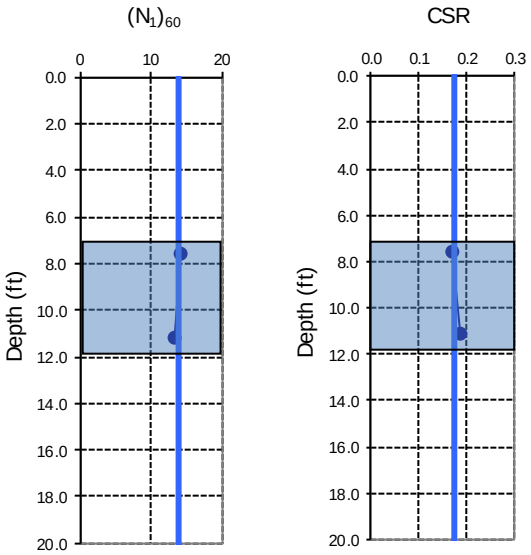
10'

20'

γ_m=110±3 pcf

γ_s=125±3 pcf



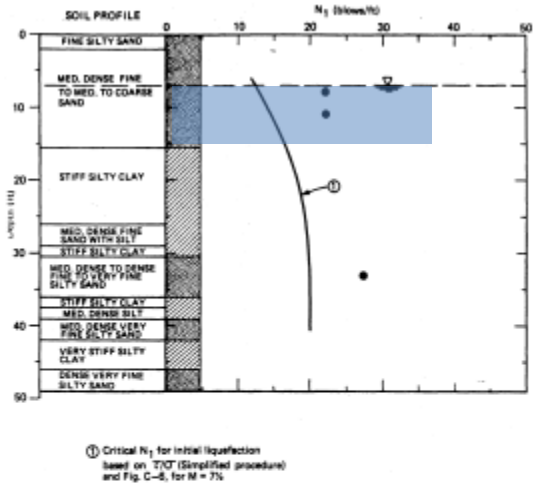


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.3	7.5	0.170	14	1.79	0.76	***14.3***
3.4	11.2	0.188	14	1.54	0.82	***13.3***
					Mean:	13.8
					St. Dev.	0.7

Case number:	40
Earthquake:	1977 Argentina
Magnitude:	7.5 (Mw) USGS Centennial Earthquake Catalog
Location:	San Juan B-5
References:	Idriss et al. (1979) Fear et al. (1995)
Nature of Failure:	No apparent damage at nearby structures; no sand boils or other surface evidence of liquefaction.
Comments:	The borehole was located in the Dept. of 9 de Julio in the Finca La Porfia about 1.5 km east of route 20. It was drilled near the center of an old filled-in tributary channel. Ground response analyses were performed based on generalized soil profile: 0-700 ft : sand with lenses of silt and clay 700-1400 ft : gravel >1400 ft : limestone, claystone and sand stone Shear moduli values were estimated from SPT values; Borehole1,3&6 and 2,4&5 were grouped for the analyses. PGA record was made available by INPRES in San Juan and corrected by USGS. A PGA of 0.187 g was recorded at the subject seismographic station which was located within 10 km of the study area. Average unit weight of soil was estimated as 110 pcf. Dry Unit weight was reported in the range of 86-93 pcf in Table B-1. Estimated rod energy was 45% N values were back calculated from N_1 plots by using above summarized info.

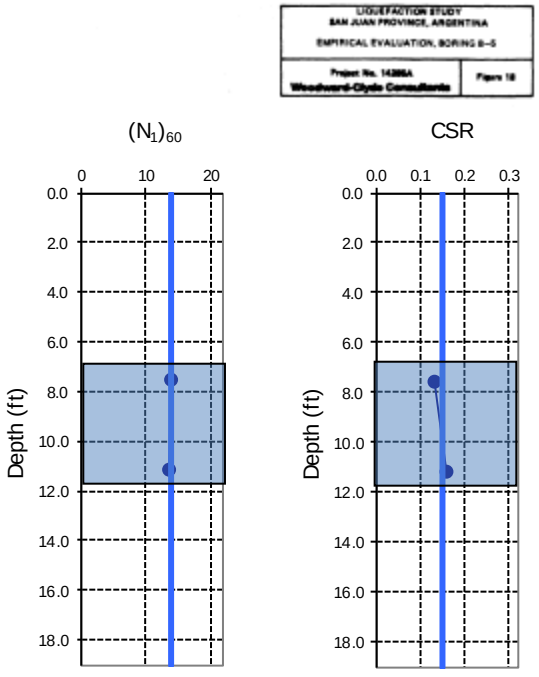
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.240 ± 0.050		0.24
Data Class	B			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	7.0 - 12.0	10.2	10.0	% PI			
Depth to GWT (ft)	7.0	6.9	7.0				
σ_v (psf)	1082.5 ± 106.6	1106.9	1100.0	N	15.3 ± 0.1	14.0	14.0
σ_v' (psf)	926.5 ± 58.5	919.0	910.0	C_R	0.80	0.85	
a_{max} (g)	0.200 ± 0.030	0.20	0.200	C_S	1.00	1.00	
r_d	0.99 ± 0.044	0.98	0.980	C_B	1.00	1.00	
CSR	0.150 ± 0.024	0.154	0.155	C_E	0.75	0.75	0.750
Equivalent Magnitude	7.5	7.5		C_N	1.51	1.53	1.4
MSF		1.00	0.97	$(N_1)_{60}$	13.8 ± 0.1	13.6	14.50
CSRN		0.142	0.15				



γ_m=110±3 pcf

γ_s=125±3 pcf

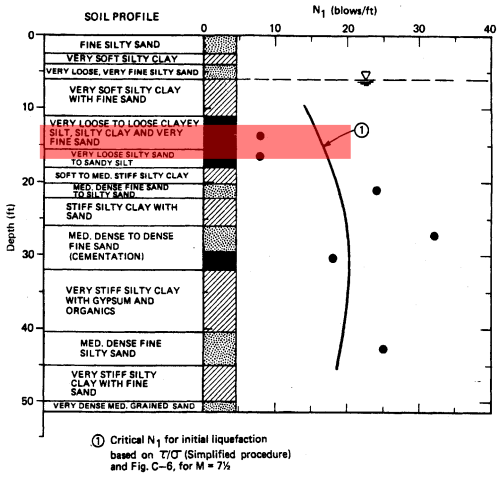


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.3	7.5	0.134	15	1.62	0.76	***13.9***
3.4	11.2	0.160	15.5	1.43	0.82	***13.7***
					Mean:	13.8
					St. Dev.	0.1

Case number:	41
Earthquake:	1977 Argentina
Magnitude:	7.5 (Mw) USGS Centennial Earthquake Catalog
Location:	San Juan B-6
References:	Idriss et al. (1979) Fear et al. (1995)
Nature of Failure:	Nearby structures were damaged; sand boils; stairway collapsed, concrete posts were sheared off.
Comments:	The borehole is located in the Dept. of 25 de Mayo at the Finca Santiago adjacent to La Plata Highway. Ground response analyses were performed based on generalized soil profile: 0-700 ft : sand with lenses of silt and clay 700-1400 ft : gravel >1400 ft : limestone, claystone and sand stone Shear moduli values were estimated from SPT values; Borehole1,3&6 and 2,4&5 were grouped for the anayses. PGA record was made available by INPRES in San Juan and corrected by USGS. A PGA of 0.187 g was recorded at the seismographic station which was located within 10 km of the study area. Average unit weight of soil was estimated as 110 pcf. Dry Unit weight was reported in the range of 86-93 pcf in Table B-1. Estimated rod energy was 45% N values were back calculated from N_1 plots by using above summarized info.

Summary of Data

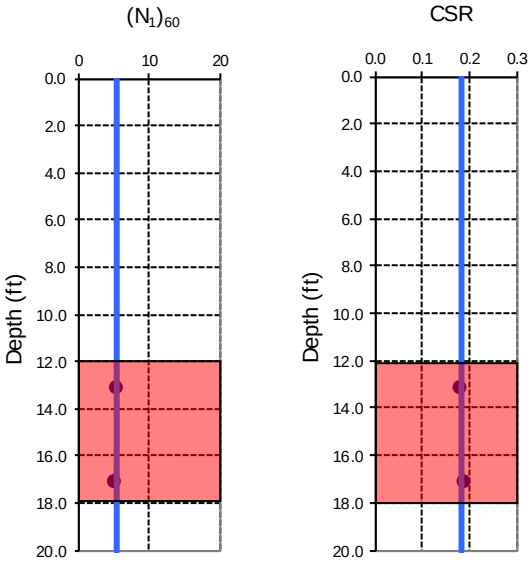
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.100 ± 0.050		SM-ML
Data Class	B			% Fines	50.0 ± 2.0	50.0	50
Critical Depth Range	12.0 - 18.0	17.1	17.0	% PI			
Depth to GWT (ft)	6.0	5.9	6.0				
σ_v (psf)	1740.0 ± 124.3	1879.7	1870.0	N	6.0 ± 0.2	6.0	6.0
σ_v' (psf)	1178.4 ± 68.0	1169.6	1180.0	C_R	0.87	0.95	
a_{max} (g)	0.200 ± 0.030	0.20	0.200	C_S	1.00	1.00	
r_d	0.96 ± 0.065	0.96	0.960	C_B	1.00	1.00	
CSR	0.184 ± 0.031	0.198	0.195	C_E	0.75	0.75	0.750
Equivalent Magnitude	7.5	7.5		C_N	1.34	1.36	1.3
MSF		1.00	0.97	$(N_1)_{60}$	5.3 ± 0.2	5.8	6.00
CSRN		0.187	0.19				



γ_m=110±3 pcf

γ_s=120±3 pcf

LIQUEFACTION STUDY
SAN JUAN PROVINCE, ARGENTINA
EMPIRICAL EVALUATION, BORING B–6
Project No. 14285A
Woodward-Clyde Consultants
Figure 19

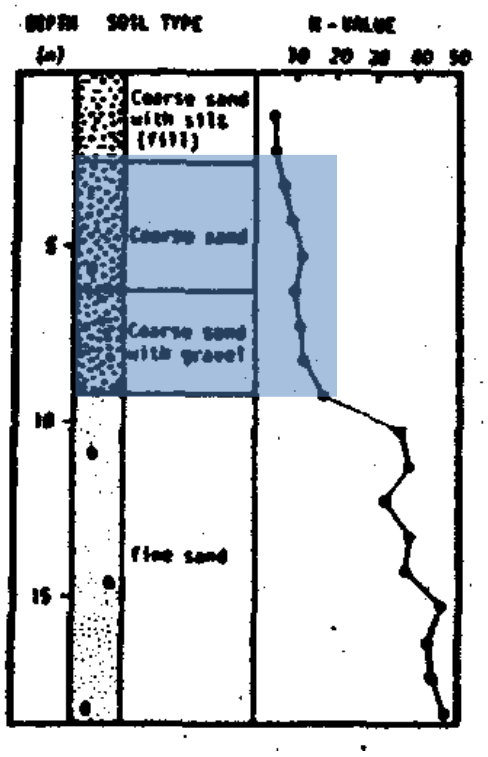


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4	13.1	0.178	6	1.41	0.85	***5.4***
5.2	17.1	0.189	6	1.28	0.90	***5.2***
					Mean:	5.3
					St. Dev.	0.2

Case number:	42
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Arahama
References:	Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No apparent evidence of liquefaction
Comments:	The site is located in Watari Town, district of Arahama. Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Arahama area is covered with very soft sandy deposits; about 9 m in thickness, N values generally less than 10. SPT values were taken before main earthquake (M=7.4) in 1974 SPT energy is estimated as 65 % by Seed et al. (84)

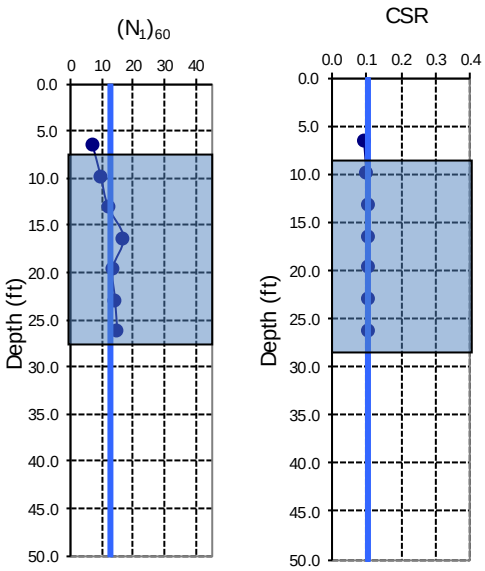
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.450 ± 0.050		0.6
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	6.6 - 26.2	21.0	21.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	1938.5 ± 395.9	2527.1	2520.0	N	8.9 ± 2.4	10.0	10.0
σ_v' (psf)	1102.1 ± 194.1	1399.3	1400.0	C_R	0.92	0.95	
a_{max} (g)	0.100 ± 0.030	0.10	0.100	C_S	1.00	1.00	
r_d	0.94 ± 0.070	0.90	0.950	C_B	1.00	1.00	
CSR	0.107 ± 0.033	0.105	0.110	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.5	6.5		C_N	1.39	1.23	1.2
MSF		1.34	1.16	$(N_1)_{60}$	12.4 ± 3.3	12.8	13.00
CSRN		0.076	0.10				



γ_m=110±3 pcf

γ_s=120±3 pcf

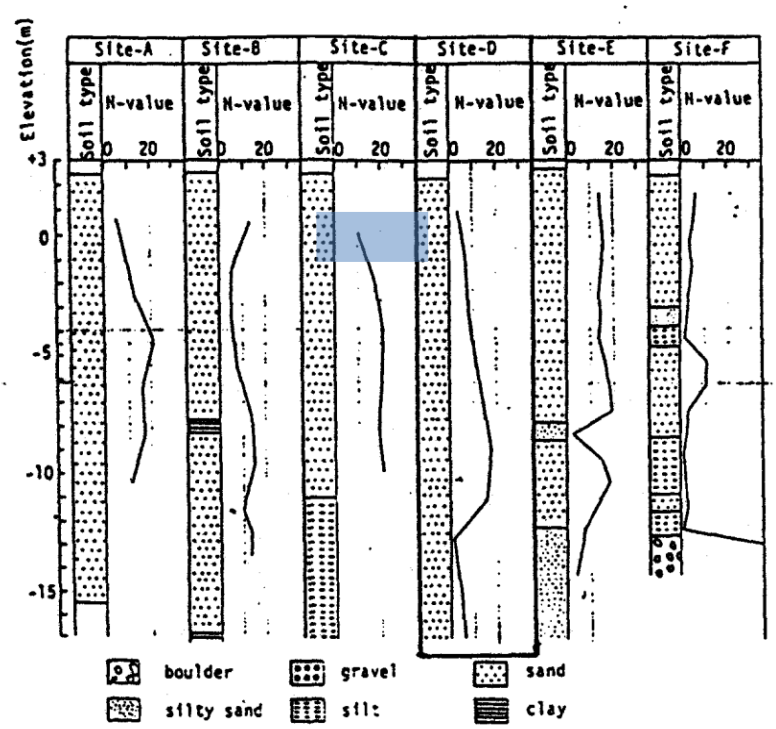


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	0.091	4	1.99	0.80	***6.9***
3.0	9.8	0.100	6	1.71	0.85	***9.5***
4.0	13.1	0.105	8	1.52	0.89	***11.8***
5.0	16.4	0.107	12	1.39	0.92	***16.7***
6.0	19.7	0.107	10	1.28	0.95	***13.3***
7.0	23.0	0.106	11	1.20	0.98	***14.0***
8.0	26.2	0.103	12	1.13	1.00	***14.8***
					Mean:	12.4
					St. Dev.	3.3

Case number:	43
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Hiyori-18
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Hiyori-18 site was claimed by Tokimatsu (1993) to be by Site C. SPT energy was estimated as 65 % by Seed et al. (84)

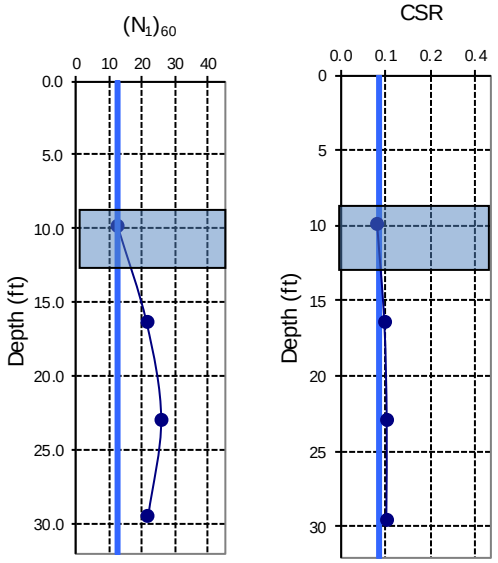
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.150 ± 0.050		0.15
Data Class	B			% Fines	20.0 ± 2.0	20.0	20
Critical Depth Range	8.2 - 13.1	17.1	17.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0	D N fines			
σ_v (psf)	1199.5 ± 101.7	2046.8	2040.0	N	9.1 ± 1.5	9.0	9.0
σ_v' (psf)	1033.4 ± 55.8	1482.9	1480.0	C_R	0.86	0.95	
a_{max} (g)	0.140 ± 0.042	0.14	0.140	C_S	1.00	1.00	
r_d	0.98 ± 0.048	0.93	0.960	C_B	1.00	1.00	
CSR	0.103 ± 0.032	0.116	0.120	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.5	6.5		C_N	1.43	1.19	1.2
MSF		1.34	1.14	$(N_1)_{60}$	12.2 ± 2.0	11.1	11.50
CSRN		0.084	0.11				



γ_m=110±3 pcf

γ_s=120±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.0	9.8	0.099	9.0	1.46	0.85	***12.2***
5.0	16.4	0.120	17.0	1.25	0.92	21.3
7.0	23.0	0.127	22.0	1.10	0.98	25.9
9.0	29.5	0.125	20.0	1.00	1.00	21.8
11.0	36.1	0.120	18.0	0.92	1.00	18.1
					Mean:	12.2
					St. Dev.	2.0

assigned

Case number:	44
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Ishinomaki-2
References:	Ishihara et al. (1980) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 The site is near the fishery port in the city of Ishinomaki. Loose sand deposit to a depth of 12 m has blow count values on the order of 5. The site has been improved by vibroflotation and compaction piles in 1975. No significant change in SPT values before and after the earthquake. SPT energy was estimated as 65 % by Seed et al. (84)

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.150 ± 0.050		0.15
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	4.6 - 19.7	11.5	13.0	% PI			
Depth to GWT (ft)	4.6 ± 2.0	4.6	5.0				
σ_v (psf)	1410.8 ± 303.6	1378.4	1560.0	N	4.0 ± 0.3	3.7	4.0
σ_v' (psf)	939.9 ± 179.9	939.8	1060.0	C_R	0.88	0.85	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.92 ± 0.054	0.96	0.970	C_B	1.00	1.00	
CSR	0.107 ± 0.036	0.109	0.110	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.5	6.5		C_N	1.50	1.61	1.3
MSF		1.34	1.10	$(N_1)_{60}$	5.7 ± 0.5	5.5	6.00
CSRN		0.076	0.10				

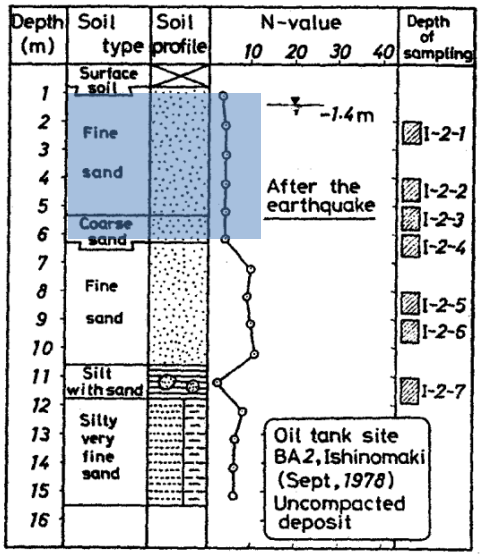


Fig. 14. Standard penetration resistance and depths of Osterberg sampling at the uncompacted site after the earthquake

γ_m=110±3 pcf

γ_s=120±3 pcf

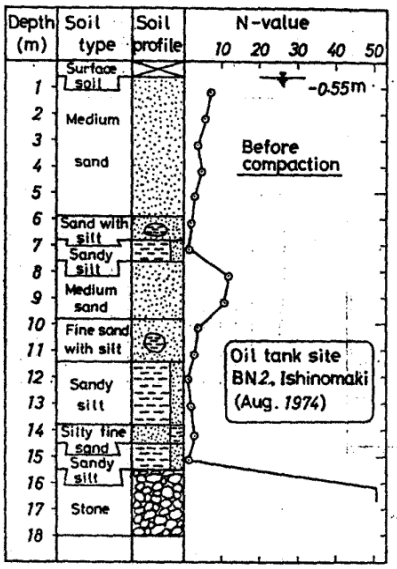
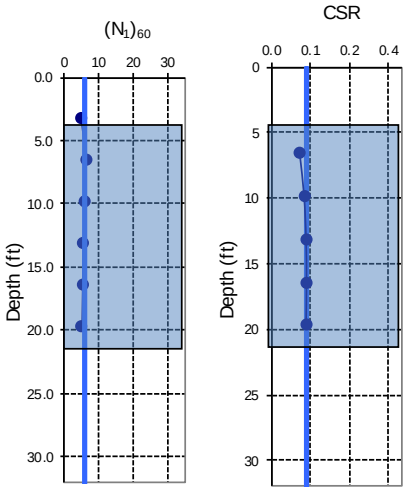


Fig. 11. A soil profile and standard penetration resistance before compaction

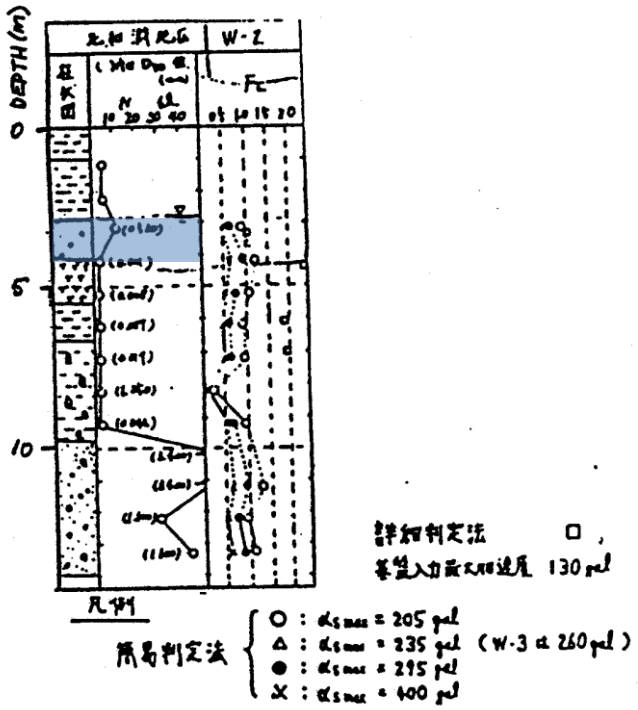


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	3.0	2.00	0.73	4.8	too close to surf.
2.0	6.6	0.090	4.0	1.85	0.80	***6.4***	
3.0	9.8	0.103	4.0	1.62	0.85	***6.0***	
4.0	13.1	0.108	4.0	1.46	0.89	***5.6***	
5.0	16.4	0.110	4.0	1.34	0.92	***5.4***	
6.0	19.7	0.108	4.0	1.24	0.95	***5.2***	
					Mean:	5.7	
					St. Dev.	0.5	

Case number:	45
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Kitawabuchi 2
References:	Iwasaki et al. (1981) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

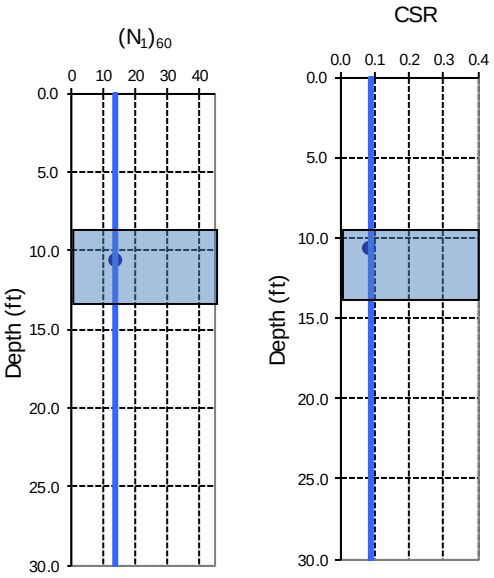
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.530 ± 0.050		0.53
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	9.8 - 13.1	11.2	11.0	% PI			
Depth to GWT (ft)	9.8 ± 0.3	10.2	10.0				
σ_v (psf)	1181.1 ± 72.4	1294.9	1300.0	N	11.1 ± 1.6	11.0	11.0
σ_v' (psf)	1078.7 ± 45.3	1232.2	1240.0	C_R	0.87	0.85	
a_{max} (g)	0.140 ± 0.042	0.14	0.140	C_S	1.00	1.00	
r_d	0.88 ± 0.052	0.96	0.980	C_B	1.00	1.00	
CSR	0.087 ± 0.027	0.092	0.095	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.40	1.32	1.2
MSF		1.34	1.19	$(N_1)_{60}$	13.5 ± 2.0	12.3	13.50
CSRN		0.065	0.08				



γ_m=100±3 pcf

γ_s=120±3 pcf



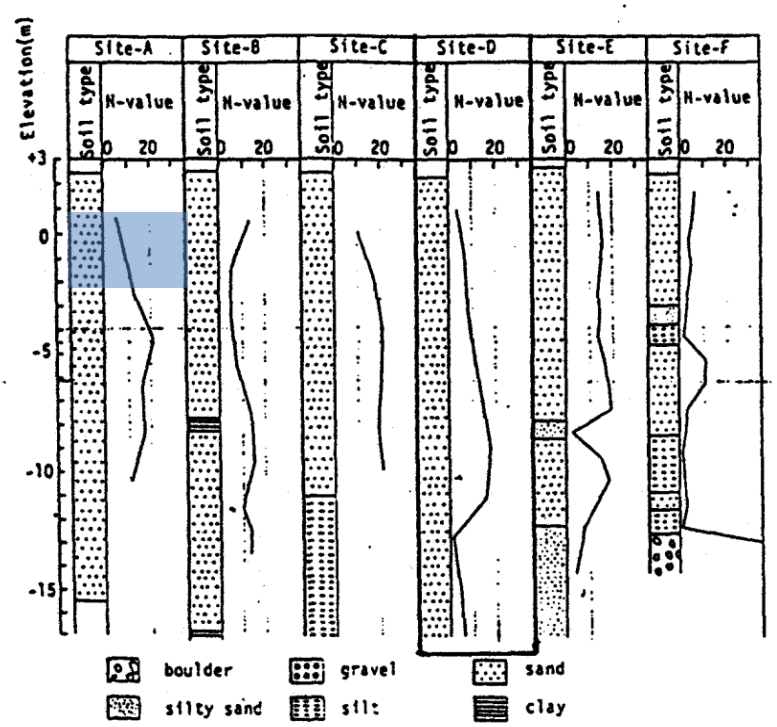
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	NA	4.0	2.0	0.73	5.9
2.0	6.6	NA	4.0	1.8	0.80	5.7
3.3	10.7	0.085	***11.0***	***1.4***	***0.9***	***13.5***
4.3	13.9	0.093	3.0	1.3	0.90	3.5
5.3	17.2	0.095	2.0	1.2	0.93	2.3
					Mean:	13.5
					St. Dev.	2.0

assigned

Case number:	46
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Nakajima-18
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Nakajima 18 site was claimed by Tokimatsu (1993) to be by Site A. SPT energy was estimated as 65 % by Seed et al. (84)

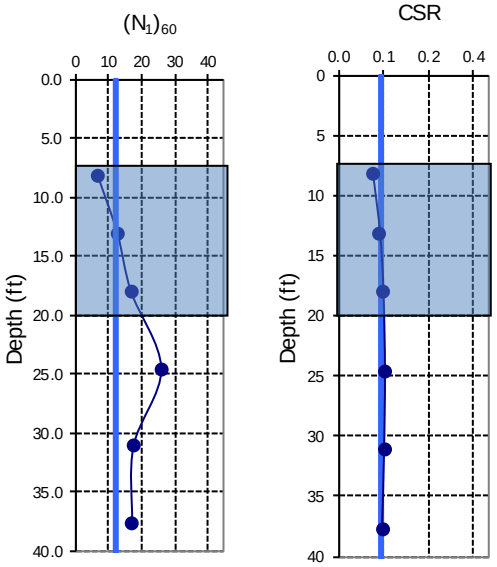
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.350 ± 0.050		0.35
Data Class	B			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	8.0 - 20.0	20.0	20.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1630.0 ± 251.8	2401.8	2400.0	N	9.6 ± 3.9	12.0	12.0
σ_v' (psf)	1255.6 ± 129.5	1649.9	1650.0	C_R	0.90	0.95	
a_{max} (g)	0.140 ± 0.042	0.14	0.140	C_S	1.00	1.00	
r_d	0.97 ± 0.061	0.91	0.960	C_B	1.00	1.00	
CSR	0.114 ± 0.036	0.120	0.125	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.5	6.5		C_N	1.30	1.13	1.1
MSF		1.34	1.14	$(N_1)_{60}$	12.2 ± 5.0	14.1	14.50
CSRN		0.088	0.11				



γ_m=110±3 pcf

γ_s=125±3 pcf

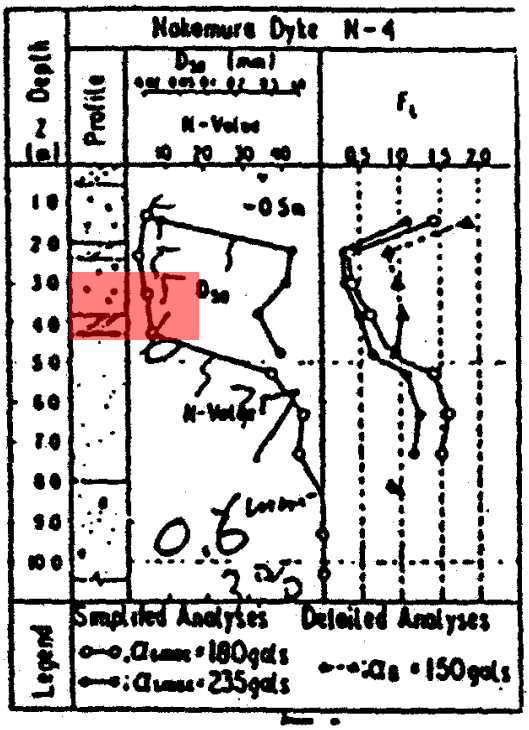


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	0.091	5.0	1.54	0.82	***6.9***
4.0	13.1	0.112	10.0	1.33	0.89	***12.9***
5.5	18.0	0.122	14.0	1.18	0.94	***16.9***
7.5	24.6	0.126	23.0	1.05	0.99	26.1
9.5	31.2	0.124	17.0	0.95	1.00	17.7
11.5	37.7	0.117	18.0	0.88	1.00	17.3
					Mean:	12.2
					St. Dev.	5.0

Case number:	47
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Nakamura-4
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils, cracking
Comments:	The site is located in the inside base towards the north end of an embankment along the left bank of the Natori River. Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

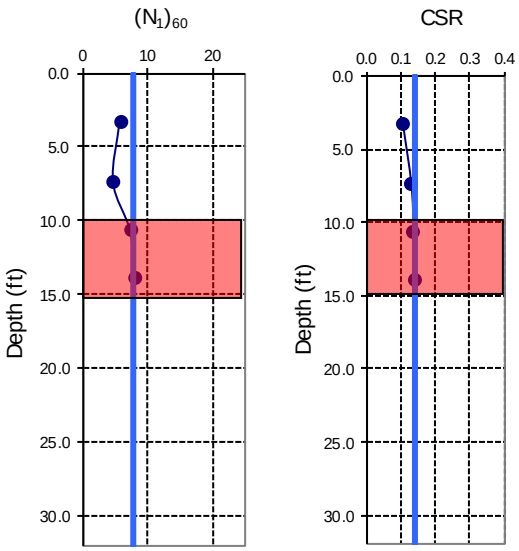
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.700 ± 0.050		0.7
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	9.8 - 16.4	9.2	11.0	% PI			
Depth to GWT (ft)	1.6 ± 0.3	1.6	2.0				
σ_v (psf)	1558.4 ± 135.8	1106.9	1320.0	N	5.6 ± 0.4	4.7	5.0
σ_v' (psf)	841.9 ± 73.7	626.6	760.0	C_R	0.89	0.85	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.98 ± 0.058	0.97	0.980	C_B	1.00	1.00	
CSR	0.142 ± 0.044	0.128	0.135	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.59	1.70	1.6
MSF		1.34	1.17	$(N_1)_{60}$	7.9 ± 0.6	6.9	8.00
CSRN		0.087	0.12				



γ_m=110±3 pcf

γ_s=120±3 pcf

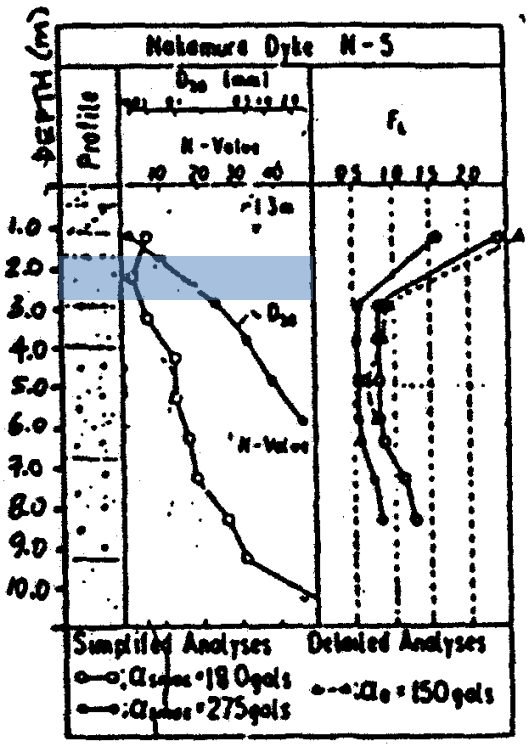


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	0.107	4	2.00	0.73	5.9	close to surf.
2.3	7.4	0.132	3	2.00	0.81	4.9	
3.3	10.7	0.139	5	1.74	0.86	***7.5***	
4.3	13.9	0.142	6	1.54	0.90	***8.3***	
					Mean:	7.9	
					St. Dev.	0.6	

Case number:	48
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Nakamura-5
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The site is located in the outside base at the north end of an embankment along the left bank of the Natori River. Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

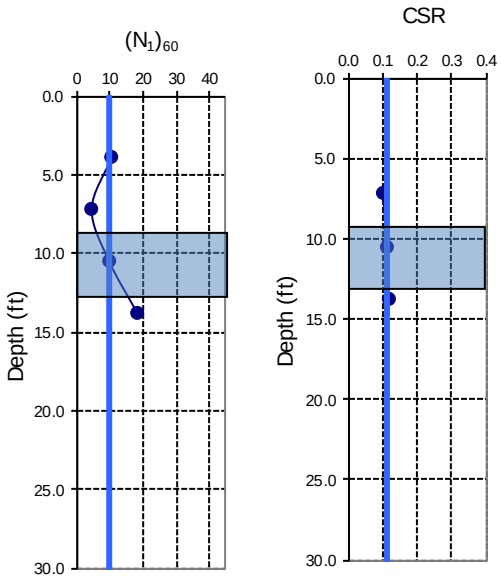
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.280 ± 0.050		0.28
Data Class	B			% Fines	4.0 ± 2.0	4.0	4
Critical Depth Range	9.0 - 13.1	11.2	11.0	% PI			
Depth to GWT (ft)	4.3 ± 0.3	4.3	4.0				
σ_v (psf)	1284.8 ± 86.0	1315.8	1320.0	N	7.1 ± 1.5	7.0	7.0
σ_v' (psf)	860.6 ± 48.9	877.2	880.0	C_R	0.86	0.85	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.97 ± 0.050	0.96	0.980	C_B	1.00	1.00	
CSR	0.113 ± 0.035	0.112	0.115	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.57	1.61	1.5
MSF		1.34	1.15	$(N_1)_{60}$	9.6 ± 2.0	9.6	10.00
CSRN		0.078	0.10				



γ_m=110±3 pcf

γ_s=120±3 pcf



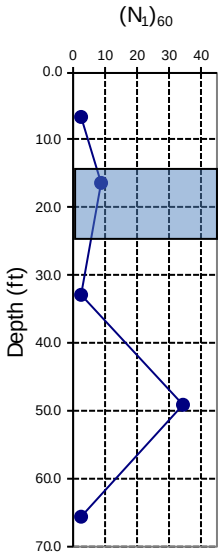
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.2	3.9	NA	7	2.00	0.75	10.5
2.2	7.2	0.099	3	1.82	0.81	4.4
3.2	10.5	0.111	7	1.60	0.86	***9.6***
4.2	13.8	0.118	14	1.44	0.89	18.1
					Mean:	9.6
					St. Dev.	2.0

assigned

Case number:	49
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Oiiri 1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995) A.S. Cakmak (1986) "Soil Liquefaction Studies in Japan", Vol. 5 No.1 Jan 1986, Soil Dynamics & Earthquake Engineering.
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Iwasaki et al (1978) paper did not mention Oiiri sites. However Cakmak et al gave a representative depth vs. N profile for Oiiri 1 SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 60 % by Seed et al. (84)

Summary of Data

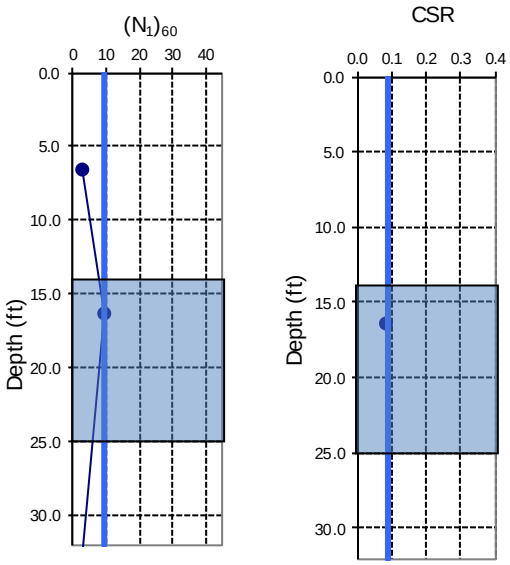
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.340 ± 0.050		0.34
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	14.0 - 25.0	21.0	21.0	% PI			
Depth to GWT (ft)	14.0 ± 0.3	14.1	14.0				
σ_v (psf)	2200.0 ± 224.6	2213.9	2220.0	N	9.2 ± 2.0	9.0	9.0
σ_v' (psf)	1856.8 ± 115.9	1775.3	1780.0	C_R	0.95	0.95	
a_{max} (g)	0.140 ± 0.042	0.14	0.140	C_S	1.00	1.00	
r_d	0.83 ± 0.081	0.90	0.950	C_B	1.00	1.00	
CSR	0.090 ± 0.029	0.102	0.110	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.07	1.10	1.1
MSF		1.34	1.16	$(N_1)_{60}$	9.3 ± 2.0	9.4	9.50
CSRN		0.075	0.10				



γ_m=110±3 pcf

γ_s=120±3 pcf

By Cakmak(1986)



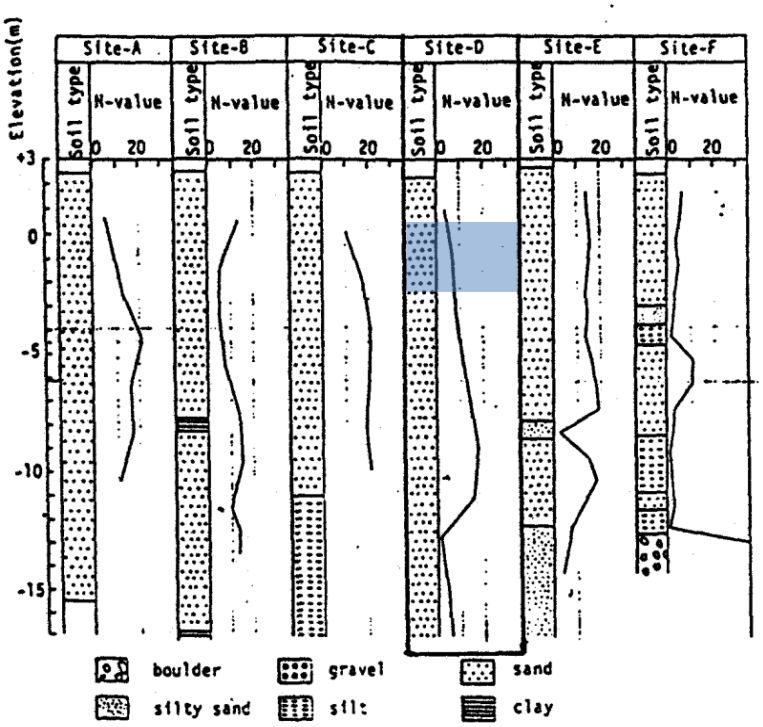
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	NA	2	1.71	0.80	2.7
5.0	16.4	0.086	9	1.12	0.92	***9.3***
10.0	32.8	0.086	3	0.90	1.00	2.7
15.0	49.2	0.078	45	0.77	1.00	34.7
20.0	65.6	0.079	4	0.68	1.00	2.7
					Mean:	9.3
					St. Dev.	2.0

assigned

Case number:	50
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Shiomi 6
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995) Tokimatsu (1983)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Shiomi 6 site was identified as Site D by Tokimatsu (1983). SPT energy was estimated as 65 % by Seed et al. (84)

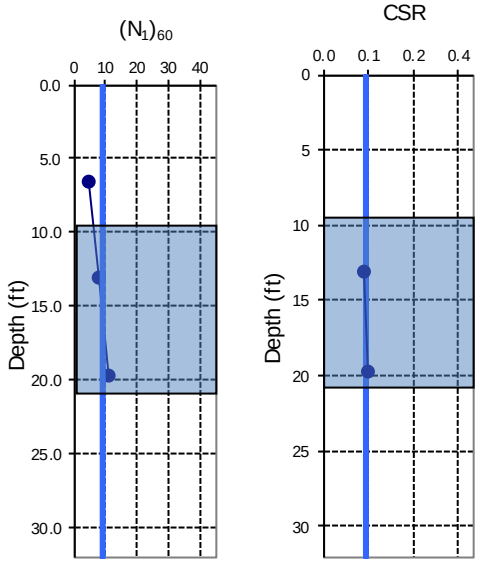
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.250 ± 0.050		0.25
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	9.8 - 19.7	13.1	13.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1691.7 ± 199.4	1566.4	1560.0	N	7.3 ± 1.7	6.0	6.0
σ_v' (psf)	1269.6 ± 100.8	1253.1	1250.0	C_R	0.90	0.85	
a_{max} (g)	0.140 ± 0.042	0.14	0.140	C_S	1.00	1.00	
r_d	0.94 ± 0.064	0.95	0.970	C_B	1.00	1.00	
CSR	0.114 ± 0.035	0.108	0.110	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.5	6.5		C_N	1.29	1.34	1.2
MSF		1.34	1.16	$(N_1)_{60}$	9.3 ± 2.2	7.5	8.00
CSRN		0.077	0.10				



γ_m=110±3 pcf

γ_s=120±3 pcf

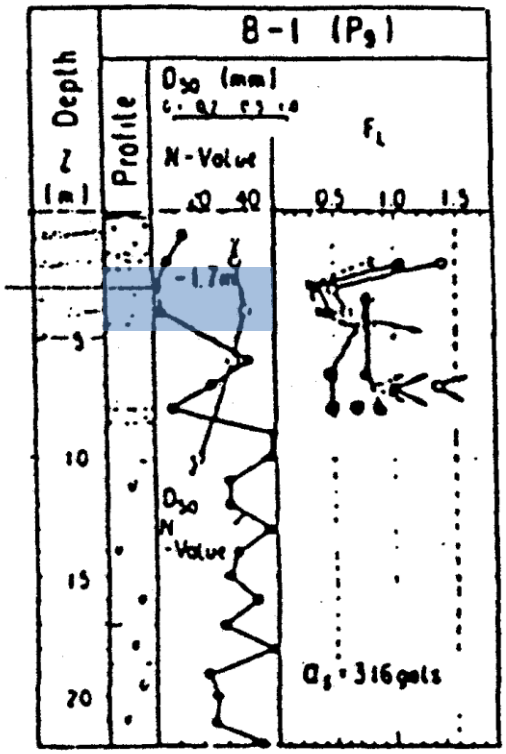


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	NA	3.0	1.71	0.80	4.5
4.0	13.1	0.110	6.0	1.34	0.89	***7.8***
6.0	19.7	0.121	9.0	1.17	0.95	***10.9***
					Mean:	9.3
					St. Dev.	2.2

Case number:	51
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Yuriage Bridge 1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

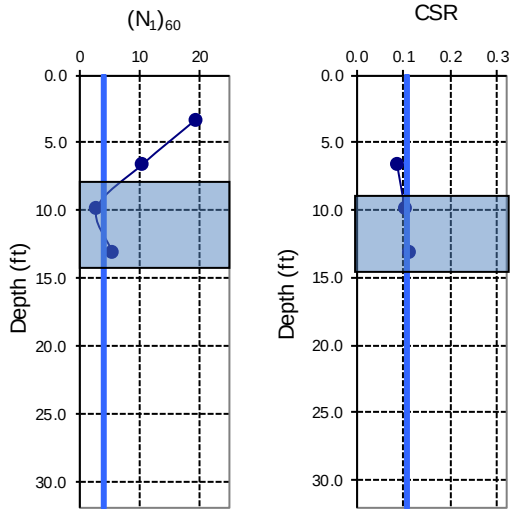
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.400 ± 0.050		0.4
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	9.8 - 13.1	14.1	14.0	% PI			
Depth to GWT (ft)	5.6 ± 0.3	5.9	6.0				
σ_v (psf)	1233.1 ± 67.9	1670.8	1680.0	N	3.0 ± 1.3	4.0	4.0
σ_v' (psf)	866.6 ± 40.4	1169.6	1180.0	C_R	0.87	0.95	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.98 ± 0.051	0.94	0.970	C_B	1.00	1.00	
CSR	0.109 ± 0.033	0.105	0.110	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.56	1.41	1.3
MSF		1.34	1.16	$(N_1)_{60}$	4.0 ± 1.8	5.4	5.00
CSRN		0.075	0.10				



γ_m=100±3 pcf

γ_s=115±3 pcf

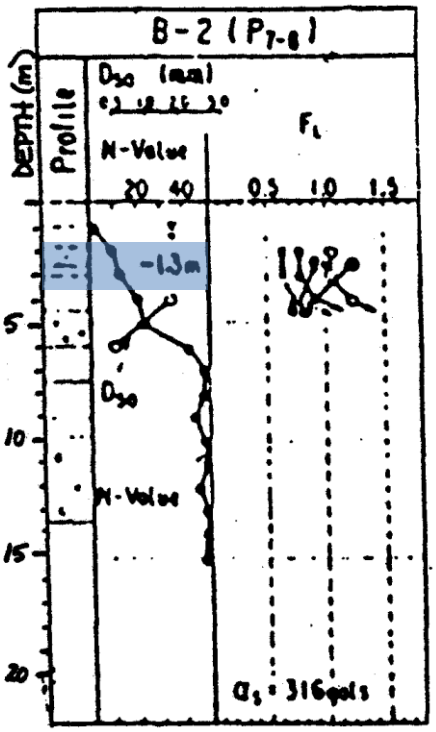


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	13	2.00	0.73	19.1	close to surf.
2.0	6.6	0.085	7	1.86	0.80	10.4	fines transition
3.0	9.8	0.103	2	1.64	0.85	***2.8***	
4.0	13.1	0.114	4	1.49	0.89	***5.3***	
					Mean:	4.0	
					St. Dev.	1.8	

Case number:	52
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Yuriage Bridge 2
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Gravel percent exceeds 20 % by weight. (Seed et al, 84) SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 67 % by Seed et al. (84)

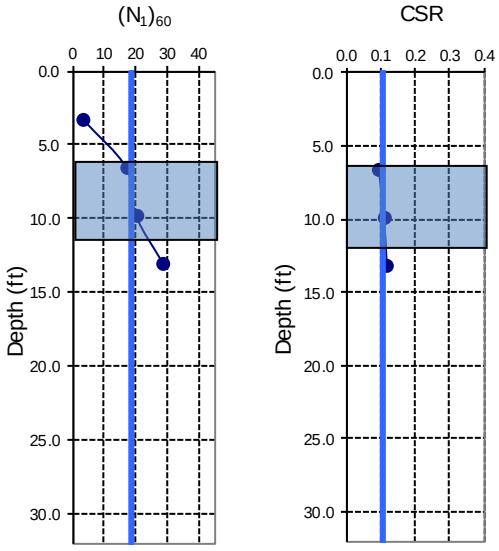
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	1.600 ± 0.050		1.6
Data Class	B			% Fines	7.0 ± 2.0	7.0	7
Critical Depth Range	6.0 - 10.0	8.2	11.0	% PI			
Depth to GWT (ft)	4.3 ± 0.3	3.9	4.0				
σ_v (psf)	893.4 ± 85.4	960.7	1320.0	N	11.4 ± 1.3	10.1	13.0
σ_v' (psf)	660.3 ± 46.4	710.1	880.0	C_R	0.82	0.85	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	1.00 ± 0.038	0.96	0.980	C_B	1.00	1.00	
CSR	0.105 ± 0.032	0.112	0.115	C_E	1.12	1.12	1.120
Equivalent Magnitude	6.5	6.5		C_N	1.79	1.68	1.5
MSF		1.34	1.15	$(N_1)_{60}$	18.7 ± 2.2	16.2	21.50
CSRN		0.076	0.10				



γ_m=100±3 pcf

γ_s=125±3 pcf

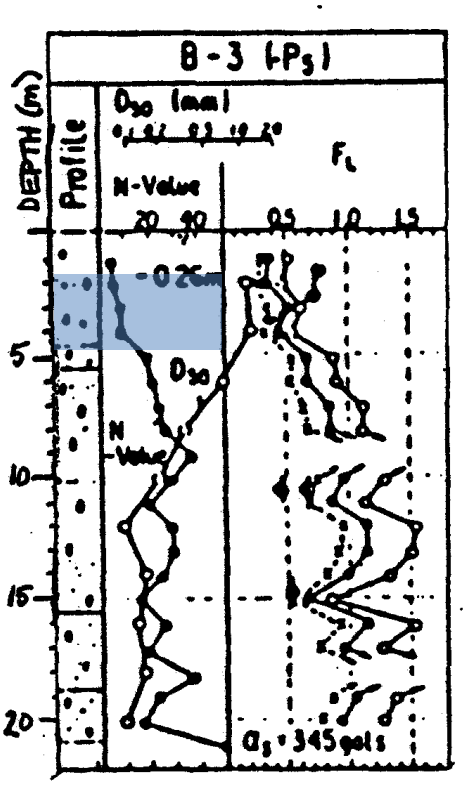


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	2	2.00	0.73	3.3	close to surf.
2.0	6.6	0.097	10	1.93	0.80	***17.2***	
3.0	9.8	0.112	13	1.65	0.85	***20.3***	
4.0	13.1	0.121	20	1.47	0.89	29.1	
					Mean:	18.7	
					St. Dev.	2.2	

Case number:	53
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Yuriage Bridge 3
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

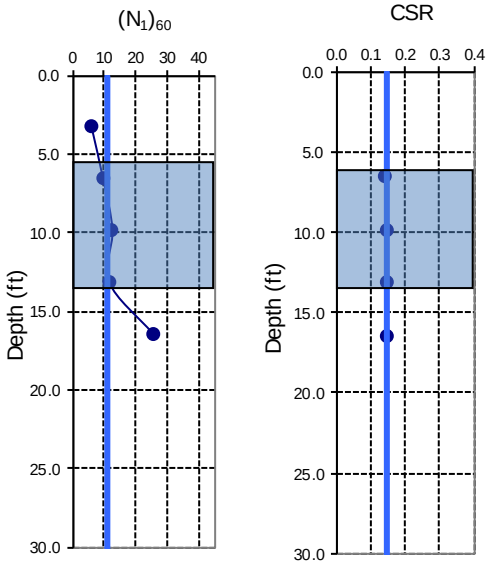
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	1.200 ± 0.050		1.2
Data Class	B			% Fines	12.0 ± 2.0	12.0	12
Critical Depth Range	6.6 - 13.1	14.1	14.0	% PI			
Depth to GWT (ft)	0.9 ± 0.3	1.0	1.0				
σ_v (psf)	1172.6 ± 134.0	1670.8	1680.0	N	7.1 ± 1.0	8.0	8.0
σ_v' (psf)	611.6 ± 70.3	877.2	870.0	C_R	0.85	0.95	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.98 ± 0.045	0.94	0.970	C_B	1.00	1.00	
CSR	0.147 ± 0.045	0.142	0.145	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.86	1.56	1.5
MSF		1.34	1.12	$(N_1)_{60}$	11.2 ± 1.5	11.8	12.00
CSRN		0.097	0.13				



γ_m=110±3 pcf

γ_s=120±3 pcf

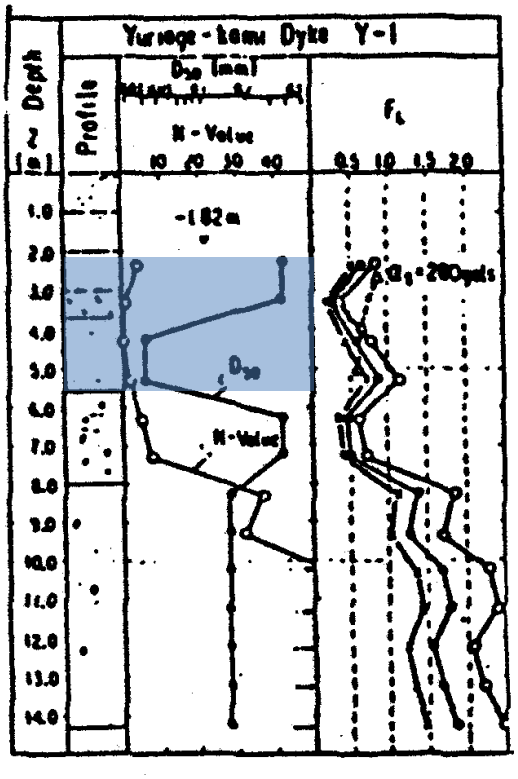


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	0.128	4	2.00	0.73	5.9	close to surf.
2.0	6.6	0.142	6	2.00	0.80	***9.6***	
3.0	9.8	0.147	8	1.86	0.85	***12.6***	
4.0	13.1	0.148	8	1.63	0.89	***11.5***	
5.0	16.4	0.148	19	1.46	0.92	25.6	
					Mean:	11.2	
					St. Dev.	1.5	

Case number:	54
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Yuriagekami-1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

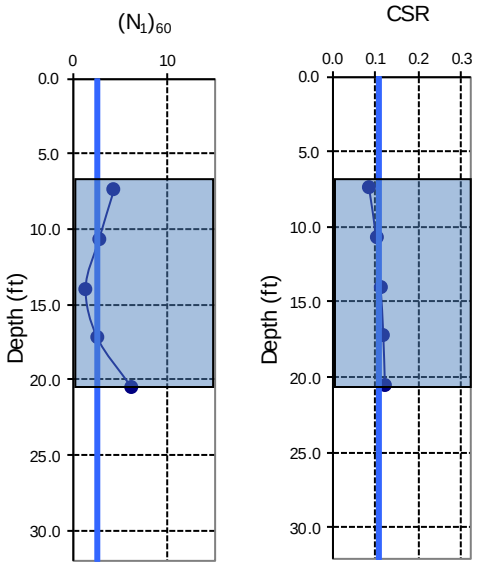
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.040 ± 0.050		0.04
Data Class	B			% Fines	60.0 ± 2.0	60.0	60
Critical Depth Range	5.9 - 18.0	18.0	18.0	% PI			
Depth to GWT (ft)	6.0 ± 0.3	5.9	6.0				
σ_v (psf)	1257.5 ± 224.0	2067.7	2060.0	N	2.0 ± 0.9	2.0	2.0
σ_v' (psf)	882.9 ± 100.8	1315.8	1310.0	C_R	0.87	0.95	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.97 ± 0.053	0.92	0.960	C_B	1.00	1.00	
CSR	0.108 ± 0.034	0.113	0.120	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.55	1.31	1.2
MSF		1.34	1.14	$(N_1)_{60}$	2.7 ± 1.2	2.5	2.50
CSRN		0.081	0.11				



γ_m=100±3 pcf

γ_s=110±3 pcf

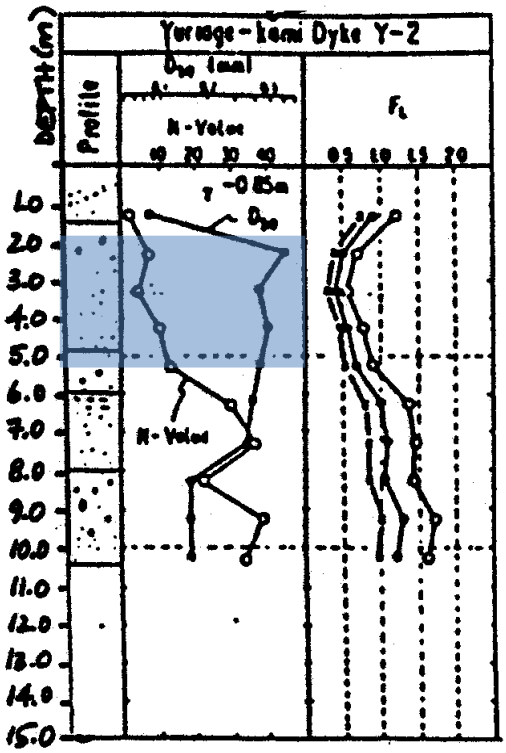


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.3	7.4	0.087	3	1.78	0.81	***4.3***
3.3	10.7	0.104	2	1.61	0.86	***2.8***
4.3	13.9	0.114	1	1.47	0.90	***1.3***
5.3	17.2	0.120	2	1.37	0.93	***2.5***
6.3	20.5	0.124	5	1.28	0.96	6.1
					Mean:	2.7
					St. Dev.	1.2

Case number:	55
Earthquake:	1978 Miyagiken-Oki
Magnitude:	6.5
Location:	Yuriagekami-2
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by the earthquakes of Magnitudes 7.3 and 7.7 SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

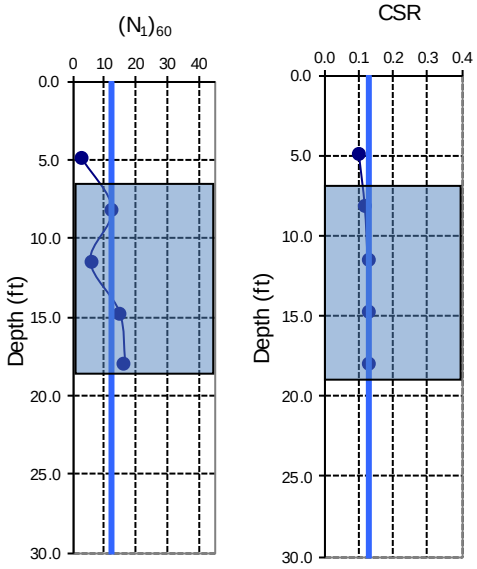
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.400 ± 0.050		0.4
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	6.6 - 18.0	14.1	14.0	% PI			
Depth to GWT (ft)	2.8 ± 0.3	3.0	3.0				
σ_v (psf)	1420.6 ± 231.7	1670.8	1680.0	N	8.8 ± 3.3	11.0	11.0
σ_v' (psf)	826.9 ± 114.9	981.6	990.0	C_R	0.88	0.95	
a_{max} (g)	0.120 ± 0.036	0.12	0.120	C_S	1.00	1.00	
r_d	0.96 ± 0.055	0.94	0.970	C_B	1.00	1.00	
CSR	0.129 ± 0.040	0.125	0.130	C_E	1.00	1.00	1.000
Equivalent Magnitude	6.5	6.5		C_N	1.60	1.45	1.4
MSF		1.34	1.13	$(N_1)_{60}$	12.3 ± 4.7	15.1	15.00
CSRN		0.086	0.12				



γ_m=100±3 pcf

γ_s=120±3 pcf

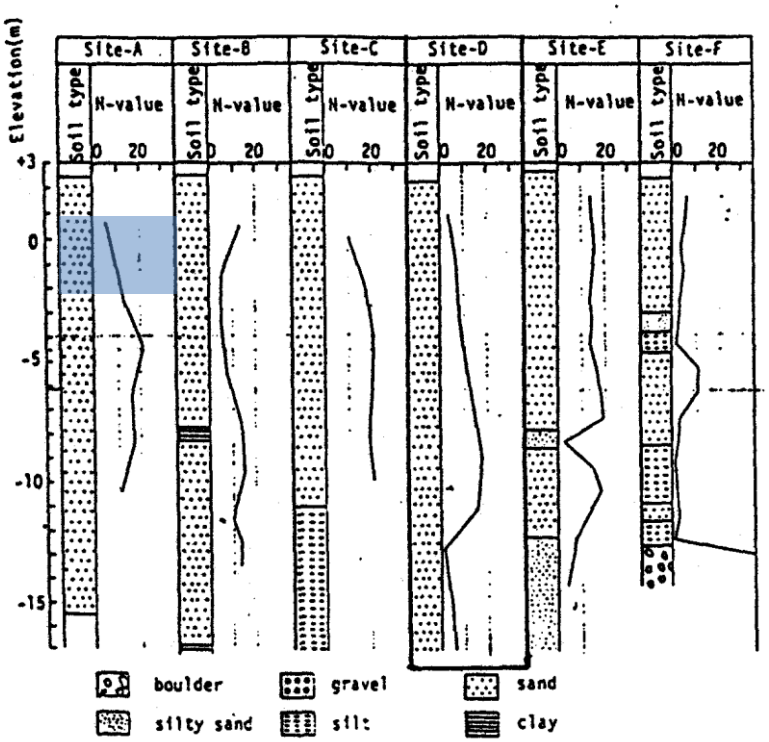


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.5	4.9	0.103	2	2.00	0.77	3.1	close to surf.
2.5	8.2	0.120	8	1.89	0.82	***12.5***	
3.5	11.5	0.128	4	1.65	0.87	***5.7***	
4.5	14.8	0.131	11	1.48	0.90	***14.7***	
5.5	18.0	0.132	13	1.35	0.94	***16.5***	
					Mean:	12.3	
					St. Dev.	4.7	

Case number:	56
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Nakajima-18
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Liquefied
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Nakajima 18 site was identified as Site A by Tokimatsu (1983). SPT energy is estimated as 65 % by Seed et al. (84)

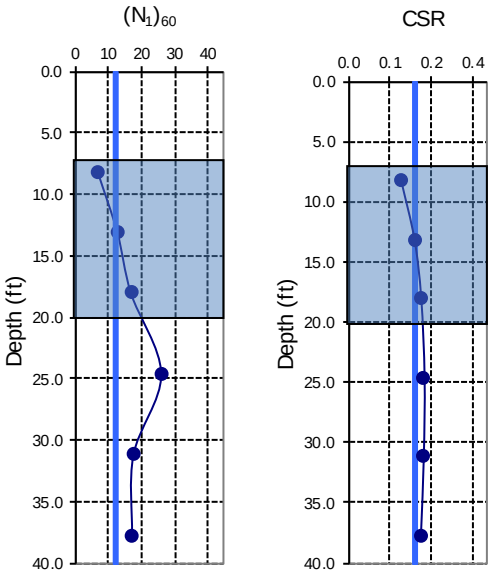
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.350 ± 0.050		0.35
Data Class	B			% Fines	3.0 ± 2.0	3.0	3
Critical Depth Range	8.0 - 20.0	20.0	20.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1630.0 ± 251.8	2401.8	2400.0	N	9.6 ± 3.9	12.0	12.0
σ_v' (psf)	1255.6 ± 129.5	1649.9	1650.0	C_R	0.90	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.97 ± 0.061	0.96	0.960	C_B	1.00	1.00	
CSR	0.197 ± 0.061	0.217	0.220	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.30	1.13	1.1
MSF		0.95	1.02	$(N_1)_{60}$	12.2 ± 5.0	14.1	14.50
CSRN		0.223	0.22				



γ_m=110±3 pcf

γ_s=125±3 pcf

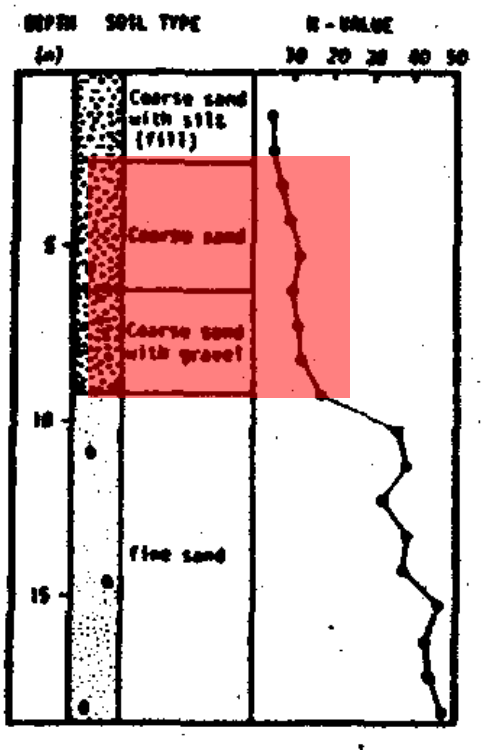


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	0.156	5.0	1.54	0.82	***6.9***
4.0	13.1	0.192	10.0	1.33	0.89	***12.9***
5.5	18.0	0.211	14.0	1.18	0.94	***16.9***
7.5	24.6	0.220	23.0	1.05	0.99	26.1
9.5	31.2	0.217	17.0	0.95	1.00	17.7
11.5	37.7	0.210	18.0	0.88	1.00	17.3
					Mean:	12.2
					St. Dev.	5.0

Case number:	57
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Arahama
References:	Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand volcanoes, houses tilted, ground settlement of ~20 cm. Uplifted buried sewage treatment tank.
Comments:	The site is located in Watari Town, district of Arahama. Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Arahama area is covered with very soft sandy deposits; about 9 m in thickness, N values generally less than 10. SPT values were taken before main earthquake (M=7.4) in 1974. SPT energy was estimated as 65 % by Seed et al. (84)

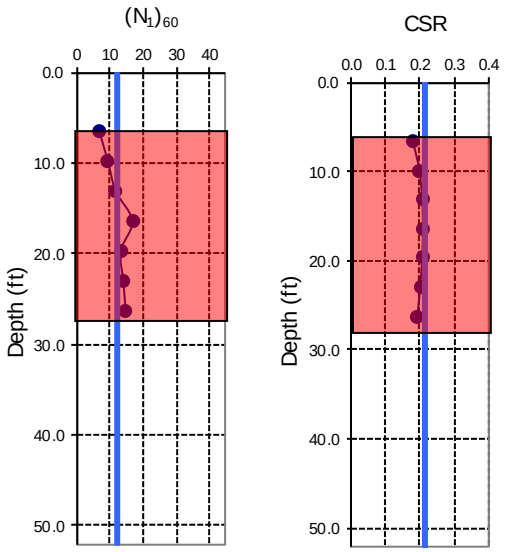
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.450 ± 0.050		0.6
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	6.6 - 26.2	21.0	21.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	1938.5 ± 395.9	2527.1	2520.0	N	8.9 ± 2.4	10.0	10.0
σ_v' (psf)	1102.1 ± 194.1	1399.3	1400.0	C_R	0.92	0.95	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.94 ± 0.070	0.95	0.950	C_B	1.00	1.00	
CSR	0.216 ± 0.067	0.223	0.220	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.39	1.23	1.2
MSF		0.95	1.00	$(N_1)_{60}$	12.4 ± 3.3	12.8	13.00
CSRN		0.225	0.22				



γ_m=110±3 pcf

γ_s=120±3 pcf

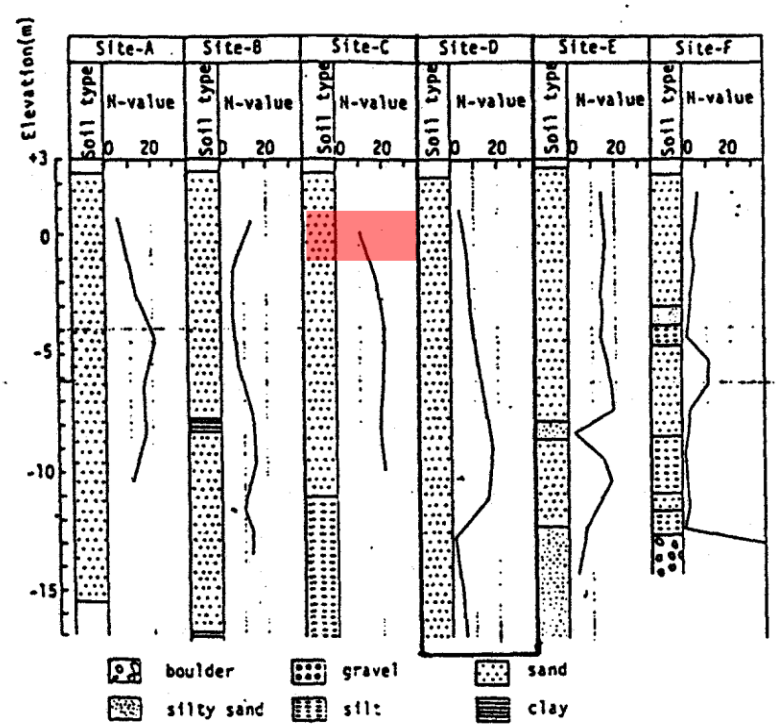


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	0.180	4	1.99	0.80	***6.9***
3.0	9.8	0.199	6	1.71	0.85	***9.5***
4.0	13.1	0.207	8	1.52	0.89	***11.8***
5.0	16.4	0.209	12	1.39	0.92	***16.7***
6.0	19.7	0.207	10	1.28	0.95	***13.3***
7.0	23.0	0.201	11	1.20	0.98	***14.0***
8.0	26.2	0.193	12	1.13	1.00	***14.8***
					Mean:	12.4
					St. Dev.	3.3

Case number:	58
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Hiyori-18
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Not specifically given, but Seed et al (1984) defined the site as "liquefied."
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Hiyori-18 site was claimed by Tokimatsu (1993) to be by Site C. SPT energy is estimated as 65 % by Seed et al. (84)

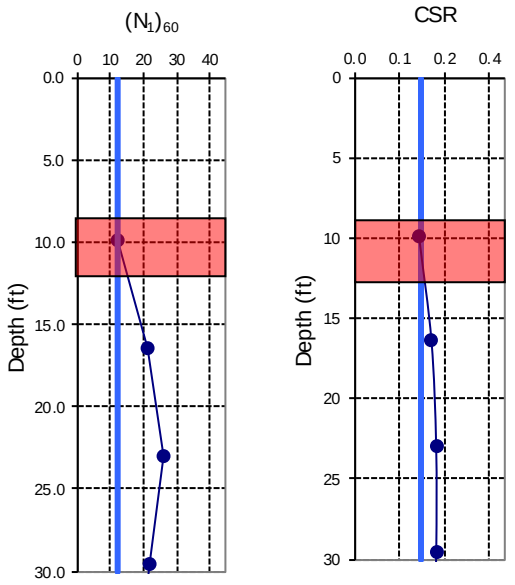
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.150 ± 0.050		0.15
Data Class	B			% Fines	20.0 ± 2.0	20.0	20
Critical Depth Range	8.2 - 13.1	17.1	17.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1199.5 ± 101.7	2046.8	2040.0	N	9.1 ± 1.5	9.0	9.0
σ_v' (psf)	1033.4 ± 55.8	1482.9	1480.0	C_R	0.86	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.98 ± 0.048	0.96	0.960	C_B	1.00	1.00	
CSR	0.177 ± 0.054	0.207	0.205	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.43	1.19	1.2
MSF		0.95	1.00	$(N_1)_{60}$	12.2 ± 2.0	11.1	11.50
CSRN		0.210	0.21				



γ_m=110±3 pcf

γ_s=120±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.0	9.8	0.171	9.0	1.46	0.85	***12.2***
5.0	16.4	0.207	17.0	1.25	0.92	21.3
7.0	23.0	0.220	22.0	1.10	0.98	25.9
9.0	29.5	0.220	20.0	1.00	1.00	21.8
11.0	36.1	0.214	18.0	0.92	1.00	18.1
					Mean:	12.2
					St. Dev.	2.0

assigned

Case number:	59
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Ishinomaki-2
References:	Ishihara et al. (1980) Fear et al. (1995)
Nature of Failure:	"Many sand boils showed up about 10 minutes after the main shock. The sand spouts reached a height of approximately 1 m. above the ground surface." Surface fissuring and cracks were also observed near the tank yard.
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The site is near the fishery port in the city of Ishinomaki. Loose sand deposit to a depth of 12 m has blow count values on the order of 5. The site has been improved by vibroflotation and compaction piles in 1975. Unimproved sand deposits in the vicinity were liquefied. The nearest strong ground motion on rock outcrop, a PGA of 289g, was recorded at Kaihoku bridge site which is located 5 km north of the oil tank site. Ishihara et al. Adopted a PGA value of 0.185g for his liquefaction analysis; considering the site response of soft soil deposits. Triaxial dynamic tests on frozen samples revealed that the FS in the depth range of 3-9 m is less than 1.(5% double amplitude axial strain in 20 cycles.) The site experienced an intensity of shaking of V on JMAMS. SPT energy is estimated as 65 % by Seed et al. (84)

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.150 ± 0.050		0.15
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	4.6 - 19.7	11.5	13.0	% PI			
Depth to GWT (ft)	4.6 ± 2.0	4.6	5.0				
σ_v (psf)	1410.8 ± 303.6	1378.4	1560.0	N	4.0 ± 0.3	3.7	4.0
σ_v' (psf)	939.9 ± 179.9	939.8	1060.0	C_R	0.88	0.85	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.93 ± 0.054	0.98	0.970	C_B	1.00	1.00	
CSR	0.181 ± 0.061	0.186	0.185	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.50	1.61	1.3
MSF		0.95	1.03	$(N_1)_{60}$	5.7 ± 0.5	5.5	6.00
CSRN		0.184	0.18				

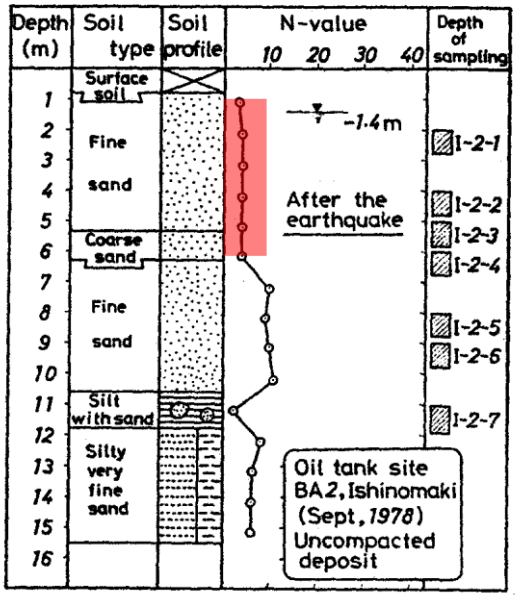


Fig. 14. Standard penetration resistance and depths of Osterberg sampling at the uncompacted site after the earthquake

γ_m=110±3 pcf

γ_s=120±3 pcf

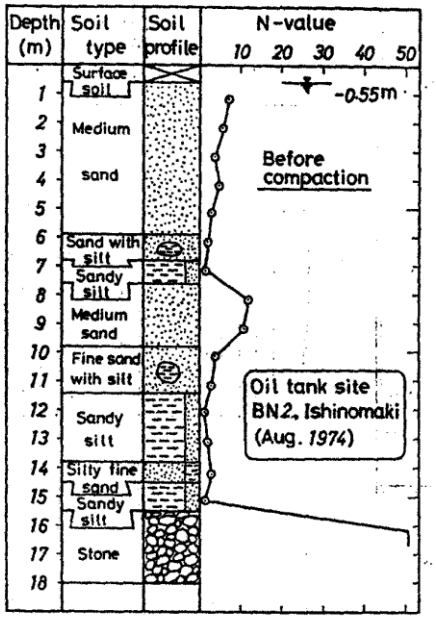
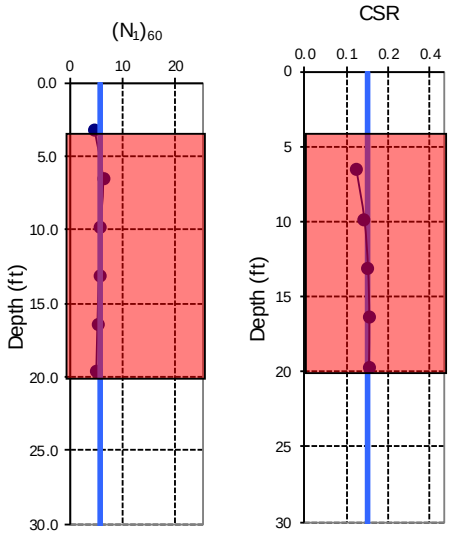


Fig. 11. A soil profile and standard penetration resistance before compaction

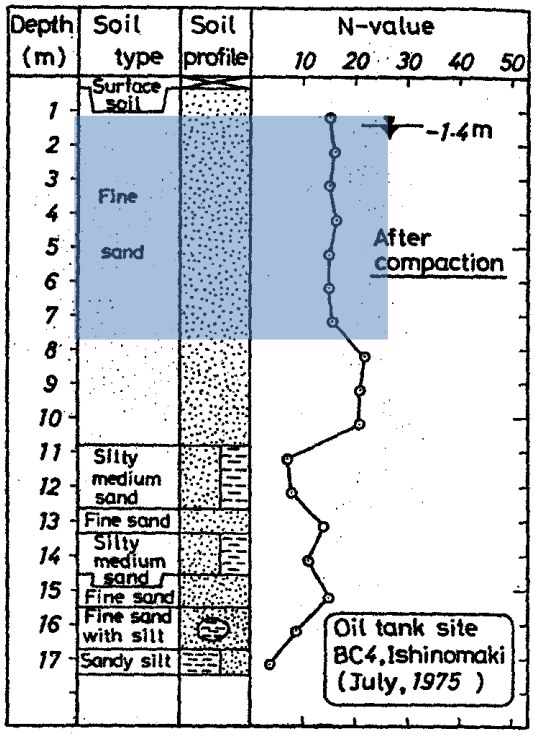


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	3.0	2.00	0.73	4.8	too close to surface
2.0	6.6	0.151	4.0	1.85	0.80	***6.4***	
3.0	9.8	0.173	4.0	1.62	0.85	***6.0***	
4.0	13.1	0.183	4.0	1.46	0.89	***5.6***	
5.0	16.4	0.186	4.0	1.34	0.92	***5.4***	
6.0	19.7	0.185	4.0	1.24	0.95	***5.2***	
					Mean:	5.7	
					St. Dev.	0.5	

Case number:	60
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Ishinomaki-4
References:	Ishihara et al. (1980) Fear et al. (1995)
Nature of Failure:	Nonliquefied
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The site is near the fishery port in the city of Ishinomaki. Loose sand deposit to a depth of 12 m has blow count values on the order of 5. The site has been improved by vibro-flotation and compaction piles in 1975. Unimproved sand deposits in the vicinity were liquefied. The nearest strong ground motion on rock outcrop, a PGA of 289g, was recorded at Kaihoku bridge site which is located 5 km north of the oil tank site. Ishihara et al. Adopted a PGA value of 0.185g for his liquefaction analysis; considering the site response of soft soil deposits. Triaxial dynamic tests on frozen samples revealed that the FS in the depth range of 3-9 m is less than 1.(5% double amplitude axial strain in 20 cycles.) 65 kN/m^3 (1316 psf) surcharge due to oil storage at the time of the earthquake SPT energy is estimated as 73 % by Seed et al. (84)

Summary of Data

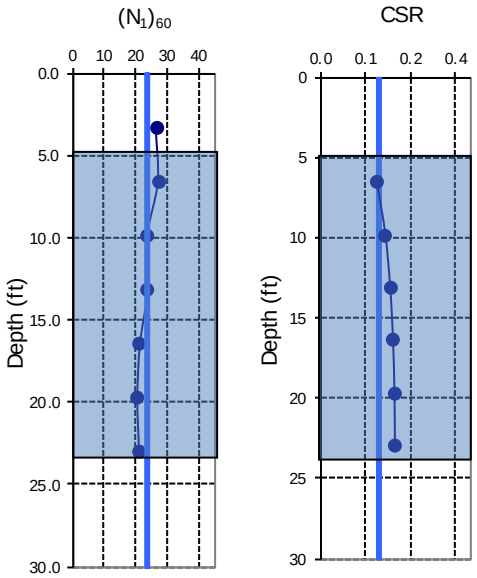
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.180 ± 0.050		0.18
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	4.6 - 23.0	14.8	20.0	% PI			
Depth to GWT (ft)	4.6 ± 0.3	4.6	5.0				
σ_v (psf)	3015.5 ± 384.0	1817.0	2400.0	N	23.4 ± 2.7	14.2	15.0
σ_v' (psf)	2442.2 ± 194.9	1190.5	1460.0	C_R	0.89	0.95	
a_{max} (g)	0.200 ± 0.060	0.20	0.200	C_S	1.00	1.00	
r_d	0.97 ± 0.060	0.97	0.960	C_B	1.00	1.00	
CSR	0.156 ± 0.048	0.188	0.205	C_E	1.21	1.21	1.210
Equivalent Magnitude	7.7	7.7		C_N	0.93	1.28	1.2
MSF		0.95	1.03	$(N_1)_{60}$	23.5 ± 2.7	20.9	21.00
CSRN		0.183	0.20				



γ_m=120±3 pcf

γ_s=125±3 pcf

Fig. 12. Standard penetration resistance after compaction

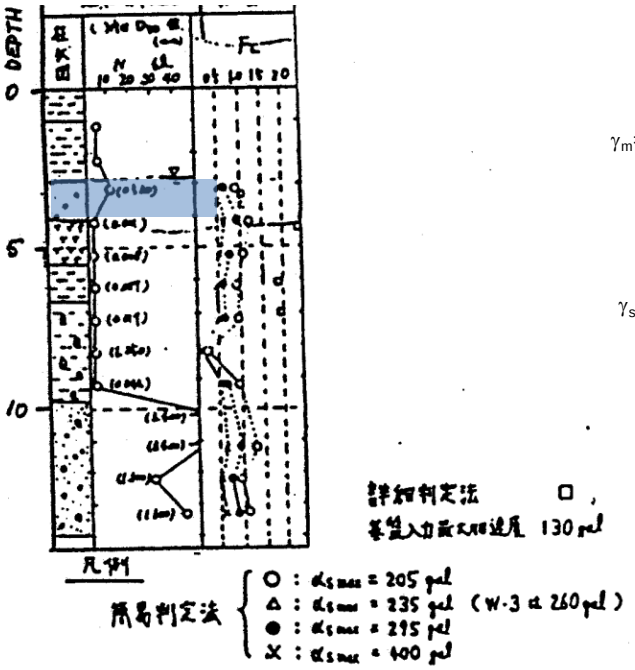


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	NA	15.0	2.00	0.73	***26.6***
2.0	6.6	0.152	16.0	1.77	0.80	***27.3***
3.0	9.8	0.176	15.0	1.55	0.85	***23.8***
4.0	13.1	0.189	16.0	1.40	0.89	***23.9***
5.0	16.4	0.197	15.0	1.28	0.92	***21.4***
6.0	19.7	0.201	15.0	1.19	0.95	***20.5***
7.0	23.0	0.202	16.0	1.12	0.98	***21.1***
					Mean:	23.5
					St. Dev.	2.7

Case number:	61
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Kitawabuchi 2
References:	Iwasaki et al. (: Fear et al. (1995) Tohno et al. (1981)
Nature of Failure:	sand boils fissures on the bank
Comments:	Kitawabuchi 2 is located on the inside of the right bank of the Eai River. The ground generally consists of sand type soil, over interbedded clay and hard clay, overlying sand. 0-10 m consists of thin sand layers and sandy silt layers with shells where N<10; below 10 m is sand Iwasaki et al. (1981) predicted the liquefied layer at depth of 3-4 m. Tohno et al (1981) : Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 liqeufaction occurred at the dike of Eai River; sand volcanoes were located on the old river bed near the right bank of the river and cracks were formed on the surface of the dike. The surface and the slope of the bank cracked at the site of the entrance to the old river bed; however the surface of the bed didn't subside The sand boil characteristics matched the characteristics of the sand layer at 3.5-5 m depth. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 60 % by Seed et al. (84)

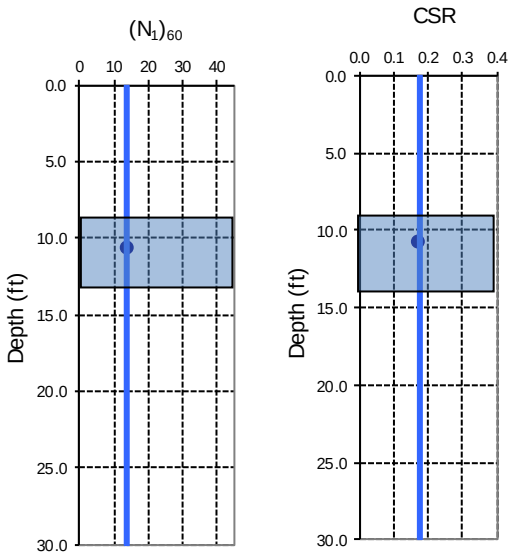
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.530 ± 0.050		0.53
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	9.8 - 13.1	11.2	11.0	% PI			
Depth to GWT (ft)	9.8 ± 0.3	10.2	10.0				
σ_v (psf)	1181.1 ± 72.4	1294.9	1300.0	N	11.1 ± 1.6	11.0	11.0
σ_v' (psf)	1078.7 ± 45.3	1232.2	1240.0	C_R	0.87	0.85	
a_{max} (g)	0.280 ± 0.084	0.28	0.280	C_S	1.00	1.00	
r_d	0.89 ± 0.052	0.98	0.980	C_B	1.00	1.00	
CSR	0.177 ± 0.054	0.187	0.185	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.40	1.32	1.2
MSF		0.95	1.00	$(N_1)_{60}$	13.5 ± 2.0	12.3	13.50
CSRN		0.187	0.19				



γ_m=100±3 pcf

γ_s=120±3 pcf



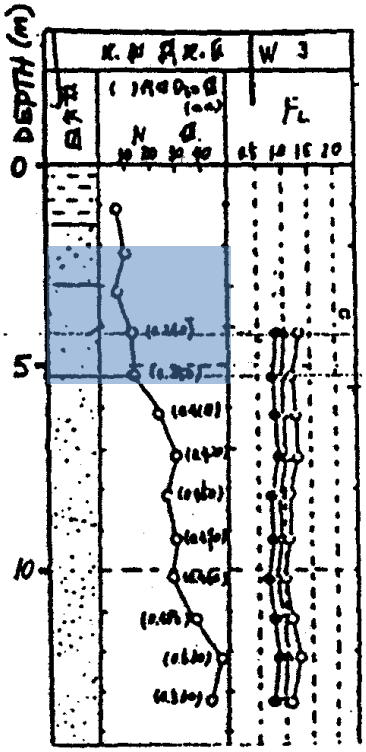
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	NA	4.0	2.0	0.7	5.9
2.0	6.6	NA	4.0	1.8	0.8	5.7
3.3	10.7	0.172	11.0	1.4	0.9	***13.5***
4.3	13.9	0.188	3.0	1.3	0.9	3.5
5.3	17.2	0.195	2.0	1.2	0.9	2.3
					Mean:	13.5
					St. Dev.	2.0

assigned

Case number:	62
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Kitawabuchi-3
References:	Iwasaki et al. (1981) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction.
Comments:	Kitawabuchi 3 is located on the inside of the right bank of the Eai River upstream from Kitawabuchi 2. The ground generally consists of sand type soil, over interbedded clay and Sand layer at depths of 3-10+ m. Tohno et al (1981) : Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Liqeufaction occurred at the dike of Eai River; sand volcanoes were located on the old river bed near the right bank of the river and cracks were formed on the surface of the dike. The surface and the slope of the bank cracked at the site of the entrance to the old river bed; however the surface of the bed didn't subside The sand boil characteristics matched the characteristics of the sand layer at 3.5-5 m depth. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 73 % by Seed et al. (84)

Summary of Data

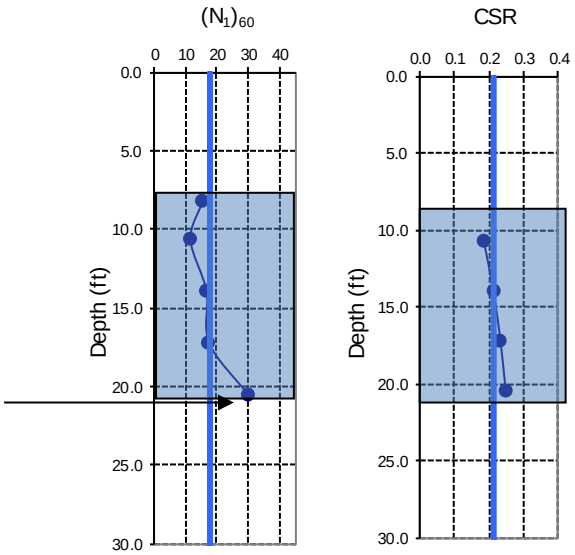
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.410 ± 0.050		0.41
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	10.0 - 18.0	15.7	20.0	% PI			
Depth to GWT (ft)	10.0 ± 3.0	10.2	10.0				
σ_v (psf)	1582.7 ± 166.8	1879.7	2400.0	N	13.2 ± 5.1	13.2	23.0
σ_v' (psf)	1331.7 ± 178.1	1524.6	1780.0	C_R	0.90	0.95	
a_{max} (g)	0.280 ± 0.084	0.28	0.320	C_S	1.00	1.00	
r_d	0.99 ± 0.061	0.97	0.960	C_B	1.00	1.00	
CSR	0.214 ± 0.073	0.216	0.235	C_E	1.21	1.21	1.210
Equivalent Magnitude	7.7	7.7		C_N	1.26	1.16	1.1
MSF		0.95	1.02	$(N_1)_{60}$	18.0 ± 6.9	17.6	29.00
CSRN		0.220	0.23				



γ_m=110±3 pcf

γ_s=120±3 pcf

Seed et al. (1984)



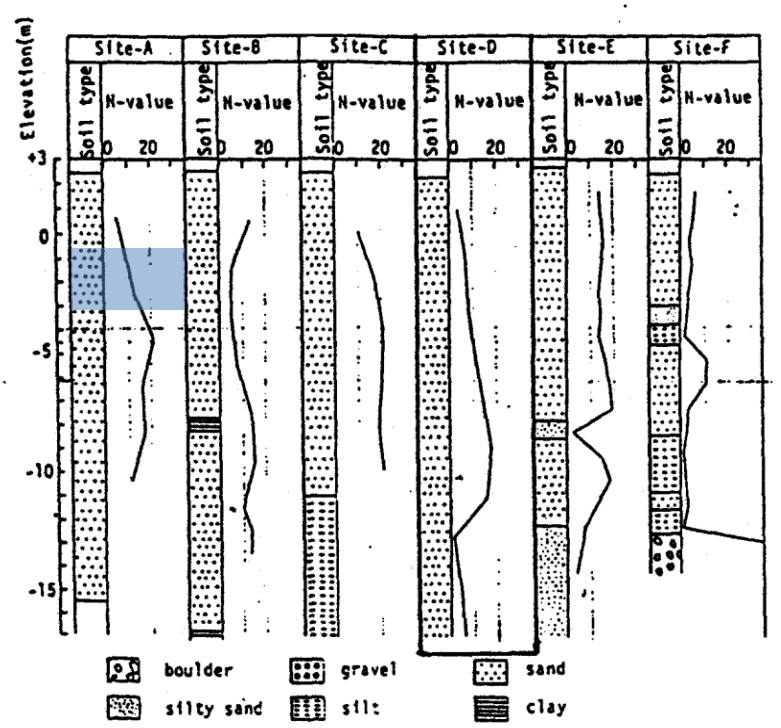
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	NA	10	1.53	0.82	***15.2***
3.3	10.7	0.188	8	1.36	0.86	***11.3***
4.3	13.9	0.214	12	1.26	0.90	***16.4***
5.3	17.2	0.233	13	1.18	0.93	***17.2***
6.3	20.5	0.247	23	1.11	0.96	***29.6***
					Mean:	18.0
					St. Dev.	6.9

Water table info is not accurate.

Case number:	63
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Nakajima-2
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction.
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Nakajima 18 site was claimed by Tokimatsu (1993) to be by Site A. SPT energy was estimated as 65 % by Seed et al. (84)

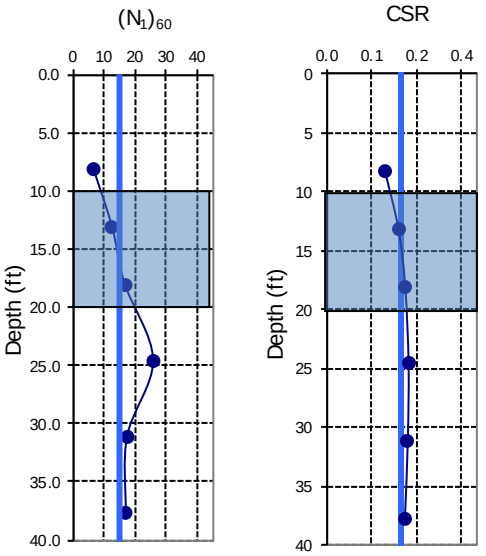
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.120 ± 0.050		0.12
Data Class	B			% Fines	26.0 ± 2.0	26.0	26
Critical Depth Range	10.0 - 20.0	15.1	15.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1755.0 ± 210.8	1796.1	1800.0	N	11.9 ± 2.3	10.0	10.0
σ_v' (psf)	1318.2 ± 110.0	1357.6	1360.0	C_R	0.91	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.97 ± 0.065	0.97	0.970	C_B	1.00	1.00	
CSR	0.201 ± 0.062	0.200	0.200	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.27	1.23	1.2
MSF		0.95	1.03	$(N_1)_{60}$	14.9 ± 2.9	12.7	13.00
CSRN		0.200	0.195				



γ_m=110±3 pcf

γ_s=125±3 pcf

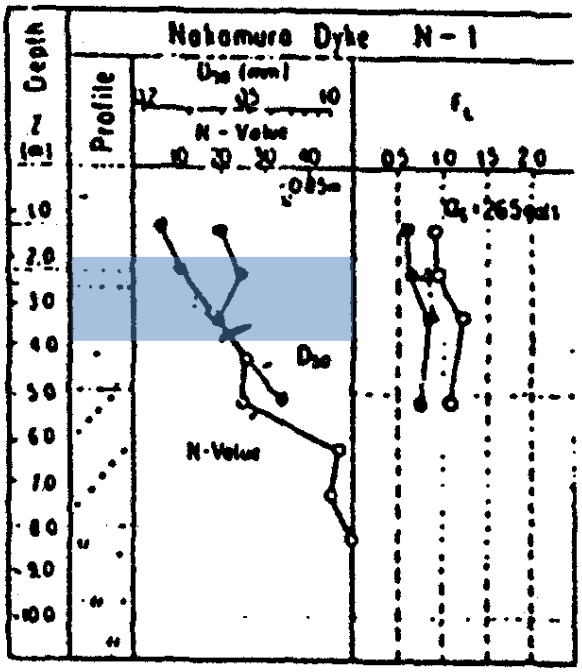


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	0.156	5.0	1.54	0.82	6.9
4.0	13.1	0.192	10.0	1.33	0.89	***12.9***
5.5	18.0	0.211	14.0	1.18	0.94	***16.9***
7.5	24.6	0.220	23.0	1.05	0.99	26.1
9.5	31.2	0.217	17.0	0.95	1.00	17.7
11.5	37.7	0.210	18.0	0.88	1.00	17.3
					Mean:	14.9
					St. Dev.	2.9

Case number:	64
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Nakamura-1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction.
Comments:	The site is located in the inside base towards the south end of an embankment along the left bank of the Natori River. Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 PGA is estimated as 0.18 g in Iwasaki (1978). Based on the results of cyclic triaxial test, the FS against liquefaction is calculated as less than 1 in the sand layer between the depths of 2-7 m. From Tohno et al. (1981) The river embankment was partially collapsed and cracked in both sides at Nakamura located ~3 km upstream from the mouth of Natori River. Sand boils were also observed after the February, 1978 (M=6.4) earthquake. The textural parameters of the liquefied soil revealed that the possibly liquefied layer is sandy deposits at 1.4-3 m. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 67 % by Seed et al. (84)

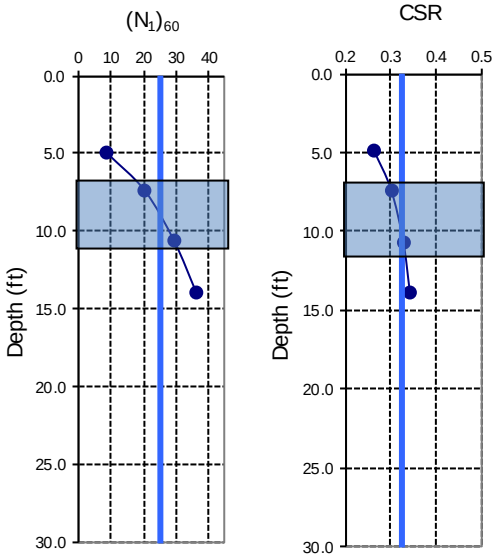
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.280 ± 0.050		0.28
Data Class	B			% Fines	4.0 ± 2.0	4.0	4
Critical Depth Range	6.6 - 13.1	11.2	11.0	% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0	3.0				
σ_v (psf)	1186.0 ± 138.6	1315.8	1320.0	N	15.8 ± 4.0	19.0	19.0
σ_v' (psf)	756.1 ± 73.4	814.5	820.0	C_R	0.85	0.85	
a_{max} (g)	0.320 ± 0.096	0.32	0.320	C_S	1.00	1.00	
r_d	0.99 ± 0.045	0.98	0.980	C_B	1.00	1.00	
CSR	0.324 ± 0.099	0.329	0.330	C_E	1.12	1.12	1.120
Equivalent Magnitude	7.7	7.7		C_N	1.67	1.45	1.5
MSF		0.95	1.02	$(N_1)_{60}$	24.9 ± 6.4	26.2	31.50
CSRN		0.315	0.33				



γ_m=110±3 pcf

γ_s=125±3 pcf

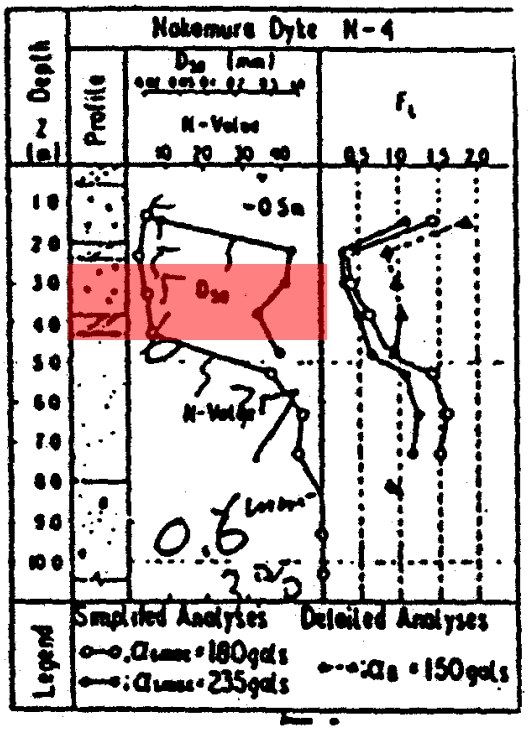


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.5	4.9	0.264	5	2.00	0.77	8.6
2.3	7.4	0.302	12	1.87	0.81	***20.4***
3.3	10.7	0.329	19	1.62	0.86	***29.4***
4.3	13.9	0.344	25	1.45	0.90	36.2
					Mean:	24.9
					St. Dev.	6.4

Case number:	65
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Nakamura-4
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils, cracking.
Comments:	The site is located in the inside base towards the north end of an embankment along the left bank of Natori River Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 PGA is estimated as 0.18 g in the paper. Based on the results of cyclic triaxial test, the FS against liquefaction is calculated as less than 1 in the sand layer between the depths of 2-7 m. From Tohno et al. (1981) The river embankment was partially collapsed and cracked in both sides at Nakamura located ~3 km upstream from the mouth of Natori River. Sand boils were also observed after the February, 1978 (M=6.4) earthquake. The textural parameters of the liquefied soil revealed that the possibly liquefied layer is sandy deposits at 1.4-3 m. SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

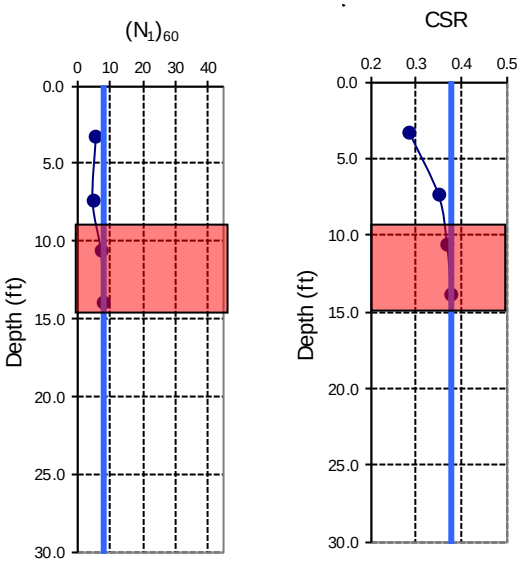
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.700 ± 0.050		0.7
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	9.8 - 16.4	9.2	11.0	% PI			
Depth to GWT (ft)	1.6 ± 0.3	1.6	2.0				
σ_v (psf)	1558.4 ± 135.8	1106.9	1320.0	N	5.6 ± 0.4	4.7	5.0
σ_v' (psf)	841.9 ± 73.7	626.6	760.0	C_R	0.89	0.85	
a_{max} (g)	0.320 ± 0.096	0.32	0.320	C_S	1.00	1.00	
r_d	0.98 ± 0.058	0.99	0.980	C_B	1.00	1.00	
CSR	0.379 ± 0.116	0.346	0.355	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.59	1.70	1.6
MSF		0.95	1.01	$(N_1)_{60}$	7.9 ± 0.6	6.9	8.00
CSRN		0.332	0.35				



γ_m=110±3 pcf

γ_s=120±3 pcf

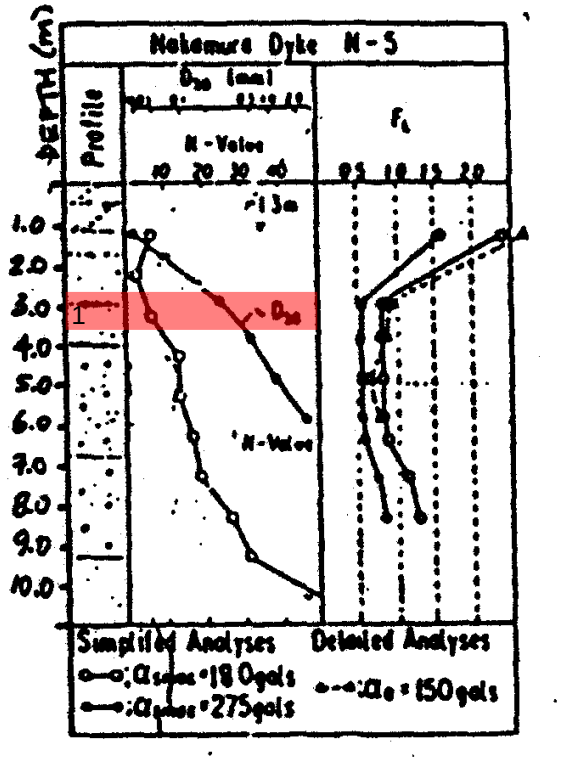


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.0	3.3	0.285	4	2.00	0.73	5.9
2.3	7.4	0.351	3	2.00	0.81	4.9
3.3	10.7	0.371	5	1.74	0.86	***7.5***
4.3	13.9	0.381	6	1.54	0.90	***8.3***
					Mean:	7.9
					St. Dev.	0.6

Case number:	66
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Nakamura-5
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils
Comments:	The site is located in the outside base at the north end of an embankment along the left bank of the Natori River. PGA is estimated as 0.18 g in the paper. Based on the results of cyclic triaxial test, the FS against liquefaction is calculated as less than 1 in the sand layer between the depths of 2-7 m. From Tohno et al. (1981): The river embankment was partially collapsed and cracked in both sides at Nakamura located ~3 km upstream from the mouth of Natori River. Sand boils were also observed after the February, 1978 (M=6.4) earthquake. The textural parameters of the liquefied soil revealed that the possibly liquefied layer is sandy deposits at 1.4-3 m. SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

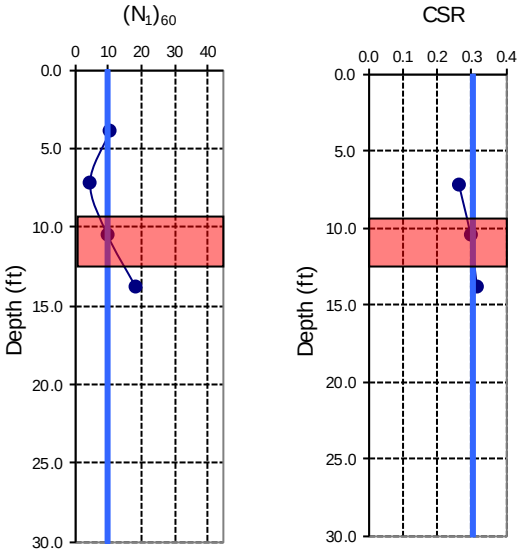
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.280 ± 0.050		0.28
Data Class	B			% Fines	4.0 ± 2.0	4.0	4
Critical Depth Range	9.0 - 13.1	11.2	11.0	% PI			
Depth to GWT (ft)	4.3 ± 0.3	4.3	4.0				
σ_v (psf)	1284.8 ± 86.0	1315.8	1320.0	N	7.1 ± 1.5	7.0	7.0
σ_v' (psf)	860.6 ± 48.9	877.2	880.0	C_R	0.86	0.85	
a_{max} (g)	0.320 ± 0.096	0.32	0.320	C_S	1.00	1.00	
r_d	0.97 ± 0.050	0.31	0.980	C_B	1.00	1.00	
CSR	0.302 ± 0.092	0.306	0.305	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.57	1.61	1.5
MSF		0.95	1.02	$(N_1)_{60}$	9.6 ± 2.0	9.6	10.00
CSRN		0.299	0.30				



γ_m=110±3 pcf

γ_s=120±3 pcf



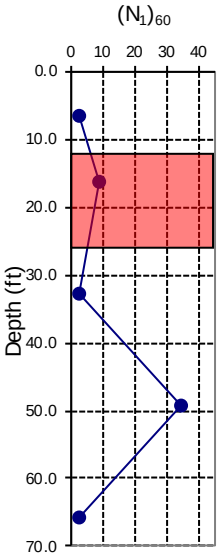
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.2	3.9	NA	7	2.00	0.75	10.5
2.2	7.2	0.264	3	1.82	0.81	4.4
3.2	10.5	0.298	7	1.60	0.86	***9.6***
4.2	13.8	0.316	14	1.44	0.89	18.1
					Mean:	9.6
					St. Dev.	2.0

assigned

Case number:	67
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Oiiri 1
References:	Iwasaki et al. (1978) No mention of Oiiri-1? Fear et al. (1995) Tohno et al. (1981) A.S. Cakmak (1986) "Soil Liquefaction Studies in Japan", Vol. 5 No.1 Jan 1986, Soil Dynamics & Earthquake Engineering.
Nature of Failure:	Not given but according to Seed et al (1984) "liquefied."
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 Iwasaki et al (1978) paper did not mention Oiiri sites. However Cakmak et al. gave a representative depth vs. N profile for Oiiri 1. Tohno et al (1981) : Liquefaction was observed at the dike of Old Kitakami River. Sand volcanoes were found on both sides of the base of the dike and in the vicinity of it. The boil material has the same characteristics of samd layer at depth of 4.5-7m SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 60 % by Seed et al. (84)

Summary of Data

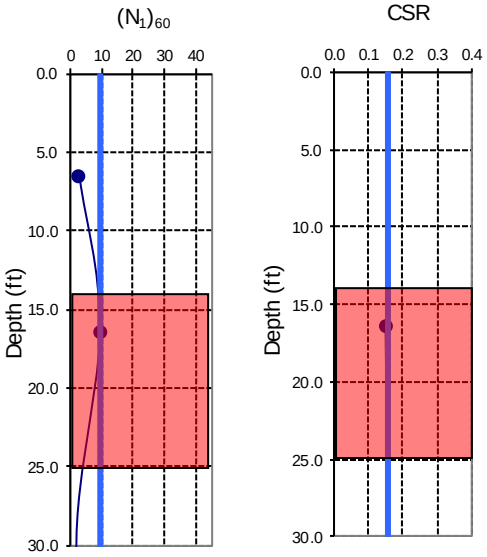
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.340 ± 0.050		0.34
Data Class	B			% Fines	5.0 ± 2.0	5.0	5
Critical Depth Range	14.0 - 25.0	21.0	21.0	% PI			
Depth to GWT (ft)	14.0 ± 0.3	14.1	14.0				
σ_v (psf)	2200.0 ± 224.6	2213.9	2220.0	N	9.2 ± 2.0	9.0	9.0
σ_v' (psf)	1856.8 ± 115.9	1775.3	1780.0	C_R	0.95	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.85 ± 0.081	0.95	0.950	C_B	1.00	1.00	
CSR	0.157 ± 0.050	0.185	0.185	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.07	1.10	1.1
MSF		0.95	1.03	$(N_1)_{60}$	9.3 ± 2.0	9.4	9.50
CSRN		0.192	0.18				



γ_m=110±3 pcf

γ_s=120±3 pcf

By Cakmak(1986)



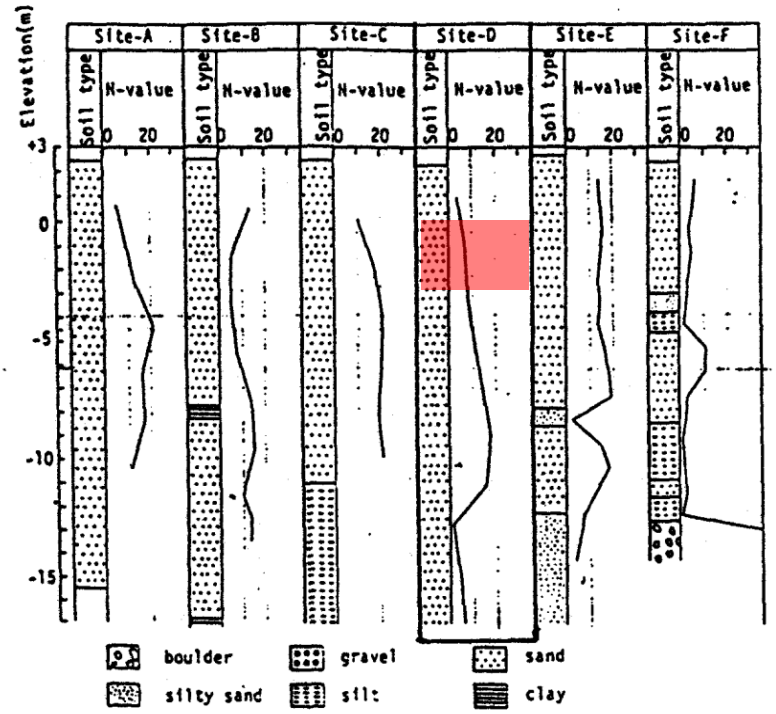
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	NA	2	1.71	0.80	2.7
5.0	16.4	0.151	9	1.12	0.92	***9.3***
10.0	32.8	0.156	3	0.90	1.00	2.7
15.0	49.2	0.147	45	0.77	1.00	34.7
20.0	65.6	0.149	4	0.68	1.00	2.7
					Mean:	9.3
					St. Dev.	2.0

assigned

Case number:	68
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Shiomi 6
References:	Tsuchida et al (1979, 1980) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Not specifically given but identified by Seed et al. (84) as "liquefied."
Comments:	Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. Only the Nakamura site liquefied during this earthquake. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The original Tsuchida (1979) paper (in Japanese) could not be accessed. However the same author's paper of 1980 in English was used to obtain borelogs. Shiomi 6 site was claimed by Tokimatsu (1993) to be by Site D. SPT energy was estimated as 65 % by Seed et al. (84)

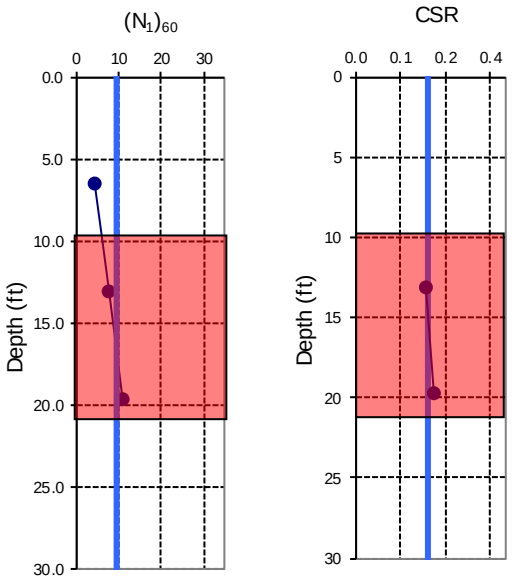
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.250 ± 0.050		0.25
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	9.8 - 19.7	13.1	13.0	% PI			
Depth to GWT (ft)	8.0 ± 0.3	7.9	8.0				
σ_v (psf)	1691.7 ± 199.4	1566.4	1560.0	N	7.3 ± 1.7	6.0	6.0
σ_v' (psf)	1269.6 ± 100.8	1253.1	1250.0	C_R	0.90	0.85	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.95 ± 0.064	0.98	0.970	C_B	1.00	1.00	
CSR	0.197 ± 0.061	0.190	0.190	C_E	1.09	1.09	1.090
Equivalent Magnitude	7.7	7.7		C_N	1.29	1.34	1.2
MSF		0.95	1.03	$(N_1)_{60}$	9.3 ± 2.2	7.5	8.00
CSRN		0.192	0.19				



γ_m=110±3 pcf

γ_s=120±3 pcf

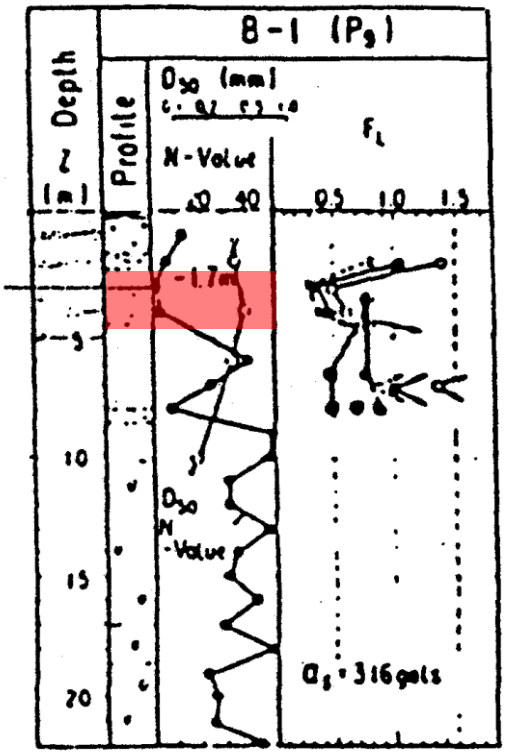


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	NA	3.0	1.71	0.80	4.5
4.0	13.1	0.190	6.0	1.34	0.89	***7.8***
6.0	19.7	0.210	9.0	1.17	0.95	***10.9***
					Mean:	9.3
					St. Dev.	2.2

Case number:	69
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriage Bridge 1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils out of surface cracks. Damage to Yuriagegami Bridge, cracked piers, etc.
Comments:	Yuriage Br. 1 is located near pier 9 of the Yuriagegami Bridge, which crosses Natori River. Pier 9 is the closest to the right bank of the river. PGA estimations of 0.24 and 0.3 g were used by Iwasaki et al (1978). Based on cyclic triaxial test results, the sand layer at depth range of 2-6 m is predicted to be liquefied. Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The dry river bed was cracked parallel to the river near Yuriage-Ohashi Bridge, and liquefied sands seeped out from the cracks. Some cracks were found near the piers of the Yuriage-Ohashi Birdge and the ground around the piers settled. The boiled soil was a poorly graded coarse sand. Iwasaki and Tokida (1980) identified the liquefied layer as at depths of 2-5 m. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 60 % by Seed et al. (84)

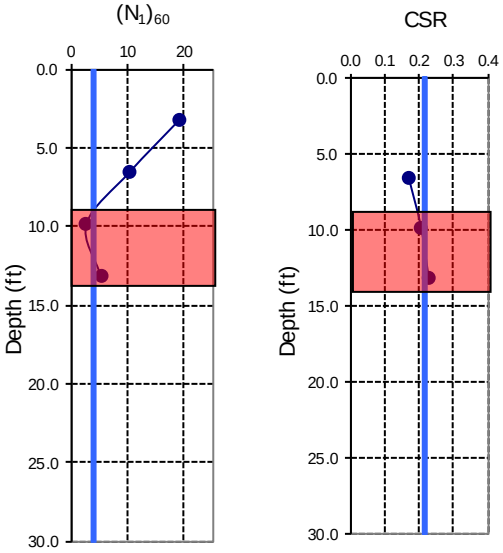
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.4
Data Class	B			% Fines	10.0 ± 2.0	10.0	10
Critical Depth Range	9.8 - 13.1	14.1	14.0	% PI			
Depth to GWT (ft)	5.6 ± 0.3	5.9	6.0				
σ_v (psf)	1233.1 ± 67.9	1670.8	1680.0	N	3.0 ± 1.3	4.0	4.0
σ_v' (psf)	866.6 ± 40.4	1169.6	1180.0	C_R	0.87	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.97 ± 0.051	0.97	0.970	C_B	1.00	1.00	
CSR	0.216 ± 0.066	0.216	0.215	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.56	1.41	1.3
MSF		0.95	1.00	$(N_1)_{60}$	4.0 ± 1.8	5.4	5.00
CSRN		0.218	0.22				



γ_m=100±3 pcf

γ_s=115±3 pcf

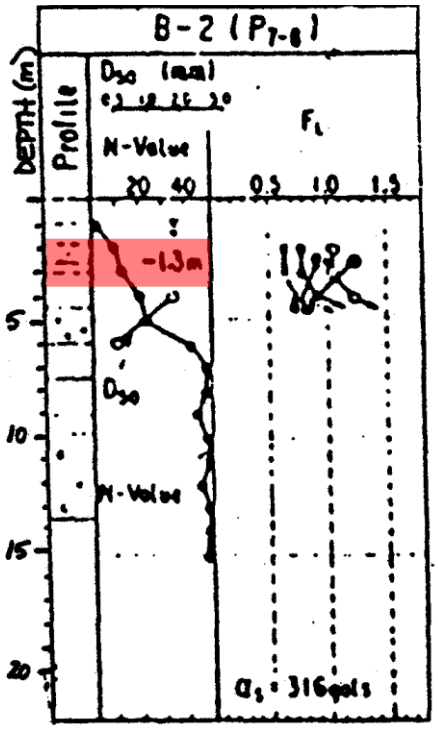


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	13	2.00	0.73	19.1	close to surf.
2.0	6.6	0.170	7	1.86	0.80	10.4	fines transition
3.0	9.8	0.206	2	1.64	0.85	***2.8***	
4.0	13.1	0.228	4	1.49	0.89	***5.3***	
					Mean:	4.0	
					St. Dev.	1.8	

Case number:	70
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriage Bridge 2
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils out of surface cracks. Damage to Yuriagegami Bridge, cracked piers, etc.
Comments:	Yuriage Br. 2 is located near pier 7 of the Yuriagegami Bridge, which crosses Natori River. Pier 7 is towards the right bank of the river. PGA estimations of 0.24 and 0.3 g were used by Iwasaki et al (1978). Based on cyclic triaxial test results, the sand layer at depth range of 2-5 m is predicted to be liquefied. Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The dry river bed was cracked parallel to the river near Yuriage-Ohashi Bridge, and liquefied sands seeped out from the cracks. Some cracks were found near the piers of the Yuriage-Ohashi Birdge and the ground around the piers settled. The boiled soil was a poorly graded coarse sand. Iwasaki and Tokida (1980) identified the liquefied layer as at depths of 2-5 m. Gravel percent exceeds 20 % by weight. (Seed et al, 84) SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 67 % by Seed et al. (84)

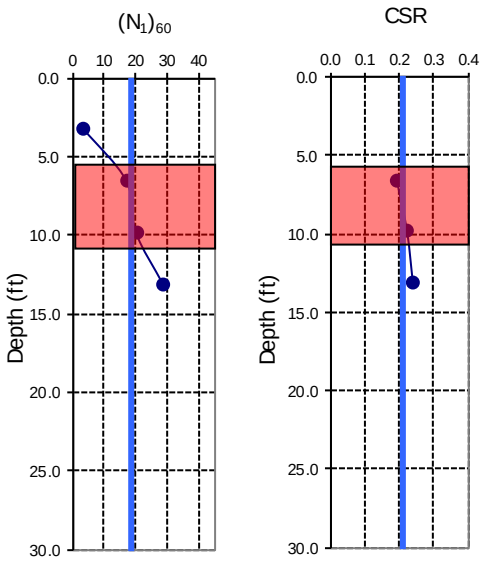
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	1.600 ± 0.050		1.6
Data Class	B			% Fines	7.0 ± 2.0	7.0	7
Critical Depth Range	6.0 - 10.0	8.2	11.0	% PI			
Depth to GWT (ft)	4.3 ± 0.3	3.9	4.0				
σ_v (psf)	893.4 ± 85.4	960.7	1320.0	N	11.4 ± 1.3	10.1	13.0
σ_v' (psf)	660.3 ± 46.4	710.1	880.0	C_R	0.82	0.85	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	1.00 ± 0.038	0.99	0.980	C_B	1.00	1.00	
CSR	0.210 ± 0.064	0.212	0.230	C_E	1.12	1.12	1.120
Equivalent Magnitude	7.7	7.7		C_N	1.79	1.68	1.5
MSF		0.95	1.02	$(N_1)_{60}$	18.7 ± 2.2	16.2	21.50
CSRN		0.203	0.23				



γ_m=100±3 pcf

γ_s=125±3 pcf

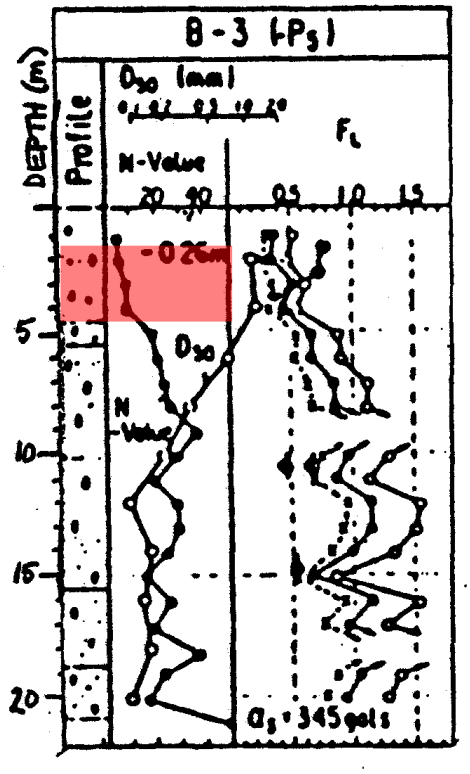


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	NA	2	2.00	0.73	3.3	close to surf.
2.0	6.6	0.195	10	1.93	0.80	***17.2***	
3.0	9.8	0.225	13	1.65	0.85	***20.3***	
4.0	13.1	0.242	20	1.47	0.89	29.1	
					Mean:	18.7	
					St. Dev.	2.2	

Case number:	71
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriage Bridge 3
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils out of surface cracks. Damage to Yuriagegami Bridge, cracked piers, etc.
Comments:	Yuriage Br. 3 is located near pier 5 of the Yuriagegami Bridge, which crosses Natori River. Pier 5 is towards the center of the river. PGA estimations of 0.24 and 0.3 g were used by Iwasaki et al (1978). Based on cyclic triaxial test results, the sand layer at depth range of 1-2 m is predicted to be liquefied. Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The dry river bed was cracked parallel to the river near Yuriage-Ohashi Bridge, and liquefied sands seeped out from the cracks. Some cracks were found near the piers of the Yuriage-Ohashi Birdge and the ground around the piers settled. The boiled soil was a poorly graded coarse sand. Iwasaki and Tokida (1980) identified the liquefied layer as at depths of 2-5 m. SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

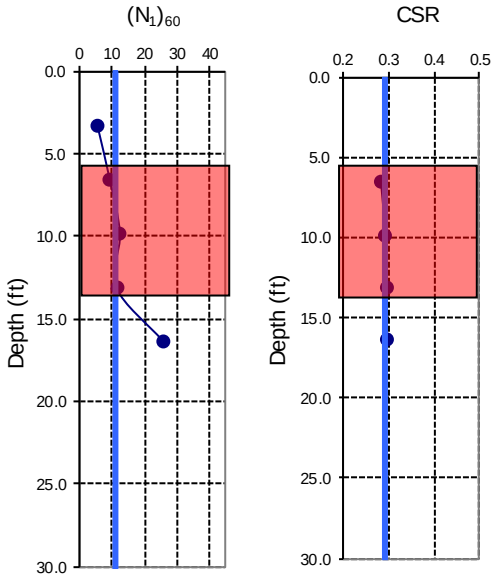
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	1.200 ± 0.050		1.2
Data Class	B			% Fines	12.0 ± 2.0	12.0	12
Critical Depth Range	6.6 - 13.1	14.1	14.0	% PI			
Depth to GWT (ft)	0.9 ± 0.3	1.0	1.0				
σ_v (psf)	1172.6 ± 134.0	1670.8	1680.0	N	7.1 ± 1.0	8.0	8.0
σ_v' (psf)	611.6 ± 70.3	877.2	870.0	C_R	0.85	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.99 ± 0.045	0.97	0.970	C_B	1.00	1.00	
CSR	0.295 ± 0.090	0.293	0.290	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.86	1.56	1.5
MSF		0.95	1.00	$(N_1)_{60}$	11.2 ± 1.5	11.8	12.00
CSRN		0.282	0.29				



γ_m=110±3 pcf

γ_s=120±3 pcf

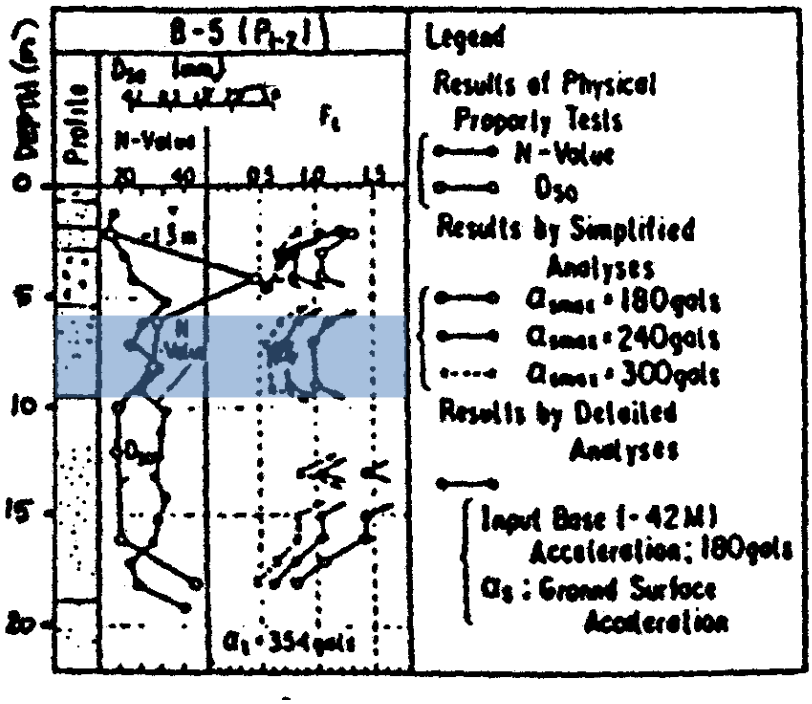


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3	0.256	4	2.00	0.73	5.9	close to surf.
2.0	6.6	0.285	6	2.00	0.80	***9.6***	
3.0	9.8	0.295	8	1.86	0.85	***12.6***	
4.0	13.1	0.298	8	1.63	0.89	***11.5***	
5.0	16.4	0.298	19	1.46	0.92	25.6	
					Mean:	11.2	
					St. Dev.	1.5	

Case number:	72
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriage Bridge 5
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction.
Comments:	Yuriage Br. 5 is located near pier 1 of the Yuriagegami Bridge, which crosses the Natori River. Pier 5 is the closest pier to the left bank of the river. PGA estimations of 0.24 and 0.3 g were used by Iwasaki et al (1978). Based on cyclic triaxial test results, the sand layer at depth range of 3-9 m is predicted to be liquefied. However no surface evindence of liquefaction exists. Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The dry river bed was cracked parallel to the river near Yuriage-Ohashi Bridge, and liquefied sands seeped out from the cracks. Some cracks were found near the piers of the Yuriage-Ohashi Birdge and the ground around the piers settled. The boiled soil was a poorly graded coarse sand. Iwasaki and Tokida (1980) identified the potentially liquefiable layer at depths of 2-5 m. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 67 % by Seed et al. (84)

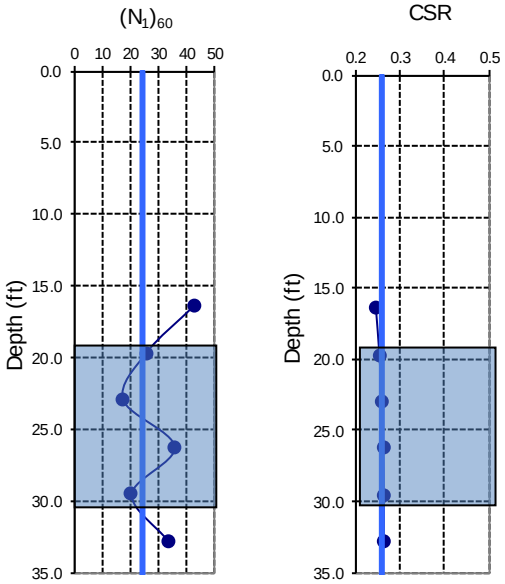
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.350 ± 0.050		0.35
Data Class	B			% Fines	17.0 ± 2.0	17.0	17
Critical Depth Range	19.7 - 29.5	24.0	24.0	% PI			
Depth to GWT (ft)	4.3 ± 0.5	3.9	4.0				
σ_v (psf)	3054.5 ± 214.3	2882.2	2880.0	N	20.5 ± 6.7	17.0	17.0
σ_v' (psf)	1785.2 ± 123.5	1629.1	1630.0	C_R	0.99	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.98 ± 0.099	0.94	0.940	C_B	1.00	1.00	
CSR	0.261 ± 0.083	0.260	0.260	C_E	1.12	1.12	1.120
Equivalent Magnitude	7.7	7.7		C_N	1.09	1.11	1.1
MSF		0.95	1.02	$(N_1)_{60}$	24.6 ± 8.0	20.1	21.00
CSRN		0.263	0.26				



γ_m=120±3 pcf

γ_s=125±3 pcf

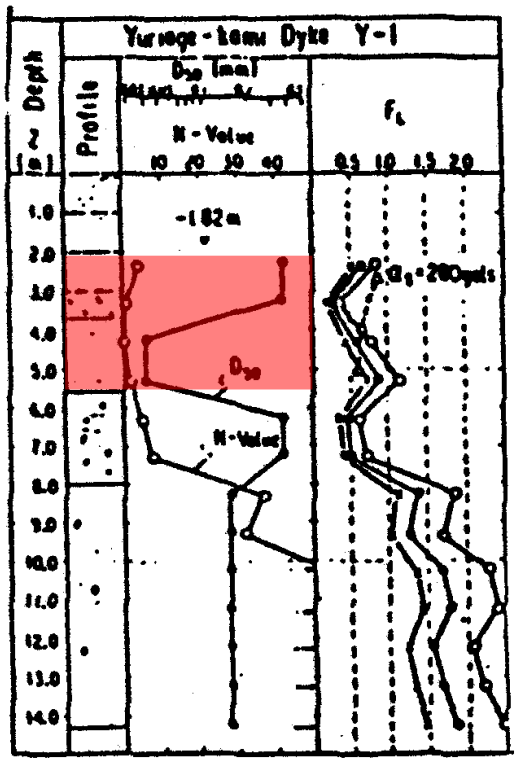


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.0	16.4	0.247	32	1.29	0.92	42.5
6.0	19.7	0.254	20	1.20	0.95	***25.4***
7.0	23.0	0.259	14	1.12	0.98	***17.1***
8.0	26.2	0.262	30	1.06	1.00	***35.5***
9.0	29.5	0.263	18	1.01	1.00	***20.2***
10.0	32.8	0.263	31	0.96	1.00	33.2
					Mean:	24.6
					St. Dev.	8.0

Case number:	73
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriagekami-1
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand volcanoes, cracking and spreading of the ground towards the pond, also damage to the bank of the dike.
Comments:	Yuriagekami is located in between the edge of the Shin-enko numa pond just just inland from the Natori rive, and the base of the dyke on the right bank of the river. PGA is estimated as 0.18 g by Iwasaki et al (1978). Based on cyclic triaxial test results, the liquefied layer is predicted to be the sand layer at a depth of 2-8 m. Tohno et al. (1981) Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7. The river embankment was partially collpased and cracked on both sides at Yuriagekami. SPT values were taken after main earthquake (M=7.4) SPT energy is estimated as 60 % by Seed et al. (84)

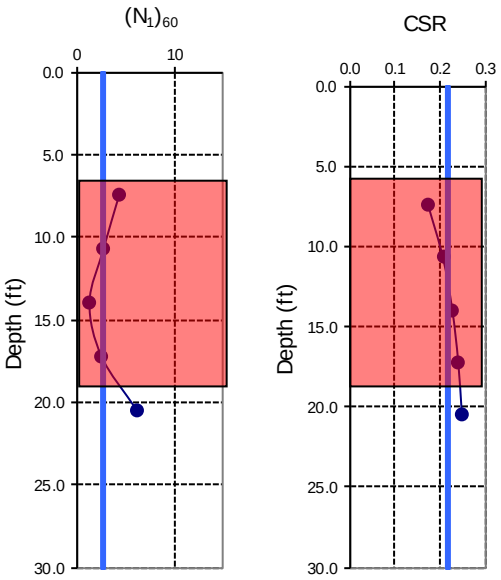
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.040 ± 0.050		0.04
Data Class	B			% Fines	60.0 ± 2.0	60.0	60
Critical Depth Range	5.9 - 18.0	18.0	18.0	% PI			
Depth to GWT (ft)	6.0 ± 0.3	5.9	6.0				
σ_v (psf)	1257.5 ± 224.0	2067.7	2060.0	N	2.0 ± 0.9	2.0	2.0
σ_v' (psf)	882.9 ± 100.8	1315.8	1310.0	C_R	0.87	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.98 ± 0.053	0.96	0.960	C_B	1.00	1.00	
CSR	0.217 ± 0.068	0.236	0.235	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.55	1.31	1.2
MSF		0.95	1.02	$(N_1)_{60}$	2.7 ± 1.2	2.5	2.50
CSRN		0.239	0.23				



γ_m=100±3 pcf

γ_s=110±3 pcf

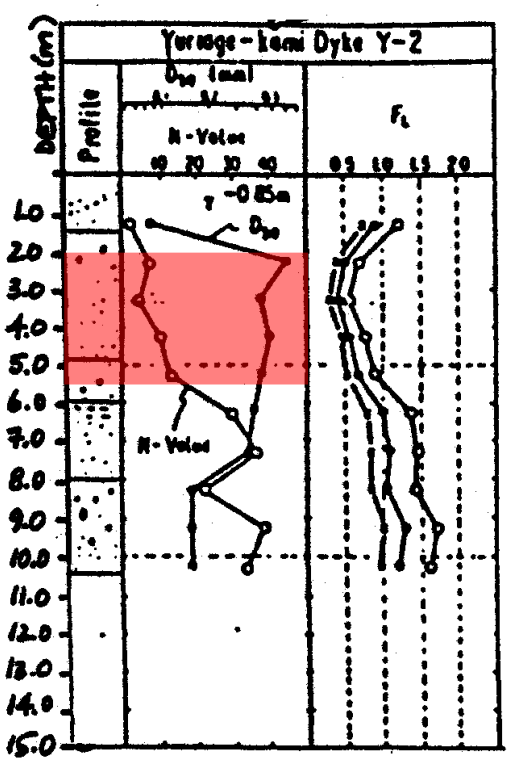


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.3	7.4	0.175	3	1.78	0.81	***4.3***
3.3	10.7	0.208	2	1.61	0.86	***2.8***
4.3	13.9	0.229	1	1.47	0.90	***1.3***
5.3	17.2	0.242	2	1.37	0.93	***2.5***
6.3	20.5	0.250	5	1.28	0.96	6.1
					Mean:	2.7
					St. Dev.	1.2

Case number:	74
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriagekami-2
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	Sand boils.
Comments:	Yuriagekami-2 is located by the edge of the Natori river, on the right bank of the river. PGA is estimated as 0.18 g by Iwasaki et al (1978). Based on cyclic triaxial test results, the liquefied layer is peridicted to be the sand layer at depth of 1-6 m. Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7. The river embankment was partially collapsed and cracked on both sides at Yuriagekami. Cracks were observed both on and at the base of the dike. Also sand volcanoes were found along the dike. SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 60 % by Seed et al. (84)

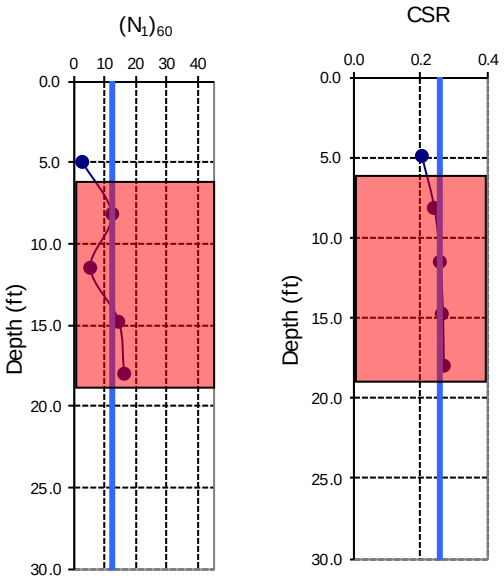
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.400 ± 0.050		0.4
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	6.6 - 18.0	14.1	14.0	% PI			
Depth to GWT (ft)	2.8 ± 0.3	3.0	3.0				
σ_v (psf)	1420.6 ± 231.7	1670.8	1680.0	N	8.8 ± 3.3	11.0	11.0
σ_v' (psf)	826.9 ± 114.9	981.6	990.0	C_R	0.88	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.97 ± 0.055	0.97	0.970	C_B	1.00	1.00	
CSR	0.260 ± 0.080	0.258	0.255	C_E	1.00	1.00	1.000
Equivalent Magnitude	7.7	7.7		C_N	1.60	1.45	1.4
MSF		0.95	1.02	$(N_1)_{60}$	12.3 ± 4.7	15.1	15.00
CSRN		0.251	0.25				



γ_m=100±3 pcf

γ_s=120±3 pcf

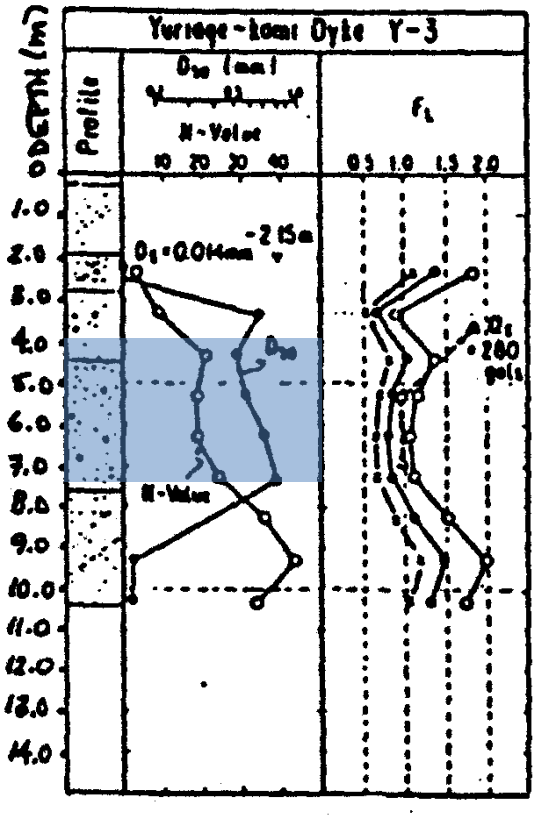


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.5	4.9	0.206	2	2.00	0.77	3.1	close to surf.
2.5	8.2	0.241	8	1.89	0.82	***12.5***	
3.5	11.5	0.257	4	1.65	0.87	***5.7***	
4.5	14.8	0.265	11	1.48	0.90	***14.7***	
5.5	18.0	0.267	13	1.35	0.94	***16.5***	
					Mean:	12.3	
					St. Dev.	4.7	

Case number:	75
Earthquake:	1978 Miyagiken-Oki
Magnitude:	7.7 (Mw) USGS Centennial Earthquake Catalog
Location:	Yuriagekami-3
References:	Iwasaki et al. (1978) Tohno et al. (1981) Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction.
Comments:	Yuriagekami-3 is located at the base of the dike on the right bank of the Natori River, not far from the Shin-enko numa pond. PGA is estimated as 0.18 g by Iwasaki et al (1978). Tohno et al. (1981): Prior to the main shock of M=7.4, an earthquake of magnitude 6.7 occurred. The subject zone is seismically very active and had been shaken in 1897 and 1936 by earthquakes of Magnitudes 7.3 and 7.7 The river embankment was partially collapsed and cracked on both sides at Yuriagekami. Cracks were observed both on and at the base of the dike. Also sand volcanoes were found along the dike. SPT values were taken after main earthquake (M=7.4) SPT energy was estimated as 67 % by Seed et al. (84)

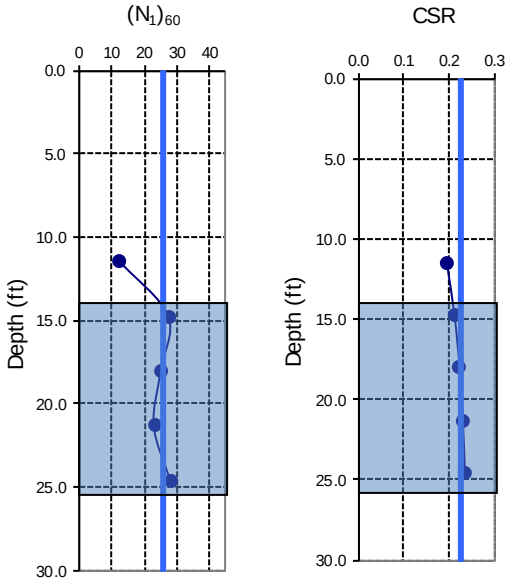
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.600 ± 0.050		0.6
Data Class	B			% Fines	0.0 ± 2.0	0.0	0
Critical Depth Range	14.8 - 24.6	18.0	18.0	% PI			
Depth to GWT (ft)	7.1 ± 0.3	6.9	7.0				
σ_v (psf)	2354.8 ± 209.6	2151.2	2160.0	N	21.1 ± 1.9	20.0	20.0
σ_v' (psf)	1566.6 ± 112.4	1462.0	1470.0	C_R	0.95	0.95	
a_{max} (g)	0.240 ± 0.072	0.24	0.240	C_S	1.00	1.00	
r_d	0.97 ± 0.082	0.96	0.960	C_B	1.00	1.00	
CSR	0.227 ± 0.071	0.220	0.220	C_E	1.12	1.12	1.120
Equivalent Magnitude	7.7	7.7		C_N	1.16	1.16	1.2
MSF		0.95	1.02	$(N_1)_{60}$	26.0 ± 2.3	24.6	26.00
CSRN		0.220	0.22				



γ_m=110±3 pcf

γ_s=125±3 pcf

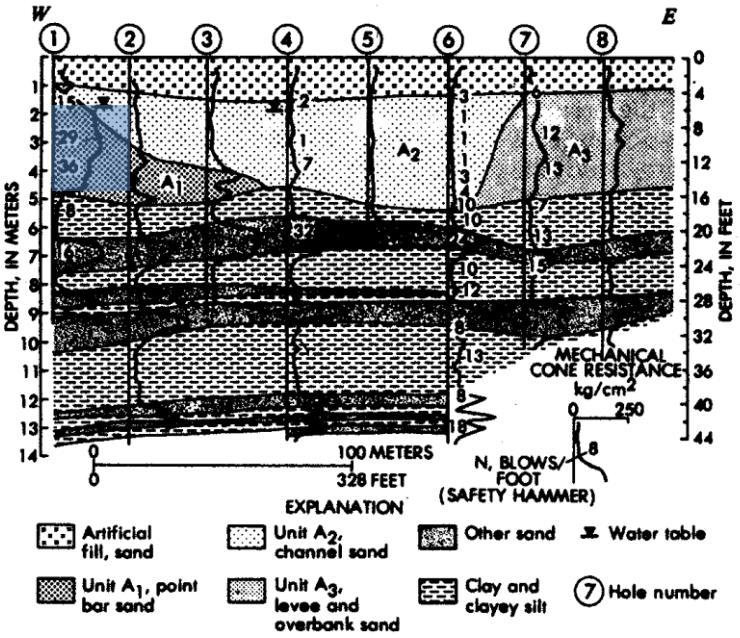


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
3.5	11.5	0.195	9	1.42	0.87	12.4	clay?
4.5	14.8	0.212	21	1.30	0.90	***27.5***	
5.5	18.0	0.223	20	1.20	0.94	***25.1***	
6.5	21.3	0.231	19	1.13	0.96	***23.0***	
7.5	24.6	0.235	24	1.06	0.99	***28.2***	
					Mean:	26.0	
					St. Dev.	2.3	

Case number:	76
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Heber Road A1
References:	Youd and Bennett (1983), "Liquefaction sites, Imperial Valley, CA", Benett et. al. (1979)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site is ~1.6 km to the fault rupture plane. AGR and BCR are two stations that are located ~ 7 kms southeast of Heber Road site. The peak acceleration recorded at station AGR at an epicentral distance of 1.8 km is 0.35 and 0.23 g. The station is underlain by deep alluvium and the motion is free field. At Bonds Corner, 5.2 km southern terminus of the fault rupture and 6 km form the southeast of the Heber Road site, PGA was recorded as 0.76 and 0.58 g. Seed et al (1984) used a PGA value of 0.78 g. It is believed that directivity effects play a major role in the intensity of shaking. Higher intensity shakings (0.83 g) were recorded at the northern section of the fault. Based on the recorded motion and earthquake specific PGA contour maps, PGA was estimated as in the range of 0.4-0.7 g. Site response analyses based PGA estimation was in the range of ~ 0.45 to 0.50 ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

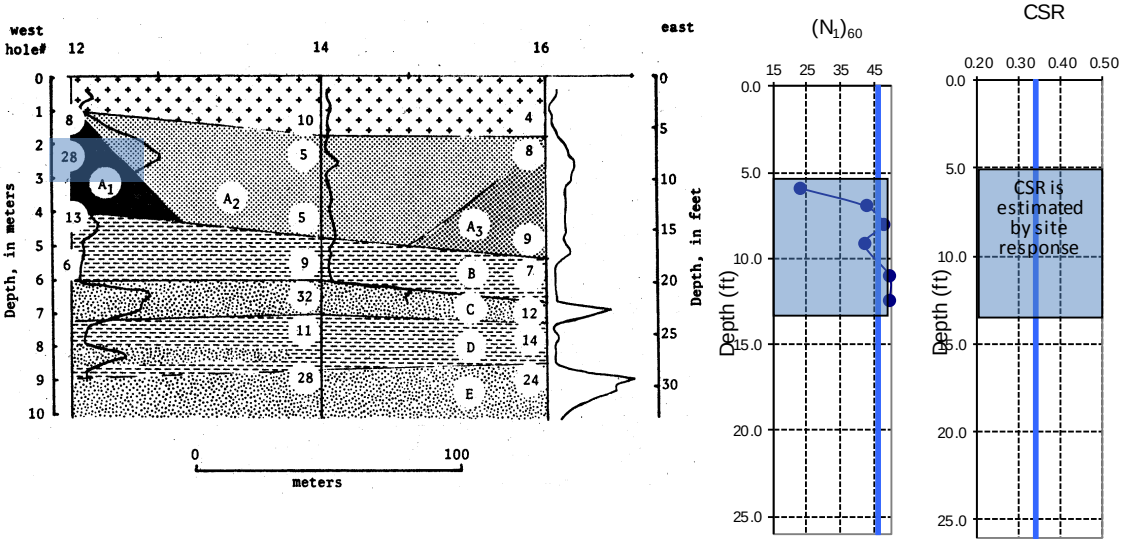
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.111 ± 0.007		0.112
Data Class	A			% Fines	13.0 ± 3.9	12.0	25
Critical Depth Range	5.9 - 16.4	9.5	12.0	% PI			
Depth to GWT (ft)	5.9	5.9	6.0				
σ_v (psf)	1246.7 ± 220.1	1106.9	1410.0	N	32.9 ± 2.6	30.4	28.0
σ_v' (psf)	919.2 ± 112.6	877.2	1035.0	C_R	0.82	0.85	
a_{max} (g)	0.470 ± 0.071	0.78	0.780	C_S	1.00	1.00	
r_d	0.82 ± 0.010	0.97	0.970	C_B	1.00	1.00	1.050
CSR	0.340 ± 0.055	0.620	0.670	C_E	1.13	1.13	
Equivalent Magnitude	6.53	6.53		C_N	1.52	1.30	1.4
MSF		1.29	1.17	$(N_1)_{60}$	46.5 ± 3.7	37.8	39.50
CSRN		0.437	0.58				



γ_m=100±3 pcf

γ_s=125±3 pcf

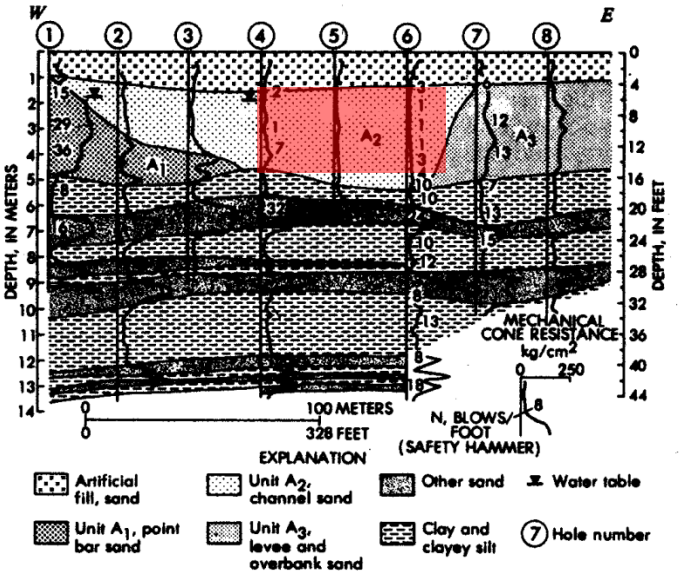


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.8	5.9		15	1.89	0.73	23.4	BH-1
2.12	7.0		28	1.80	0.75	***42.7***	BH-12
2.45	8.0		32	1.71	0.77	***47.8***	BH-1 Donut
2.8	9.2		29	1.63	0.79	***42.4***	BH-1
3.35	11.0		35	1.53	0.82	***49.7***	BH-1 Donut
3.8	12.5		36	1.45	0.84	***49.9***	BH-1
					Mean:	46.5	
					St. Dev.	3.7	

Case number:	77
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Heber Road A2
References:	Youd and Bennett (1983), "Liquefaction sites, Imperial Valley, CA", Benett et. al. (1979)
Nature of Failure:	Liquefaction-induced lateral spreading shifted the road as much as 7 ft. southward toward a 6.6 ft deep depression. Ground cracks and scaprs formed. Sand boils erupted at several placed along the spread.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site is ~1.6 km to the fault rupture plane. AGR and BCR are two stations that are located ~ 7 kms southeast of Heber Road site. The peak acceleration recorded at station AGR at an epicentral distance of 1.8 km is 0.35 and 0.23 g. The station is underlain by deep alluvium and the motion is free field. At Bonds Corner, 5.2 km southern terminus of the fault rupture and 6 km form the southeast of the Heber Road site, PGA was recorded as 0.76 and 0.58 g. Seed et al (1984) used a PGA value of 0.78 g. It is believed that directivity effects play a major role in the intensity of shaking. Higher intensity shakings (0.83 g) were recorded at the northern section of the fault. Based on the recorded motion and earthquake specific PGA contour maps, PGA was estimated as in the range of 0.4-0.7 g. Site response analyses based PGA estimation was in the range of ~ 0.45 to 0.50 ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

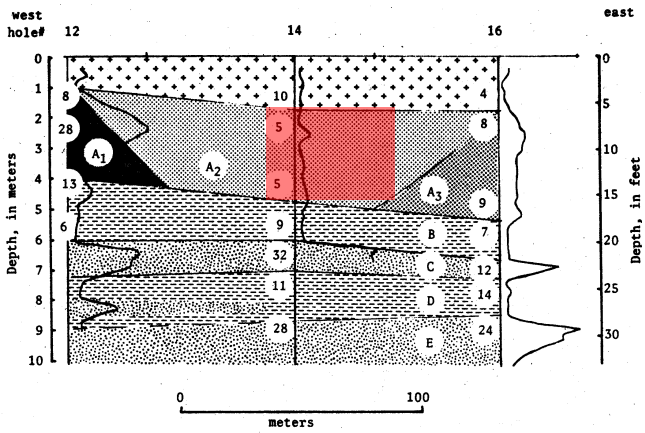
Summary of Data

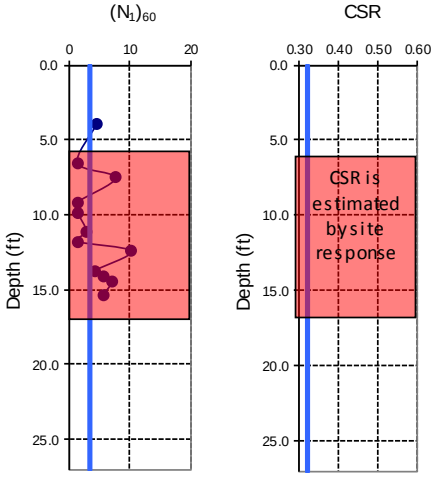
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.116 ± 0.013		0.119
Data Class	A			% Fines	20.9 ± 5.0	18.0	29
Critical Depth Range	6.0 - 15.1	12.1	12.0	% PI			
Depth to GWT (ft)	5.9	5.9	6.0				
σ_v (psf)	1103.0 ± 167.6	1420.2	1410.0	N	2.4 ± 1.5	2.0	1.0
σ_v' (psf)	812.3 ± 76.9	1044.3	1035.0	C_R	0.81	0.85	
a_{max} (g)	0.470 ± 0.071	0.78	0.780	C_S	1.00	1.00	
r_d	0.78 ± 0.020	0.96	0.970	C_B	1.00	1.00	1.050
CSR	0.324 ± 0.054	0.661	0.670	C_E	1.13	1.13	
Equivalent Magnitude	6.53	6.53		C_N	1.61	1.51	1.4
MSF		1.29	1.17	$(N_1)_{60}$	3.5 ± 2.2	2.9	1.50
CSRN		0.484	0.58				



γ_m=100±3 pcf

γ_s=110±3 pcf



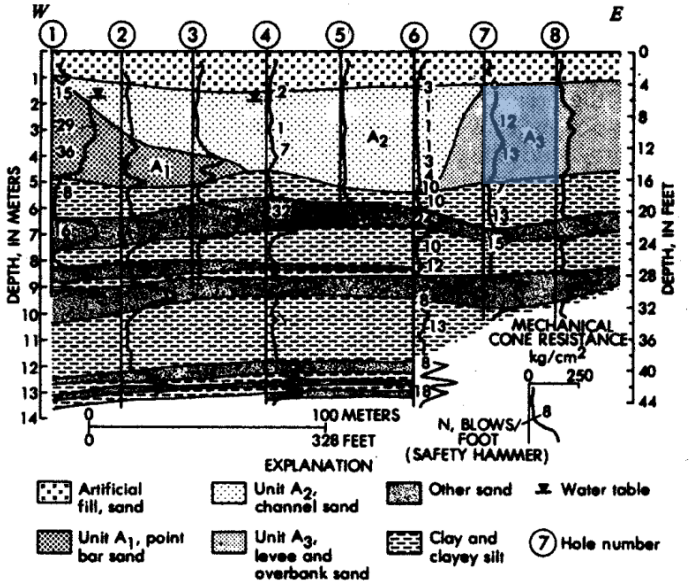


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.2	3.9		3	2.00	0.68	4.6	BH-6
2	6.6		1	1.84	0.74	***1.6***	BH-6
2.3	7.5		5	1.78	0.76	7.7	BH-14
2.8	9.2		1	1.68	0.79	***1.5***	BH-6
3	9.8		1	1.65	0.80	***1.5***	BH-4
3.4	11.2		2	1.59	0.82	***3.0***	BH-5 Donut
3.6	11.8		1	1.56	0.83	***1.5***	BH-6
3.8	12.5		7	1.53	0.84	10.2	BH-4
4.2	13.8		3	1.48	0.86	***4.3***	BH-6
4.3	14.1		4	1.47	0.86	***5.7***	BH-5 Donut
4.4	14.4		5	1.46	0.87	***7.2***	BH-14
4.7	15.4		4	1.42	0.88	***5.7***	BH-6
					Mean:	3.5	
					St. Dev.	2.2	

Case number:	78
Earthquake:	Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Heber Road A3
References:	Youd and Bennett (1983), "Liquefaction sites, Imperial Valley, CA"
Nature of Failure:	No surface manifestation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site is 1.6 km to the fault rupture plane. AGR and BCR are two stations that are located ~ 7 kms southeast of Heber Road site. The peak acceleration recorded at station AGR at an epicentral distance of 1.8 km is 0.35 and 0.23 g. The station is underlain by deep alluvium and the motion is free field. At Bonds Corner, 5.2 km southern terminus of the fault rupture and 6 km form the southeast of the Heber Road site, PGA was recorded as 0.76 and 0.58 g. Seed et al (1984) used a PGA value of 0.78 g. It is believed that directivity effects play a major role in the intensity of shaking. Higher intensity shakings (0.83 g) were recorded at the northern section of the fault. Based on the recorded motion and earthquake specific PGA contour maps, PGA was estimated as in the range of 0.4-0.7 g. Site response analyses based PGA estimation was in the range of ~ 0.45 to 0.50 ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

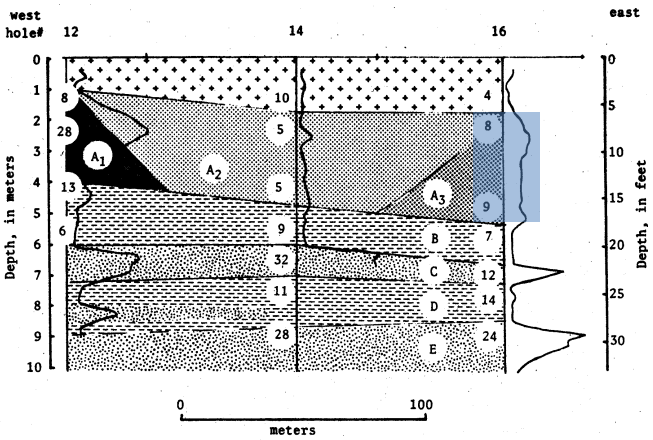
Summary of Data

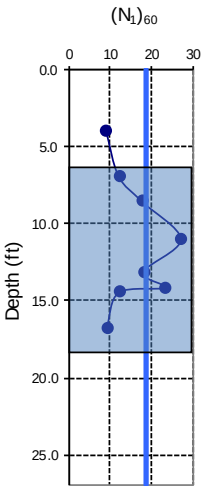
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.094 ± 0.004		0.094
Data Class	A			% Fines	25.3 ± 5.2	25.0	37
Critical Depth Range	5.9 - 16.1	13.1	14.0	% PI			
Depth to GWT (ft)	5.9	5.9	6.0				
σ_v (psf)	1200.8 ± 204.8	1649.9	1660.0	N	13.0 ± 4.2	13.0	13.0
σ_v' (psf)	883.5 ± 101.2	1169.58	1160.0	C_R	0.82	0.85	
a_{max} (g)	0.470 ± 0.071	0.78	0.780	C_S	1.00	1.00	
r_d	0.75 ± 0.025	0.950	0.960	C_B	1.00	1.00	1.050
CSR	0.311 ± 0.052	0.69	0.695	C_E	1.13	1.13	
Equivalent Magnitude	6.53	6.53		C_N	1.55	1.29	1.3
MSF		1.290	1.17	$(N_1)_{60}$	18.6 ± 6.0	16.2	17.50
CSRN			0.60				

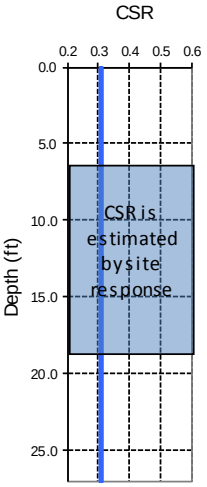


γ_m=100±3 pcf

γ_s=120±3 pcf





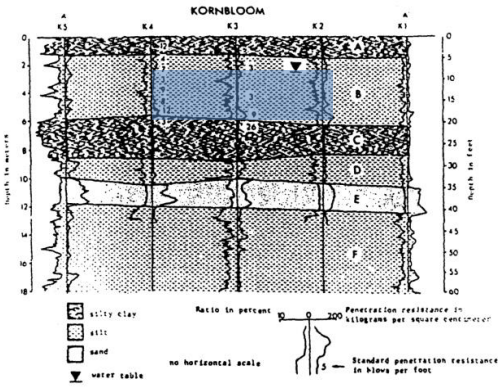


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.2	3.9		6	2.00	0.68	9.2	BH-7
2.1	6.9		8	1.81	0.75	***12.3***	BH-16
2.6	8.5		12	1.69	0.78	***17.9***	BH-7
3.35	11.0		19	1.55	0.82	***27.4***	BH-8
4	13.1		13	1.45	0.85	***18.2***	BH-7
4.3	14.1		17	1.41	0.86	***23.5***	BH-8
4.4	14.4		9	1.40	0.87	***12.4***	BH-16
5.1	16.7		7	1.32	0.89	9.4	BH-7
					Mean:	18.6	
					St. Dev.	6.0	

Case number:	79
Earthquake:	1979 Imperial Valley Earthquake
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Kornbloom B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: silty clay, stiff-lacustrine and/or flood plain deposit B: silty and sandy silt-fluvial and perhaps fluvial deltaic deposition C: silty clay to clay, stiff-lacustrine and/or flood plain deposit D: interbedded silt and sand-loose to medium dense E: Uniform sand with some silt-medium dense to dense F: interbedded silt and sand - loose to medium dense. D-F :fluvial and perhaps fluvial-deltaic deposition PGA was recorded at the ground level of a two-story building as 0.13 g (CAL) which is ~23 km from the north terminus of the fault and ~16 km north of the north terminus of the fault, PGA is recorded as 0.11 g.(Westmorland). The site is 25 km north of the north terminus of the fault and ~9 km from the westmorland seismographic station. PGA was estimated as 0.12 g by Andrus and Stoke (NCEER, 97). Due to rapid dissipation of wave energy beyond the north terminus of the fault, PGA may be lower than what is recorded at Westmorland. Based on earthquake specific attenuation relationship, PGA was estimated as 0.06-0.1 g. A value of 0.13 is adopted for the study based on site response analysis results. Energy ratio ~68 %

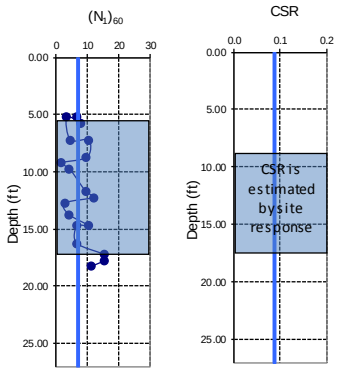
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.045 ± 0.008		0.045
Data Class	A			% Fines	83.0 ± 9.6	92.0	92
Critical Depth Range	9.0 - 17.5	14.1	14.0	% PI			
Depth to GWT (ft)	9.0	8.9	9.0				
σ_v (psf)	1410.0 ± 172.7	1608.2	1615.0	N	5.4 ± 3.1	5.0	5.0
σ_v' (psf)	1144.8 ± 88.1	1294.9	1305.0	C_R	0.85	0.85	
a_{max} (g)	0.130 ± 0.020	0.13		C_S	1.00	1.00	
r_d	0.83 ± 0.030	0.95	0.970	C_B	1.00	1.00	1.050
CSR	0.086 ± 0.014	0.099		C_E	1.13	1.13	
Equivalent Magnitude	6.53	6.53		C_N	1.36	1.29	1.2
MSF		1.29		$(N_1)_{60}$	7.1 ± 4.1	6.2	6.50
CSRN		0.073					



γ_m=100±3 pcf

γ_s=120±3 pcf

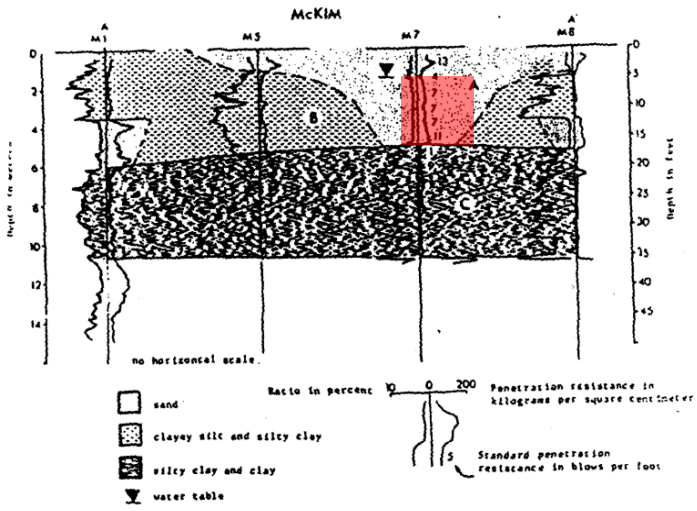


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		FC	sand	silt	clay	D50	WN	WL	PI	LI
1.60	5.25		4	2.00	0.71	6.5	K-4	88.0	12.0	65.0	23.0	0.019	26.0	29.0	11.0	0.7
1.75	5.75		5	1.92	0.72	7.9	K-4 b	84.0	16.0	74.0	10.0	0.038				
1.60	5.25		2	2.00	0.71	*3.2*	SK4-5	-	-	-	-	-				
2.21	7.25		3	1.71	0.76	*4.4*	K-3	*80.0*	*21.0*	*73.0*	*7.0*	*0.044*				
2.21	7.25		7	1.71	0.76	*10.2*	K-4	*74.0*	*26.0*	*68.0*	*6.0*	*0.052*				
2.67	8.75		7	1.56	0.78	*9.7*	K-4 b	*86.0*	*14.0*	*83.0*	*3.0*	*0.049*				
2.82	9.25		1	1.52	0.79	*1.4*	SK4-5	-	-	-	-	-				
2.97	9.75		3	1.50	0.80	*4.1*	K-4	*62.0*	*38.0*	*53.0*	*9.0*	*0.056*	*28.0*			
3.58	11.75		7	1.41	0.83	*9.3*	K-4 b	*88.0*	*12.0*	*79.0*	*9.0*	*0.042*				
3.73	12.25		9	1.40	0.84	*11.9*	K-4	*96.0*	*4.0*	*83.0*	*13.0*	*0.028*	*30.0*			
3.89	12.75		2	1.38	0.85	*2.6*	SK4-5	-	-	-	-	-				
4.19	13.75		3	1.34	0.86	*3.9*	K-3	*91.0*	*9.0*	*79.0*	*12.0*	*0.039*				
4.50	14.75		8	1.31	0.87	*10.4*	K-4	*83.0*	*17.0*	*74.0*	*9.0*	*0.047*	*29.0*			
4.50	14.75		5	1.31	0.87	*6.5*	K-4 b	*81.0*	*19.0*	*71.0*	*10.0*	*0.042*				
4.95	16.25		5	1.27	0.89	*6.4*	SK4-5	-	-	-	-	-				
5.26	17.25		12	1.24	0.90	*15.2*	K-4	*89.0*	*11.0*	*83.0*	*6.0*	*0.048*				
5.41	17.75		12	1.23	0.90	15.1	K-4 b	44.0	56.0	40.0	4.0	0.700				
5.56	18.25		9	1.22	0.91	11.3	K-3	30.0	70.0	29.0	1.0	0.082				
					Mean:	7.1		83.0	17.1	74.6	8.4	0.045				
					St. Dev.	4.1		9.6	9.7	9.3	3.0	0.008				

Case number:	80
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	McKim Ranch A
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984)
Nature of Failure:	Fissures developed and as many as one hundred sand boils erupted at this location. The sand boils occurred between two continuous fissures, some 10-15 m apart, that parallel each other in an arcuate north-to-south zone approximately 1.8 km long.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Located between the Alamo River and the Redwood Canal. A: very fine sand (fining upwards)-loose sand that filled the relic channel. B: stiff silty clay McKim Ranch Site is 6 km from the northern terminus of the fault rupture. The earthquake specific PGA contour maps indicate a PGA level of 0.5-0.8g. 2-3 km away from site, at the seismagraphic station EO4 located on deep alluvium (>300m), PGA was recorded as 0.49g. Energy ratio ~68 %

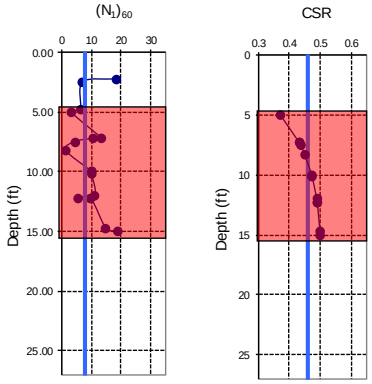
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.106 ± 0.005		0.105
Data Class	B			% Fines	19.8 ± 3.5	31	31
Critical Depth Range	5.0 - 13.0	6.9	7.0	% PI			
Depth to GWT (ft)	5.0	4.9	5.0				
σ_v (psf)	1030.0 ± 161.2	793.6	800.0	N	5.4 ± 2.6	3	3.0
σ_v' (psf)	780.4 ± 80.7	668.3	675.0	C_R	0.79	0.80	0.8
a_{max} (g)	0.510 ± 0.153	0.51	0.510	C_S	1.00	1.00	
r_d	0.94 ± 0.042	0.98	0.990	C_B	1.00	1.00	
CSR	0.411 ± 0.127	0.385	0.390	C_E	1.13	1.13	1.050
Equivalent Magnitude	6.53	6.53		C_N	1.65	1.70	1.6
MSF		1.29	1.16	$(N_1)_{60}$	7.9 ± 3.8	4.6	4.00
CSRN		0.271	0.34				



γ_m=110±3 pcf

γ_s=120±3 pcf



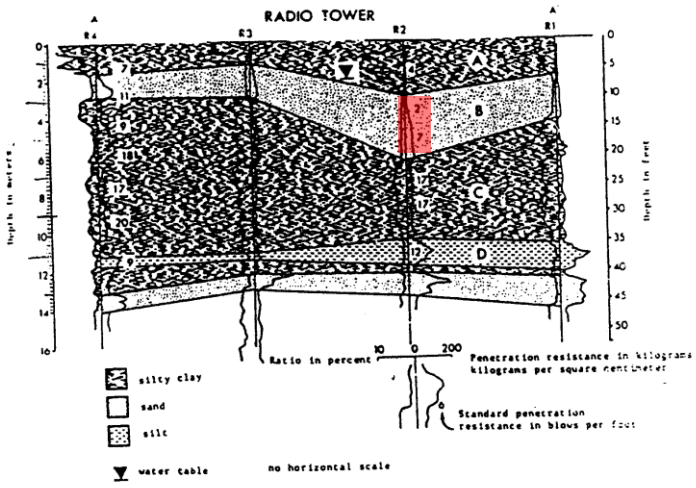
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
0.69	2.25	NA	13	2.00	0.62	18.3	M-7
0.76	2.50	NA	5	2.00	0.63	7.2	SM-7
1.45	4.75	NA	4	2.00	0.70	***6.3***	***M-7***
1.52	5.00	0.322	2	1.96	0.71	***3.1***	***SM-7***
2.21	7.25	0.382	9	1.76	0.76	***13.6***	***M-2***
2.21	7.25	0.382	7	1.76	0.76	***10.6***	***M-7***
2.29	7.50	0.387	3	1.75	0.76	***4.5***	***SM-7***
2.51	8.25	0.400	1	1.69	0.78	***1.5***	***M-3***
3.05	10.00	0.423	7	1.59	0.81	***10.2***	***SM-7***
3.09	10.15	0.425	7	1.58	0.81	***10.1***	***M-7***
3.66	12.00	0.440	8	1.49	0.84	***11.3***	***SM-7***
3.73	12.25	0.441	4	1.48	0.84	***5.6***	***M-3***
3.73	12.25	0.441	7	1.48	0.84	***9.8***	***M-7***
4.50	14.75	0.450	11	1.38	0.87	15.0	M-7
4.57	15.00	0.450	14	1.37	0.87	19.0	SM-7
					Mean:	7.9	
					St. Dev.	3.8	

Depth (ft)	FC	sand	silt	clay	D50	
2.25	31.0	69.0	18.0	13.0	0.085	M-7
2.50	-	-	-	-	-	SM-7
4.75	*-*	*-*	*-*	*-*	*-*	***M-7***
5.00	*-*	*-*	*-*	*-*	*-*	***SM-7***
7.25	*-*	*-*	*-*	*-*	*-*	***M-2***
7.25	***24.0***	***76.0***	***14.0***	***10.0***	***0.100***	***M-7***
7.50	*-*	*-*	*-*	*-*	*-*	***SM-7***
8.25	*-*	*-*	*-*	*-*	*-*	***M-3***
9.40	***21.0***	***79.0***	***11.0***	***10.0***	***0.105***	***M-7***
10.00	*-*	*-*	*-*	*-*	*-*	***SM-7***
10.15	***16.0***	***84.0***	***9.0***	***7.0***	***0.110***	***M-7***
12.00	*-*	*-*	*-*	*-*	*-*	***SM-7***
12.25	*-*	*-*	*-*	*-*	*-*	***M-3***
12.25	***18.0***	***82.0***	***11.0***	***7.0***	***0.110***	***M-7***
14.75	22.0	78.0	15.0	7.0	0.093	M-7
15.00	-	-	-	-	-	SM-7
	19.8	80.3	11.3	8.5	0.106	
	3.5	3.5	2.1	1.7	0.005	

Case number:	81
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Radio Tower B1
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	Liquefied during both the 1979 and 1981 earthquakes. Several sandboils expelled considerable volumes of water and sand and created a new pond near the radio tower at the site. Small fissures devepoed around the pond and larger fissures developed at the margin of the river flood plane locally in the adjacent bluffs.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand Radio Tower Site is 12 km from the northern terminus of the fault rupture. 0.22g and 0.16 of PGA was recorded at BRA recording site which is 9-10 km from the northern terminus of the fault rupture. Based on the earthquake specific PGA contours, the estimated PGA at the River site is ~0.2 g. However, site response analyses results suggested a slightly lower value: 0.18 g Energy ratio ~68 %

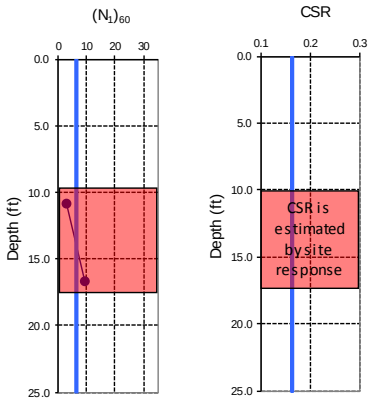
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.074 ± 0.037		0.047
Data Class	B			% Fines	43.5 ± 29.0	64.0	75
Critical Depth Range	9.8 - 18.0	11.2	11.0	% PI			
Depth to GWT (ft)	6.6	6.9	7.0				
σ_v (psf)	1505.1 ± 160.1	1294.9	1270.0	N	4.5 ± 3.4	2.0	2.0
σ_v' (psf)	1044.5 ± 79.5	1044.3	1020.0	C_R	0.86	0.85	
a_{max} (g)	0.180 ± 0.027	0.20	0.200	C_S	1.00	1.00	1.050
r_d	0.97 ± 0.030	0.96	0.960	C_B	1.00	1.00	
CSR	0.164 ± 0.026	0.154	0.160	C_E	1.13	1.13	
Equivalent Magnitude	6.53	6.53		C_N	1.42	1.49	1.4
MSF		1.29	1.19	$(N_1)_{60}$	6.2 ± 4.7	2.9	3.00
CSRN		0.112	0.14				



γ_m=100±3 pcf

γ_s=115±3 pcf



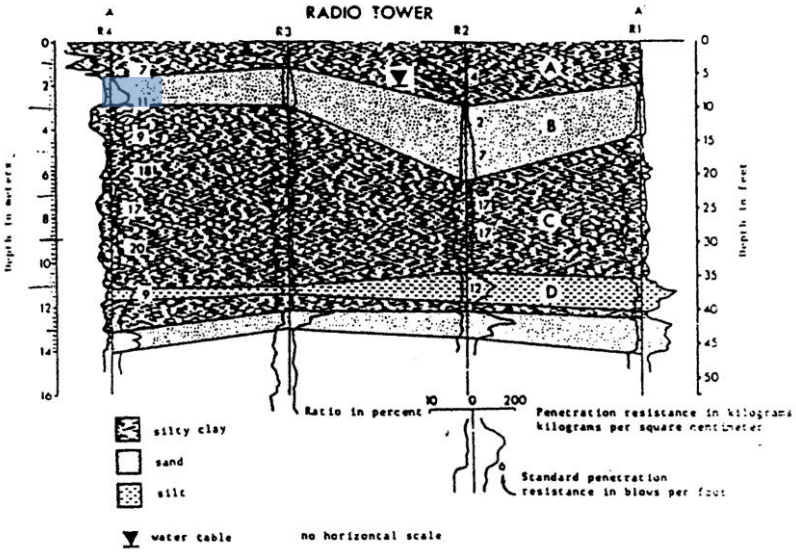
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.8		2	1.55	0.82	***2.9***
5.1	16.7		7	1.33	0.89	***9.5***
					Mean:	6.2
					St. Dev.	4.7

Depth (m)	FC	sand	silt	clay	D50
3.3	***64.0***	***36.0***	***59.0***	***5.0***	***0.047***
5.1	***23.0***	***77.0***	***22.0***	***1.0***	***0.100***
23.15	*18.0*	*82.0*	*13.0*	*5.0*	*0.125*
	43.5	56.5	40.5	3.0	0.1
	29.0	29.0	26.2	2.8	0.0

Case number:	82
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Radio Tower B2
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	Liquefied during both the 1979 and 1981 earthquakes. Several sandboils expelled considerable volumes of water and sand and created a new pond near the radio tower at the site. Small fissures developed around the pond and larger fissures developed at the margin of the river flood plane locally in the adjacent bluffs. This borelog is from the "nonliquefied" section
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand Radio Tower Site is 12 km from the northern terminus of the fault rupture. 0.22g and 0.16 of PGA is recorded at BRA recording site which is 9-10 km from the northern terminus of the fault rupture. The earthquake specific PGA contours estimated a PGA of ~ 0.2 g at the River site. Based on site response analysis results a PGA value of 0.18 g is adopted Energy ratio ~68 %

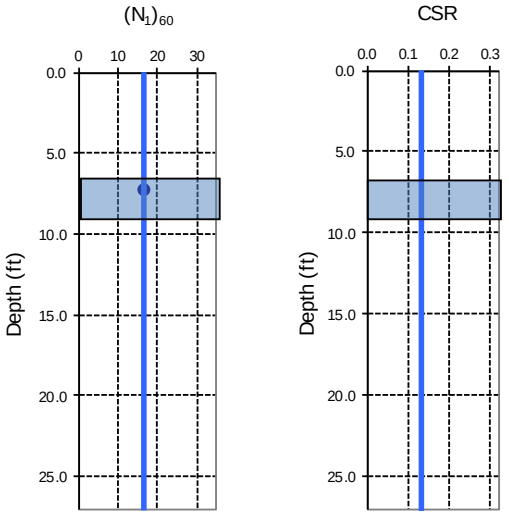
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.100 ± 0.050		0.1
Data Class	B			% Fines	18.0 ± 2.0	30.0	30
Critical Depth Range	6.6 - 9.8	7.5	7.5	% PI			
Depth to GWT (ft)	6.6	6.9	7.0				
σ_v (psf)	852.8 ± 67.4	835.4	830.0	N	11.3 ± 1.4	11.0	11.0
σ_v' (psf)	750.5 ± 39.0	793.6	800.0	C_R	0.77	0.80	0.8
a_{max} (g)	0.180 ± 0.027	0.20	0.200	C_S	1.00	1.00	
r_d	0.99 ± 0.020	0.98	0.980	C_B	1.00	1.00	
CSR	0.132 ± 0.021	0.132	0.130	C_E	1.13	1.13	1.050
Equivalent Magnitude	6.53	6.53		C_N	1.68	1.53	1.5
MSF		1.29	1.13	$(N_1)_{60}$	16.6 ± 2.0	15.2	13.00
CSRN		0.093	0.12				



γ_m=100±3 pcf

γ_s=120±3 pcf



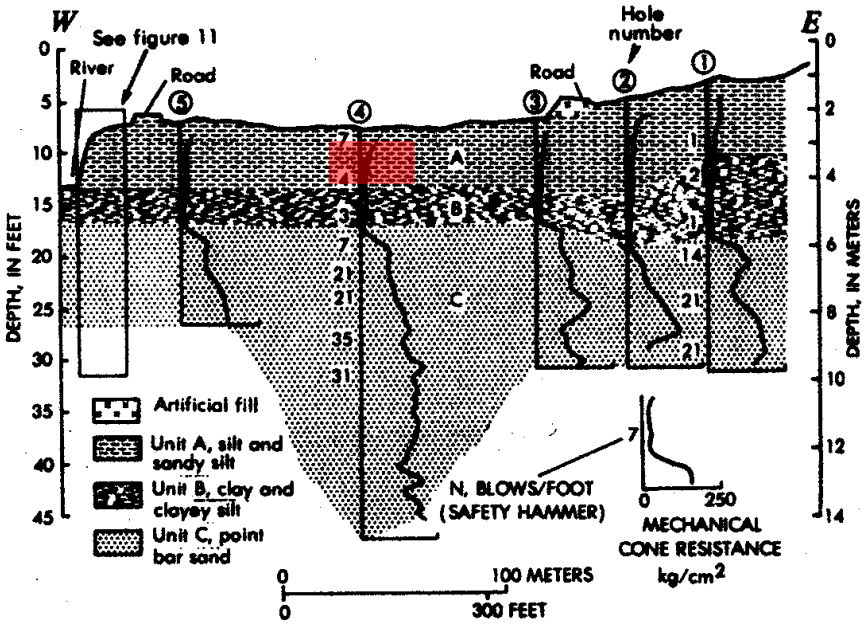
CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	silt	clay	D50
2.3	7.3		11	1.74	0.76	***16.6***	***18.0***	***17.0***	***1.0***	***0.10***
					Mean:	16.6				
					St. Dev.	2.0	assigned			

Case number:	83
Earthquake:	1979 Imperial Valley
Magnitude:	6.53 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	River Park A
References:	Youd et al. (1983), "Liquefaction Sites, Imperial Valley, CA
Nature of Failure:	Hundreds of sand boils, a slump, and numerous small ground cracks. Water continued to seep from a few sand boils for two weeks.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. River Park Site is 3.4 km from the northern terminus of the fault rupture.PGA of 0.22-0.16g were recorded at Brawley Airport recording site which is 5.9 km from the northern terminus of the fault rupture. Based on the earthquake specific PGA contours, the estimated PGA at the River site is ~0.2 g. The directivity and the basin effects are significant. Contrary, site response analyses results suggest a lower value (probably due to significant strain softening) ~0.16 g ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T Driver and safety hammer. Energy ratio ~68 % SPT blowcounts were taken in 1979 and 1981.

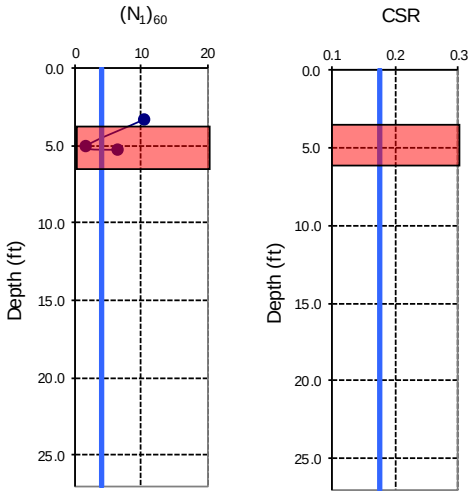
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.010 ± 0.050		0.04
Data Class	B			% Fines	91.0 ± 2.0	80.0	80
Critical Depth Range	1.0 - 5.9	5.9	6.0	% PI			
Depth to GWT (ft)	1.0	1.0	1.0				
σ_v (psf)	369.8 ± 90.3	731.0	735.0	N	2.7 ± 2.3	3.0	3.0
σ_v' (psf)	216.8 ± 42.7	417.7	425.0	C_R	0.66	0.80	0.8
a_{max} (g)	0.160 ± 0.024	0.24	0.240	C_S	1.00	1.00	
r_d	0.99 ± 0.015	0.98	0.990	C_B	1.00	1.00	
CSR	0.176 ± 0.032	0.266	0.265	C_E	1.13	1.13	1.050
Equivalent Magnitude	6.5	6.5		C_N	2.00	1.70	1.9
MSF		1.29	1.15	$(N_1)_{60}$	4.0 ± 3.4	4.6	4.50
CSRN		0.187	0.23				



γ_m=100±3 pcf

γ_s=110±3 pcf



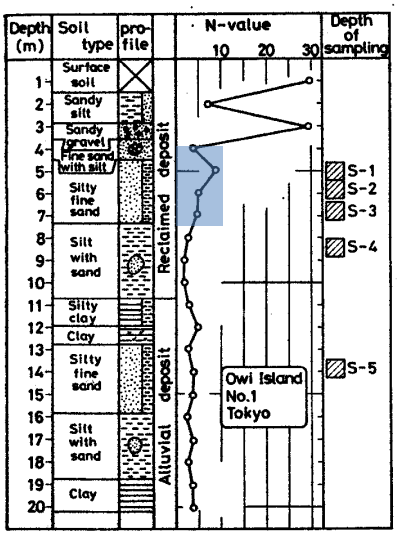
CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		FC	sand	silt	clay	D50
1.00	3.3		7	2.00	0.66	10.4	BH-4 (1979)	-	-	-	-	0.071
1.52	5.0		1	2.00	0.71	***1.6***	BH-15 (1981)	***91.0***	***9.0***	***74.0***	***17.0***	***0.010***
1.60	5.2		4	2.00	0.71	***6.5***	BH-4 (1979)	-	-	-	-	-
					Mean:	4.0		91.0	9.0	74.0	17.0	0.010
					St. Dev.	3.4		2.0	-	-	-	0.050

Case number:	84
Earthquake:	1980 Mid-Chiba
Magnitude:	6.1
Location:	Owi-1
References:	Ishihara et al (1981) Fear et al. (1995)
Nature of Failure:	No significant pore pressure generation
Comments:	Owi Island No 1 is located on the west side of Tokyo Bay. The island was reclaimed from 1961 to 1969 with soils dredged fom the nearby seabed to open a navigation route.Soils with different grain sizes were transported in slurry through pipes and discharged in the area. When the soil surface reached the sea level, several kinds of waste soils were dumped on top of the dredged fill. and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively Two set of piezometers were embedded at two depths, I and II were installed 5m apart at depths of 6 and 13 m. III and IV were also embedded 5 m apart at depths 6 and 14 m parallel to, and 5 m from the first set. A seismograph was installed at the surface. A PGA of 45 gal due to a small magnitude EQ Mw ~4-5 has not triggered the pore pressure transducers. Piezometers I and II were destroyed during the earthquake. The epicenter was located 15 km southeast of the City of Chiba, and had a focal depth of ~20 km. The intensity of shaking sorrounding the Tokyo Bay was V (JMAS). PGA was recorded as 95 and 65 gal respectively in two normal directions Pore pressure started to increase after the PGA exceeded 60 gal and excess pore pressure reached to the maximum values of 0.75 and 1.32 m at depths 6 and 14 m. 50 % of the excess pore pressure dissipated within 10 seconds. No excess pore pressure remained at depth 14 m (FC=27) after 2 hours. Seed et al estimated the CSR that will trigger the liquefaction based on laboratory test results on frozen samples. It is believed that he used number of cycles of 5-6 to represent a M=6.1 earthquake. CSR that will cause liquefaction in 5-6 cycles was predicted for depths 4.9 and 13.9 as 0.37 and 0.35 respectively. based on lab data. The CSR estimated due to earthquake was estimated as 0.232 and 0.224. So he predicted that a PGA of 0.145g (~1.5x0.95) would cause liquefaction. CSR estimates were not multiplied by 0.65 SPT values were taken before the earthquake. SPT energy is estimated as 65 % by Seed et al. (84)

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.120 ± 0.026		0.18
Data Class	A			% Fines	30.0 ± 16.5	13.0	13
Critical Depth Range	13.1 - 23.0	20.0	20.0	% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.0	3.0				
σ_v (psf)	2165.4 ± 202.0	2255.6	2260.0	N	5.8 ± 2.3	5.0	5.0
σ_v' (psf)	1244.1 ± 106.5	1190.5	1200.0	C_R	0.94	0.95	
a_{max} (g)	0.079 ± 0.004	0.10	0.100	C_S	1.00	1.00	
r_d	0.81 ± 0.076	0.89	0.960	C_B	1.15	1.00	
CSR	0.072 ± 0.008	0.104	0.120	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.1	6.0		C_N	1.30	1.36	1.3
MSF		1.48	1.26	$(N_1)_{60}$	8.9 ± 3.5	7.1	7.00
CSRN		0.067	0.10				



γ_m=120±3 pcf

γ_s=120±3 pcf

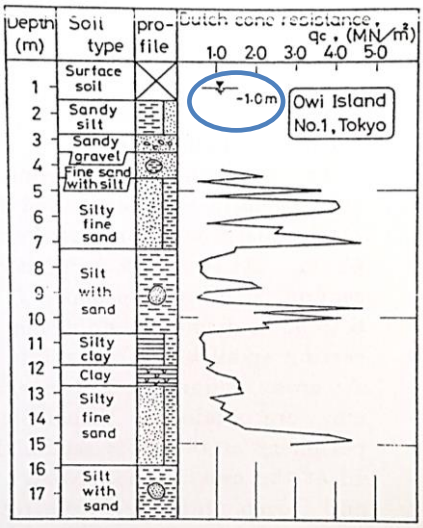
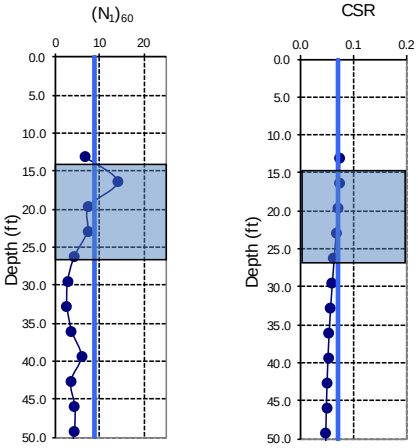


Fig. 9. Dutch cone penetration test results at the test site



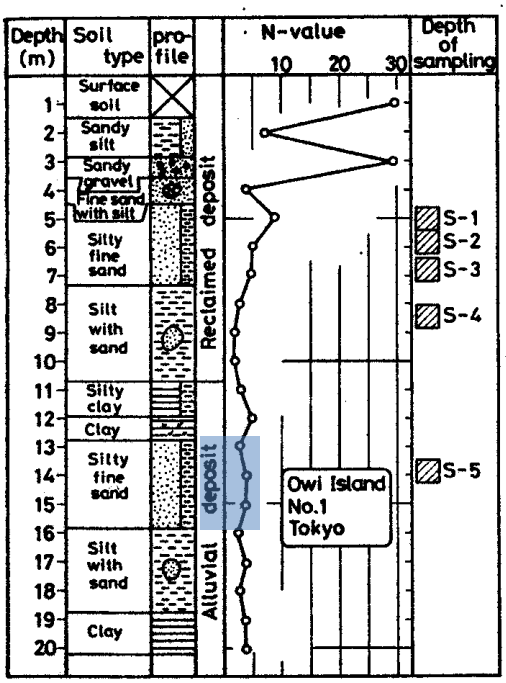
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4	13.1	0.074	4	1.48	0.89	*6.6*
5	16.4	0.073	9	1.36	0.92	*14.1*
6	19.7	0.070	5	1.26	0.95	*7.5*
7	23.0	0.067	5	1.18	0.98	*7.2*
8	26.2	0.064	3	1.11	1.00	4.2
9	29.5	0.060	2	1.05	1.00	2.6
10	32.8	0.057	2	1.01	1.00	2.5
11	36.1	0.054	3	0.96	1.00	3.6
12	39.4	0.052	5	0.93	1.00	5.8
13	42.7	0.050	3	0.89	1.00	3.4
14	45.9	0.049	4	0.86	1.00	4.3
15	49.2	0.048	4	0.83	1.00	4.2
16.0	52.5	0.048	3	0.81	1.00	3.0
					Mean:	8.9
					St. Dev.	3.5

	d (m)	D_{10} (mm)	D_{50} (mm)	D_{60} (mm)	FC %
	4.9	*-*	*0.090*	*0.120*	*47.0*
	5.8	*0.045*	*0.140*	*0.160*	*14.0*
	6.7	*0.070*	*0.130*	*0.160*	*29.0*
Mean:	5.8	0.058	0.120	0.147	30.0
St. Dev.	0.9	0.035	0.026	0.023	16.5

Case number:	85
Earthquake:	1980 Mid-Chiba
Magnitude:	6.1
Location:	Owi-2
References:	Ishihara et al (1981) Fear et al. (1995)
Nature of Failure:	No significant pore pressure generation
Comments:	Owi Island is located on the west side of Tokyo Bay. The island was reclaimed from 1961 to 1969 with soils dredged fom the nearby seabed to open a navigation route.Soils with different grain sizes were transported in slurry through pipes and discharged in the area. When the soil surface reached the sea level, several kinds of waste soils were dumped on top of the dredged fill. and alluvial deposits to 200-300'. In the vicinity of river the ground is relatively Two set of piezometers were embedded at two depths, I and II were installed 5m apart at depths of 6 and 13 m. III and IV were also embedded 5 m apart at depths 6 and 14 m parallel to, and 5 m from the first set. A seismograph was installed at the surface. A PGA of 45 gal due to a small magnitude EQ Mw ~4-5 has not triggered the pore pressure transducers. Piezometers I and II were destroyed during the earthquake. The epicenter was located 15 km southeast of the City of Chiba, and had a focal depth of ~20 km. The intensity of shaking sorrounding the Tokyo Bay was V JMAS PGA was recorded as 95 and 65 gal respectively in two normal directions Pore pressure started to increase after the PGA exceeded 60 gal and excess pore pressure reached to the maximum values of 0.75 and 1.32 m at depths 6 and 14 m. 50 % of the excess pore pressure dissipated within 10 seconds. No excess pore pressure remained at depth 14 m (FC=27) after 2 hours. Seed et al estimated the CSR that will trigger the liquefaction based on laboratory test results of frozen samples. It is believed that he used number of cycles of 5-6 to represent a M=6.1 earthquake. CSR that will cause liquefaction in 5-6 cycles is predicted for depths 4.9 and 13.9 as 0.37 and 0.35 respectively. based on lab data. The CSR estimated due to earthquake was estimated as 0.232 and 0.224. So he predicted that a PGA of 0.145g (~1.5x0.95) would cause liquefaction. CSR estimates were not multiplied by 0.65 SPT values were taken before the earthquake. SPT energy is estimated as 65 % by Seed et al. (84)

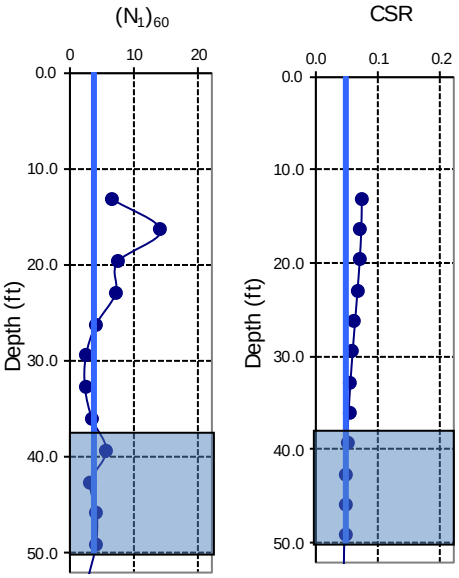
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.170 ± 0.050		0.17
Data Class	B			% Fines	27.0 ± 2.0	27.0	27
Critical Depth Range	42.7 - 52.5	46.9	47.0	% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.0	3.0				
σ_v (psf)	5708.7 ± 237.7	5304.9	5310.0	N	3.5 ± 0.6	4.0	4.0
σ_v' (psf)	2944.9 ± 164.4	2568.9	2560.0	C_R	1.00	1.00	
a_{max} (g)	0.079 ± 0.004	0.10	0.100	C_S	1.00	1.00	
r_d	0.49 ± 0.149	0.69	0.790	C_B	1.15	1.00	
CSR	0.049 ± 0.015	0.089	0.105	C_E	1.09	1.09	1.090
Equivalent Magnitude	6.1	6.0		C_N	0.85	0.90	0.9
MSF		1.48	1.24	$(N_1)_{60}$	3.7 ± 0.6	3.9	4.00
CSRN		0.061	0.09				



γ_m=120±3 pcf

γ_s=120±3 pcf

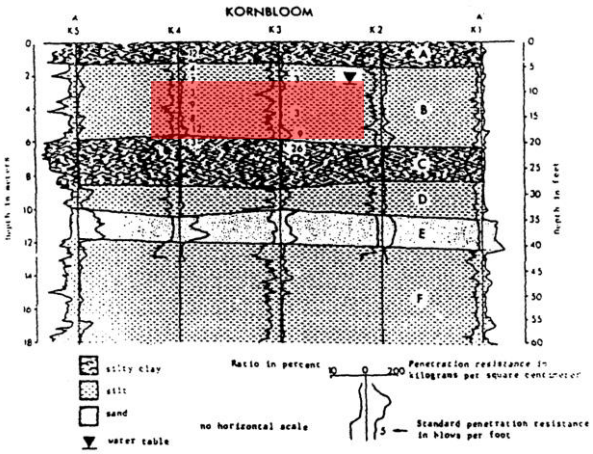


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4	13.1	0.074	4	1.48	0.89	6.6
5	16.4	0.073	9	1.36	0.92	14.1
6	19.7	0.070	5	1.26	0.95	7.5
7	23.0	0.067	5	1.18	0.98	7.2
8	26.2	0.064	3	1.11	1.00	4.2
9	29.5	0.060	2	1.05	1.00	2.6
10	32.8	0.057	2	1.01	1.00	2.5
11	36.1	0.054	3	0.96	1.00	3.6
12	39.4	0.052	5	0.93	1.00	5.8
13	42.7	0.050	3	0.89	1.00	***3.4***
14	45.9	0.049	4	0.86	1.00	***4.3***
15	49.2	0.048	4	0.83	1.00	***4.2***
16.0	52.5	0.048	3	0.81	1.00	***3.0***
					Mean:	3.7
					St. Dev.	0.6

Case number:	86
Earthquake:	1981 WestMorland
Magnitude:	5.6 (M_L)
Location:	Kornbloom B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	More than one hundred sand boils erupted at this site. The boils are widely scattered but generally occurred in poorly defined bands. Some of the boils erupted along short ground fissures. The unique aspect of the Kornbloom site is that the liquefied sediments were primarily silty material. …the extent of the liquefiable silt layer is from 5-22'.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: silty clay, stiff-lacustrine and/or flood plain deposit B: silty and sandy silt-fluvial and perhaps fluvial deltaic deposition C: silty clay to clay, stiff-lacustrine and/or flood plain deposit D: interbedded silt and sand-loose to medium dense E: Uniform sand with some silt-medium dense to dense F: interbedded silt and sand - loose to medium dense. D-F :fluvial and perhaps fluvial-deltaic deposition PGA was estimated based on earthquake specific attenuation relationship for alluvium sites as min. 0.24, mean 0.28 and max 0.46 g. at the site. Based on site specific response analyses, PGA was estimated as 0.19 g Energy ratio ~68 %

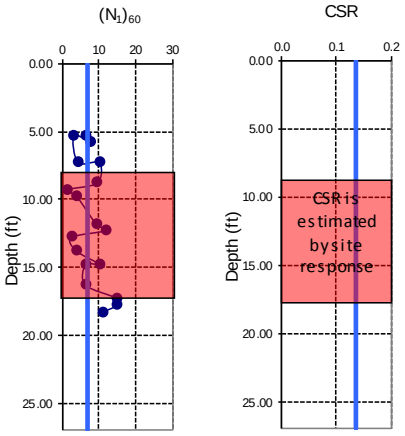
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.045 ± 0.008		0.045
Data Class	A			% Fines	83.0 ± 9.6	92.0	92
Critical Depth Range	9.0 - 17.5	14.1	14.0	% PI			
Depth to GWT (ft)	9.0	8.9	9.0				
σ_v (psf)	1410.0 ± 172.7	1608.2	1615.0	N	5.4 ± 3.1	5.0	5.0
σ_v' (psf)	1144.8 ± 88.1	1294.9	1305.0	C_R	0.85	0.85	
a_{max} (g)	0.190 ± 0.029	0.32	0.320	C_S	1.00	1.00	
r_d	0.90 ± 0.012	0.93	0.970	C_B	1.00	1.00	1.050
CSR	0.137 ± 0.022	0.240	0.260	C_E	1.13	1.13	
Equivalent Magnitude	5.9	5.9		C_N	1.36	1.29	1.2
MSF		1.52	1.41	$(N_1)_{60}$	7.1 ± 4.1	6.2	6.50
CSRN		0.151	0.19				



γ_m=100±3 pcf

γ_s=120±3 pcf

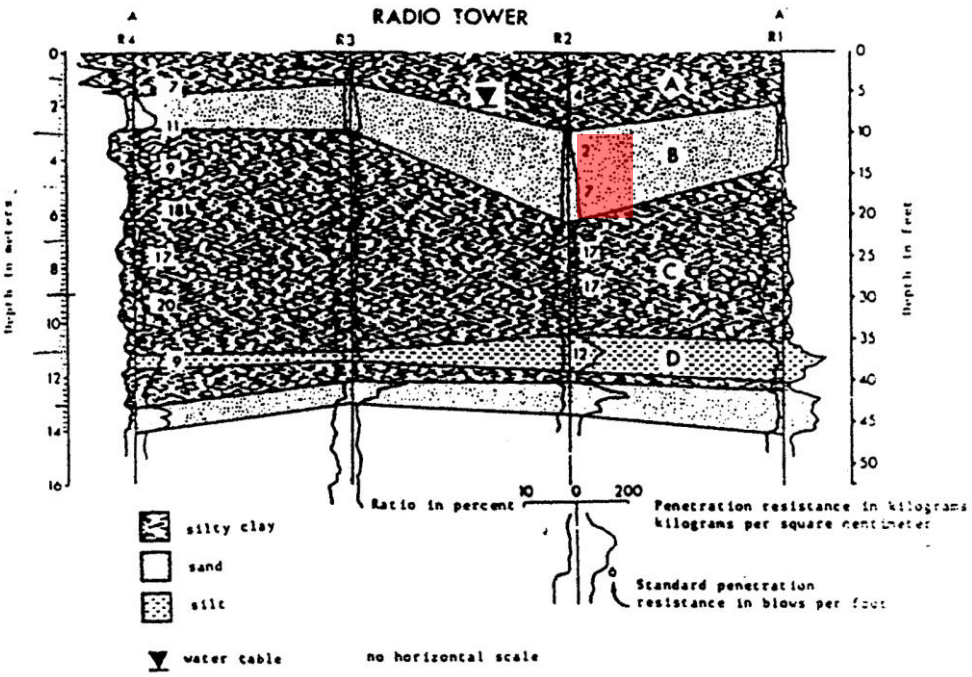


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		FC	sand	silt	clay	D50	WN	WL	PI	LI
1.60	5.25		4	2.00	0.71	6.5	K-4	88.0	12.0	65.0	23.0	0.019	26.0	29.0	11.0	0.7
1.75	5.75		5	1.92	0.72	7.9	K-4 b	84.0	16.0	74.0	10.0	0.038				
1.60	5.25		2	2.00	0.71	***3.2***	SK4-5	-	-	-	-	-				
2.21	7.25		3	1.71	0.76	***4.4***	K-3	***80.0***	***21.0***	***73.0***	***7.0***	***0.044***				
2.21	7.25		7	1.71	0.76	***10.2***	K-4	***74.0***	***26.0***	***68.0***	***6.0***	***0.052***				
2.67	8.75		7	1.56	0.78	***9.7***	K-4 b	***86.0***	***14.0***	***83.0***	***3.0***	***0.049***				
2.82	9.25		1	1.52	0.79	***1.4***	SK4-5	-	-	-	-	-				
2.97	9.75		3	1.50	0.80	***4.1***	K-4	***62.0***	***38.0***	***53.0***	***9.0***	***0.056***	***28.0***			
3.58	11.75		7	1.41	0.83	***9.3***	K-4 b	***88.0***	***12.0***	***79.0***	***9.0***	***0.042***				
3.73	12.25		9	1.40	0.84	***11.9***	K-4	***96.0***	***4.0***	***83.0***	***13.0***	***0.028***	***30.0***			
3.89	12.75		2	1.38	0.85	***2.6***	SK4-5	-	-	-	-	-				
4.19	13.75		3	1.34	0.86	***3.9***	K-3	***91.0***	***9.0***	***79.0***	***12.0***	***0.039***				
4.50	14.75		8	1.31	0.87	***10.4***	K-4	***83.0***	***17.0***	***74.0***	***9.0***	***0.047***	***29.0***			
4.50	14.75		5	1.31	0.87	***6.5***	K-4 b	***81.0***	***19.0***	***71.0***	***10.0***	***0.042***				
4.95	16.25		5	1.27	0.89	***6.4***	SK4-5	-	-	-	-	-				
5.26	17.25		12	1.24	0.90	***15.2***	K-4	***89.0***	***11.0***	***83.0***	***6.0***	***0.048***				
5.41	17.75		12	1.23	0.90	15.1	K-4 b	44.0	56.0	40.0	4.0	0.700				
5.56	18.25		9	1.22	0.91	11.3	K-3	30.0	70.0	29.0	1.0	0.082				
					Mean:	7.1		83.0	17.1	74.6	8.4	0.045				
					St. Dev.	4.1		9.6	9.7	9.3	3.0	0.008				

Case number:	87
Earthquake:	Westmorland
Magnitude:	5.6 (M_L)
Location:	Radio Tower B1
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	Liquefied during both the 1979 and 1981 earthquakes. Several sandboils expelled considerable volumes of water and sand and created a new pond near the radio tower at the site. Small fissures devepoed around the pond and larger fissures developed at the margin of the river flood plane locally in the adjacent bluffs.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand Radio Tower Site is 10 km from the epicenter of the earthquake. At Brawley seismographic station which is ~ 4 km from the River Park site and at an epicentral distance of 16 km, PGA was recorded as 0.17 and 0.15g (deep alluvium site). Based on earthquake specific attenuation relationships the mean and standard deviation of PGA were estimated as 0.18 ± 0.025 PGA was estimated as ~ 0.20 g by Seed et al (1984) Based on site response analyses results, PGA was estimated as 0.17 g Energy ratio ~68 %

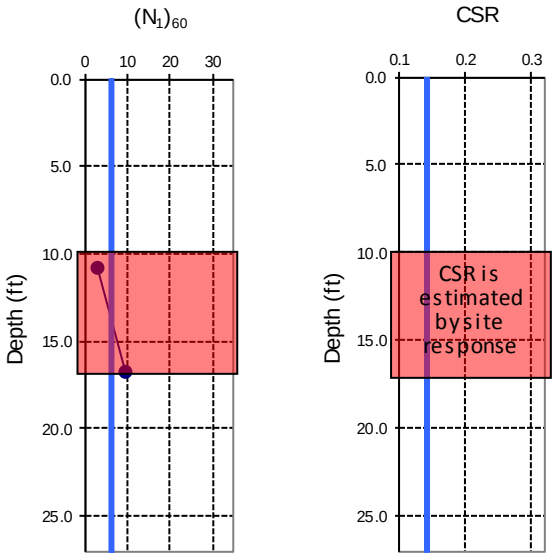
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.074 ± 0.037		0.047
Data Class	B			% Fines	43.5 ± 29.0	64.0	75
Critical Depth Range	9.8 - 18.0	11.2	11.0	% PI			
Depth to GWT (ft)	6.6	6.9	7.0				
σ_v (psf)	1505.1 ± 160.1	1294.9	1270.0	N	4.5 ± 3.4	2.0	2.0
σ_v' (psf)	1044.5 ± 79.5	1044.3	1020.0	C_R	0.86	0.85	
a_{max} (g)	0.170 ± 0.026	0.20	0.200	C_S	1.00	1.00	1.050
r_d	0.89 ± 0.012	0.95	0.960	C_B	1.00	1.00	
CSR	0.142 ± 0.022	0.152	0.160	C_E	1.13	1.13	
Equivalent Magnitude	5.9	5.9		C_N	1.42	1.49	1.4
MSF		1.52	1.45	$(N_1)_{60}$	6.2 ± 4.7	2.9	3.00
CSRN		0.094	0.11				



γ_m=100±3 pcf

γ_s=115±3 pcf



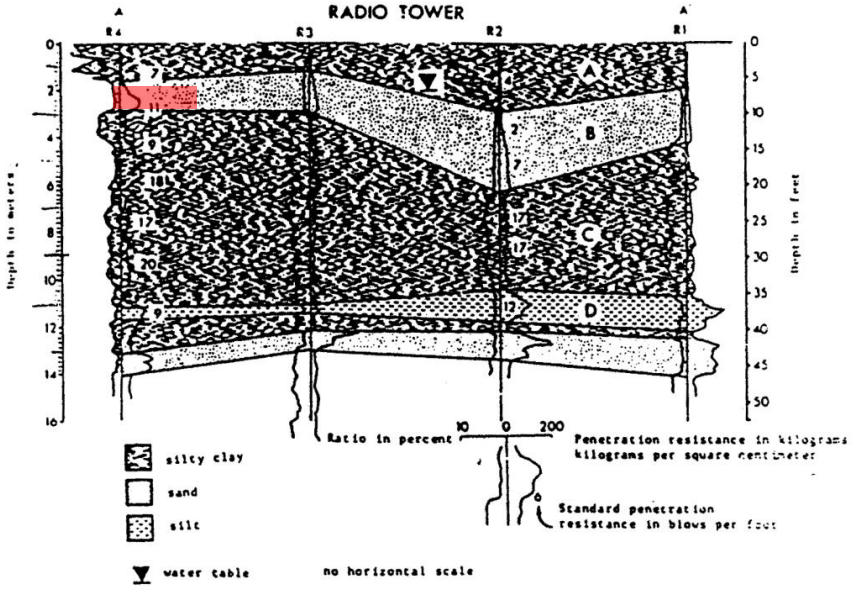
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.8		2	1.55	0.82	***2.9***
5.1	16.7		7	1.33	0.89	***9.5***
					Mean:	6.2
					St. Dev.	4.7

Depth (m)	FC	sand	silt	clay	D50
3.3	***64.0***	***36.0***	***59.0***	***5.0***	***0.047***
5.1	***23.0***	***77.0***	***22.0***	***1.0***	***0.100***
23.2	*18.0*	*82.0*	*13.0*	*5.0*	*0.125*
	43.5	56.5	40.5	3.0	0.074
	29.0	29.0	26.2	2.8	0.037

Case number:	88
Earthquake:	Westmorland
Magnitude:	5.6 (M_L)
Location:	Radio Tower B2
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	Liquefied during both the 1979 and 1981 earthquakes. Several sandboils expelled considerable volumes of water and sand and created a new pond near the radio tower at the site. Small fissures devepoed around the pond and larger fissures developed at the margin of the river flood plane locally in the adjacent bluffs. This borelog is from the "nonliquefied" section
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand Radio Tower Site is 10 km from the epicenter of the earthquake. At Brawley seismographic station which is ~ 4 km from the River Park site and at an epicentral distance of 16 km, PGA was recorded as 0.17and 0.15g (deep alluvium site). Based on earthquake specific attenuation relationships the mean and standard deviation of PGA were estimated as 0.18 ± 0.025 PGA was estimated by Seed et al (1984) ~0.20 g Based on site response analyses results, PGA was estimated as 0.16 g. Energy ratio ~68 %

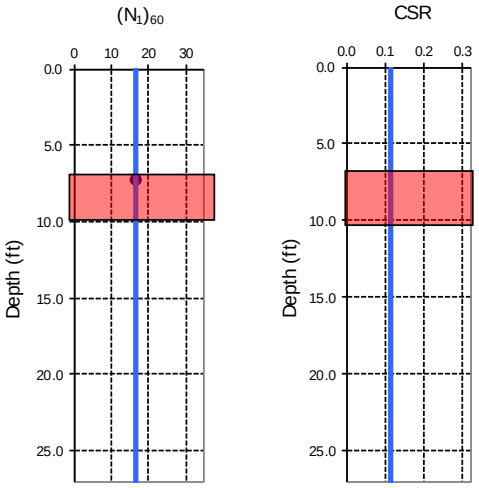
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.100 ± 0.050		0.1
Data Class	B			% Fines	18.0 ± 2.0	30.0	30
Critical Depth Range	6.6 - 9.8	7.5	7.5	% PI			
Depth to GWT (ft)	6.6	6.9	7.0				
σ_v (psf)	852.8 ± 67.4	835.4	830.0	N	11.3 ± 1.4	11.0	11.0
σ_v' (psf)	750.5 ± 39.0	793.6	800.0	C_R	0.77	0.80	0.8
a_{max} (g)	0.160 ± 0.024	0.20	0.200	C_S	1.00	1.00	
r_d	0.98 ± 0.010	0.97	0.980	C_B	1.00	1.00	
CSR	0.116 ± 0.018	0.131	0.130	C_E	1.13	1.13	1.050
Equivalent Magnitude	5.9	5.9		C_N	1.68	1.53	1.5
MSF		1.52	1.37	$(N_1)_{60}$	16.6 ± 2.0	15.2	13.00
CSRN		0.078	0.10				



γ_m=100±3 pcf

γ_s=120±3 pcf



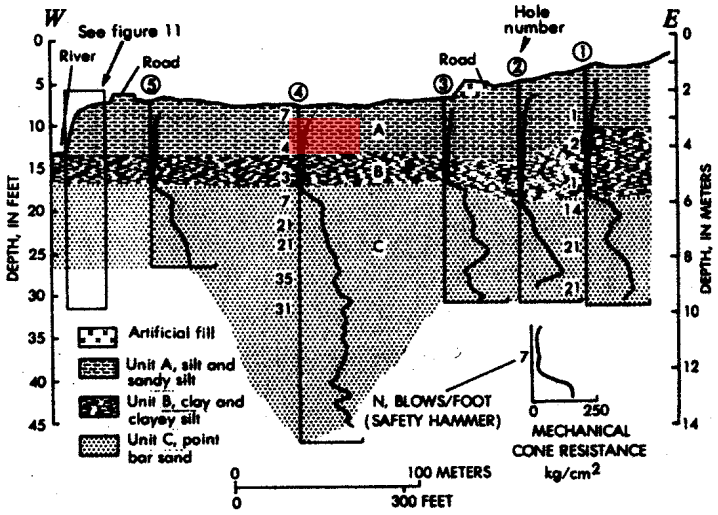
CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	silt	clay	D50
2.3	7.3		11	1.74	0.76	***16.6***	***18.0***	***17.0***	***1.0***	***0.1***
					Mean:	16.6				
					St. Dev.	2.0	assigned			

Case number:	89
Earthquake:	Westmorland
Magnitude:	5.6 (M_L)
Location:	River Park A
References:	Youd et al. (1983), "Liquefaction sites, Imperial Valley, CA
Nature of Failure:	Hundreds of sand boils, a slump, and numerous small ground cracks. Water continued to seep from a few sand boils for two weeks.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. River Park Site is 14 km from the epicenter of the earthquake. At Brawley seismographic station which is ~ 4 km from the River Park site and at an epicentral distance of 16 km, PGA was recorded as 0.17 and 0.15g (deep alluvil site). PGA was estimated by Seed et al (1984) ~0.21 g Based on site response analyses results, PGA was estimated as 0.17 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T Driver and safety hammer. Energy ratio ~68 % SPT blowcounts were taken in 1979 and 1981.

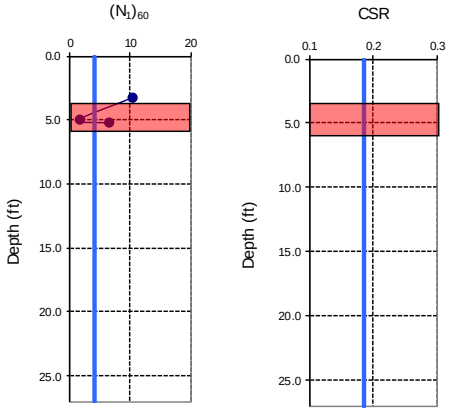
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.010 ± 0.050		0.04
Data Class	B			% Fines	91.0 ± 2.0	80.0	80
Critical Depth Range	1.0 - 5.9	5.9	6.0	% PI			
Depth to GWT (ft)	1.0	1.0	1.0				
σ_v (psf)	369.8 ± 90.3	731.0	735.0	N	2.7 ± 2.3	3.0	3.0
σ_v' (psf)	216.8 ± 42.7	417.7	425.0	C_R	0.66	0.80	0.8
a_{max} (g)	0.170 ± 0.026	0.21	0.210	C_S	1.00	1.00	
r_d	0.99 ± 0.003	0.98	0.990	C_B	1.00	1.00	
CSR	0.187 ± 0.034	0.231	0.235	C_E	1.13	1.13	1.050
Equivalent Magnitude	5.9	5.9		C_N	2.00	1.70	1.9
MSF		1.52	1.42	$(N_1)_{60}$	4.0 ± 3.4	4.6	4.50
CSRN		0.138	0.17				



γ_m=100±3 pcf

γ_s=110±3 pcf



CSR is estimated by site response

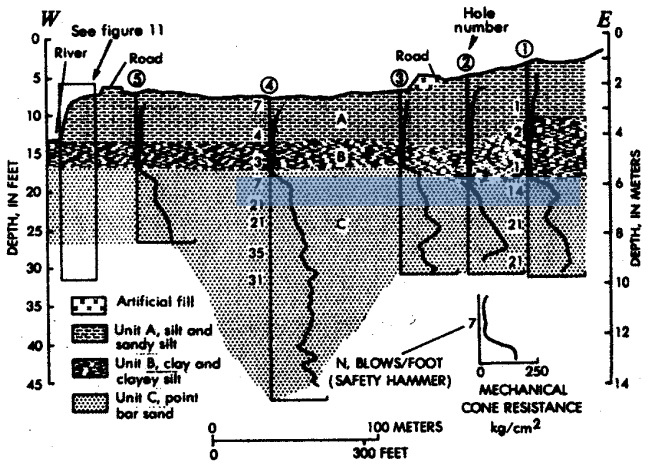
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.0	3.3		7	2.00	0.66	10.4	BH-4 (1979)
1.5	5.0		1	2.00	0.71	***1.6***	BH-15 (1981)
1.6	5.2		4	2.00	0.71	***6.5***	BH-4 (1979)
					Mean:	4.0	
					St. Dev.	3.4	

FC	sand	silt	clay	D50
-	-	-	-	0.071
91.0	***9.0***	***74.0***	***17.0***	***0.010***
-	-	-	-	-
91.0	9.0	74.0	17.0	0.010
2.0	-	-	-	0.050

Case number:	90
Earthquake:	Westmorland
Magnitude:	5.6 (M_L)
Location:	River Park C
References:	Youd et al. (1983), "Liquefaction sites, Imperial Valley, CA
Nature of Failure:	Hundreds of sand boils, a slump, and numerous small ground cracks. Water continued to seep from a few sand boils for two weeks.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. River Park Site is 14 km from the epicenter of the earthquake. At Brawley seismographic station which is ~ 4 km from the River Park site and at an epicentral distance of 16 km, PGA was recorded as 0.17 g (deep allivual site) . PGA was estimated by Seed et al (1984) ~0.21 g.Site response analysis results suggest a PGA value of 0.17 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T Driver and safety hammer. Energy ratio ~68 % SPT blowcounts were taken in 1979 and 1981.

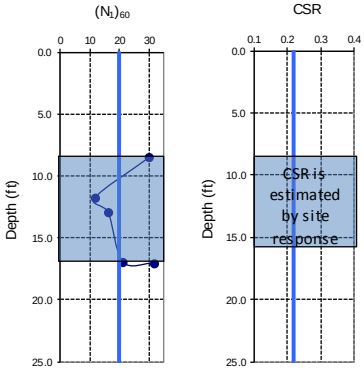
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.142 ± 0.020		0.15
Data Class	A			% Fines	13.2 ± 4.4	18.0	18
Critical Depth Range	11.0 - 17.0	14.1	14.0	% PI			
Depth to GWT (ft)	1.0	1.0	1.0				
σ_v (psf)	1595.0 ± 121.6	1733.5	1735.0	N	12.4 ± 4.9	11.0	11.0
σ_v' (psf)	783.8 ± 67.6	939.8	930.0	C_R	0.86	0.85	
a_{max} (g)	0.170 ± 0.026	0.21	0.210	C_S	1.00	1.00	
r_d	0.97 ± 0.010	0.93	0.960	C_B	1.00	1.00	
CSR	0.218 ± 0.034	0.237	0.245	C_E	1.13	1.13	1.050
Equivalent Magnitude	5.9	5.9		C_N	1.64	1.44	1.4
MSF		1.52	1.40	$(N_1)_{60}$	19.9 ± 7.8	15.2	16.00
CSRN		0.142	0.18				



γ_m=100±3 pcf

γ_s=115±3 pcf



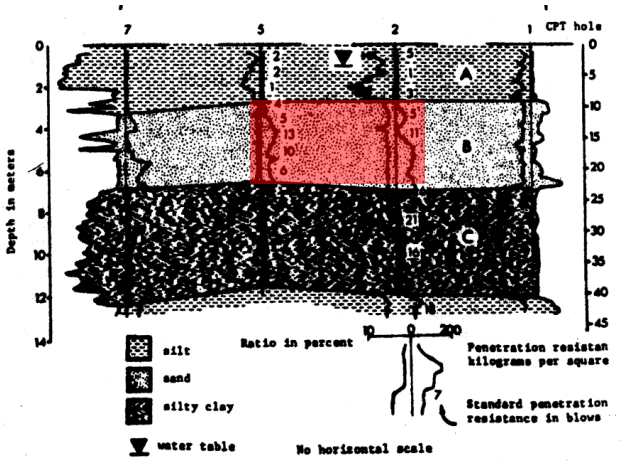
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.6	8.5		17	2.00	0.78	***30.1***	BH-13
3.6	11.8		7	1.78	0.83	***11.8***	BH-4
3.96	13.0		10	1.70	0.85	***16.4***	BH-6
5.18	17.0		14	1.50	0.90	***21.3***	BH-1
5.2	17.1		21	1.50	0.90	32.0	BH-4
					Mean:	19.9	
					St. Dev.	7.8	

Depth (m)	FC	sand	silt	clay	D50	
3.3	**12**	**88**	**9**	**3**	**0.110**	BH-4
3.5	**13**	**87**	**11**	**2**	**0.120**	BH-4
3.9	**11**	**88**	**9**	**2**	**0.135**	BH-4
4.4	**11**	**89**	**10**	**1**	**0.140**	BH-4
4.6	**10**	**90**	**9**	**1**	**0.145**	BH-4
5.1	**10**	**90**	**10**	**0**	**0.145**	BH-4
5.2	**9**	**91**	**9**	**0**	**0.180**	BH-1
5.2	**14**	**86**	**14**	**0**	**0.170**	BH-1
5.3	**13**	**87**	**10**	**3**	**0.145**	BH-4
5.7	**18**	**82**	**14**	**4**	**0.140**	BH-4
6.0	**24**	**76**	**20**	**4**	**0.135**	BH-4
6.4	7	93	7	0	0.115	BH-1
7.8	6	94	6	0	0.145	BH-1
Mean:	13.2	86.7	11.4	1.8	0.142	
St. Dev.	4.4	4.3	3.4	1.5	0.020	

Case number:	91
Earthquake:	1981 WestMorland
Magnitude:	5.6 (M_L)
Location:	Wildlife B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	Numerous sand boils, fissures, and slumped banks occurred along both sides of a 3 km long stretch of Alamo River within the Wildlife Management Area. Sand boils and fissures extended as far as 100 m from the river.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very loose and very soft interbedded micaceous sandy silt, silt and clayey silts (0-2.5 m) flood plain deposits B: very loose to loose moderately sorted sandy silt (2.5-6.8m); point bar deposits C: silty clay (medium to very stiff) and clayey silt (medium to stiff); lacustrine deposit. PGA was recorded at a one-story building underlain by deep alluvium as 0.21 g at epicentral distance of 9.2 km. There is no surface faulting. The site is located ~9.5 km from the epicenter. Also at WLF station PGA was recorded as 0.19 and 0.18 g. Based on site response analyses results, PGA was estimated as as 0.23 g. Energy ratio ~68 %

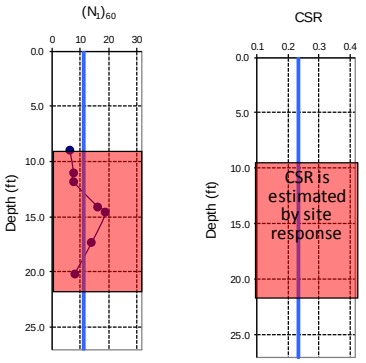
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.109 ± 0.038		0.09
Data Class	A			% Fines	26.2 ± 7.6	30.0	40
Critical Depth Range	9.0 - 22.0	15.1	16.0	% PI			
Depth to GWT (ft)	3.0	3.9	3.0				
σ_v (psf)	1800.0 ± 262.9	1817.0	1960.0	N	7.9 ± 3.5	7.1	9.0
σ_v' (psf)	1020.0 ± 131.5	1127.8	1150.0	C_R	0.88	0.95	
a_{max} (g)	0.230 ± 0.012	0.26	0.260	C_S	1.00	1.00	
r_d	0.89 ± 0.013	0.92	0.970	C_B	1.00	1.00	1.050
CSR	0.235 ± 0.014	0.267	0.280	C_E	1.13	1.13	
Equivalent Magnitude	5.9	5.9		C_N	1.44	1.36	1.3
MSF		1.52	1.40	$(N_1)_{60}$	11.3 ± 5.0	10.3	11.50
CSRN		0.164	0.20				



γ_m=100±3 pcf

γ_s=120±3 pcf



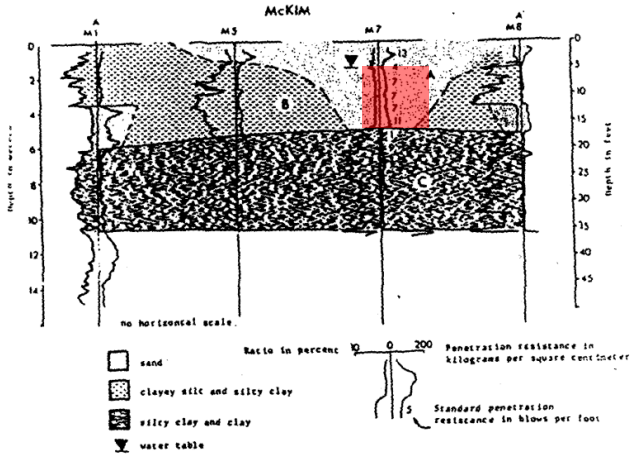
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.75	9.0		4	1.81	0.79	***6.5***	Hole-5
3.35	11.0		5	1.67	0.82	***7.8***	Hole-2
3.6	11.8		5	1.62	0.83	***7.6***	Hole-5
4.3	14.1		11	1.50	0.86	***16.2***	Hole-2
4.45	14.6		13	1.48	0.87	***18.9***	Hole-5
5.3	17.4		10	1.37	0.90	***14.0***	Hole-5
6.15	20.2		6	1.28	0.93	***8.1***	Hole-5
					Mean:	11.3	
					St. Dev.	5.0	

Depth (ft)	FC	Sand	Silt	Clay	D50	
10.5	***34.0***	***66.0***	***34.0***	***0.0***	***0.2***	Hole-2
11.2	***17.0***	***83.0***	***17.0***	***0.0***	***0.1***	Hole-2
13.5	***27.0***	***73.0***	***26.0***	***1.0***	***0.1***	Hole-2
14.2	***20.0***	***80.0***	***20.0***	***0.0***	***0.1***	Hole-2
18.3	***33.0***	***67.0***	***29.0***	***4.0***	***0.1***	Hole-2
22.8	*18.0*	*82.0*	*17.0*	*1.0*	*0.1*	Hole-2
Mean:	26.2	73.8	25.2	1.0	0.109	
St. Dev.	7.6	7.6	6.8	1.7	0.038	

Case number:	92
Earthquake:	Westmorland
Magnitude:	5.6 (M_L), 5.9(Mw)
Location:	McKim Ranch A
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Located between the Alamo River and the Redwood Canal. A: very fine sand (fining upwards)-loose sand that filled the relic channel. B: stiff silty clay Mc Kim Ranch is ~25 km from the epicenter of the earthquake. At SUP seismographic station which is ~ located at an epicentral distance of 24 km, PGA was recorded as 0.1 g (deep alluvial site). Based on earthquake specific attenuation relationship, the mean and standard deviation were estimated as 0.12 ± 0.06 g. PGA was estimated by Seed et al (1984) ~0.09 g . Energy ratio ~68 %

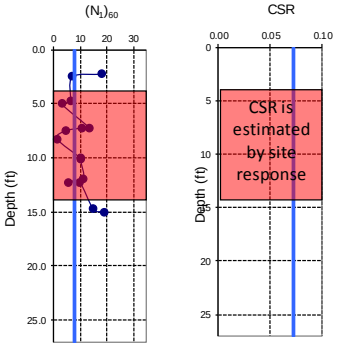
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.106 ± 0.005		0.105
Data Class	A			% Fines	19.8 ± 3.5	31.0	31
Critical Depth Range	5.0 - 13.0	6.9	7.0	% PI			
Depth to GWT (ft)	5.0	4.9	5.0				
σ_v (psf)	1030.0 ± 161.2	793.6	800.0	N	5.4 ± 2.6	3.0	3.0
σ_v' (psf)	780.4 ± 80.7	668.3	675.0	C_R	0.79	0.80	0.8
a_{max} (g)	0.090 ± 0.014	0.09	0.090	C_S	1.00	1.00	
r_d	0.93 ± 0.010	0.97	0.990	C_B	1.00	1.00	1.050
CSR	0.072 ± 0.012	0.068	0.070	C_E	1.13	1.13	
Equivalent Magnitude	5.9	5.9		C_N	1.65	1.70	1.6
MSF		1.52	1.40	$(N_1)_{60}$	7.9 ± 3.8	4.6	4.00
CSRN		0.040	0.05				



γ_m=110±3 pcf

γ_s=120±3 pcf



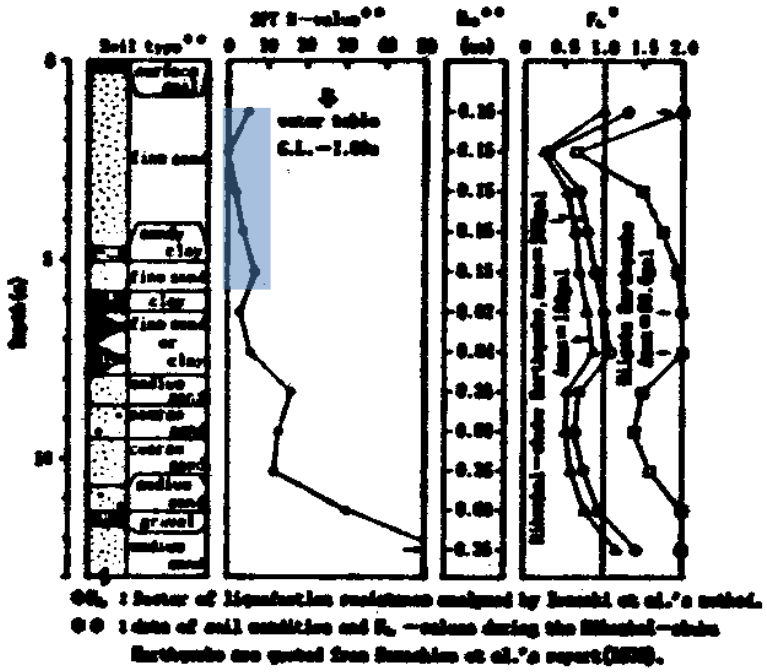
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
0.7	2.3		13	2.00	0.62	18.3	M-7
0.8	2.5		5	2.00	0.63	7.2	SM-7
1.4	4.8		4	2.00	0.70	***6.3***	***M-7***
1.5	5.0		2	1.96	0.71	***3.1***	***SM-7***
2.2	7.3		9	1.76	0.76	***13.6***	***M-2***
2.2	7.3		7	1.76	0.76	***10.6***	***M-7***
2.3	7.5		3	1.75	0.76	***4.5***	***SM-7***
2.5	8.3		1	1.69	0.78	***1.5***	***M-3***
3.0	10.0		7	1.59	0.81	***10.2***	***SM-7***
3.1	10.2		7	1.58	0.81	***10.1***	***M-7***
3.7	12.0		8	1.49	0.84	***11.3***	***SM-7***
3.7	12.3		4	1.48	0.84	***5.6***	***M-3***
3.7	12.3		7	1.48	0.84	***9.8***	***M-7***
4.5	14.8		11	1.38	0.87	15.0	M-7
4.6	15.0		14	1.37	0.87	19.0	SM-7
					Mean:	7.9	
					St. Dev.	3.8	

Depth (ft)	FC	sand	silt	clay	D50	
2.3	31.0	69.0	18.0	13.0	0.085	M-7
2.5	-	-	-	-	-	SM-7
4.8	-	-	-	-	-	***M-7***
5.0	-	-	-	-	-	***SM-7***
7.3	-	-	-	-	-	***M-2***
7.3	***24.0***	***76.0***	***14.0***	***10.0***	***0.100***	***M-7***
7.5	-	-	-	-	-	***SM-7***
8.3	-	-	-	-	-	***M-3***
9.4	***21.0***	***79.0***	***11.0***	***10.0***	***0.105***	***M-7***
10.0	-	-	-	-	-	***SM-7***
10.2	***16.0***	***84.0***	***9.0***	***7.0***	***0.110***	***M-7***
12.0	-	-	-	-	-	***SM-7***
12.3	-	-	-	-	-	***M-3***
12.3	***18.0***	***82.0***	***11.0***	***7.0***	***0.110***	***M-7***
14.8	22.0	78.0	15.0	7.0	0.093	M-7
15.0	-	-	-	-	-	SM-7
	19.8	80.3	11.3	8.5	0.106	
	3.5	3.5	2.1	1.7	0.005	

Case number:	93
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.1 (M)
Location:	Aki City (Arayamotomachi)
References:	Yasuda and Tohno (1988)
Nature of Failure:	No Surface evidence of liquefaction
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at Aomori Prefecture but not at Akita. At Arayamotomachi Town in Akita City, liquefaction was caused along a narrow area by the main shock of Nihonkaichubu earthquake. The damage was concentrated in a small zone about 100 m in width and 500 m in length, in an area of land recently reclaimed over an old river by filling with sand from adjacent sand dunes. At a point near the center of the liquefied area, a layer of very loose fine sand, with SPT N values of 0-6, forms a deposit extending from the ground surface to about 6 m in depth. The water table is very shallow, only 1 m in depth. The maximum surface acceleration was given as 0.2 and 0.15 g for the main and after shock respectively. The site has also liquefied during Niigata EQ. It is believed that a thin soil layer has liquefied during Niigata EQ. However, several meters of soil thickness is estimated to have been liquefied during Nihonkai-Chubu Earthquake.

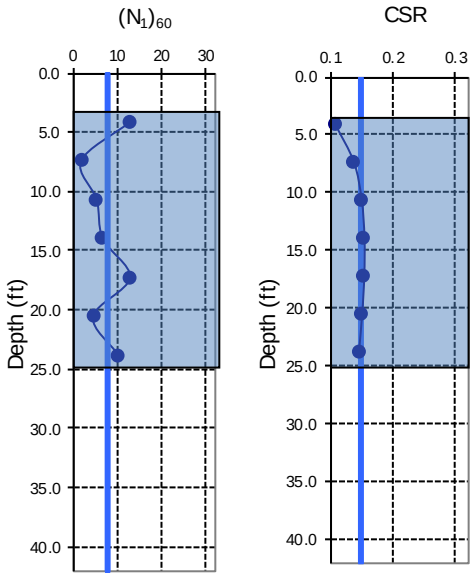
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.150 ± 0.050		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	3.3 - 18.0	14.1		% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.3					
σ_v (psf)	1177.0 ± 284.0	1441.1		N	4.4 ± 2.7	2.6	
σ_v' (psf)	716.4 ± 132.4	772.8		C_R	0.86	0.95	
a_{max} (g)	0.150 ± 0.045	0.15		C_S	1.00	1.00	
r_d	0.93 ± 0.048	0.95		C_B	1.00	1.00	
CSR	0.150 ± 0.047	0.172		C_E	1.22	1.22	
Equivalent Magnitude	7.1	6.8		C_N	1.72	1.70	
MSF		1.20		$(N_1)_{60}$	7.9 ± 4.8	5.1	
CSRN		0.133					



γ_m=100±3 pcf

γ_s=115±3 pcf

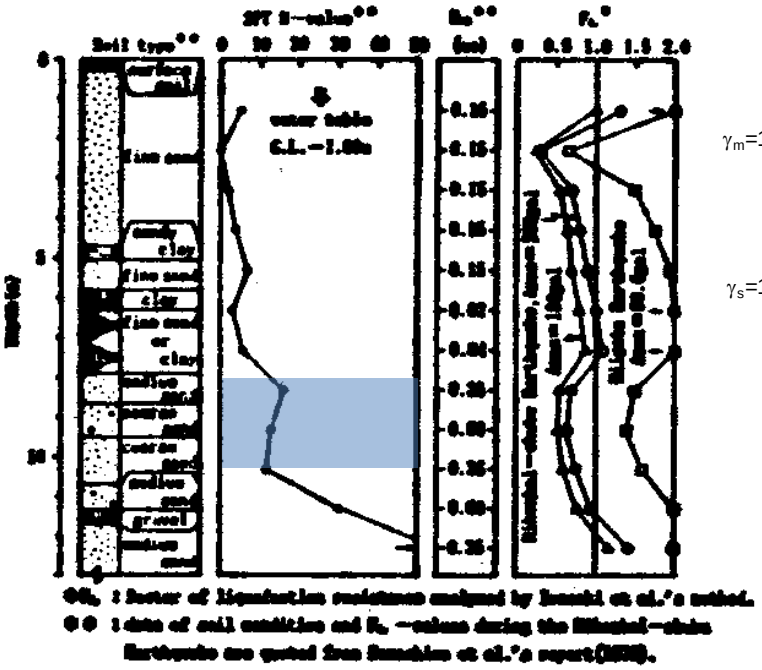


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.3	4.1	0.109	7.0	2.00	0.75	***12.8***	fine sand
2.3	7.4	0.138	1.0	1.97	0.81	***1.9***	fine sand
3.3	10.7	0.150	3.0	1.72	0.86	***5.4***	fine sand
4.3	13.9	0.154	4.0	1.54	0.90	***6.7***	fine sand
5.3	17.2	0.154	8.0	1.41	0.93	***12.8***	fine sand
6.3	20.5	0.151	3.0	1.31	0.96	4.6	clay seam
7.3	23.8	0.146	7.0	1.23	0.98	10.3	fine sand or clay
					Mean:	7.9	
					St. Dev.	4.8	

Case number:	94
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.1 (M)
Location:	Aki City (Arayamotomachi Coarse Sand)
References:	Yasuda and Tohno (1988)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at Aomori Prefecture but not at Akita. At Arayamotomachi Town in Akita City, liquefaction was caused along a narrow area by the main shock of Nihonkaichubu earthquake. The damage was concentrated in a small zone about 100 m in width and 500 m in length, in an area of land recently reclaimed over an old river by filling with sand from djacent sand dunes. At a point near the center of the liquefied area, a layer of very loose fine sand, with SPT N values of 0-6, forms a deposit extending from the ground surface to about 6 m in depth. The water table is very shallow, only 1 m in depth. The maximum surface acceleration was given as 0.2 and 0.15 g for the main and after shock respectively. The site has also liquefied during Niigata EQ. It is believed that a thin soil layer has liquefied during Niigata EQ. However, several meters of soil thickness is estimated to have been liquefied during Nihonkai-Chubu Earthquake.

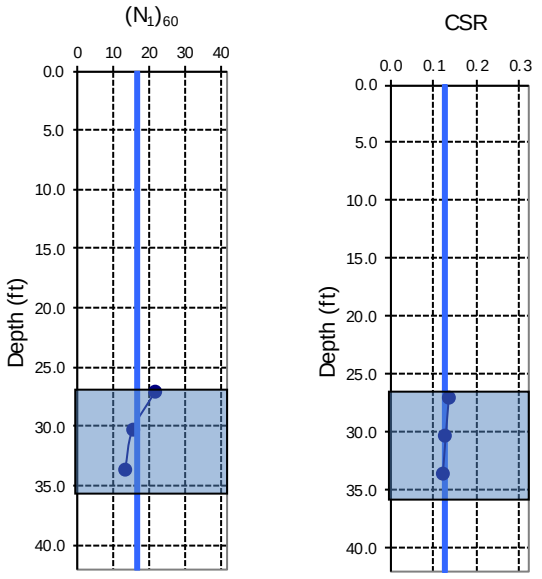
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.420 ± 0.050		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	26.2 - 34.4	30.2		% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.3					
σ_v (psf)	3576.1 ± 183.4	3299.9		N	13.1 ± 3.3	13.1	
σ_v' (psf)	1887.1 ± 114.2	1608.2		C_R	1.00	1.00	
a_{max} (g)	0.150 ± 0.045	0.15		C_S	1.00	1.00	
r_d	0.70 ± 0.118	0.86		C_B	1.00	1.00	
CSR	0.129 ± 0.044	0.172		C_E	1.22	1.22	
Equivalent Magnitude	7.1	6.8		C_N	1.06	1.13	
MSF		1.20		$(N_1)_{60}$	16.9 ± 4.3	18.1	
CSRN		0.139					



γ_m=100±3 pcf

γ_s=120±3 pcf

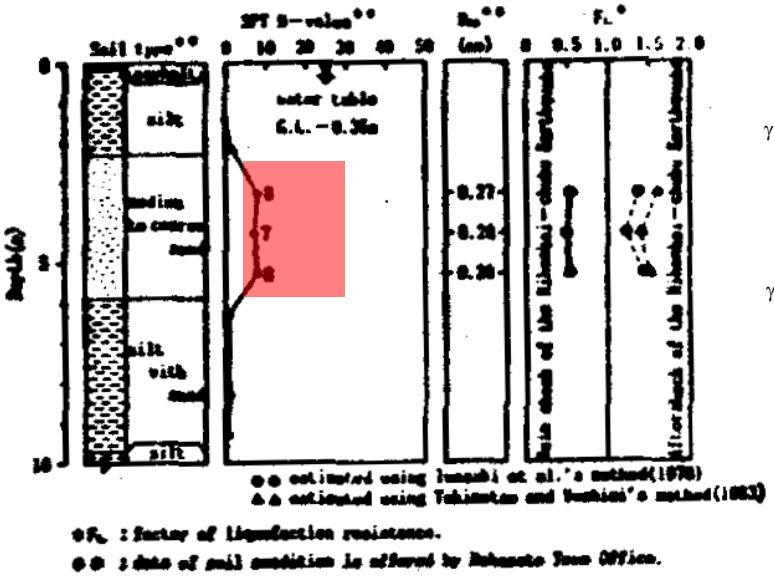


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
8.3	27.1	0.135	16.0	1.12	1.00	***21.7***
9.3	30.3	0.129	12.0	1.06	1.00	***15.5***
10.3	33.6	0.123	11.0	1.01	1.00	***13.5***
					Mean:	16.9
					St. Dev.	4.3

Case number:	95
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.1 (M)
Location:	Nakazata Town (Takeda Elementary School)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils, settled structures.
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at the Takeda Elementary School in Nakazato Town. The school was built on the old bed of the Iwaki River. During the main shock on May 26, the ground beneath the schoolhouses (two-story wood structures) and sweeped out sand and water then was submerged. The aftershock of June 21 again caused sand and water to sweep out from the ground. According to some teachers, the area where sand volcanoes were observed almost coincided with the area where sand volcaones had been caused by the main shock. The maximum surface acceleration was suggested to be 0.28 and 0.11 g for the main and after shock respectively based on the values recorded at the Tsugaru-ohashi Bridge about 6 km from the school. Slightly higher value is ~0.12g is adopted due to possible amplification at extremely soft upper 5 ft.

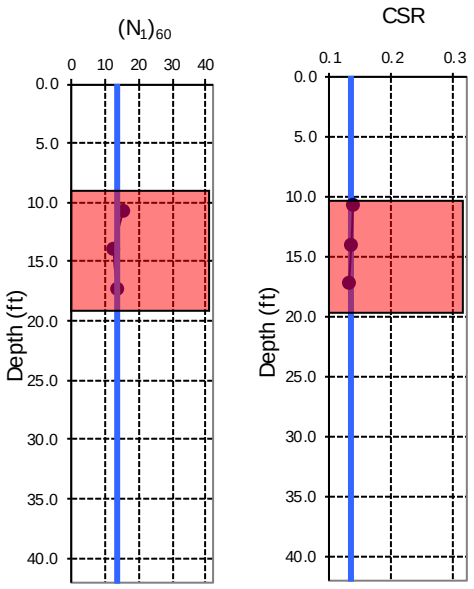
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.240 ± 0.050		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	8.2 - 21.3	14.1		% PI			
Depth to GWT (ft)	1.1 ± 0.4	1.1					
σ_v (psf)	1680.6 ± 254.9	1691.7		N	7.9 ± 0.9	7.4	
σ_v' (psf)	831.0 ± 123.3	877.2		C_R	0.90	0.95	
a_{max} (g)	0.120 ± 0.036	0.11		C_S	1.00	1.00	
r_d	0.85 ± 0.064	0.95		C_B	1.00	1.00	
CSR	0.134 ± 0.042	0.132		C_E	1.22	1.22	
Equivalent Magnitude	7.1	6.8		C_N	1.60	1.56	
MSF		1.20		$(N_1)_{60}$	13.8 ± 1.5	13.3	
CSRN		0.101					



γ_m=100±3 pcf

γ_s=115±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.7	0.139	8.0	1.85	0.86	***15.5***
4.3	13.9	0.135	7.0	1.64	0.90	***12.5***
5.3	17.2	0.130	8.0	1.48	0.93	***13.4***
					Mean:	13.8
					St. Dev.	1.5

Case number:	96
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Akita Station
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. The Akita Station is the strong ground motion station site. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. Geometric mean of 190 and 205 Gals is equal to 0.204 g is assigned to the site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. (1989) for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. Soil profile data is missing just N values and FC exist. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data could not be obtained. Site response by Iai et.al. exists however soil profile not known. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 75 m away from the channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 75 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated. In Idriss and Boulanger (2010) Akita Station is divided into two case since there exist 2 borehole datas. Note that for one borehole FC data is missing. In addition two boreholes at one site is against statistically independent assumption. In Cetin 2014 Akita (1) and (2) are composed and represented as Akita Station by taking into account of two borehole data.

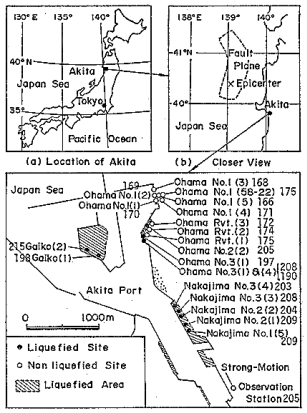
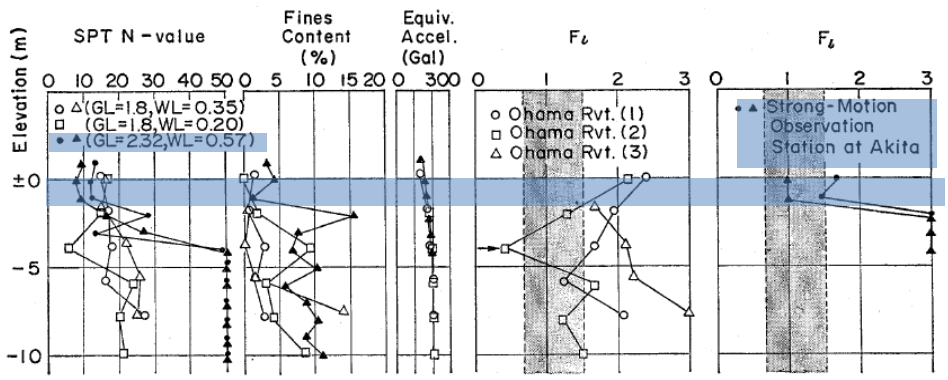


Fig. 8. Investigated sites in Akita Port

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No			D_{50}	- ± -		
Data Class	B			% Fines	2.7 ± 1.5		
Critical Depth Range	6.6 - 13.1			% PI	-		
Depth to GWT (ft)	5.7 ± 3.0						
σ_v (psf)	1181.1 ± 132.9			N	10.3 ± 1.8		
σ_v' (psf)	925.2 ± 198.6			C_R	0.85		
a_{max} (g)	0.204 ± 0.010			C_S	1.00		
r_d	0.999 ± 0.045			C_B	1.00		
CSR	0.169 ± 0.037			C_E	1.22		
Equivalent Magnitude	7.7			C_N	1.51		
MSF	0.93			$(N_1)_{60}$	16.0 ± 2.8		
CSRN	0.18						

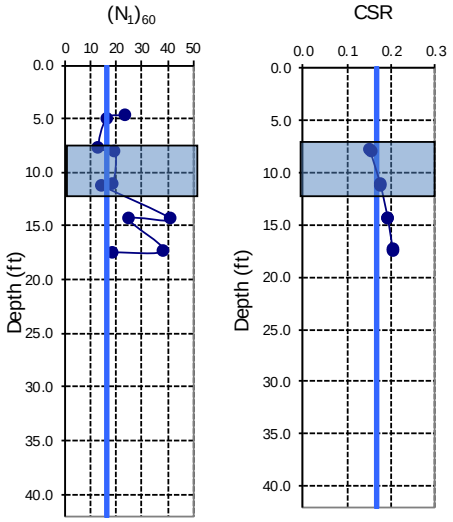


γ_m=120±3 pcf

γ_s=120±3 pcf

(e) Ohama No. 2 Former Revetment and the strong-motion observation station (Nonliquefied)

Fig. 9. Ground conditions and resulted liquefaction potentials at Akita Port during 1983 Nihonkai-Chubu Earthquake

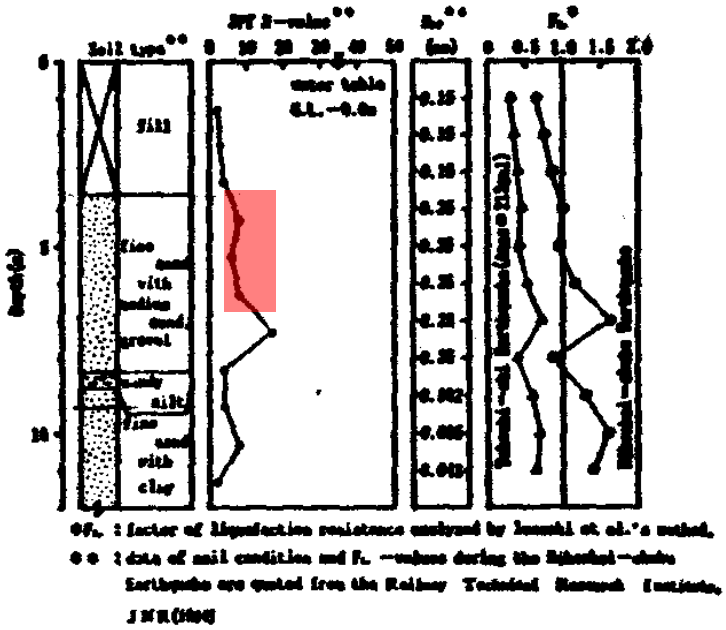


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
1.41	4.6	NA	13	1.95	0.76	*23.5*	-	Akita Station (1)
1.50	4.9	NA	9	1.89	0.77	***15.9***	***3.0***	Akita Station (2)
2.35	7.7	0.153	8	1.62	0.82	***12.9***	***4.0***	Akita Station (2)
2.43	8.0	0.155	12	1.61	0.82	***19.3***	**-**	Akita Station (1)
3.36	11.0	0.176	12	1.46	0.86	***18.4***	**-**	Akita Station (1)
3.40	11.2	0.177	9	1.45	0.86	***13.8***	***1.0***	Akita Station (2)
4.37	14.3	0.192	28	1.34	0.90	*41.0*	-	Akita Station (1)
4.37	14.3	0.192	17	1.34	0.90	*24.9*	*16.0*	Akita Station (2)
5.26	17.3	0.203	27	1.25	0.93	*38.2*	*8.0*	Akita Station (2)
5.32	17.5	0.203	13	1.25	0.93	*18.3*	-	Akita Station (1)
					Mean:	16.0	2.7	
					St. Dev.	2.8	1.5	

Case number:	97
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (M)
Location:	Aomori City (Aomori Station)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils, settled structures.
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at the Aomori Station. The station faces the sea and the north part of the station is used as a landing place for ferryboats. The north part was built on reclaimed land constructed in 1945, but the south part was built on natural land. Liquefaction was induced at the north part of the station during the main shock of the Nihonakai-chubu EQ. Many sand volcanoes, settlement of rails and buildings and uplift of buried tanks occurred. The maximum surface acceleration at Aomari Port, about 1 km north of Aomori Station was 0.116 g

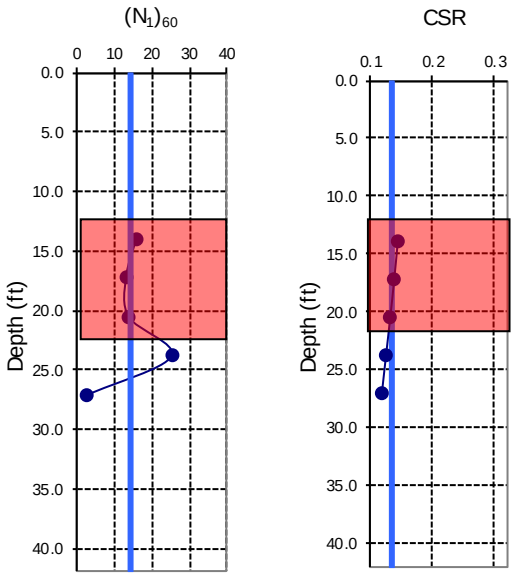
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.250 ± 0.050		
Data Class	A			% Fines	3.0 ± 2.0	3.0	
Critical Depth Range	13.1 - 24.6	18.7		% PI			
Depth to GWT (ft)	0.0 ± 0.4	0.0					
σ_v (psf)	2263.8 ± 236.6	1984.1		N	9.0 ± 0.9	9.0	
σ_v' (psf)	1086.6 ± 124.8	793.6		C_R	0.94	0.95	
a_{max} (g)	0.116 ± 0.006	0.12		C_S	1.00	1.00	
r_d	0.86 ± 0.079	0.96		C_B	1.00	1.00	
CSR	0.135 ± 0.015	0.178		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.40	1.58	
MSF		0.95		$(N_1)_{60}$	14.4 ± 1.4	16.5	
CSRN		0.171					



γ_m=100±3 pcf

γ_s=120±3 pcf

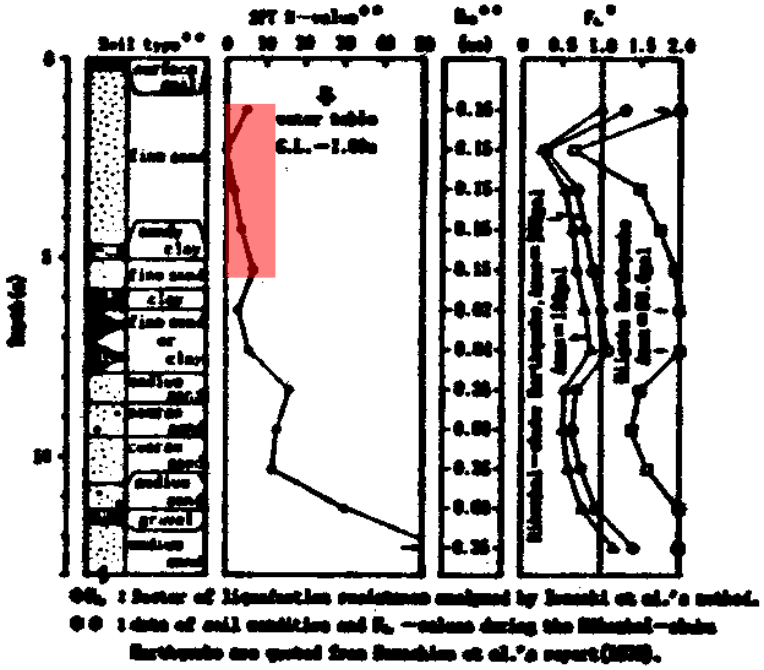


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
4.3	13.9	0.143	9.0	1.62	0.90	***15.9***	
5.3	17.2	0.138	8.0	1.46	0.93	***13.2***	
6.3	20.5	0.132	9.0	1.34	0.96	***14.0***	
7.3	23.8	0.126	17.0	1.24	0.98	25.3	Gravel?
8.3	27.1	0.120	2.0	1.17	1.00	2.8	Clay?
					Mean:	14.4	
					St. Dev.	1.4	

Case number:	98
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (M)
Location:	Aki City (Arayamotomachi)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils, settled structures, sewage tanks floated.
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at Aomori Prefecture but not at Akita. At Arayamotomachi Town in Akita City, liquefaction was caused along a narrow area by the main shock of Nihonkaichubu earthquake. The damage was concentrated in a small zone about 100 m in width and 500 m in length, in an area of land recently reclaimed over an old river by filling with sand from djacent sand dunes. At a point near the center of the liquefied area, a layer of very loose fine sand, with SPT N values of 0-6, forms a deposit extending from the ground surface to about 6 m in depth. The water table is very shallow, only 1 m in depth. The maximum surface acceleration was given as 0.2 and 0.15 g for the main and after shock respectively. The site has also liquefied during Niigata EQ. It is believed that a thin soil layer has liquefied during Niigata EQ. However, several meters of soil thickness is estimated to have been liquefied during Nihonkai-Chubu Earthquake.

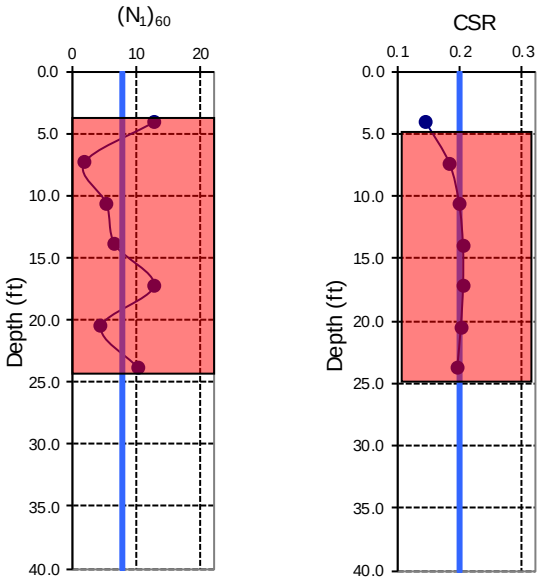
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.150 ± 0.050		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	3.3 - 18.0	14.1		% PI			
Depth to GWT (ft)	3.3 ± 0.3	3.3					
σ_v (psf)	1177.0 ± 284.0	1441.1		N	4.4 ± 2.7	2.6	
σ_v' (psf)	716.4 ± 132.4	772.8		C_R	0.86	0.95	
a_{max} (g)	0.200 ± 0.060	0.20		C_S	1.00	1.00	
r_d	0.94 ± 0.048	0.97		C_B	1.00	1.00	
CSR	0.200 ± 0.062	0.234		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.72	1.70	
MSF		0.95		$(N_1)_{60}$	7.9 ± 4.8	5.1	
CSRN		0.230					



γ_m=100±3 pcf

γ_s=115±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.3	4.1	0.145	7.0	2.00	0.75	***12.8***	fine sand
2.3	7.4	0.184	1.0	1.97	0.81	***1.9***	fine sand
3.3	10.7	0.200	3.0	1.72	0.86	***5.4***	fine sand
4.3	13.9	0.207	4.0	1.54	0.90	***6.7***	fine sand
5.3	17.2	0.207	8.0	1.41	0.93	***12.8***	fine sand
6.3	20.5	0.203	3.0	1.31	0.96	4.6	clay seam
7.3	23.8	0.197	7.0	1.23	0.98	10.3	fine sand or clay
					Mean:	7.9	
					St. Dev.	4.8	

Case number:	99
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Gaiko-1&2
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Gaiko-1 and Gaiko-2 is located approximatly 2.76 km away from strong ground motion station in addition the two sites are very close to each other and soil profiles are very similar. In Cetin 2015 database these two sites are decided to be combained and represetnetd as Gaiko-1&2. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 215 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 215 and 205 Gals are obtained. Geometric mean of 215 and 204 Gals is equal to 0.214 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 37 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 37 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

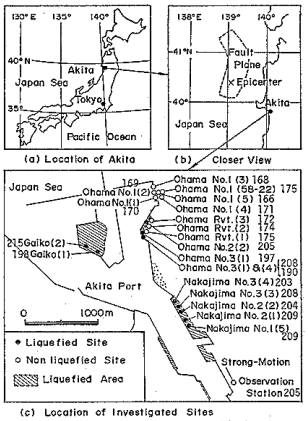
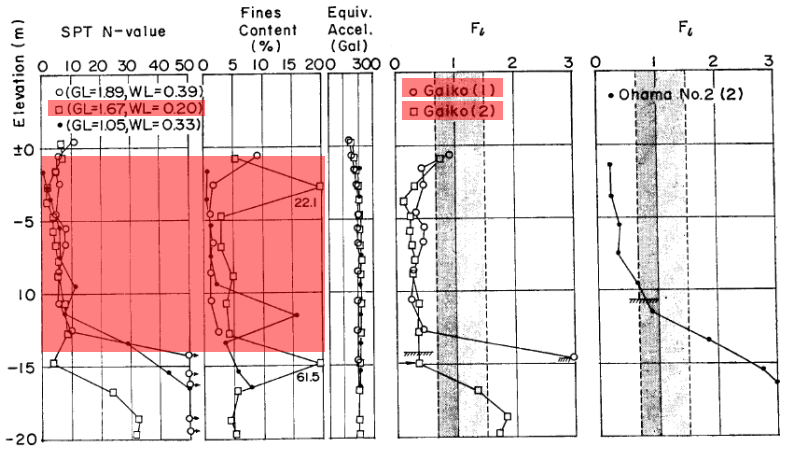


Fig. 8. Investigated sites in Akita Port

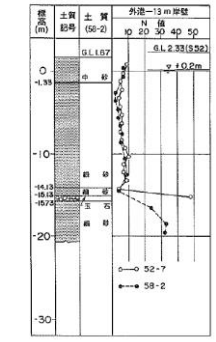
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes			D50	- ± -		
Data Class	B			% Fines	5.5 ± 6.9		
Critical Depth Range (m)	9.8 - 59.1			% PI	-		
Depth to GWT (ft)	4.8 ± 0.3						
σ_v (psf)	4085.6 ± 988.4			N	5.8 ± 1.8		
σ_v' (psf)	2237.0 ± 481.2			CR	1.00		
a_{max} (g)	0.214 ± 0.032			CS	1.00		
r_d	0.622 ± 0.131			CB	1.00		
CSR	0.158 ± 0.041			CE	1.22		
Equivalent Magnitude	7.7			CN	0.97		
MSF				$(N_1)_{60}$	6.9 ± 2.1		
CSRN							



γ_m=110±3 pcf

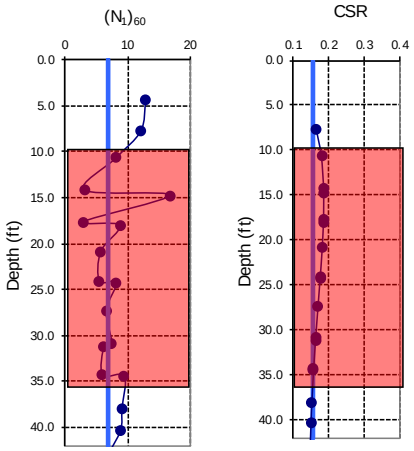
γ_s=120±3 pcf



図－3.1.101 外港－13m岸壁（N値）土質（58－2）

(a) Gaiko and Ohama No.2 (Liquefied)

0-3	medium sand
3-15.8	sand
15.8-16.8	sand
16.8-17.4	**
17.4-22.7	sand

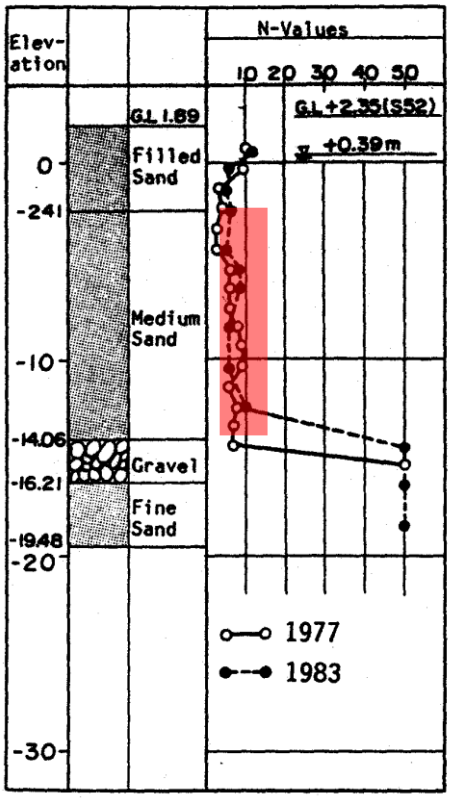


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC		
1.3	4.4	NA	7	2.00	0.76	*12.9*	-	medium sand	Gaiko-2
2.4	7.8	0.166	7	1.74	0.82	*12.1*	*5.0*	medium sand	Gaiko-2
3.3	10.7	0.181	5	1.56	0.86	***8.1***	-	sand	Gaiko-2
4.3	14.2	0.188	2	1.41	0.90	***3.1***	***22.0***	sand	Gaiko-2
4.5	14.8	0.188	11	1.39	0.90	*16.8*	-	-	Gaiko-1
5.4	17.6	0.188	2	1.29	0.93	***2.9***	-	sand	Gaiko-2
5.5	18.0	0.187	6	1.28	0.94	***8.7***	***9.0***	-	Gaiko-1
6.4	20.9	0.184	4	1.21	0.96	***5.6***	***3.0***	sand	Gaiko-2
7.4	24.1	0.178	4	1.13	0.99	***5.4***	-	sand	Gaiko-2
7.4	24.3	0.178	6	1.13	0.99	***8.2***	***2.0***	-	Gaiko-1
8.3	27.4	0.171	5	1.08	1.00	***6.5***	***3.0***	sand	Gaiko-2
9.4	30.9	0.164	6	1.02	1.00	***7.4***	-	sand	Gaiko-2
9.5	31.2	0.164	5	1.02	1.00	***6.2***	***1.0***	-	Gaiko-1
10.5	34.4	0.158	5	0.97	1.00	***5.9***	***3.0***	sand	Gaiko-2
10.5	34.4	0.158	8	0.97	1.00	***9.5***	-	-	Gaiko-1
11.6	38.1	0.153	8	0.93	1.00	***9.1***	***2.0***	-	Gaiko-1
12.3	40.4	0.150	8	0.91	1.00	***8.8***	***4.0***	sand	Gaiko-2
13.5	44.3	0.147	6	0.87	1.00	***6.3***	***1.0***	-	Gaiko-1
14.4	47.2	0.145	9	0.84	1.00	***9.2***	***4.0***	sand	Gaiko-2
15.7	51.5	0.143	6	0.81	1.00	***5.9***	***1.0***	-	Gaiko-1
16.4	53.8	0.143	4	0.79	1.00	***3.9***	***20.0***	sand	Gaiko-2
17.5	57.4	0.142	10	0.77	1.00	***9.4***	***2.0***	-	Gaiko-1
18.4	60.5	0.142	24	0.75	1.00	*22.0*	*6.0*	sand	Gaiko-2
20.3	66.5	0.142	33	0.72	1.00	*28.9*	*4.0*	sand	Gaiko-2
21.3	70.0	0.142	32	0.70	1.00	*27.4*	*5.0*	sand	Gaiko-2
					Mean:	**6.9**	**5.5**		
					St. Dev	**2.1**	**6.9**		

Case number:	100
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (M)
Location:	Gaiko Wharf B-2
References:	Hamada (1992)
Nature of Failure:	Lateral spreading, sand boils, ground fissures
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area . Akita Harbor suffered severe damage to its quay walls due to liquefaction. At Gaiko, which is an area newly reclaimed from the sea, liquefaction-induced ground displacements were observed. The quay wall, made of concrete caisson moved about 1.5 m and was tilted about 3 degrees The maximum surface acceleration at Akita City 115 km from the epicenter was recorded as 0.235 and 0.219 g.

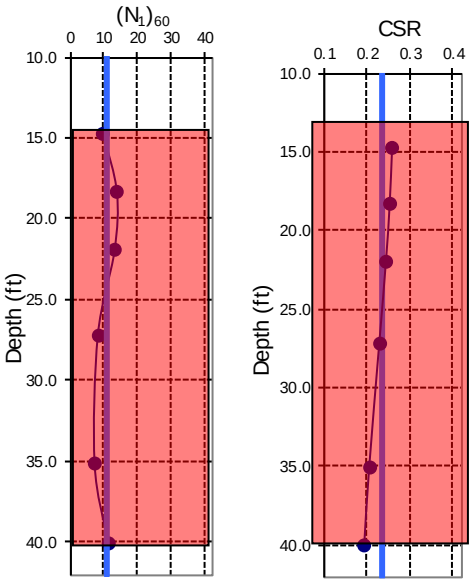
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.250 ± 0.050		
Data Class	B			% Fines	1.0 ± 2.0	1.0	
Critical Depth Range	8.2 - 41.0	24.6		% PI			
Depth to GWT (ft)	1.3 ± 0.3	1.3					
σ_v (psf)	2940.0 ± 659.9	2568.9		N	7.8 ± 1.9	7.7	
σ_v' (psf)	1484.4 ± 323.0	1106.9		C_R	0.99	0.95	
a_{max} (g)	0.227 ± 0.068	0.23		C_S	1.00	1.00	
r_d	0.82 ± 0.099	0.94		C_B	1.00	1.00	
CSR	0.239 ± 0.078	0.320		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.19	1.39	
MSF		0.95		$(N_1)_{60}$	11.2 ± 2.7	12.4	
CSRN		0.317					



γ_m=110±3 pcf

γ_s=120±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.4	7.9	0.256	7.0	2.00	0.82	***13.9***
4.5	14.8	0.260	6.0	1.52	0.90	***10.0***
5.6	18.4	0.255	9.0	1.37	0.94	***14.1***
6.7	22.0	0.246	9.0	1.26	0.97	***13.4***
8.3	27.2	0.231	6.0	1.14	1.00	***8.3***
10.7	35.1	0.208	6.0	1.01	1.00	***7.3***
12.2	40.0	0.197	10	0.94	1.00	***11.5***
					Mean:	11.2
					St. Dev.	2.7

Case number:	101
Earthquake:	1983 M=7.7 Nihonka-chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Hakodate
References:	Iai. et. al. (1989)
Nature of Failure:	-
Comments:	In Table 1 additional investigated sites where strong-motion observation stations exist are tabulated. In Figure 12 location of investigated site and epicenters of the earthquakes are shown. Hakodate site is a non-liquefied site. In Hakodate maximum accelereation values measured as 48 gals (NS) and 52 gals (EW) geometric mean of which is 0.051 g. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. Soil profile data is missing just N values. FC values exist for three depth moreover the values reflects clayey behaviour. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data could not be obtained. There is an ambiguity if the site is free field or not in the vicinty of Pacific Ocean. In addition the location of the site may not be correctly located in Figure 12 since the scale is very small and the coordinates of the borehole are not available. The location of the station could not be found. By taking into account the above mentioned reasons the site is rejected.

Table 1. Strong-motion observation stations and earthquakes

Investigated Sites	Max. Accel. (Gal) NS	EW	UD	Accelerograph	Epicentral Distance (km)	Magnitude	Date of Earthquake
Hachinohe	233	181	119	SMAC-B2	178	7.9	May 16, 1968**
Hakodate	48*	52*	36*	ERS	211	7.7	May 26, 1983***
Hososhima	182	242	48	SMAC-B2	89	7.5	April 1, 1968
Hososhima	190	268	102	SMAC-B2	47	7.1	Aug. 7, 1984
Kushiro	166	120	51	SMAC-B2	127	7.4	June 17, 1973****
Muroran	204	135	70	SMAC-B2	280	7.9	May 16, 1968**
Tokachi	113*	168*	46*	ERS	64	7.1	March 21, 1982*****
Tomakomai	161	115	20	SMAC-B2	50	7.1	Jan. 23, 1981

* SMAC-B2 Equivalent (i.e. modified, by digital filtering technique, into the maximum acceleration which would be obtained by the SMAC-B2 accelerograph)
** 1968 Tokachi-Oki Earthquake
*** 1983 Nihonkai-Chubu Earthquake
**** 1973 Nemuro-Hanto-Oki Earthquake
***** 1982 Urakawa-Oki Earthquake

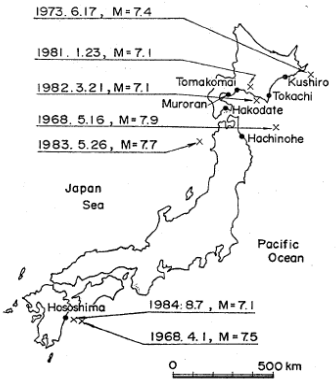
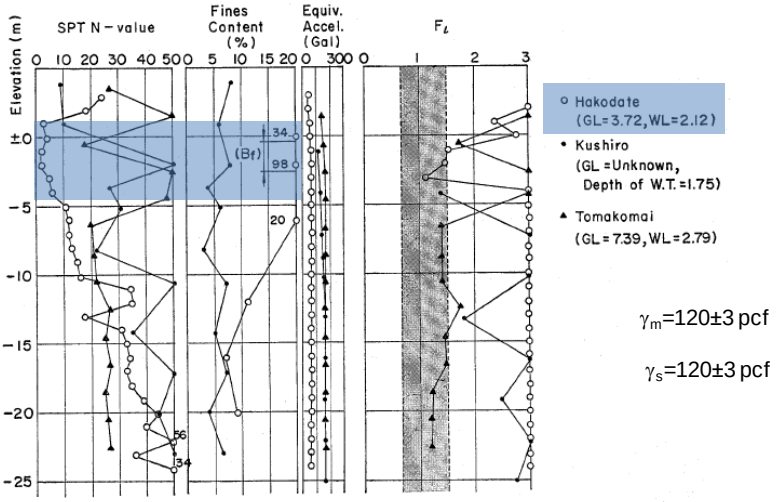


Fig. 12. Locations of other investigated ports and the epicenters of the earthquakes

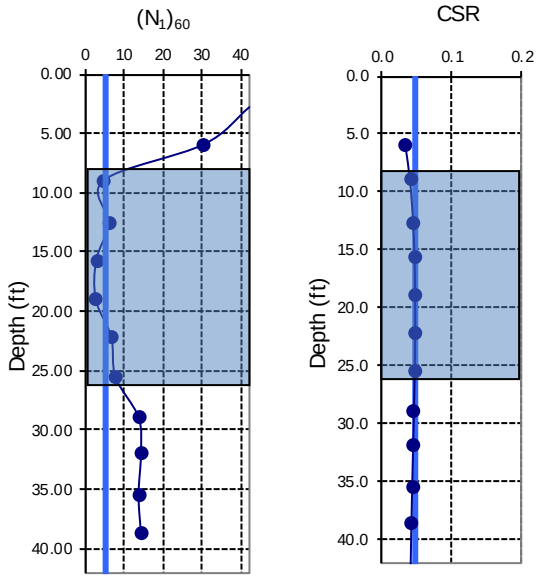
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	- ± -	-	
Data Class	A	-		% Fines	66.0 ± 45.3	66.0	
Critical Depth Range	8.2 - 26.2	14.1		% PI	-	-	
Depth to GWT (ft)	5.2 ± 3.0	5.2					
σ_v (psf)	2066.9 ± 363.0	1691.7		N	3.7 ± 1.5	2.6	
σ_v' (psf)	1319.7 ± 258.1	1127.8		C_R	0.93	0.95	
a_{max} (g)	0.051 ± 0.003	0.052		C_S	1.00	1.00	
r_d	0.917 ± 0.073	0.970		C_B	1.00	1.00	
CSR	0.048 ± 0.008	0.049		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.27	1.41	
MSF		0.95		$(N_1)_{60}$	5.2 ± 2.1	4.2	
CSRN		0.05					



(b) Hakodate, Kushiro, and Tomakomai (Nonliquefied)

Fig. 13. Ground conditions and resulted liquefaction potentials at other strong-motion observation stations



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
0.86	2.81	NA	24	2.00	0.72	42.3	-
1.81	5.94	0.035	18	1.78	0.79	30.6	-
2.74	8.98	0.041	3	1.58	0.83	*4.8*	-
3.85	12.63	0.045	4	1.42	0.88	*6.1*	*34*
4.78	15.69	0.047	2	1.31	0.91	*2.9*	-
5.79	19.00	0.048	2	1.22	0.94	*2.8*	*98*
6.75	22.16	0.048	5	1.15	0.97	*6.8*	-
7.78	25.54	0.047	6	1.08	1.00	*7.9*	-
8.81	28.91	0.046	11	1.03	1.00	13.8	-
9.74	31.97	0.045	12	0.99	1.00	14.4	20
10.81	35.46	0.044	12	0.94	1.00	13.8	-
11.79	38.67	0.043	13	0.91	1.00	14.4	-
12.80	42.00	0.042	15	0.88	1.00	16.0	-
13.92	45.67	0.041	16	0.85	1.00	16.5	-
14.78	48.48	0.041	35	0.82	1.00	35.1	-
15.83	51.94	0.040	35	0.80	1.00	34.0	-
					Mean:	**5.2**	**66.0**
					St. Dev.	**2.1**	**45.3**

Case number:	102
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Nakajima No. 1 (5)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Nakajima No. 1 (5) is 1.03 km away from strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 209 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 209 and 199 Gals are obtained. Geometric mean of 209 and 199 Gals is equal to 0.208 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. Exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 22 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 22 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated. There exist two possibly liqueifed layer.

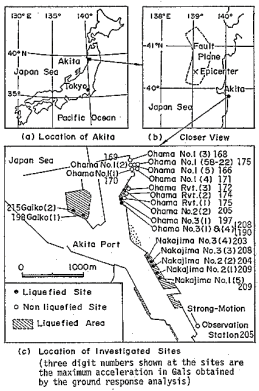
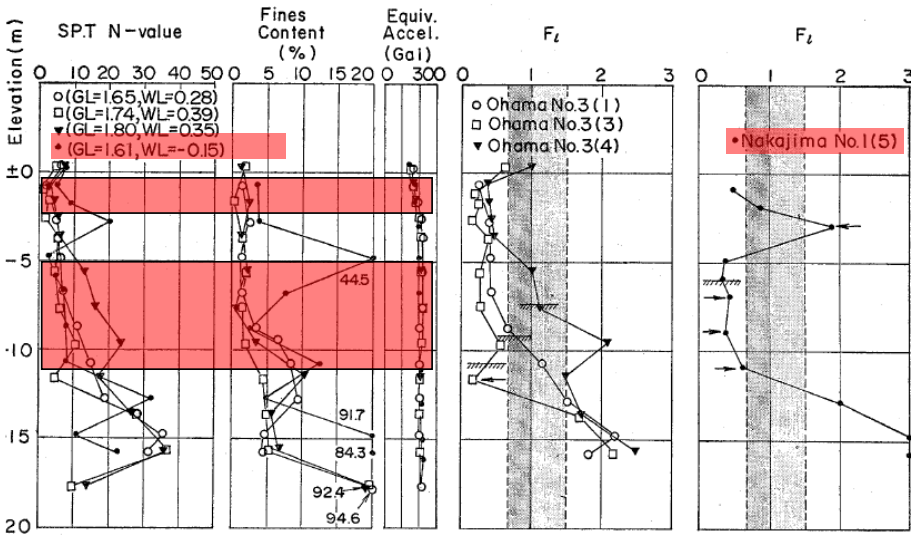


Fig. 8. Investigated sites in Akita Port

Summary of Data

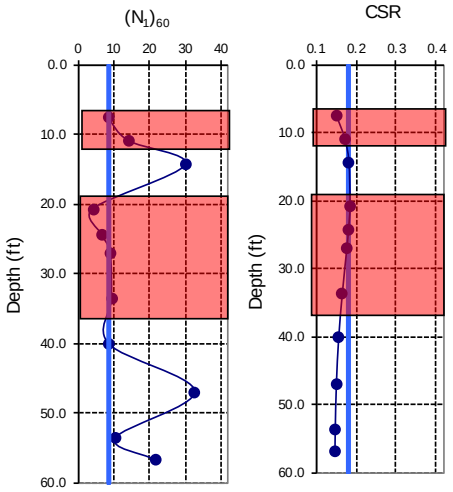
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	B	-		% Fines	14.6 ± 17.4	8.0	
Critical Depth Range	6.6 - 42.7	21.2		% PI	-	-	
Depth to GWT (ft)	5.8 ± 0.3	4.8					
σ_v (psf)	2895.0 ± 724.2	2589.8		N	6.5 ± 2.2	7.3	
σ_v' (psf)	1719.9 ± 351.8	1545.5		C_R	0.99	0.95	
a_{max} (g)	0.208 ± 0.031	0.205		C_S	1.00	1.00	
r_d	0.790 ± 0.099	0.950		C_B	1.00	1.00	
CSR	0.180 ± 0.036	0.210		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.11	1.18	
MSF		0.95		$(N_1)_{60}$	8.7 ± 3.0	9.9	
CSRN		0.22					

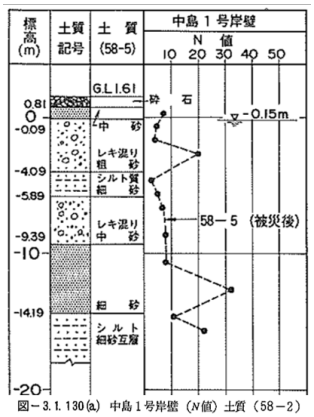


(b) Ohama No.3 and Nakajima No.1 (Liquefied)

γ_m=110±3 pcf

γ_s=120±3 pcf





図－3.1.130(a) 中島1号岸壁 (N値) 土質 (58－2)

sand	0.8-1.61
sand-gravel	1.61-5.7
silt-sand	5.7-7.5
sand-gravel	7.5-11.0
sand	11.0-15.8
silt-sand	15.8-20.0

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
1.3	4.4	NA	8.0	2.00	0.76	***14.7***	***3.0***	sand
2.3	7.6	0.150	5.0	1.69	0.81	***8.4***	***4.0***	sand-gravel
3.3	11.0	0.171	9.0	1.51	0.86	***14.2***	***-***	sand-gravel
4.4	14.4	0.181	20.0	1.37	0.90	***30.0***	***4.0***	sand-gravel
6.3	20.8	0.184	3.0	1.19	0.96	***4.2***	***45.0***	silt-sand
7.4	24.4	0.180	5.0	1.11	0.99	***6.7***	***-***	silt-sand
8.2	27.0	0.176	7.0	1.07	1.00	***9.1***	***8.0***	silt-sand
10.3	33.7	0.165	8.0	0.97	1.00	***9.5***	***3.0***	sand-gravel
12.2	40.2	0.156	8.0	0.90	1.00	***8.8***	***13.0***	sand
14.3	47.0	0.150	32.0	0.84	1.00	***32.7***	***5.0***	sand
16.4	53.7	0.147	11.0	0.79	1.00	***10.6***	***91.7***	silt-sand
17.3	56.8	0.147	23.0	0.77	1.00	***21.5***	***84.3***	silt-sand
					Mean:	**8.7**	**14.6**	
					St. Dev.	**3.0**	**17.4**	

Case number: 103
Earthquake: 1983 Nihonkai-Chubu
Magnitude: 7.7 (Mw by USGS, NISEE)
Location: Nakajima No. 2 (1)
References: Iai. et. al. (1989)

Nature of Failure: Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)

Comments:

Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8.
Nakajima No. 2 (1) is 1.12 km away from strong ground motion station.
SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983)
In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 209 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 209 and 199 Gals are obtained.
Geometric mean of 209 and 199 Gals is equal to 0.208 g is assigned to site.
SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases.
The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. Site response by Iai et.al. exists.
From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained.
There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth.
From Figure 8 the site is 32 m away from channel.
Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available.
Additionally Quay walls are believed to be located 32 m away from site.
Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.
In Iai et.al. first SPT-N value is missing hence It is taken from Technical Report of the Port and Horbour Research Institute Ministry of Transport.

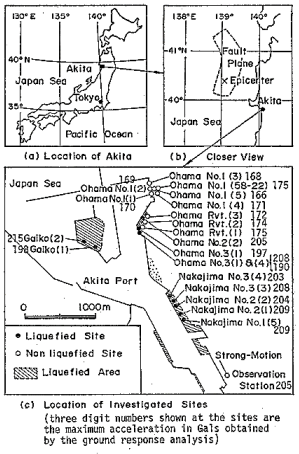
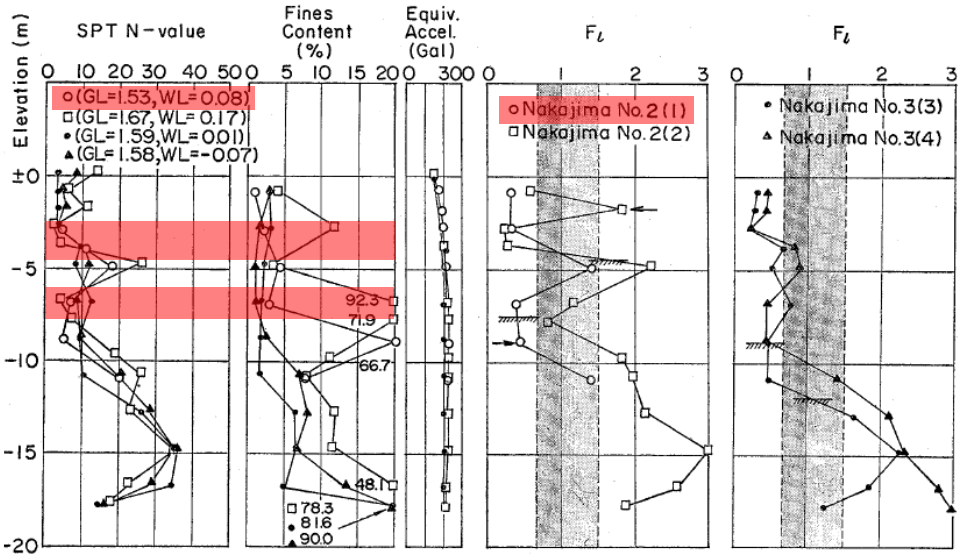


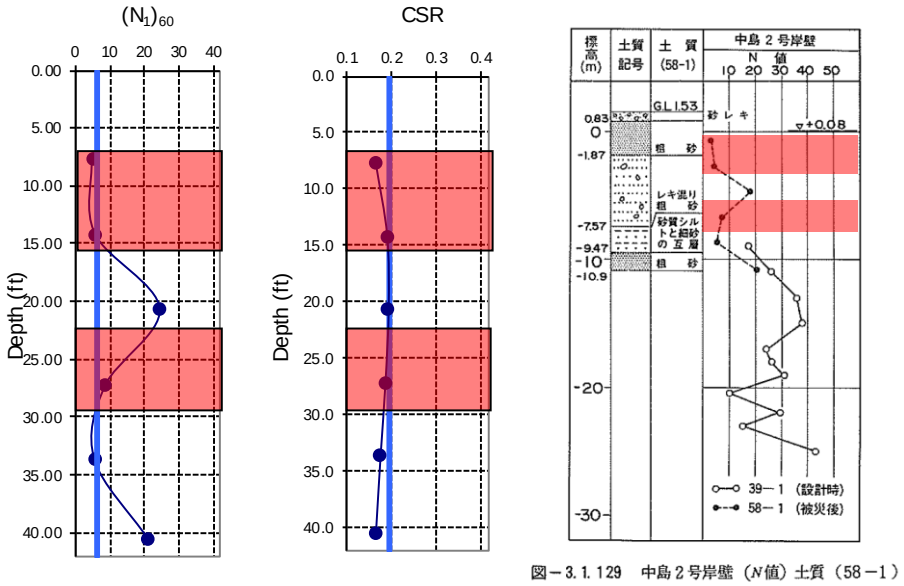
Fig. 8. Investigated sites in Akita Port

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	A	-		% Fines	2.0 ± 1.0	3.0	
Critical Depth Range	6.6 - 29.5	23.4		% PI	-	-	
Depth to GWT (ft)	4.8 ± 0.3	4.8					
σ_v (psf)	2231.8 ± 480.3	2840.4		N	4.7 ± 1.4	10.4	
σ_v' (psf)	1402.7 ± 243.9	1691.7		C_R	0.94	0.95	
a_{max} (g)	0.207 ± 0.031	0.205		C_S	1.00	1.00	
r_d	0.904 ± 0.076	0.940		C_B	1.00	1.00	
CSR	0.193 ± 0.034	0.213		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.23	1.12	
MSF		0.95		$(N_1)_{60}$	6.6 ± 2.0	13.5	
CSRN		0.22					



(c) Nakajima No.2 and No.3 (Liquefied)



図－3.1.129 中島2号岸壁（N値）土質（58－1）

0-0.7
0.7-3.40 sand
3.40-9.10 sand gravel
9.10-11.00 silt sand
11.00-12.43 sand

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
2.36	7.74	0.163	3	1.67	0.82	***5.0***	***1.0***	sand
4.32	14.18	0.190	4	1.35	0.90	***5.9***	***2.0***	sand gravel
6.31	20.69	0.193	18	1.16	0.96	*24.4*	*4.0*	sand gravel
8.32	27.30	0.186	7	1.03	1.00	***8.8***	***3.0***	sand gravel
10.25	33.64	0.175	5	0.94	1.00	*5.7*	*66.7*	silt sand
12.35	40.53	0.164	20	0.87	1.00	*21.1*	*8.0*	sand
					Mean:	6.6	2.0	
					St. Dev.	2.0	1.0	

Case number:	104
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Nakajima No. 2 (2)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Nakajima No. 2 (2) is 1.24 km away from strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 204 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 204 and 194 Gals are obtained. Geometric mean of 204 and 194 Gals is equal to 0.203 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 37 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 37 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

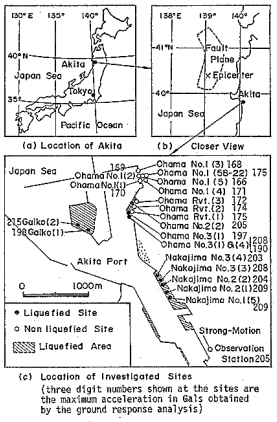
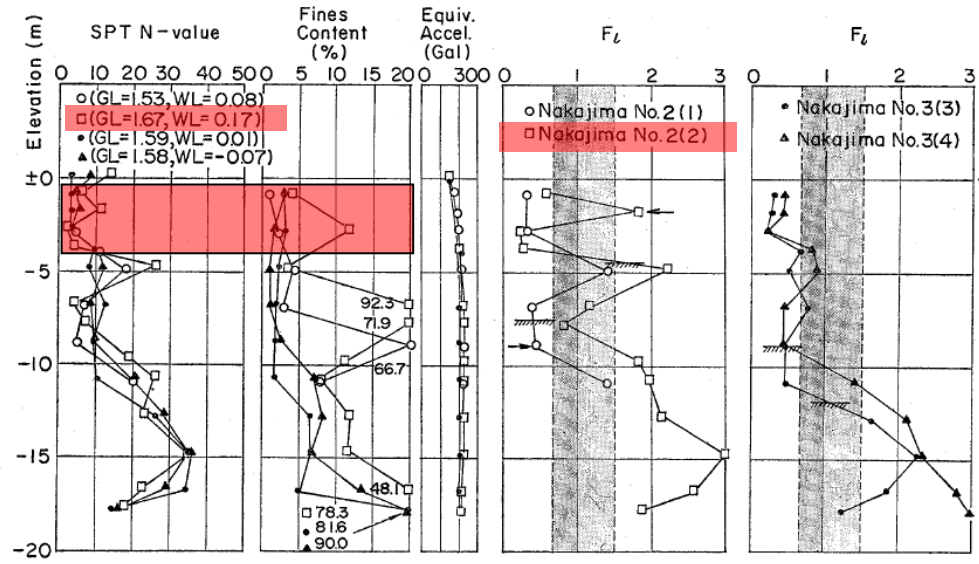


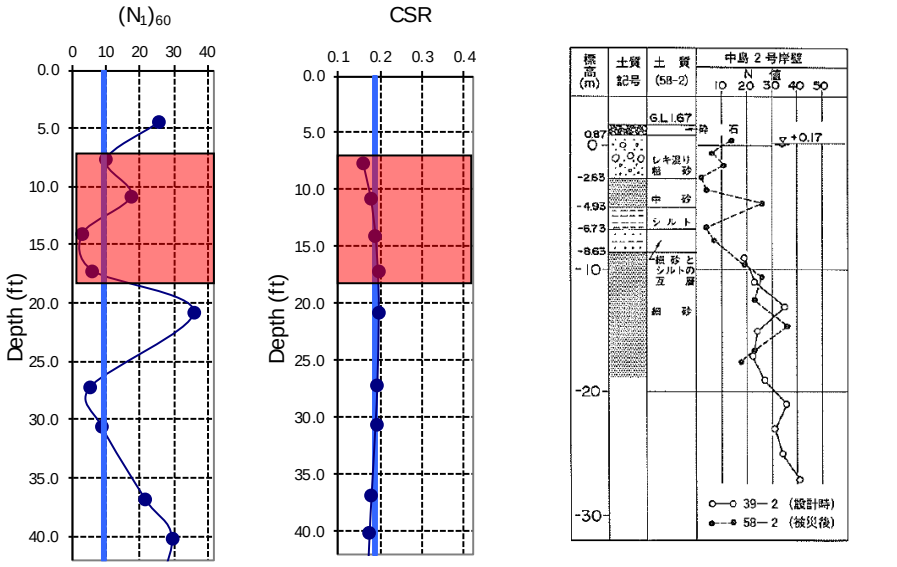
Fig. 8. Investigated sites in Akita Port

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	A	-		% Fines	8.0 ± 5.7	7.0	
Critical Depth Range	6.6 - 18.7	12.4		% PI	-	-	
Depth to GWT (ft)	4.9 ± 0.3	4.9					
σ_v (psf)	1515.7 ± 244.3	1482.9		N	5.9 ± 4.0	6.0	
σ_v' (psf)	1034.6 ± 121.2	1002.5		C_R	0.88	0.85	
a_{max} (g)	0.203 ± 0.030	0.205		C_S	1.00	1.00	
r_d	0.961 ± 0.056	0.980		C_B	1.00	1.00	
CSR	0.186 ± 0.031	0.191		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.43	1.50	
MSF		1.0		$(N_1)_{60}$	9.0 ± 6.2	9.3	
CSRN		0.2					



(c) Nakajima No. 2 and No. 3 (Liquefied)



図－3.1.130(b) 中島1号岸壁（N値）土質（58－2）

0.8-4.3	sand gravel
4.3-6.6	sand
6.6-8.4	silt
8.4-10.3	silt sand
10.3-...	sand

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
1.4	4.5	NA	14	1.98	0.76	*25.7*	-	sand gravel
2.3	7.7	0.159	6	1.68	0.82	***10.0***	***4.0***	sand gravel
3.3	10.8	0.179	11	1.51	0.86	***17.3***	***-***	sand gravel
4.3	14.0	0.190	2	1.38	0.90	***3.0***	***12.0***	sand gravel
5.2	17.2	0.196	4	1.28	0.93	***5.8***	***-***	sand
6.3	20.8	0.198	26	1.19	0.96	*36.0*	*3.0*	sand
8.3	27.3	0.195	4	1.06	1.00	*5.2*	*20.0*	silt
9.3	30.6	0.190	7	1.01	1.00	*8.6*	*20.0*	silt sand
11.2	36.9	0.181	19	0.93	1.00	*21.6*	*11.0*	sand
12.3	40.2	0.176	27	0.90	1.00	*29.5*	*8.0*	sand
14.3	46.9	0.168	24	0.84	1.00	*24.5*	*12.0*	sand
16.5	54.0	0.163	35	0.79	1.00	*33.5*	*12.0*	sand
18.2	59.8	0.162	23	0.75	1.00	*21.0*	*20.0*	sand
19.2	63.1	0.161	18	0.73	1.00	*16.0*	-	sand
					Mean:	9.0	8.0	
					St. Dev.	6.2	5.7	

Case number:	105
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Nakajima No. 3 (3)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Nakajima No. 3 (3) is 1.3 km away from strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 208 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 208 and 198 Gals are obtained. Geometric mean of 208 and 198 Gals is equal to 0.207 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 47 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 47 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

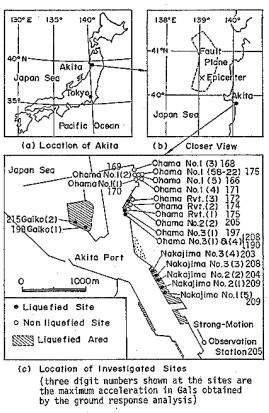
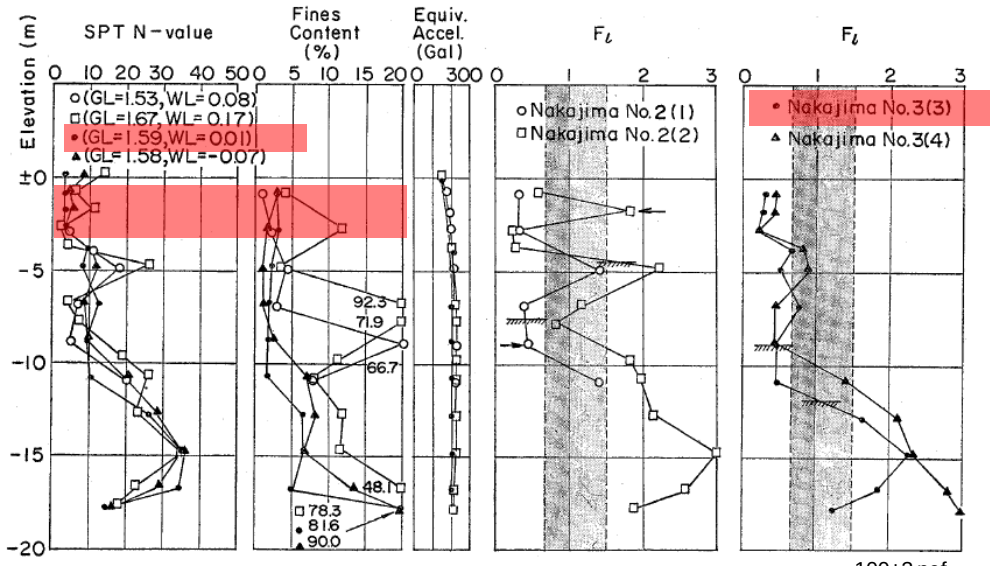


Fig. 8. Investigated sites in Akita Port

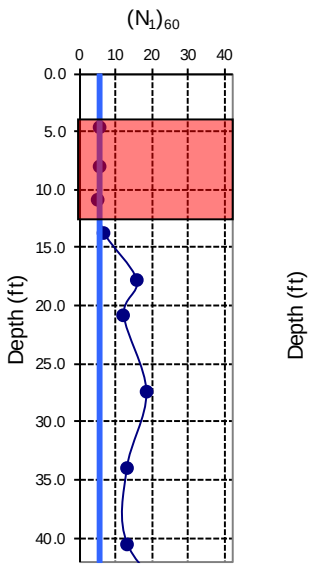
Summary of Data

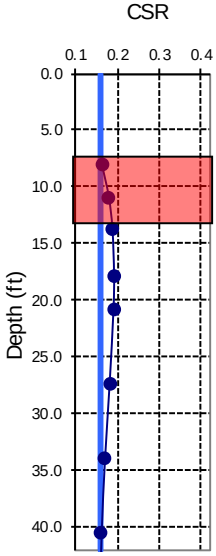
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	A	-		% Fines	3.0 ± 2.0	2.0	
Critical Depth Range	3.3 - 12.5	19.8		% PI	-	-	
Depth to GWT (ft)	5.2 ± 0.3	5.2					
σ_v (psf)	814.3 ± 169.4	2401.8		N	3.0 ± 0.1	7.3	
σ_v' (psf)	646.4 ± 76.6	1482.9		C_R	0.82	0.95	
a_{max} (g)	0.207 ± 0.031	0.205		C_S	1.00	1.00	
r_d	0.952 ± 0.037	0.960		C_B	1.00	1.00	
CSR	0.161 ± 0.030	0.205		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.81	1.21	
MSF		0.95		$(N_1)_{60}$	5.4 ± 0.2	10.2	
CSRN		0.21					

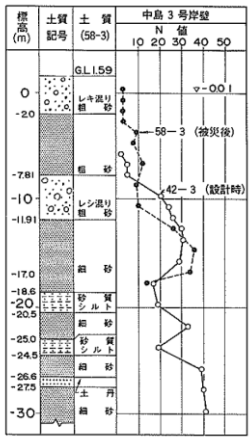


(c) Nakajima No. 2 and No. 3 (Liquefied)

γ_m=100±3 pcf
γ_s=110±3 pcf







図－3.1.127 中島3号岸壁（N値）土質（58－3）

0-3.59	gravel sand
3.59-3.59	sand
9.4-13.5	gravel sand
13.5-20.19	sand
20.19-22.09	silt
22.09-26.09	sand
26.09-26.59	silt
26.59-28.19	sand
28.19-29.09	silt
29.09-31.59	sand

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
1.4	4.7	NA	3	2.00	0.76	***5.6***	-	gravel sand
2.5	8.1	0.163	3	1.80	0.82	***5.4***	***3.0***	gravel sand
3.3	10.9	0.181	3	1.64	0.86	***5.1***	-	gravel sand
4.2	13.7	0.190	4	1.51	0.89	***6.6***	***3.0***	sand
5.4	17.8	0.194	10	1.37	0.93	***15.6***	-	sand
6.4	20.9	0.193	8	1.29	0.96	***12.1***	***5.0***	sand
8.4	27.5	0.183	13	1.16	1.00	***18.3***	***7.0***	sand
10.4	34.0	0.171	10	1.06	1.00	***12.9***	***9.0***	gravel sand
12.4	40.6	0.162	11	0.98	1.00	***13.1***	***11.0***	gravel sand
14.4	47.1	0.157	27	0.92	1.00	***30.1***	***13.0***	sand
16.3	53.4	0.155	36	0.87	1.00	***38.0***	***15.0***	sand
18.3	60.0	0.154	35	0.82	1.00	***35.0***	***17.0***	sand
19.3	63.4	0.154	15	0.80	1.00	***14.6***	***18.0***	sand
					Mean:	**5.4**	**3.0**	
					St. Dev.	**0.2**	**2.0**	assigned

Case number:	106
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Nakajima No. 3 (4)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Nakajima No. 3 (4) is 1.40 km awayfrom strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 203 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 203 and 193 Gals are obtained. Geometric mean of 203 and 193 Gals is equal to 0.202 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 38 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 38 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

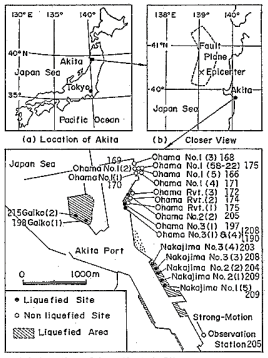
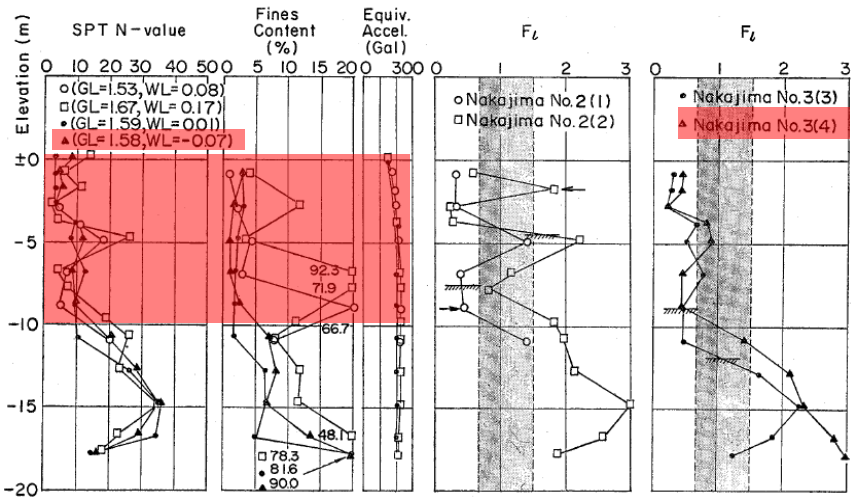


Fig. 8. Investigated sites in Akita Port

Summary of Data

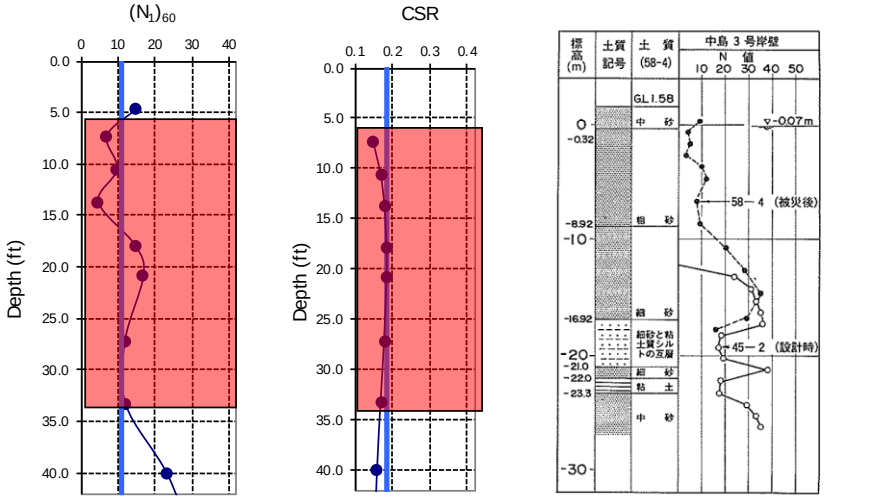
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	A	-		% Fines	2.0 ± 1.0	2.0	
Critical Depth Range	5.9 - 36.7	18.8		% PI	-	-	
Depth to GWT (ft)	5.4 ± 0.3	5.0					
σ_v (psf)	2504.9 ± 618.9	2276.5		N	7.8 ± 3.0	8.0	
σ_v' (psf)	1512.0 ± 300.7	1420.2		C_R	0.96	0.95	
a_{max} (g)	0.202 ± 0.030	0.205		C_S	1.00	1.00	
r_d	0.857 ± 0.087	0.960		C_B	1.00	1.00	
CSR	0.187 ± 0.035	0.206		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.18	1.24	
MSF		0.95		$(N_1)_{60}$	10.8 ± 4.2	11.5	
CSRN		0.21					



(c) Nakajima No.2 and No.3 (Liquefied)

γ_m=110±3pcf

γ_s=120±3 pcf



0-1.9	medium sand
1.9-10.5	sand
10.5-18.5	sand
18.5-22.58	silt
22.58-23.58	sand
23.58-24.88	clay
24.88-31.58	sand

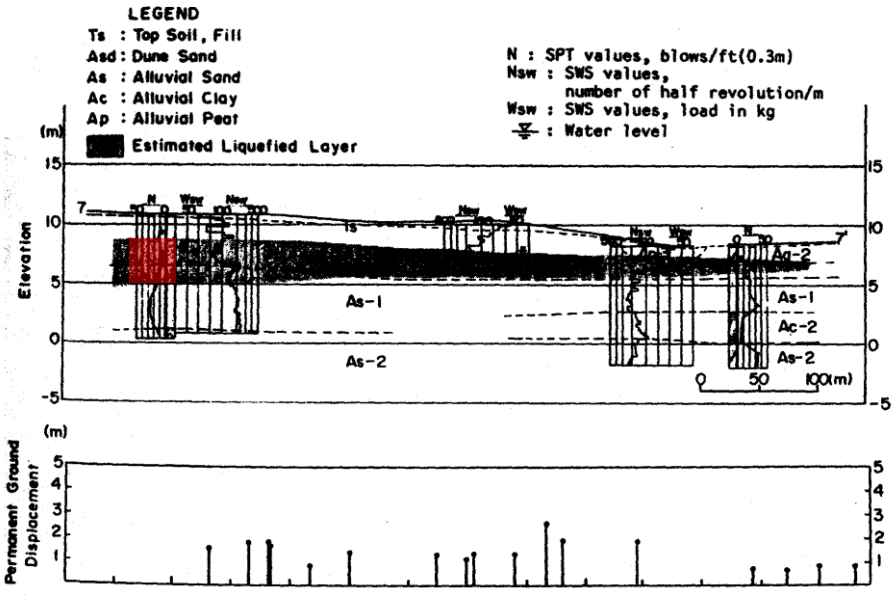
図－3.1.128 中島3号岸壁（N値）土質（58－4）

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
1.4	4.7	NA	8	2.00	0.76	*14.9*	-	medium sand
2.2	7.3	0.149	4	1.73	0.81	***6.8***	***3.0***	sand
3.2	10.5	0.170	6	1.54	0.86	***9.6***	**-**	sand
4.2	13.8	0.181	3	1.40	0.89	***4.6***	***2.0***	sand
5.5	17.9	0.187	10	1.27	0.94	***14.4***	**-**	sand
6.4	20.8	0.187	12	1.19	0.96	***16.7***	***1.0***	sand
8.3	27.2	0.180	9	1.07	1.00	***11.7***	***1.0***	sand
10.2	33.4	0.170	10	0.98	1.00	***11.9***	***3.0***	sand
12.2	40.1	0.160	21	0.90	1.00	*23.1*	*7.0*	sand
14.2	46.5	0.154	29	0.85	1.00	*29.8*	*8.0*	sand
16.3	53.4	0.150	37	0.79	1.00	*35.7*	*7.0*	sand
18.1	59.4	0.149	30	0.76	1.00	*27.6*	*14.0*	sand
19.3	63.3	0.148	17	0.73	1.00	*15.2*	*20.0*	sand
					Mean:	**10.8**	**2.0**	
					St. Dev.	**4.2**	**1.0**	

Case number:	107
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (M)
Location:	Noshiro N-7
References:	Hamada (1992)
Nature of Failure:	Lateral spreading
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area The site is gently sloped (1-3%). A particulary large displacement occurred on the slopes of Maeyama, a sand hill about 10 m higher than the surrounding area. On the northern slope of the hill, displacements began near the top and extended horizontally for more than 300 m in a northerly direction The maximum surface acceleration at Akita City 115 km from the epicenter was 0.235 and 0.219 g. The PGA at the site, 100 km away from the epicenter was estimated as 0.25 g.

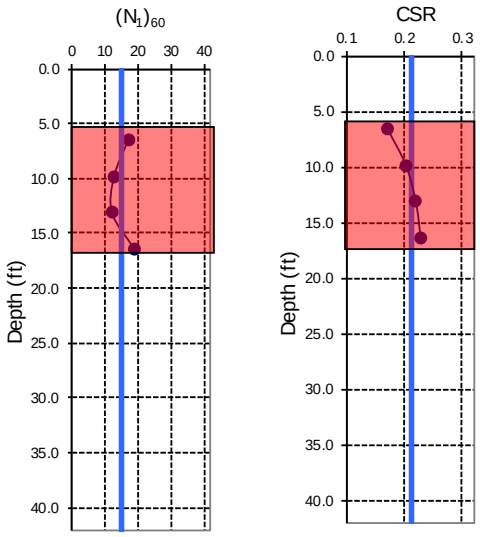
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	yes		D_{50}	0.250 ± 0.050		
Data Class	B			% Fines	1.0 ± 2.0	1.0	
Critical Depth Range	6.6 - 16.4	11.5		% PI			
Depth to GWT (ft)	5.7 ± 0.3	5.6					
σ_v (psf)	1320.5 ± 198.4	1148.7		N	9.8 ± 2.1	9.8	
σ_v' (psf)	962.3 ± 98.8	793.6		C_R	0.87	0.85	
a_{max} (g)	0.250 ± 0.075	0.25		C_S	1.00	1.00	
r_d	0.95 ± 0.052	0.98		C_B	1.00	1.00	
CSR	0.212 ± 0.066	0.232		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.48	1.60	
MSF		0.95		$(N_1)_{60}$	15.3 ± 3.3	16.2	
CSRN		0.222					



γ_m=110±3 pcf

γ_s=120±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.0	6.6	0.171	10.0	1.77	0.80	***17.1***
3.0	9.8	0.202	8.0	1.56	0.85	***12.9***
4.0	13.1	0.219	8.0	1.42	0.89	***12.2***
5.0	16.4	0.228	13.0	1.30	0.92	***19.0***
					Mean:	15.3
					St. Dev.	3.3

Case number:	108
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Ohama No. 2 (2)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Ohama No. 2 (2) is 2.53 km away from strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 205 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 205 and 195 Gals are obtained. Geometric mean of 205 and 195 Gals is equal to 0.204 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data is obtained. Site response by Iai et.al. exists. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 38 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 38 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

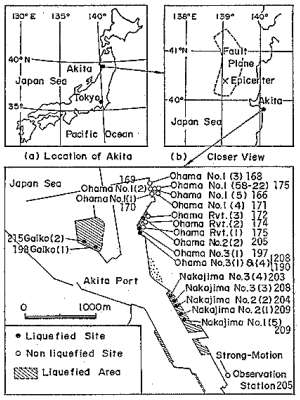
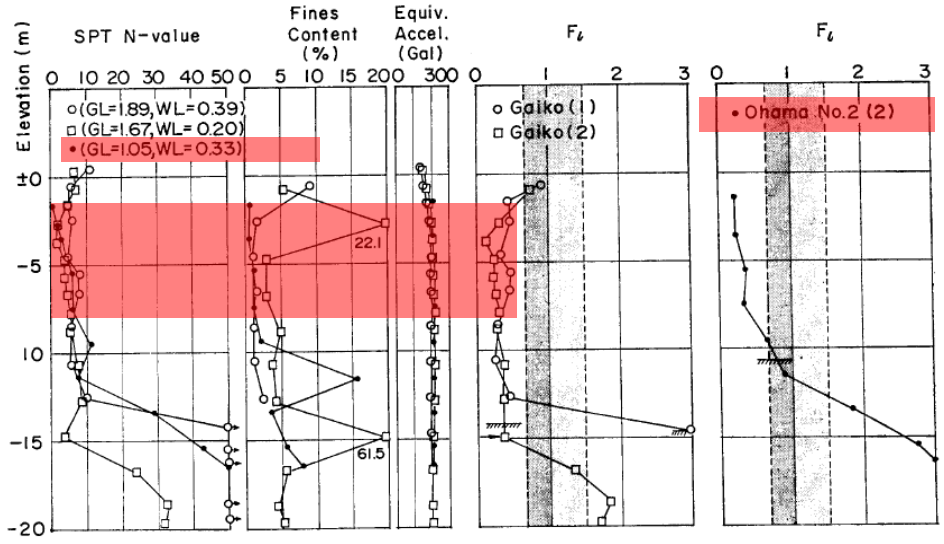


Fig. 8. Investigated sites in Akita Port

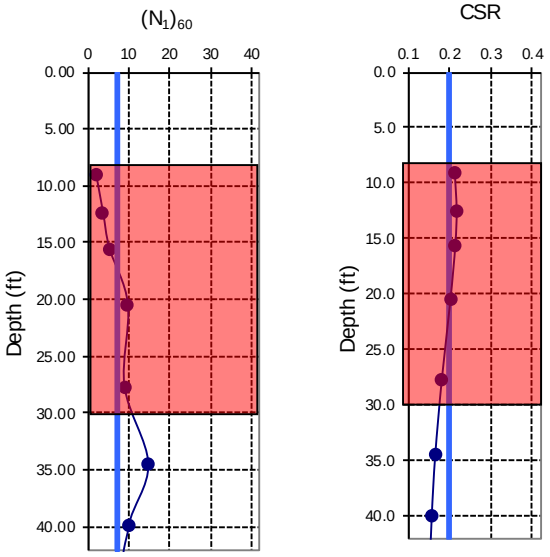
Summary of Data							
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	- ± -	-	
Data Class	A	-		% Fines	1.1 ± 0.5	2.0	
Critical Depth Range	7.2 - 37.1	17.1		% PI	-	-	
Depth to GWT (ft)	2.4 ± 0.3	2.4					
σ_v (psf)	2412.4 ± 550.6	2088.5		N	4.6 ± 2.9	3.3	
σ_v' (psf)	1177.9 ± 244.8	1169.6		C_R	0.97	0.95	
a_{max} (g)	0.204 ± 0.031	0.205		C_S	1.00	1.00	
r_d	0.728 ± 0.090	0.960		C_B	1.00	1.00	
CSR	0.198 ± 0.039	0.229		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.34	1.43	
MSF		0.95		$(N_1)_{60}$	7.3 ± 4.6	5.4	
CSRN		0.23					

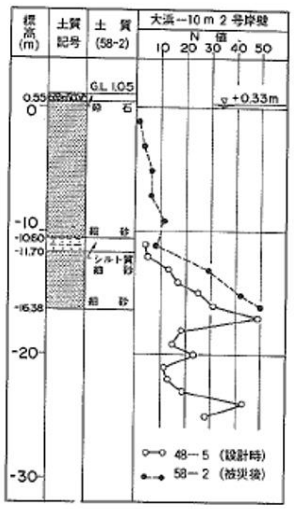


(a) Gaiko and Ohama No.2 (Liquefied)

γ_m=100±3 pcf

γ_s=110±3 pcf





図－3.1.115 大浜－10m 2号岸壁 (N値) 土質 (58－2)

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
2.77	9.09	0.213	1	1.95	0.84	**2.0**	**1.0**	sand
3.80	12.47	0.217	2	1.72	0.88	**3.7**	**-**	sand
4.75	15.58	0.214	3	1.56	0.91	**5.2**	**0.5**	sand
6.25	20.51	0.202	6	1.39	0.96	**9.7**	**1.0**	sand
8.45	27.72	0.182	6	1.21	1.00	**8.8**	**1.0**	sand
10.53	34.55	0.167	11	1.09	1.00	**14.6**	**2.0**	sand
12.17	39.93	0.159	8	1.02	1.00	10.0	16.0	silt
13.55	44.46	0.154	9	0.97	1.00	10.6	4.0	sand
13.95	45.77	0.153	29	0.96	1.00	33.8	6.0	sand
16.55	54.30	0.150	43	0.88	1.00	46.2	8.0	sand
					Mean:	**7.3**	**1.1**	
					St. Dev.	**4.6**	**0.5**	

Case number:	109
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (Mw by USGS, NISEE)
Location:	Ohama No. Rvt. (1)
References:	Iai. et. al. (1989)
Nature of Failure:	Liquefied and nonliquefied areas are defined by the boundary lines between artifically filled areas and the areas of the Quaternary sediments which are shown in photos provided by Iai et. al. (1989)
Comments:	Tsuchida et. al (1985) shows the liquefied and non liquefied sites on Akita Port in Figure 8. Ohama No. Rvt. (1) is 2.43 km away from strong ground motion station. SMAC_B2 type accelerograph measured an acceleration value of 190 and 205 Gals in NS and EW directions. "These two values are used for computing the rock outcrop motion. Then this rock outcrop motion is used as input for the earthquake response analysis at several sites." (Kurata et. Al. 1983) In Figure 8 three digit numbers shows the maximum acceleration values obtained from ground response analysis. However this procedure is done for one component which is 175 Gals (EW) In order ro take into account multi-directional effects site response by Iai et.al. was scaled, maximum acceleration values of 175 and 166 Gals are obtained. Geometric mean of 175 and 166 Gals is equal to 0.174 g is assigned to site. SPT procedures were not clearly documented. However, on page 66 in Iai et. al. for comparison with Seed et. al. 1985, an average Japanese SPT energy ratio of 73% was reported to be used. On the basis of this, with some uncertainty Iai et. al. N values may be adopted as N_{73} values. Note that Seed et. al. (1985) adopted an energy ratio of 73% for liquefied cases. The rationale behind this choice could be field data which is unknown to us or simply a conservative engineering judgement. Soil profile data is missing just N values and FC exist. From Technical Report of the Port and Horbour Research Institute Ministry of Transport, Japan No. 511 Marc. 1985 the borehole data could not be obtained. Site response by Iai et.al. exists however soil profile not known. There is an ambiguity if the site is free field or not in the vicinty of Akita Port Channel which is 10 m in depth. From Figure 8 the site is 76 m away from channel. Note that the location of the site may not be correctly located in Figure 8 since the scale is very small and the coordinates of the borehole are not available. Additionally Quay walls are believed to be located 76 m away from site. Presence of Quay walls and due to proximity to the Akita Port Channel which 10 m free field stress boundary conditions may not be fulfilled. Hence cases within 20 m to the Quay walls and channel are eliminated.

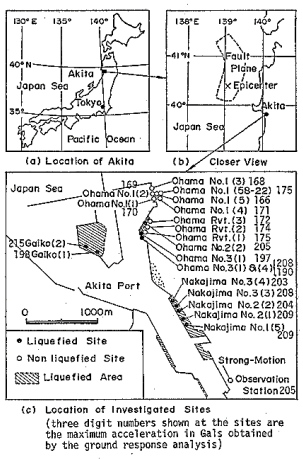
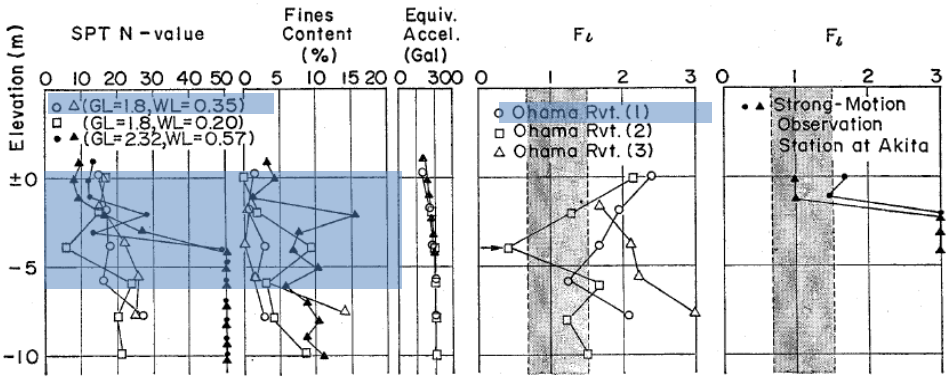


Fig. 8. Investigated sites in Akita Port

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	- ± -	-	
Data Class	B	-		% Fines	3.0 ± 1.0	2.0	
Critical Depth Range	11.6 - 24.8	14.9		% PI		-	
Depth to GWT (ft)	4.8 ± 3.0	4.8					
σ_v (psf)	2251.1 ± 278.5	1796.1		N	17.2 ± 2.1	15.8	
σ_v' (psf)	1412.3 ± 224.5	1148.7		C_R	0.94	0.95	
a_{max} (g)	0.174 ± 0.026	0.205		C_S	1.00	1.00	
r_d	0.973 ± 0.076	0.970		C_B	1.00	1.00	
CSR	0.175 ± 0.038	0.200		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.22	1.28	
MSF		0.95		$(N_1)_{60}$	24.0 ± 2.9	23.5	
CSRN		0.19					

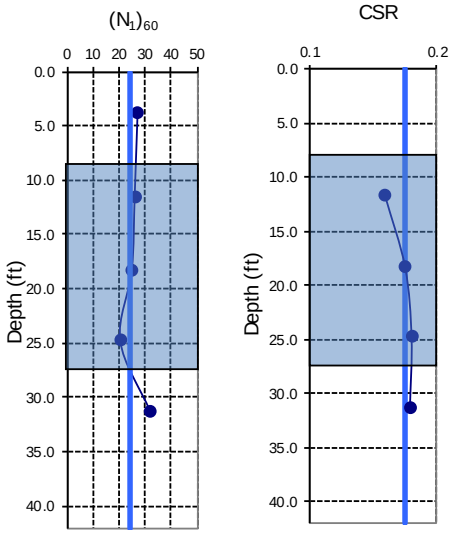


γ_m=120±3 pcf

γ_s=125±3 pcf

(e) **Ohama No. 2 Former Revetment and the strong-motion observation station (Nonliquefied)**

Fig. 9. Ground conditions and resulted liquefaction potentials at Akita Port during 1983 Nihonkai-Chubu Earthquake

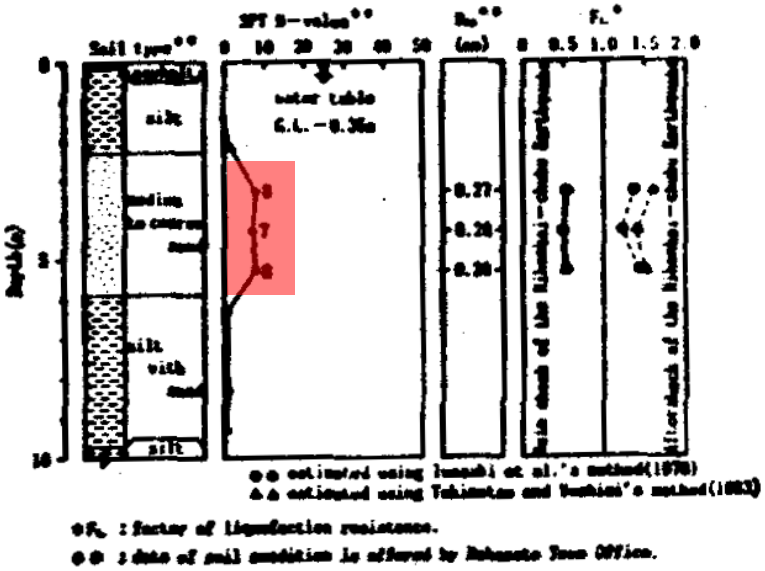


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
1.18	3.9	NA	15	2.00	0.75	27.3	3.0
3.54	11.6	0.160	17	1.46	0.87	***26.2***	***2.0***
5.58	18.3	0.176	18	1.22	0.94	***25.1***	***4.0***
7.56	24.8	0.181	16	1.08	0.99	***20.8***	***3.0***
9.53	31.3	0.179	27	0.97	1.00	32.0	4.0
					Mean:	**24.0**	**3.0**
					St. Dev.	**2.9**	**1.0**

Case number:	110
Earthquake:	1983 Nihonkai-Chubu
Magnitude:	7.7 (M)
Location:	Nakazata Town (Takeda Elementary School)
References:	Yasuda and Tohno (1988)
Nature of Failure:	Sand boils, settled structures.
Comments:	The Nihonkai-chubu Earthquake, with a magnitude of 7.7 occurred on May 26, 1983 in the northwest Japan and produced liquefaction sites in the coastal plain of Akita and Aomori Prefectures. About one month later, a magnitude 7.1 earthquake again hit the area and caused liquefaction at the Takeda Elementary School in Nakazato Town. The school was built on the old bed of the Iwaki River. During the main shock on May 26, the ground beneath the schoolhouses (two-story wood structures) and spewed out sand and water then was submerged. The aftershock of June 21 again caused sand and water to swep out from the ground. According to some teachers, the area where sand volcanoes were observed almost coincided with the area where sand volcaones had been caused by the ,main shock. The maximum surface acceleration was assumed to be 0.278 and 0.109 g for the main and after shock respectively based on the values recorded at the Tsugaru-ohashi Bridge about 6 km from the school.

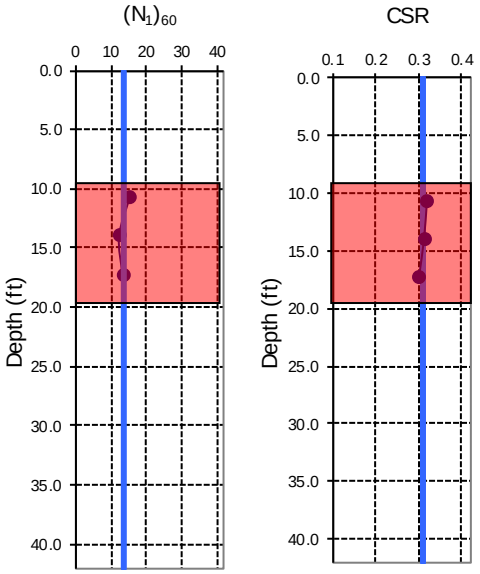
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.240 ± 0.050		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	8.2 - 21.3	14.1		% PI			
Depth to GWT (ft)	1.1 ± 0.4	1.3					
s_v (psf)	1680.6 ± 254.9	1691.7		N	7.9 ± 0.9	7.4	
s_v' (psf)	831.0 ± 123.3	877.2		C_R	0.90	0.95	
a_{max} (g)	0.278 ± 0.083	0.28		C_S	1.00	1.00	
r_d	0.85 ± 0.064	0.97		C_B	1.00	1.00	
CSR	0.312 ± 0.097	0.397		C_E	1.22	1.22	
Equivalent Magnitude	7.7	7.7		C_N	1.60	1.56	
MSF		0.95		$(N_1)_{60}$	13.8 ± 1.5	13.3	
CSRN		0.381					



γ_m=100±3 pcf

γ_s=115±3 pcf

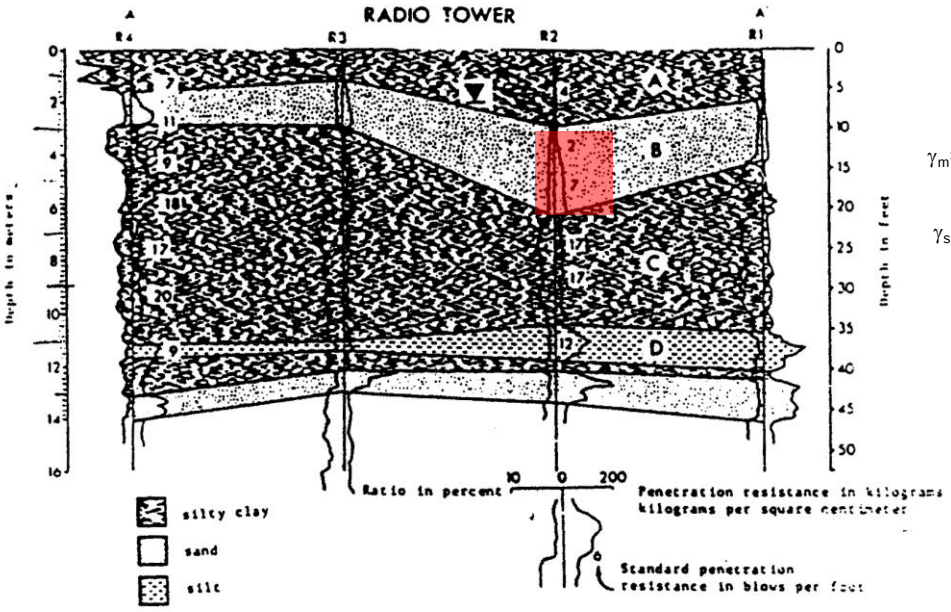


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.7	0.322	8.0	1.85	0.86	***15.5***
4.3	13.9	0.315	7.0	1.64	0.90	***12.5***
5.3	17.2	0.303	8.0	1.48	0.93	***13.4***
					Mean:	13.8
					St. Dev.	1.5

Case number	111
Earthquake:	1987 Elmore Ranch
Magnitude:	6.2 (Mw)
Location:	Radio Tower B1
References:	James Gerard Bierschwale (1984), Andrus and Stokoe (1997) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand PGA was recorded at the surface and at 6.7 m depth of Wildlife site as 0.126 and 0.077g respectively. The epicentral distance is ~25 km sligthly further than Wildlife site.The earthquake is very poorly monitored. Only one seismogram is available. Andrus and Stoke (NCEER, 97) predicted the PGA at the site as 0.11g. A PGA of 0.09 g was adopted for these studies. Energy ratio ~68 %

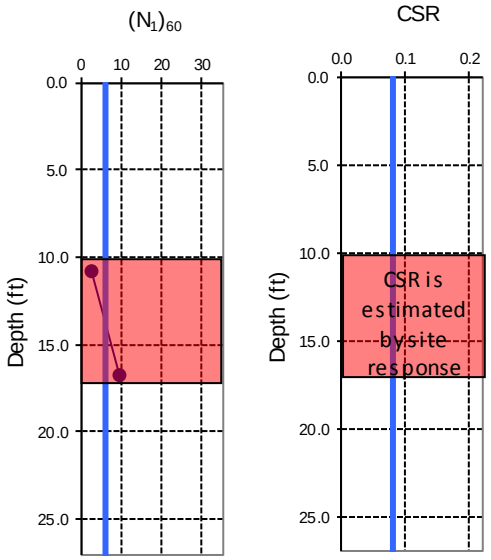
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.074 ± 0.037		
Data Class	B			% Fines	43.5 ± 29.0	64.0	
Critical Depth Range	9.8 - 18.0	11.2		% PI			
Depth to GWT (ft)	6.6	6.9					
σ_v (psf)	1505.1 ± 160.1	1294.9		N	4.5 ± 3.4	2.0	
σ_v' (psf)	1044.5 ± 79.5	1044.3		C_R	0.86	0.85	
a_{max} (g)	0.090 ± 0.014	0.09		C_S	1.00	1.00	
r_d	0.97 ± 0.032	0.96		C_B	1.00	1.00	
CSR	0.082 ± 0.013	0.069		C_E	1.13	1.13	
Equivalent Magnitude	6.2	6.2		C_N	1.42	1.49	
MSF		1.40		$(N_1)_{60}$	6.2 ± 4.7	2.9	
CSRN		0.046					



γ_m=100±3 pcf

γ_s=115±3 pcf



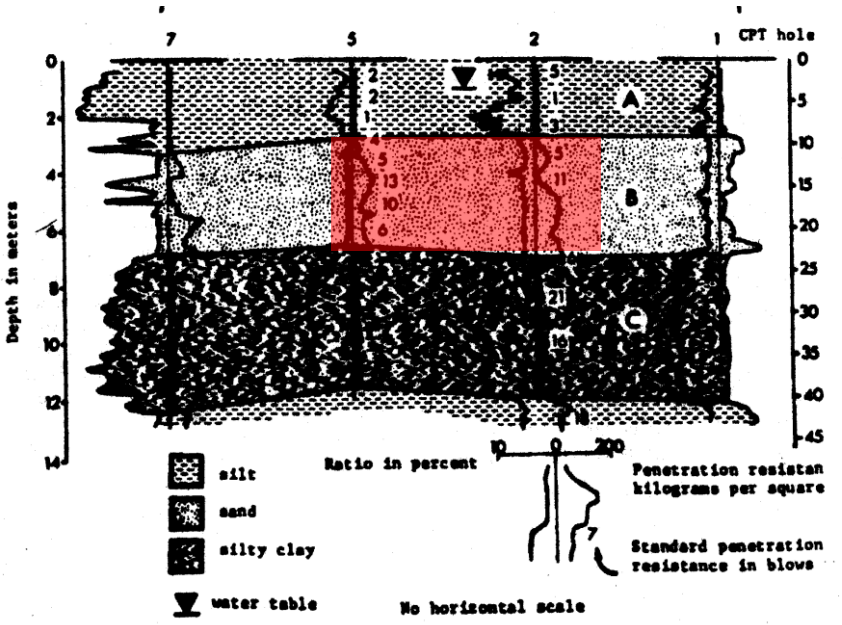
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.8		2	1.55	0.82	***2.9***
5.1	16.7		7	1.33	0.89	***9.5***
					Mean:	6.2
					St. Dev.	4.7

Depth (m)	FC	sand	silt	clay	D50
3.3	***64.0***	***36.0***	***59.0***	***5.0***	***0.047***
5.1	***23.0***	***77.0***	***22.0***	***1.0***	***0.100***
23.15	*18.0*	*82.0*	*13.0*	*5.0*	*0.125*
	43.5	56.5	40.5	3.0	0.074
	29.0	29.0	26.2	2.8	0.037

Case number:	112
Earthquake:	1987 Elmore Ranch
Magnitude:	6.2 (Mw)
Location:	Wildlife B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	Site has not liquefied. Neither pore pressure ratios exceeding 1 nor surface evidence of liquefaction.
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The site is very closely monitored. Strong ground motion seismometers are available at the surface and at 6.7 m depth. Piezometers were installed almost at every meter depth along the potentially liquefiable layer of thickness 4 m. A: very loose and very soft interbedded micaceous sandy silt, silt and clayey silts (0-2.5 m) flood plain deposits B: very loose to loose moderately sorted sandy silt (2.5-6.8m); point bar deposits C: silty clay (medium to very stiff) and clayey silt (medium to stiff); lacustrine deposit. PGA was recorded at the surface and at 6.7 m depth as 0.126 and 0.077g. respectively. The epicentral distance is 22.9 km. Energy ratio ~68 %

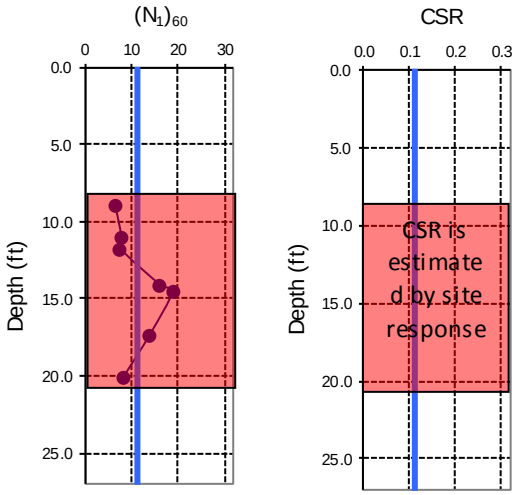
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.109 ± 0.038		
Data Class	A			% Fines	26.2 ± 7.6	30.0	
Critical Depth Range	9.0 - 22.0	15.1		% PI			
Depth to GWT (ft)	3.0	3.9					
σ_v (psf)	1800.0 ± 262.9	1817.0		N	7.9 ± 3.5	7.1	
σ_v' (psf)	1020.0 ± 131.5	1127.8		C_R	0.88	0.95	
a_{max} (g)	0.130 ± 0.020	0.13		C_S	1.00	1.00	
r_d	0.75 ± 0.035	0.93		C_B	1.00	1.00	
CSR	0.112 ± 0.018	0.138		C_E	1.13	1.13	
Equivalent Magnitude	6.2	6.2		C_N	1.44	1.36	
MSF		1.40		$(N_1)_{60}$	11.3 ± 5.0	10.3	
CSRN		0.092					



γ_m=100±3 pcf

γ_s=120±3 pcf



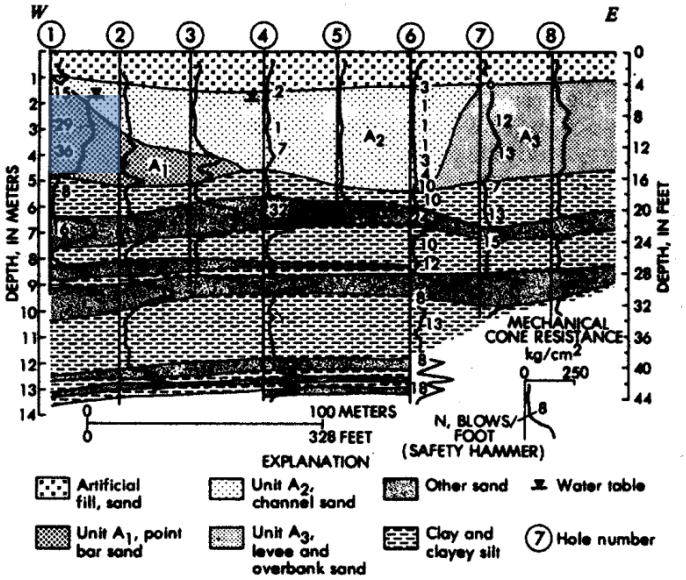
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.75	9.0		4	1.81	0.79	***6.5***	Hole-5
3.35	11.0		5	1.67	0.82	***7.8***	Hole-2
3.6	11.8		5	1.62	0.83	***7.6***	Hole-5
4.3	14.1		11	1.50	0.86	***16.2***	Hole-2
4.45	14.6		13	1.48	0.87	***18.9***	Hole-5
5.3	17.4		10	1.37	0.90	***14.0***	Hole-5
6.15	20.2		6	1.28	0.93	***8.1***	Hole-5
					Mean:	11.3	
					St. Dev.	5.0	

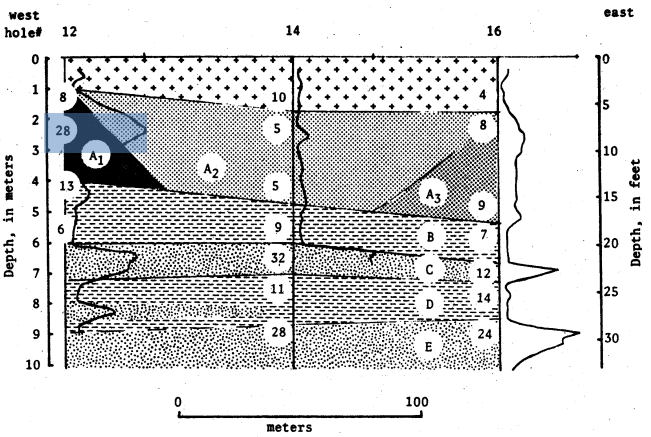
Depth (ft)	FC	Sand	Silt	Clay	D50	
10.5	***34.0***	***66.0***	***34.0***	***0.0***	***0.176***	Hole-2
11.2	***17.0***	***83.0***	***17.0***	***0.0***	***0.096***	Hole-2
13.5	***27.0***	***73.0***	***26.0***	***1.0***	***0.090***	Hole-2
14.2	***20.0***	***80.0***	***20.0***	***0.0***	***0.100***	Hole-2
18.3	***33.0***	***67.0***	***29.0***	***4.0***	***0.082***	Hole-2
22.8	*18.0*	*82.0*	*17.0*	*1.0*	*0.102*	Hole-2
Mean:	26.2	73.8	25.2	1.0	0.109	
St. Dev.	7.6	7.6	6.8	1.7	0.038	

Case number:	113
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Heber Road A1
References:	Youd et al. (1983), "Liquefaction sites, Imperial Valley, CA Andrus and Stokoe, 1997
Nature of Failure:	No surface manifestation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site is ~60 km to the epicenter. Due to the lack of significant surface trace of the fault rupture, it is difficult to estimate the closest distance to the fault rupture. A soil PGA of 0.36g was recorded at El Centro array site # 5, which is ~15 km north of Heber Road site. Based on attenuation relationships, site response analyses and available recordings, a PGA in the range of 0.12-0.16 was estimated. Based on site response analyses results, a PGA value of 0.16 g was adopted. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

Summary of Data

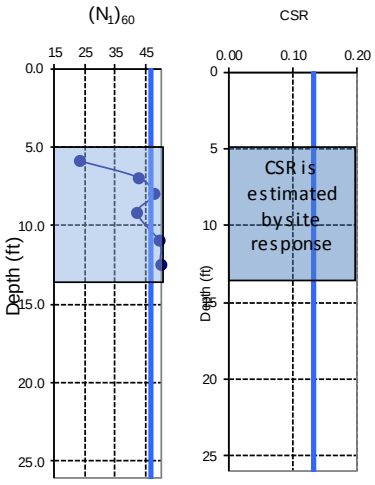
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.111 ± 0.007		
Data Class	A			% Fines	13.0 ± 3.9	12.0	
Critical Depth Range	5.9 - 16.4	11.2		% PI			
Depth to GWT (ft)	5.9	6.9					
σ_v (psf)	1246.7 ± 220.1	1294.9		N	32.9 ± 2.6	30.4	
σ_v' (psf)	919.2 ± 112.6	1044.3		C_R	0.82	0.85	
a_{max} (g)	0.156 ± 0.023	0.20		C_S	1.00	1.00	
r_d	0.96 ± 0.022	0.96		C_B	1.00	1.00	
CSR	0.132 ± 0.022	0.154		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.52	1.30	
MSF		1.29		$(N_1)_{60}$	46.5 ± 3.7	37.8	
CSRN		0.113					





γ_m=100±3 pcf

γ_s=125±3 pcf

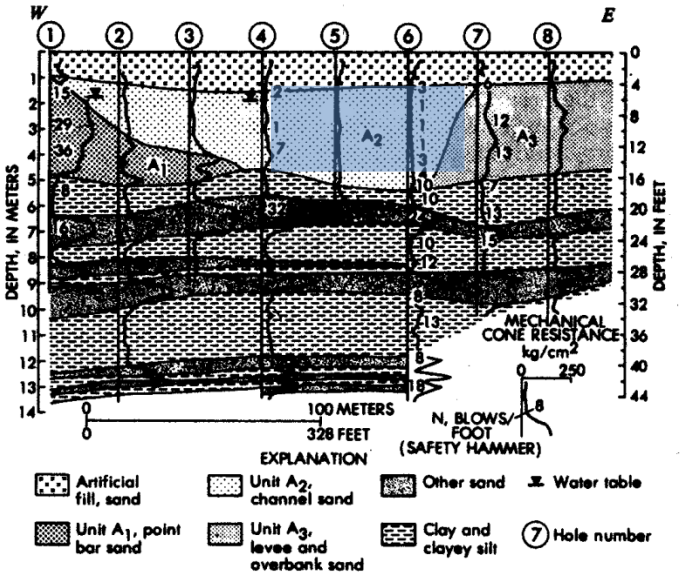


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.8	5.9		15	1.89	0.73	23.4	BH-1
2.12	7.0		28	1.80	0.75	***42.7***	BH-12
2.45	8.0		32	1.71	0.77	***47.8***	BH-1 Donut
2.8	9.2		29	1.63	0.79	***42.4***	BH-1 Donut
3.35	11.0		35	1.53	0.82	***49.7***	BH-1
3.8	12.5		36	1.45	0.84	***49.9***	BH-1
					Mean:	46.5	
					St. Dev.	3.7	

Case number:	114
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Heber Road A2
References:	Youd et al. (1982), "Liquefaction sites, Imperial Valley, CA Andrus and Stokoe, 1997
Nature of Failure:	No surface manifestation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site ~60 km to the epicenter. Due to the lack of significant surface trace of the fault rupture, it is difficult to estimate the closest distance to the fault rupture. A soil PGA of 0.36g was recorded at El Centro array site # 5, which is ~15 kms north of Heber Road site. Based on attenuation relationships, site response analyses and available recordings, a PGA in the range of 0.12-0.16 was estimated. Site response analysis results suggests a PGA value of 0.15 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

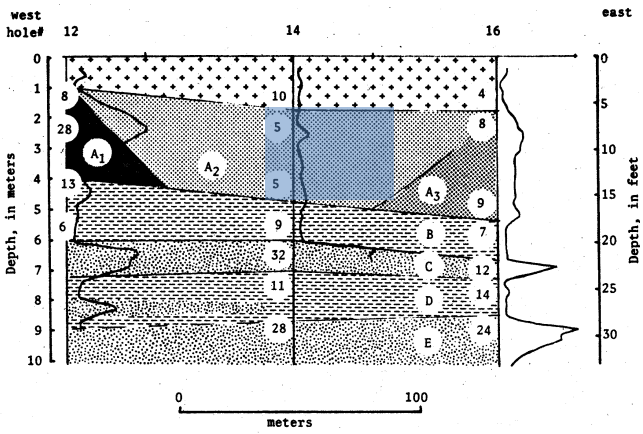
Summary of Data

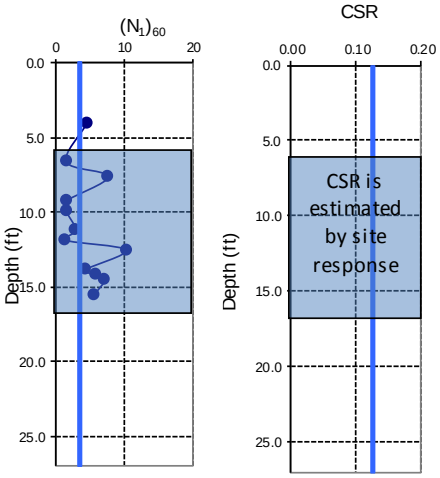
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.116 ± 0.013		
Data Class	A			% Fines	20.9 ± 5.0	18.0	
Critical Depth Range	6.0 - 15.1	12.1		% PI			
Depth to GWT (ft)	5.9	5.9					
σ_v (psf)	1103.0 ± 167.6	1420.2		N	2.4 ± 1.5	2.0	
σ_v' (psf)	812.3 ± 76.9	1044.3		C_R	0.81	0.85	
a_{max} (g)	0.150 ± 0.023	0.15		C_S	1.00	1.00	
r_d	0.95 ± 0.024	0.96		C_B	1.00	1.00	
CSR	0.126 ± 0.021	0.127		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.61	1.51	
MSF		1.29		$(N_1)_{60}$	3.5 ± 2.2	2.9	
CSRN		0.093					



γ_m=100±3 pcf

γ_s=110±3 pcf



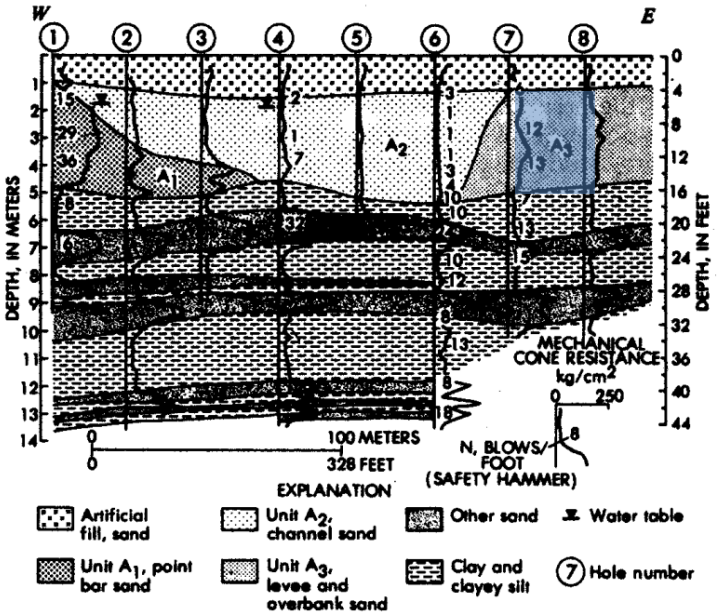


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.2	3.9		3	2.00	0.68	4.6	BH-6
2	6.6		1	1.84	0.74	***1.6***	BH-6
2.3	7.5		5	1.78	0.76	7.7	BH-14
2.8	9.2		1	1.68	0.79	***1.5***	BH-6
3	9.8		1	1.65	0.80	***1.5***	BH-4
3.4	11.2		2	1.59	0.82	***3.0***	BH-5 Donut
3.6	11.8		1	1.56	0.83	***1.5***	BH-6
3.8	12.5		7	1.53	0.84	10.2	BH-4
4.2	13.8		3	1.48	0.86	***4.3***	BH-6
4.3	14.1		4	1.47	0.86	***5.7***	BH-5 Donut
4.4	14.4		5	1.46	0.87	***7.2***	BH-14
4.7	15.4		4	1.42	0.88	***5.7***	BH-6
					Mean:	3.5	
					St. Dev.	2.2	

Case number:	115
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Heber Road A3
References:	Youd et al. (1982), "Liquefaction sites, Imperial Valley, CA
Nature of Failure:	No surface manifestation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Heber Road site ~60 km to the epicenter. Due to the lack of significant surface trace of the fault rupture, it is difficult to estimate the closest distance to the fault rupture. A soil PGA of 0.36 g was recorded at El Centro array site # 5, which is ~15 kms north of Heber Road site. Based on attenuation relationships, site response analyses and available recordings, a PGA in the range of 0.12-0.16 g was estimated. Site response analysis results suggest a PGA value of 0.13 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T- Driver and safety hammer for the drillings in 1979 and 1981. Donut Hammer with rope cathead was used for the drillings in in 1982. Energy ratios are adopted as 68 and 66 % for safety and donut hammers respectively.

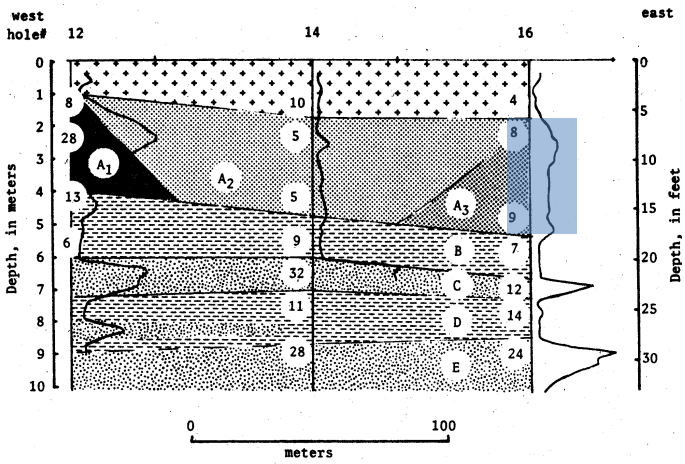
Summary of Data

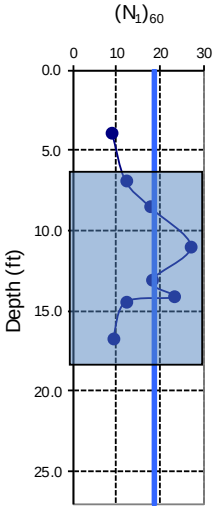
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.094 ± 0.004		
Data Class	A			% Fines	25.3 ± 5.2	25.0	
Critical Depth Range	5.9 - 16.1	13.1		% PI			
Depth to GWT (ft)	5.9	5.9					
σ_v (psf)	1200.8 ± 204.8	1649.9		N	13.0 ± 4.2	13.0	
σ_v' (psf)	883.5 ± 101.2	1169.6		C_R	0.82	0.85	
a_{max} (g)	0.130 ± 0.020	0.13		C_S	1.00	1.00	
r_d	0.95 ± 0.025	0.95		C_B	1.00	1.00	
CSR	0.109 ± 0.018	0.115		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.55	1.29	
MSF		1.29		$(N_1)_{60}$	18.6 ± 6.0	16.2	
CSRN		0.082					

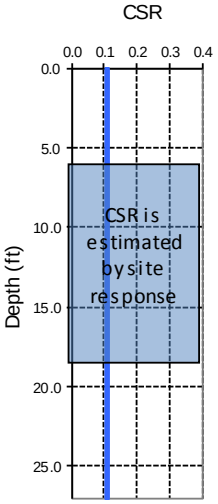


γ_m=100±3 pcf

γ_s=120±3 pcf





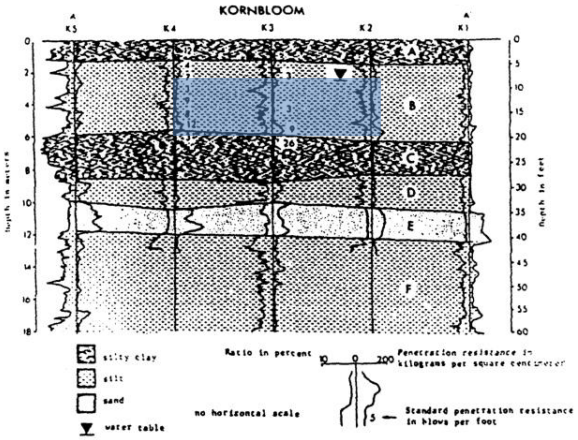


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
1.2	3.9		6	2.00	0.68	9.2	BH-7
2.1	6.9		8	1.81	0.75	***12.3***	BH-16
2.6	8.5		12	1.69	0.78	***17.9***	BH-7
3.35	11.0		19	1.55	0.82	***27.4***	BH-8
4	13.1		13	1.45	0.85	***18.2***	BH-7
4.3	14.1		17	1.41	0.86	***23.5***	BH-8
4.4	14.4		9	1.40	0.87	***12.4***	BH-16
5.1	16.7		7	1.32	0.89	9.4	BH-7
					Mean:	18.6	
					St. Dev.	6.0	

Case number:	116
Earthquake:	1987 Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Kornbloom B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	No surface evidence of liquefaction
Comments:	Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: silty clay, stiff-lacustrine and/or flood plain deposit B: silty and sandy silt-fluvial and perhaps fluvial deltaic deposition C: silty clay to clay, stiff-lacustrine and/or flood plain deposit D: interbedded silt and sand-loose to medium dense E: Uniform sand with some silt-medium dense to dense F: interbedded silt and sand - loose to medium dense. D-F :fluvial and perhaps fluvial-deltaic deposition PGA was recorded at the Kornbloom road as 0.14 g (temp. seismograph) Site response analysis results suggest a slightly higher PGA of 0.17 g Energy ratio ~68 %

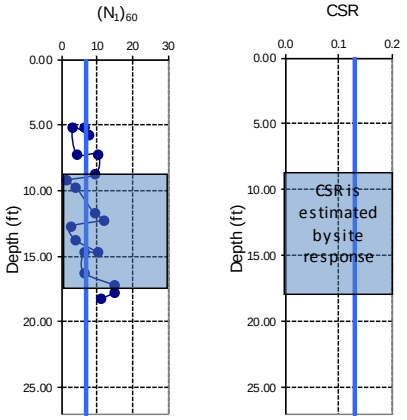
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.045 ± 0.008		
Data Class	A			% Fines	83.0 ± 9.6	92.0	
Critical Depth Range	9.0 - 17.5	14.1		% PI			
Depth to GWT (ft)	9.0	8.9					
σ_v (psf)	1410.0 ± 172.7	1608.2		N	5.4 ± 3.1	5.0	
σ_v' (psf)	1144.8 ± 88.1	1294.9		C_R	0.85	0.85	
a_{max} (g)	0.174 ± 0.026	0.17		C_S	1.00	1.00	
r_d	0.94 ± 0.030	0.95		C_B	1.00	1.00	
CSR	0.131 ± 0.021	0.132		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.36	1.29	
MSF		1.29		$(N_1)_{60}$	7.1 ± 4.1	6.2	
CSRN		0.098					



γ_m=100±3 pcf

γ_s=120±3 pcf

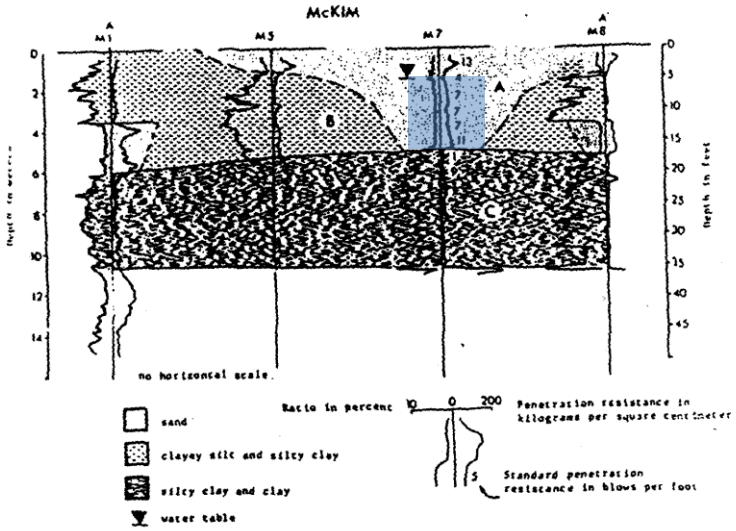


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		FC	sand	silt	clay	D50	WN	WL	PI	LI
1.60	5.25		4	2.00	0.71	6.5	K-4	88.0	12.0	65.0	23.0	0.019	26.0	29.0	11.0	0.7
1.75	5.75		5	1.92	0.72	7.9	K-4 b	84.0	16.0	74.0	10.0	0.038				
1.60	5.25		2	2.00	0.71	***3.2***	SK4-5	-	-	-	-	-				
2.21	7.25		3	1.71	0.76	***4.4***	K-3	***80.0***	***21.0***	***73.0***	***7.0***	***0.044***				
2.21	7.25		7	1.71	0.76	***10.2***	K-4	***74.0***	***26.0***	***68.0***	***6.0***	***0.052***				
2.67	8.75		7	1.56	0.78	***9.7***	K-4 b	***86.0***	***14.0***	***83.0***	***3.0***	***0.049***				
2.82	9.25		1	1.52	0.79	***1.4***	SK4-5	-	-	-	-	-				
2.97	9.75		3	1.50	0.80	***4.1***	K-4	***62.0***	***38.0***	***53.0***	***9.0***	***0.056***	***28.0***			
3.58	11.75		7	1.41	0.83	***9.3***	K-4 b	***88.0***	***12.0***	***79.0***	***9.0***	***0.042***				
3.73	12.25		9	1.40	0.84	***11.9***	K-4	***96.0***	***4.0***	***83.0***	***13.0***	***0.028***	***30.0***			
3.89	12.75		2	1.38	0.85	***2.6***	SK4-5	-	-	-	-	-				
4.19	13.75		3	1.34	0.86	***3.9***	K-3	***91.0***	***9.0***	***79.0***	***12.0***	***0.039***				
4.50	14.75		8	1.31	0.87	***10.4***	K-4	***83.0***	***17.0***	***74.0***	***9.0***	***0.047***	***29.0***			
4.50	14.75		5	1.31	0.87	***6.5***	K-4 b	***81.0***	***19.0***	***71.0***	***10.0***	***0.042***				
4.95	16.25		5	1.27	0.89	***6.4***	SK4-5	-	-	-	-	-				
5.26	17.25		12	1.24	0.90	***15.2***	K-4	***89.0***	***11.0***	***83.0***	***6.0***	***0.048***				
5.41	17.75		12	1.23	0.90	15.1	K-4 b	44.0	56.0	40.0	4.0	0.700				
5.56	18.25		9	1.22	0.91	11.3	K-3	30.0	70.0	29.0	1.0	0.082				
					Mean:	7.1		83.0	17.1	74.6	8.4	0.045				
					St. Dev.	4.1		9.6	9.7	9.3	3.0	0.008				

Case number:	117
Earthquake:	Superstition Hills
Magnitude:	6.7 (Mw)
Location:	McKim Ranch A
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale (1984), Andrus and Stokoe, (1997)
Nature of Failure:	No surface manifastation of liquefaction
Comments:	Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. Located between the Alamo River and the Redwood Canal. A: very fine sand (fining upwards)-loose sand that filled the relic channel. B: stiff silty clay Radio Tower Site is ~ 40 km away from the epicenter of the earthquake. The recorded PGA at the El Centro differential array site # 5 (0.36 g) suggest a a high PGA. Contrary to this site response analyses results suggest a a lower PGA (0.16 g) Energy ratio ~68 %

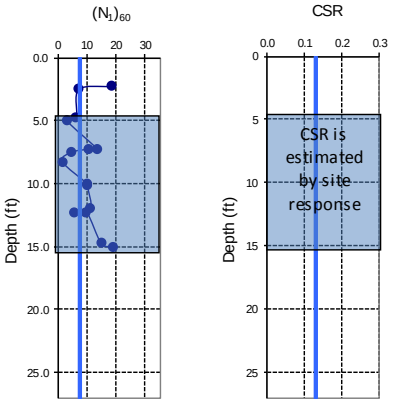
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.106 ± 0.005		
Data Class	B			% Fines	19.8 ± 3.5	31.0	
Critical Depth Range	5.0 - 13.0	6.9		% PI			
Depth to GWT (ft)	5.0	4.9					
σ_v (psf)	1030.0 ± 161.2	793.6		N	5.4 ± 2.6	3.0	
σ_v' (psf)	780.4 ± 80.7	668.3		C_R	0.79	0.80	
a_{max} (g)	0.160 ± 0.048	0.16		C_S	1.00	1.00	
r_d	0.95 ± 0.025	0.98		C_B	1.00	1.00	
CSR	0.130 ± 0.040	0.121		C_E	1.13	1.13	
Equivalent Magnitude	6.5	6.5		C_N	1.65	1.70	
MSF		1.29		$(N_1)_{60}$	7.9 ± 3.8	4.6	
CSRN		0.085					



γ_m=110±3 pcf

γ_s=120±3 pcf



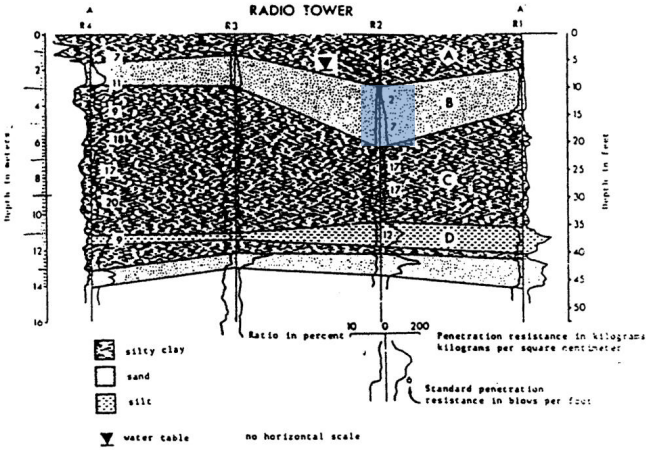
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
0.69	2.3		13	2.00	0.62	18.3	M-7
0.76	2.5		5	2.00	0.63	7.2	SM-7
1.45	4.8		4	2.00	0.70	***6.3***	***M-7***
1.52	5.0		2	1.96	0.71	***3.1***	***SM-7***
2.21	7.3		9	1.76	0.76	***13.6***	***M-2***
2.21	7.3		7	1.76	0.76	***10.6***	***M-7***
2.29	7.5		3	1.75	0.76	***4.5***	***SM-7***
2.51	8.3		1	1.69	0.78	***1.5***	***M-3***
3.05	10.0		7	1.59	0.81	***10.2***	***SM-7***
3.09	10.2		7	1.58	0.81	***10.1***	***M-7***
3.66	12.0		8	1.49	0.84	***11.3***	***SM-7***
3.73	12.3		4	1.48	0.84	***5.6***	***M-3***
3.73	12.3		7	1.48	0.84	***9.8***	***M-7***
4.50	14.8		11	1.38	0.87	15.0	M-7
4.57	15.0		14	1.37	0.87	19.0	SM-7
					Mean:	7.9	
					St. Dev.	3.8	

Depth (ft)	FC	sand	silt	clay	D_{50}	
2.3	31.0	69.0	18.0	13.0	0.1	M-7
2.5	-	-	-	-	-	SM-7
4.8	***-***	***-***	***-***	***-***	***-***	***M-7***
5.0	***-***	***-***	***-***	***-***	***-***	***SM-7***
7.3	***-***	***-***	***-***	***-***	***-***	***M-2***
7.3	***24.0***	***76.0***	***14.0***	***10.0***	***0.1***	***M-7***
7.5	***-***	***-***	***-***	***-***	***-***	***SM-7***
8.3	***-***	***-***	***-***	***-***	***-***	***M-3***
9.4	***21.0***	***79.0***	***11.0***	***10.0***	***0.1***	***M-7***
10.0	***-***	***-***	***-***	***-***	***-***	***SM-7***
10.2	***16.0***	***84.0***	***9.0***	***7.0***	***0.1***	***M-7***
12.0	***-***	***-***	***-***	***-***	***-***	***SM-7***
12.3	***-***	***-***	***-***	***-***	***-***	***M-3***
12.3	***18.0***	***82.0***	***11.0***	***7.0***	***0.1***	***M-7***
14.8	22.0	78.0	15.0	7.0	0.1	M-7
15.0	-	-	-	-	-	SM-7
	19.8	80.3	11.3	8.5	0.106	
	3.5	3.5	2.1	1.7	0.005	

Case number:	118
Earthquake:	1987 Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Radio Tower B1
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand PGA was recorded at the surface and at 6.7 m depth of Wildlife site as 0.18 and 0.17g respectively. The epicentral distance is ~30.2 km same as Wildlife site. Andrus and Stoke (NCEER, 97) predicted the PGA at the site as 0.2 g same as the PGA at Wildlife site. Site response analyses results also verify that Energy ratio ~68 %

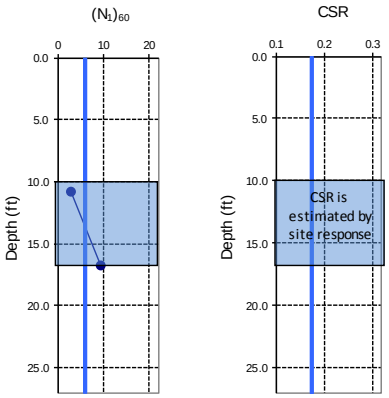
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.074 ± 0.037		
Data Class	B			% Fines	43.5 ± 29.0	64.0	
Critical Depth Range	9.8 - 18.0	11.2		% PI			
Depth to GWT (ft)	6.6	6.9					
σ_v (psf)	1505.1 ± 160.1	1294.9		N	4.5 ± 3.4	2.0	
σ_v' (psf)	1044.5 ± 79.5	1044.3		C_R	0.86	0.85	
a_{max} (g)	0.200 ± 0.030	0.20		C_S	1.00	1.00	
r_d	0.94 ± 0.032	0.96		C_B	1.00	1.00	
CSR	0.176 ± 0.028	0.154		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.42	1.49	
MSF		1.29		$(N_1)_{60}$	6.2 ± 4.7	2.9	
CSRN		0.113					



γ_m=100±3 pcf

γ_s=115±3 pcf



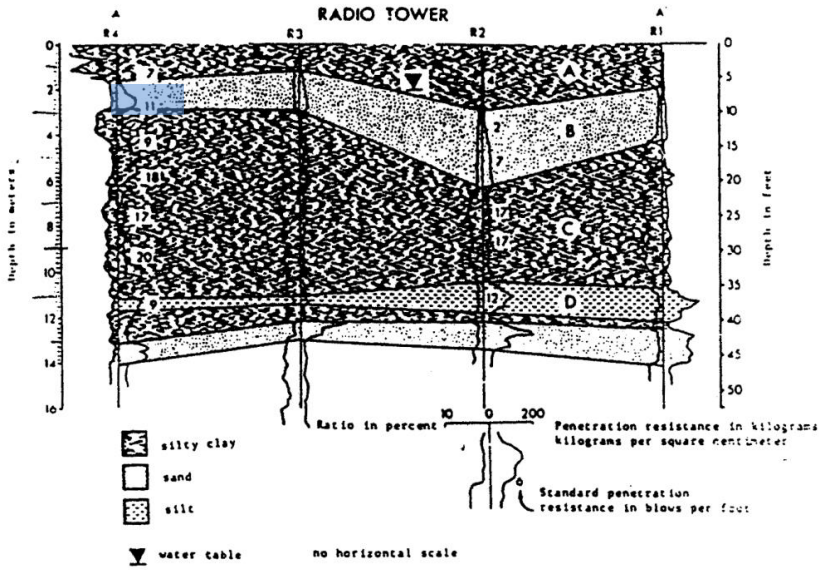
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.8		2	1.55	0.82	***2.9***
5.1	16.7		7	1.33	0.89	***9.5***
					Mean:	6.2
					St. Dev.	4.7

Depth (m)	FC	sand	silt	clay	D_{50}
3.3	***64.0***	***36.0***	***59.0***	***5.0***	***0.047***
5.1	***23.0***	***77.0***	***22.0***	***1.0***	***0.100***
23.15	*18.0*	*82.0*	*13.0*	*5.0*	*0.125*
	43.5	56.5	40.5	3.0	0.074
	29.0	29.0	26.2	2.8	0.037

Case number:	119
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Radio Tower B2
References:	James Gerard Bierschwale. (1984) Bennett et al. (1984) and Fear et al. (1995)
Nature of Failure:	No surface manifastation of liquefaction
Comments:	Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. A: very poorly sorted, soft brown clay and silt-flood plain deposit B: brown, loose to med.dense silt to fine sand- sediment at base is well sorted very fine sand fining upwards to poorly sorted silt-channel deposit C: very stiff brown clayey silt to clay-lacustrine D: med dense sand PGA was recorded at the surface and at 6.7 m depth of Wildlife site as 0.18 and 0.17g respectively. The epicentral distance is ~30.2 km same as Wildlife site. Andrus and Stoke (NCEER, 97) predicted the PGA at the site as 0.2 g same as the PGA at Wildlife site. Site response results suggest a PGA of 0.18g Energy ratio ~68 %

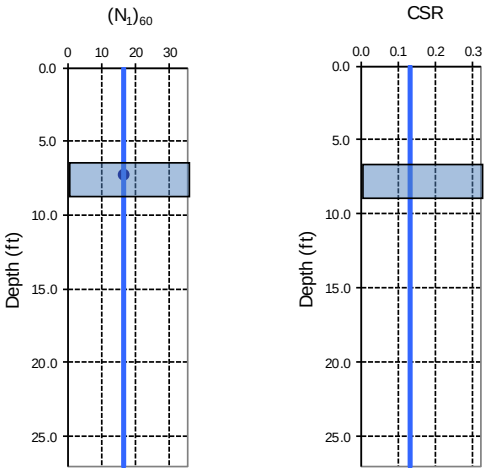
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.100 ± 0.050		
Data Class	B			% Fines	18.0 ± 2.0	30.0	
Critical Depth Range	6.6 - 9.8	7.5		% PI			
Depth to GWT (ft)	6.6	6.9					
σ_v (psf)	852.8 ± 67.4	835.4		N	11.3 ± 1.4	11.0	
σ_v' (psf)	750.5 ± 39.0	793.6		C_R	0.77	0.80	
a_{max} (g)	0.180 ± 0.027	0.18		C_S	1.00	1.00	
r_d	0.99 ± 0.020	0.98		C_B	1.00	1.00	
CSR	0.132 ± 0.021	0.119		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.68	1.53	
MSF		1.29		$(N_1)_{60}$	16.6 ± 2.0	15.2	
CSRN		0.084					



γ_m=100±3 pcf

γ_s=120±3 pcf



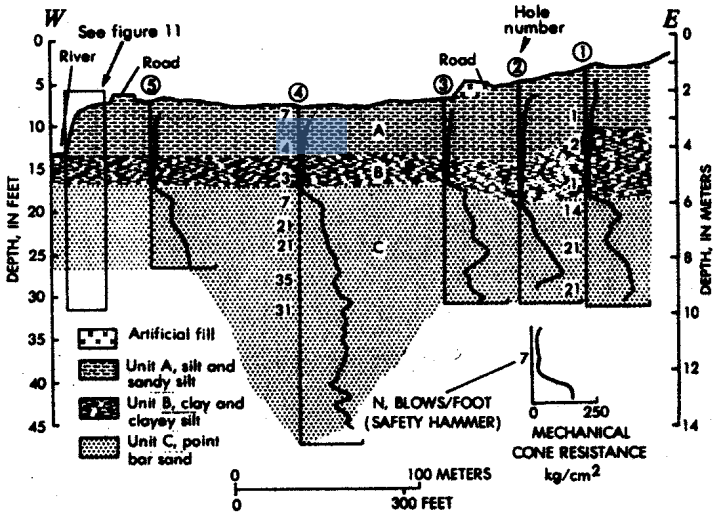
CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	silt	clay	D50
2.3	7.3		11	1.74	0.76	*16.6*	*18.0*	*17.0*	*1.0*	*0.1*
					Mean:	16.6				
					St. Dev.	2.0	assigned			

Case number:	120
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	River Park A
References:	Youd et al. (1983), "Liquefaction sites, Imperial Valley, CA
Nature of Failure:	No surface manifastation of liquefaction
Comments:	Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. River Park Site is ~30 km from the epicenter of the earthquake. PGA's values of 0.46 and 0.16 g were recorded at Parachute (15 kms southwest) and Brawley (~8 kms east) stations, respectively. Site response analysis result suggest a PGA value of 0.19 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T Driver and safety hammer. Energy ratio ~68 % SPT blowcounts were taken in 1979 and 1981.

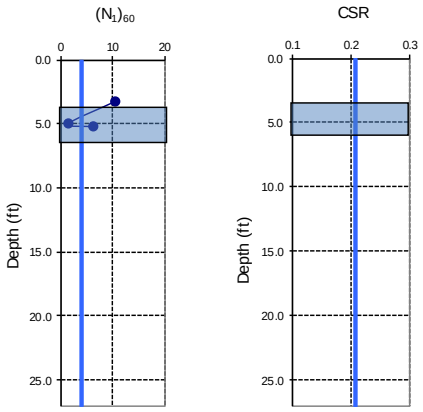
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.010 ± 0.050		
Data Class	B			% Fines	91.0 ± 2.0	80.0	
Critical Depth Range	1.0 - 5.9	5.9		% PI			
Depth to GWT (ft)	1.0	1.0					
σ_v (psf)	369.8 ± 90.3	731.0		N	2.7 ± 2.3	3.0	
σ_v' (psf)	216.8 ± 42.7	417.7		C_R	0.66	0.80	
a_{max} (g)	0.190 ± 0.029	0.19		C_S	1.00	1.00	
r_d	0.99 ± 0.010	0.98		C_B	1.00	1.00	
CSR	0.209 ± 0.038	0.210		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	2.00	1.70	
MSF		1.29		$(N_1)_{60}$	4.0 ± 3.4	4.6	
CSRN		0.149					



γ_m=100±3 pcf

γ_s=110±3 pcf



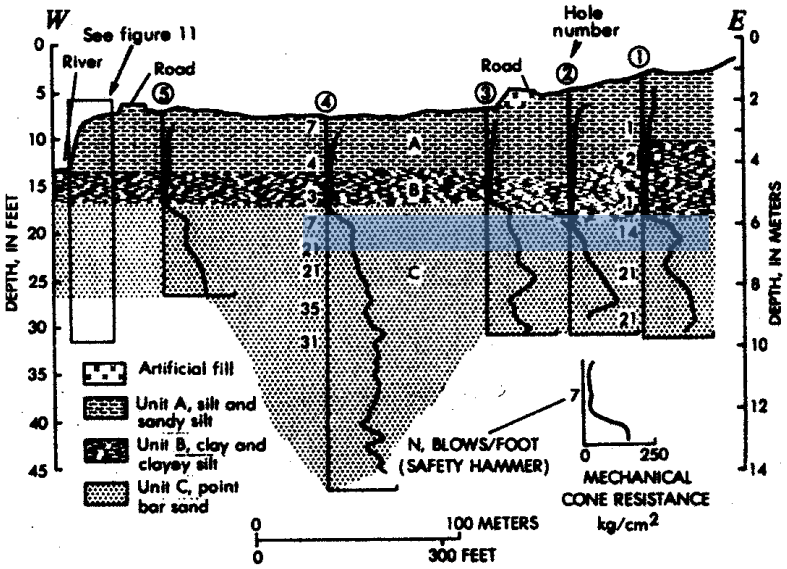
CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		FC	sand	silt	clay	D50
1	3.3		7	2.00	0.66	10.4	BH-4 (1979)	-	-	-	-	0.071
1.52	5.0		1	2.00	0.71	***1.6***	BH-15 (1981)	***91.0***	***9.0***	***74.0***	***17.0***	***0.010***
1.6	5.2		4	2.00	0.71	***6.5***	BH-4 (1979)	-	-	-	-	-
					Mean:	4.0		91.0	9.0	74.0	17.0	0.010
					St. Dev.	3.4		2.0	-	-	-	0.050

Case number:	121
Earthquake:	Superstition Hills
Magnitude:	6.54 (Mw)
Location:	River Park C
References:	Youd et al. (1982), "Liquefaction sites, Imperial Valley, CA
Nature of Failure:	No surface manifastation of liquefaction
Comments:	The Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much as 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The principal rivers are the Alamo and New Rivers along which many sand boils were developed on fluvial deposits. River Park Site is ~30 km from the epicenter of the earthquake. PGA's values of 0.46 and 0.16 g were recorded at Parachute (15 kms southwest) and Brawley (~8 kms east) stations, respectively. Site response analysis result suggest a PGA value of 0.19 g. ASTM D1586-67 procedures were followed for SPT tests. 10" outside diameter hollow-stem auger was used with Safe-T Driver and safety hammer. Energy ratio ~68 % SPT blowcounts were taken in 1979 and 1981.

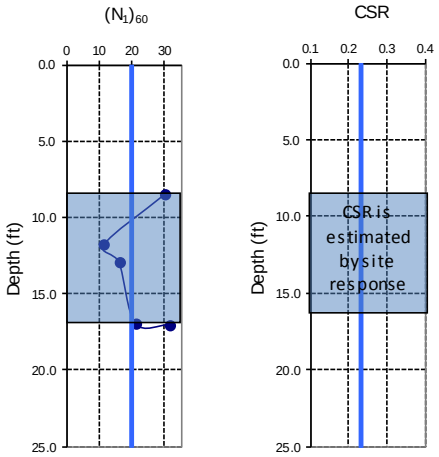
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.142 ± 0.020		
Data Class	A			% Fines	13.2 ± 4.4	18.0	
Critical Depth Range	11.0 - 17.0	14.1		% PI			
Depth to GWT (ft)	1.0	1.0					
σ_v (psf)	1595.0 ± 121.6	1733.5		N	12.4 ± 4.9	11.0	
σ_v' (psf)	783.8 ± 67.6	939.8		C_R	0.86	0.85	
a_{max} (g)	0.190 ± 0.029	0.19		C_S	1.00	1.00	
r_d	0.93 ± 0.025	0.95		C_B	1.00	1.00	
CSR	0.234 ± 0.037	0.218		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.64	1.44	
MSF		1.29		$(N_1)_{60}$	19.9 ± 7.8	15.2	
CSRN		0.154					



γ_m=100±3 pcf

γ_s=115±3 pcf



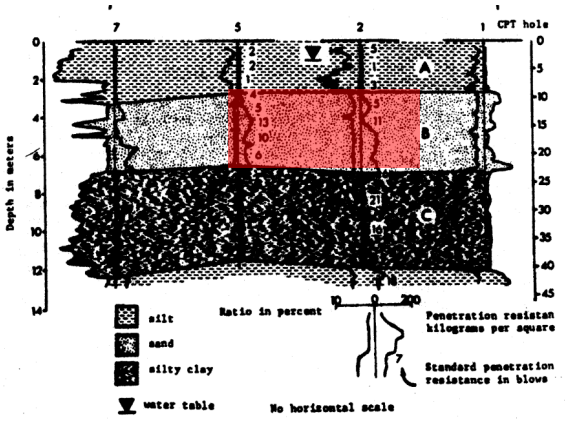
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.6	8.5		17	2.00	0.78	***30.1***	BH-13
3.6	11.8		7	1.78	0.83	***11.8***	BH-4
3.96	13.0		10	1.70	0.85	***16.4***	BH-6
5.18	17.0		14	1.50	0.90	***21.3***	BH-1
5.2	17.1		21	1.50	0.90	32.0	BH-4
					Mean:	19.9	
					St. Dev.	7.8	

Depth (m)	FC	sand	silt	clay	D50	
3.3	**12**	**88**	**9**	**3**	**0.110**	BH-4
3.5	**13**	**87**	**11**	**2**	**0.120**	BH-4
3.9	**11**	**88**	**9**	**2**	**0.135**	BH-4
4.4	**11**	**89**	**10**	**1**	**0.140**	BH-4
4.6	**10**	**90**	**9**	**1**	**0.145**	BH-4
5.1	**10**	**90**	**10**	**0**	**0.145**	BH-4
5.2	**9**	**91**	**9**	**0**	**0.180**	BH-1
5.2	**14**	**86**	**14**	**0**	**0.170**	BH-1
5.3	**13**	**87**	**10**	**3**	**0.145**	BH-4
5.7	**18**	**82**	**14**	**4**	**0.140**	BH-4
6.0	**24**	**76**	**20**	**4**	**0.135**	BH-4
6.4	7	93	7	0	0.115	BH-1
7.8	6	94	6	0	0.145	BH-1
Mean:	13.2	86.7	11.4	1.8	0.142	
St. Dev.	4.4	4.3	3.4	1.5	0.020	

Case number:	122
Earthquake:	1987 Superstition Hills
Magnitude:	6.54 (Mw)
Location:	Wildlife B
References:	Bennett et al. (1984) and Fear et al. (1995) James Gerard Bierschwale. (1984) Youd, T.L., Wieczorek G. F., (1984), "Liquefaction during the 1981 and Previous Earthquakes Near Westmorland, CA" Open File Rep. 84-680
Nature of Failure:	Site has liquefied. Pore pressure ratios exceeded 1
Comments:	Imperial Valley lies in the southern part of the Salton Basin, which was formed by northward extension of the crustal rifting that opened the Gulf of California. The basin is underlain by as much ad 20,000 ft of Tertiary and Quaternary marine, continental, and lacustrine sediment. Holocene sediment is as thick as 330 ft. The site is very closely monitored. Strong ground motion seismometers were installed at the surface and at 6.7 m depth. Piezometers were installed almost at every meter along the potentially liquefiable layer of thickness 4 m. A: very loose and very soft interbedded micaceous sandy silt, silt and clayey silts (0-2.5 m) flood plain deposits B: very loose to loose moderately sorted sandy silt (2.5-6.8m); point bar deposits C: silty clay (medium to very stiff) and clayey silt (medium to stiff); lacustrine deposit. PGA was recorded at the surface and at 6.7 m depth as 0.2 and 0.17g. respectively. The epicentral distance is 30.2 km. Site response analyses results estimate PGA in two direction as 0.18 g and 0.23 g geometric mean of which is 0.205 g. Energy ratio ~68 %

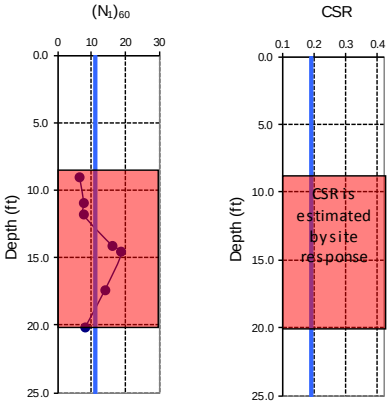
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.109 ± 0.038		
Data Class	A			% Fines	26.2 ± 7.6	30.0	
Critical Depth Range	9.0 - 22.0	15.1		% PI			
Depth to GWT (ft)	3.0	3.9					
σ_v (psf)	1800.0 ± 262.9	1817.0		N	7.9 ± 3.5	7.1	
σ_v' (psf)	1020.0 ± 131.5	1127.8		C_R	0.88	0.95	
a_{max} (g)	0.205 ± 0.031	0.21		C_S	1.00	1.00	
r_d	0.82 ± 0.035	0.94		C_B	1.00	1.00	
CSR	0.192 ± 0.031	0.216		C_E	1.13	1.13	
Equivalent Magnitude	6.54	6.54		C_N	1.44	1.36	
MSF		1.29		$(N_1)_{60}$	11.3 ± 5.0	10.3	
CSRN		0.157					



γ_m=100±3 pcf

γ_s=120±3 pcf



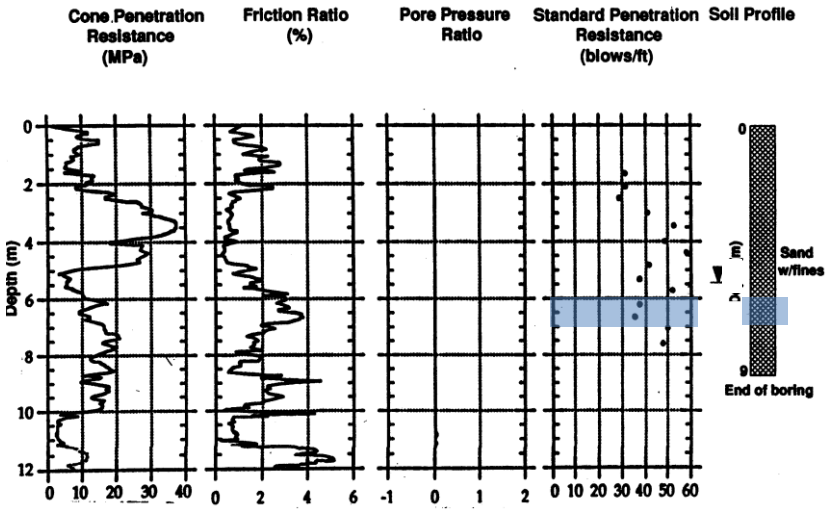
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.75	9.0		4	1.81	0.79	***6.5***	Hole-5
3.35	11.0		5	1.67	0.82	***7.8***	Hole-2
3.6	11.8		5	1.62	0.83	***7.6***	Hole-5
4.3	14.1		11	1.50	0.86	***16.2***	Hole-2
4.45	14.6		13	1.48	0.87	***18.9***	Hole-5
5.3	17.4		10	1.37	0.90	***14.0***	Hole-5
6.15	20.2		6	1.28	0.93	***8.1***	Hole-5
					Mean:	11.3	
					St. Dev.	5.0	

Depth (ft)	FC	Sand	Silt	Clay	D50	
10.5	***34.0***	***66.0***	***34.0***	***0.0***	***0.2***	Hole-2
11.2	***17.0***	***83.0***	***17.0***	***0.0***	***0.1***	Hole-2
13.5	***27.0***	***73.0***	***26.0***	***1.0***	***0.1***	Hole-2
14.2	***20.0***	***80.0***	***20.0***	***0.0***	***0.1***	Hole-2
18.3	***33.0***	***67.0***	***29.0***	***4.0***	***0.1***	Hole-2
22.8	*18.0*	*82.0*	*17.0*	*1.0*	*0.1*	Hole-2
Mean:	26.2	73.8	25.2	1.0	0.109	
St. Dev.	7.6	7.6	6.8	1.7	0.038	

Case number:	123
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Alameda Bay Farm Island (Dike Location)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	No surface manifestation of liquefaction. Site was improved by vibraflotation
Comments:	Located immediately north of Oakland International Airport Most of the western part of Bay Farm Island consists of sandy hydraulic fill, underlain by bay mud and deeper stiffer soil. Considerable liquefaction occurred at the northwest corner and at points along the west edge of the fill. Soil liquefaction in the northern part of the island was largely confined to artificially fill land. Numerous sand boils~ 3m in diameter erupted in Harbor Bay Plaza area. Fill in the perimeter dike area was densified by vibroflotation. Densification prevented soil liquefaction of the western-perimeter dike. Complex stratigraphy of thin (<1m thick) interbeds of sand and finer material A layer of noncohesive sand and silty sand at 2-3 m. Below this layer to 6.5 m is a set of interlayered deposits. PGA is recorded as 0.27 & 0.21g at the Alameda Naval Air Station seismogram which is located on a 152 m deep soil site. Measurements performed after the earthquake. Depth to water table is given as 2-3 m in Kayen et al (1998). However in Mitchell et al it was given as 5.5 m. P-wave plots suggests a water depth of 5.5 m.

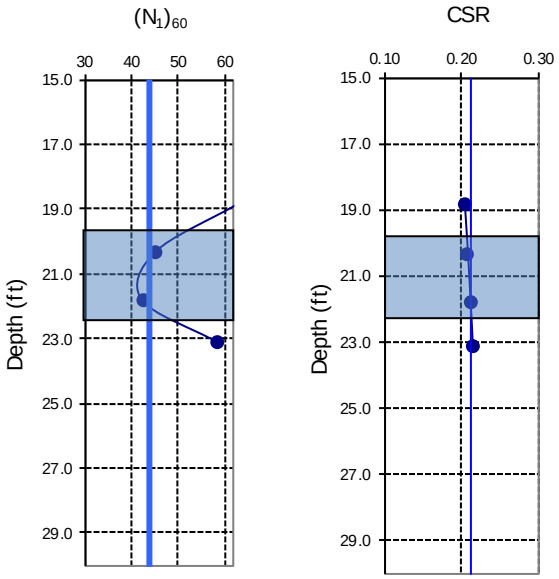
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No	No	D_{50}	0.280 ± 0.050		? ? ?
Data Class	B			% Fines	7.0 ± 2.0	7.0	sand sand
Critical Depth Range	19.7 - 23.0	21.3	5.30 6.70 m	% PI			
Depth to GWT (ft)	9.8	9.8	3 m				
σ_v (psf)	2616.5 ± 82.1	2610.7	0.95 1.21 kg/cm³	N	37.1 ± 1.7	37.0	37 35
σ_v' (psf)	1899.9 ± 59.4	1900.6	0.72 0.84 kg/cm³	C_R	0.94	0.95	↑
a_{max} (g)	0.240 ± 0.036	0.24	0.270	C_S	1.30	1.30	0.900
r_d	0.99 ± 0.087	0.92	0.95 0.93	C_B	1.00	1.00	
CSR	0.212 ± 0.037	0.198	0.219 0.236	C_E	0.92	0.92	↓
Equivalent Magnitude	6.93	6.93		C_N	1.06	1.03	1.16 1.09
MSF		1.16		$(N_1)_{60}$	43.8 ± 2.0	43.3	42.2 37.5
CSRN		0.165	0.207 0.223				



γ_m=120±3 pcf

γ_s=125±3 pcf

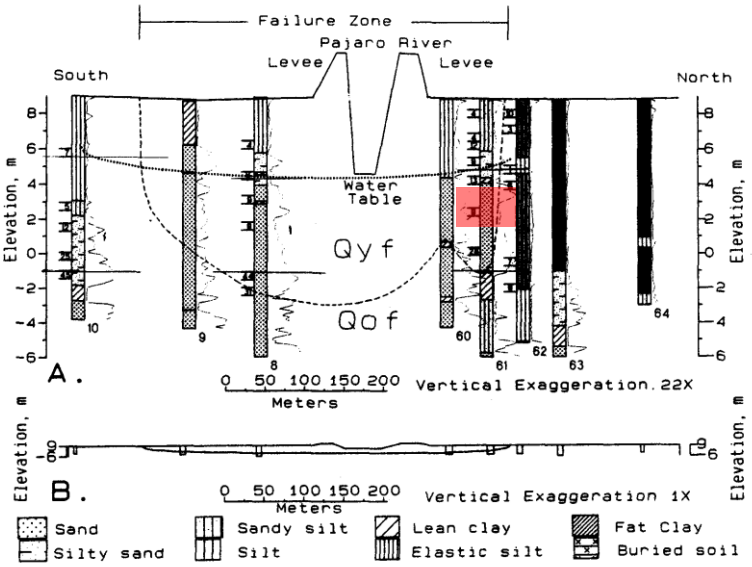


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.75	18.9	0.204	52	1.10	0.92	62.5
6.2	20.3	0.209	38	1.07	0.93	***45.2***
6.65	21.8	0.213	36	1.05	0.94	***42.4***
7.05	23.1	0.217	50	1.03	0.95	58.3
					Mean:	43.8
					St. Dev.	2.0

Case number:	124
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Farris Farm Site
References:	Holzer et al. (1994), "Observed and predicted ground deformation-Miller Farm lateral spread, Watsonville, California
Nature of Failure:	Lateral spread near the Pajaro River. South of the river ~0.2 m high head scarp which could be followed for 1.7 km. Maximum horizontal displacement of ~30 cm just west of the study area. Sand boils along the area affected by lateral spreading. Eyewitnesses reported that sediment-laden water flowed gently from vents on the lateral spread during and until 20 minuntes after the earthquake.
Comments:	Located at the south side of the Pajara River at Clint Miller Farms near Watsonville, CA. Composed of younger and older holocene fluvial deposits The upper 4-6 m of the younger flood-plain deposits is silty and contains only a few thin sand layers. In the depth interval from 4.6-9.1, sand percentages range 67-100. This layer includes most of the liquefied layer. Average fines content of the sandy layers if 15%, ranging from 22 % at Miller Farm to 8% at Farris Farm. The depth to the water table was ~4.6 m 10 days after the main shock. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g In Open File Report 95-663, Holzer et al., the PGA was given as 0.53g. Watsonville region was shaken at MMI scale of 8.(~ 0.25-0.5 g ?) The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.5-0.65 g PGA was recorded as 0.36 g at Watsonville. Based on site response analyses results a PGA of 0.37 g was adopted for the analyses. Strong ground motion lasted ~ 6 seconds. SPT performed after the earthquake

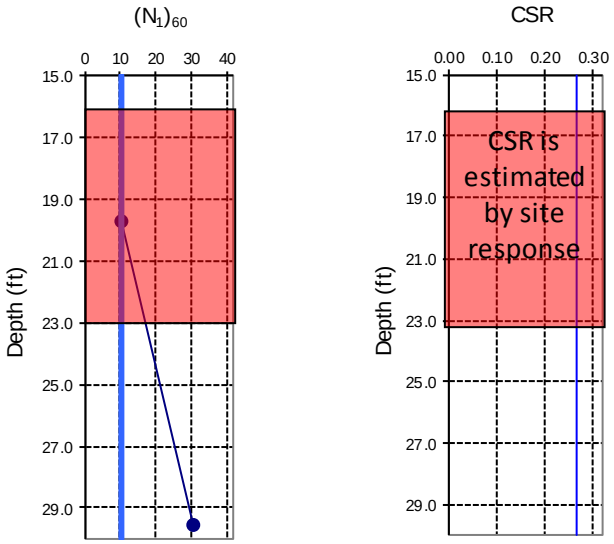
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.200 ± 0.050		
Data Class	B			% Fines	8.0 ± 2.0	8.0	
Critical Depth Range	16.4 - 23.0	19.7		% PI			
Depth to GWT (ft)	14.8	14.8		D N fines			
σ_v (psf)	2066.9 ± 139.4	2213.9			9.0 ± 1.7	9.0	
σ_v' (psf)	1759.8 ± 79.4	1921.5		C_R	0.92	0.95	
a_{max} (g)	0.370 ± 0.056	0.37		C_S	1.00	1.00	
r_d	0.95 ± 0.020	0.93		C_B	1.00	1.00	
CSR	0.268 ± 0.041	0.259		C_E	1.13	1.13	
Equivalent Magnitude	6.93	6.93		C_N	1.10	1.05	
MSF		1.16		$(N_1)_{60}$	10.3 ± 2.0	10.2	
CSRN		0.221					



γ_m=100±3 pcf

γ_s=120±3 pcf



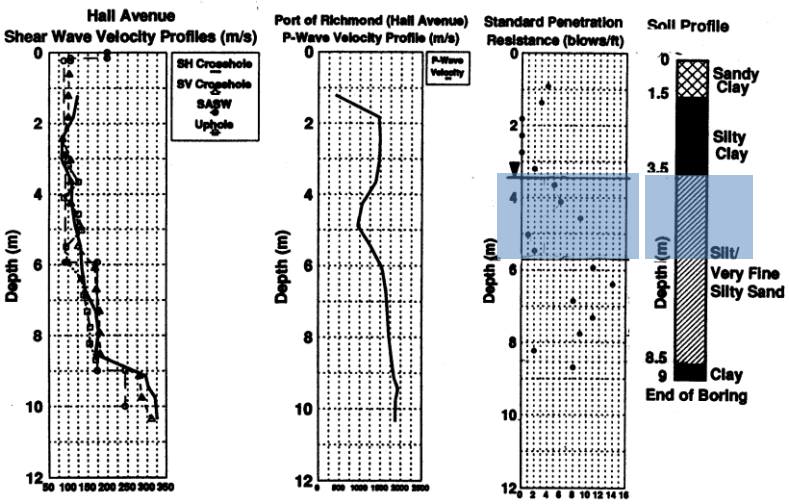
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
6	19.7		9	1.10	0.92	***10.3***
9	29.5		28	0.95	1.00	30.3
					Mean:	10.3
					St. Dev.	2.0

assigned

Case number:	125
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Port of Richmond (Hall Avenue)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	No surface evidence of liquefaction was observed at Hall Avenue. However, between depths of about 1.5-3.5 m, the sampler sank under its own weight in a layer of very soft silty clay.
Comments:	The site is close to POR 2-3-4 (~ 1 km) by the Hall Street The site represents the most distant point, approximately 85 km north of the rupture zone The liquefaction zone overlies deposits of soft bay mud, underlain by deeper deposits of stiffer, overconsolidated noncohesive and cohesive soils Up to 0.8 m depth the soil is characterized by oxidized, tan-brown crusted silty sand. 0.8-1.8 m oxidized tan-brown silty sand. 1.8-4.1 m oxidized tan-brown sandy clay to reduced sandy clay.4.1-7.8 m, is a hydraulic fill layer of olive-gray fine-silty sand containing shell fragments. The thick deposits amplified the level of ground motion at the site PGA was estimated as 0.18 g by performing a site response analysis (SHAKE) using the input motion from Richmond City Hall (PGA=0.13g) by Kayen et al. Based on site response analyses results, a PGA of 0.14 was adopted. Depth to water table in Kayen et al. (1998) was reported as 2.5 m contrary to 3.5 m reported in Mitchell et al. (1994)

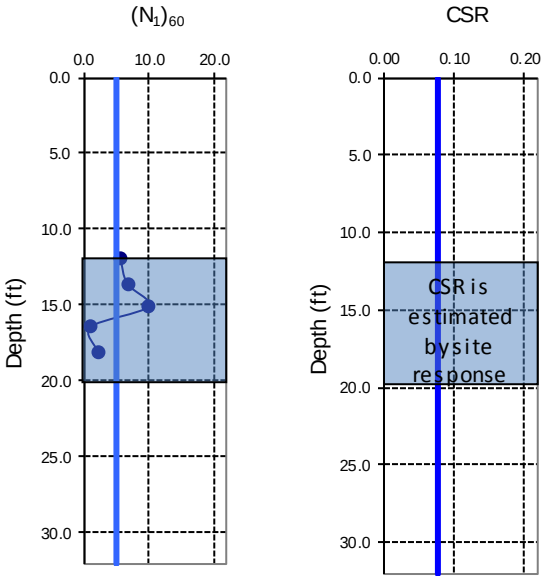
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.090 ± 0.050		
Data Class	A			% Fines	30.0 ± 2.0	30.0	
Critical Depth Range	18.9 - 11.5	15.1		% PI			
Depth to GWT (ft)	11.5	11.5					
σ_v (psf)	1572.8 ± 146.1	1566.4		N	4.6 ± 3.2	4.6	
σ_v' (psf)	1342.4 ± 75.5	1336.7		C_R	0.88	0.95	
a_{max}	0.140 ± 0.021	0.14		C_S	1.10	1.10	
r_d	0.72 ± 0.013	0.95		C_B	1.00	1.00	
CSR	0.077 ± 0.012	0.102		C_E	0.92	0.92	
Equivalent Magnitude	6.93	6.93		C_N	1.26	1.28	
MSF		1.16		$(N_1)_{60}$	5.1 ± 3.6	5.7	
CSRN		0.084					



γ_m=100±3 pcf

γ_s=115±3 pcf

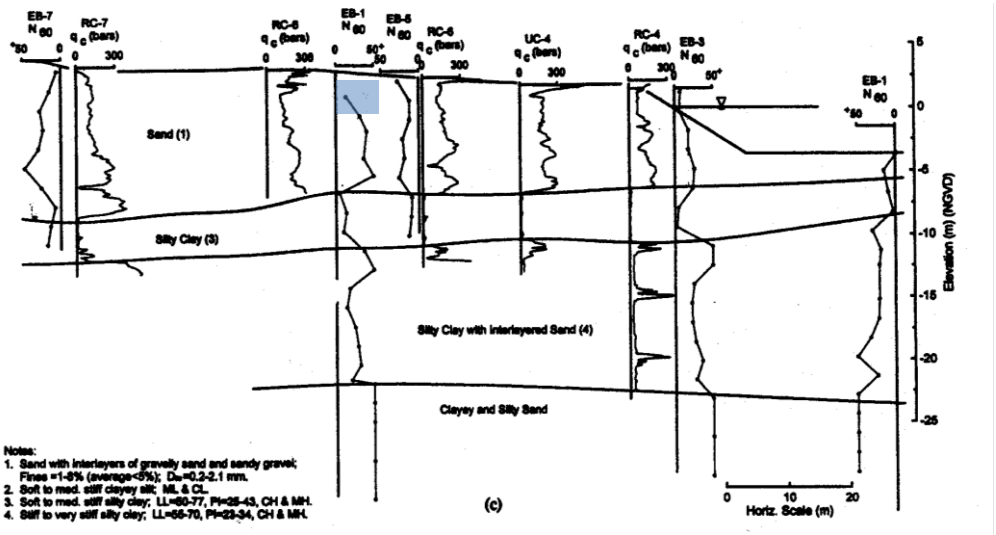


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.65	12.0		5	1.34	0.84	***5.7***
4.15	13.6		6	1.30	0.86	***6.7***
4.6	15.1		9	1.26	0.88	***10.0***
5	16.4		1	1.23	0.89	***1.1***
5.5	18.0		2	1.19	0.91	***2.2***
					Mean:	5.1
					St. Dev.	3.6

Case number:	126
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	MBARI Facilities Building 3 EB-1
References:	Boulanger et al. (1996), "Liquefaction at Moass Landing During Loma Prieta Earthquake"
Nature of Failure:	Extensive ground deformations were observed along Sandholdt Road, including the portion alongside the MBARI facilities. Cracks parallel to the front line. Away from Sandholdt Road, surface evidence of liquefaction appeared to occur in localized areas.
Comments:	Located on Monterey Bay about midway between the cities of Santa Cruz and Monterey, CA. The MBARI facilities include three main buildings and a pier located along Sandholt Road on the Moss Landing spit. Historical shoreline suggest that the shoreline in the vicinity of the site has fluctuated significantly. Shallower soil strata may have been deposited recently. The area is underlain by Holocene deposits consisting of relatively thick sands offshore and estuarine and fluvial deposits onshore, up to about 60-m deep. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m The site is underlain by an 8-12 m thick layer of poorly graded sand with interlayers of gravelly sand and sandy gravel in its lower portion and occasional thin layers of clayey silt throughout. PGA was recorded in a four-story building 8km from rupture plane as 0.39 g (alluvial site). Also San Juan Bautista seismographic station, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site was predicted in the range of 0.2-0.3g Based on site response analyses results, a PGA value of 0.24 g was adopted. EB-1 and EB-5 are ~50ft apart Difficult to choose a representative SPT value SPT performed after the earthquake

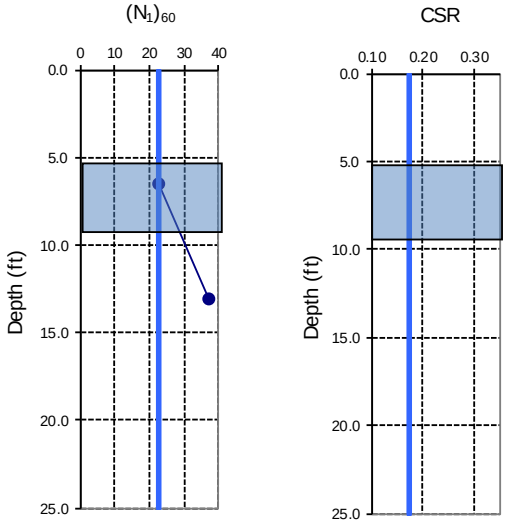
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.600 ± 0.050		
Data Class	B			% Fines	1.0 ± 2.0	1.0	
Critical Depth Range	6.6 - 9.8	6.6		% PI			
Depth to GWT (ft)	6.6	6.6					
σ_v (psf)	926.8 ± 71.4	731.0		N	18.5 ± 1.6	18.0	
σ_v' (psf)	824.5 ± 42.3	731.0		C_R	0.77	0.80	
a_{max} (g)	0.240 ± 0.036	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.007	0.99		C_B	1.00	1.00	
CSR	0.174 ± 0.027	0.179		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.60	1.57	
MSF		1.16		$(N_1)_{60}$	22.9 ± 2.0	22.6	
CSRN		0.140					



γ_m=110±3 pcf

γ_s=125±3 pcf



CSR is estimated by site response

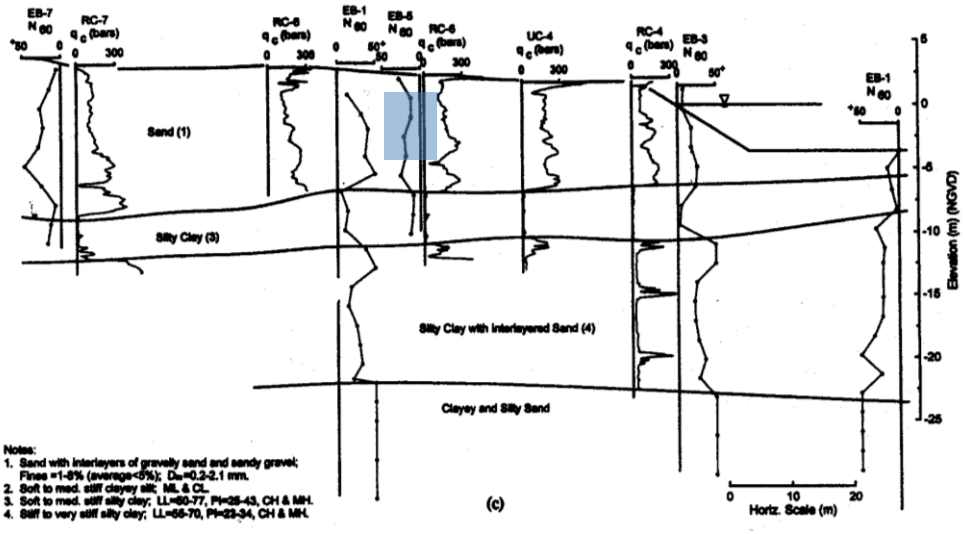
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2	6.6		18	1.71	0.74	***22.9***
4	13.1		32	1.37	0.85	37.2
					Mean:	22.9
					St. Dev.	2.0

assigned

Case number:	127
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	MBARI Facilities Building 3 EB-5
References:	Boulanger et al. (1996), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	Extensive ground deformations were observed along Sandholdt Road, including the portion alongside the MBARI facilities. Cracks parallel to the front line. Away from Sandholdt Road, surface evidence of liquefaction appeared to occur in localized areas.
Comments:	Located on Monterey Bay about midway between the cities of Santa Cruz and Monterey, CA. The MBARI facilities include three main buildings and a pier located along Sandholt Road on the Moss Landing spit. Historical shoreline suggest that the shoreline in the vicinity of the site has fluctuated significantly. Shallower soil strata may have been deposited recently. The area is underlain by Holocene deposits consisting of relatively thick sands offshore and estuarine and fluvial deposits onshore, up to about 60-m deep. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m The site is underlain by an 8-12 m thick layer of poorly graded sand with interlayers of gravelly sand and sandy gravel in its lower portion and occasional thin layers of clayey silt throughout. PGA is recorded in a four-story building 8km from rupture plane as 0.39 g which is underlain by alluvium. Also San Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3 g. Based on site response analyses results, PGA is adopted as 0.27 g, slightly higher than the value adopted by authors of the source document (0.25 g) EB-1 and EB-5 are ~50ft apart. SPT performed before the earthquake.

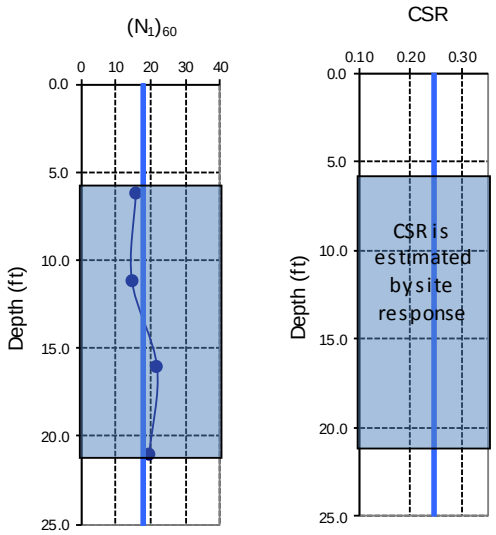
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.600 ± 0.050		
Data Class	A			% Fines	1.0 ± 2.0	1.0	
Critical Depth Range	5.9 - 21.0	11.2		% PI			
Depth to GWT (ft)	5.9	5.9					
σ_v (psf)	1592.8 ± 315.8	1315.8		N	15.2 ± 2.8	12.0	
σ_v' (psf)	1122.0 ± 160.7	981.6		C_R	0.86	0.85	
a_{max} (g)	0.270 ± 0.041	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.007	0.97		C_B	1.00	1.00	
CSR	0.247 ± 0.040	0.235		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.37	1.46	
MSF		1.16		$(N_1)_{60}$	17.8 ± 3.3	14.9	
CSRN		0.187					



γ_m=110±3 pcf

γ_s=125±3 pcf

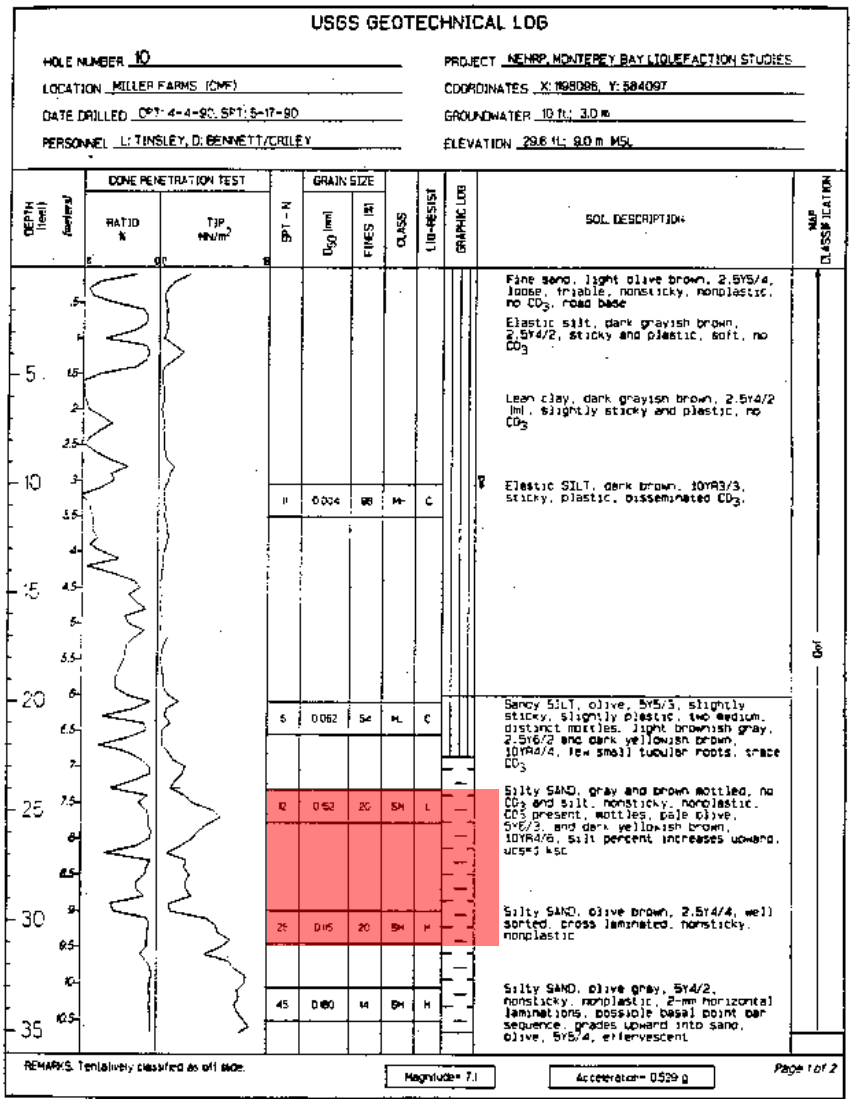


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.9	6.2		12	1.78	0.73	***15.7***
3.4	11.2		12	1.47	0.82	***14.5***
4.9	16.1		19	1.28	0.89	***21.6***
6.4	21.0		18	1.15	0.94	***19.4***
					Mean:	17.8
					St. Dev.	3.3

Case number:	128
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Miller Farm CMF 10
References:	Bennett and Tinsley, 1995, "Open File Report 95-663"
Nature of Failure:	Lateral spread, slide, and sand boils.
Comments:	Located at the east of Watsonville, Monterey. Both SPT and CPT test were performed. Surface samples were taken. Grain size characteristices were given in detail. Usually SPT's were made 3-5 ft away from the CPT boreholes to obtain samples. SPT procedures followed the guidelines outlined in D1586-67 (ASTM). Hollow-stem augers (10" outside diameter, 4" inside diameter with split liners) 140 lb "inhole sampling hammer dropped at 30" height by "Safe-T-Driver" hoist The overall efficiency of the hammer-hoist system is 68 %. Undisturbed samples were taken using thin-walled Shelby tubes. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g. In Open File Report 95-663, Holzer et al., the PGA was given as 0.53g. Watsonville region was shaken at MMI scale of 8.(~ 0.25-0.50 g ?). The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.50-0.65 g A PGA value of 0.36g was recorded at Watsonville. Based on site response analyses results, a PGA value of 0.41 is adopted. Strong ground motion lasted ~ 6 seconds. CMF3-5-8-10 are ~ 1000 ft apart. SPT performed after the earthquake.

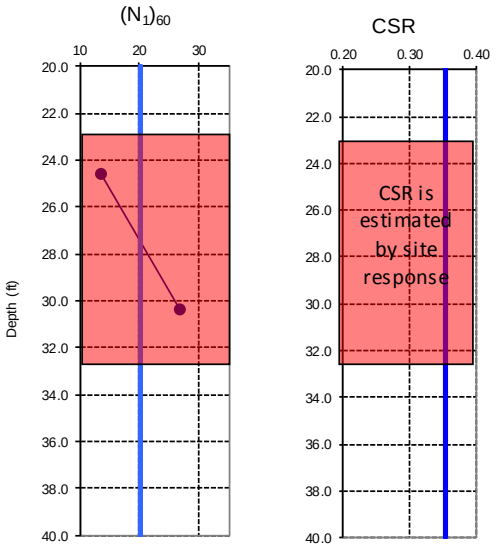
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.134 ± 0.026		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	23.0 - 32.8	27.6		% PI			
Depth to GWT (ft)	9.8	9.8					
σ_v (psf)	3338.3 ± 214.2	3299.9		N	18.3 ± 3.2	19.0	
σ_v' (psf)	2212.3 ± 121.3	2193.0		C_R	0.99	0.95	
a_{max} (g)	0.410 ± 0.062	0.39		C_S	1.00	1.00	
r_d	0.88 ± 0.024	0.89		C_B	1.00	1.00	
CSR	0.354 ± 0.054	0.338		C_E	1.13	1.13	
Equivalent Magnitude	6.93	6.93		C_N	0.98	0.99	
MSF		1.16		$(N_1)_{60}$	20.1 ± 3.5	20.2	
CSRN		0.292					



γ_m=110±3 pcf

γ_s=125±3 pcf

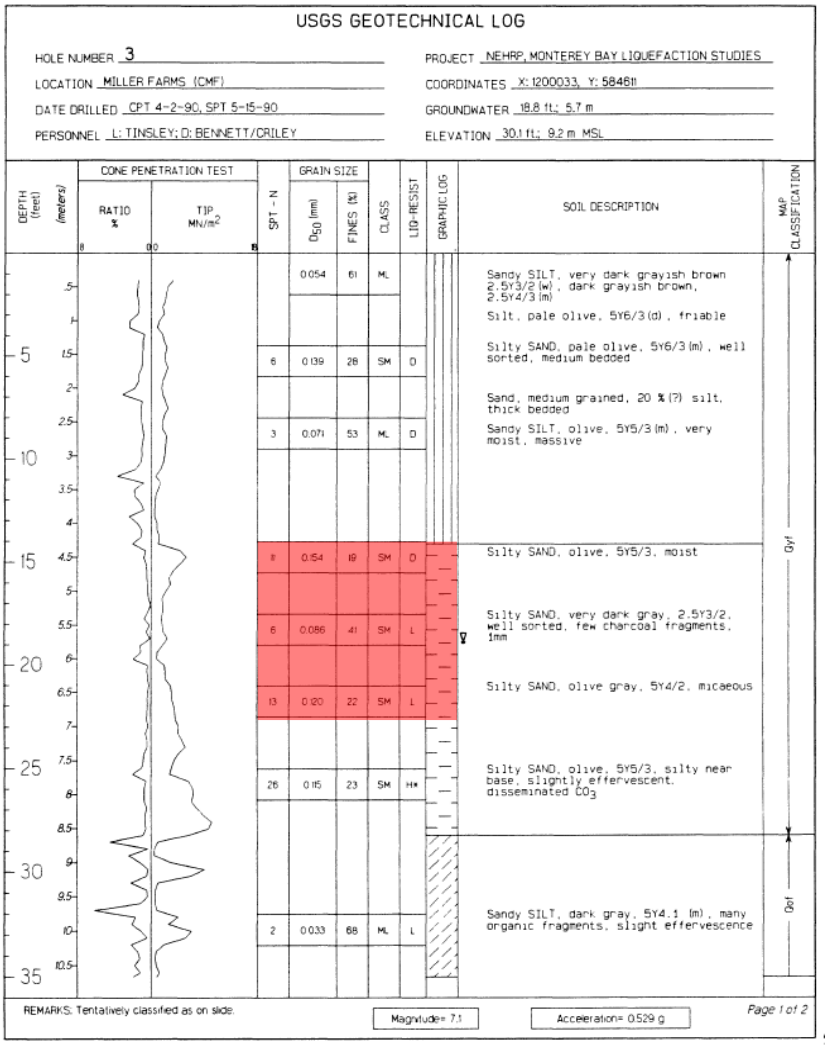


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC %	D50
7.5	24.6		12	1.03	0.97	***13.5***	***20.0***	***0.152***
9.25	30.3		25	0.95	1.00	***26.8***	***20.0***	***0.115***
					Mean:	20.1	20.0	0.134
					St. Dev.	3.5	2.0	0.026

Case number:	129
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Miller Farm CMF 3
References:	Bennett and Tinsley, 1995, "Open File Report 95-663."
Nature of Failure:	Lateral spread, slide, and sand boils.
Comments:	Located at the east of Watsonville, Monterey. Both SPT and CPT test were performed. Surface samples were taken. Grain size characteristices were given in detail. Usually SPT's were made 3-5 ft away from the CPT boreholes to obtain samples. SPT procedures followed the guidelines outlined in D1586-67 (ASTM). Hollow-stem augers (10" outside diameter, 4" inside diameter with split liners) 140 lb "inhole sampling hammer dropped at 30" height by "Safe-T-Driver" hoist. The overall efficiency of the hammer-hoist system is 68 %. Undisturbed samples were taken using thin-walled Shelby tubes. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g In Open File Report 95-663, Holzer et al., the PGA was given as 0.53 g. Watsonville region was shaken at MMI scale of 8.(~ 0.25-0.50 g ?). The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.50-0.65 g. A PGA value of 0.36 g was recorded at Watsonville Based on site response analyses results, a PGA value of 0.46 is adopted. Strong ground motion lasted ~ 6 seconds. CMF3-5-8-10 are ~ 1000 ft apart. The SPT blowcount 26 at depth ~8m was eliminated due to high CPT tip resistance at that depth. Unlikely to be liquefied? SPT performed after the earthquake.

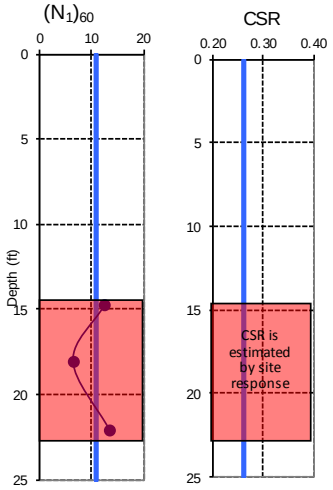
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.120 ± 0.034		
Data Class	A			% Fines	27.3 ± 11.9	32.0	
Critical Depth Range	18.9 - 24.6	20.3		% PI			
Depth to GWT (ft)	18.7	16.1					
σ_v (psf)	2436.4 ± 132.5	2380.9		N	10.3 ± 3.7	9.2	
σ_v' (psf)	2247.1 ± 83.8	2109.4		C_R	0.94	0.95	
a_{max} (g)	0.460 ± 0.069	0.39		C_S	1.00	1.00	
r_d	0.83 ± 0.019	0.92		C_B	1.00	1.00	
CSR	0.263 ± 0.041	0.252		C_E	1.13	1.13	
Equivalent Magnitude	6.93	6.93		C_N	0.97	1.00	
MSF		1.16		$(N_1)_{60}$	10.7 ± 3.8	9.9	
CSRN		0.219					



γ_m=120±3 pcf

γ_s=125±3 pcf

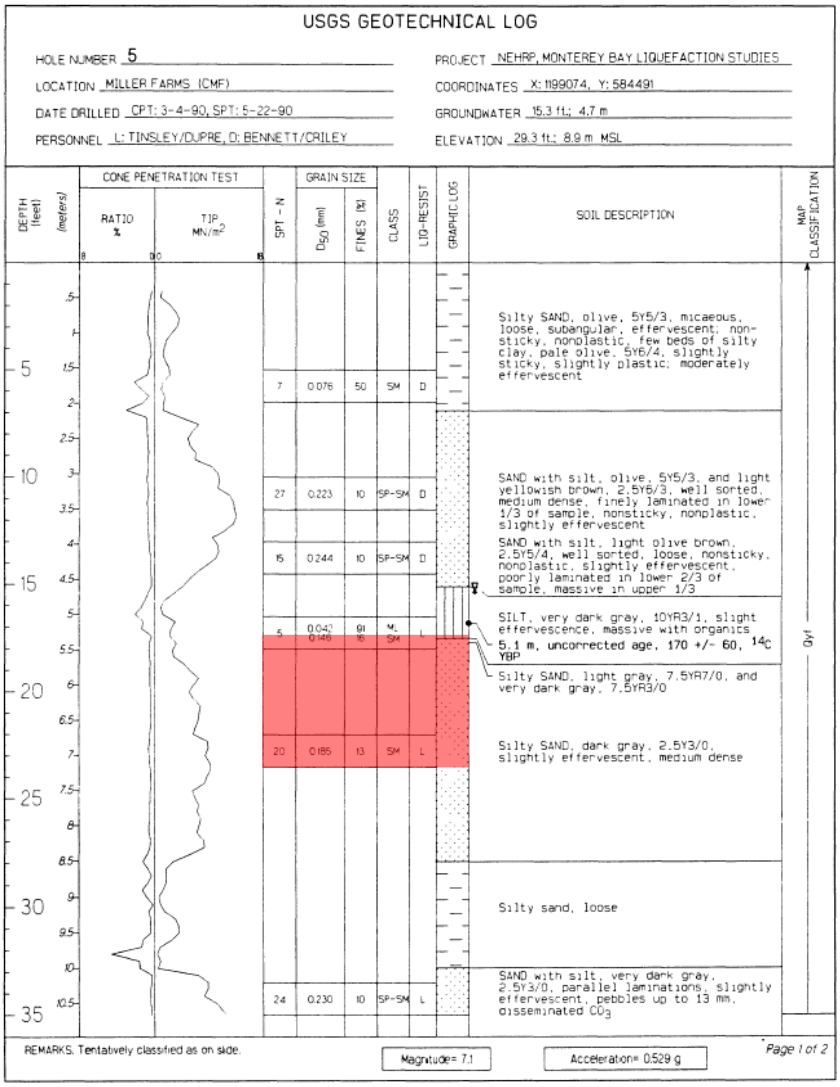


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC %	D_{50} (mm)
4.5	14.8		11	1.14	0.87	***12.4***	***19.0***	***0.154***
5.5	18.0		6	1.03	0.91	***6.4***	***41.0***	***0.086***
6.75	22.1		13	0.96	0.95	***13.5***	***22.0***	***0.120***
					Mean:	10.7	27.3	0.120
					St. Dev.	3.8	11.9	0.034

Case number:	130
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Miller Farm CMF 5
References:	Bennett and Tinsley, 1995, "Open File Report 95-663"
Nature of Failure:	Lateral spread, slide, and sand boils.
Comments:	Located at the east of Watsonville, Monterey. Both SPT and CPT test were performed. Surface samples were taken. Grain size characteristices were given in detail. Usually SPT's were made 3-5 ft away from the CPT boreholes to obtain samples. SPT procedures followed the guidelines outlined in D1586-67 (ASTM). Hollow-stem augers (10" outside diameter, 4" inside diameter with split liners) 140 lb "inhole sampling hammer dropped at 30" height by "Safe-T-Driver" hoist. The overall efficiency of the hammer-hoist system is 68 %. Undisturbed samples were taken using thin-walled Shelby tubes. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g. In Open File Report 95-663, Holzer et al., the PGA was given as 0.53 g. Watsonville region was shaken at MMI scale of 8.(~ 0.25-0.50 g ?). The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.50-0.65 g. A PGA value of 0.36 g was recorded at Watsonville Based on site response analyses results, a PGA value of 0.41g is adopted Strong ground motion lasted ~ 6 seconds. CMF3-5-8-10 are ~ 1000 ft apart. SPT performed after the earthquake.

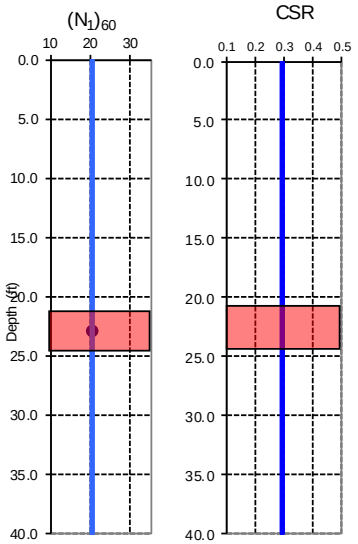
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.185 ± 0.050		
Data Class	B			% Fines	13.0 ± 2.0	13.0	
Critical Depth Range	18.0 - 27.9	23.0		% PI			
Depth to GWT (ft)	15.4	15.4					
σ_v (psf)	2793.6 ± 211.4	2715.1		N	20.0 ± 1.9	20.0	
σ_v' (psf)	2322.8 ± 116.6	2255.6		C_R	0.95	0.95	
a_{max} (g)	0.410 ± 0.062	0.39		C_S	1.00	1.00	
r_d	0.90 ± 0.016	0.91		C_B	1.00	1.00	
CSR	0.290 ± 0.044	0.280		C_E	1.13	1.13	
Equivalent Magnitude	6.9	6.9		C_N	0.95	0.97	
MSF		1.16		$(N_1)_{60}$	20.6 ± 2.0	20.9	
CSRN		0.243					



γ_m = 120±3 pcf

γ_s = 125±3 pcf



CSR is estimated by site response

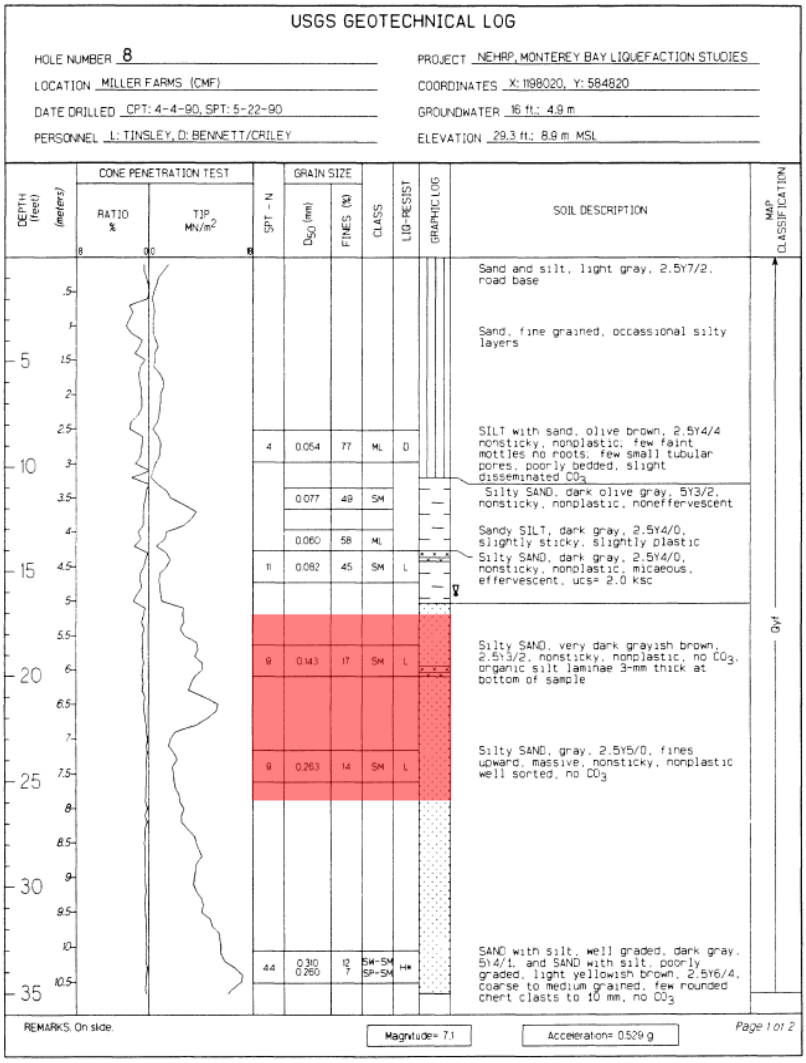
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
7	23.0		20	0.95	0.95	***20.6***
					Mean:	20.6
					St. Dev.	2.0

FC %	D_{50} (mm)
13	***0.185***
16	0.146
13.0	0.185
2.0	0.050

Case number:	131
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Miller Farm CMF 8
References:	Bennett and Tinsley, 1995, "Open File Report 95-663"
Nature of Failure:	Lateral spread, slide, and sand boils.
Comments:	Located at the east of Watsonville, Monterey. Both SPT and CPT test were performed. Surface samples were taken. Grain size characteristices were given in detail. Usually SPT's were made 3-5 ft away from the CPT boreholes to obtain samples. SPT procedures followed the guidelines outlined in D1586-67 (ASTM). Hollow-stem augers (10" outside diameter, 4" inside diameter with split liners) 140 lb "inhole sampling hammer dropped at 30" height by "Safe-T-Driver" hoist. The overall efficiency of the hammer-hoist system is 68 %. Undisturbed samples were taken using thin-walled Shelby tubes. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g In Open File Report 95-663, Holzer et al., the PGA was given as 0.53 g. Watsonville region was shaken at MMI scale of 8 (~ 0.25-0.50 g ?). The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.50-0.65 g. A PGA value of 0.36 g was recorded at Watsonville Based on site response analyses results, a PGA value of 0.46 g is adopted. Strong ground motion lasted ~ 6 seconds. CMF3-5-8-10 are ~ 1000 ft apart. SPT performed after the earthquake.

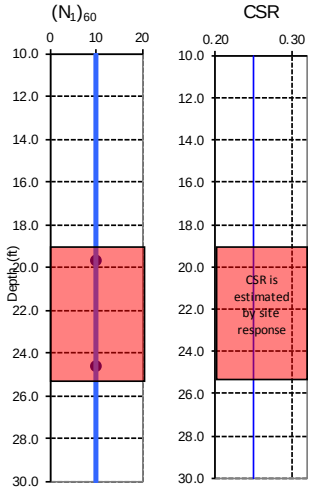
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.203 ± 0.085		
Data Class	B			% Fines	15.5 ± 2.1	25.0	
Critical Depth Range	16.4 - 26.2	19.7		% PI			
Depth to GWT (ft)	16.1	14.4					
σ_v (psf)	2237.5 ± 203.4	2318.3		N	9.0 ± 0.9	8.8	
σ_v' (psf)	1910.0 ± 108.0	1984.1		C_R	0.94	0.95	
a_{max} (g)	0.460 ± 0.069	0.39		C_S	1.00	1.00	
r_d	0.73 ± 0.013	0.93		C_B	1.00	1.00	
CSR	0.250 ± 0.040	0.274		C_E	1.13	1.13	
Equivalent Magnitude	6.93	6.93		C_N	1.05	1.03	
MSF		1.16		$(N_1)_{60}$	10.0 ± 1.0	9.8	
CSRN		0.234					



γ_m= 100 ± 3 pcf

γ_s= 120 ± 3 pcf

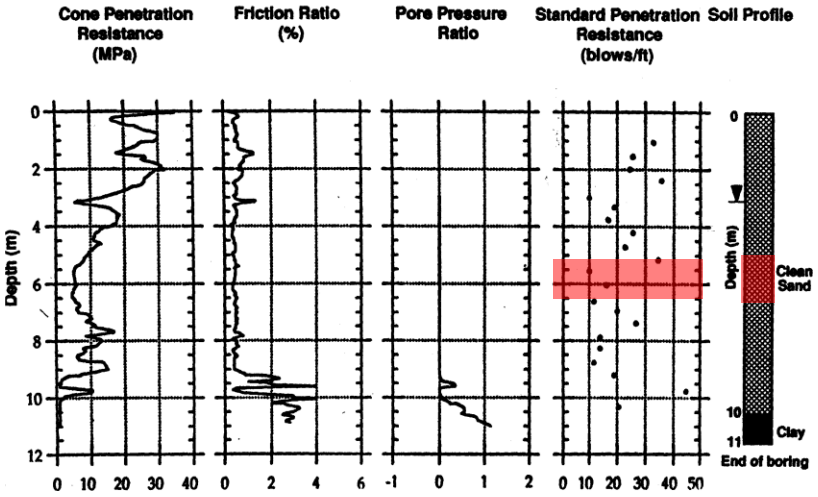


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	D50
6	19.7		9	1.08	0.92	***10.2***	***17.0***	***0.143***
7.5	24.6		9	1.00	0.97	***9.9***	***14.0***	***0.263***
					Mean:	10.0	15.5	0.203
					St. Dev.	1.0	2.1	0.085

Case number:	132
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Port of Oakland (POO7-2)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	Significant settlements (max ~ 30 cm.) occurred over most of the fill. Significant lateral spreading along most of the fill, causing longitudinal fissures. A 10 m diameter. 0.3 m high sand boil was observed in the paved container south of the C.F.S. Building near Berth 38.
Comments:	Located immediately south of the San Francisco - Oakland Bay Bridge Most of the surface fills at the Port of Oakland were hydraulically placed to sea level, above which a combination of hydraulic fill and dumped fill was placed. A relatively thick asphalt was laid on top of the unimproved fill. Underlain by 180 m of stiffer Pleistocene glacial and interglacial deposits. Upper 3-4 m, surface layer of high penetration resistance; below this layer lies a looser deposits of marine sand. Liquefaction appears to have occurred in the depth range of 4-8 m. PGA is recorded as 0.29 & 0.27g at the Oakland Outer Harbor seismogram which is located on a 155 m deep soil site. PGA is estimated as 0.22 g based on site response analyses results. Measurements performed after the earthquake. The POO7-2 and POO7-3 are approximately 15 m apart.

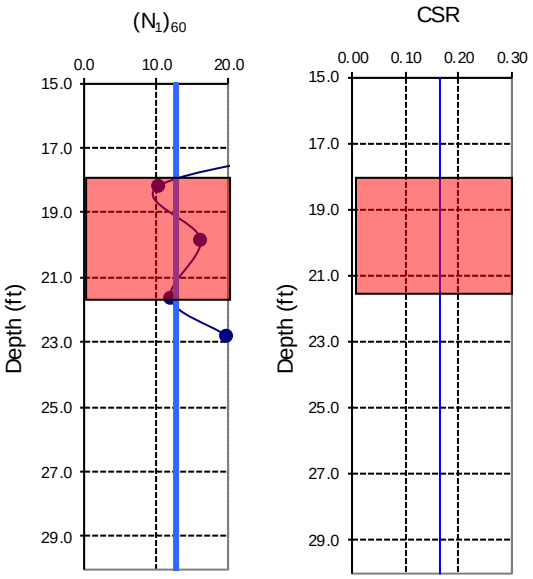
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	yes	D_{50}	0.300 ± 0.050		? ? ?
Data Class	A			% Fines	3.0 ± 2.0	3.0	sand sand sand
Critical Depth Range	18.0 - 22.3	20.7	3 5.50 8.75	% PI			
Depth to GWT (ft)	9.8	9.8	3 m				
σ_v (psf)	2472.9 ± 98.6	2527.1	0.54 0.99 1.58kg/cm^2	N	12.7 ± 3.1	14.4	10 10 12
σ_v' (psf)	1828.1 ± 64.1	1858.8	0.54 0.74 1.00kg/cm^2	C_R	0.93	0.95	
a_{max}	0.220 ± 0.033	0.28	0.285	C_S	1.10	1.10	0.900
r_d	0.86 ± 0.018	0.92	0.97 0.95 0.91	C_B	1.00	1.00	
CSR	0.165 ± 0.025	0.229	0.18 0.234 0.266	C_E	0.92	0.92	
Equivalent Magnitude	6.93	6.93		C_N	1.08	1.11	1.3 1.15 1.0
MSF		1.16		$(N_1)_{60}$	12.8 ± 3.1	15.4	11.7 10.4 10.8
CSRN		0.194	0.17 0.221 0.251				



γ_m=120±3 pcf

γ_s=125±3 pcf



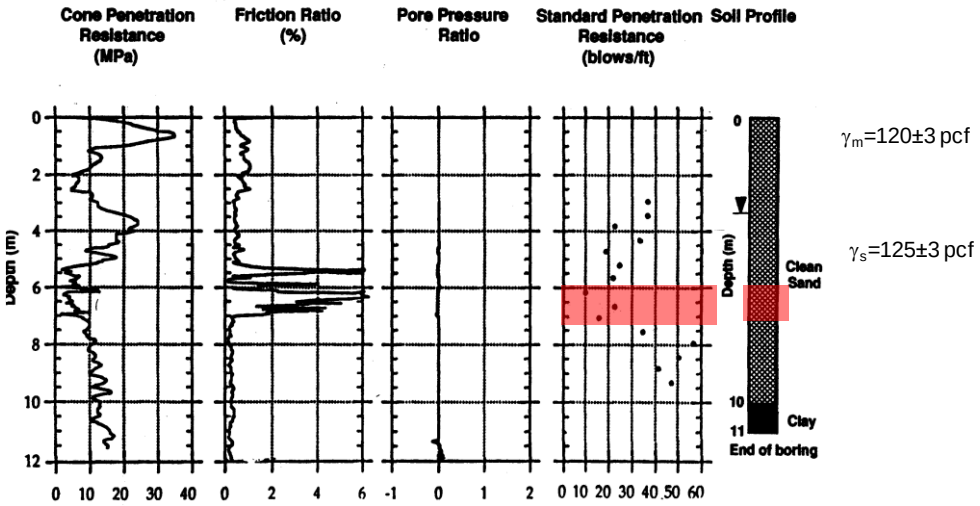
CSR is estimated by site response

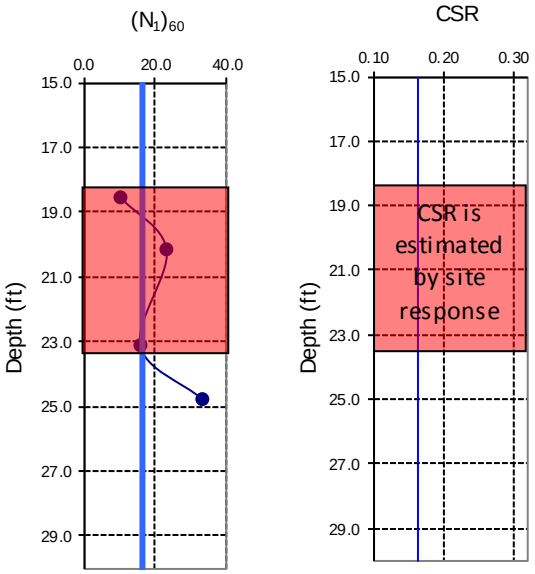
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.15	16.9		35	1.14	0.90	36.1
5.55	18.2		10	1.11	0.91	***10.2***
6.05	19.8		16	1.08	0.93	***16.2***
6.6	21.7		12	1.05	0.94	***12.0***
6.95	22.8		20	1.03	0.95	19.8
					Mean:	12.8
					St. Dev.	3.1

Case number:	133
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Port of Oakland (POO7-3)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	Significant settlements (max ~ 30 cm.) occurred over most of the fill. Significant lateral spreading along most of the fill, causing longitudinal fissures A 10 m diameter. 0.3 m high sand boil was observed in the paved container south of the C.F.S. Building near Berth 38. Muddy sand unit at depths 5-7 m
Comments:	Located immediately south of the San Francisco - Oakland Bay Bridge. Most of the surface fills at the Port of Oakland were hydraulically placed to sea level, above which a combination of hydraulic fill and dumped fill was placed. A relatively thick asphalt was laid on top of the unimproved fill Underlain by 180 m of stiffer Pleistocene glacial and interglacial deposits Upper 3-4 m, surface layer of high penetration resistance; Below this layer lies a looser deposits of marine sand. Borehole is characterized by high N values (~21) for the entire submerged soil profile, except for a muddy-sand unit between 5-7m depth. Liquefied? PGA is recorded as 0.29 & 0.27g at the Oakland Outer Harbor seismogram which is located on a 155 m deep soil site. PGA is estimated as 0.22 g based on site response analyses results. Measurements performed after the earthquake. The POO7-2 and POO7-3 are approximately 15 m apart. The SPT blowcount (~22) at depth ~6.6 m does not appear on the borelog excel spreadsheet given in Appedix of Mitchell et al. Eliminated for the analysis.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	marginal	Yes	D_{50}	0.320 ± 0.050		? ? ?
Data Class	A			% Fines	5.0 ± 2.0	3.0	sand, sand/sand &fines
Critical Depth Range	16.4 - 23.0	20.7	6.2 7.0 m	% PI			
Depth to GWT (ft)	9.8	9.8	3 m				
σ_v (psf)	2411.4 ± 142.9	2527.1	1.12 1.26 kg/cm^2	N	16.2 ± 6.4	16.0	10.0 15.5
σ_v' (psf)	1797.2 ± 82.0	1858.8	0.80 0.86 kg/cm^2	C_R	0.92	0.95	
a_{max}	0.220 ± 0.033	0.28	0.285	C_S	1.10	1.10	0.900
r_d	0.83 ± 0.018	0.92	0.94 0.93	C_B	1.00	1.00	
CSR	0.164 ± 0.024	0.229	0.244 0.252	C_E	0.92	0.92	
Equivalent Magnitude	6.93	6.93		C_N	1.09	1.10	1.11 1.08
MSF		1.16		$(N_1)_{60}$	16.4 ± 6.5	17.0	10.0 15.5
CSRN		0.194	0.23 0.238				



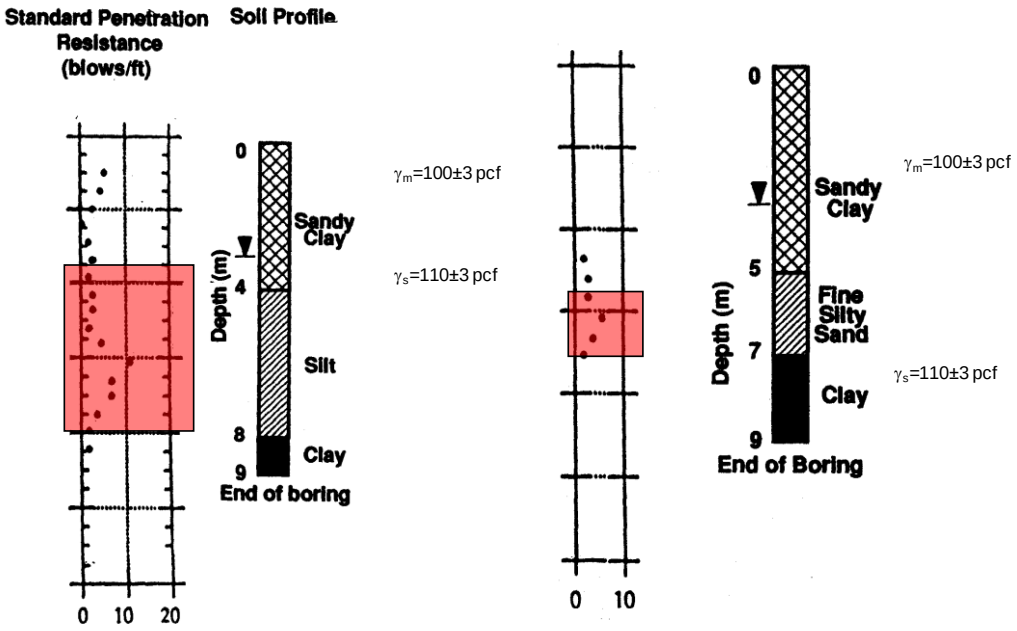


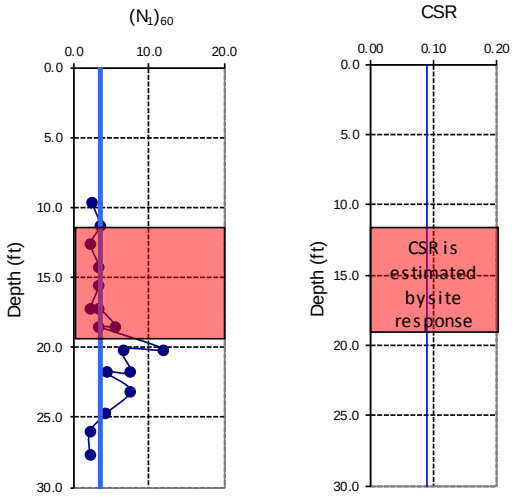
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
5.65	18.5		10	1.11	0.91	**10.2**
6.15	20.2		23	1.08	0.93	***23.2***
7.05	23.1		16	1.03	0.95	***15.8***
7.55	24.8		34	1.00	0.97	33.2
					Mean:	16.4
					St. Dev.	6.5

Case number:	134
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Port of Richmond (POR-2&3&4)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	Four large and a dozen smaller sand boils vented fine sand and silty sand form the underlying fill. Seepage continued for as long as 24 hours. Ground settlement and minor lateral spreading both aprroximately 2-8 cm.
Comments:	The liquefied area is a zone approximately 250 ft wide and 1000 ft long at the foot of Harbor Way Road. The site represents the most distant point, approximately 85 km north of the rupture zone. The liquefaction zone overlies deposits of soft bay mud, underlain by deeper deposits of stiffer, overconsolidated noncohesive and cohesive soils. Up to 0.8 m depth the soil is characterized by oxidized, tan-brown crusted silty sand. 0.8-1.8 m oxidized tan-brown silty sand. 1.8-4.1 m oxidized tan-brown sandy clay to reduced sandy clay.4.1-7.8 m, is a hydraulic fill layer of olive-gray fine-silty sand containing shell fragments. The thick deposits amplified the level of ground motion at the site. At adjacent sites (POR-3 & 4) SPT, CPT and shear wave velocity tests were performed. POR-2 &3&4 are aprroximately 10-15m apart. Correlated? PGA is estimated as 0.18 g by performing a site response analysis (SHAKE) using the input motion from Richmond City Hall (PGA=0.13g ~2km) by Kayen et al. Based on site response analyses results performed by using the recording scaled to the geometric mean of the two orthonormal components. PGA is estimated as 0.15. The SPT blowcount ~11 is eliminated due to unrepresentative nature of it. Depth to water table in Kayen et al. (1998) is reported as 2.5 m contrary to 3.5 m reported in Mitchell et al. (1994).

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.090 ± 0.050		? ?
Data Class	A			% Fines	50.0 ± 2.0	50.0	sand wt. fines sand
Critical Depth Range	19.0 - 13.1	19.4	5.25 7.25	% PI			
Depth to GWT (ft)	11.5	11.5	2.5				
σ_v (psf)	1652.0 ± 114.0	2025.9	0.95 1.35 kg/cm^2	N	3.2 ± 1.0	4.3	2 4
σ_v' (psf)	1366.2 ± 63.2	1524.6	0.67 0.85 kg/cm^3	C_R	0.89	0.95	
a_{max}	0.150 ± 0.023	0.18	0.160	C_S	1.10	1.10	0.900
r_d	0.78 ± 0.017	0.93	0.95 0.93	C_B	1.00	1.00	
CSR	0.090 ± 0.014	0.142	0.135 0.148	C_E	0.92	0.92	
Equivalent Magnitude	6.93	6.93		C_N	1.24	1.27	1.2 1.08
MSF		1.16	1.06	$(N_1)_{60}$	3.5 ± 1.1	5.1	2.2 3.9
CSRN		0.119	0.127 0.14				



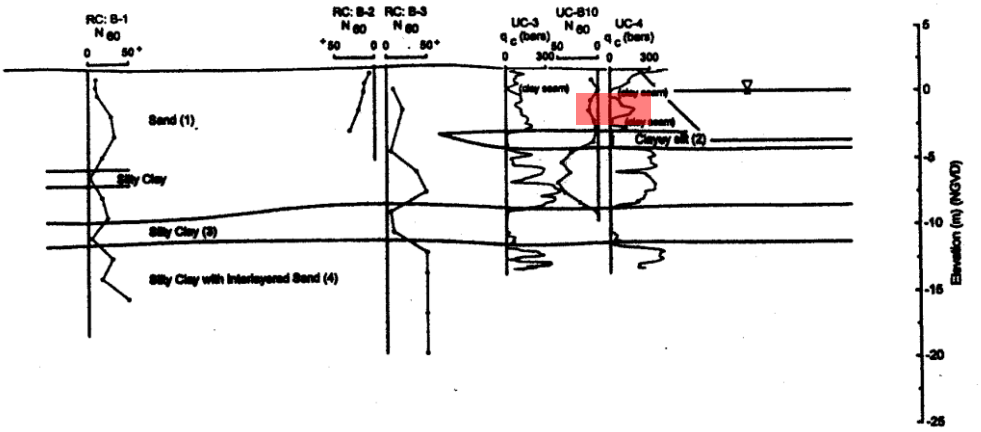


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		
2.95	9.7		2	1.48	0.80	2.4	POR-2	sandy clay
3.45	11.3		3	1.37	0.83	3.4	POR-2	sandy clay
3.85	12.6		2	1.33	0.84	2.3	POR-2	sandy clay
4.35	14.3		3	1.29	0.87	***3.4***	POR-2	
4.75	15.6		3	1.26	0.88	***3.3***	POR-2	
5.25	17.2		2	1.22	0.90	***2.2***	POR-2	
5.25	17.2		3	1.22	0.90	***3.3***	POR-3&4	
5.65	18.5		5	1.19	0.91	***5.5***	POR-2	
5.65	18.5		3	1.19	0.91	***3.3***	POR-3&4	
6.15	20.2		11	1.16	0.93	12.0	POR-2	
6.15	20.2		6	1.16	0.93	6.5	POR-3&4	
6.65	21.8		7	1.14	0.94	7.6	POR-2	
6.65	21.8		4	1.14	0.94	4.3	POR-3&4	
7.05	23.1		7	1.11	0.95	7.5	POR-2	
7.55	24.8		4	1.09	0.97	4.3	POR-2	
7.95	26.1		2	1.07	0.98	2.1	POR-2	
8.45	27.7		2	1.05	0.99	2.1	POR-2	
					Mean:	3.5		
					St. Dev.	1.1		

Case number:	135
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Sandholdt Road UC-B10
References:	Boulanger et al. (1996), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	Extensive ground deformations were observed along Sandholdt Road, including the portion alongside the MBARI facilities. Cracks parallel to the front line. Away from Sandholdt Road, surface evidence of liquefaction appeared to occur in localized areas.
Comments:	Located on Monterey Bay about midway between the cities of Santa Cruz and Monterey, CA. The MBARI facilities include three main buildings and a pier located along Sandholt Road on the Moss Landing spit. Historical shoreline suggest that the shoreline in the vicinity of the site has fluctuated significantly. Shallower soil strata may have been deposited recently. The area is underlain by Holocene deposits consisting of relatively thick sands offshore and estuarine and fluvial deposits onshore, up to about 60-m deep. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m The site is underlain by an 8-12 m thick layer of poorly graded sand with interlayers of gravelly sand and sandy gravel in its lower portion and occasional thin layers of clayey silt throughout. PGA is recorded in a four-story building 8km from rupture plane as 0.39 g which is underlain by alluvium. Also San Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted in the range of 0.2-0.3 g. PGA is chosen for the analysis as 0.26 g almost identical with the value chosen by the authors of the source document (0.25 g) ~30 ft away from 30 degree 15' slope. SPT performed after the earthquake.

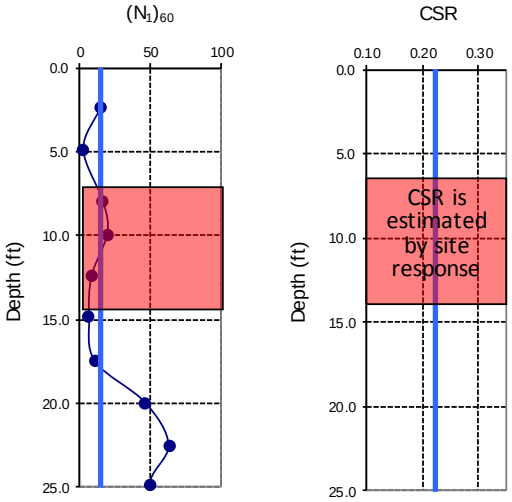
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.800 ± 0.050		
Data Class	A			% Fines	2.0 ± 2.0	2.0	
Critical Depth Range	8.0 - 13.0	9.8		% PI			
Depth to GWT (ft)	5.6	5.9					
σ_v (psf)	1203.9 ± 102.5	1148.7		N	9.2 ± 3.8	9.5	
σ_v' (psf)	898.8 ± 55.2	898.1		C_R	0.81	0.85	
a_{max} (g)	0.260 ± 0.039	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.008	0.97		C_B	1.00	1.00	
CSR	0.224 ± 0.035	0.226		C_E	1.25	1.25	
Equivalent Magnitude	6.93	6.93		C_N	1.53	1.52	
MSF		1.16		$(N_1)_{60}$	14.3 ± 5.9	15.3	
CSRN		0.177					



γ_m=110±3 pcf

γ_s=120±3 pcf

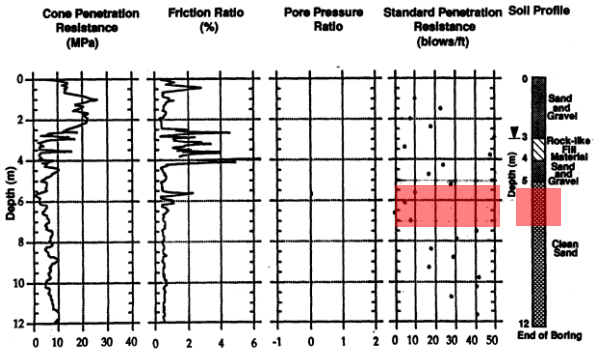


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
0.7	2.4		9	2.00	0.63	*14.1*
1.5	4.9		1	1.99	0.70	*1.7*
2.4	8.0		10	1.67	0.77	***16.1***
3.0	9.9		12	1.56	0.80	***18.9***
3.8	12.4		5	1.45	0.84	***7.6***
4.5	14.9		4	1.36	0.87	*5.9*
5.3	17.5		7	1.28	0.90	*10.1*
6.1	19.9		33	1.21	0.93	*46.3*
6.9	22.5		46	1.15	0.95	*63.0*
7.6	24.8		37	1.11	0.97	*49.6*
8.4	27.4		50	1.06	0.99	*65.6*
9.1	29.8		42	1.03	1.00	*53.9*
9.9	32.4		24	0.99	1.00	*29.7*
10.6	34.8		3	0.96	1.00	*3.6*
					Mean:	14.2
					St. Dev.	5.9

Case number:	136
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	San Francisco Oakland Bay Bridge (SFOBB-1&2)
References:	Mitchell et al. (1994), "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT and Shear Wave Velocity Kayen et al. (1998), "Soil Liquefaction in the East Bay during the earthquake
Nature of Failure:	Significant settlements (max ~ 40 cm.) occurred over most of the mole. In some places differential settlements produced hummocky surface with permanent waves as large as 15 cm in amplitude. Significant lateral spreading along most of the fill, causing longitudinal fissures. Many of the fissures exuded fine to silty sand. Numerous sand boils erupted along the median strip of the roadway. Fissure width openings ~7-8 cm and exhibited maximum differential vertical offset of 3-5 cm.
Comments:	The San Francisco- Oakland Bay Bridge mole (peninsula-approach fill) is located at immediate south of Emeryville. SFOBB-1 was drilled at the easternmost end of the Toll Plaza Parking lot. No piezocone pore-pressure generation during cone penetration until bay mud was reached at 15 m depth. 0-2.8 m depth, moderately dense surface crust; 2.8-4.5 m interbedded sequence of cohesionless silty sand and cohesive clay. Between 4.5 to 7.5 m lies cohesionless material with low q_c values ~5MPa. Drill rod and hammer sank under their own weight at 6.6 m depth. PGA is recorded as 0.29 & 0.27 at the Oakland Outer Harbor seismogram which is located on a 155 m deep soil site. PGA is estimated as 0.25 g based on site response analyses results. The average of recorded and site response based estimated PGA values was adopted. Depth to water table in Kayen et al. (1998) is reported as 2.0 m contrary to 3.0 m reported in Mitchell et al. (1994). Measurements performed after the earthquake. The measurements could be correlated with SFOBB-2.

Summary of Data

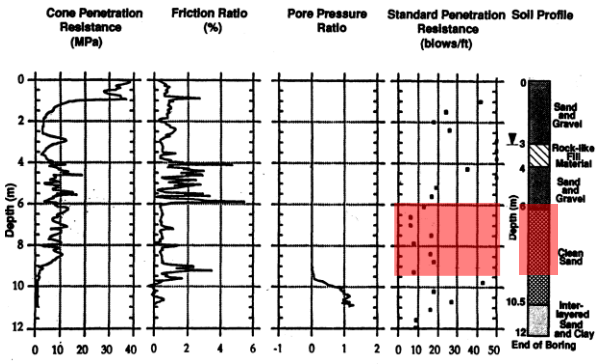
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes	Yes	D_{50}	0.280 ± 0.050		?
Data Class	A			% Fines	8.0 ± 2.0	8.0	sand sand
Critical Depth Range	18.0 - 23.0	20.7	6.7m 6.1m	% PI			
Depth to GWT (ft)	9.8	9.8	2.0 m				
σ_v (psf)	2513.9 ± 111.4	2464.5	1.21 1.10 kg/cm^2	N	7.5 ± 2.0	7.5	1 5
σ_v' (psf)	1848.6 ± 69.5	1796.1	0.74 0.69 kg/cm^2	C_R	0.93	0.95	
a_{max}	0.270 ± 0.041	0.27	0.285	C_S	1.20	1.20	0.900
r_d	0.79 ± 0.013	0.92	0.93 0.94	C_B	1.00	1.00	
CSR	0.190 ± 0.029	0.222	0.283 0.278	C_E	0.92	0.92	
Equivalent Magnitude	6.93	6.93		C_N	1.07	1.10	1.15 1.18
MSF		1.16		$(N_1)_{60}$	8.2 ± 2.2	8.6	1 5.3
CSRN		0.189	0.267 0.262				



γ_m=120±3 pcf

γ_s=130±3 pcf

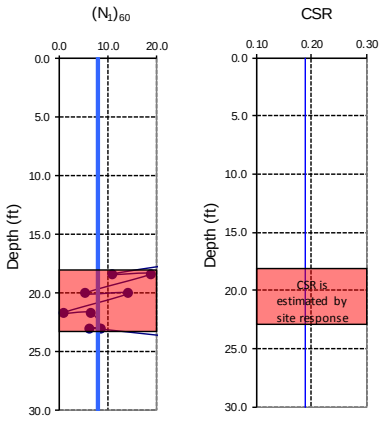
γ_s=115±3 pcf



γ_m=120±3 pcf

γ_s=130±3 pcf

γ_s=125±3 pcf

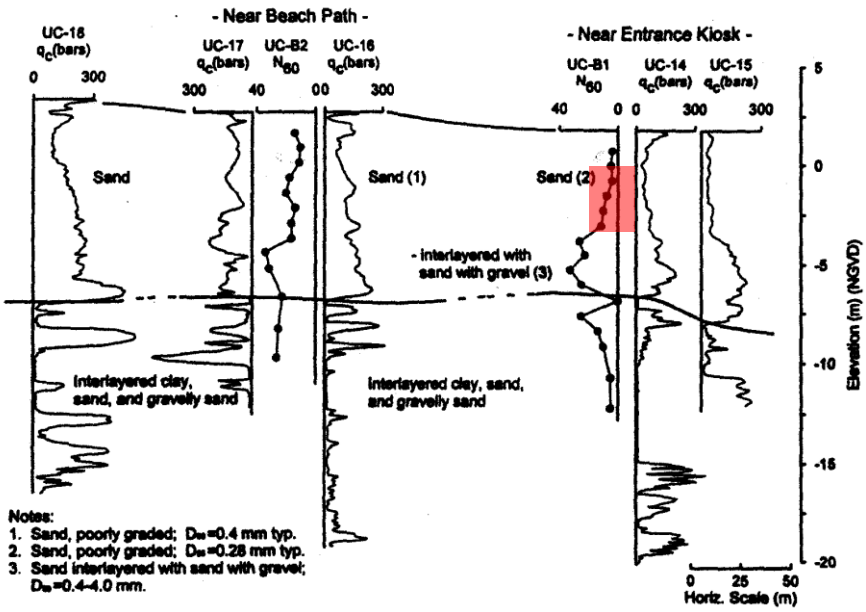


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		
5.2	17.1		28	1.14	0.90	31.5	SFOBB-1	
5.6	18.4		10	1.11	0.91	***11.1***	SFOBB-1	
5.6	18.4		17	1.11	0.91	18.9	SFOBB-2	Gravel
6.1	20.0		5	1.08	0.93	5.5	SFOBB-1	Fines?
6.1	20.0		13	1.08	0.93	14.3	SFOBB-2	Gravel?
6.6	21.7		1	1.05	0.94	1.1	SFOBB-1	Eliminate
6.6	21.7		6	1.05	0.94	***6.5***	SFOBB-2	
7	23.0		8	1.03	0.95	***8.6***	SFOBB-1	
7	23.0		6	1.03	0.95	***6.5***	SFOBB-2	
7.5	24.6		41	1.00	0.97	43.7	SFOBB-1	
					Mean:	8.2		
					St. Dev.	2.2		

Case number:	137
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Moss Landing State Beach UC-B1
References:	Boulanger et al. (1996), "Liquefaction at Moass Landing During Loma
Nature of Failure:	Liquefaction-induced lateral spreading caused extensive damage to the access road. The tide gate was destroyed due to liquefaction flow slide. Road surface slumping about 1.3 m. Lateral spreading were roughly 0.3-0.6 m horizontal with vertical offset up to 0.3 m. Sand boils and ground cracks on the north side of the access road.
Comments:	Located on Monterey Bay about midway between the cities of Santa Cruz and Monterey, CA. The area is underlain by Holocene deposits consisting of relatively thick sands offshore and estuarine and fluvial deposits onshore, up to about 60-m deep. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m. The upper 8-10 m of soils encountered were poorly graded, clean sands to gravelly sands having no more than 3 % fines. Underlying the sands was a complex sequence of interlayered clay, sand, and gravelly sand. PGA is recorded in a four-story building 8km from rupture plane as 0.39 g which is underlain by alluvium. Also San Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3g PGA adopted for the analysis based on site response analyses results (~0.29 g) is slightly higher than the value suggested by the authors of the source document. SPT performed after the earthquake.

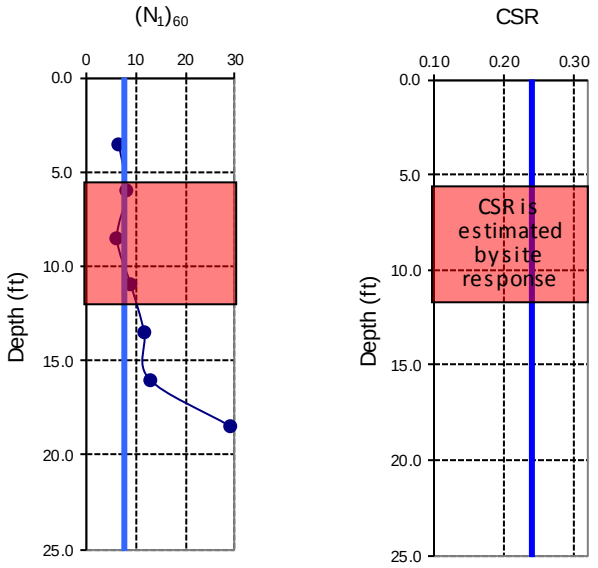
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.260 ± 0.050		
Data Class	A			% Fines	1.7 ± 0.6	1.0	
Critical Depth Range	5.9 - 12.0	11.2		% PI			
Depth to GWT (ft)	5.9	5.9					
σ_v (psf)	1030.5 ± 128.6	1274.0		N	5.1 ± 0.9	6.3	
σ_v' (psf)	840.4 ± 68.1	960.7		C_R	0.79	0.85	
a_{max} (g)	0.290 ± 0.044	0.28		C_S	1.00	1.00	
r_d	0.95 ± 0.010	0.97		C_B	1.00	1.00	
CSR	0.240 ± 0.035	0.234		C_E	1.25	1.25	
Equivalent Magnitude	6.93	6.93		C_N	1.59	1.53	
MSF		1.16		$(N_1)_{60}$	7.9 ± 1.4	10.3	
CSRN		0.187					



γ_m=110±3 pcf

γ_s=125±3 pcf

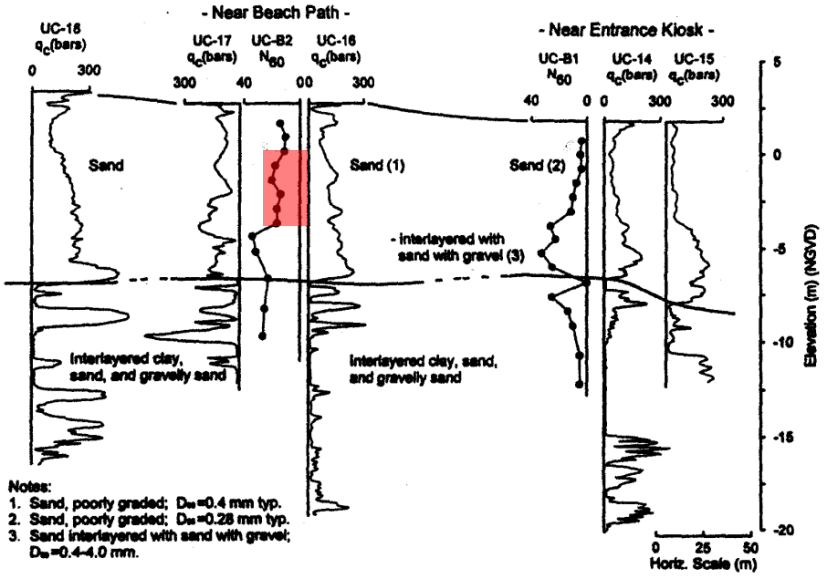


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
1.1	3.5		4	2.00	0.66	6.6	0.0
1.8	6.0		5	1.80	0.73	***8.2***	***2.0***
2.6	8.5		4	1.61	0.78	***6.3***	***2.0***
3.4	11.0		6	1.48	0.82	***9.1***	***1.0***
4.1	13.5		8	1.37	0.86	11.7	1.0
4.9	16.0		9	1.29	0.89	12.8	1.0
5.6	18.5		21	1.21	0.91	29.1	1.0
					Mean:	7.9	1.7
					St. Dev.	1.4	0.6

Case number:	138
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Moss Landing State Beach UC-B2
References:	Boulanger et al. (1996), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	Liquefaction-induced lateral spreading caused extensive damage to the access road. The tide gate was destroyed due to liquefaction flow slide. Road surface slumping about 1.3 m. Lateral spreading were roughly 0.3-0.6 m horizontal with vertical offset up to 0.3 m. Sand boils and ground cracks on the north side of the access road.
Comments:	Located on Monterey Bay about midway between the cities of Santa Cruz and Monterey, CA. The area is underlain by Holocene deposits consisting of relatively thick sands offshore and estuarine and fluvial deposits onshore, up to about 60-m deep. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m. The upper 8-10 m of soils encountered were poorly graded, clean sands to gravelly sands having no more than 3 % fines. Underlying the sands was a complex sequence of interlayered clay, sand, and gravelly sand. PGA is recorded in a four-story building 8km from rupture plane as 0.39 g which is underlain by alluvium. Also San Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3g Based on site response analyses results, a PGA value of 0.24 g is adopted which is slightly lower than the value suggested by the authors of the source document (0.25 g). SPT performed after the earthquake.

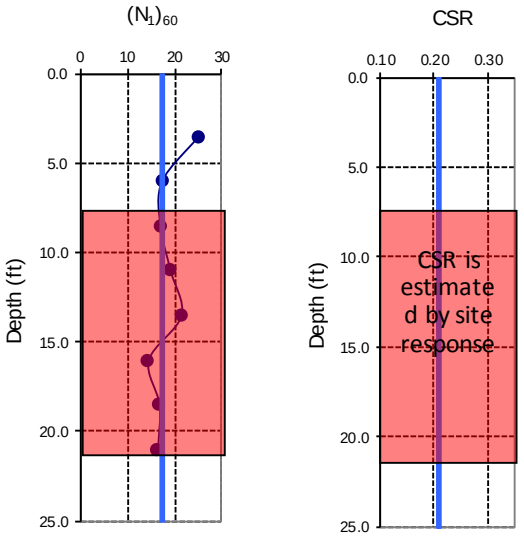
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.400 ± 0.050		
Data Class	A			% Fines	1.0 ± 2.0	1.0	
Critical Depth Range	9.0 - 22.0	16.1		% PI			
Depth to GWT (ft)	9.0	8.5					
σ_v (psf)	1892.5 ± 272.9	1879.7		N	13.3 ± 1.8	12.8	
σ_v' (psf)	1486.9 ± 140.7	1399.3		C_R	0.88	0.95	
a_{max} (g)	0.240 ± 0.036	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.013	0.95		C_B	1.00	1.00	
CSR	0.210 ± 0.031	0.229		C_E	1.25	1.25	
Equivalent Magnitude	6.93	6.93		C_N	1.19	1.20	
MSF		1.16		$(N_1)_{60}$	17.4 ± 2.4	18.4	
CSRN		0.188					



γ_m=120±3 pcf

γ_s=125±3 pcf

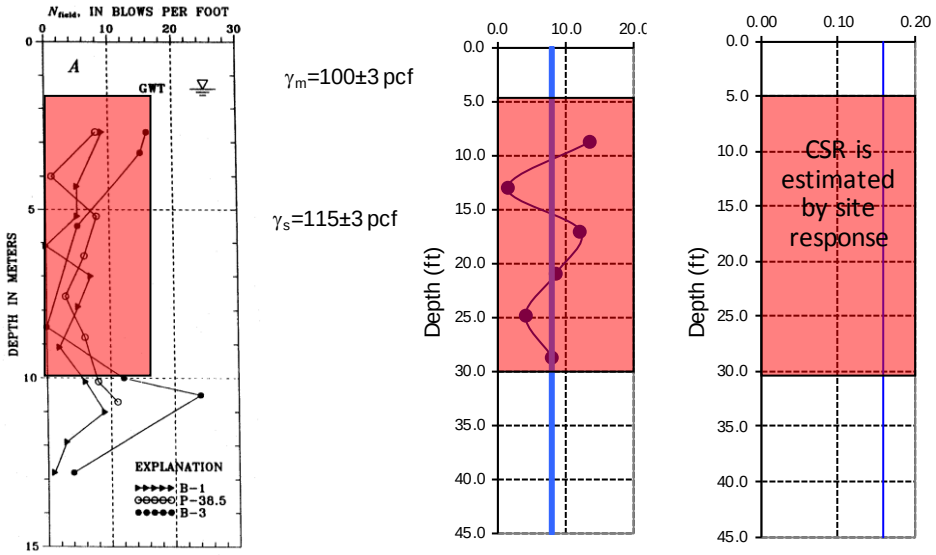


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1.1	3.5		15	2.00	0.66	24.9
1.8	6.0		11	1.71	0.73	17.2
2.6	8.5		12	1.44	0.78	***16.8***
3.4	11.0		14	1.33	0.82	***19.0***
4.1	13.5		16	1.25	0.86	***21.3***
4.9	16.0		11	1.18	0.89	***14.4***
5.6	18.5		13	1.12	0.91	***16.7***
6.4	21.0		13	1.08	0.94	***16.4***
					Mean:	17.4
					St. Dev.	2.4

Case number:	139
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Treasure Island Naval Station
References:	De Alba et al., Youd and Shakal (1994), "Deep Instrumentation Array at the Treasure Island Naval Station" Rollins and Mchood (1994), "Ground Response on Treasure Island"
Nature of Failure:	No surface evidence of liquefaction; however significant change in frequency content of the ground motion after 15 s. compared to the recorded motion at 322 ft.
Comments:	Treasure Island is a 400-acre manmade island immediately northwest of the rock outcrop on Yerba Buena Island in San Francisco Bay. The original surface soils included a shallow-water sandbar or spit extending northwestward from Yerba Buena Island, and soft silty clay known as bay mud surrounding the sandbar to the north and east. Approximately 65% of the island was built on the sandbar and the rest on the bay mud. Treasure Island was constructed by hydraulic and clamshell dredging. A perimeter rock dike was built in two to four stages on a bed of coarse sand placed over bay mud. The structure is essentially an upstream-constructed hydraulic fill. The soils on Treasure Island may be grouped into four broad categories: fill, native shoal sand, recent bay sedimentary deposits, and older bay sedimentary deposits. Both the fill and native shoal sand consists predominantly of sand and varying amounts of clay, silt and gravel; however, the fill is somewhat looser. The depth and thickness of soil layers vary significantly around the island. The thickness of the fill and native shoal sand ranges from 10.7 m at the south end to 15.2 m in the north. The depth of recent bay sedimentary deposits ranges from 10.7 m in the south. Bedrock elevation has been confirmed at a depth of 85 m. PGA is recorded on Yerba Buena Island as 0.06 and 0.03 g (rock outcrop) and 0.16 and 0.11 g onTreasure Island. A PGA value of 0.18 g is adopted based on site response analyses results. Depth to water table is reported as 1.5 m.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.170 ± 0.050		
Data Class	A			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	29.5 - 4.9	21.3		% PI			
Depth to GWT (ft)	4.9	4.9					
σ_v (psf)	1907.0 ± 473.3	2422.7		N	5.1 ± 3.0	4.3	
σ_v' (psf)	1139.3 ± 219.5	1399.3		C_R	0.90	0.95	
a_{max}	0.180 ± 0.027	0.16		C_S	1.10	1.10	
r_d	0.88 ± 0.012	0.92		C_B	1.00	1.00	
CSR	0.160 ± 0.028	0.165		C_E	1.13	1.13	
Equivalent Magnitude	6.93	6.93		C_N	1.36	1.24	
MSF		1.16		$(N_1)_{60}$	7.7 ± 4.6	6.4	
CSRN		0.137					



P-38.5

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.7	8.9		8	1.74	0.79	***13.6***
4	13.1		1	1.51	0.85	***1.6***
5.2	17.1		8	1.37	0.90	***12.2***
6.4	21.0		6	1.26	0.94	***8.8***
7.6	24.9		3	1.17	0.97	***4.2***
8.8	28.9		6	1.10	1.00	***8.2***
					Mean:	8.1
					St. Dev.	4.6

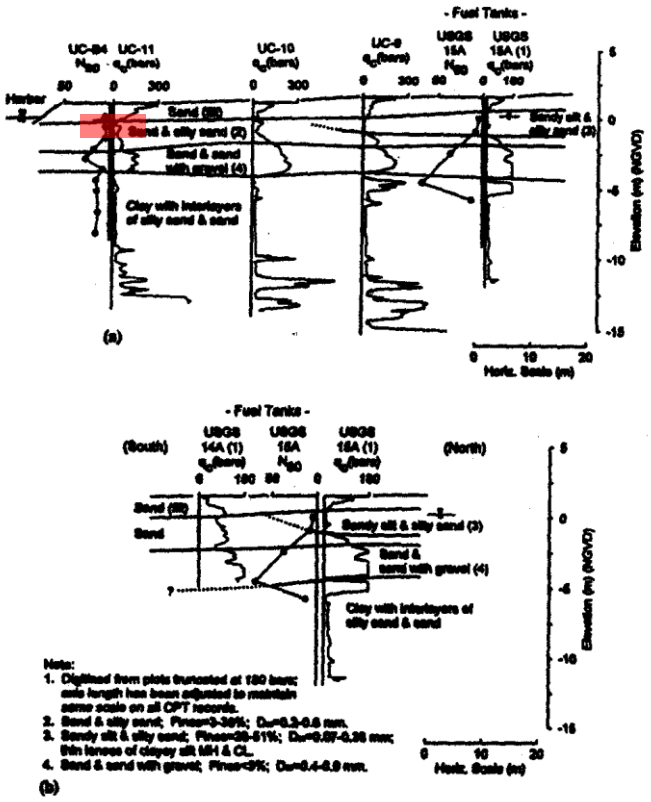
B-1

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.7	8.9		9	1.74	0.79	***15.3***
4.2	13.8		5	1.49	0.86	***8.0***
5.2	17.1		5	1.37	0.90	***7.7***
6	19.7		0	1.29	0.92	***0.0***
7	23.0		7	1.21	0.95	***10.1***
7.9	25.9		5	1.15	0.98	***7.0***
9.1	29.9		2	1.08	1.00	***2.7***
					Mean:	7.3
					St. Dev.	5.0

Case number:	140
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Woodward Marine UC-B4
References:	Boulanger et al. (1996), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	Liquefaction-induced ground spreading was evident across much of the site. Along the shoreline, jetty piles were displaced horizontally, and the boat ramp and adjacent asphalt pavement experienced cracking and settlement. Several cracks up to 5 cm wide. The oil and gas tanks settled and tilted 1-6^{o}. Gray coarse to medium grained sand was vented through some of the cracks. Sand dikes were traced from the ground surface to a depth of about 0.8 m through a gravelly sand and gray, medium-grained sand to a gray silty sand.
Comments:	Located on Salomon Way off Sandholdt Road near the northern end of the Moss Landing spit and along the South Harbor. The Woodward Marine facilities include a boat launch ramp, fuel service dock and oil and gas storage tanks. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m. A ~1m thick layer of sandy fill blankets the site. By the shoreline, the fill is underlain by a roughly 2.2 m thick layer of sand and silty sand. By the fuel tanks, the fill is underlain by a 1-2 m thick layer of soft silty sand, sandy silt and silt. This soft layer of silty sand to silt is underlain by a thin zone of sand which appears similar to the sand encountered at similar depths near the shoreline. These materials are underlain by a ~ 2m thick layer of denser sand. Beneath this layer, there is a layer of high plasticity clay with interlayers of dense silty sand and sand. PGA is recorded in a four-story building 8 km from rupture plane as 0.39 g which is underlain by alluvium. Also Sab Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3 g. PGA is chosen for the analysis as 0.25 g same as the value chosen by the authors of the source document. Also site response analyses results supported this conclusion. SPT performed after the earthquake.

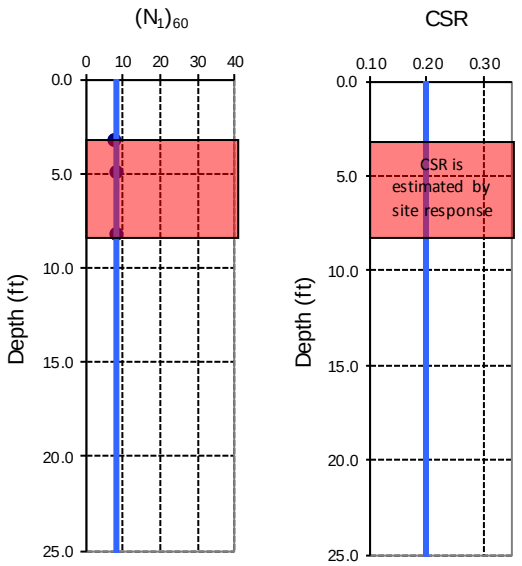
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.100 ± 0.050		
Data Class	A			% Fines	35.0 ± 2.0	35.0	
Critical Depth Range	3.3 - 8.2	5.9		% PI			
Depth to GWT (ft)	3.3	3.3					
σ_v (psf)	656.2 ± 99.2	668.3		N	5.8 ± 0.2	6.7	
σ_v' (psf)	502.6 ± 51.3	522.1		C_R	0.72	0.80	
a_{max} (g)	0.250 ± 0.038	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.005	0.99		C_B	1.00	1.00	
CSR	0.200 ± 0.034	0.233		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	2.00	1.70	
MSF		1.16		$(N_1)_{60}$	8.3 ± 0.3	9.1	
CSRN		0.183					



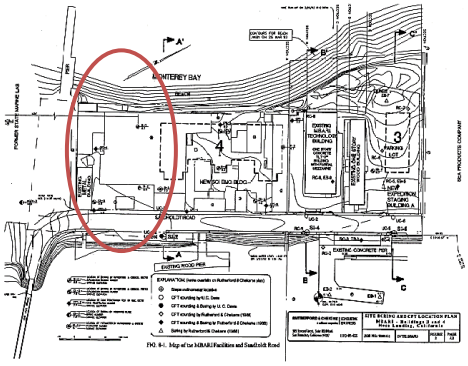
γ_m=110±3 pcf

γ_s=120±3 pcf



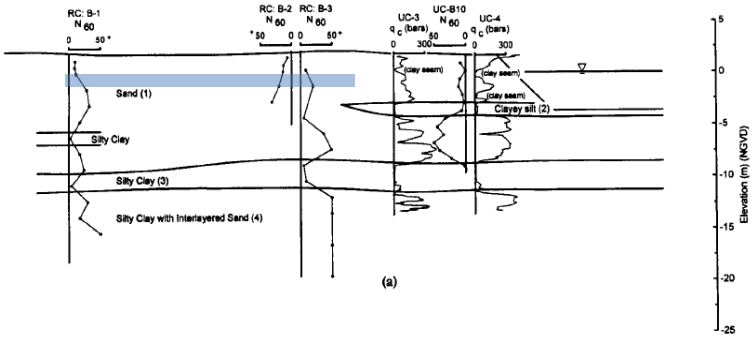
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
1	3.3		6	2.00	0.66	***7.9***
1.5	4.9		6	2.00	0.70	***8.4***
2.5	8.2		6	1.81	0.77	***8.4***
					Mean:	8.3
					St. Dev.	0.3

Case number:	141
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	General Fish
References:	Boulanger et al. (1995), Boulanger et al. (1997)
Nature of Failure:	"The Site of General Fish Company experienced minor lateral movements. Ground cracks and some cracks in the buildings (up to 8 cm) were observed. A significant ground crack was observed at the location of EB-5 borehole. Lateral movement of the site was tottally estimated as 5-7.5 cm." (Barminski 1993)
Comments:	It is stated that the site is somewhere near to existing metal building. In original document it is stated that "the General Fish Company building were constructed of wood but the details are not known". Boulanger et.al. (1995) The site consists of 8-12 m of poorly graded sand (FC=5%; 1-8%, D_{50}=0.20-2.1 mm) with some interlayers. For SI-2 inclinometer D_{50}=0.40-0.70 mm. Sand layer is underlained by clay layer of low plasticity (ML and CL). The magnitude of the Loma Prieta is given by USGS equal to 6.9. The acceleration value is recommended by Boulanger (1995) 0.20-0.30 g. In Cetin 2014 it is taken as 0.25 g. However in Boulanger 2010 study it is taken as 0.28 g. In Boulanger et.al (1997) it is stated that "a peak horizantal acceleration of about 0.25 g is a reasonable estimate for evaluation the effects of soil behavior during the 1989 Loma Prieta earthquake at Moss Landing .In Cetin 2014 a_{max} is taken as 0.25 g. "The SPT were performed by Rutherford and Chekene (1988,1993) by Taber Consultans using a Concore rig. A fish tail bit was used for drilling. SPT testes were performed by using a safety hammer, a cathead and rope system with 2 wraps of the rope around the cathead and a 5-cm-outside diameter split spoon sampler with room for liners. A few samples were obtained using a California split spoon sampler (6.3-cm outside-diameter). Energy measurements for this rig and the procedures while drilling at shallow depth in soft clay on another profect indicated an average energy ratio of 65%.Given the differences in energy ratios that may arise from different operators and soil conditions it seems reasonable to conclude that the blow counts reported by R&C (1988,1993) can be essentially taken as N_{60} values." Boulanger et.al. (1995). In Boulanger et.al (1995) borehole x-axis was named as $N_{1,60}$, In Boulanger et.al (1997) x-axis named as N_{60}. In this study these values are taken as N_{60} values. GWT is taken as 1.7 m. There exist ambiguity in Boulanger et.al (1995) 1. FC data is missing. The data is given in the output section on a graph however since the ground level information is missing it is unclear in which depth these fines content can be assigned. In the text in Boulanger et.al (1995) it is stated that "The site consists of 8-12 m of poorly graded sand (FC=5%; 1-8%, D50=0.20-2.1 mm) with some interlayers. In this study FC=5% is taken for all depths. Sea depth is 3 m. B1 is located 28 m away from sea, B2 is located 69 m away from sea and B3 is located 71 m away from sea. Thus, the site is a free field.



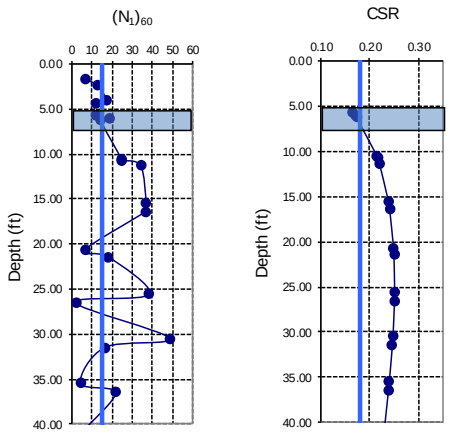
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.550 ± 0.050	-	
Data Class	B	-		% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	5.0 - 8.3	8.2		% PI	-	-	
Depth to GWT (ft)	5.5	4.6					
σ_v (psf)	749.0 ± 70.8	939.8		N	11.5 ± 2.3	16.9	
σ_v' (psf)	676.2 ± 38.6	731.0		C_R	0.74	0.80	
a_{max} (g)	0.250 ± 0.075	0.280		C_S	1.00	1.00	
r_d	0.99 ± 0.007	0.980		C_B	1.00	1.00	
CSR	0.179 ± 0.054	0.232		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.77	1.58	
MSF		1.16		$(N_1)_{60}$	15.1 ± 3.0	21.4	
CSRN		0.18					



γ_m=110±3 pcf

γ_s=125±3 pcf

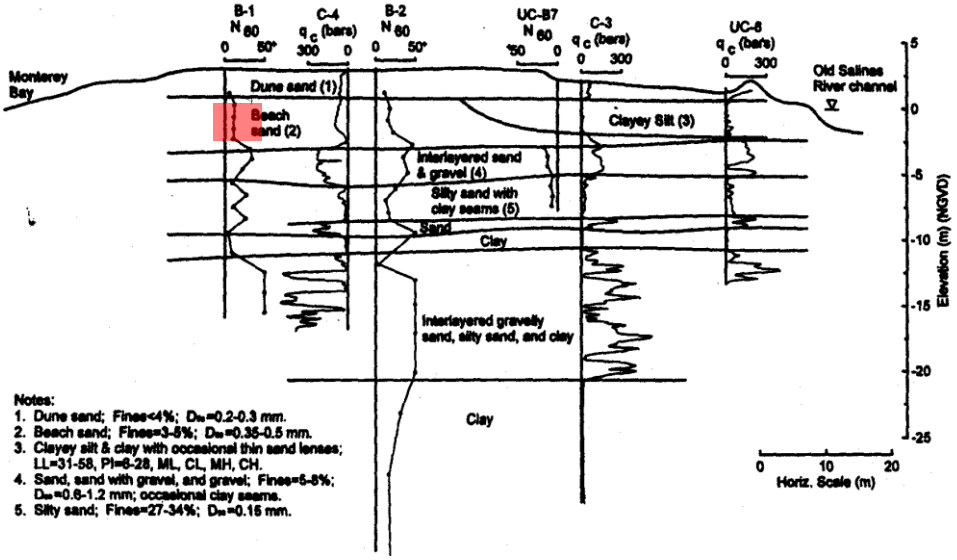


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	Borehole	Soil Type
0.5	1.64	NA	6	2.00	0.60	7.2	5.0	**B2**	Sand(1)
0.7	2.33	NA	10	2.00	0.63	12.5	5.0	**B1**	Sand(1)
1.2	4.04	NA	13	2.00	0.68	***17.7***	***5.0***	***B2***	Sand(1)
1.3	4.33	NA	9	2.00	0.69	***12.4***	***5.0***	***B1***	Sand(1)
1.7	5.71	0.165	9	1.85	0.72	***12.0***	***5.0***	***B3***	Sand(1)
1.8	6.04	0.170	14	1.82	0.73	***18.6***	***5.0***	***B2***	Sand(1)
1.9	6.20	0.173	11	1.81	0.73	***14.6***	***5.0***	***B1***	Sand(1)
3.2	10.50	0.215	20	1.52	0.81	24.7	5.0	**B2**	Sand(1)
3.3	10.70	0.216	20	1.51	0.82	24.6	5.0	**B3**	Sand(1)
3.4	11.29	0.220	28	1.48	0.83	34.2	5.0	**B1**	Sand(1)
4.7	15.49	0.238	32	1.31	0.88	37.0	5.0	**B2**	Sand(1)
5.0	16.37	0.241	32	1.28	0.89	36.6	5.0	**B1**	Sand(1)
6.3	20.70	0.249	6	1.17	0.93	6.5	5.0	**B3**	Sand(1)
6.5	21.42	0.250	17	1.15	0.94	18.4	5.0	**B1**	Sand(1)
7.8	25.59	0.251	37	1.07	0.97	38.4	5.0	**B3**	Sand(1)
8.1	26.51	0.251	2	1.05	0.98	2.1	5.0	**B1**	Silty Clay
9.3	30.48	0.248	49	0.99	1.00	48.4	5.0	**B3**	Sand(1)
9.6	31.50	0.247	17	0.97	1.00	16.6	5.0	**B1**	Sand(1)
10.8	35.50	0.240	5	0.92	1.00	4.6	5.0	**B3**	Silty Clay (3)
11.1	36.48	0.239	24	0.91	1.00	21.9	5.0	**B1**	Sand(1)
12.3	40.35	0.231	9	0.87	1.00	7.8	5.0	**B3**	Silty Clay (3)
12.6	41.47	0.229	4	0.86	1.00	3.4	5.0	**B1**	Silty Clay (3)
13.9	45.60	0.221	50	0.82	1.00	41.2	5.0	**B3**	Silty Clay with interlayered Sand(4)
14.2	46.59	0.219	30	0.82	1.00	24.5	5.0	**B1**	Silty Clay with interlayered Sand(4)
15.8	51.67	0.212	18	0.78	1.00	14.0	5.0	**B1**	Silty Clay with interlayered Sand(4)
17.3	56.63	0.207	50	0.75	1.00	37.3	5.0	**B1**	Silty Clay with interlayered Sand(4)
					Mean:	15.1	5.0		
					St. Dev.	3.0	2.0		

Case number:	142
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Marine Laboratory B-1
References:	Boulanger et al. (1996), "Liquefaction at Moass Landing During Loma Prieta Earthquake"
Nature of Failure:	Liquefaction-induced lateral spreading and differential settlement. Buildings spread 1-1.3 m at the ground surface. Fine grained clayey silt boil in the voleyball court near the southeaast of the site.
Comments:	Located on the Moss Landing spit, just south of the timber bridge. The facilities consisted of one- and two-story, woodframe structures. and oil and gas storage tanks. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m The clayey silt stratum beneath the east side is 1.5-3 m thick, and is interbedded between dune and beach sands. PGA was recorded in a four-story building 8 km from rupture plane as 0.39 g (alluvial site). Also San Juan Bautista seismographic station, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3g Based on site response analyses results, a PGA value of 0.24 g was adopted for these studies SPT performed after the earthquake

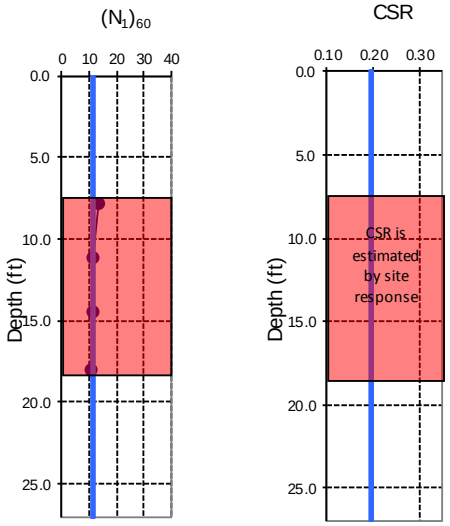
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.800 ± 0.050		
Data Class	A			% Fines	3.0 ± 2.0	3.0	
Critical Depth Range	7.9 - 18.0	15.1		% PI			
Depth to GWT (ft)	7.9	7.9					
σ_v (psf)	1501.8 ± 213.8	1817.0		N	10.3 ± 1.0	11.0	
σ_v' (psf)	1184.5 ± 110.7	1357.6		C_R	0.85	0.95	
a_{max} (g)	0.240 ± 0.036	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.011	0.95		C_B	1.00	1.00	
CSR	0.196 ± 0.031	0.230		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.34	1.25	
MSF		1.16		$(N_1)_{60}$	11.7 ± 1.1	13.1	
CSRN		0.189					



γ_m=110±3 pcf

γ_s=125±3 pcf

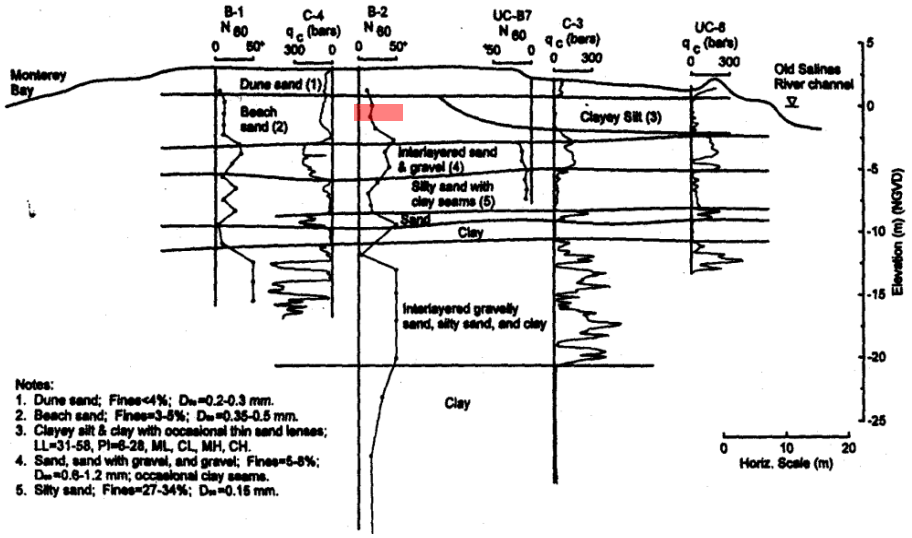


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.4	7.9		11	1.56	0.77	***13.2***
3.4	11.2		10	1.41	0.82	***11.6***
4.4	14.4		10	1.29	0.87	***11.2***
5.5	18.0		10	1.19	0.91	***10.8***
					Mean:	11.7
					St. Dev.	1.1

Case number:	143
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Marine Laboratory B-2
References:	Boulanger et al. (1996), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	Liquefaction-induced lateral spreading and differential settlement. Buildings spread 1-1.3 m at the ground surface. Fine grained clayey silt boil in the volleyball court near the southeaast of the site.
Comments:	Located on the Moss Landing spit, just south of the timber bridge. The facilities consisted of one- and two-story, woodframe structures. and oil and gas storage tanks. The Moss Landing spit is underlain by littoral soils comprising eolian, fluvial, estuarine, and beach deposits of gravel, sand, silt, and clay deposited in a shoreline environment within the zone of tidal fluctuation. Pleistocene and miocene deposits extend to a depth of ~2000 m The clayey silt stratum beneath the east side is 1.5-3 m thick, and is interbedded between dune and beach sands. PGA is recorded in a four-story building 8km from rupture plane as 0.39 g which is underlain by alluvium. Also San Juan Bautista seismogram, ~20 km from the rupture plane recorded 0.15 g (stiff alluvium). The Moss Landing area is located ~21 km from the rupture plane. Due to deep alluvium which is soft near the surface, PGA at the site is predicted between 0.2-0.3g Based on site response analyses results, a PGA value of 0.26 g is adopted for this study (a PGA of 0.25 is suggested in the source document). UC-B1 and UC-B2 are ~100 ft apart. SPT performed after the earthquake.

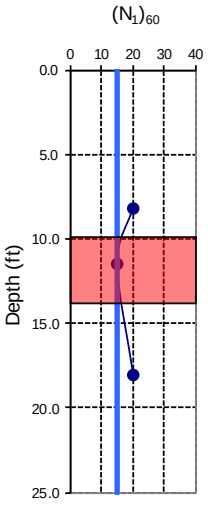
Summary of Data

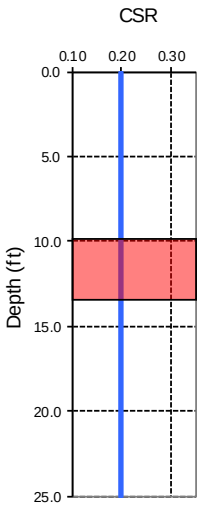
	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.500 ± 0.050		
Data Class	B			% Fines	3.0 ± 2.0	3.0	
Critical Depth Range	10.0 - 13.0	11.5		% PI			
Depth to GWT (ft)	8.2	8.2					
σ_v (psf)	1314.5 ± 68.0	1357.6		N	13.0 ± 1.7	13.0	
σ_v' (psf)	1108.7 ± 43.4	1148.7		C_R	0.83	0.85	
a_{max} (g)	0.260 ± 0.039	0.28		C_S	1.00	1.00	
r_d	0.99 ± 0.008	0.97		C_B	1.00	1.00	
CSR	0.198 ± 0.030	0.207		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.38	1.35	
MSF		1.16		$(N_1)_{60}$	14.9 ± 2.0	14.9	
CSRN		0.167					



γ_m=110±3 pcf

γ_s=125±3 pcf





CSR is estimated by site response

Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2		17	1.53	0.77	20.2
3.5	11.5		13	1.38	0.83	***14.9***
5.5	18.0		19	1.18	0.91	20.4
					Mean:	14.9
					St. Dev.	2.0

Case number:	144
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Marine Laboratory F1-F7
References:	Boulanger et al. (1995), "Liquefaction at Moss Landing During Loma Prieta Earthquake"
Nature of Failure:	"The MLML facilities damaged beyond repair by liquefaction-induced lateral spreading and differential settlement of the site. The survey data shows that the building spread 1 to 1.3 m at the ground surface. Severe cracking occured in the foundation and slab." Boulanger et.al. (1995)
Comments:	MLML consist of two building and a west wing surrounding a courtuyard. The northern building is one and two storey wood frame structure founded on spread footings. The southern one and the west wing were one storey wood frame and shear wall structure with concrete slab-on-grade floors. The eastern part of southern building was founded on 0.4 m diameter cast in place concrete piers up to 5.5 m deep. Field investigations were perfomed by WCC (1990) and UC Davis. Additional data were obtained from Fugro (1980). B1,B2,B3 belongs to Woodward-Cylde Consultants (1990) There exist two additional cases: Marine Lab B1 and B2. SPT were performed using a CME automatic trip hammer and a safety hammer with a cathead and rope system. The sampler was a standard 5 cm outside diameter split spoon with room for liners. Sample taken with the liners omitted. Energy hammer ratios were estimated as 63% and 75% for the safety hammer and automatic trip hammer systems. Fugro's boreholes located 80 m away from the site. "SPT hammers were of two types: a rope around the cathead "donut" hammer provided by drillers (total weigth of 225 ponds)i and a "free-fall", mechanical trip hammer (total weigth of 210 pounds) manufactured by Pilcon Engineering. Sampling spoon were newly constructed following ASTM-guidlines with the exception that space for liners was provided, but liners were not used. " (Fugro Inc.) In Boulanger et. al. 1995 UCB-6-12 and F1-6 were used.However UCB 6-12 boreholes were unavailable. From Boulanger et. al. 1995 it can be inferred that these boreholes are CPT logs. Therefore F1-7 logs are used in Marine Laboratory F1-F7 site.

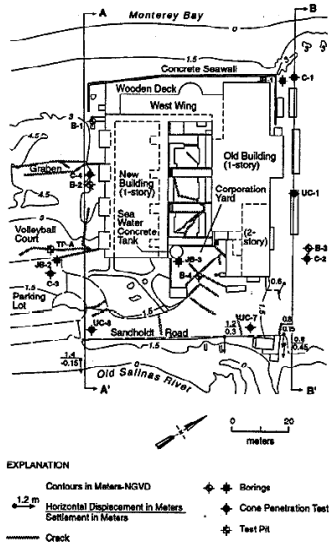
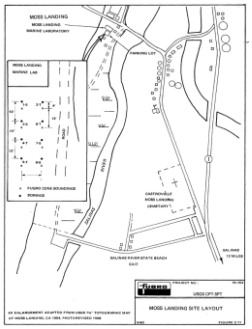
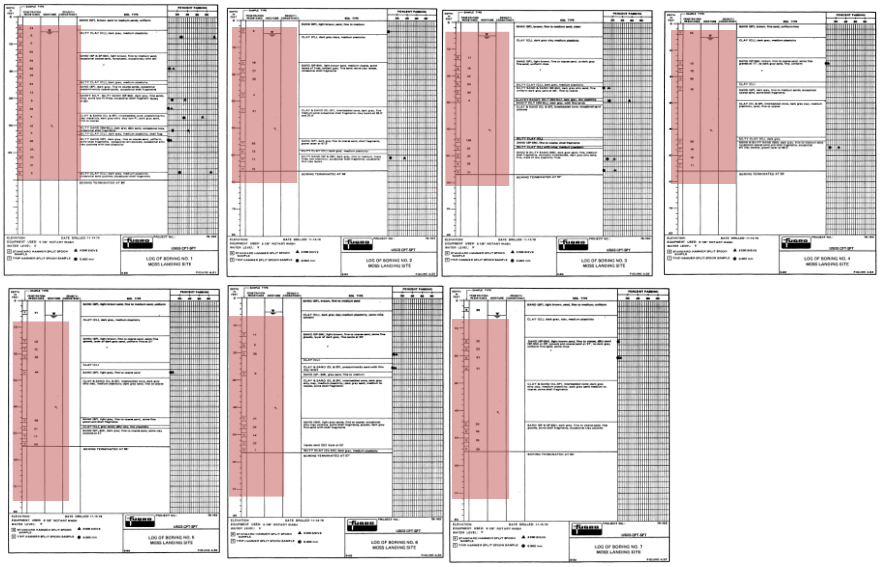


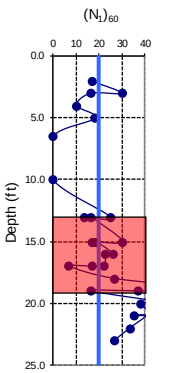
FIG. 9-1. Map of the Moss Landing Marine Laboratory (MLML) Site

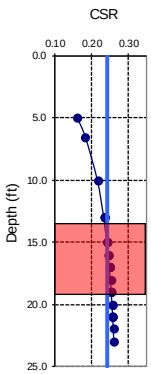
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.425 ± 0.075	-	
Data Class	B	-		% Fines	3.0 ± 1.4	3.0	
Critical Depth Range	11.5 - 18.5	17.4		% PI	-	-	
Depth to GWT (ft)	5.0	4.9					
σ_v (psf)	1850.0 ± 149.6	2130.3		N	13.8 ± 4.7	12.0	
σ_v' (psf)	1226.0 ± 80.8	1336.7		C_R	0.87	0.95	
a_{max} (g)	0.250 ± 0.075	0.280		C_S	1.00	1.00	
r_d	0.99 ± 0.007	0.940		C_B	1.00	1.00	
CSR	0.243 ± 0.073	0.270		C_E	1.25	1.25	
Equivalent Magnitude	6.93	6.9		C_N	1.31	1.23	
MSF		1.16		$(N_1)_{60}$	19.8 ± 6.7	17.6	
CSRN		0.22					



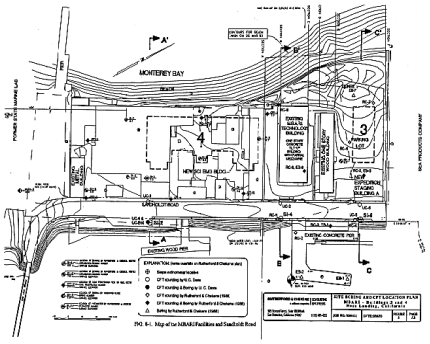






Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	Borehole	Soil Type	Hammer Type
0.61	2.0	NA	13	2.00	0.61	*16.7*	-	S-1	sand	S
0.91	3.0	NA	22	2.00	0.65	*30.0*	-	S-7	sand	S
0.91	3.0	NA	10	2.00	0.65	*16.2*	-	T-4	sand	T
1.22	4.0	NA	6	2.00	0.68	*10.2*	2	T-2	sand	T
1.52	5.0	0.162	11	1.88	0.71	*18.2*	-	T-5	sand	T
1.98	6.5	0.184	0	1.75	0.74	*0.0*	-	S-1	silty clay	S
3.05	10.0	0.217	0	1.52	0.81	*0.0*	-	S-1	silty clay	S
3.96	13.0	0.234	20	1.39	0.85	*24.7*	12	S-1	silty clay	S
3.96	13.0	0.234	11	1.39	0.85	***13.6***	-	S-3	sand	S
3.96	13.0	0.234	11	1.39	0.85	***16.2***	-	T-6	sand	T
4.57	15.0	0.243	25	1.31	0.87	***30.2***	***2.0***	S-7	sand	S
4.57	15.0	0.243	12	1.31	0.87	***17.2***	***4.0***	T-4	sand	T
4.57	15.0	0.243	13	1.31	0.87	***18.7***	-	T-5	sand	T
4.88	16.0	0.246	22	1.28	0.89	***26.2***	-	S-1	sand	S
4.88	16.0	0.246	16	1.28	0.89	***22.7***	-	T-2	sand	T
5.18	17.0	0.249	19	1.25	0.90	***22.4***	-	S-3	sand	S
5.18	17.0	0.249	12	1.25	0.90	***16.8***	-	T-4	sand	T
5.18	17.0	0.249	5	1.25	0.90	***7.0***	-	T-6	sand	T
5.49	18.0	0.252	23	1.22	0.91	***26.8***	-	S-7	sand	S
5.79	19.0	0.255	65	1.20	0.92	*75.0*	-	S-1	sand	S
5.79	19.0	0.255	27	1.20	0.92	*37.1*	-	T-2	sand	T
5.79	19.0	0.255	12	1.20	0.92	*16.5*	-	T-5	sand	T
6.10	20.0	0.257	44	1.17	0.93	*50.2*	-	S-3	sand	S
6.10	20.0	0.257	28	1.17	0.93	*38.1*	8	T-6	sand	T
6.40	21.0	0.259	42	1.15	0.94	*47.5*	-	S-1	sand	S
6.40	21.0	0.259	57	1.15	0.94	*64.4*	3	S-7	sand	S
6.40	21.0	0.259	26	1.15	0.94	*35.0*	-	T-4	sand	T
6.40	21.0	0.259	30	1.15	0.94	*40.4*	-	T-5	sand	T
6.71	22.0	0.260	25	1.13	0.95	*33.3*	-	T-2	sand	T
7.01	23.0	0.262	24	1.11	0.95	*26.6*	-	S-3	silty clay	S
					Mean:	19.8	3.0			
					St. Dev.	6.7	1.4			

Case number:	145
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	MBARI N0.4-B4B5EB2EB3
References:	Boulanger et al. (1995), "Liquefaction at Moass Landing During Loma Prieta Earthquake"
Nature of Failure:	"Ground cracks were mapped at a few spots toward the eastern side of the current site of Building 4 and the lot on its south side" Boulanger et. al. 1997
Comments:	The MBARI facilities include three main buildings and a pier located along Sandholt Road on the Moss Landing spit. Construction of No.4 building started in 1994. B4, B5, EB2, EB3 is used for the liquefaction analysis by Boulanger et. al. 1995. However in Cetin 2014 B5 and EB3 was used due to fact that B4 borehole data was unavailable and from EB-3 borehole data consists of silty clay layers. The site consists of 8-12 m of poorly graded sand (FC=5%; 1-8%, D50=0.20-2.1 mm) with some interlayers. For SI-2 inlinometer D50=0.40-0.70 mm. Sand layer is underlained by clay layer of low plasticity (ML and CL). The magnitude of the Loma Prieta is given by USGS equal to 6.9. The acceleration value is recommended by Boulanger (1995) 0.20-0.30 g. In Cetin 2014 it is taken as 0.25 g. However in Boulanger 2010 study it is taken as 0.28 g. "The SPT were performed by Rutherford and Chekene (1988,1993) by Taber Consultans using a Concore rig. A fish tail bit was used for drilling. SPT testes were performed by using a safety hammer, a cathead and rope system with 2 wraps of the rope around the cathead and a 5-cm-outside diameter split spoon sampler with room for liners. A few samples were obtained using a California split spoon sampler (6.3-cm outside-diameter). Energy measurements for this rig and the procedures while drilling at shallow depth in soft clay on another profectindicated an average anergy ratio of 65%.Given the differences in energy ratios that may arise from different opertaors and soil conditionsi it seems reasonable to conclude that the blow counts reported by R&C (1988,1993) can be essentially taken as N60 values." Boulanger et.al. (1995) From B5 water table is located at 2.3 m depth and from EB3 it is located at 1.6 m. An average value of 1.95 m is assigned as ground water table for the site. FC data is missing. The FC data is given in the output section in Boulanger 1995 on a graph however since the ground level information is missing it is unclear in which depth these fines content can be assigned. In the text in Boulanger et.al (1995) it is stated that "The site consists of 8-12 m of poorly graded sand (FC=5%; 1-8%, D50=0.20-2.1 mm) with some interlayers. In this study FC=5% is taken for all depths. Sea depth is 3 m. B4 is located 59 m away from sea, EB3 is located 33 m away from sea and EB2 is located 84 m away from sea. Thus, the site is a free field.



Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.550 ± 0.150	-	
Data Class	B	-		% Fines	5.0 ± 2.0	5.00	
Critical Depth Range	7.9 - 25.3	11.15		% PI	-	-	
Depth to GWT (ft)	6.4	6.23					
s_v (psf)	2039.0 ± 364.1	1294.9		N	23.6 ± 5.5	18.00	
s_v' (psf)	1405.3 ± 206.5	1002.5		C_R	0.89	0.85	
a_{max} (g)	0.250 ± 0.075	0.28		C_S	1.00	1.00	
r_d	0.98 ± 0.007	0.97		C_B	1.00	1.00	
CSR	0.230 ± 0.072	0.23		C_E	1.00	1.00	
Equivalent Magnitude	6.93	6.93		C_N	1.23	1.38	
MSF		1.16		$(N_1)_{60}$	25.8 ± 6.0	21.20	
CSRN		0.18					

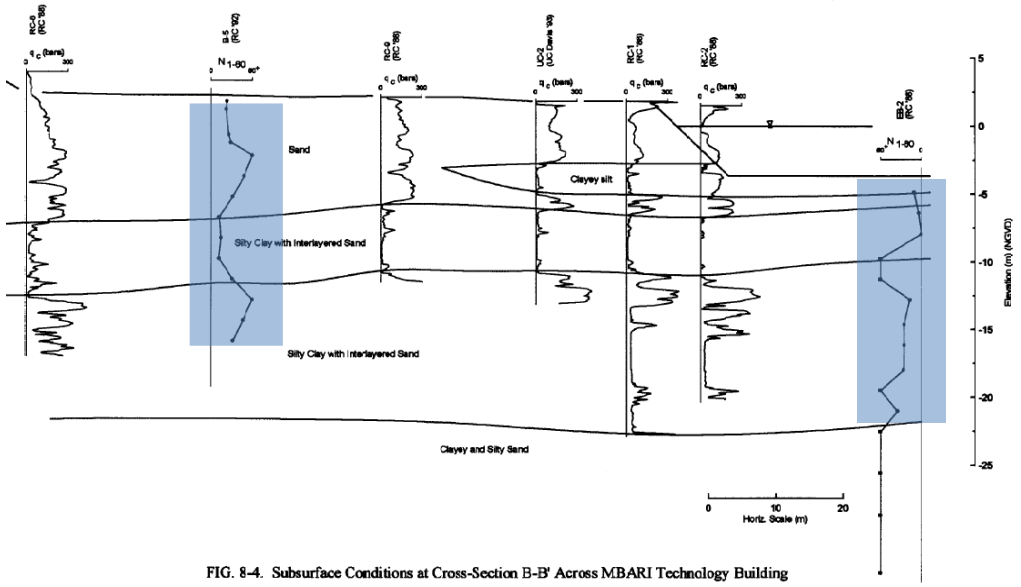


FIG. 8-4. Subsurface Conditions at Cross-Section B-B' Across MBARI Technology Building

γ_m=120±3 pcf

γ_s=125±3 pcf

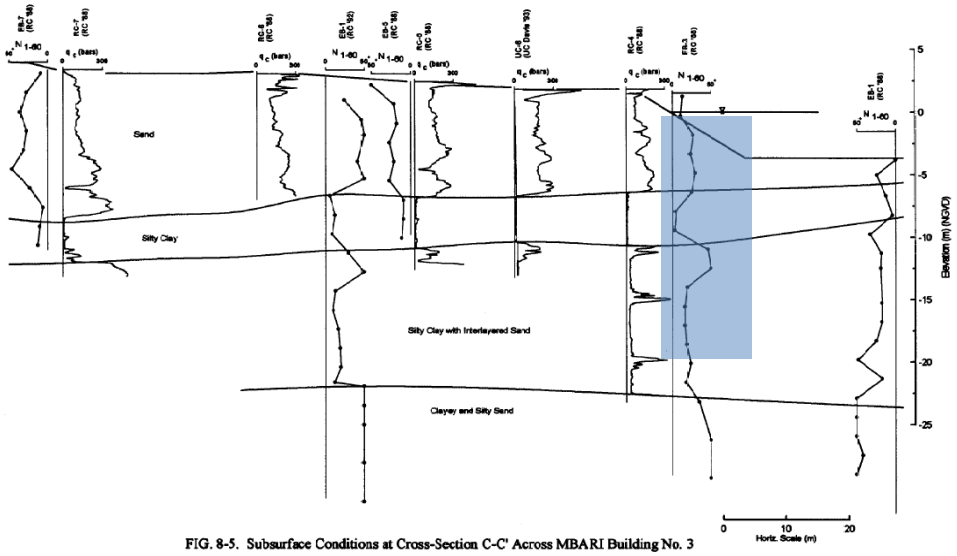
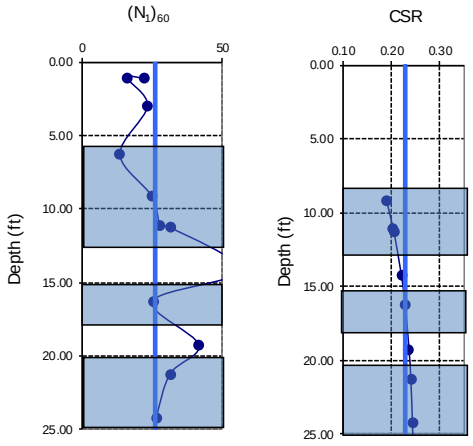


FIG. 8-5. Subsurface Conditions at Cross-Section C-C' Across MBARI Building No. 3

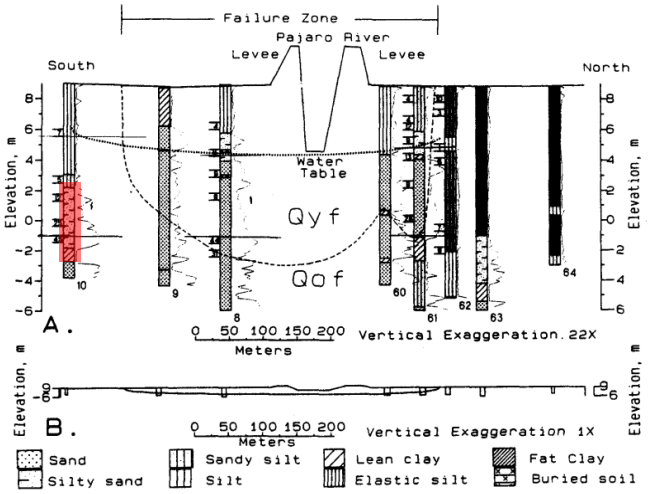


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	Borehole	Soil Type
0.34	1.12	NA	19	2.00	0.58	*21.9*	*5.0*	B5	sand
0.34	1.12	NA	14	2.00	0.58	*16.2*	*5.0*	EB3	sand
0.92	3.02	NA	18	2.00	0.65	*23.4*	*5.0*	B5	sand
1.92	6.30	NA	11	1.67	0.74	***13.5***	***5.0***	EB3	sand
2.79	9.15	0.190	21	1.50	0.79	***24.9***	***5.0***	B5	sand
3.39	11.12	0.205	24	1.41	0.82	***27.8***	***5.0***	B5	sand
3.43	11.25	0.206	27	1.40	0.82	***31.3***	***5.0***	EB3	sand
4.33	14.21	0.221	50	1.30	0.86	*56.1*	*5.0*	B5	sand
4.96	16.27	0.229	23	1.24	0.89	***25.3***	***5.0***	EB3	sand
5.87	19.26	0.237	39	1.16	0.92	*41.6*	*5.0*	B5	sand
6.47	21.23	0.241	30	1.12	0.94	***31.4***	***5.0***	EB3	sand
7.38	24.21	0.244	26	1.06	0.96	***26.6***	***5.0***	B5	sand
7.99	26.21	0.245	25	1.03	0.98	*25.1*	*5.0*	EB3	silty clay
8.9	29.20	0.245	9	0.98	1.00	*8.8*	*5.0*	B5	sand
9.54	31.30	0.243	5	0.95	1.00	*4.8*	*5.0*	EB3	silty clay
10.38	34.06	0.241	12	0.92	1.00	*11.0*	*5.0*	B5	silty clay2
11.06	36.29	0.238	3	0.90	1.00	*2.7*	*5.0*	EB3	silty clay
11.93	39.14	0.233	9	0.87	1.00	*7.8*	*5.0*	B5	silty clay2
12.58	41.27	0.230	46	0.85	1.00	*38.9*	*5.0*	EB3	silty clay2
13.43	44.06	0.225	25	0.82	1.00	*20.6*	*5.0*	B5	silty clay2
14.08	46.19	0.222	49	0.81	1.00	*39.5*	*5.0*	EB3	silty clay2
14.96	49.08	0.217	50	0.78	1.00	*39.2*	*5.0*	B5	silty clay3
15.58	51.12	0.215	20	0.77	1.00	*15.4*	*5.0*	EB3	silty clay2
16.49	54.10	0.211	38	0.75	1.00	*28.5*	*5.0*	B5	silty clay3
17.16	56.30	0.209	16	0.74	1.00	*11.8*	*5.0*	EB3	silty clay2
17.98	58.99	0.207	26	0.72	1.00	*18.8*	*5.0*	B5	silty clay3
18.66	61.22	0.206	17	0.71	1.00	*12.1*	*5.0*	EB3	silty clay2
20.17	66.17	0.204	19	0.68	1.00	*13.0*	*5.0*	EB3	silty clay2
21.68	71.13	0.204	23	0.66	1.00	*15.2*	*5.0*	EB3	silty clay2
23.21	76.15	0.203	17	0.64	1.00	*10.9*	*5.0*	EB3	silty clay2
24.77	81.27	0.203	35	0.62	1.00	*21.8*	*5.0*	EB3	silty clay2
					Mean:	25.8	5.0		
					St. Dev.	6.0	2.0		

Case number:	146
Earthquake:	Loma Prieta
Magnitude:	6.93 (Mw) Pacific Earthquake Engineering Research Center: NGA Database
Location:	Miller Farm Site
References:	Holzer et al. (1994), "Observed and predicted ground deformation-Miller Farm lateral spread, Watsonville, California."
Nature of Failure:	Lateral spread near the Pajaro River. South of the river ~0.2 m high head scarp which could be followed for 1.7 km. Maximum horizontal displacement of ~30 cm just west of the study area. Sand boils along the area affected by lateral spreading. Eyewitnesses reported that sediment-laden water flowed gently from vents on the lateral spread during and until 20 minuntes after the earthquake.
Comments:	Located at the south side of the Pajara River at Clint Miller Farms near Watsonville, CA. Composed of younger and older holocene fluvial deposits. The upper 4-6 m of the younger flood-plain deposits is silty and contains only a few thin sand layers. In the depth interval from 4.6-9.1, sand percentages range from 67-100. This layer includes most of the liquefied layer. Average fines content of the sandy layers if 15%, ranging from 22 % at Miller Farm to 8% at Farris Farm. The depth to the water table was 4.6 m 10 days after the main shock. PGA is recorded in a four-story building 1km west of Miller Farm as 0.39 g. In Open File Report 95-663, Holzer et al., the PGA was given as 0.53 g. Watsonville region was shaken at MMI scale of 8.(~ 0.25-0.50 g ?) The earthquake specific attenuation-relationship suggests for alluvium sites at 4.6 miles (closest distance to the fault) ~0.50-0.65 g. A PGA value of 0.36 g was recorded at Watsonville. Based on site response analyses results, a PGA value of 0.42 is adopted. Strong ground motion lasted ~ 6 seconds. SPT performed after the earthquake.

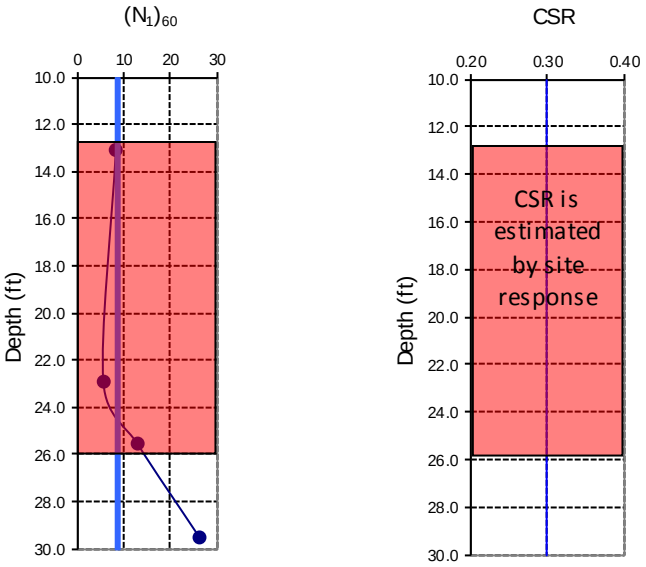
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes			D_{50}	0.160 ± 0.050		
Data Class	A			% Fines	22.0 ± 2.0		
Critical Depth Range	13.1 - 26.2			% PI			
Depth to GWT (ft)	13.1						
σ_v (psf)	2263.8 ± 277.0			N	8.0 ± 3.4		
σ_v' (psf)	1854.3 ± 144.8			C_R	0.92		
a_{max} (g)	0.420 ± 0.063			C_S	1.00		
r_d	0.90 ± 0.017			C_B	1.00		
CSR	0.300 ± 0.048			C_E	1.13		
Equivalent Magnitude	6.9			C_N	1.07		
MSF				$(N_1)_{60}$	8.9 ± 3.8		
CSRN							



γ_m=110±3 pcf

γ_s=125±3 pcf

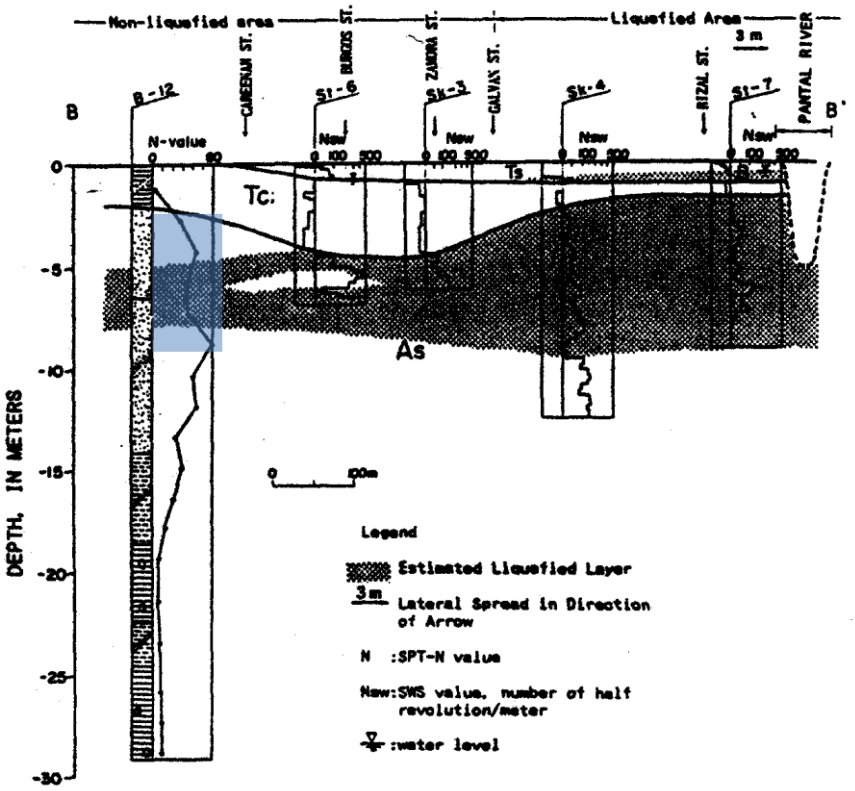


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
4	13.1		7	1.21	0.85	***8.2***	
7	23.0		5	1.01	0.95	***5.5***	HOLZER
7.8	25.6		12	0.98	0.97	***12.9***	HOLZER
9	29.5		25	0.93	1.00	26.2	HOLZER
					Mean:	8.9	
					St. Dev.	3.8	

Case number:	147
Earthquake:	1990 Luzon
Magnitude:	7.7 (M_w)
Location:	Cereenan St. B-12
References:	Wakamatsu (1992)
Nature of Failure:	No surface evidence of liquefaction
Comments:	The Luzon, Philippines Earthquake of July 16, 1990 occurred as result of relative movement between the Eurasian and Philippine Sea plate. A fine soil layer with SPT-values between 10 and 40, exists near the ground surface or several meters below it. Under this layer, there exists a sand layer containing silt and clayey layer. The ground surface is composed of sandy soil, which is thought to be fill, as well as natural levee and or point-bar deposits, or clayey soil from back marsh or swamp deposits. The thickness of the liquefied layer is on the order of 5-10 m. In addition the unliquefied layer above it is thin, being less than 3 m deep. The SPT energy used in Philippines is about half of the Japanese energy values. PGA was estimated as 0.2 g in the source document. PGA contour maps suggests a value in between 0.2-0.3g.

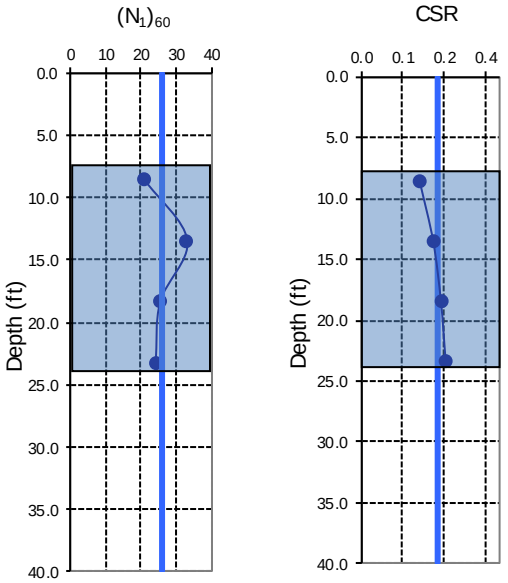
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.200 ± 0.050		
Data Class	B			% Fines	19.0 ± 2.0	19.0	
Critical Depth Range	7.9 - 24.6	16.4		% PI	5		
Depth to GWT (ft)	7.5 ± 0.3	7.5					
σ_v (psf)	1916.8 ± 350.3	1963.2		N	34.8 ± 7.0	34.7	
σ_v' (psf)	1374.3 ± 178.5	1420.2		C_R	0.92	0.95	
a_{max} (g)	0.250 ± 0.075	0.25		C_S	1.00	1.00	
r_d	0.99 ± 0.069	0.97		C_B	1.00	1.00	
CSR	0.225 ± 0.070	0.218		C_E	0.65	0.65	
Equivalent Magnitude	7.7	7.7		C_N	1.24	1.16	
MSF		0.95		$(N_1)_{60}$	25.8 ± 5.2	24.9	
CSRN		0.213					



γ_m=110±3 pcf

γ_s=125±3 pcf

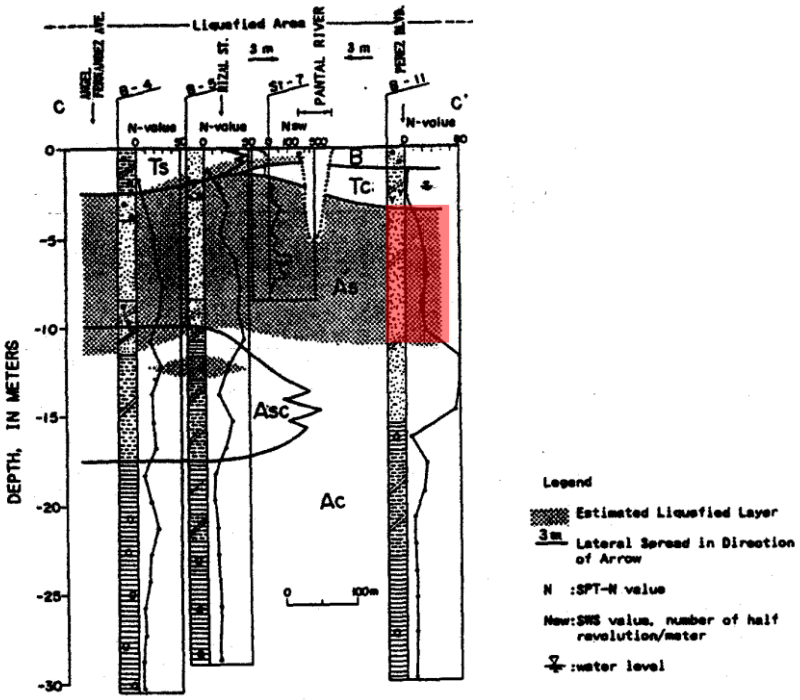


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.6	8.5	0.173	25.0	1.54	0.83	***20.7***
4.1	13.5	0.211	43.0	1.33	0.89	***33.1***
5.6	18.4	0.233	35.0	1.18	0.94	***25.3***
7.1	23.3	0.246	35.0	1.08	0.98	***24.1***
					Mean:	25.8
					St. Dev.	5.2

Case number:	148
Earthquake:	1990 Luzon
Magnitude:	7.7 (M_w)
Location:	Perez Blv. B-11
References:	Wakamatsu (1992)
Nature of Failure:	Cracks, lateral spread and tilting of bulidings
Comments:	The Luzon, Philippines Earthquake of July 16, 1990 occurred as result of relative movement between the Eurasian and Philippine Sea plate. A fine soil layer with SPT-values between 10 and 40, exists near the ground surface or several meters below it. Under this layer, there exists a sand layer containing silt and clayey layer. The ground surface is composed of sandy soil, which is thought to be fill, as well as natural levee and or point-bar deposits, or clayey soil from back marsh or swamp deposits. The thickness of the liquefied layer is on the order of 5-10 m. In addition the unliquefied layer above it is thin, being less than 3 m deep. The SPT energy used in Philippines is about half of the Japanese energy values. PGA was estimated as 0.2 g in the source document. PGA contour maps suggests a value in between 0.2-0.3g.

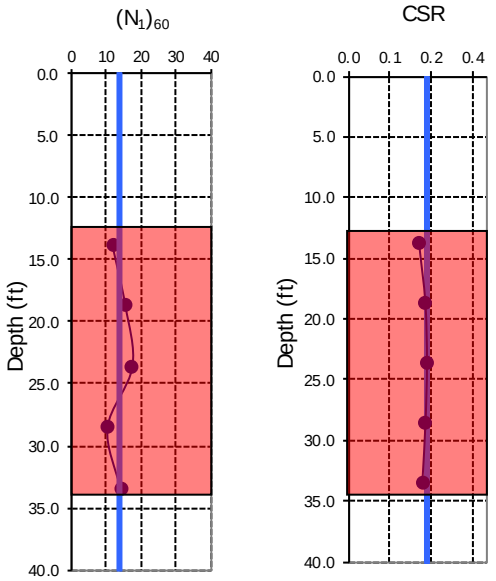
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.200 ± 0.050		
Data Class	B			% Fines	19.0 ± 2.0	19.0	
Critical Depth Range	13.1 - 34.4	23.6		% PI	5		
Depth to GWT (ft)	7.5 ± 0.3	7.5					
σ_v (psf)	2784.6 ± 447.6	2903.1		N	19.9 ± 4.0	19.9	
σ_v' (psf)	1771.2 ± 229.2	1879.7		C_R	0.98	0.95	
a_{max} (g)	0.250 ± 0.075	0.25		C_S	1.00	1.00	
r_d	0.89 ± 0.096	0.94		C_B	1.00	1.00	
CSR	0.227 ± 0.073	0.236		C_E	0.65	0.65	
Equivalent Magnitude	7.7	7.7		C_N	1.09	1.06	
MSF		0.95		$(N_1)_{60}$	13.9 ± 2.8	13.0	
CSRN		0.245					



γ_m=100±3 pcf

γ_s=125±3 pcf

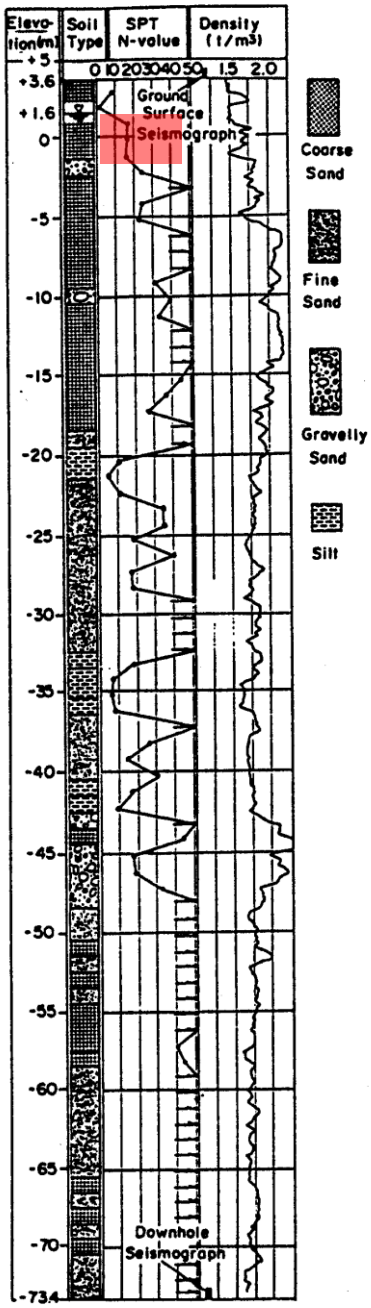


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4.2	13.8	0.209	15.0	1.36	0.89	***11.9***
5.7	18.7	0.223	21.0	1.21	0.94	***15.5***
7.2	23.6	0.227	25.0	1.10	0.98	***17.5***
8.7	28.5	0.223	16.0	1.01	1.00	***10.5***
10.2	33.5	0.216	23.0	0.94	1.00	***14.1***
					Mean:	13.9
					St. Dev.	2.8

Case number:	149
Earthquake:	1993 Kushiro-Oki
Magnitude:	7.8 (Mjma)
Location:	Kushiro Port Strong Motion Recording Station
References:	Iai et al (1994)
Nature of Failure:	Liquefaction of the backfill sand of quay walls; displacements of quay walls. Sudden change in the response of ground after 30 seconds.
Comments:	On January 15, 1993, an earthquake of magnitude 7.8 in JMA scale hit a northern island of Japan called Hokkaido. The epicenter was located 15 km south of Kushiro City. Kushiro Port was shaken with a peak horizontal acceleration of 0.47 and 0.34g. The earthquake caused liquefaction of untreated backfill sand at the port, resulting in damage to many of the quay walls. The extent of the damage was generally moderate; the displacements of the quay walls were less than about 5% of the water depth in front of the quay walls. In contrast, the quay walls having the backfill sand compacted as measures against liquefaction survived the earthquake without damage. Kushiro Port has been developed at the estuary of the Kushiro River. Most of the port area has been reclaimed from the sea by filling the sand dredged from the nearby sea. The natural ground along the former beach line is of sand dune origin being dense with SPT N values of about 50. The fill above the original ground gradually increases its thickness from none at the former beach line to about 10 m at the southern end of the piers in the West Port District. SPT was conducted before the earthquake.

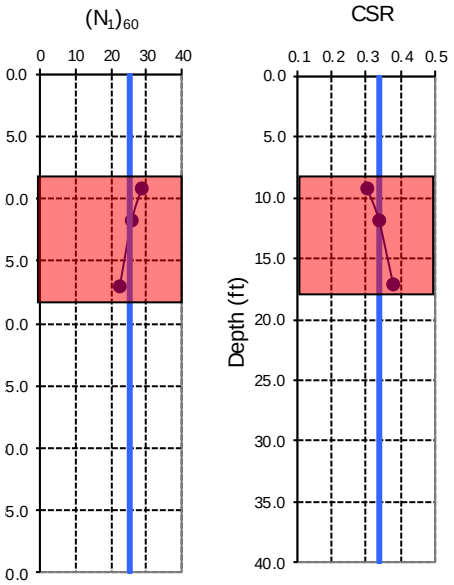
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.350 ± 0.050		
Data Class	A			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	5.2 - 18.4	12.5		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	1378.0 ± 274.6	1357.6		N	16.9 ± 2.0	17.4	
σ_v' (psf)	1050.4 ± 139.9	981.6		C_R	0.87	0.85	
a_{max} (g)	0.400 ± 0.060	0.47		C_S	1.00	1.00	
r_d	0.99 ± 0.053	0.98		C_B	1.00	1.00	
CSR	0.338 ± 0.059	0.410		C_E	1.22	1.30	
Equivalent Magnitude	7.6	7.6		C_N	1.42	1.35	
MSF		0.97		$(N_1)_{60}$	25.4 ± 3.0	25.9	
CSRN		0.383					



γ_m=110±3 pcf

γ_s=125±3 pcf

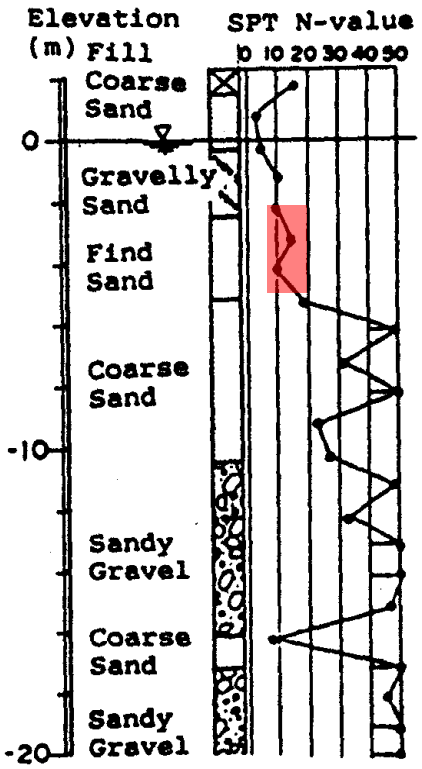


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.8	9.2	0.306	18.0	1.55	0.84	***28.3***
3.6	11.8	0.338	17.0	1.42	0.87	***25.6***
5.2	17.1	0.378	16.0	1.24	0.93	***22.4***
					Mean:	25.4
					St. Dev.	3.0

Case number:	150
Earthquake:	1993 Kushiro-Oki
Magnitude:	7.8 (Mjma)
Location:	Quay Wall Site A
References:	Iai et al (1994)
Nature of Failure:	Settlement and lateral movement of quay wall couple of meters.
Comments:	On January 15, 1993, an earthquake of magnitude 7.8 in JMA scale hit a northern island of Japan called Hokkaido. The epicenter was located 15 km south of Kushiro City. Kushiro Port was shaken with a peak horizontal acceleration of 0.47 and 0.34g. The earthquake caused liquefaction of untreated backfill sand at the port, resulting in damage to many of the quay walls. The extent of the damage was generally moderate; the displacements of the quay walls were less than about 5% of the water depth in front of the quay walls. In contrast, the quay walls having the backfill sand compacted as measures against liquefaction survived the earthquake without damage. Kushiro Port is developed at the estuary of the Kushiro River. Most of the port area has been reclaimed from the sea by filling the sand dredged from the nearby sea. The natural ground along the former beach line is of sand dune origin being dense with SPT N values of about 50. The fill above the original ground gradually increases its thickness from none at the former beach line to about 10 m at the southern end of the piers in the West Port District. SPT was conducted before the earthquake.

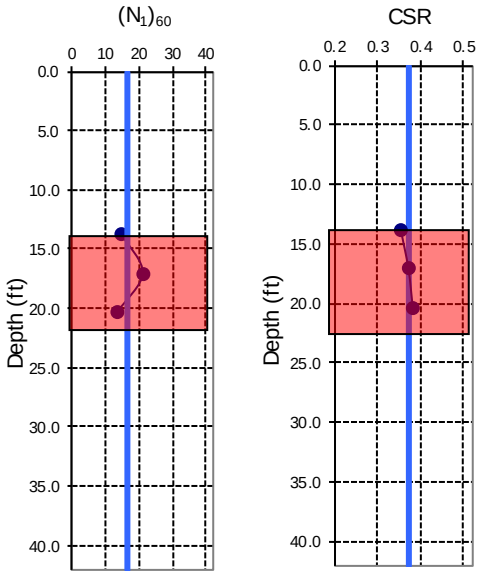
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.340 ± 0.050		
Data Class	A			% Fines	2.0 ± 2.0	2.0	
Critical Depth Range	13.1 - 21.3	17.1		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	2021.8 ± 175.1	2088.5		N	11.7 ± 2.9	11.7	
σ_v' (psf)	1356.5 ± 94.3	1420.2		C_R	0.93	0.95	
a_{max} (g)	0.400 ± 0.060	0.40		C_S	1.00	1.00	
r_d	0.96 ± 0.073	0.96		C_B	1.00	1.00	
CSR	0.372 ± 0.063	0.366		C_E	1.22	1.22	
Equivalent Magnitude	7.6	7.6		C_N	1.25	1.21	
MSF		0.97		$(N_1)_{60}$	16.5 ± 4.1	16.4	
CSRN		0.359					



γ_m=105±3 pcf

γ_s=125±3 pcf

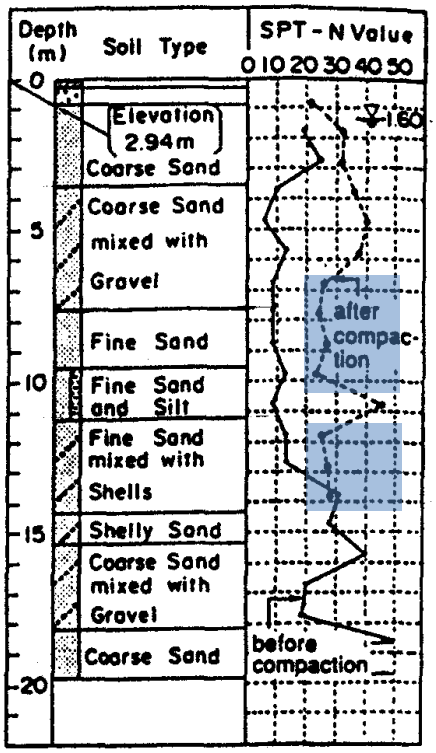


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4.2	13.8	0.353	10.0	1.36	0.89	***14.8***
5.2	17.1	0.372	15.0	1.25	0.93	***21.2***
6.2	20.3	0.382	10.0	1.17	0.96	***13.6***
					Mean:	16.5
					St. Dev.	4.1

Case number:	151
Earthquake:	1993 Kushiro-Oki
Magnitude:	7.8 (Mjma)
Location:	Quay Wall Site D
References:	Iai et al (1994)
Nature of Failure:	No surface evidence of liquefaction
Comments:	On January 15, 1993, an earthquake of magnitude 7.8 in JMA scale hit a northern island of Japan called Hokkaido. The epicenter was located 15 km south of Kushiro City. Kushiro Port was shaken with a peak horizontal acceleration of 0.47 and 0.34g. The earthquake caused liquefaction of untreated backfill sand at the port, resulting in damage to many of the quay walls. The extent of the damage was generally moderate; the displacements of the quay walls were less than about 5% of the water depth in front of the quay walls. In contrast, the quay walls having the backfill sand compacted as measures against liquefaction survived the earthquake without damage. Kushiro Port was developed at the estuary of the Kushiro River. Most of the port area has been reclaimed from the sea by filling the sand dredged from the nearby sea. The natural ground along the former beach line is of sand dune origin being dense with SPT N values of about 50. The fill above the original ground gradually increases its thickness from none at the former beach line to about 10 m at the southern end of the piers in the West Port District. SPT was conducted before the earthquake. The Site had been improved by sand compaction piles.

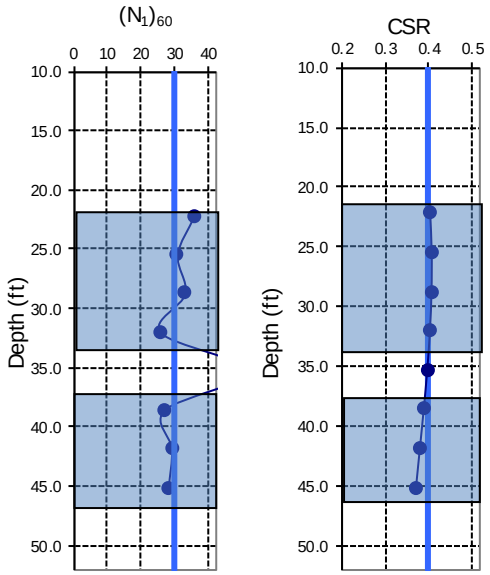
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.340 ± 0.050		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	24.6 - 45.9	35.4		% PI			
Depth to GWT (ft)	5.2 ± 0.3	5.2					
σ_v (psf)	4382.4 ± 453.6	4344.2		N	26.7 ± 3.1	26.8	
σ_v' (psf)	2509.2 ± 241.2	2464.5		C_R	1.00	1.00	
a_{max} (g)	0.400 ± 0.060	0.40		C_S	1.00	1.00	
r_d	0.88 ± 0.134	0.89		C_B	1.00	1.00	
CSR	0.398 ± 0.086	0.408		C_E	1.22	1.22	
Equivalent Magnitude	7.6	7.6		C_N	0.92	0.95	
MSF		0.97		$(N_1)_{60}$	29.8 ± 3.5	30.9	
CSRN		0.434					



γ_m=120±3 pcf

γ_s=125±3 pcf

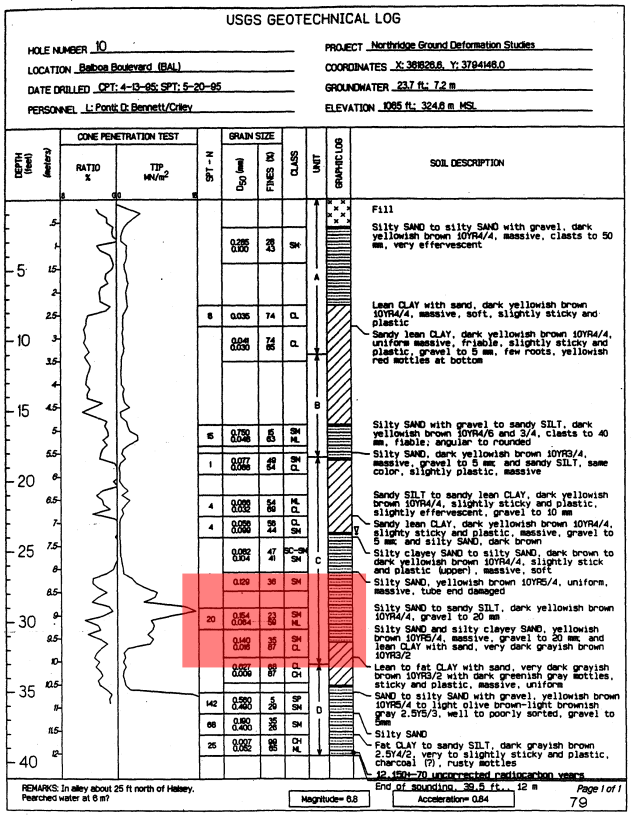


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
6.8	22.1	0.405	27.0	1.12	0.97	*35.7*	
7.8	25.4	0.408	24.0	1.06	0.99	*30.7*	
8.8	28.7	0.408	27.0	1.00	1.00	*33.0*	
9.8	32.0	0.404	22.0	0.96	1.00	*25.7*	
10.8	35.3	0.398	46.0	0.92	1.00	51.4	gravel?
11.8	38.5	0.390	25.0	0.88	1.00	*26.9*	
12.8	41.8	0.381	28.0	0.85	1.00	*29.0*	
13.8	45.1	0.372	28.0	0.82	1.00	*28.0*	
					Mean:	29.8	
					St. Dev.	3.5	

Case number:	152
Earthquake:	1994 Northridge Earthquake
Magnitude:	6.7 (M_w)
Location:	Balboa Blv. Unit C
References:	Bennett et al. (1998)
Nature of Failure:	Cracks and lateral spread
Comments:	The Northridge Earthquake of January 17, 1994 caused widespread ground deformations on the gently sloping alluvial fan surface of the San Fernando Valley. The San Fernando Valley is a structural valley in southern California, which is filled with up to 4500 m of Tertiary sedimentary rock and alluvial sediment. Holocene age alluvial gravels, sands, and finer sediment with an aggregate thickness ranging from ~8-12 m immediately underlie the gently sloping valley floor.The alluvium was deposited by sediment-laden floodwaters that flowed out of the surrounding mountain canyons. The Balboa Boulevard site is on the northern margin of the San Fernando Valley where a 5 km long belt defined by a complex pattern of ground cracks formed in the Granada and Mission Hills. Ground cracks in most of the belt developed in Holocene alluvium on a gentle south-sloping surface. The site is underlain by Holocene silty sand to lean clay that ranges in thickness from 8-10 m. The uppermost unit, A, is less than 1 m thick and consists of fill and reworked sandy silt and lean clay with sand. The upper part of the primary Holocene section, unit B, consists of sheet flood and debris flow deposits and the lower part unit C, consists of fluvial deposits. The locations of maximum compression and extension correlate closely with where the water table falls below the base of the Holocene fluvial unit C. The part of unit C below water table has an average fines content of 52% and clay content of 18 %. Only 8 SPTs in unit C are below water table. Unit D is much coarser grained but is still very silty. Average fines content is 48%. 44 SPT's in unit D. PGA was recorded as 0.83 and 0.47g 2.3 km east of the study site at Los Angles Department of Water and Power Rinaldi Receiving station. Other nearby recorded PGA's were as high as 0.94 g; contoured regional maps of peak acceleration imply a value of 0.85 g. Site response analyses results suggests a value of 0.7 g.

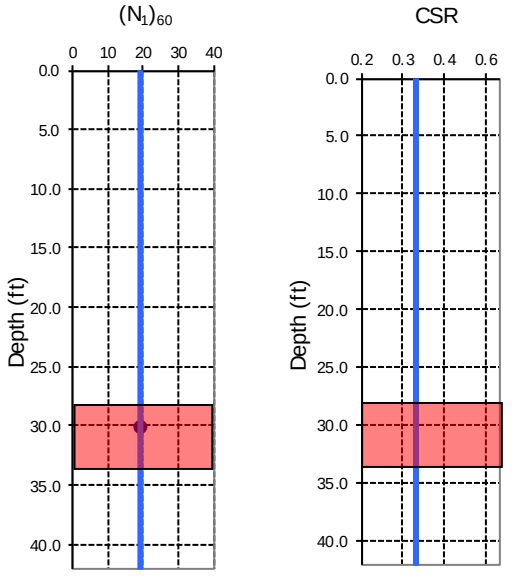
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.099 ± 0.040		
Data Class	B			% Fines	48.0 ± 15.0	50.0	
Critical Depth Range	27.1 - 32.0	27.9		% PI	11		
Depth to GWT (ft)	23.6 ± 0.4	23.6					
σ_v (psf)	3336.6 ± 126.0	3258.1		N	19.9 ± 4.2	13.6	
σ_v' (psf)	2968.1 ± 91.3	2986.6		C_R	1.00	1.00	
a_{max} (g)	0.693 ± 0.104	0.84		C_S	1.00	1.00	
r_d	0.71 ± 0.005	0.87		C_B	1.00	1.00	
CSR	0.357 ± 0.054	0.428		C_E	1.13	1.13	
Equivalent Magnitude	6.7	6.7		C_N	0.84	0.86	
MSF		1.24		$(N_1)_{60}$	19.0 ± 4.0	13.1	
CSRN		0.362					



γ_m=110±3 pcf

γ_s=125±3 pcf



CSR is estimated by site response

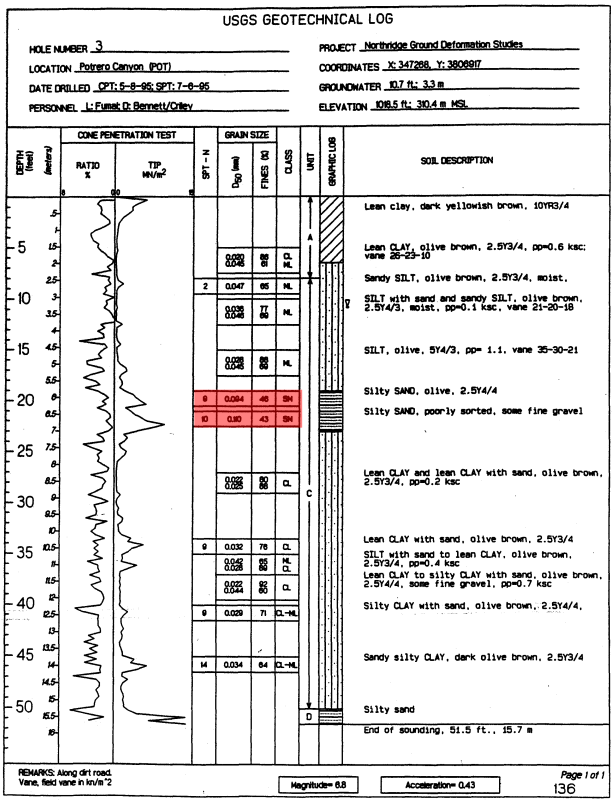
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
9.2	30.2		20.0	0.84	1.00	***19.0***
					Mean:	19.0
					St. Dev.	4.0

given in the source document

Case number:	153
Earthquake:	1994 Northridge Earthquake
Magnitude:	6.7 (Mw)
Location:	Potrero Canyon Unit C
References:	Bennett et al. (1998)
Nature of Failure:	Cracks and lateral spread
Comments:	The Northridge Earthquake of January 17, 1994 caused widespread ground deformations on the gently sloping alluvial fan surface of the San Fernando Valley. The San Fernando Valley is a structural valley in southern California, which is filled with up to 4500 m of Tertiary sedimentary rock and alluvial sediment. Holocene age alluvial gravels, sands, and finer sediment with an aggregate thickness ranging from ~8-12 m immediately underlie the gently sloping valley floor.The alluvium was deposited by sediment-laden floodwaters that flowed out of the surrounding mountain canyons. Potrero Canyon is a narrow 5 km long E-W valley incised into Pliocene sedimentary rock on the north flank of the Santa Susana Mountains. Altough it is about 22 km NNW of the epicenter of the Northridge earthquake, it is near the up dip projection of the seismogenic rupture surface. Ground failure in Potrero Canyon consisted of two zones of ground cracks along the north and south edges of the valley. Cracks formed in alluvium at or near the contact between the valley fill and the Pliocene bedrock forming the adjacent valley walls. Horizontal displacements across ground cracks were at least 10 cm. The Valley is underlain by fine grained Holocene sediment, units A and C, most of which is below the water table. Depth to groundwater was about 2.5 m. The uppermost unit, A, is a homogeneous lean clay to silt with an average thickness of 2.7 m. The remainder of Holocene section, unit C, consists of lean clay to sandy silt with lenses of silt and silty sand, C1. The Holocene sediment rests on dense silty sand, which may either Pleistocene or part of Pliocene Pico Formation, unit D. The average fines and clay contents for unit C are 57 and 11 % respectively. PGA was recorded as 0.43 and 0.37g 3.8 km east of the study site at NWS. Contoured regional maps of peak acceleration imply a value of 0.35 g

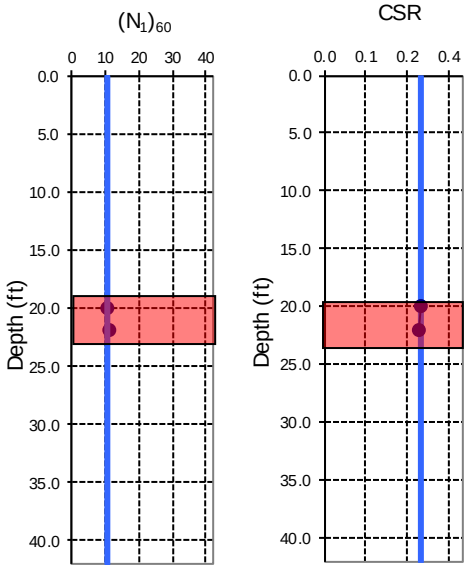
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.102 ± 0.011		
Data Class	B			% Fines	44.5 ± 2.1	64.0	
Critical Depth Range	19.7 - 23.0	23.3		% PI	5		
Depth to GWT (ft)	10.8 ± 0.4	6.6					
σ_v (psf)	2503.3 ± 82.1	2903.1		N	9.5 ± 0.6	7.4	
σ_v' (psf)	1848.2 ± 59.1	1837.9		C_R	0.94	0.95	
a_{max} (g)	0.399 ± 0.120	0.43		C_S	1.00	1.00	
r_d	0.79 ± 0.087	0.91		C_B	1.00	1.00	
CSR	0.279 ± 0.089	0.320		C_E	1.13	1.13	
Equivalent Magnitude	6.7	6.7		C_N	1.07	1.07	
MSF		1.24		$(N_1)_{60}$	10.8 ± 0.7	8.5	
CSRN		0.254					



γ_m=110±3 pcf

γ_s=125±3 pcf



Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC (%)	D50 (mm)
6.1	20.0	0.279	9.0	1.09	0.93	***10.4***	***46.0***	***0.094***
6.7	22.0	0.278	10.0	1.06	0.95	***11.3***	***43.0***	***0.110***
					Mean:	10.8	44.5	0.102
					St. Dev.	0.7	2.1	0.011

Case number:	154
Earthquake:	1994 Northridge Earthquake
Magnitude:	6.7 (Mw)
Location:	Wynne Ave. Unit C1
References:	Bennett et al. (1998)
Nature of Failure:	Cracks and lateral spread
Comments:	The Northridge Earthquake of January 17, 1994 caused widespread ground deformations on the gently sloping alluvial fan surface of the San Fernando Valley. The San Fernando Valley is a structural valley in southern California, which is filled with up to 4500 m of Tertiary sedimentary rock and alluvial sediment. Holocene age alluvial gravels, sands, and finer sediment with an aggregate thickness ranging from ~8-12 m immediately underlie the gently sloping valley floor.The alluvium was deposited by sediment-laden floodwaters that flowed out of the surrounding mountain canyons. The Wynne Ave. site is located couple of kms north east from the epicenter. Ground failure at the Wynne Ave. site consisted of an 150 m long by 12 m wide down dropped block. Vertical offset across cracks ranged from 100-200 mm. Groundwater beneath the failure zone was was at a depth of about 4.3 m. The site is underlain by 2 m of silty sand and sandy lean clay, unit A, which in turn is underlain by 12-14 m of Holocene sandy silt to lean clay, unit B and C. Unit B consists of multiple fining upward sequences of overbank and sheetflood deposits that were deposited in a flood basin between major distributary flood channels. Unit C, which is texturally comparable to B, is distinguished by the presence of lenticular sand layers. Immediately beneath the failure, two sand layers are intercalated in unit C at depths of 6 and 10 m. The shallower sand layer, C1, is about 2 m thick. Deeper sand layer C2, is 1.5 thick. Both layers have average fines and clay content of 38 and 11 %. PGA was recorded as 0.47 and 0.36g 1.7 km south-east of the study site at NRG. Contoured regional maps of peak acceleration imply a value of 0.50 g. Site response based estimation of PGA is 0.54 g

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.106 ± 0.037		
Data Class	A			% Fines	42.4 ± 8.9	33.0	
Critical Depth Range	18.9 - 22.1	22.0		% PI			
Depth to GWT (ft)	13.7 ± 1.5	14.1					
σ_v (psf)	2358.1 ± 85.5	2694.2		N	10.3 ± 1.5	11.0	
σ_v' (psf)	1931.6 ± 93.1	2193.0		C_R	0.93	0.95	
a_{max} (g)	0.540 ± 0.081	0.51		C_S	1.00	1.00	
r_d	0.86 ± 0.040	0.91		C_B	1.00	1.00	
CSR	0.350 ± 0.061	0.390		C_E	1.13	1.13	
Equivalent Magnitude	6.7	6.7		C_N	1.05	0.98	
MSF		1.24		$(N_1)_{60}$	11.4 ± 1.7	11.6	
CSRN		0.317					

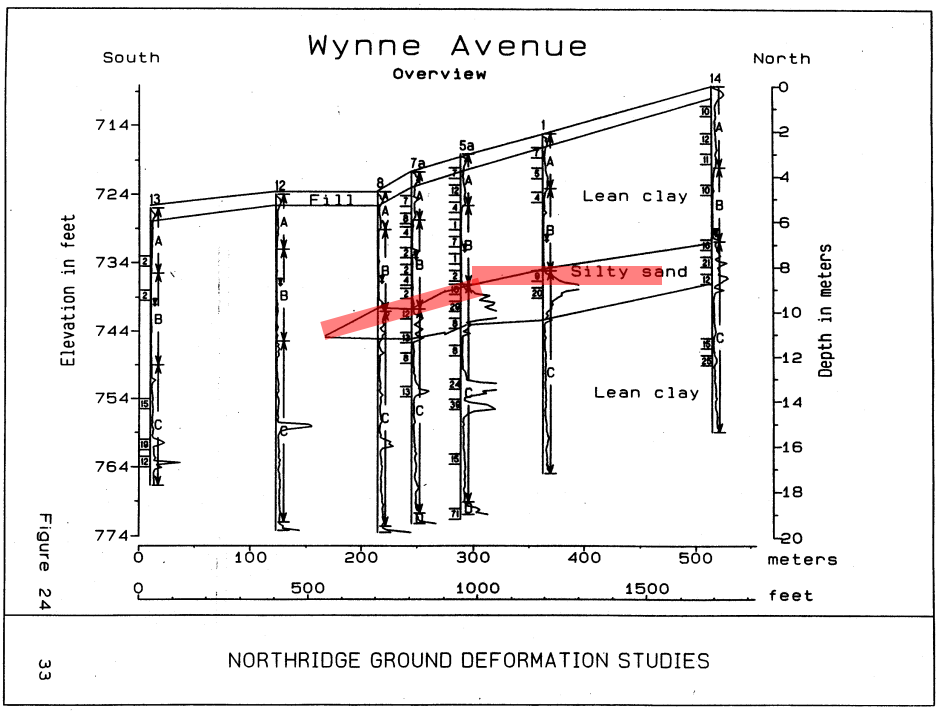


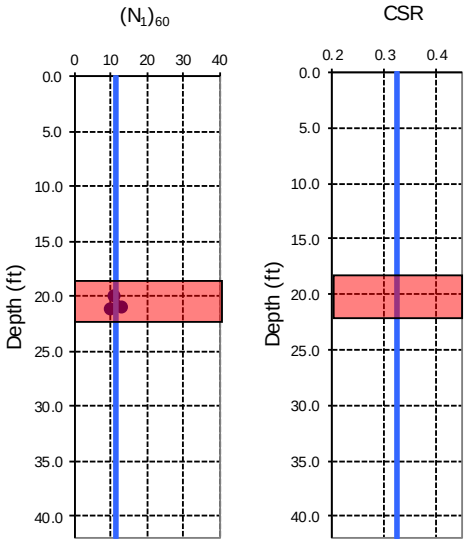
Figure 24

NORTHRIDGE GROUND DEFORMATION STUDIES

33

γ_m=110±3 pcf

γ_s=125±3 pcf



	D_{50} (mm)	FC	
BH-5a	***0.160***	***31.0***	SM
BH-7a	***0.098***	***43.0***	SM
BH-7a	***0.068***	***54.0***	SM??
BH-1a	***0.079***	***47.0***	SM
BH-1a	***0.123***	***37.0***	SM
	0.106	42.4	
	0.037	8.9	

CSR is estimated by site response

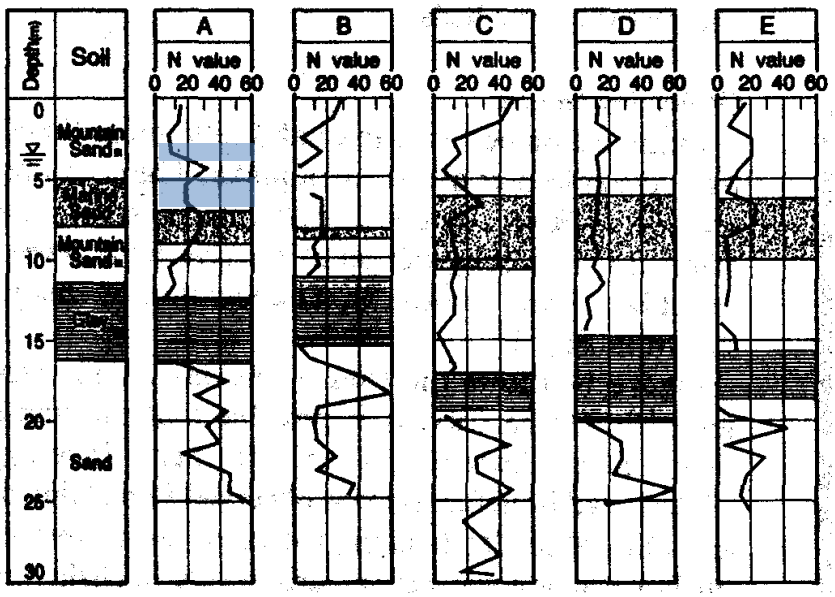
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
6.1	20.0		10.0	1.06	0.93	***11.1***	BH-5a
6.4	21.0		12.0	1.04	0.94	***13.2***	BH-7a
6.5	21.2		9.0	1.04	0.94	***9.9***	BH-1a
					Mean:	11.4	
					St. Dev.	1.7	

454545

Case number:	155
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Ashiyama A (Mountain Sand 1)
References:	Shibata et al.(1996)
Nature of Failure:	From the cracks in roads, building foundations, and around utility poles, massive amounts of sand were ejected. Many two-story houses tilted and settled differentially. However, this site is located in the non-liquefied zone.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed art numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and inland areas with high ground water tables. The reclaimed island known as Ashiyahama was reclamed by Hyogo Prefecture using mainly Masado, weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. The reclamation of Ashiyahama began in 1969 and completed in 1975. An alluvial clay layer of 3-5 m in thickness was deposited just beneath the old sea level, and the total thickness of the reclamation is 13-15 m. The fill material used was mainly debris from mountainside construction sites, but also a marine sand layer having an N value of ~10 exists about 5 m below the ground surface. The erupted sand has the characteristics of marine sand layer. In the vicinity of the site, PGA was recorded as 0.3-0.4g at liquefied sites and 0.6-0.8 g at non-liquefied sites. The PGA value of 0.326g, was recorded at Uozakihama, which is 2-3 km south-west of Ashiyahama. A PGA value of 0.4 g was adopted. The average Japanese SPT energy ratio of 73% is used.

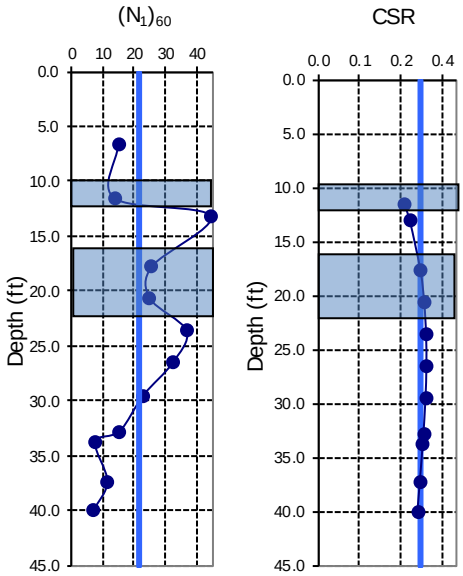
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.126 ± 0.050		
Data Class	B			% Fines	18.0 ± 2.0	18.0	
Critical Depth Range	11.5 - 22.6	17.1		% PI			
Depth to GWT (ft)	11.5 ± 0.3	11.5					
σ_v (psf)	1960.3 ± 235.6	2025.9		N	16.6 ± 5.3	16.6	
σ_v' (psf)	1612.3 ± 123.3	1670.8		C_R	0.93	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.94 ± 0.072	0.94		C_B	1.00	1.00	
CSR	0.297 ± 0.065	0.295		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.15	1.10	
MSF		1.17		$(N_1)_{60}$	21.5 ± 6.8	21.1	
CSRN		0.243					



γ_m=110±3 pcf

γ_s=125±3 pcf

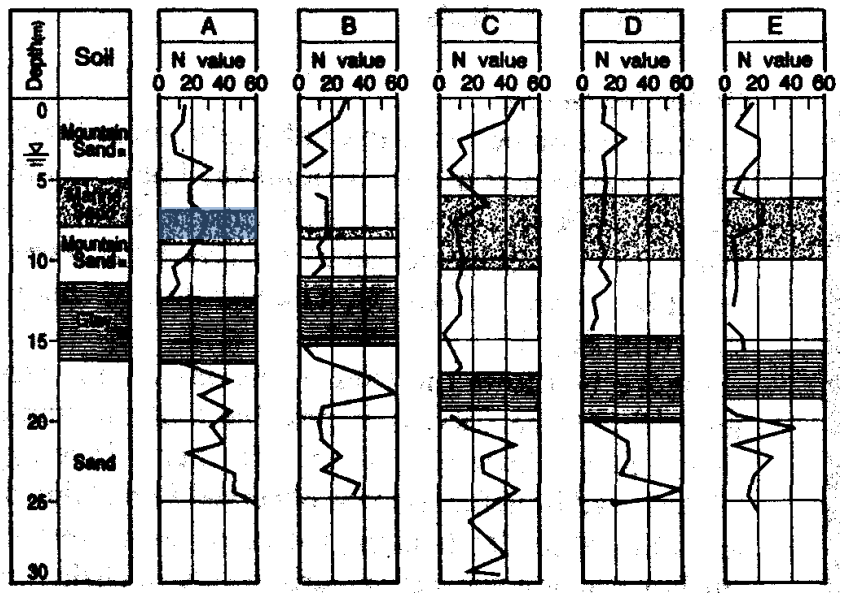


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.0	6.6	NA	9.0	1.71	0.80	15.0	
3.5	11.5	0.252	10.0	1.29	0.87	***13.7***	
4.0	13.1	0.269	33.0	1.24	0.89	44.3	Gravel ?
5.4	17.7	0.300	20.0	1.13	0.93	***25.7***	
6.3	20.7	0.311	20.0	1.07	0.96	***25.0***	
7.2	23.6	0.316	30.0	1.02	0.98	36.7	
8.1	26.6	0.317	27.0	0.98	1.00	32.2	
9.0	29.5	0.315	20.0	0.94	1.00	22.9	
10.0	32.8	0.308	14.0	0.90	1.00	15.4	
10.3	33.8	0.306	7.0	0.89	1.00	7.6	
11.4	37.4	0.297	11.0	0.86	1.00	11.5	
12.2	40.0	0.290	7.0	0.83	1.00	7.1	
					Mean:	21.5	
					St. Dev.	6.8	

Case number:	156
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Ashiyama A (Marine Sand)
References:	Shibata et al.(1996)
Nature of Failure:	From the cracks in roads, building foundations, and around utility poles, massive amounts of sand were ejected. Many two-story houses tilted and settled differentially. However, this site is located in the non-liquefied zone.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and inland areas with high ground water tables. The island known as Ashiyahama was reclamed by Hyogo Prefecture using mainly Masado, weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. The reclamation of Ashiyahama began in 1969 and completed in 1975. An alluvial clay layer of 3-5 m in thickness was deposited just beneath the old sea level, and the total thickness of the reclamation is 13-15 m. The fill material used was mainly debris from mountainside construction sites, but also marine sand layer having an N value of ~10 exists about 5 m below the ground surface. The erupted sand has the characteristics of marine sand layer. In the vicinity of the site, PGA was recorded as 0.3-0.4 g at liquefied sites and 0.6-0.8 g at non-liquefied sites. The PGA value of 0.326g, was recorded at Uozakihama, which is 2-3 km south-west of Ashiyahama. A PGA value of 0.4 g was adopted.The average Japanese SPT energy ratio of 73% is used.

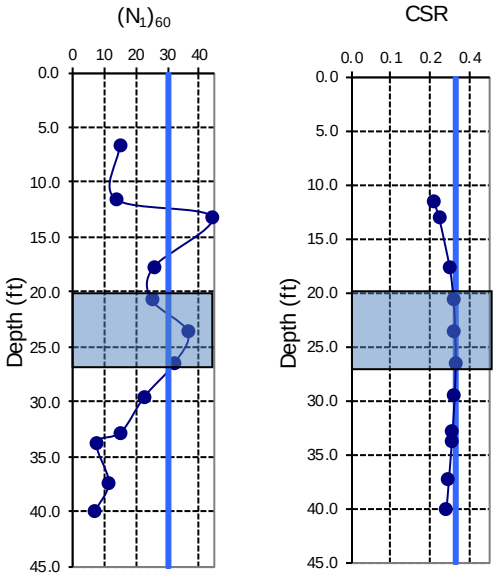
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No			D_{50}	0.185 ± 0.050		
Data Class	B			% Fines	2.0 ± 2.0		
Critical Depth Range	22.6 - 29.5			% PI			
Depth to GWT (ft)	11.5 ± 0.3						
σ_v (psf)	3088.1 ± 154.0			N	25.5 ± 5.8		
σ_v' (psf)	2177.1 ± 92.1			C_R	1.00		
a_{max} (g)	0.400 ± 0.080			C_S	1.00		
r_d	0.86 ± 0.104			C_B	1.00		
CSR	0.317 ± 0.074			C_E	1.22		
Equivalent Magnitude	6.9			C_N	0.99		
MSF				$(N_1)_{60}$	30.6 ± 7.0		
CSRN							



γ_m=110±3 pcf

γ_s=125±3 pcf

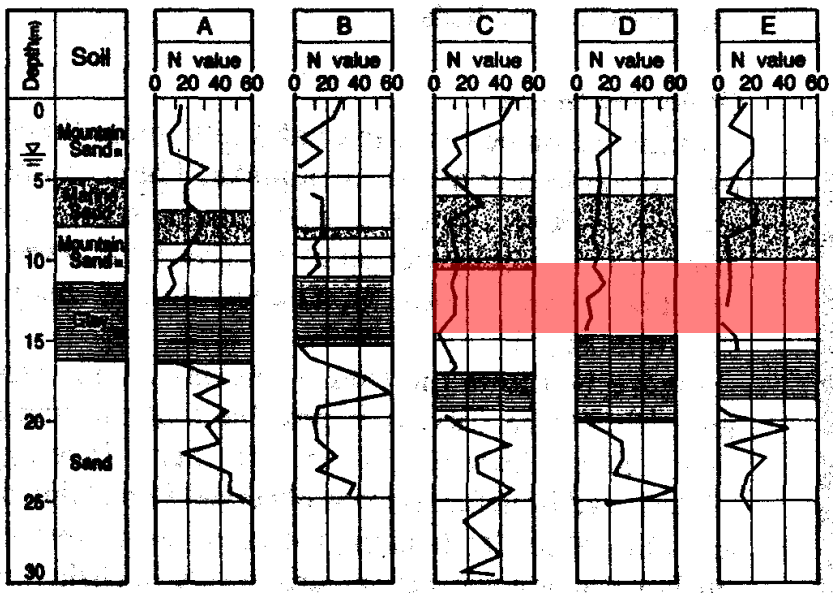


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.0	6.6	NA	9.0	1.71	0.80	15.0	
3.5	11.5	0.252	10.0	1.29	0.87	13.7	
4.0	13.1	0.269	33.0	1.24	0.89	44.3	Gravel ?
5.4	17.7	0.300	20.0	1.13	0.93	25.7	
6.3	20.7	0.311	20.0	1.07	0.96	25.0	
7.2	23.6	0.316	30.0	1.02	0.98	***36.7***	
8.1	26.6	0.317	27.0	0.98	1.00	***32.2***	
9.0	29.5	0.315	20.0	0.94	1.00	***22.9***	
10.0	32.8	0.308	14.0	0.90	1.00	15.4	
10.3	33.8	0.306	7.0	0.89	1.00	7.6	
11.4	37.4	0.297	11.0	0.86	1.00	11.5	
12.2	40.0	0.290	7.0	0.83	1.00	7.1	
					Mean:	30.6	
					St. Dev.	7.0	

Case number:	157
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (ML)
Location:	Ashiyama C&D&E (Mountain Sand 2)
References:	Shibata et al.(1996)
Nature of Failure:	From the cracks in roads, building foundations, and around utility poles, massive amounts of sand were ejected. Many two-story houses tilted and settled differentially.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed art numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and inland areas with high ground water tables. The island known as Ashiyahama was reclaimed by Hyogo Prefecture using mainly Masado, weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. The reclamation of Ashiyahama began in 1969 and completed in 1975. An alluvial clay layer of 3-5 m in thickness was deposited just beneath the old sea level, and the total thickness of the reclamation is 13-15 m. The fill material used was mainly debris from mountainside construction sites, but also a merine sand layer having an N value of ~10 exists about 5 m below the ground surface. The erupted sand has the characteristics of marine sand layer. In the vicinity of the site, PGA was recorded as 0.3-0.4g at liquefied sites and 0.6-0.8 g at non-liquefied sites. The PGA value of 0.326g,recorded at Uozakihama, which is 2-3 km south-west of Ashiyahama. A PGA value of 0.40 g was adopted. The average Japanese SPT energy ratio of 73% is used.

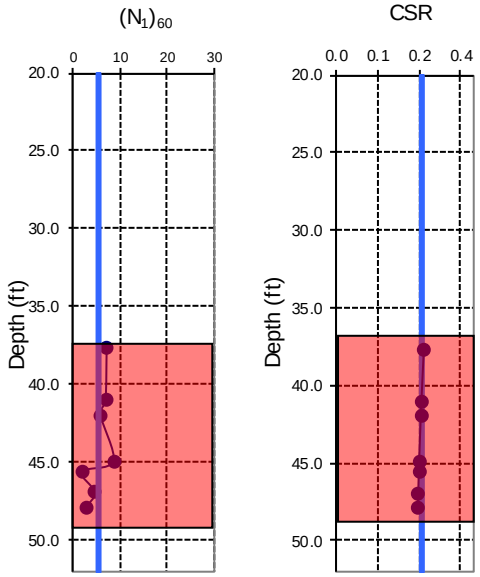
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes			D_{50}	0.126 ± 0.050		
Data Class	B			% Fines	18.0 ± 2.0		
Critical Depth Range	36.1 - 49.2			% PI			
Depth to GWT (ft)	11.5 ± 0.2						
σ_v (psf)	5159.1 ± 291.0			N	5.7 ± 2.7		
σ_v' (psf)	3214.2 ± 169.5			C_R	1.00		
a_{max} (g)	0.400 ± 0.080			C_S	1.00		
r_d	0.59 ± 0.149			C_B	1.00		
CSR	0.246 ± 0.079			C_E	1.22		
Equivalent Magnitude	6.9			C_N	0.81		
MSF				$(N_1)_{60}$	5.6 ± 2.7		
CSRN							



γ_m=110±3 pcf

γ_s=125±3 pcf

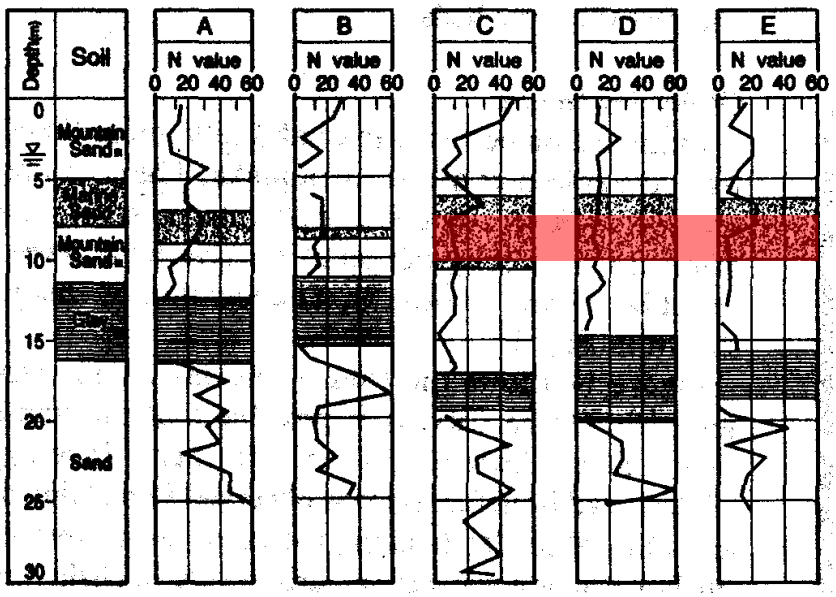


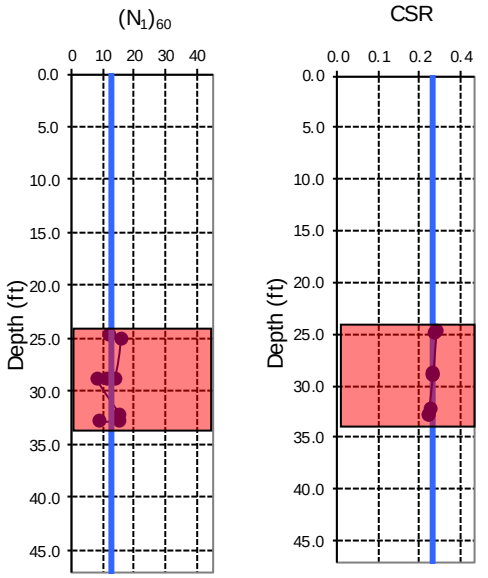
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$		
11.5	37.7	0.257	7.0	0.85	1.00	***7.3***	Site D	
12.5	41.0	0.249	7.0	0.82	1.00	***7.0***	Site E	
12.8	42.0	0.247	6.0	0.82	1.00	***6.0***	Site E	
13.7	44.9	0.242	9.0	0.79	1.00	***8.7***	Site D	
13.9	45.6	0.241	2.0	0.79	1.00	***1.9***	Site E	
14.3	46.9	0.239	5.0	0.78	1.00	4.7	Site D	clay?
14.6	47.9	0.238	3.0	0.77	1.00	***2.8***	Site C	
					Mean:	5.6		
					St. Dev.	2.7		

Case number:	158
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Ashiyama C&D&E (Marine Sand)
References:	Shibata et al.(1996)
Nature of Failure:	From the cracks in roads, building foundations, and around utility poles, massive amounts of sand were ejected. Many two-story houses tilted and settled differentially.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and inland areas with high ground water tables. The reclaimed island known as Ashiyahama was reclamed by Hyogo Prefecture using mainly Masado, weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. The reclamation of Ashiyahama began in 1969 and completed in 1975. An alluvial clay layer of 3-5 m in thickness was deposited just beneath the old sea level, and the total thickness of the reclamation is 13-15 m. The fill material used was mainly debris from mountainside construction sites, but also a marine sand layer having an N value of ~10 exists about 5 m below the ground surface. The erupted sand has the characteristics of marine sand layer. In the vicinity of the site, PGA was recorded as 0.3-0.4g at liquefied sites and 0.6-0.8 g at non-liquefied sites. The PGA value of 0.326g, was recorded at Uozakihama, which is 2-3 km south-west of Ashiyahama. A PGA value of 0.4 g was adopted. The average Japanese SPT energy ratio of 73% is used.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.185 ± 0.050		
Data Class	B			% Fines	2.0 ± 2.0	2.0	
Critical Depth Range	24.6 - 32.8	28.9		% PI			
Depth to GWT (ft)	11.5 ± 0.2	11.5					
σ_v (psf)	3416.2 ± 181.8	3467.0		N	10.9 ± 2.6	10.9	
σ_v' (psf)	2341.4 ± 106.0	2401.8		C_R	1.00	1.00	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.74 ± 0.113	0.88		C_B	1.00	1.00	
CSR	0.281 ± 0.071	0.331		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.95	0.94	
MSF		1.17		$(N_1)_{60}$	12.6 ± 3.0	12.5	
CSRN		0.286					



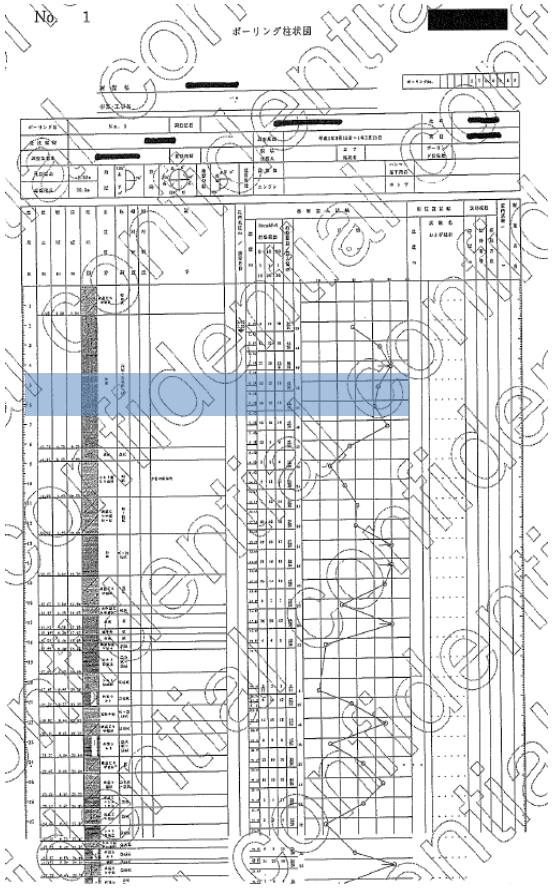


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
7.5	24.6	0.289	10.0	1.01	0.99	***12.1***	Site C
7.6	24.9	0.288	13.0	1.00	0.99	***15.7***	Site D
8.8	28.7	0.281	12.0	0.95	1.00	***13.9***	Site C
8.8	28.9	0.281	10.0	0.95	1.00	***11.5***	Site D
8.8	28.9	0.281	7.0	0.95	1.00	***8.1***	Site E
9.8	32.2	0.272	14.0	0.91	1.00	***15.5***	Site D
10.0	32.8	0.270	14.0	0.90	1.00	***15.4***	Site C
10.0	32.8	0.270	8.0	0.90	1.00	***8.8***	Site E
					Mean:	12.6	
					St. Dev.	3.0	

Case number:	159
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 1
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Non-liquefied
Comments:	

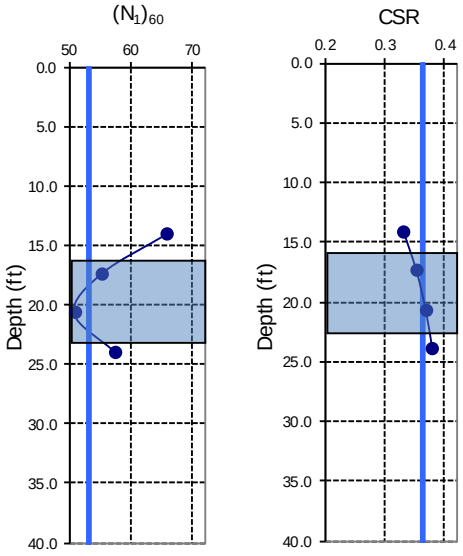
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	3.5 ± 4.9	3.0	
Critical Depth Range	16.4 - 23.0	19.0		% PI			
Depth to GWT (ft)	7.7 ± 0.3	7.9					
σ_v (psf)	2541.8 ± 153.8	2360.1		N	42.4 ± 2.5	42.1	
σ_v' (psf)	1794.6 ± 91.3	1670.8		C_R	0.95	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.99 ± 0.082	0.93		C_B	1.00	1.00	
CSR	0.365 ± 0.079	0.345		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.09	1.07	
MSF		1.17		$(N_1)_{60}$	53.2 ± 3.2	52.0	
CSRN		0.275					



γ_m=120±3 pcf

γ_s=135±3 pcf

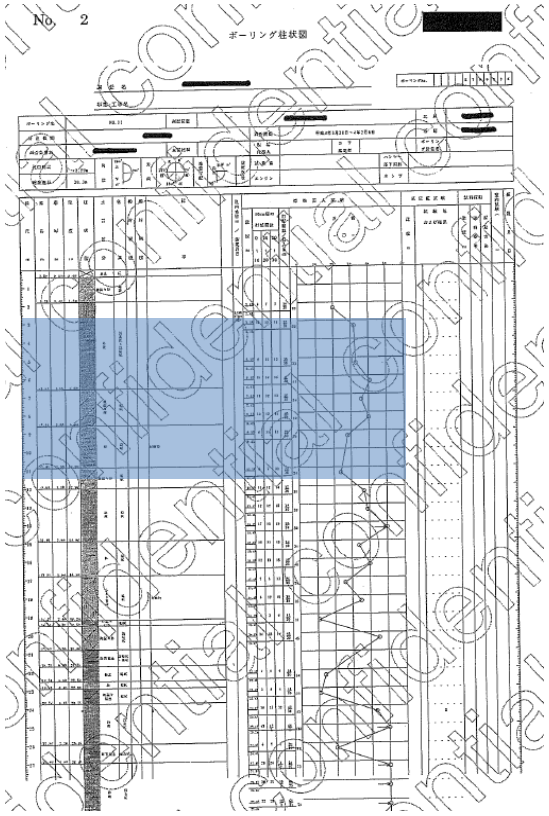


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.3	14.1	0.333	49.0	1.23	0.90	66.0	
5.3	17.4	0.354	43.0	1.14	0.93	*55.5*	*0.0*
6.3	20.7	0.369	41.0	1.06	0.96	*50.9*	*7.0*
7.3	24.0	0.380	48.0	1.00	0.98	57.6	
					Mean:	53.2	3.5
					St. Dev.	3.2	4.9

Case number:	160
Earthquake:	1995 Hypgoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 2
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	

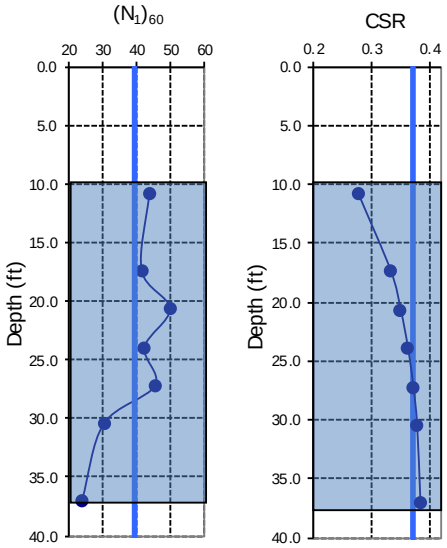
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	14.8 ± 11.5	3.0	
Critical Depth Range	16.4 - 39.4	19.0		% PI			
Depth to GWT (ft)	9.5 ± 0.3	7.9					
σ_v (psf)	3622.5 ± 520.5	2360.1		N	35.2 ± 8.1	42.1	
σ_v' (psf)	2474.0 ± 285.1	1670.8		C_R	1.00	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.98 ± 0.110	0.93		C_B	1.00	1.00	
CSR	0.373 ± 0.086	0.345		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.92	1.07	
MSF		1.17		$(N_1)_{60}$	39.6 ± 9.1	52.0	
CSRN		0.275					



γ_m=120±3 pcf

γ_s=135±3 pcf

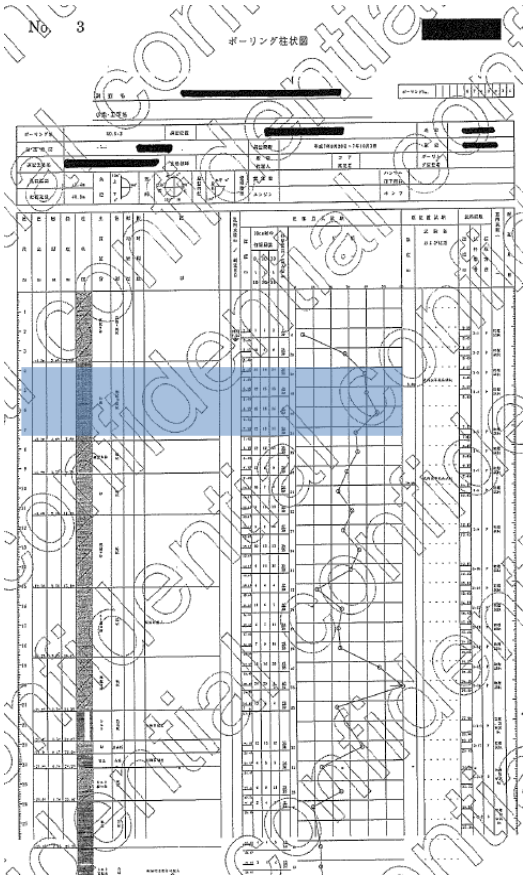


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.3	10.8	0.277	32.0	1.31	0.86	*43.8*	
5.3	17.4	0.333	33.0	1.11	0.93	*41.5*	*8.0*
6.3	20.7	0.350	41.0	1.04	0.96	*49.8*	*0.0*
7.3	24.0	0.362	36.0	0.98	0.98	*42.4*	*7.0*
8.3	27.2	0.372	40.0	0.93	1.00	*45.4*	*30.0*
9.3	30.5	0.378	28.0	0.89	1.00	*30.4*	*22.0*
11.3	37.1	0.384	24.0	0.82	1.00	*24.0*	*22.0*
					Mean:	39.6	14.8
					St. Dev.	9.1	11.5

Case number:	161
Earthquake:	1995 Hypgoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 3
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	

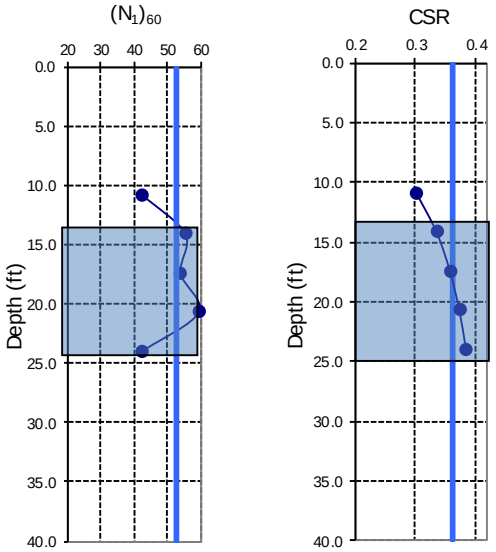
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	3.3 ± 2.2	3.0	
Critical Depth Range	11.5 - 24.6	19.0		% PI			
Depth to GWT (ft)	8.2 ± 0.3	8.2					
σ_v (psf)	2149.0 ± 298.0	2276.5		N	39.5 ± 5.5	40.0	
σ_v' (psf)	1534.8 ± 163.6	1608.2		C_R	0.94	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.99 ± 0.076	0.93		C_B	1.00	1.00	
CSR	0.362 ± 0.078	0.344		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.17	1.08	
MSF		1.17		$(N_1)_{60}$	52.8 ± 7.4	49.8	
CSRN		0.271					



γ_m=100±3 pcf

γ_s=135±3 pcf

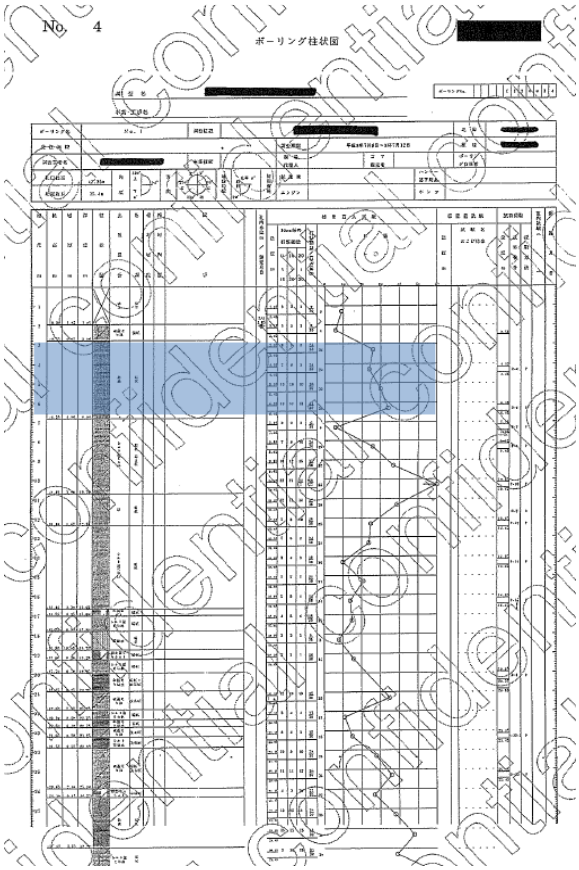


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.3	10.8	0.301	28.0	1.45	0.86	42.4	
4.3	14.1	0.335	39.0	1.30	0.90	*55.4*	*5.0*
5.3	17.4	0.358	40.0	1.19	0.93	*54.0*	*4.0*
6.3	20.7	0.374	46.0	1.11	0.96	*59.4*	*0.0*
7.3	24.0	0.385	34.0	1.04	0.98	*42.3*	*4.0*
					Mean:	52.8	3.3
					St. Dev.	7.4	2.2

Case number:	162
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 4
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	

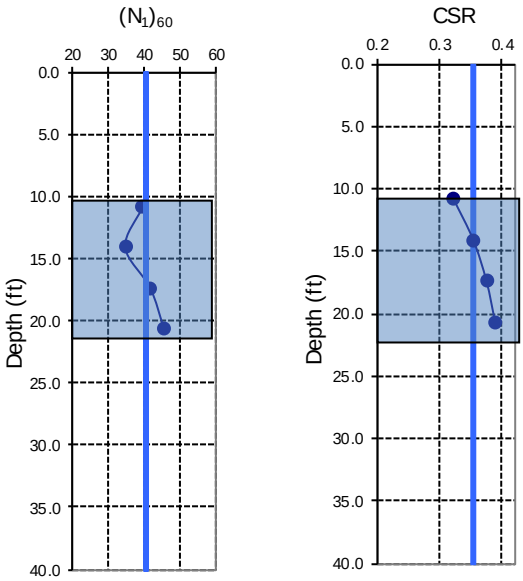
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	1.3 ± 2.3	1.0	
Critical Depth Range	9.8 - 18.0	14.1		% PI			
Depth to GWT (ft)	6.7 ± 0.3	6.9					
σ_v (psf)	1642.1 ± 173.5	1587.3		N	27.8 ± 3.0	25.8	
σ_v' (psf)	1191.7 ± 91.7	1127.8		C_R	0.90	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.99 ± 0.061	0.95		C_B	1.00	1.00	
CSR	0.354 ± 0.075	0.350		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.33	1.23	
MSF		1.17		$(N_1)_{60}$	40.4 ± 4.4	36.6	
CSRN		0.272					



γ_m=110±3 pcf

γ_s=125±3 pcf

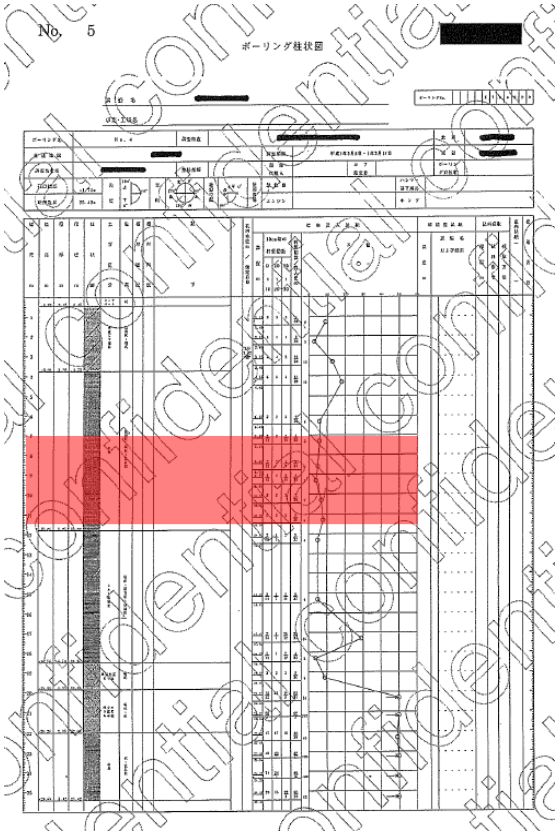


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.3	10.8	0.324	26.0	1.46	0.86	*39.6*	*0.0*
4.3	14.1	0.355	24.0	1.33	0.90	*34.8*	*4.0*
5.3	17.4	0.375	30.0	1.23	0.93	*41.6*	*0.0*
6.3	20.7	0.389	34.0	1.15	0.96	*45.4*	
					Mean:	40.4	1.3
					St. Dev.	4.4	2.3

Case number:	163
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 5
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Surface manifestation of liquefaction
Comments:	

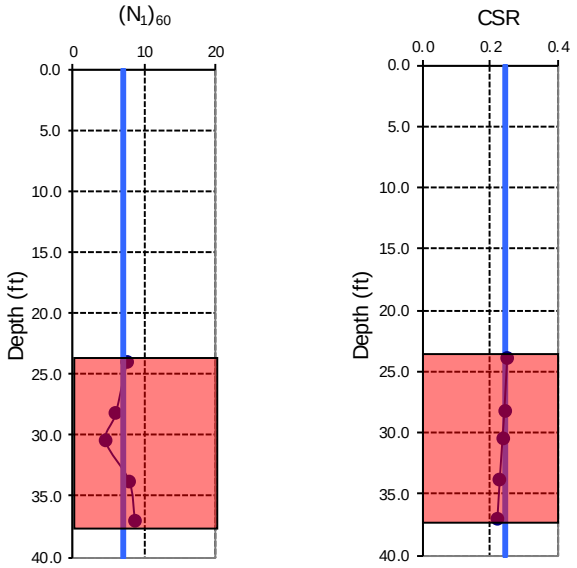
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	1.3 ± 2.5	1.0	
Critical Depth Range	21.3 - 36.1	29.2		% PI			
Depth to GWT (ft)	9.9 ± 0.3	9.8					
σ_v (psf)	3345.8 ± 302.1	3613.2		N	5.8 ± 1.3	5.4	
σ_v' (psf)	2172.7 ± 156.2	2422.7		C_R	1.00	1.00	
a_{max} (g)	0.350 ± 0.070	0.35		C_S	1.00	1.00	
r_d	0.70 ± 0.113	0.88		C_B	1.00	1.00	
CSR	0.244 ± 0.063	0.298		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.99	0.92	
MSF		1.17		$(N_1)_{60}$	7.0 ± 1.6	6.1	
CSRN		0.258					



γ_m=110±3 pcf

γ_s=120±3 pcf

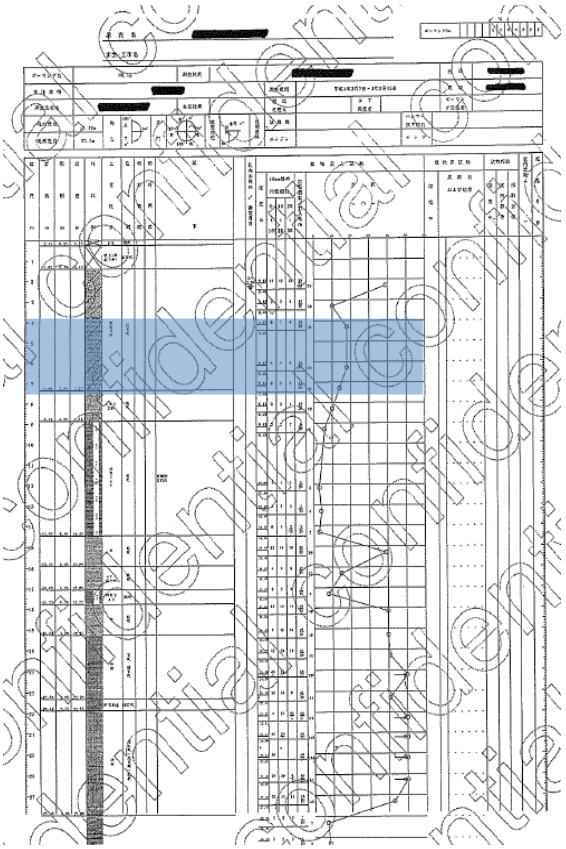


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
7.3	24.0	0.254	6.0	1.06	0.98	***7.6***	***0.0***
8.6	28.2	0.245	5.0	0.99	1.00	***6.0***	***0.0***
9.3	30.5	0.239	4.0	0.96	1.00	***4.7***	***0.0***
10.3	33.8	0.231	7.0	0.93	1.00	***7.9***	***5.0***
11.3	37.1	0.223	8.0	0.89	1.00	***8.7***	
					Mean:	7.0	1.3
					St. Dev.	1.6	2.5

Case number:	164
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 6
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	No surface manifestation of liquefaction.
Comments:	

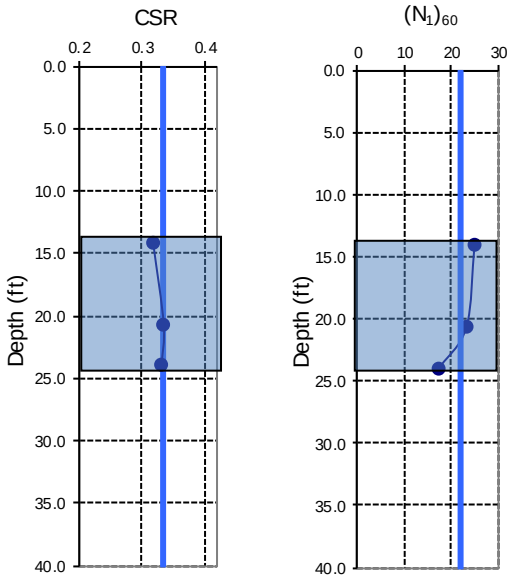
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	24.7 ± 9.2	21.0	
Critical Depth Range	14.1 - 24.0	19.4		% PI			
Depth to GWT (ft)	7.5 ± 0.3	7.5					
σ_v (psf)	2340.9 ± 209.2	2234.7		N	16.7 ± 3.0	13.4	
σ_v' (psf)	1624.3 ± 112.0	1503.8		C_R	0.95	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.89 ± 0.079	0.93		C_B	1.00	1.00	
CSR	0.332 ± 0.073	0.360		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.14	1.15	
MSF		1.17		$(N_1)_{60}$	21.9 ± 3.9	17.8	
CSRN		0.293					



γ_m=120±3 pcf

γ_s=125±3 pcf

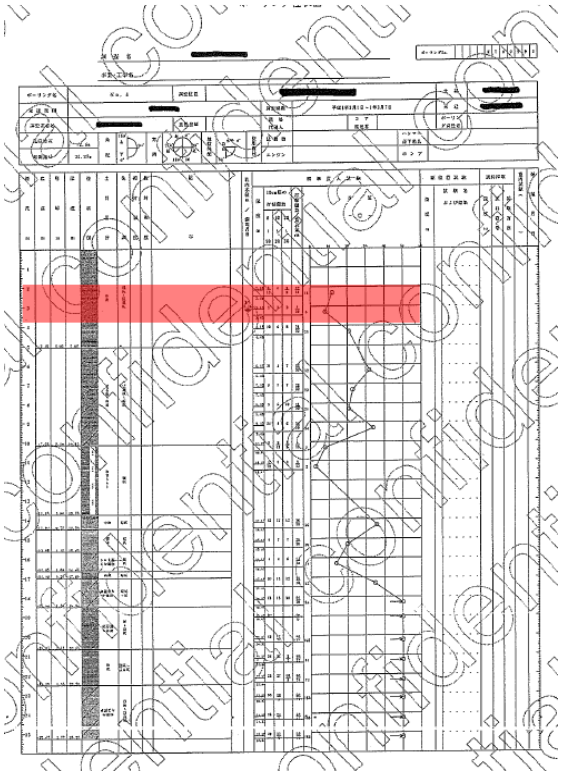


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.3	14.1	0.318	18.0	1.27	0.90	***24.9***	***30.0***
6.3	20.7	0.333	18.0	1.11	0.96	***23.2***	***14.0***
7.3	24.0	0.330	14.0	1.05	0.98	***17.5***	***30.0***
					Mean:	21.9	24.7
					St. Dev.	3.9	9.2

Case number:	165
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 7
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Surface manifestation of liquefaction
Comments:	

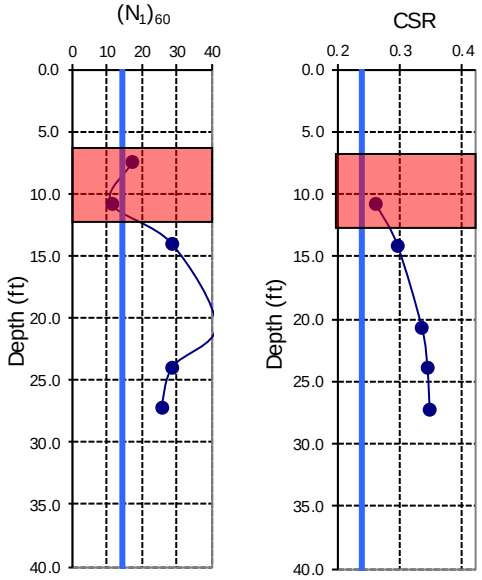
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	5.9 - 12.5	10.8		% PI			
Depth to GWT (ft)	10.4 ± 0.3	10.5					
σ_v (psf)	998.7 ± 134.9	1294.9		N	15.6 ± 5.5	8.0	
σ_v' (psf)	1072.4 ± 72.1	1253.1		C_R	0.84	0.85	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.99 ± 0.043	0.93		C_B	1.00	1.00	
CSR	0.239 ± 0.052	0.314		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.40	1.32	
MSF		1.17		$(N_1)_{60}$	22.3 ± 7.9	10.9	
CSRN		0.262					



γ_m=110±3 pcf

γ_s=120±3pcf

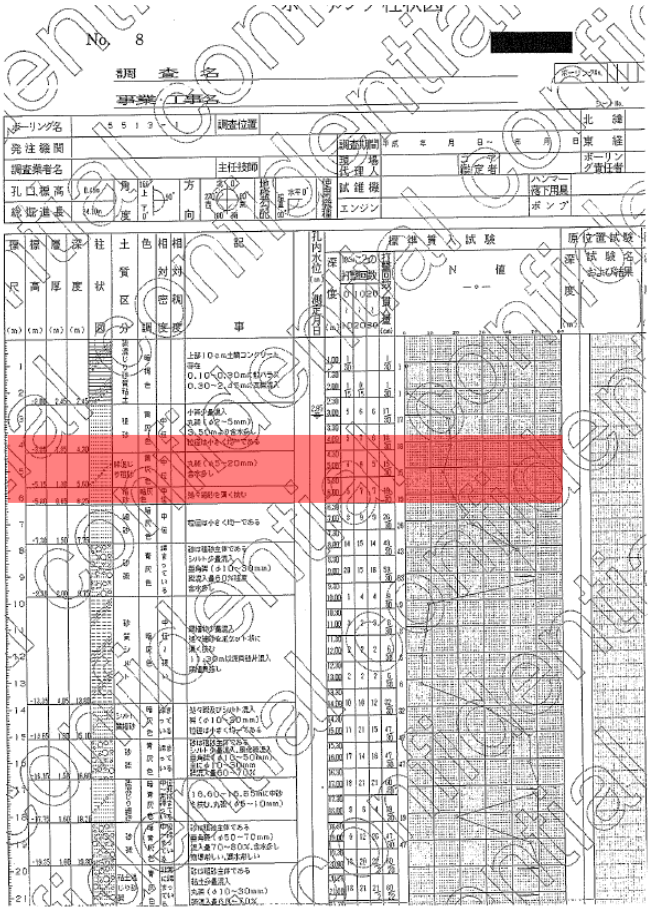


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
2.3	7.5	NA	11.0	1.60	0.81	**17.4**	**0.0**	interpreted
3.3	10.8	0.262	8.0	1.35	0.86	**11.3**	**0.0**	interpreted
4.3	14.1	0.296	21.0	1.25	0.90	28.7	0.0	
6.3	20.7	0.335	32.0	1.10	0.96	41.2	12.0	
7.3	24.0	0.344	23.0	1.05	0.98	28.9	0.0	
8.3	27.2	0.348	21.0	1.00	1.00	25.6	0.0	
					Mean:	14.3	0.0	
					St. Dev.	4.3	2.0	assigned

Case number:	166
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 8
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Surface manifestation of liquefaction
Comments:	

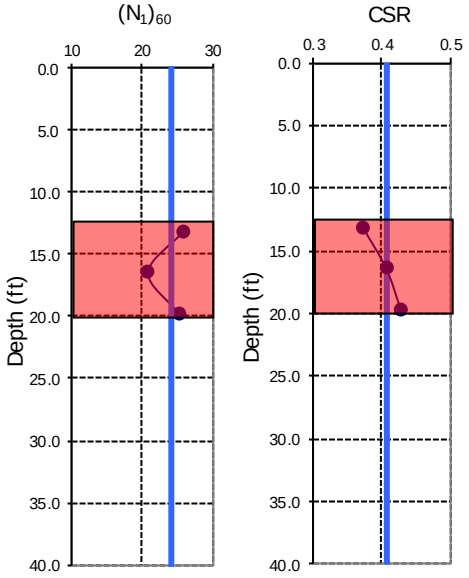
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	13.1 - 19.7	16.4		% PI			
Depth to GWT (ft)	9.7 ± 0.3	9.8					
σ_v (psf)	1808.6 ± 141.4	1775.3		N	17.4 ± 2.0	17.4	
σ_v' (psf)	1388.9 ± 77.9	1357.6		C_R	0.92	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.96 ± 0.070	0.94		C_B	1.00	1.00	
CSR	0.407 ± 0.087	0.402		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.23	1.20	
MSF		1.17		$(N_1)_{60}$	24.0 ± 2.8	24.1	
CSRN		0.321					



γ_m=100±3 pcf

γ_s=125±3 pcf



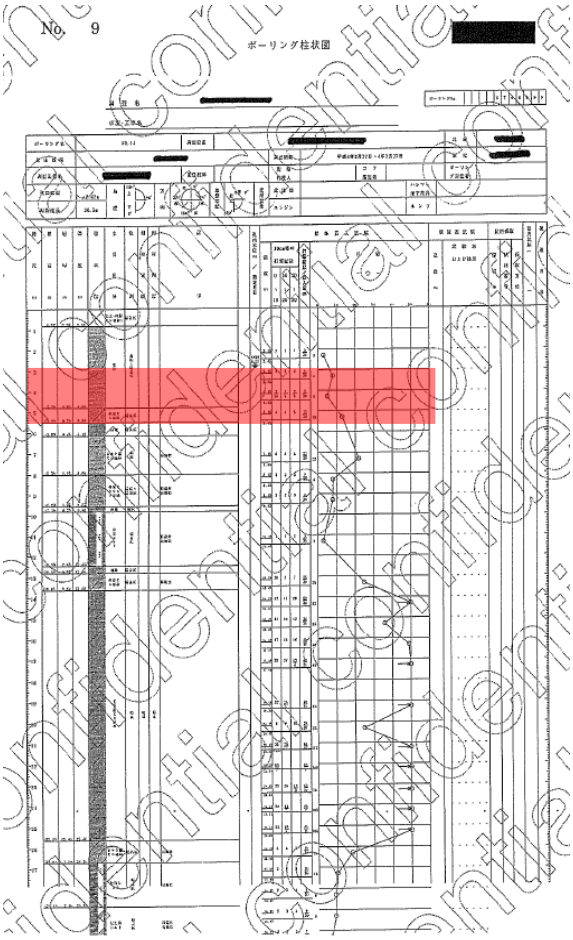
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.374	18.0	1.34	0.89	***26.0***	***0.0***
5.0	16.4	0.407	15.0	1.23	0.92	***20.7***	***0.0***
6.0	19.7	0.427	19.0	1.15	0.95	***25.3***	***0.0***
					Mean:	24.0	0.0
					St. Dev.	2.8	2.0

assigned

Case number:	167
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 9
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	

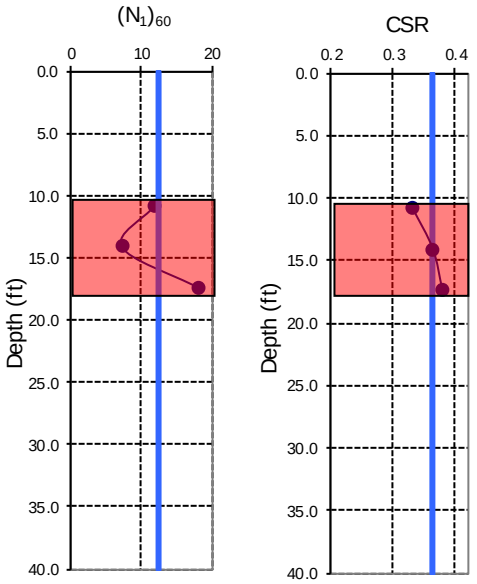
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	2.3 ± 4.0	2.0	
Critical Depth Range	10.8 - 17.4	14.1		% PI			
Depth to GWT (ft)	9.1 ± 0.3	9.2					
σ_v (psf)	1531.5 ± 129.6	1649.9		N	8.6 ± 3.8	8.3	
σ_v' (psf)	1218.3 ± 67.3	1336.7		C_R	0.90	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.89 ± 0.061	0.95		C_B	1.00	1.00	
CSR	0.364 ± 0.078	0.383		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.32	1.27	
MSF		1.17		$(N_1)_{60}$	12.4 ± 5.5	12.2	
CSRN		0.313					



γ_m=105±3 pcf

γ_s=115±3 pcf

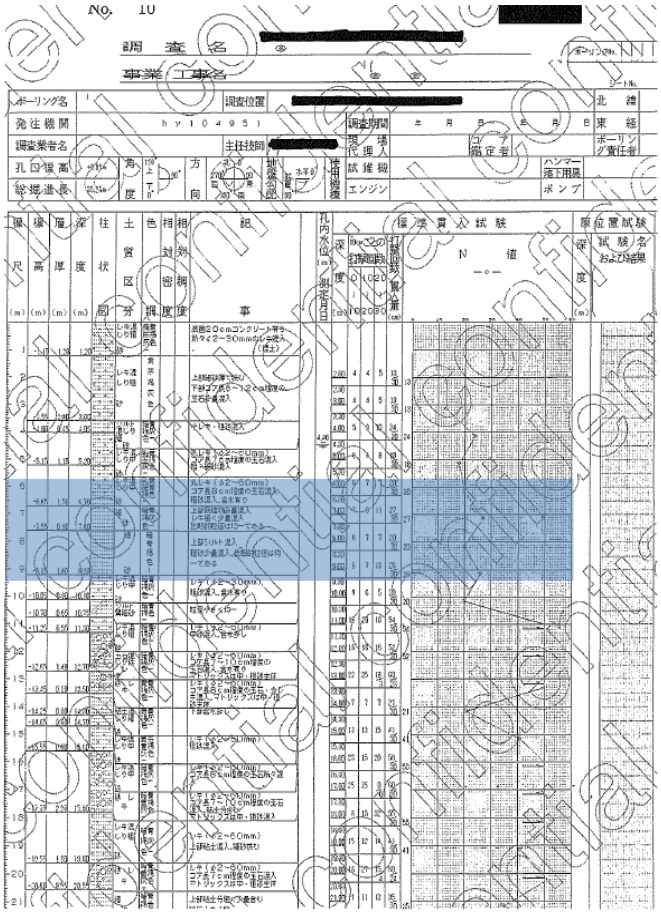


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.3	10.8	0.332	8.0	1.42	0.86	***11.9***	***7.0***
4.3	14.1	0.364	5.0	1.32	0.90	***7.2***	***0.0***
5.3	17.4	0.378	13.0	1.23	0.93	***18.2***	***0.0***
					Mean:	12.4	2.3
					St. Dev.	5.5	4.0

Case number:	168
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 10
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	No surface manifestation of liquefaction
Comments:	

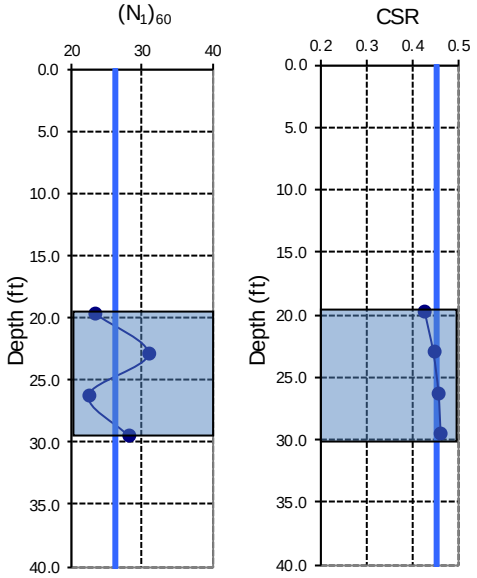
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	8.8 ± 2.5	9.0	
Critical Depth Range	19.7 - 29.5	24.6		% PI			
Depth to GWT (ft)	14.6 ± 0.3	14.8					
σ_v (psf)	3002.6 ± 211.8	2861.3		N	23.1 ± 3.5	24.1	
σ_v' (psf)	2380.3 ± 116.9	2234.7		C_R	0.99	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.92 ± 0.099	0.90		C_B	1.00	1.00	
CSR	0.455 ± 0.104	0.450		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.94	0.98	
MSF		1.17		$(N_1)_{60}$	26.2 ± 4.0	27.4	
CSRN		0.388					



γ_m=120±3 pcf

γ_s=125±3 pcf

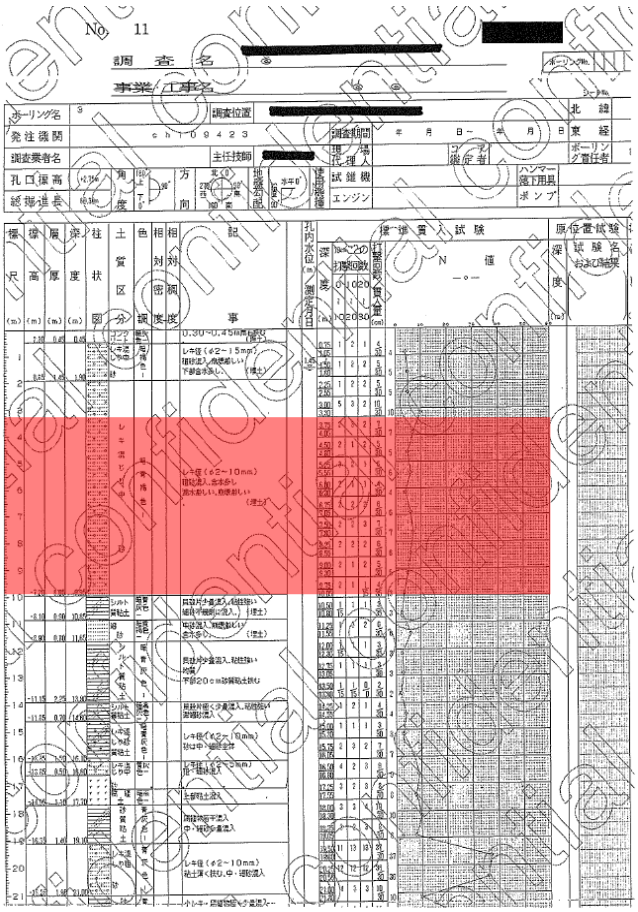


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
6.0	19.7	0.428	20.0	1.01	0.95	*23.4*	*5.0*
7.0	23.0	0.448	27.0	0.96	0.98	*30.9*	*10.0*
8.0	26.2	0.460	20.0	0.92	1.00	*22.5*	*10.0*
9.0	29.5	0.465	26.0	0.89	1.00	*28.1*	*10.0*
					Mean:	26.2	8.8
					St. Dev.	4.0	2.5

Case number:	169
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 11
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

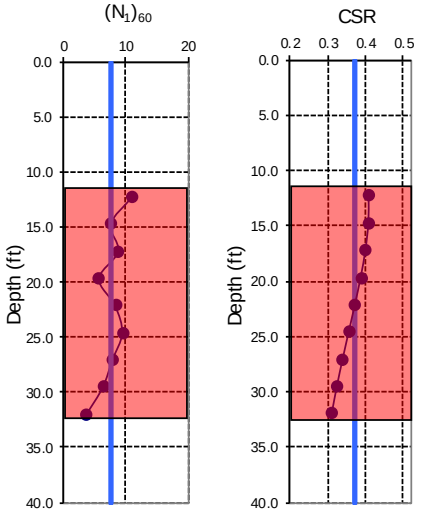
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	12.3 - 32.0	22.3		% PI			
Depth to GWT (ft)	4.8 ± 0.3	4.9					
σ_v (psf)	2609.9 ± 397.4	2380.9		N	5.6 ± 1.6	5.6	
σ_v' (psf)	1524.9 ± 197.2	1294.9		C_R	0.97	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.67 ± 0.090	0.91		C_B	1.00	1.00	
CSR	0.374 ± 0.091	0.546		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.18	1.31	
MSF		1.17		$(N_1)_{60}$	7.7 ± 2.2	8.5	
CSRN		0.447					



γ_m=110±3 pcf

γ_s=120±3 pcf



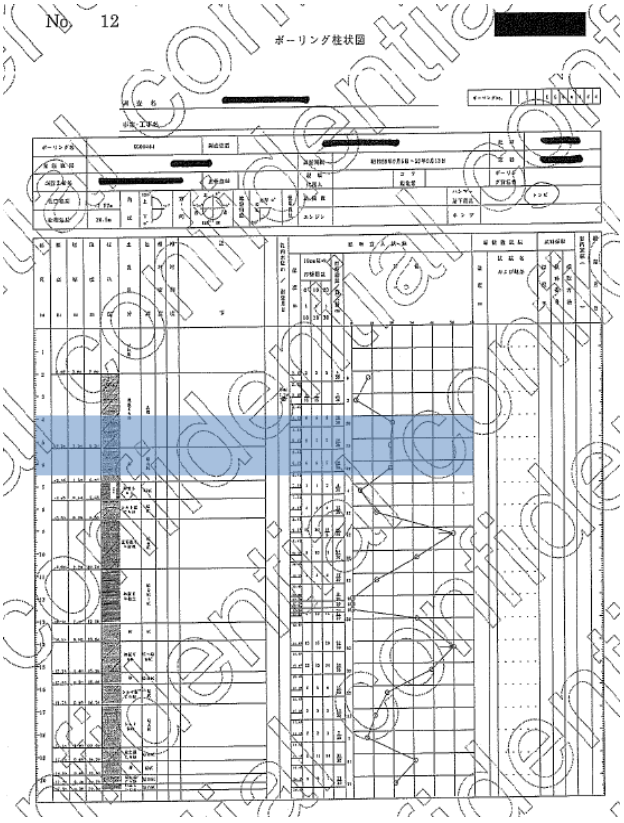
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.8	12.3	0.410	7.0	1.49	0.88	***11.1***	***5.0***
4.5	14.8	0.410	5.0	1.39	0.90	***7.6***	***5.0***
5.3	17.2	0.402	6.0	1.31	0.93	***8.9***	***5.0***
6.0	19.7	0.389	4.0	1.24	0.95	***5.7***	***5.0***
6.8	22.1	0.374	6.0	1.18	0.97	***8.3***	***5.0***
7.5	24.6	0.357	7.0	1.13	0.99	***9.5***	***5.0***
8.3	27.1	0.341	6.0	1.08	1.00	***7.9***	***5.0***
9.0	29.5	0.325	5.0	1.04	1.00	***6.3***	***5.0***
9.8	32.0	0.311	3.0	1.01	1.00	***3.7***	***5.0***
					Mean:	7.7	5.0
					St. Dev.	2.2	2.0

assg

Case number:	170
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 12
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Liquefied
Comments:	

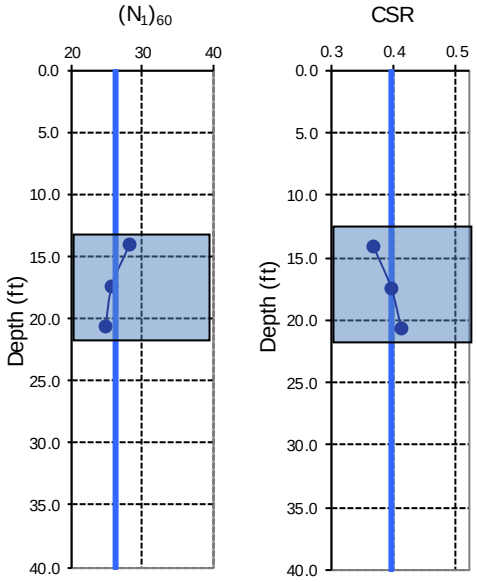
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	14.0 ± 14.0	14.0	
Critical Depth Range	14.1 - 20.7	17.4		% PI			
Depth to GWT (ft)	10.5 ± 0.3	10.5					
σ_v (psf)	1911.1 ± 142.0	1921.5		N	19.6 ± 1.3	18.6	
σ_v' (psf)	1481.2 ± 78.9	1503.8		C_R	0.92	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.94 ± 0.073	0.94		C_B	1.00	1.00	
CSR	0.396 ± 0.086	0.393		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.20	1.14	
MSF		1.17		$(N_1)_{60}$	26.2 ± 1.7	24.7	
CSRN		0.316					



γ_m=100±3 pcf

γ_s=125±3 pcf

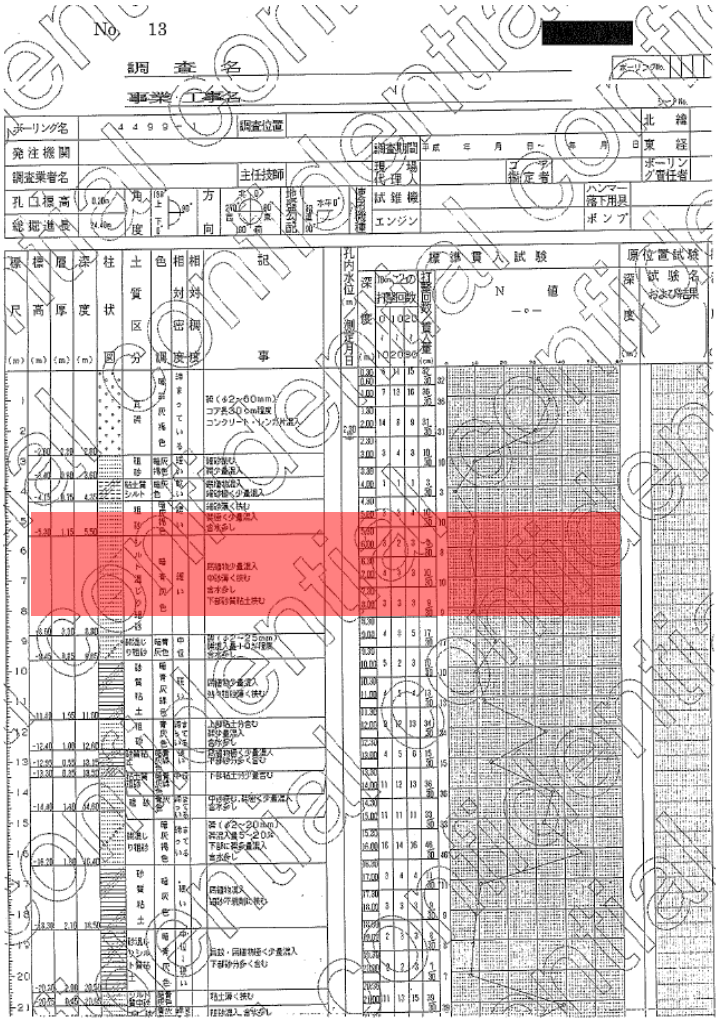


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.3	14.1	0.368	20.0	1.29	0.90	*28.1*	*4.0*
5.3	17.4	0.396	19.0	1.20	0.93	*25.7*	*30.0*
6.3	20.7	0.413	19.0	1.12	0.96	*24.8*	*8.0*
					Mean:	26.2	14.0
					St. Dev.	1.7	14.0

Case number:	171
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 13
References:	Kobe City Office, 1999 (In Japanese), Confidential
Nature of Failure:	Liquefied
Comments:	

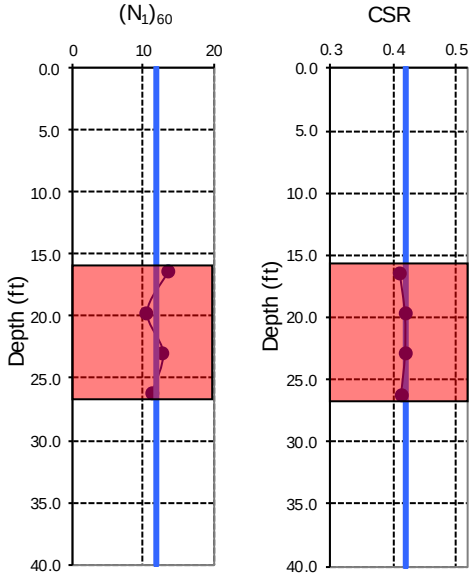
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	15.0 ± 10.0	15.0	
Critical Depth Range	16.4 - 26.2	21.3		% PI			
Depth to GWT (ft)	7.5 ± 0.3	7.5					
σ_v (psf)	2596.8 ± 202.4	2422.7		N	9.3 ± 1.1	9.5	
σ_v' (psf)	1736.9 ± 107.5	1545.5		C_R	0.96	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.87 ± 0.087	0.92		C_B	1.00	1.00	
CSR	0.421 ± 0.095	0.464		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.10	1.16	
MSF		1.17		$(N_1)_{60}$	12.0 ± 1.4	12.7	
CSRN		0.383					



γ_m=125±3 pcf

γ_s=120±3 pcf

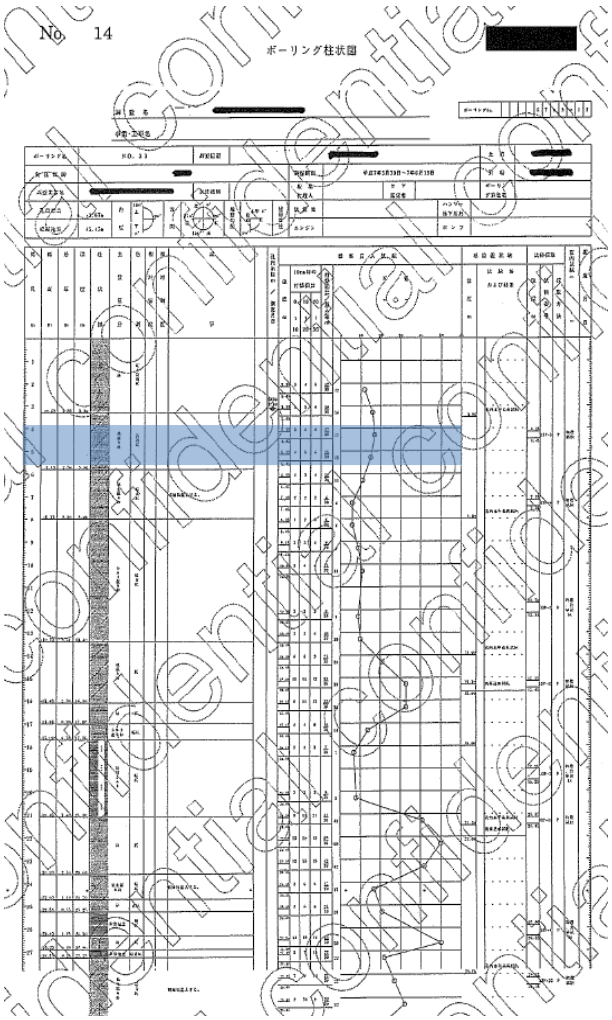


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
5.0	16.4	0.411	10.0	1.21	0.92	***13.5***	***0.0***
6.0	19.7	0.420	8.0	1.14	0.95	***10.5***	***20.0***
7.0	23.0	0.420	10.0	1.07	0.98	***12.8***	***20.0***
8.0	26.2	0.413	9.0	1.02	1.00	***11.2***	***20.0***
					Mean:	12.0	15.0
					St. Dev.	1.4	10.0

Case number:	172
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 14
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Non-Liquefied
Comments:	

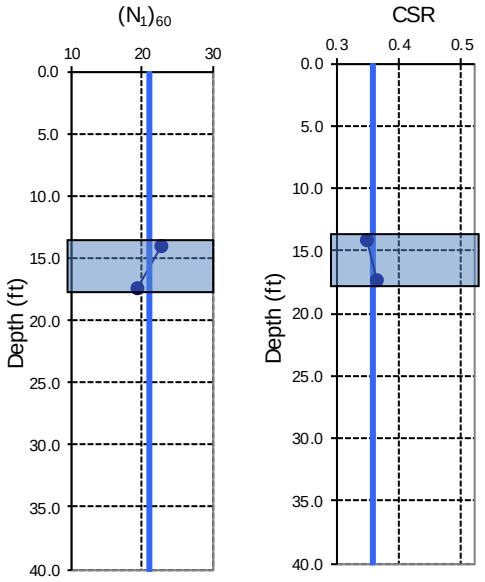
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	18.5 ± 16.3	19.0	
Critical Depth Range	14.1 - 17.4	15.7		% PI			
Depth to GWT (ft)	10.2 ± 0.3	10.2					
σ_v (psf)	1866.8 ± 76.8	1796.1		N	16.0 ± 1.8	15.0	
σ_v' (psf)	1518.8 ± 51.3	1441.1		C_R	0.91	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.90 ± 0.068	0.95		C_B	1.00	1.00	
CSR	0.360 ± 0.077	0.382		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.18	1.17	
MSF		1.17		$(N_1)_{60}$	21.0 ± 2.3	20.3	
CSRN		0.307					



γ_m=115±3 pcf

γ_s=125±3 pcf

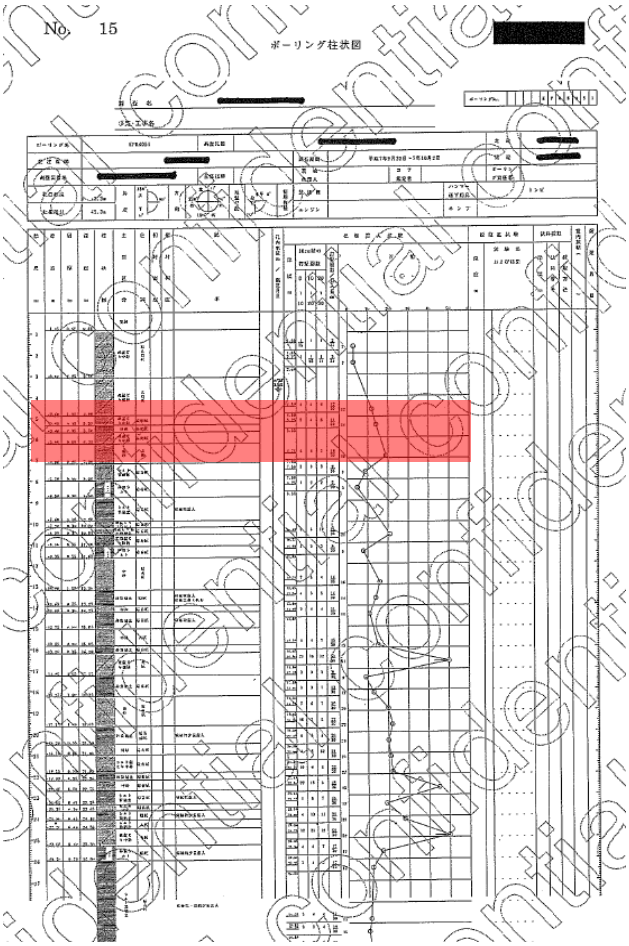


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.3	14.1	0.350	17.0	1.22	0.90	***22.7***	***7.0***
5.3	17.4	0.366	15.0	1.14	0.93	***19.4***	***30.0***
					Mean:	21.0	18.5
					St. Dev.	2.3	16.3

Case number:	173
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 15
References:	Kobe City Office, Construction Engineering Research Institute (1999)
Nature of Failure:	Liquefied
Comments:	

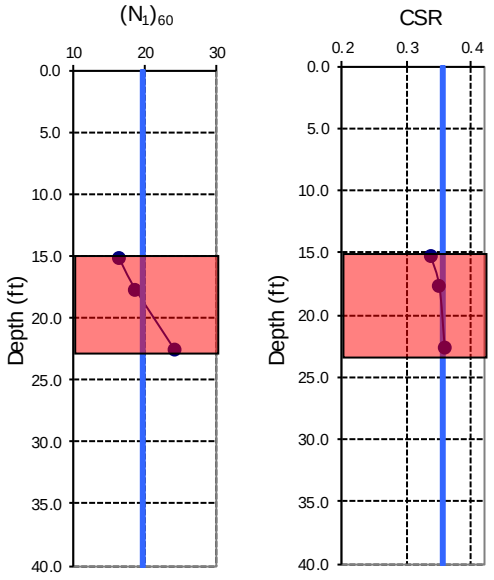
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	4.7 ± 4.5	5.0	
Critical Depth Range	15.3 - 22.6	18.7		% PI			
Depth to GWT (ft)	12.0 ± 0.3	12.1					
σ_v (psf)	2069.0 ± 159.5	2130.3		N	15.0 ± 3.1	15.1	
σ_v' (psf)	1633.9 ± 88.2	1712.6		C_R	0.94	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.86 ± 0.079	0.93		C_B	1.00	1.00	
CSR	0.356 ± 0.079	0.375		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.14	1.10	
MSF		1.17		$(N_1)_{60}$	19.6 ± 4.0	19.2	
CSRN		0.312					



γ_m=100±3 pcf

γ_s=125±3 pcf

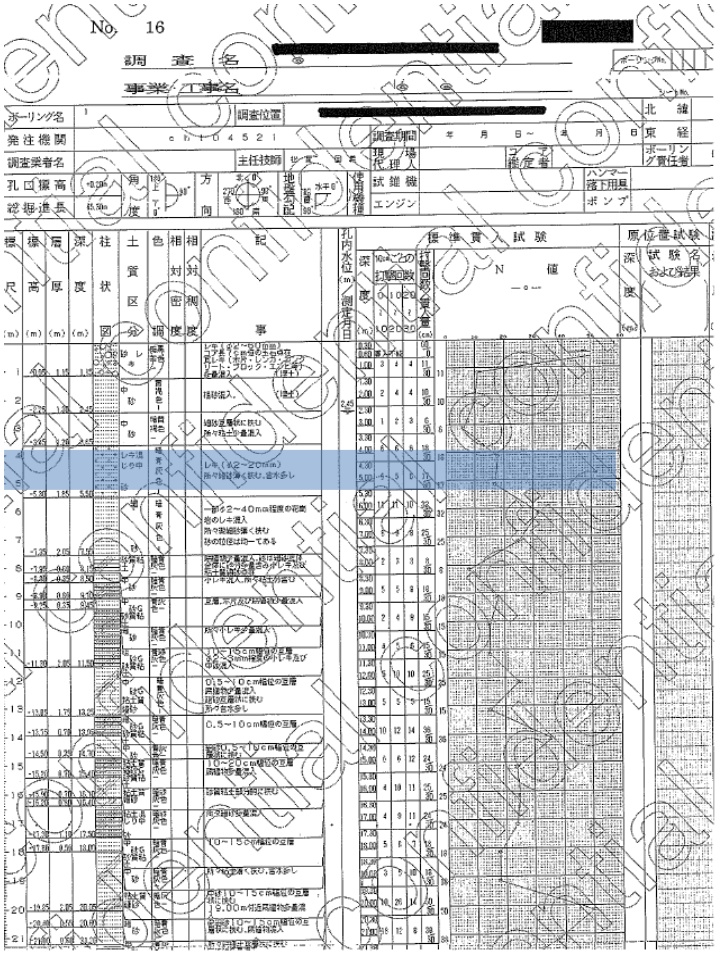


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.7	15.3	0.337	12.0	1.23	0.91	***16.3***	***5.0***
5.4	17.7	0.351	14.0	1.17	0.93	***18.5***	***0.0***
6.9	22.6	0.360	19.0	1.07	0.97	***24.0***	***9.0***
					Mean:	19.6	4.7
					St. Dev.	4.0	4.5

Case number:	174
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 16
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

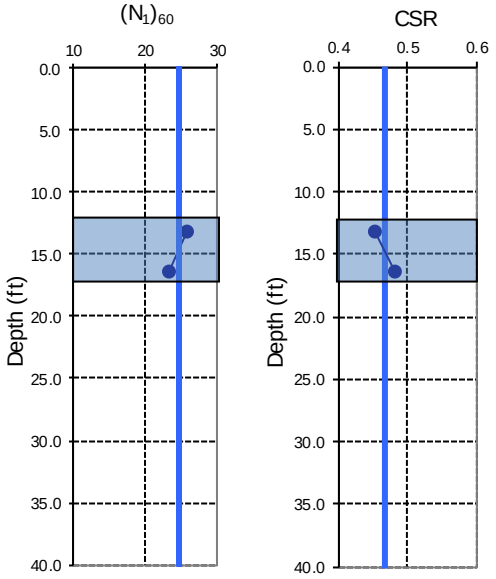
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	13.1 - 16.4	14.8		% PI			
Depth to GWT (ft)	8.0 ± 0.3	8.2					
σ_v (psf)	1724.9 ± 75.4	1670.8		N	17.6 ± 1.2	17.5	
σ_v' (psf)	1305.2 ± 48.6	1253.1		C_R	0.90	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.95 ± 0.064	0.95		C_B	1.00	1.00	
CSR	0.489 ± 0.104	0.495		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.27	1.23	
MSF		1.17		$(N_1)_{60}$	24.6 ± 1.7	25.0	
CSRN		0.390					



γ_m=110±3 pcf

γ_s=125±3 pcf



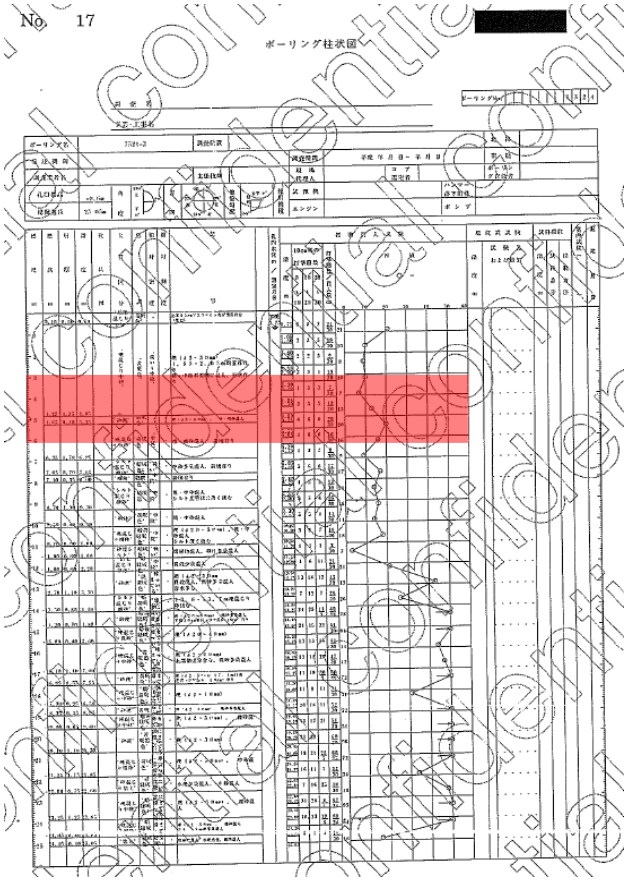
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.472	18.0	1.33	0.89	*25.8*	*5.0*
5.0	16.4	0.502	17.0	1.23	0.92	*23.4*	*5.0*
					Mean:	24.6	5.0
					St. Dev.	1.7	2.0

assigned

Case number:	175
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 17
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

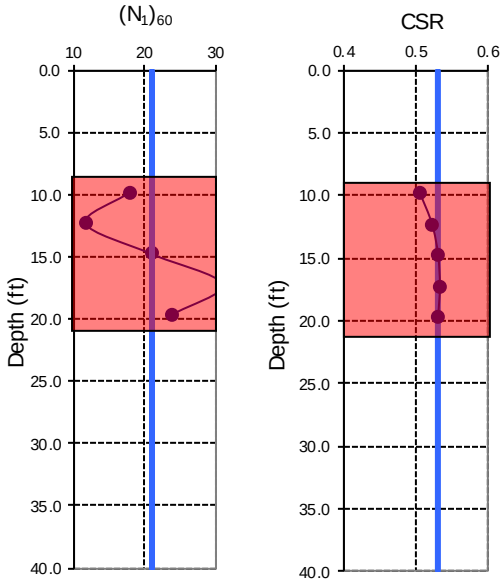
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	9.8 - 19.7	14.8		% PI			
Depth to GWT (ft)	2.5 ± 0.3	2.6					
σ_v (psf)	1747.0 ± 200.4	1670.8		N	13.1 ± 4.4	12.6	
σ_v' (psf)	979.3 ± 102.9	898.1		C_R	0.90	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.95 ± 0.064	0.95		C_B	1.00	1.00	
CSR	0.551 ± 0.117	0.574		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.47	1.45	
MSF		1.17		$(N_1)_{60}$	21.2 ± 7.1	21.1	
CSRN		0.446					



γ_m=110±3 pcf

γ_s=120±3 pcf



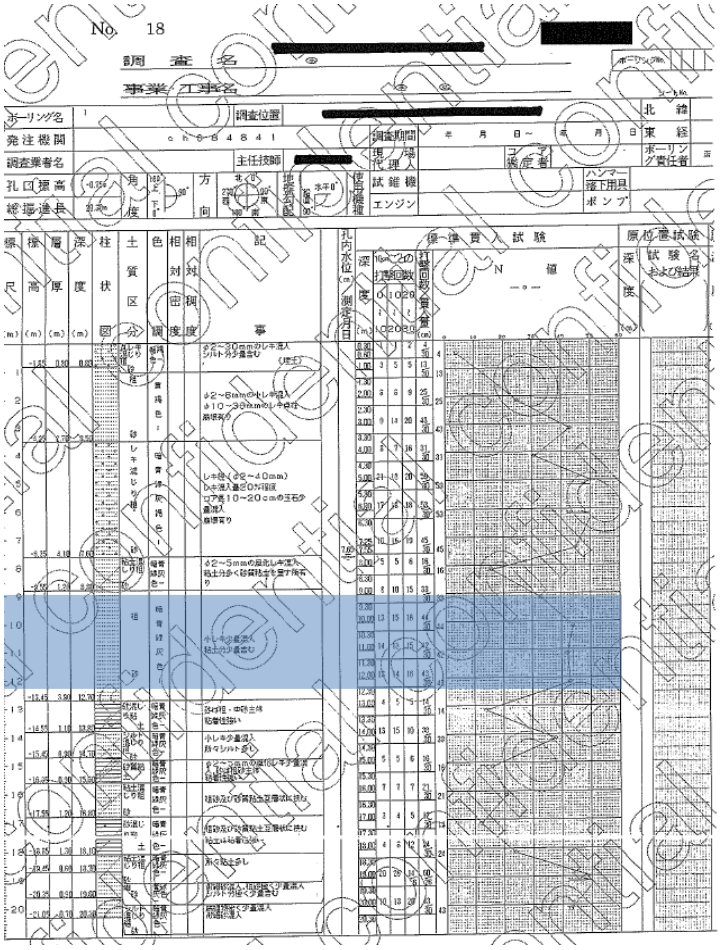
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.526	10.0	1.74	0.85	***18.0***	***5.0***
3.8	12.3	0.543	7.0	1.59	0.88	***11.9***	***5.0***
4.5	14.8	0.551	13.0	1.47	0.90	***21.0***	***5.0***
5.3	17.2	0.554	20.0	1.37	0.93	***31.1***	***5.0***
6.0	19.7	0.551	16.0	1.29	0.95	***24.0***	***5.0***
					Mean:	21.2	5.0
					St. Dev.	7.1	2.0

assigned

Case number:	176
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 18
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

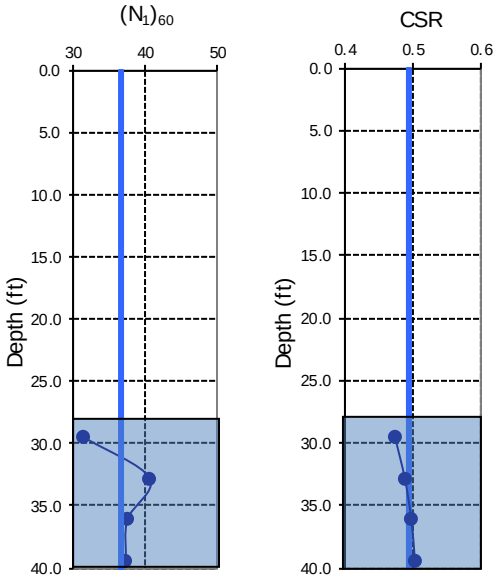
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	29.5 - 39.4	34.4		% PI			
Depth to GWT (ft)	25.1 ± 0.3	25.3					
σ_v (psf)	4399.6 ± 235.6	4156.2		N	40.5 ± 4.2	40.5	
σ_v' (psf)	3816.1 ± 144.5	3571.4		C_R	1.00	1.00	
a_{max} (g)	0.700 ± 0.140	0.70		C_S	1.00	1.00	
r_d	0.94 ± 0.131	0.85		C_B	1.00	1.00	
CSR	0.493 ± 0.121	0.448		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.74	0.86	
MSF		1.17		$(N_1)_{60}$	36.7 ± 3.8	42.6	
CSRN		0.452					



γ_m=125±3 pcf

γ_s=135±3 pcf

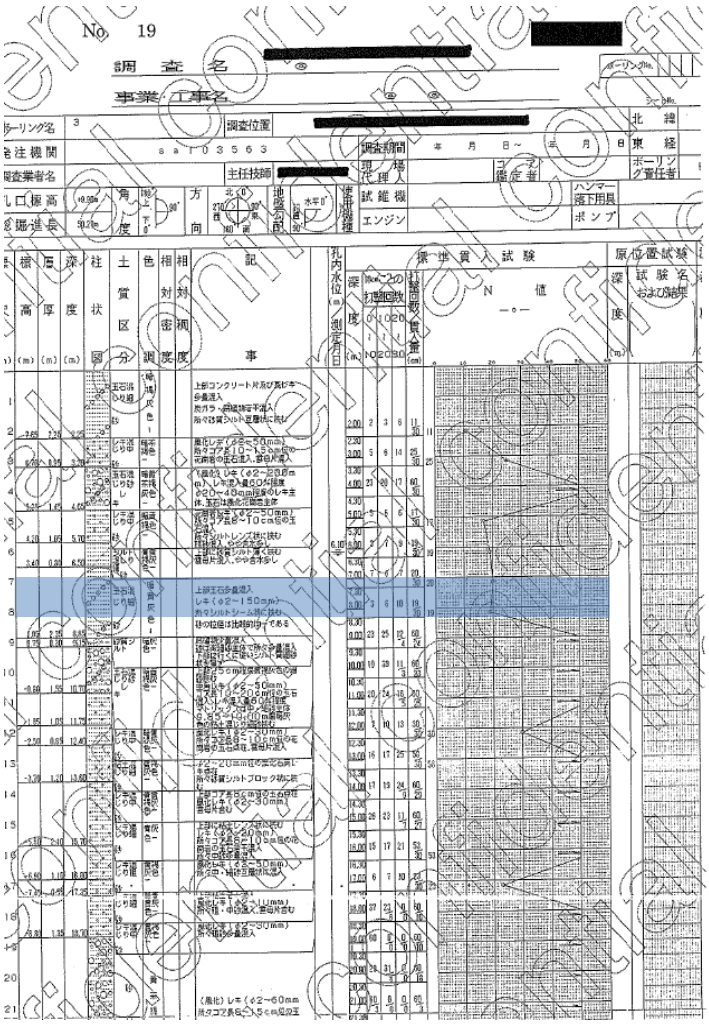


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
9.0	29.5	0.473	33.0	0.78	1.00	*31.4*	*0.0*
10.0	32.8	0.488	44.0	0.76	1.00	*40.5*	*0.0*
11.0	36.1	0.498	42.0	0.73	1.00	*37.5*	*0.0*
12.0	39.4	0.503	43.0	0.71	1.00	*37.3*	*0.0*
					Mean:	36.7	0.0
					St. Dev.	3.8	2.0

Case number:	177
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 19
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

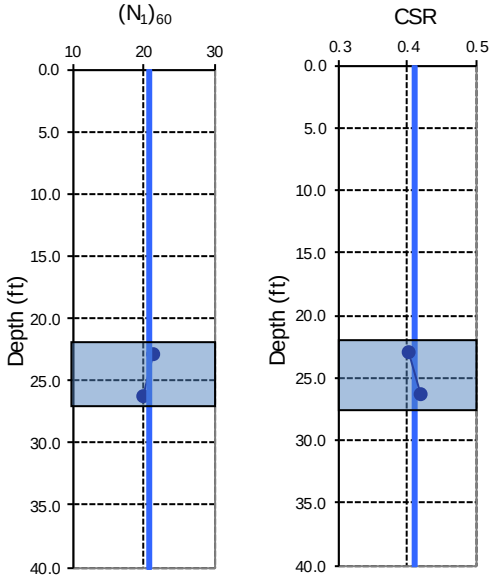
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	10.0 ± 2.0	10.0	
Critical Depth Range	23.0 - 26.2	24.6		% PI			
Depth to GWT (ft)	20.0 ± 0.3	20.0					
σ_v (psf)	2975.7 ± 92.0	2861.3		N	19.5 ± 0.9	20.0	
σ_v' (psf)	2689.1 ± 72.5	2589.8		C_R	0.99	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.95 ± 0.099	0.90		C_B	1.00	1.00	
CSR	0.411 ± 0.093	0.391		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.89	0.92	
MSF		1.17		$(N_1)_{60}$	20.8 ± 1.0	21.3	
CSRN		0.344					



γ_m=120±3 pcf

γ_s=125±3 pcf



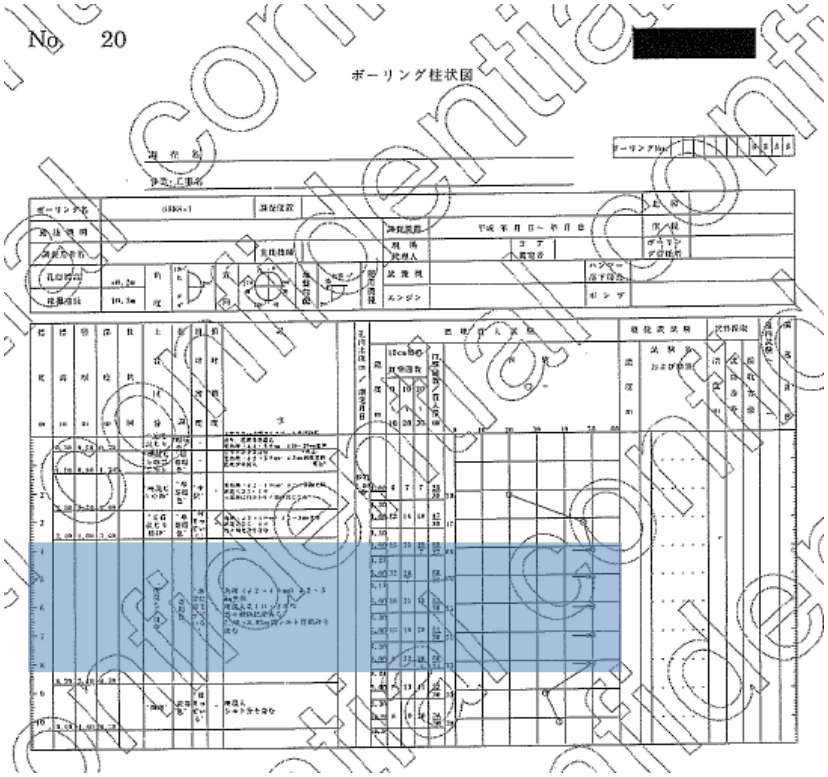
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
7.0	23.0	0.401	20.0	0.90	0.98	***21.5***	***10.0***
8.0	26.2	0.420	19.0	0.87	1.00	***20.1***	***10.0***
					Mean:	20.8	10.0
					St. Dev.	1.0	2.0

assigned

Case number:	178
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 20
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

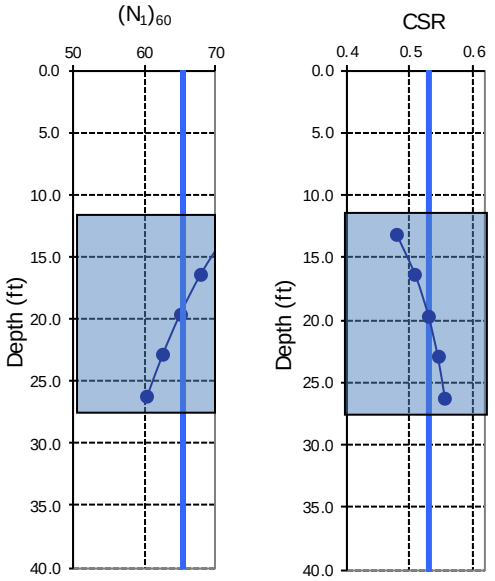
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	13.1 - 26.2	19.7		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	2493.4 ± 298.6	2380.9		N	50.4 ± 3.5	50.8	
σ_v' (psf)	1674.5 ± 165.2	1566.4		C_R	0.95	0.95	
a_{max} (g)	0.550 ± 0.110	0.55		C_S	1.00	1.00	
r_d	0.99 ± 0.082	0.93		C_B	1.00	1.00	
CSR	0.530 ± 0.115	0.505		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.12	1.08	
MSF		1.17		$(N_1)_{60}$	65.5 ± 4.5	63.7	
CSRN		0.396					



γ_m=110±3 pcf

γ_s=135±3 pcf

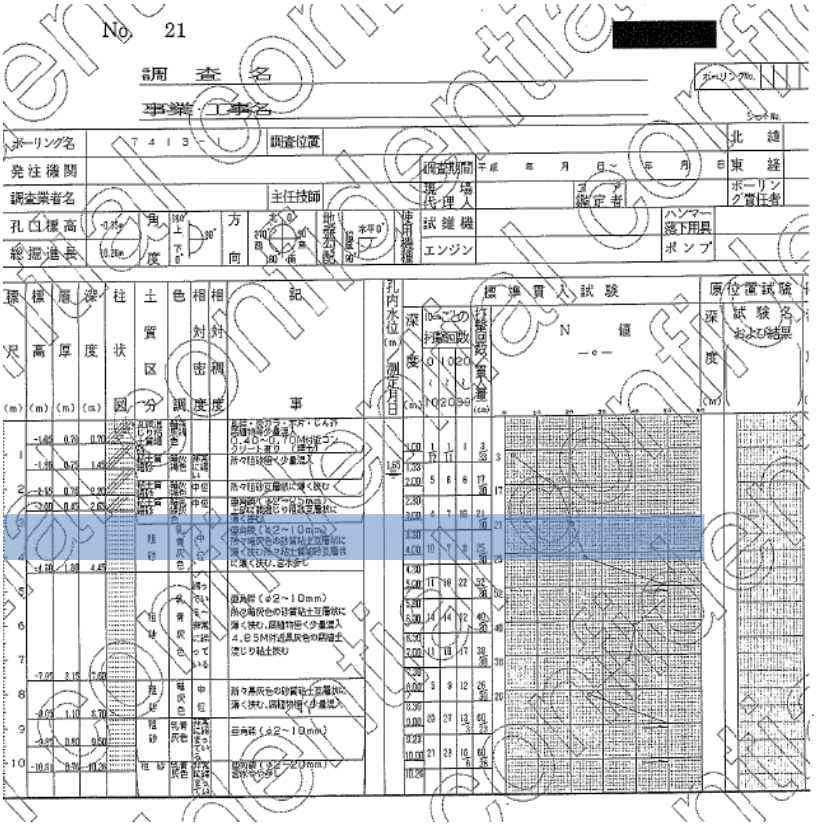


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.479	50.0	1.33	0.89	*71.7*	*0.0*
5.0	16.4	0.509	50.0	1.21	0.92	*68.0*	*0.0*
6.0	19.7	0.530	50.0	1.12	0.95	*65.0*	*0.0*
7.0	23.0	0.545	50.0	1.05	0.98	*62.5*	*0.0*
8.0	26.2	0.556	50.0	0.99	1.00	*60.3*	*0.0*
					Mean:	65.5	0.0
					St. Dev.	4.5	2.0

Case number:	179
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 21
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

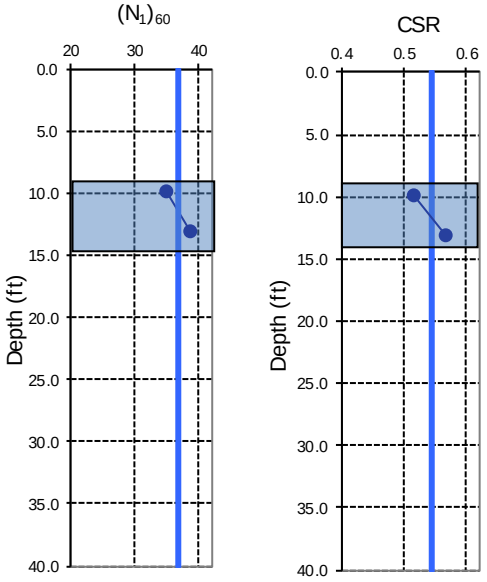
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	5.4 ± 0.3	5.6					
σ_v (psf)	1300.0 ± 73.0	1294.9		N	23.0 ± 1.8	24.4	
σ_v' (psf)	921.3 ± 43.5	919.0		C_R	0.87	0.85	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.99 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.545 ± 0.114	0.531		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.52	1.32	
MSF		1.17		$(N_1)_{60}$	36.8 ± 2.8	33.5	
CSRN		0.412					



γ_m=100±3 pcf

γ_s=125±3 pcf

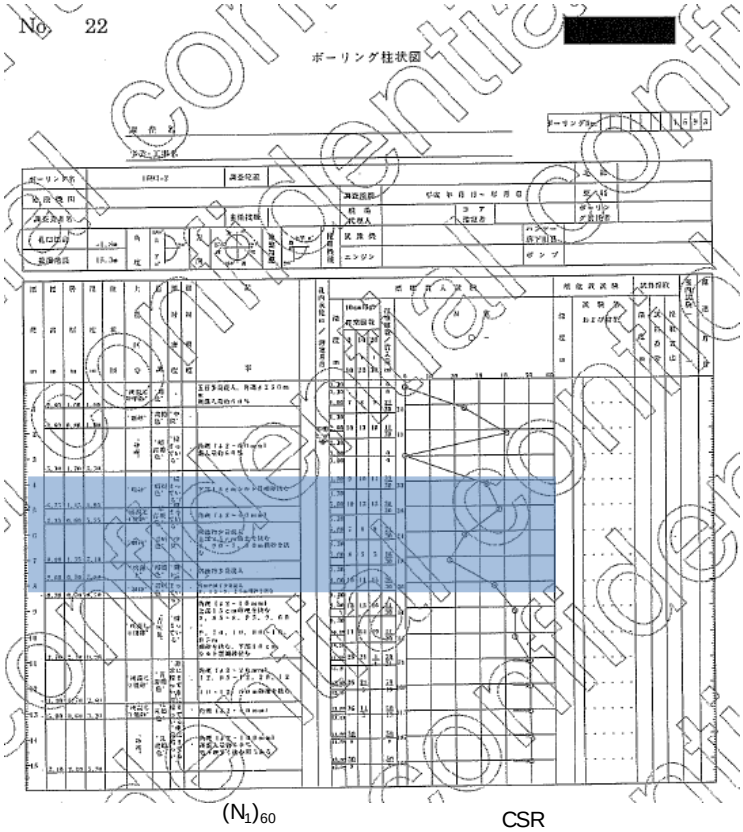


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.518	21.0	1.61	0.85	*34.8*	*0.0*
4.0	13.1	0.567	25.0	1.44	0.89	*38.8*	*0.0*
					Mean:	36.8	0.0
					St. Dev.	2.8	2.0

Case number:	180
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 22
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

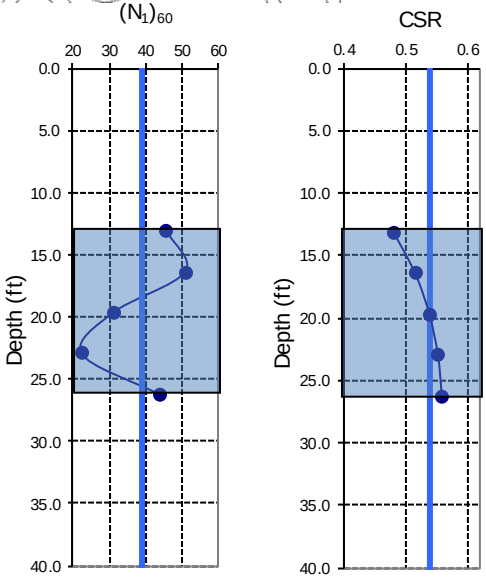
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	6.0 ± 5.5	6.0	
Critical Depth Range	13.1 - 26.2	19.7		% PI			
Depth to GWT (ft)	7.9 ± 0.3	7.9					
σ_v (psf)	2421.3 ± 276.7	2380.9		N	30.0 ± 9.0	30.8	
σ_v' (psf)	1684.3 ± 144.4	1649.9		C_R	0.95	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.96 ± 0.082	0.93		C_B	1.00	1.00	
CSR	0.539 ± 0.119	0.524		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.12	1.08	
MSF		1.17		$(N_1)_{60}$	38.9 ± 11.7	38.6	
CSRN		0.416					



γ_m=120±3 pcf

γ_s=125±3 pcf

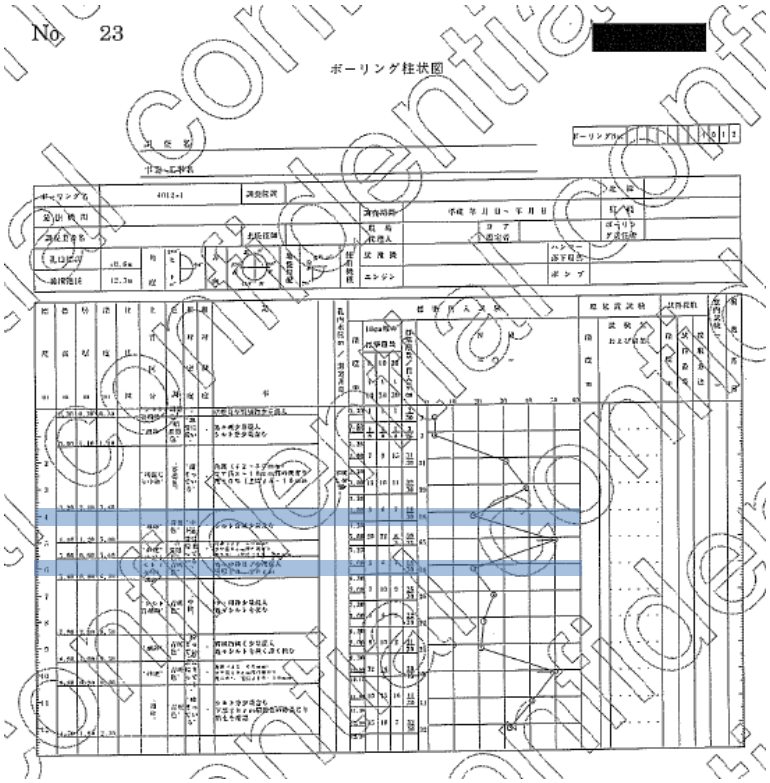


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.482	33.0	1.29	0.89	*45.9*	*0.0*
5.0	16.4	0.516	38.0	1.20	0.92	*50.9*	*0.0*
6.0	19.7	0.539	24.0	1.12	0.95	*31.1*	*10.0*
7.0	23.0	0.553	18.0	1.06	0.98	*22.6*	*10.0*
8.0	26.2	0.559	36.0	1.01	1.00	*44.0*	*10.0*
					Mean:	38.9	6.0
					St. Dev.	11.7	5.5

Case number:	181
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 23
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

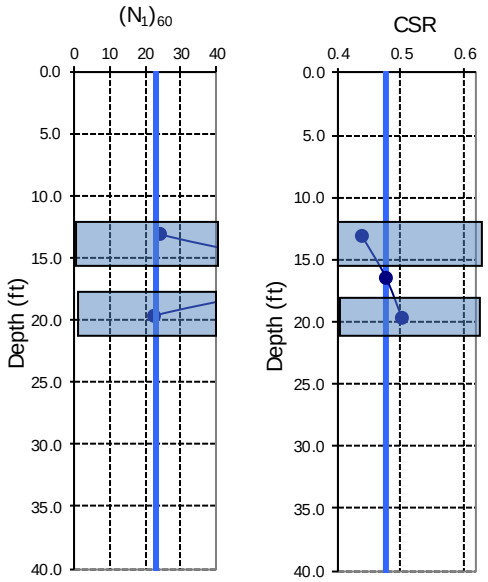
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	10.0 ± 2.0	10.0	
Critical Depth Range	13.1 - 19.7	16.4		% PI			
Depth to GWT (ft)	9.8 ± 0.3	9.8					
σ_v (psf)	2001.3 ± 141.2	1921.5		N	18.0 ± 0.8	18.1	
σ_v' (psf)	1591.9 ± 79.0	1503.8		C_R	0.92	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.97 ± 0.070	0.94		C_B	1.00	1.00	
CSR	0.477 ± 0.102	0.468		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.15	1.15	
MSF		1.17		$(N_1)_{60}$	23.3 ± 1.0	24.0	
CSRN		0.379					



γ_m=120±3 pcf

γ_s=125±3 pcf

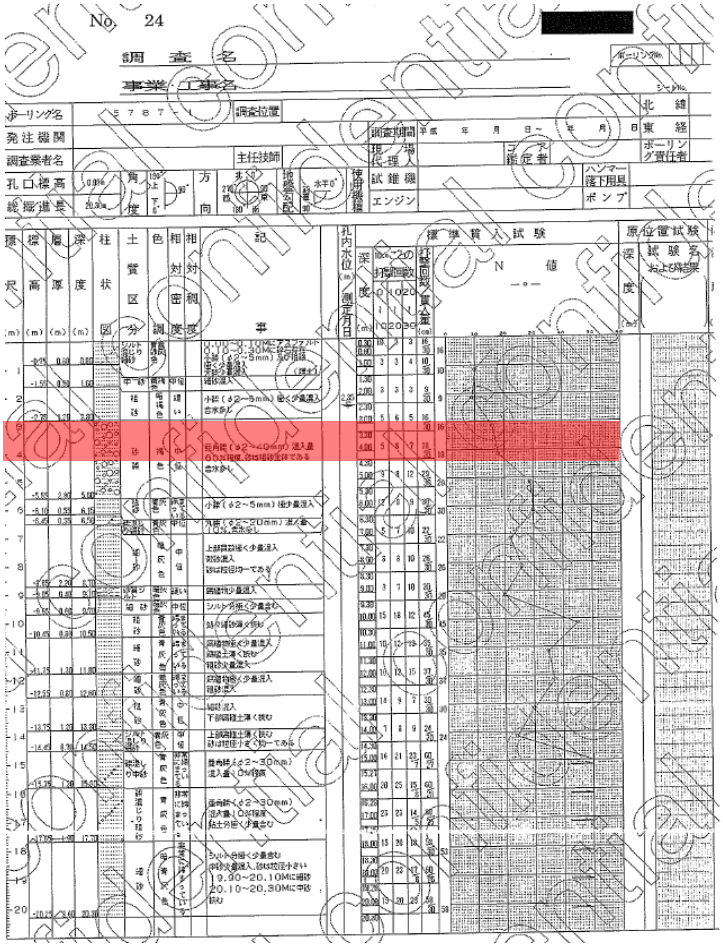


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.440	18.0	1.24	0.89	***24.0***	***10.0***
5.0	16.4	0.477	50.0	1.15	0.92	64.6	0.0
6.0	19.7	0.503	18.0	1.09	0.95	***22.6***	***10.0***
					Mean:	23.3	10.0
					St. Dev.	1.0	2.0

Case number:	182
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 24
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

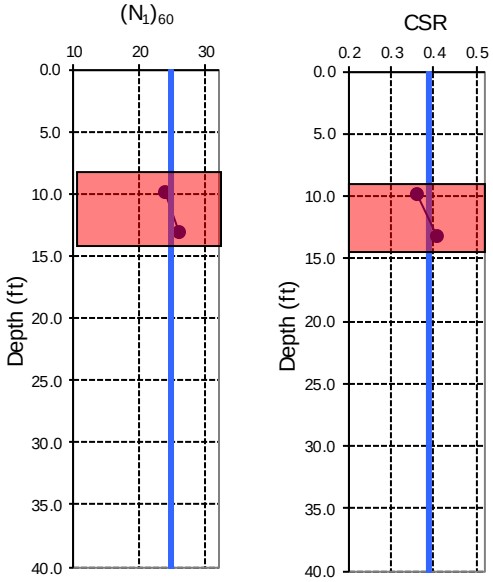
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	7.7 ± 0.3	7.9					
σ_v (psf)	1339.4 ± 70.5	1315.8		N	17.0 ± 1.0	18.0	
σ_v' (psf)	1104.0 ± 44.2	1065.2		C_R	0.87	0.85	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.98 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.388 ± 0.081	0.383		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.38	1.32	
MSF		1.17		$(N_1)_{60}$	24.8 ± 1.4	24.6	
CSRN		0.297					



γ_m=115±3 pcf

γ_s=120±3 pcf

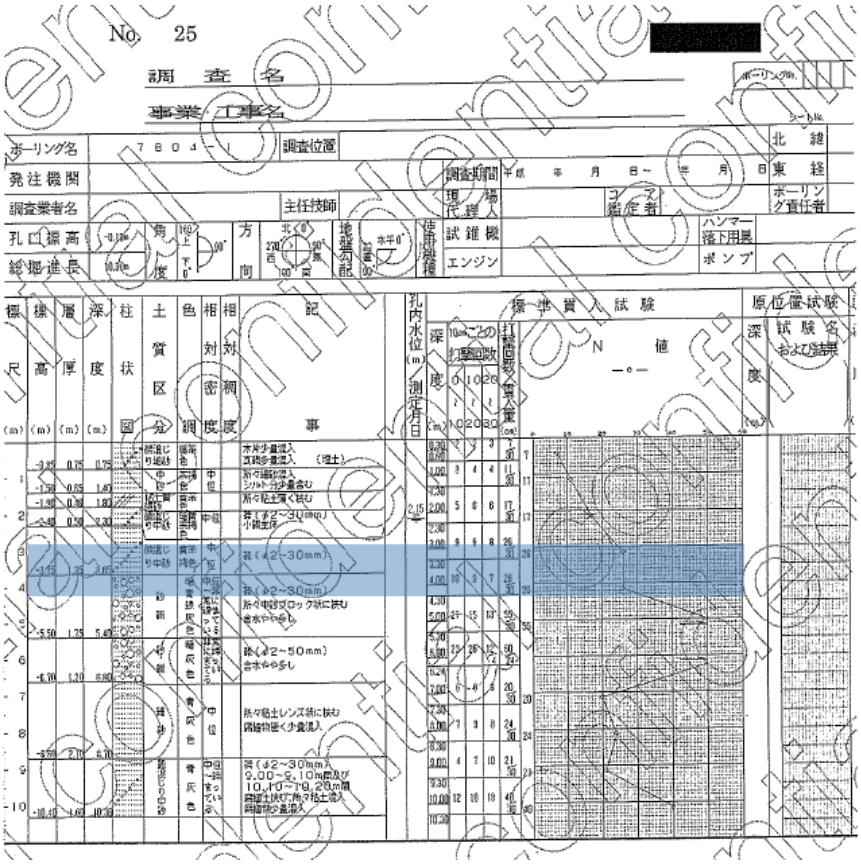


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.363	16.0	1.45	0.85	***23.9***	***0.0***
4.0	13.1	0.408	18.0	1.33	0.89	***25.8***	***0.0***
					Mean:	24.8	0.0
					St. Dev.	1.4	2.0

Case number:	183
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 25
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Nonliquefied
Comments:	

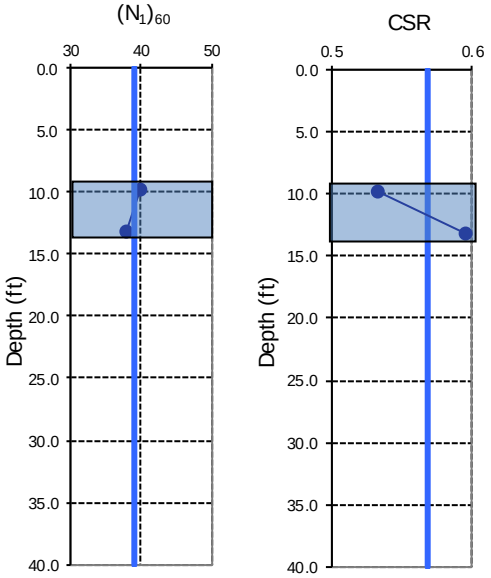
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	2.5 ± 3.5	3.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	7.1 ± 0.3	7.2					
σ_v (psf)	1329.6 ± 72.9	1336.7		N	26.1 ± 0.9	27.5	
σ_v' (psf)	1053.2 ± 44.7	1044.3		C_R	0.87	0.85	
a_{max} (g)	0.700 ± 0.140	0.70		C_S	1.00	1.00	
r_d	0.99 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.568 ± 0.118	0.555		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.42	1.25	
MSF		1.17		$(N_1)_{60}$	39.0 ± 1.4	35.8	
CSRN		0.431					



γ_m=110±3 pcf

γ_s=125±3 pcf

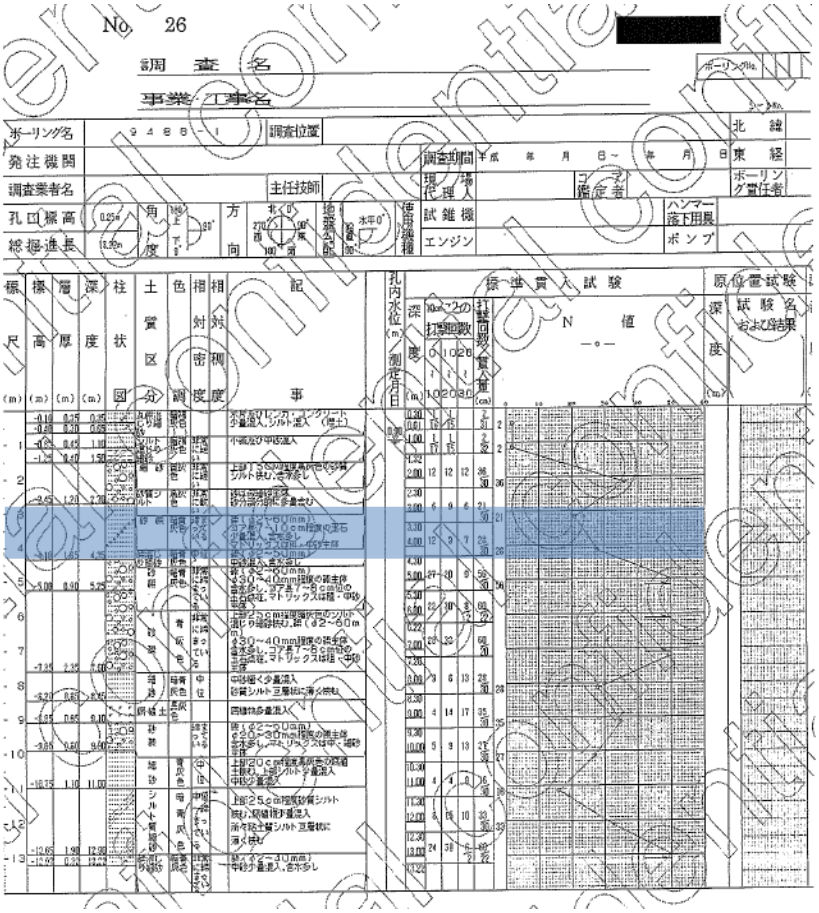


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.534	26.0	1.49	0.85	***40.0***	***5.0***
4.0	13.1	0.596	26.0	1.35	0.89	***38.0***	***0.0***
					Mean:	39.0	2.5
					St. Dev.	1.4	3.5

Case number:	184
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 26
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

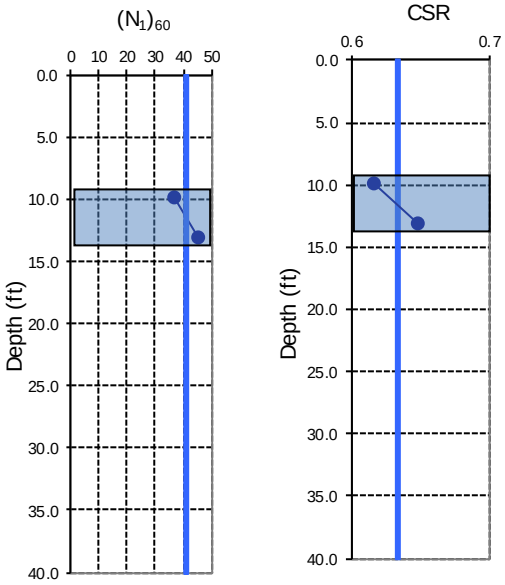
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	3.0 ± 0.3	3.0					
σ_v (psf)	1361.5 ± 73.9	1315.8		N	24.5 ± 3.6	26.0	
σ_v' (psf)	829.3 ± 45.1	772.8		C_R	0.87	0.85	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.99 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.634 ± 0.132	0.633		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.60	1.37	
MSF		1.17		$(N_1)_{60}$	41.2 ± 6.1	37.0	
CSRN		0.491					



γ_m=100±3 pcf

γ_s=125±3 pcf

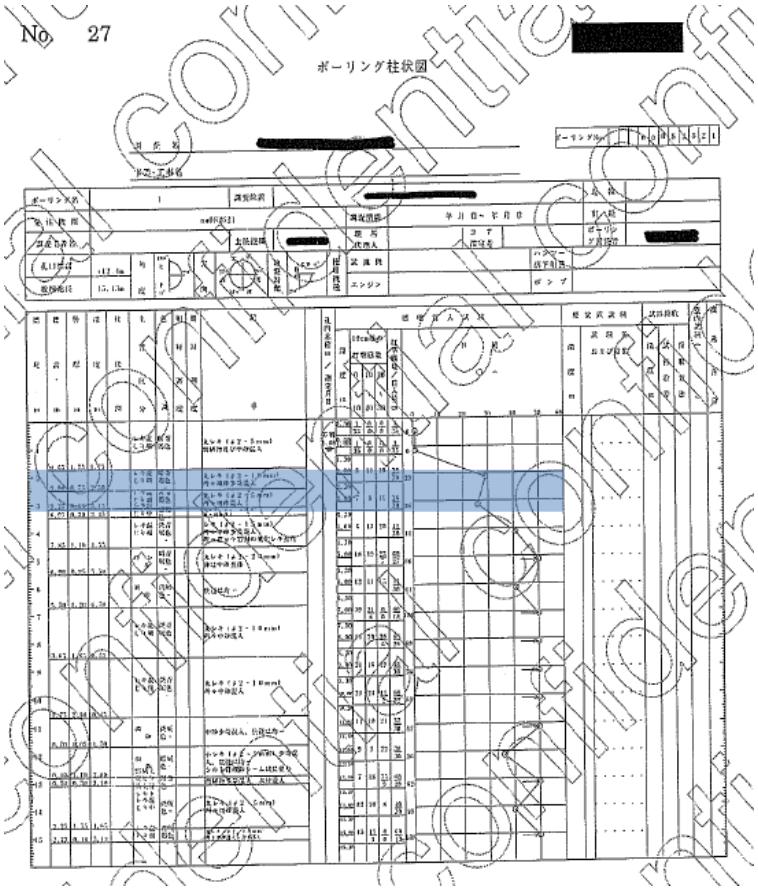


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.616	21.0	1.71	0.85	***36.9***	***0.0***
4.0	13.1	0.648	28.0	1.51	0.89	***45.5***	***0.0***
					Mean:	41.2	0.0
					St. Dev.	6.1	2.0

Case number:	185
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 27
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

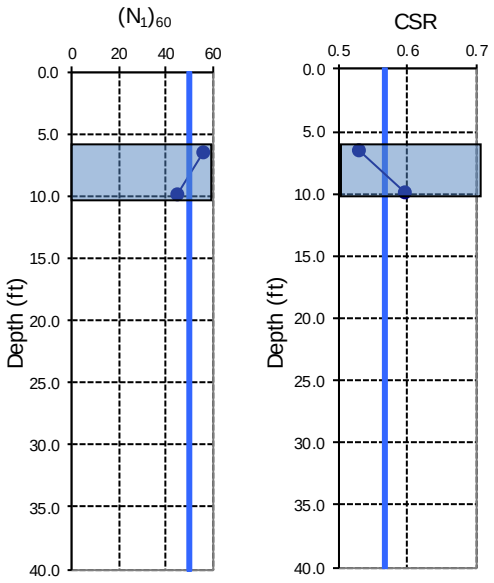
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	10.0 ± 2.0	10.0	
Critical Depth Range	6.6 - 9.8	8.2		% PI			
Depth to GWT (ft)	3.4 ± 0.3	3.6					
σ_v (psf)	939.1 ± 71.0	898.1		N	27.7 ± 4.1	27.6	
σ_v' (psf)	642.3 ± 40.1	605.7		C_R	0.82	0.85	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	1.00 ± 0.039	0.98		C_B	1.00	1.00	
CSR	0.569 ± 0.117	0.568		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.82	1.43	
MSF		1.17		$(N_1)_{60}$	50.4 ± 7.5	40.8	
CSRN		0.441					



γ_m=100±3 pcf

γ_s=125±3 pcf

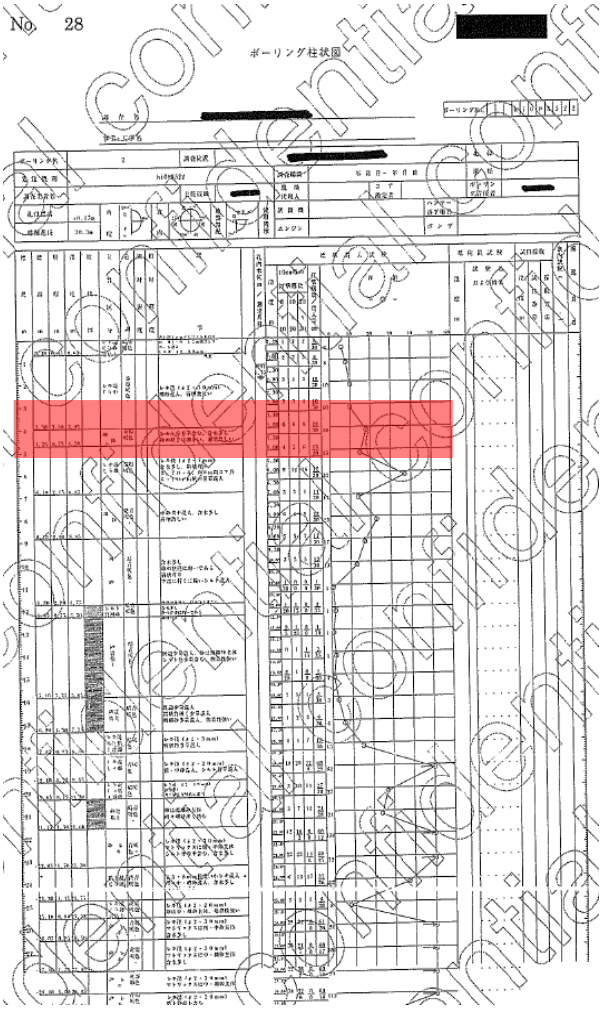


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
2.0	6.6	0.530	29.0	1.98	0.80	*55.7*	*10.0*
3.0	9.8	0.597	26.0	1.69	0.85	*45.1*	*10.0*
					Mean:	50.4	10.0
					St. Dev.	7.5	2.0

Case number:	186
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 28
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

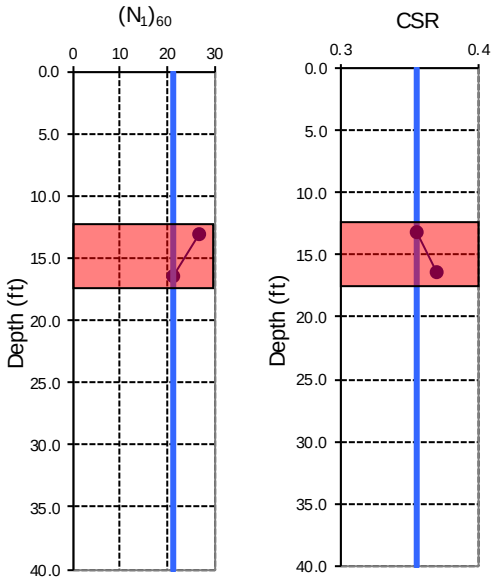
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	8.0 ± 2.0	8.0	
Critical Depth Range	9.8 - 16.4	11.5		% PI			
Depth to GWT (ft)	5.7 ± 0.3	5.9					
σ_v (psf)	1554.3 ± 139.6	1294.9		N	14.3 ± 3.7	14.3	
σ_v' (psf)	1093.7 ± 75.3	919.0		C_R	0.89	0.85	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.96 ± 0.058	0.97		C_B	1.00	1.00	
CSR	0.355 ± 0.075	0.348		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.39	1.42	
MSF		1.17		$(N_1)_{60}$	21.5 ± 5.6	21.1	
CSRN		0.270					



γ_m=110±3 pcf

γ_s=125±3 pcf

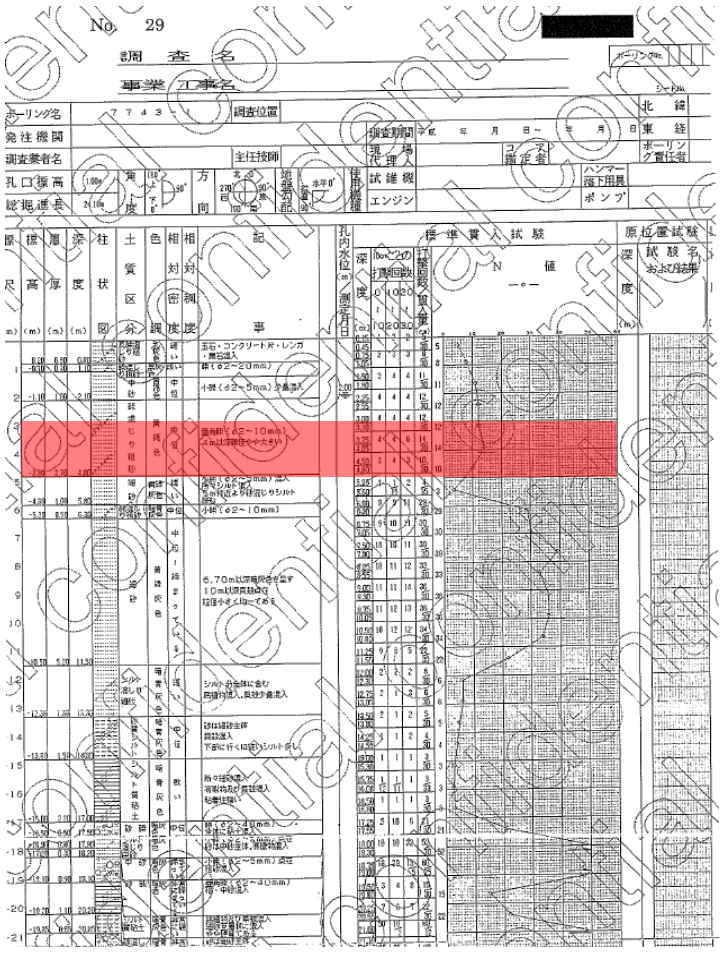


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC	
3.0	9.8	0.327	10.0	1.54	0.85	***15.9***	***4.0***	interpreted
4.0	13.1	0.355	18.0	1.39	0.89	***27.0***	***10.0***	
5.0	16.4	0.371	15.0	1.28	0.92	***21.5***	***10.0***	
					Mean:	21.5	8.0	
					St. Dev.	5.6	2.0	

Case number:	187
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 29
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

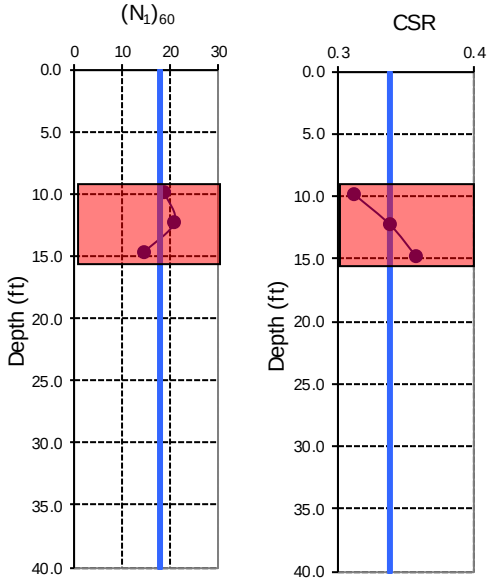
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 14.8	12.5		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	1439.5 ± 105.9	1399.3		N	12.1 ± 2.2	12.4	
σ_v' (psf)	1081.2 ± 59.4	1023.4		C_R	0.88	0.85	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.98 ± 0.055	0.96		C_B	1.00	1.00	
CSR	0.339 ± 0.071	0.337		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.40	1.39	
MSF		1.17		$(N_1)_{60}$	18.0 ± 3.3	17.9	
CSRN		0.264					



γ_m=110±3 pcf

γ_s=125±3 pcf



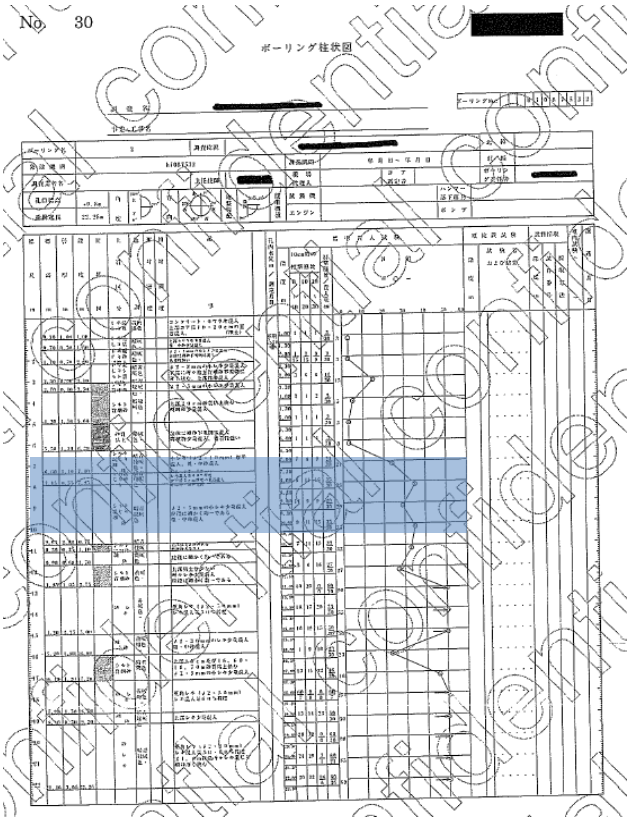
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.313	12.0	1.51	0.85	***18.7***	***0.0***
3.8	12.3	0.339	14.0	1.40	0.88	***20.9***	***0.0***
4.5	14.8	0.357	10.0	1.31	0.90	***14.4***	***0.0***
					Mean:	18.0	0.0
					St. Dev.	3.3	2.0

assigned

Case number:	188
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 30
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-Liquefied
Comments:	

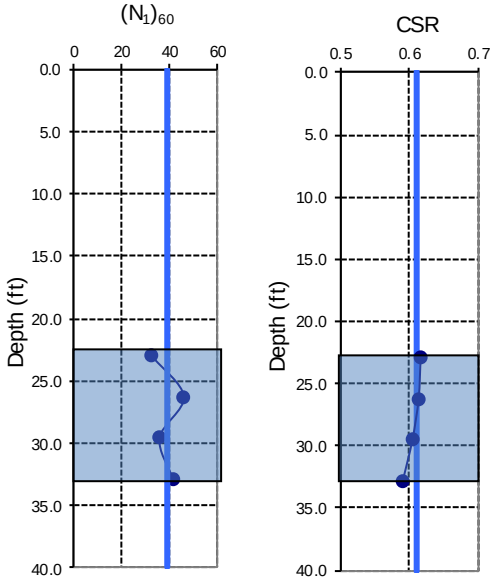
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	10.0 ± 2.0	10.0	
Critical Depth Range	23.0 - 32.8	27.9		% PI			
Depth to GWT (ft)	4.9 ± 0.3	4.9					
σ_v (psf)	3362.9 ± 217.0	3049.3		N	30.6 ± 4.6	30.5	
σ_v' (psf)	1929.8 ± 125.0	1629.1		C_R	1.00	1.00	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.90 ± 0.110	0.88		C_B	1.00	1.00	
CSR	0.611 ± 0.144	0.649		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.05	1.08	
MSF		1.17		$(N_1)_{60}$	39.0 ± 5.8	40.1	
CSRN		0.514					



γ_m=100±3 pcf

γ_s=125±3 pcf



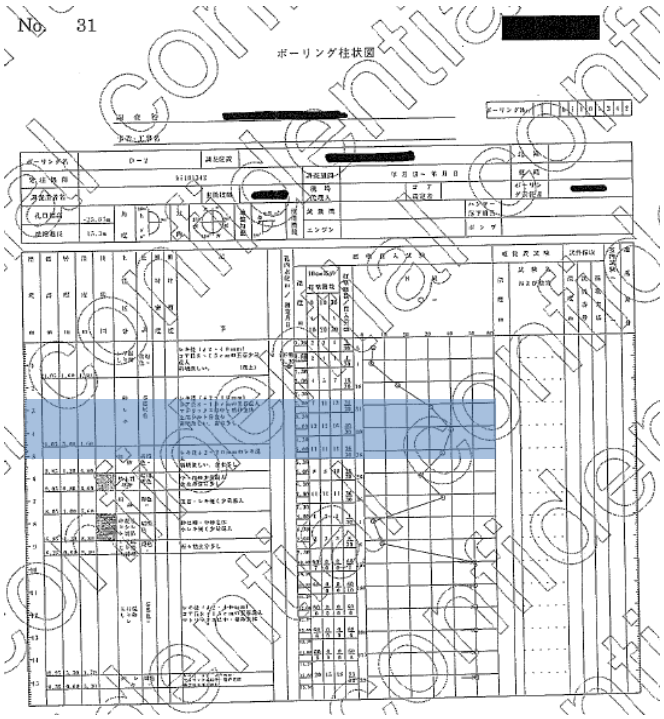
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
7.0	23.0	0.618	24.0	1.14	0.98	*32.6*	*10.0*
8.0	26.2	0.615	35.0	1.08	1.00	*45.8*	*10.0*
9.0	29.5	0.606	29.0	1.02	1.00	*36.0*	*10.0*
10.0	32.8	0.592	35.0	0.97	1.00	*41.4*	*10.0*
					Mean:	39.0	10.0
					St. Dev.	5.8	2.0

assigned

Case number:	189
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 31
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

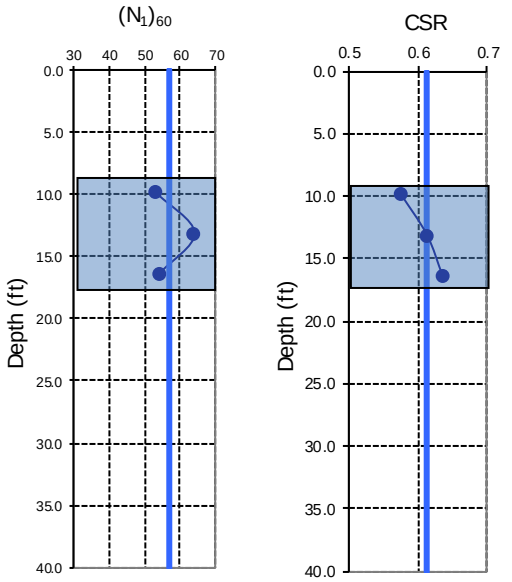
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 16.4	13.1		% PI			
Depth to GWT (ft)	3.9 ± 0.3	3.9					
σ_v (psf)	1542.0 ± 140.2	1524.6		N	35.8 ± 3.7	34.8	
σ_v' (psf)	968.8 ± 75.6	960.7		C_R	0.89	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.99 ± 0.058	0.96		C_B	1.00	1.00	
CSR	0.614 ± 0.129	0.599		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.48	1.23	
MSF		1.17		$(N_1)_{60}$	57.0 ± 5.9	49.7	
CSRN		0.465					



γ_m=100±3 pcf

γ_s=125±3 pcf



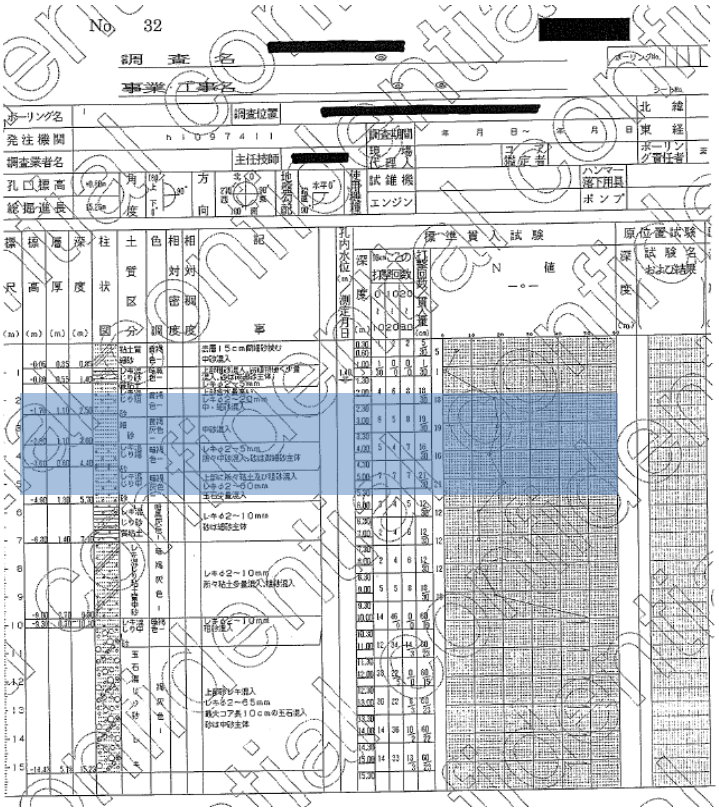
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.574	31.0	1.66	0.85	*53.2*	*0.0*
4.0	13.1	0.614	40.0	1.48	0.89	*63.8*	*0.0*
5.0	16.4	0.637	36.0	1.34	0.92	*54.2*	*0.0*
					Mean:	57.0	0.0
					St. Dev.	5.9	2.0

assigned

Case number:	190
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 32
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

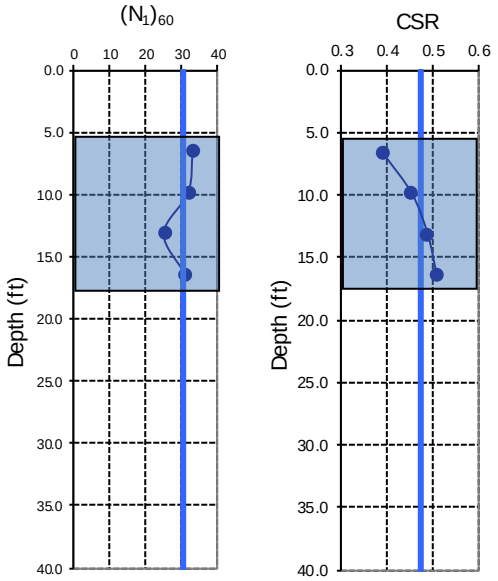
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	6.3 ± 4.8	6.0	
Critical Depth Range	6.6 - 16.4	11.5		% PI			
Depth to GWT (ft)	4.6 ± 0.3	4.6					
σ_v (psf)	1320.5 ± 206.7	1274.0		N	18.8 ± 2.2	20.1	
σ_v' (psf)	890.6 ± 106.2	856.3		C_R	0.87	0.85	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.98 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.474 ± 0.100	0.472		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.54	1.40	
MSF		1.17		$(N_1)_{60}$	30.5 ± 3.6	29.1	
CSRN		0.367					



γ_m=100±3 pcf

γ_s=125±3 pcf

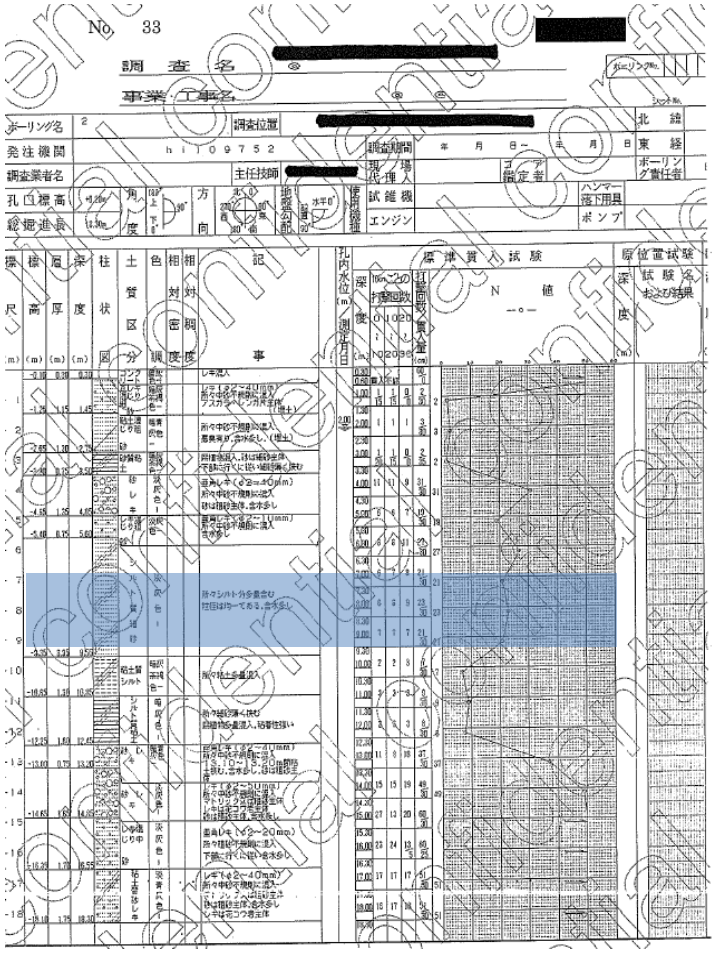


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
2.0	6.6	0.391	18.0	1.91	0.80	*33.3*	*0.0*
3.0	9.8	0.454	19.0	1.64	0.85	*32.1*	*10.0*
4.0	13.1	0.489	16.0	1.46	0.89	*25.2*	*10.0*
5.0	16.4	0.509	21.0	1.33	0.92	*31.3*	*5.0*
					Mean:	30.5	6.3
					St. Dev.	3.6	4.8

Case number:	191
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 33
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

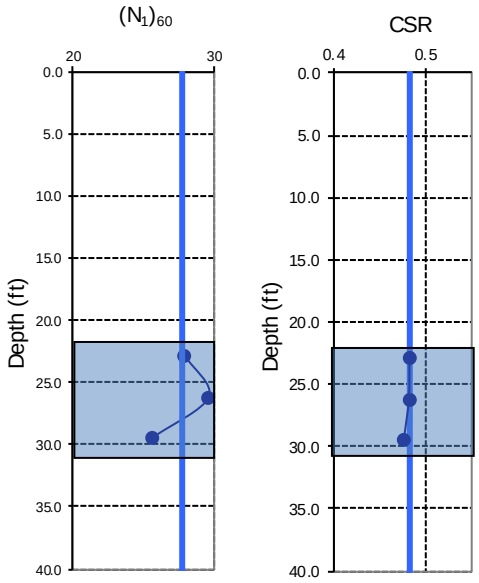
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	50.0 ± 2.0	50.0	
Critical Depth Range	23.0 - 29.5	26.2		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	3116.8 ± 150.4	2965.7		N	21.6 ± 1.6	21.3	
σ_v' (psf)	1888.5 ± 93.2	1733.5		C_R	1.00	1.0	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.90 ± 0.104	0.89		C_B	1.00	1.00	
CSR	0.483 ± 0.112	0.496		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.06	1.1	
MSF		1.17		$(N_1)_{60}$	27.8 ± 2.0	27.9	
CSRN		0.404					



γ_m=100±3 pcf

γ_s=125±3 pcf

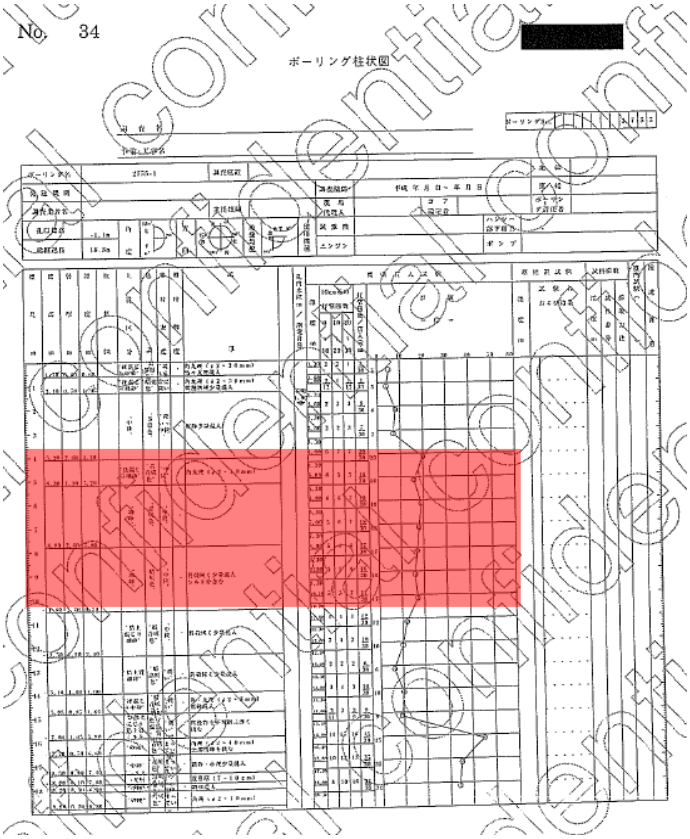


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
7.0	23.0	0.484	21.0	1.12	0.98	***28.0***	***50.0***
8.0	26.2	0.483	23.0	1.06	1.00	***29.6***	***50.0***
9.0	29.5	0.477	21.0	1.01	1.00	***25.7***	***50.0***
					Mean:	27.8	50.0
					St. Dev.	2.0	2.0

Case number:	192
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 34
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

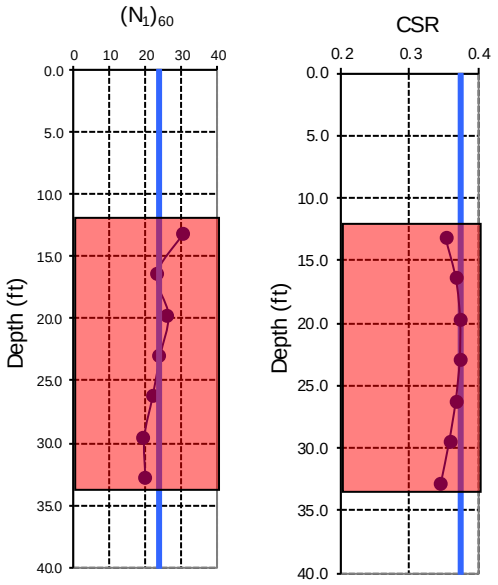
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	9.3 ± 1.9	9.0	
Critical Depth Range	13.1 - 32.8	23.0		% PI			
Depth to GWT (ft)	5.9 ± 0.3	5.9					
σ_v (psf)	2752.6 ± 413.7	2589.8		N	17.7 ± 2.9	18.3	
σ_v' (psf)	1688.1 ± 212.8	1524.6		C_R	0.98	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.88 ± 0.093	0.91		C_B	1.00	1.00	
CSR	0.374 ± 0.085	0.403		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.12	1.14	
MSF		1.17		$(N_1)_{60}$	23.5 ± 3.8	24.2	
CSRN		0.326					



γ_m=105±3 pcf

γ_s=125±3 pc

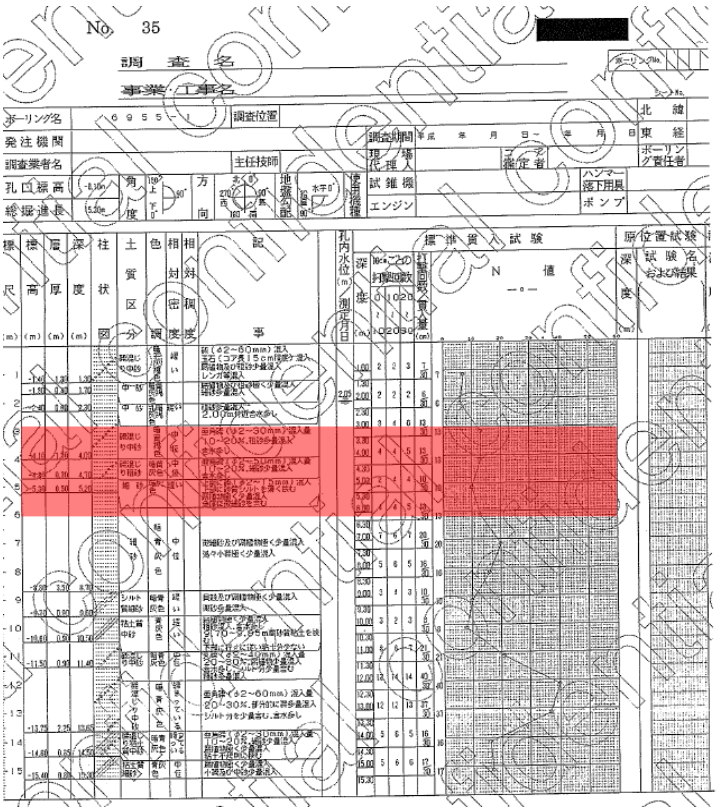


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.354	20.0	1.41	0.89	***30.3***	***5.0***
5.0	16.4	0.369	16.0	1.29	0.92	***23.1***	***10.0***
6.0	19.7	0.375	19.0	1.19	0.95	***26.3***	***10.0***
7.0	23.0	0.374	18.0	1.12	0.98	***24.0***	***10.0***
8.0	26.2	0.368	17.0	1.06	1.00	***21.9***	***10.0***
9.0	29.5	0.359	16.0	1.00	1.00	***19.5***	***10.0***
10.0	32.8	0.347	17.0	0.96	1.00	***19.8***	***10.0***
					Mean:	23.5	9.3
					St. Dev.	3.8	1.9

Case number:	193
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 35
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

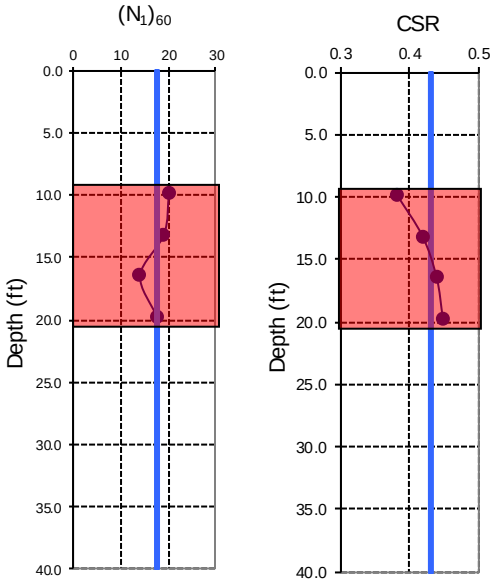
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	6.3 ± 4.8	6.0	
Critical Depth Range	9.8 - 19.7	14.8		% PI			
Depth to GWT (ft)	6.7 ± 0.3	6.9					
σ_v (psf)	1744.6 ± 207.5	1649.9		N	12.3 ± 1.9	12.3	
σ_v' (psf)	1243.0 ± 108.3	1148.7		C_R	0.90	0.95	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.95 ± 0.064	0.95		C_B	1.00	1.00	
CSR	0.432 ± 0.093	0.445		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.30	1.32	
MSF		1.17		$(N_1)_{60}$	17.7 ± 2.7	18.9	
CSRN		0.352					



γ_m=110±3 pcf

γ_s=125±3 pcf

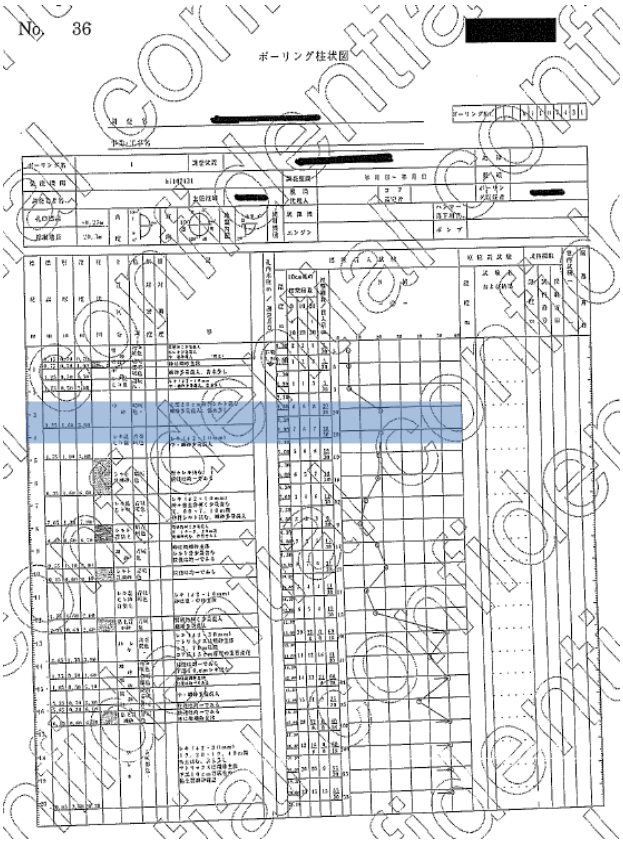


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.382	13.0	1.50	0.85	***20.1***	***5.0***
4.0	13.1	0.419	13.0	1.36	0.89	***19.1***	***0.0***
5.0	16.4	0.440	10.0	1.25	0.92	***14.1***	***10.0***
6.0	19.7	0.450	13.0	1.17	0.95	***17.6***	***10.0***
					Mean:	17.7	6.3
					St. Dev.	2.7	4.8

Case number:	194
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 36
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

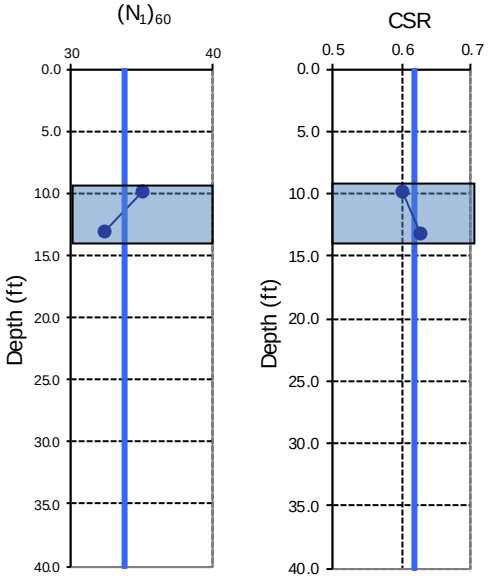
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	2.5 ± 3.5	3.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	3.1 ± 0.3	3.0					
σ_v (psf)	1358.3 ± 73.8	1274.0		N	20.1 ± 1.1	21.2	
σ_v' (psf)	834.2 ± 44.9	751.9		C_R	0.87	0.85	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	0.97 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.617 ± 0.129	0.639		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.59	1.44	
MSF		1.17		$(N_1)_{60}$	33.7 ± 1.8	31.6	
CSRN		0.496					



γ_m=100±3 pcf

γ_s=125±3 pcf

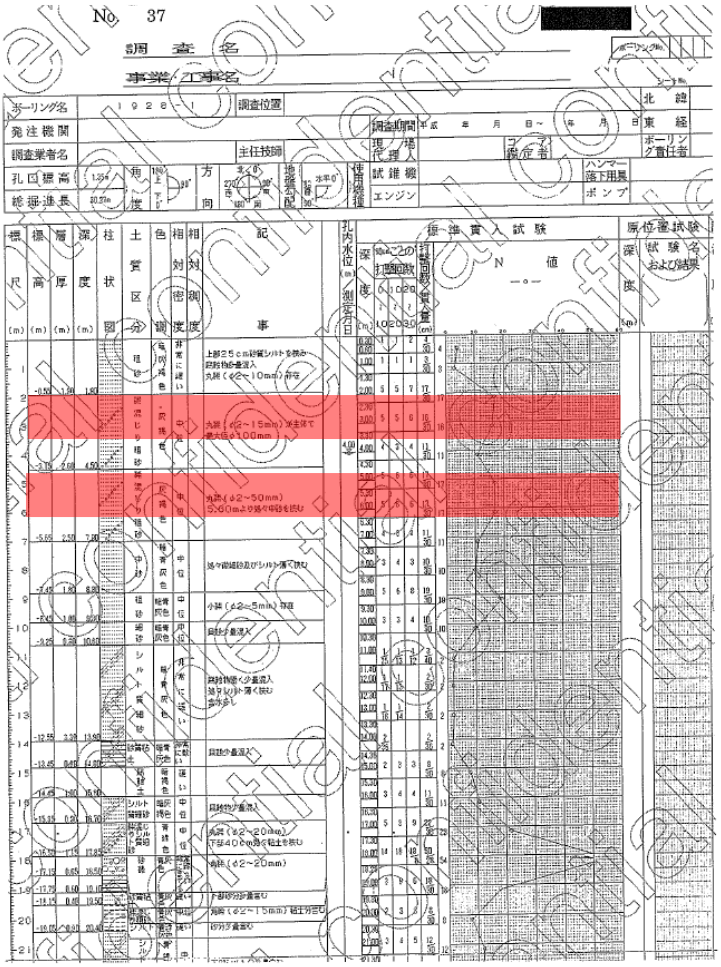


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.601	20.0	1.70	0.85	*35.0*	*5.0*
4.0	13.1	0.628	20.0	1.50	0.89	*32.4*	*0.0*
					Mean:	33.7	2.5
					St. Dev.	1.8	3.5

Case number:	195
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 37
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

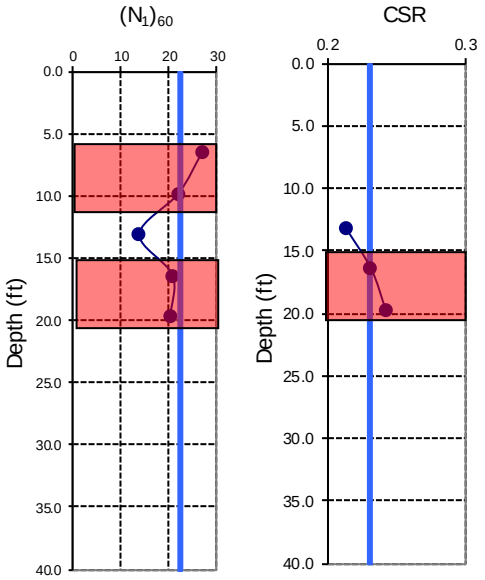
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	13.1 - 19.7	16.4		% PI			
Depth to GWT (ft)	13.1 ± 0.3	13.1					
σ_v (psf)	1984.9 ± 142.6	1858.8		N	18.4 ± 2.5	15.0	
σ_v' (psf)	1780.2 ± 81.4	1649.9		C_R	0.92	0.95	
a_{max} (g)	0.350 ± 0.070	0.35		C_S	1.00	1.00	
r_d	0.91 ± 0.070	0.94		C_B	1.00	1.00	
CSR	0.232 ± 0.050	0.241		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.09	1.12	
MSF		1.17		$(N_1)_{60}$	22.5 ± 3.1	19.3	
CSRN		0.200					



γ_m=120±3 pcf

γ_s=125±3 pcf



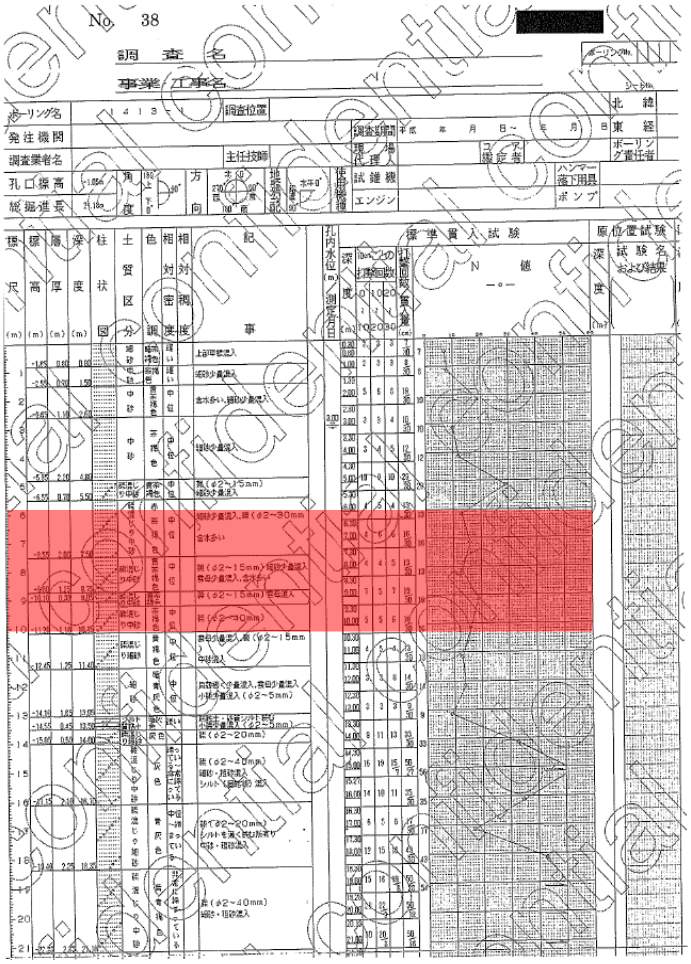
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
2.0	6.6	NA	17.0	1.64	0.80	***27.0***	***0.0***
3.0	9.8	NA	16.0	1.34	0.85	***22.1***	***0.0***
4.0	13.1	0.214	11.0	1.16	0.89	*13.8*	*0.0*
5.0	16.4	0.232	17.0	1.09	0.92	***20.8***	***0.0***
6.0	19.7	0.242	17.0	1.03	0.95	***20.3***	***0.0***
					Mean:	22.5	0.0
					St. Dev.	3.1	2.0

assigned

Case number:	196
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 38
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

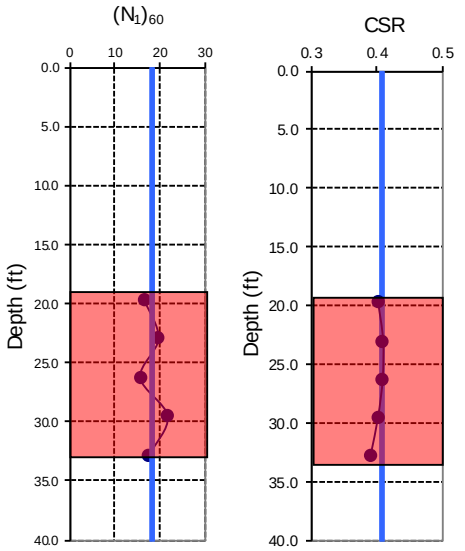
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	19.7 - 32.8	26.2		% PI			
Depth to GWT (ft)	9.8 ± 0.3	9.8					
σ_v (psf)	3182.4 ± 279.4	2986.6		N	15.2 ± 2.1	15.1	
σ_v' (psf)	2158.8 ± 149.3	1963.2		C_R	1.00	1.00	
a_{max} (g)	0.500 ± 0.100	0.50		C_S	1.00	1.00	
r_d	0.85 ± 0.104	0.89		C_B	1.00	1.00	
CSR	0.409 ± 0.096	0.441		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.99	1.03	
MSF		1.17		$(N_1)_{60}$	18.3 ± 2.5	19.1	
CSRN		0.373					



γ_m=115±3 pcf

γ_s=125±3 pcf



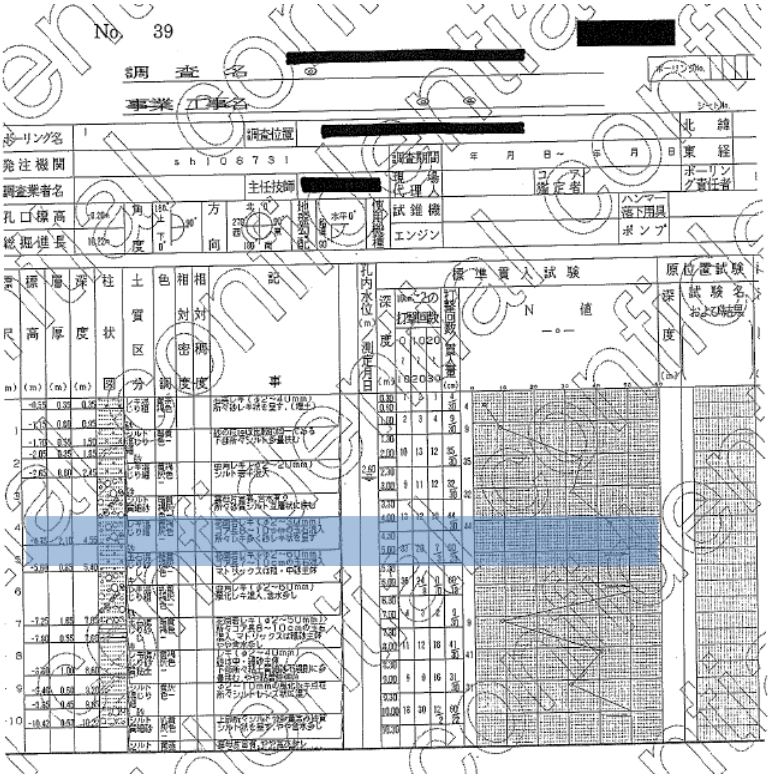
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
6.0	19.7	0.403	13.0	1.10	0.95	***16.5***	***5.0***
7.0	23.0	0.409	16.0	1.04	0.98	***19.8***	***5.0***
8.0	26.2	0.409	13.0	0.99	1.00	***15.7***	***5.0***
9.0	29.5	0.403	19.0	0.95	1.00	***21.9***	***5.0***
10.0	32.8	0.393	16.0	0.91	1.00	***17.7***	***5.0***
					Mean:	18.3	5.0
					St. Dev.	2.5	2.0

assigned

Case number:	197
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 39
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

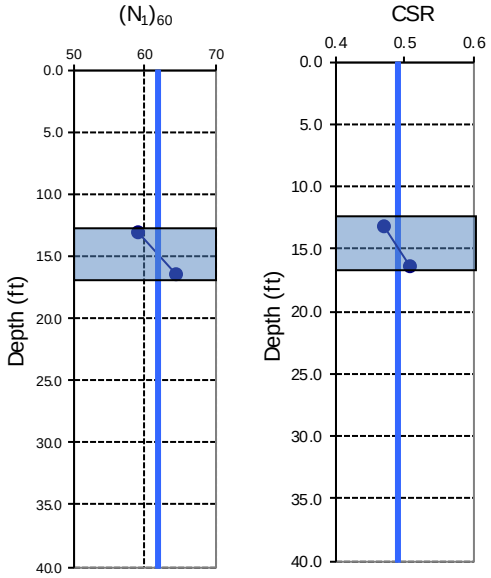
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	13.1 - 16.4	14.8		% PI			
Depth to GWT (ft)	8.5 ± 0.3	8.5					
σ_v (psf)	1865.2 ± 80.5	1754.4		N	47.0 ± 2.8	47.0	
σ_v' (psf)	1476.2 ± 52.8	1378.4		C_R	0.90	0.95	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	1.00 ± 0.064	0.95		C_B	1.00	1.00	
CSR	0.492 ± 0.104	0.476		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.20	1.12	
MSF		1.17		$(N_1)_{60}$	61.9 ± 3.7	61.0	
CSRN		0.370					



γ_m=120±3 pcf

γ_s=135±3 pcf



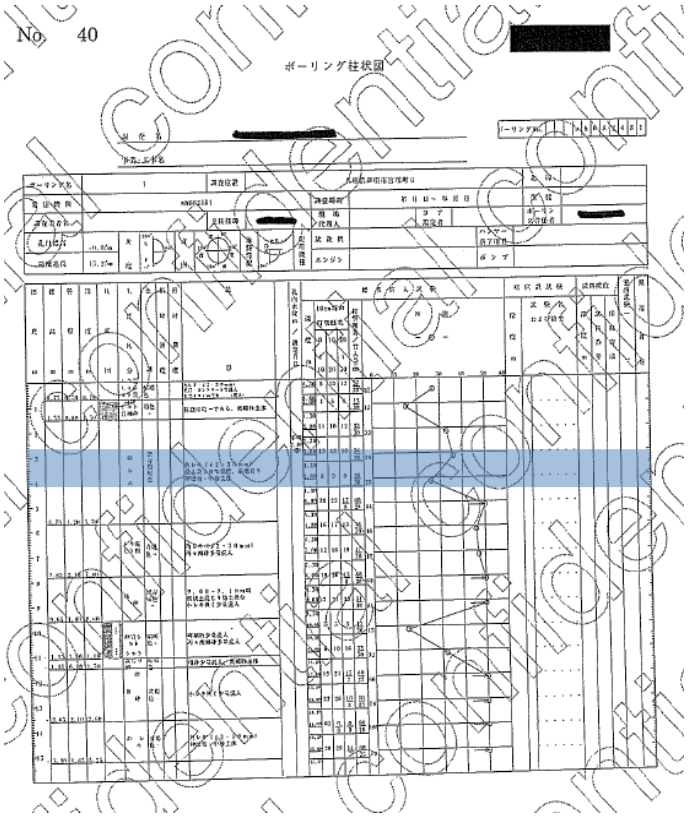
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.472	44.0	1.25	0.89	***59.3***	***0.0***
5.0	16.4	0.509	50.0	1.15	0.92	***64.5***	***0.0***
					Mean:	61.9	0.0
					St. Dev.	3.7	2.0

assigned

Case number:	198
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 40
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Non-liquefied
Comments:	

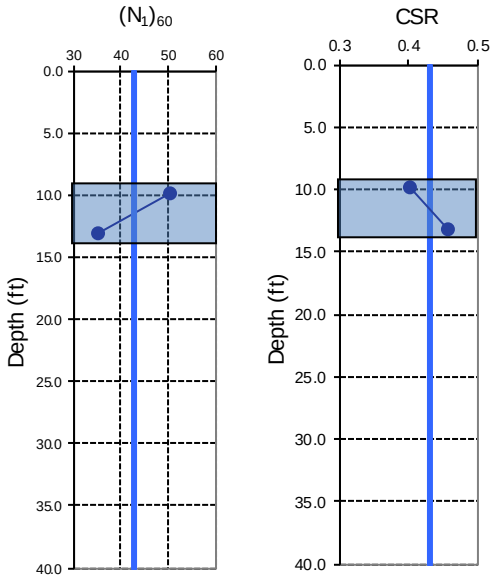
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	9.8 - 13.1	11.5		% PI			
Depth to GWT (ft)	9.2 ± 0.3	9.2					
σ_v (psf)	1389.4 ± 74.0	1378.4		N	31.1 ± 7.9	32.5	
σ_v' (psf)	1246.1 ± 47.7	1232.2		C_R	0.87	0.85	
a_{max} (g)	0.600 ± 0.120	0.60		C_S	1.00	1.00	
r_d	1.00 ± 0.052	0.97		C_B	1.00	1.00	
CSR	0.433 ± 0.090	0.421		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.30	1.18	
MSF		1.17		$(N_1)_{60}$	42.8 ± 10.8	39.7	
CSRN		0.326					



γ_m=120±3 pcf

γ_s=125±3 pcf



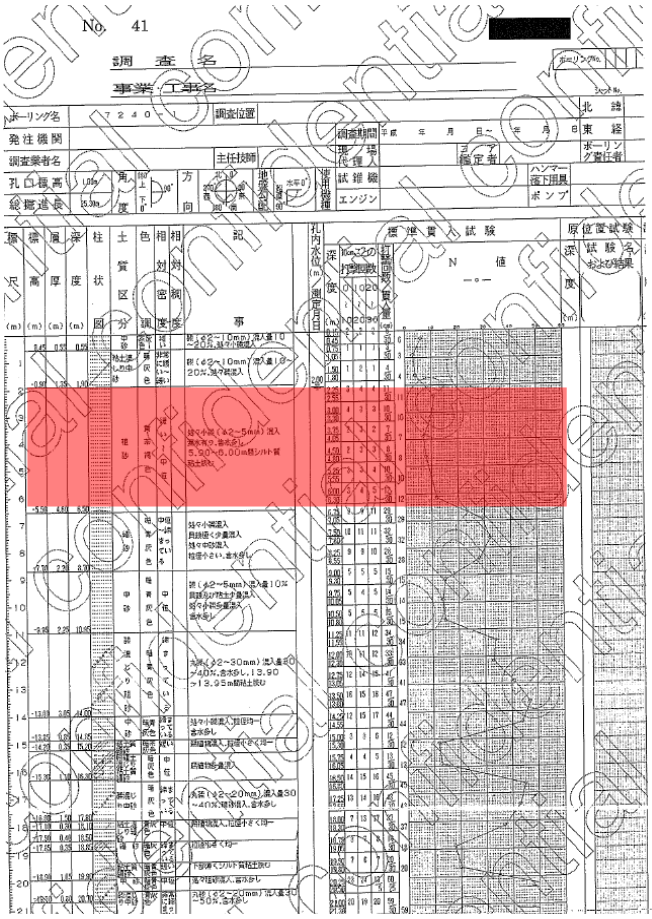
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.402	36.0	1.36	0.85	***50.4***	***0.0***
4.0	13.1	0.458	26.0	1.25	0.89	***35.1***	***0.0***
					Mean:	42.8	0.0
					St. Dev.	10.8	2.0

assigned

Case number:	199
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 41
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

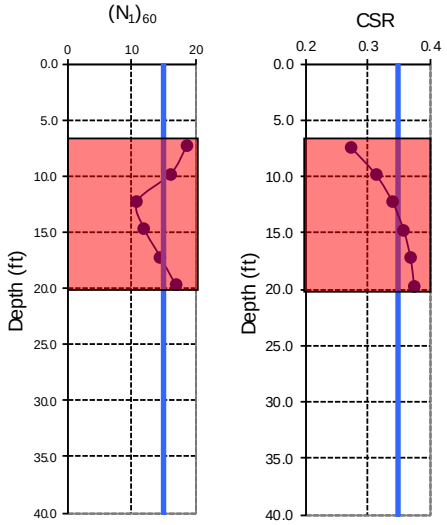
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	0.0 ± 2.0	0.0	
Critical Depth Range	7.4 - 19.7	13.5		% PI			
Depth to GWT (ft)	6.6 ± 0.3	6.6					
σ_v (psf)	1492.8 ± 247.8	1482.9		N	9.7 ± 2.0	9.2	
σ_v' (psf)	1057.7 ± 122.2	1044.3		C_R	0.89	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.96 ± 0.059	0.96		C_B	1.00	1.00	
CSR	0.351 ± 0.076	0.352		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.41	1.41	
MSF		1.17		$(N_1)_{60}$	14.9 ± 3.0	15.0	
CSRN		0.279					



γ_m=100±3 pcf

γ_s=120±3 pcf

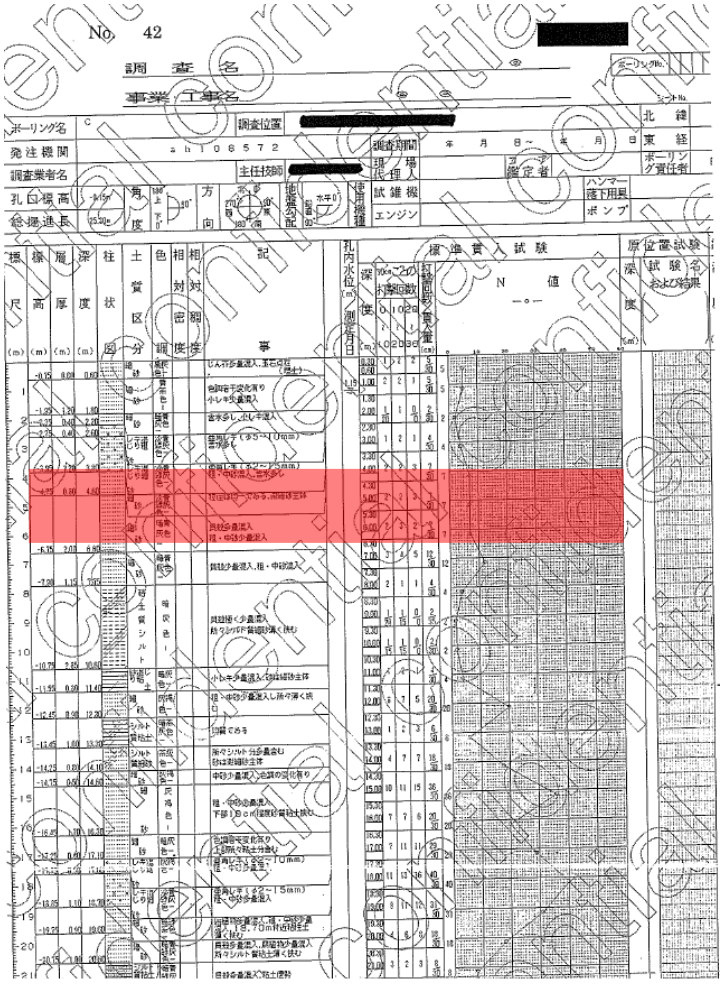


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
2.3	7.4	0.274	11.0	1.73	0.81	***18.8***	***0.0***
3.0	9.8	0.314	10.0	1.58	0.85	***16.3***	***0.0***
3.8	12.3	0.341	7.0	1.46	0.88	***10.9***	***0.0***
4.5	14.8	0.358	8.0	1.37	0.90	***12.1***	***0.0***
5.3	17.2	0.369	10.0	1.29	0.93	***14.6***	***0.0***
6.0	19.7	0.375	12.0	1.22	0.95	***17.0***	***0.0***
					Mean:	14.9	0.0
					St. Dev.	3.0	2.0

Case number:	200
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 42
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

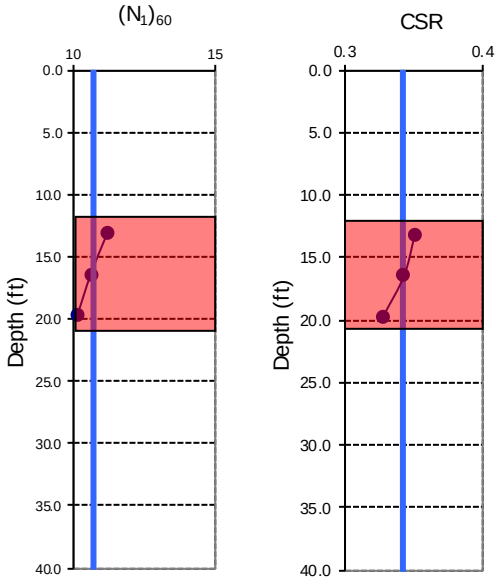
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	10.0 ± 2.0	10.0	
Critical Depth Range	13.1 - 19.7	16.4		% PI			
Depth to GWT (ft)	3.8 ± 0.3	3.9					
σ_v (psf)	1930.8 ± 137.1	1754.4		N	7.0 ± 0.3	7.0	
σ_v' (psf)	1142.6 ± 76.0	960.7		C_R	0.92	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.78 ± 0.070	0.94		C_B	1.00	1.00	
CSR	0.343 ± 0.076	0.445		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.36	1.48	
MSF		1.17		$(N_1)_{60}$	10.7 ± 0.5	12.1	
CSRN		0.352					



γ_m=110±3 pcf

γ_s=120±3 pcf

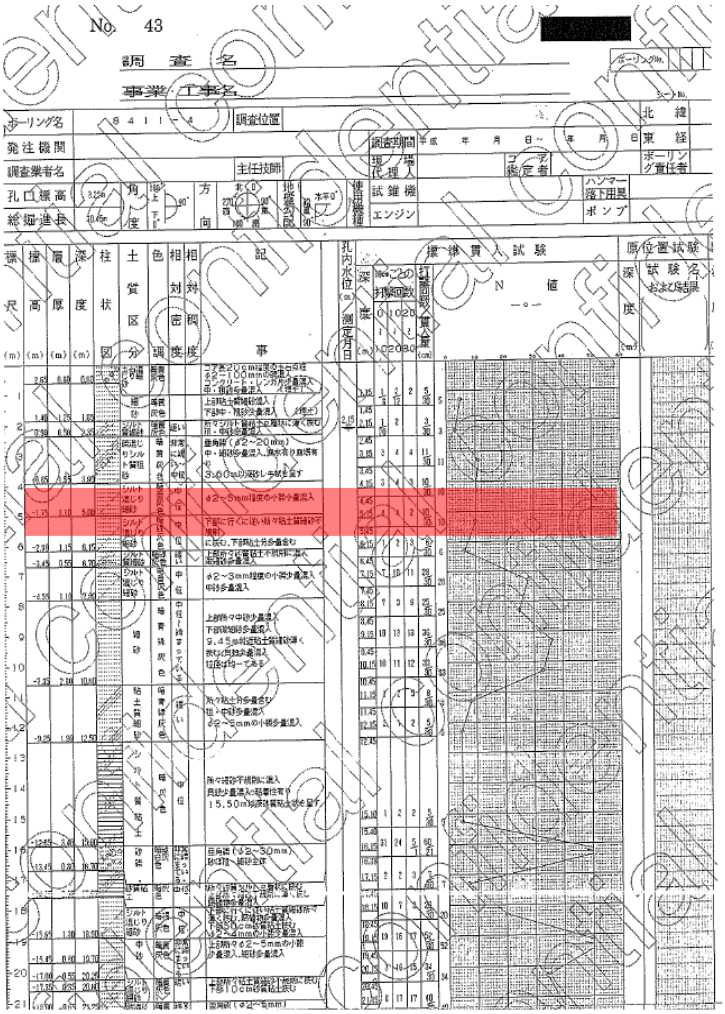


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.0	13.1	0.351	7.0	1.49	0.89	***11.3***	***10.0***
5.0	16.4	0.343	7.0	1.36	0.92	***10.7***	***10.0***
6.0	19.7	0.329	7.0	1.26	0.95	***10.2***	***10.0***
					Mean:	10.7	10.0
					St. Dev.	0.5	2.0

Case number:	201
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 43
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

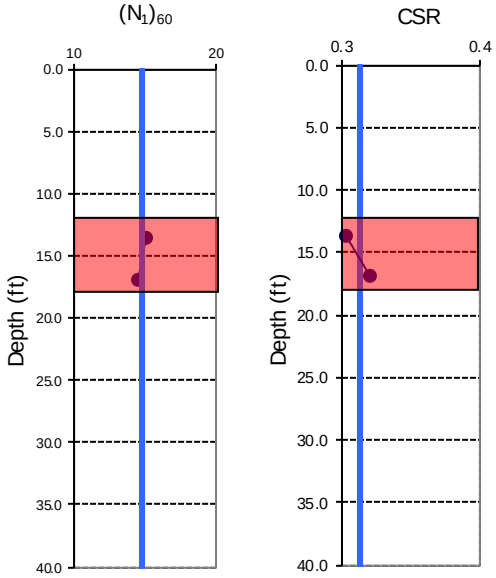
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	13.8 - 17.1	15.4		% PI			
Depth to GWT (ft)	7.1 ± 0.3	7.2					
σ_v (psf)	1709.3 ± 73.6	1670.8		N	10.0 ± 0.3	10.0	
σ_v' (psf)	1187.3 ± 47.2	1148.7		C_R	0.91	0.95	
a_{max} (g)	0.350 ± 0.070	0.35		C_S	1.00	1.00	
r_d	0.96 ± 0.066	0.95		C_B	1.00	1.00	
CSR	0.314 ± 0.067	0.311		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.34	1.31	
MSF		1.17		$(N_1)_{60}$	14.8 ± 0.5	15.2	
CSRN		0.246					



γ_m=100±3 pcf

γ_s=120±3 pcf



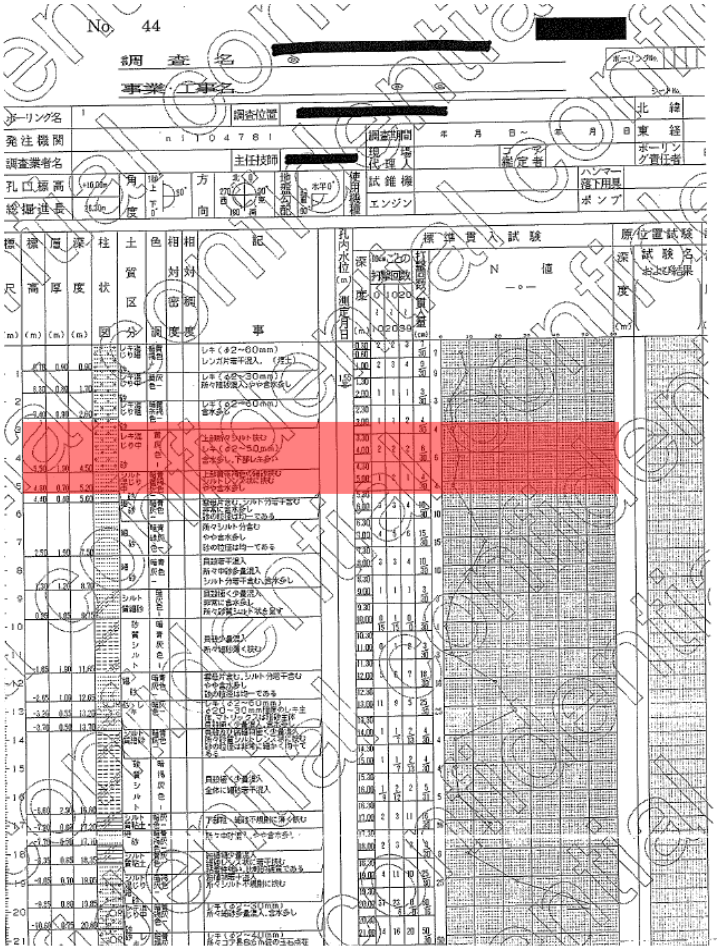
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
4.2	13.6	0.303	10.0	1.40	0.89	***15.2***	***20.0***
5.2	16.9	0.321	10.0	1.29	0.93	***14.5***	***20.0***
					Mean:	14.8	20.0
					St. Dev.	0.5	2.0

assigned

Case number:	202
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Kobe No : 44
References:	Tokimatsu, K. Personal Communication (2000)
Nature of Failure:	Liquefied
Comments:	

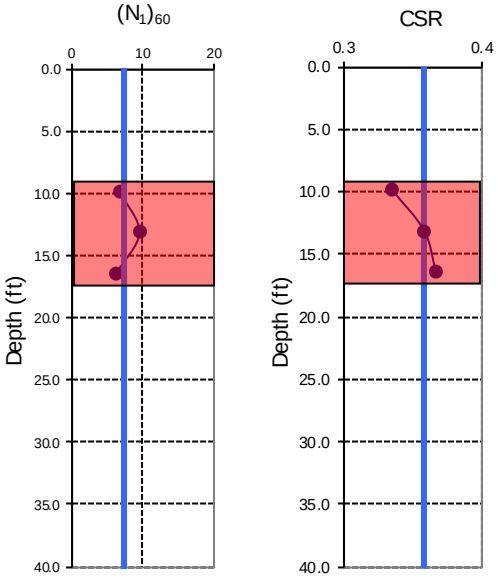
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.000 ± 0.000		
Data Class	B			% Fines	5.0 ± 2.0	5.0	
Critical Depth Range	9.8 - 16.4	13.1		% PI			
Depth to GWT (ft)	5.1 ± 0.3	5.2					
σ_v (psf)	1443.6 ± 123.6	1399.3		N	4.7 ± 1.2	4.4	
σ_v' (psf)	942.0 ± 62.2	898.1		C_R	0.89	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.90 ± 0.058	0.96		C_B	1.00	1.00	
CSR	0.359 ± 0.076	0.388		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.50	1.61	
MSF		1.17		$(N_1)_{60}$	7.6 ± 1.9	8.3	
CSRN		0.308					



γ_m=110±3 pcf

γ_s=110±3 pcf



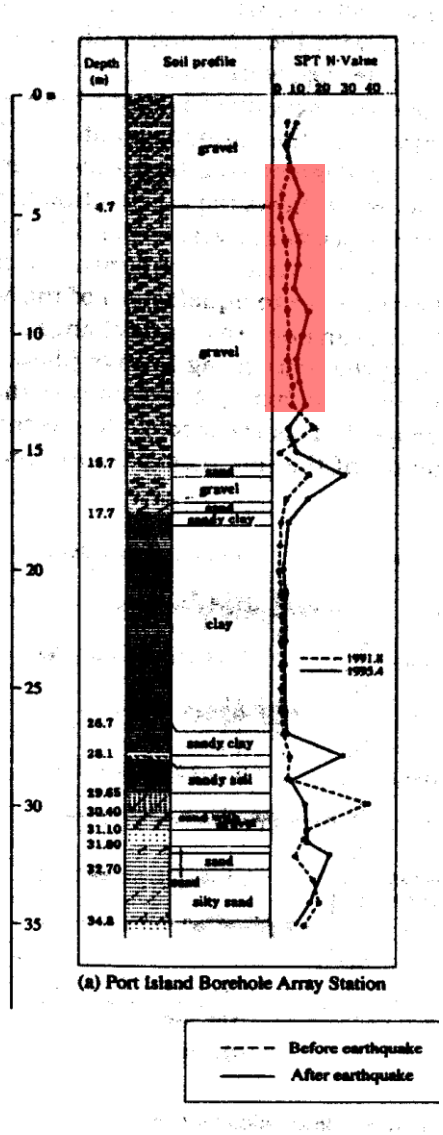
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	FC
3.0	9.8	0.335	4.0	1.64	0.85	***6.8***	***5.0***
4.0	13.1	0.359	6.0	1.50	0.89	***9.7***	***5.0***
5.0	16.4	0.368	4.0	1.39	0.92	***6.2***	***5.0***
					Mean:	7.6	5.0
					St. Dev.	1.9	2.0

assigned

Case number:	203
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (ML)
Location:	Port Island Borehole Array Site
References:	Shibata et al.(1996)
Nature of Failure:	Densification after earthquake, pore pressure increase, change in frequency content of the strong motion recordings at depths 0 and 16 m. Liquefied.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed art numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Ilsand was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at the site. Based on the recordings, it became apparent that the site has liquefied before 6th second. Also the liquefied layer was estimated at depths 0-16 m. The average Japanese SPT energy ratio of 73% is used.

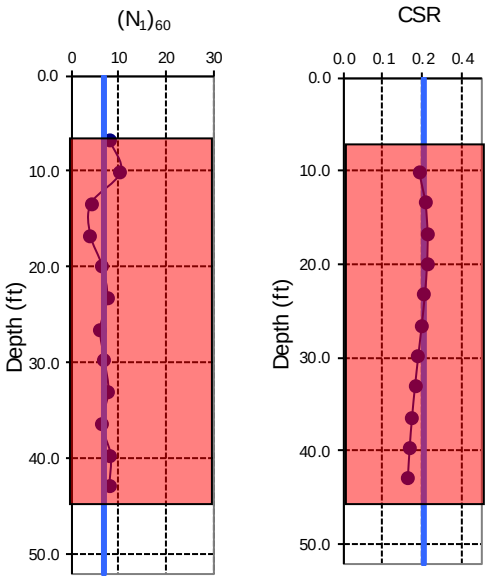
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.400 ± 0.020		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	6.9 - 43.0	25.6		% PI			
Depth to GWT (ft)	7.9 ± 0.3	7.9					
σ_v (psf)	2998.7 ± 754.0	3111.9		N	5.7 ± 1.4	5.7	
σ_v' (psf)	1934.1 ± 381.0	2005.0		C_R	0.99	0.95	
a_{max} (g)	0.340 ± 0.068	0.34		C_S	1.00	1.00	
r_d	0.72 ± 0.100	0.90		C_B	1.00	1.00	
CSR	0.245 ± 0.061	0.307		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.05	1.03	
MSF		1.17		$(N_1)_{60}$	7.1 ± 1.8	6.8	
CSRN		0.260					



γ_m=110±3 pcf

γ_s=125±3 pcf

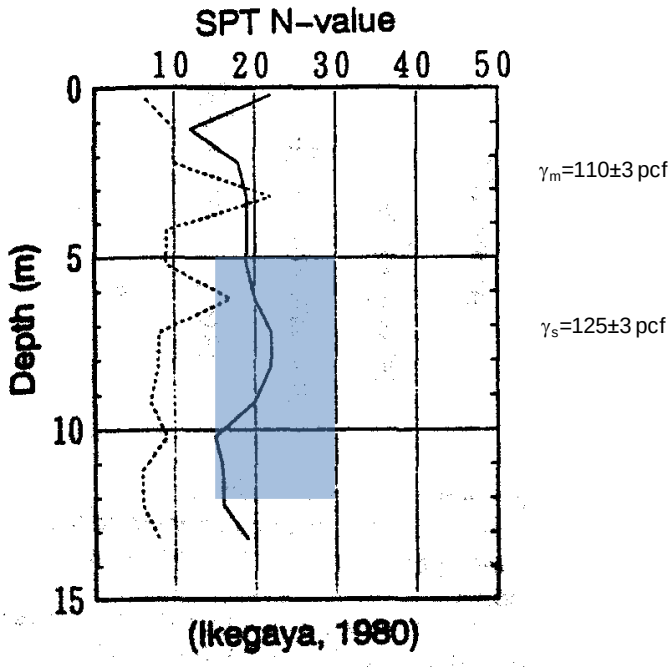


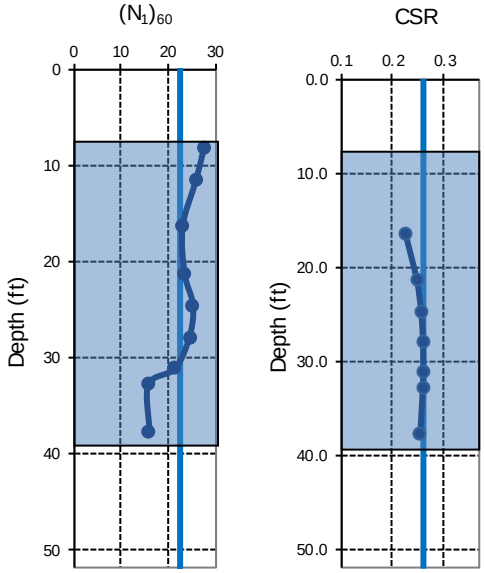
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.1	6.9	NA	5.0	1.67	0.80	***8.2***
3.1	10.2	0.233	7.0	1.45	0.85	***10.5***
4.1	13.5	0.251	3.0	1.32	0.89	***4.3***
5.1	16.7	0.258	3.0	1.22	0.92	***4.1***
6.1	20.0	0.256	5.0	1.14	0.95	***6.6***
7.1	23.3	0.249	6.0	1.07	0.98	***7.7***
8.1	26.6	0.240	5.0	1.02	1.00	***6.2***
9.1	29.9	0.230	6.0	0.97	1.00	***7.1***
10.1	33.1	0.220	7.0	0.93	1.00	***7.9***
11.1	36.4	0.211	6.0	0.89	1.00	***6.5***
12.1	39.7	0.204	8.0	0.86	1.00	***8.4***
13.1	43.0	0.199	8.0	0.83	1.00	***8.1***
					Mean:	7.1
					St. Dev.	1.8

Case number:	204
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (ML)
Location:	Port Island Improved Site (Ikegaya, 80)
References:	Yasuda et al.(1996) Ikegaya (1980) in Japanese
Nature of Failure:	No subsidence and no damage to structures were observed in the zones densified by rod (vibro) compaction.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Island was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The improved site is located on Port Island. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not affected the PGA significantly because the fling was recorded at very early stages of the shaking. The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.400 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	16.4 - 39.4	27.9		% PI			
Depth to GWT (ft)	16.4 ± 0.3	16.4					
σ_v (psf)	3239.8 ± 482.2	3320.8		N	20.3 ± 3.8	20.2	
σ_v' (psf)	2523.3 ± 247.4	2610.7		C_R	1.00	1.00	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.87 ± 0.110	0.88		C_B	1.00	1.00	
CSR	0.291 ± 0.071	0.293		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.92	0.92	
MSF		1.17		$(N_1)_{60}$	22.6 ± 4.2	22.7	
CSRN		0.260					



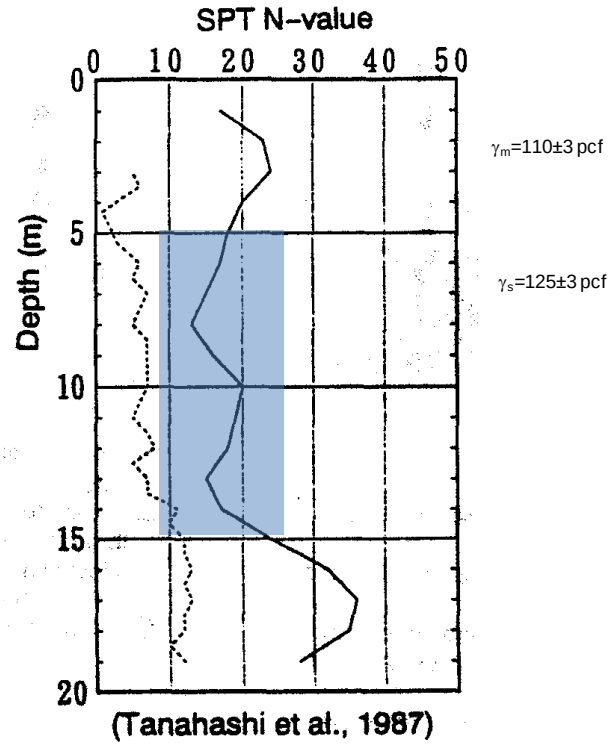


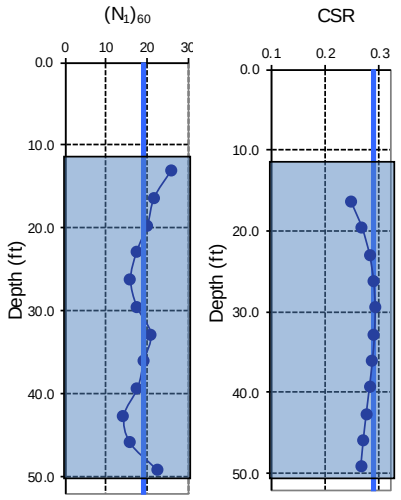
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
2.5	8.2	NA	18.0	1.53	0.82	*27.6*
3.5	11.5	NA	19.0	1.29	0.87	*26.0*
5.0	16.4	0.249	19.0	1.08	0.92	*23.1*
6.5	21.3	0.276	20.0	1.00	0.96	*23.5*
7.5	24.6	0.286	22.0	0.96	0.99	*25.3*
8.5	27.9	0.291	22.0	0.92	1.00	*24.5*
9.5	31.2	0.292	20.0	0.88	1.00	*21.4*
10.0	32.8	0.291	15.0	0.86	1.00	*15.8*
11.5	37.7	0.285	16.0	0.82	1.00	*16.0*
					Mean:	22.6
					St. Dev.	4.2

Case number:	205
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Port Island Improved Site (Tanahashi 87)
References:	Yasuda et al.(1996) Tanahashi (1978) in Japanese
Nature of Failure:	No subsidence and no damage to structures were observed in the zones densified by rod (vibro) compaction.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Ilsand was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The improved site is located on Port Island. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not affected the PGA significantly because the fling was recorded at very early stages of the shaking. The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.400 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	16.4 - 49.2	32.8		% PI			
Depth to GWT (ft)	16.4 ± 0.3	16.4					
σ_v (psf)	3855.0 ± 687.1	3947.3		N	18.2 ± 3.2	18.2	
σ_v' (psf)	2831.4 ± 349.6	2924.0		C_R	1.00	1.00	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.82 ± 0.126	0.86		C_B	1.00	1.00	
CSR	0.291 ± 0.075	0.301		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.86	0.88	
MSF		1.17		$(N_1)_{60}$	19.1 ± 3.4	19.5	
CSRN		0.270					



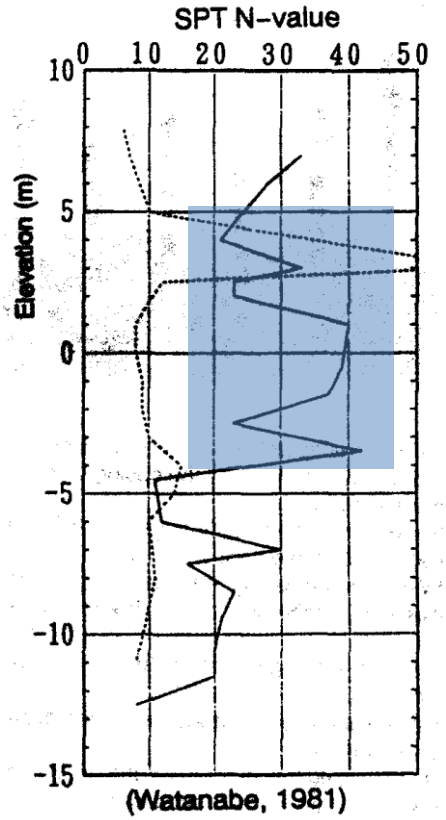


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
4.0	13.1	NA	20.0	1.21	0.89	*26.1*
5.0	16.4	0.249	18.0	1.08	0.92	*21.8*
6.0	19.7	0.269	17.0	1.03	0.95	*20.2*
7.0	23.0	0.282	15.0	0.98	0.98	*17.4*
8.0	26.2	0.290	14.0	0.94	1.00	*15.9*
9.0	29.5	0.292	16.0	0.90	1.00	*17.5*
10.0	32.8	0.291	20.0	0.86	1.00	*21.0*
11.0	36.1	0.287	19.0	0.83	1.00	*19.3*
12.0	39.4	0.282	18.0	0.81	1.00	*17.7*
13.0	42.7	0.276	15.0	0.78	1.00	*14.3*
14.0	45.9	0.271	17.0	0.76	1.00	*15.7*
15.0	49.2	0.266	25.0	0.74	1.00	*22.5*
					Mean:	19.1
					St. Dev.	3.4

Case number:	206
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (ML)
Location:	Port Island Improved Site (Watanabe 81)
References:	Yasuda et al.(1996) Tanahashi (1978) in Japanese
Nature of Failure:	No subsidence and no damage to structures were observed in the zones densified by rod (vibro) compaction.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Island was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The improved site is located on Port Island. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34 g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not affected the PGA significantly because the fling was recorded at very early stages of the shaking. The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

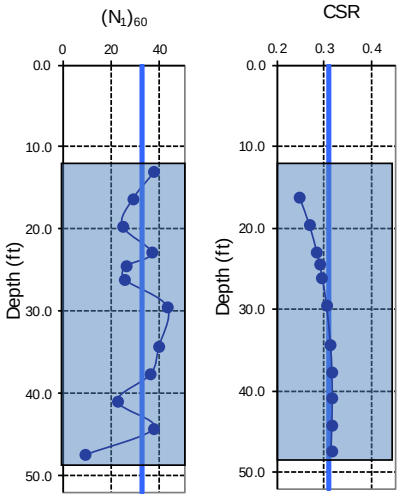
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	No	No		D_{50}	0.400 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	16.4 - 45.9	31.2		% PI			
Depth to GWT (ft)	16.4 ± 0.3	16.4					
σ_v (psf)	3723.8 ± 643.2	3738.5		N	31.0 ± 6.7	30.9	
σ_v' (psf)	2802.5 ± 339.4	2819.5		C_R	1.00	1.00	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.96 ± 0.121	0.87		C_B	1.00	1.00	
CSR	0.332 ± 0.080	0.299		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.87	0.92	
MSF		1.17		$(N_1)_{60}$	32.8 ± 7.1	34.6	
CSRN		0.278					



γ_m=110±3 pcf

γ_s=130±3 pcf

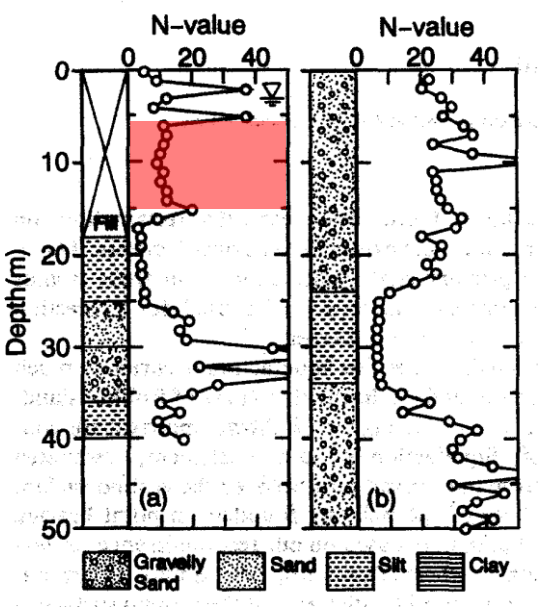


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
4.0	13.1	NA	29.0	1.21	0.89	*37.9*	
5.0	16.4	0.258	24.0	1.08	0.92	*29.1*	
6.0	19.7	0.283	21.0	1.02	0.95	*24.8*	
7.0	23.0	0.302	32.0	0.97	0.98	*36.9*	
7.5	24.6	0.310	23.0	0.95	0.99	*26.2*	
8.0	26.2	0.317	23.0	0.93	1.00	*25.9*	
9.0	29.5	0.328	40.0	0.89	1.00	*43.2*	
10.5	34.4	0.338	39.0	0.84	1.00	*39.7*	
11.5	37.7	0.342	37.0	0.81	1.00	*36.3*	
12.5	41.0	0.343	24.0	0.78	1.00	*22.8*	
13.5	44.3	0.342	41.0	0.76	1.00	*37.8*	
14.5	47.6	0.340	11.0	0.74	1.00	9.8	clay layer
					Mean:	32.8	
					St. Dev.	7.1	

Case number:	207
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Port Island Site I
References:	Tokimatsu et al. (1996)
Nature of Failure:	Up to 50 cm of settlements; sand boils
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Island was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The improved site is located on Port Island. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not to affect the PGA because the fling was recorded at very early stages of the shaking. The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

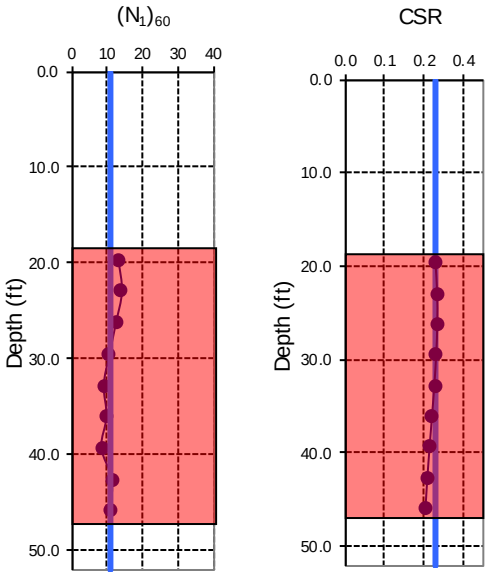
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.400 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	19.7 - 45.9	32.8		% PI			
Depth to GWT (ft)	9.8 ± 0.3	9.8					
σ_v (psf)	3838.6 ± 530.3	4010.0		N	9.7 ± 1.6	9.7	
σ_v' (psf)	2405.5 ± 263.4	2568.9		C_R	1.00	1.00	
a_{max} (g)	0.340 ± 0.068	0.34		C_S	1.00	1.00	
r_d	0.78 ± 0.126	0.86		C_B	1.00	1.00	
CSR	0.274 ± 0.071	0.295		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.94	0.91	
MSF		1.17		$(N_1)_{60}$	11.1 ± 1.8	10.8	
CSRN		0.258					



γ_m=110±3 pcf

γ_s=120±3 pcf

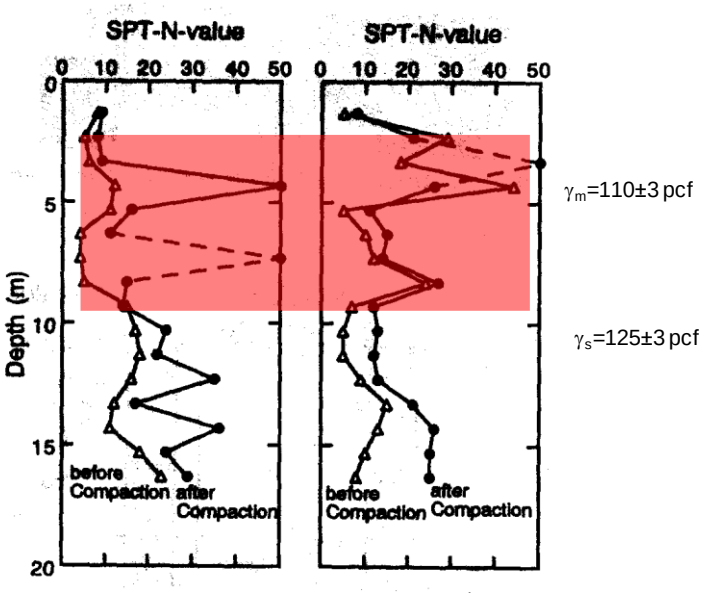


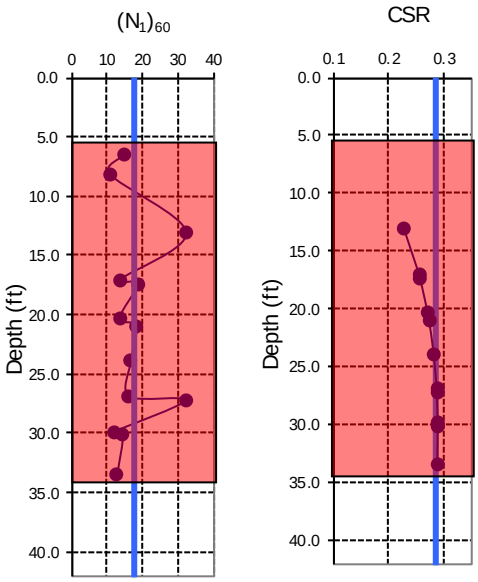
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
6.0	19.7	0.277	10.0	1.13	0.95	***13.1***
7.0	23.0	0.283	11.0	1.07	0.98	***14.0***
8.0	26.2	0.283	10.0	1.02	1.00	***12.4***
9.0	29.5	0.280	9.0	0.98	1.00	***10.7***
10.0	32.8	0.274	8.0	0.94	1.00	***9.1***
11.0	36.1	0.267	9.0	0.90	1.00	***9.9***
12.0	39.4	0.259	8.0	0.87	1.00	***8.5***
13.0	42.7	0.252	11.0	0.84	1.00	***11.3***
14.0	45.9	0.246	11.0	0.82	1.00	***10.9***
					Mean:	11.1
					St. Dev.	1.8

Case number:	208
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Rokko Island Building D
References:	Tokimatsu et al. (1996)
Nature of Failure:	Ground settlement, tilted structures
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Island was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The southern part of Rokko Island consist of mainly of crsuhed tuff and sand-stone of the Kobe and Osaka Groups from Rokko Mountains. The soils of southern Rokko Island includes more fines and clay, i.e about 20 % and 10 % compared with about 15 % and 5 % of decomposed granite (Masado). Building D is located in the southern part of Rokko Island. Site has been improved by sand compaction piles. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not affected the PGA significantly because the fling was recorded at very early stages of the shaking. Rokko Island is predicted to have experienced the same level of shaking The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.800 ± 0.050		
Data Class	B			% Fines	25.0 ± 2.0	25.0	
Critical Depth Range	13.1 - 36.1	24.6		% PI			
Depth to GWT (ft)	13.1 ± 0.3	13.1					
σ_v (psf)	2878.9 ± 481.3	2944.8		N	14.8 ± 6.0	14.8	
σ_v' (psf)	2162.4 ± 245.7	2234.7		C_R	0.99	0.95	
a_{max} (g)	0.400 ± 0.080	0.40		C_S	1.00	1.00	
r_d	0.93 ± 0.099	0.90		C_B	1.00	1.00	
CSR	0.322 ± 0.075	0.310		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.99	0.98	
MSF		1.17		$(N_1)_{60}$	17.6 ± 7.1	16.8	
CSRN		0.267					



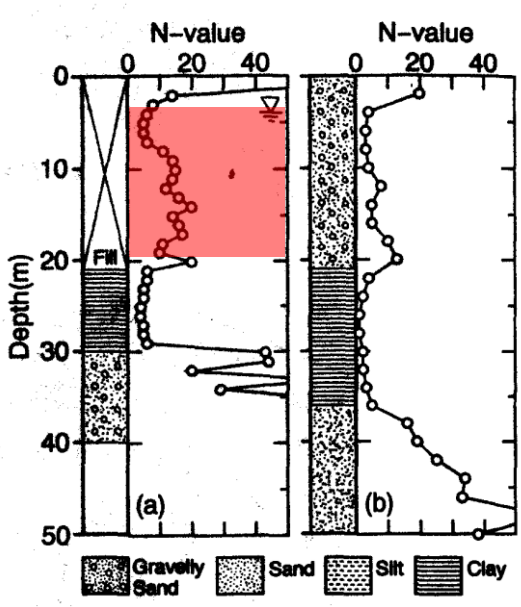


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$	
2.0	6.6	NA	9.0	1.71	0.80	***15.0***	1
2.5	8.2	NA	7.0	1.53	0.82	***10.7***	2
4.0	13.1	0.255	25.0	1.21	0.89	***32.7***	2
5.2	17.1	0.288	11.0	1.12	0.93	***13.9***	2
5.3	17.4	0.291	15.0	1.11	0.93	***18.9***	1
6.2	20.3	0.307	11.0	1.06	0.96	***13.5***	1
6.4	21.0	0.310	15.0	1.05	0.96	***18.3***	2
7.3	24.0	0.321	14.0	1.00	0.98	***16.7***	2
8.2	26.9	0.327	14.0	0.96	1.00	***16.3***	1
8.3	27.2	0.327	28.0	0.95	1.00	***32.5***	2
9.1	29.9	0.329	11.0	0.92	1.00	***12.3***	2
9.2	30.2	0.330	13.0	0.92	1.00	***14.5***	1
10.2	33.5	0.328	12.0	0.88	1.00	***12.9***	2
					Mean:	17.6	
					St. Dev.	7.1	

Case number:	209
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Rokko Island Site G
References:	Tokimatsu et al. (1996)
Nature of Failure:	Up to 50 cm of settlements; sand boils
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The entire area of the first stage reclaimed land for Port Island and the northern area of Rokko Island were filled mainly with Masado. The first stage reclamation of Port Island was carried out at its northern part of the island from 1966-1980. Rokko Island was constructed from 1972 to 1990. The second stage reclamation of Port Island started in 1986 and would be finished by 1996. The southern part of Rokko Island consist of mainly of crsuhed tuff and sand-stone of the Kobe and Osaka Groups from Rokko Mountains. The soils of southern Rokko Island includes more fines and clay, i.e about 20 % and 10 % compared with about 15 % and 5 % of decomposed granite (Masado). Site G is located in the northern part of Rokko Island. Masado is a weathered and decomposed low plasticity, low cohesion, high permeability, erodible granite. It is a sandy soil with much gravel, silt and clay. The mean grain diameter is mostly 0.2-6 mm. The contents of gravel and fines are about 0-65 % and 5-35 % respectively. PGA was recorded as 0.34g at Port Island. The recorded motion was obtained at a liquefied site. Liquefaction of the site is believed not affected the PGA significantly because the fling was recorded at very early stages of the shaking. Rokko Island is predicted to have experienced the same level of shaking The water table is at depth 4-6 m at the island. The average Japanese SPT energy ratio of 73% is used.

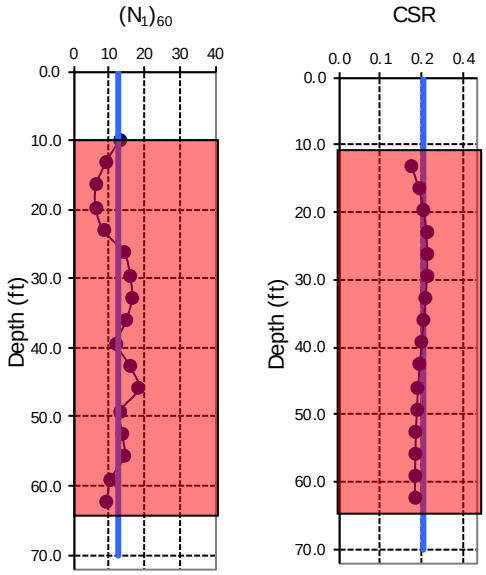
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.400 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	13.1 - 62.3	37.7		% PI			
Depth to GWT (ft)	13.1 ± 0.3	13.1					
σ_v (psf)	4396.3 ± 987.8	4573.9		N	12.0 ± 3.4	12.0	
σ_v' (psf)	2860.9 ± 480.0	3049.3		C_R	1.00	1.00	
a_{max} (g)	0.340 ± 0.068	0.34		C_S	1.00	1.00	
r_d	0.72 ± 0.142	0.83		C_B	1.00	1.00	
CSR	0.245 ± 0.070	0.275		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	0.86	0.84	
MSF		1.17		$(N_1)_{60}$	12.5 ± 3.6	12.3	
CSRN		0.246					



γ_m=110±3 pcf

γ_s=120±3 pcf

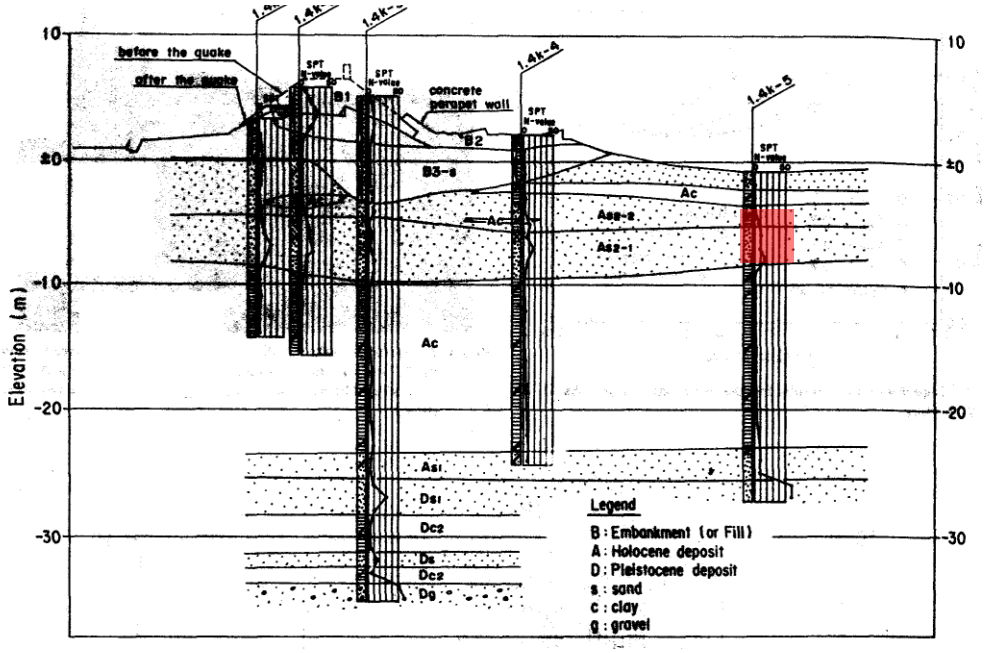


Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.0	9.8	NA	9.0	1.40	0.85	***13.0***
4.0	13.1	0.212	7.0	1.21	0.89	***9.1***
5.0	16.4	0.234	5.0	1.14	0.92	***6.4***
6.0	19.7	0.248	5.0	1.08	0.95	***6.2***
7.0	23.0	0.256	7.0	1.03	0.98	***8.5***
8.0	26.2	0.258	12.0	0.98	1.00	***14.3***
9.0	29.5	0.257	14.0	0.94	1.00	***16.0***
10.0	32.8	0.254	15.0	0.91	1.00	***16.5***
11.0	36.1	0.248	14.0	0.87	1.00	***14.9***
12.0	39.4	0.242	12.0	0.85	1.00	***12.4***
13.0	42.7	0.237	16.0	0.82	1.00	***16.0***
14.0	45.9	0.232	19.0	0.80	1.00	***18.4***
15.0	49.2	0.228	14.0	0.78	1.00	***13.2***
16.0	52.5	0.225	15.0	0.76	1.00	***13.8***
17.0	55.8	0.224	16.0	0.74	1.00	***14.3***
18.0	59.1	0.223	12.0	0.72	1.00	***10.5***
19.0	62.3	0.222	11.0	0.70	1.00	***9.4***
					Mean:	12.5
					St. Dev.	3.6

Case number:	210
Earthquake:	1995 Hyogoken-Nambu
Magnitude:	7.2 (M_L)
Location:	Torishima Dike
References:	Matsuo (1996)
Nature of Failure:	Failed dike; sand boils near dike.
Comments:	On January 17, 1995, an earthquake of magnitude 7.2 (Richter scale) struck the southern part of Hyogo Prefecture. Sand boils and liquefied, ejected soil were observed at numerous sites, mainly in the coastal areas of Hyogo and Osaka Prefectures. In particular, massive liquefaction was observed on man-made islands and in inland areas with high ground water tables. The most severely damaged were the dikes of Yodo-gawa, which, flowing through Osaka is the largest river in Kansai area. A length of 2 km of Torishima Dike, the left levee of Yodo-gawa, was severely damaged. The dike was designed to protect against high sea tide of 5.2 m above mean sea level. The core of the dike was a soil embankment, and the river side surface was protected by concrete parapet wall, the design height of which was 8.1 m from the sea level. The dike is underlain by Holocene deposit with a thickness of 30 m, called the Umeda layer. The deposit consists of, from the top to bottom, the upper sandy layer (As2), clay layer (Ac), and the lower sandy layer (As11). The thickness of Holocene deposit decreases downstream. The As2 layer is also divided by the upper (As2-2) and lower (As2-1) layer. The sand boils observed after the quake are considered to have been ejected from the upper sandy layer. The average of the main grain size and fines content of the As2 layer are 0.2-0.3 mm and 20 % respectively. Cyclic undrain TX compression tests were made on samples from the depth 8.0-8.6 m. Triaxial cyclic stress ratio on frozen samples required to cause 5 % axial starin in 20 cycles is about 0.3. PGA was recorded along Yodo River at many stations. It ranges from 0.15-0.30 g mostly concentrating around 0.22-0.26g. A value of 0.25 g was adopted for the analysis. The borings were obtained in February and March 1995, before the earthquake. The average Japanese SPT energy ratio of 73% is used.

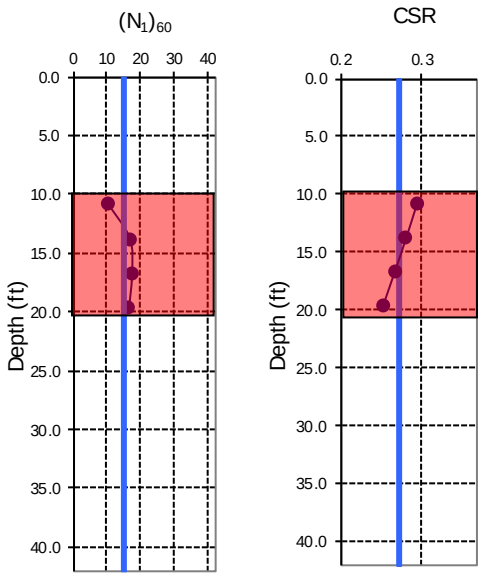
Summary of Data

	Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)		Cetin et al. (2016)	Idriss&Boulanger (2010)	Seed et.al. (1984)
Liquefied?	Yes	Yes		D_{50}	0.200 ± 0.050		
Data Class	B			% Fines	20.0 ± 2.0	20.0	
Critical Depth Range	9.8 - 21.3	15.4		% PI			
Depth to GWT (ft)	0.0 ± 0.3	0.0					
σ_v (psf)	1792.2 ± 225.0	1942.3		N	8.5 ± 1.9	8.5	
σ_v' (psf)	819.7 ± 112.3	960.7		C_R	0.91	0.95	
a_{max} (g)	0.250 ± 0.050	0.25		C_S	1.00	1.00	
r_d	0.81 ± 0.067	0.95		C_B	1.00	1.00	
CSR	0.286 ± 0.063	0.308		C_E	1.22	1.22	
Equivalent Magnitude	6.9	6.9		C_N	1.61	1.42	
MSF		1.17		$(N_1)_{60}$	15.2 ± 3.4	14.0	
CSRN		0.240					



γ_s=110±3 pcf

γ_s=115±3 pcf

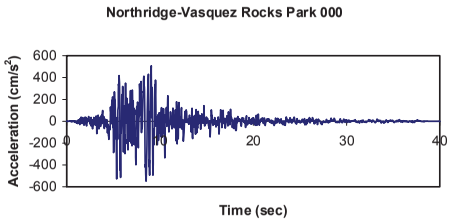


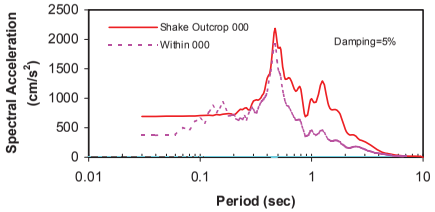
Depth (m)	Depth (ft)	CSR	N	C_N	C_R	$(N1)_{60}$
3.3	10.8	0.313	5.0	1.93	0.86	***10.1***
4.2	13.8	0.297	9.0	1.71	0.89	***16.7***
5.1	16.7	0.279	10.0	1.55	0.92	***17.4***
6.0	19.7	0.262	10.0	1.43	0.95	***16.5***
					Mean:	15.2
					St. Dev.	3.4

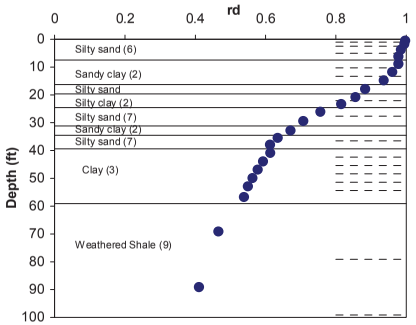
APPENDIX B

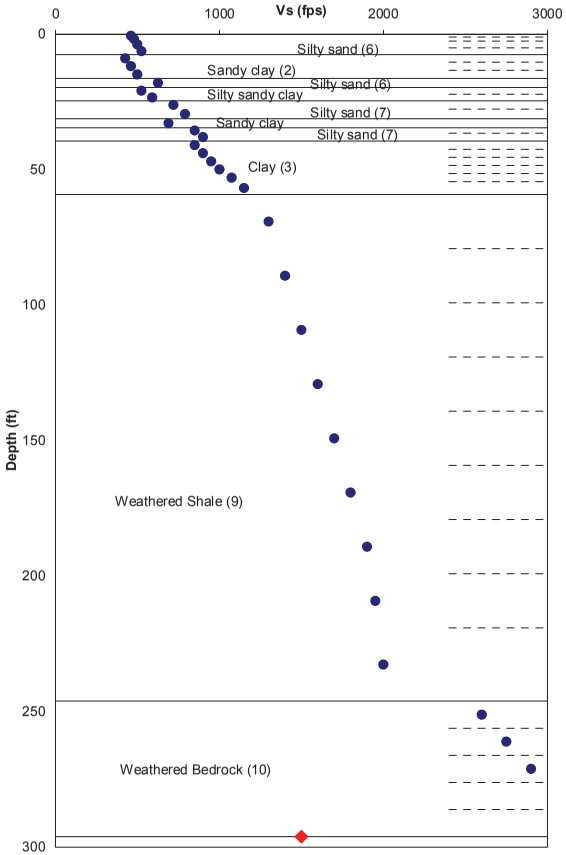
CASE HISTORY SITE RESPONSE ANLAYSIS

Balboa Boulevard-Northridge Earthquake (1994)

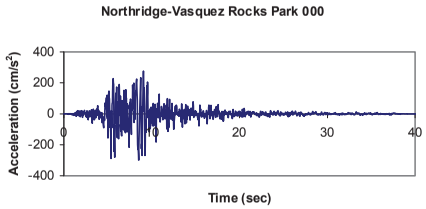


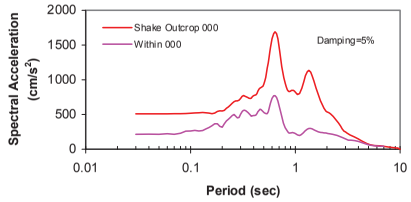


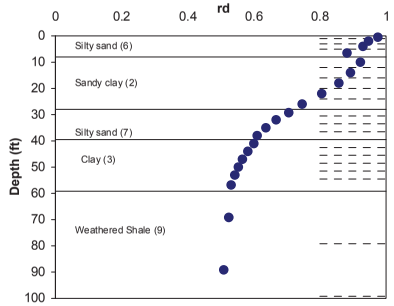


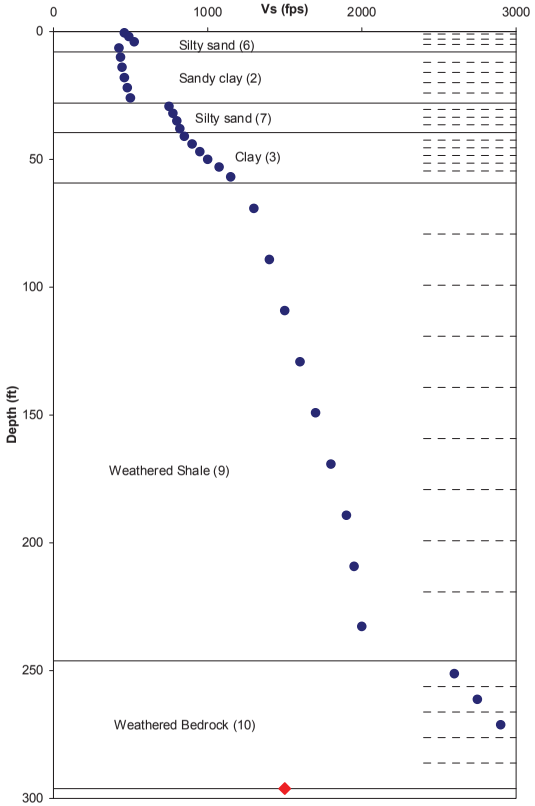


Malden Street-Northridge Earthquake (1994)

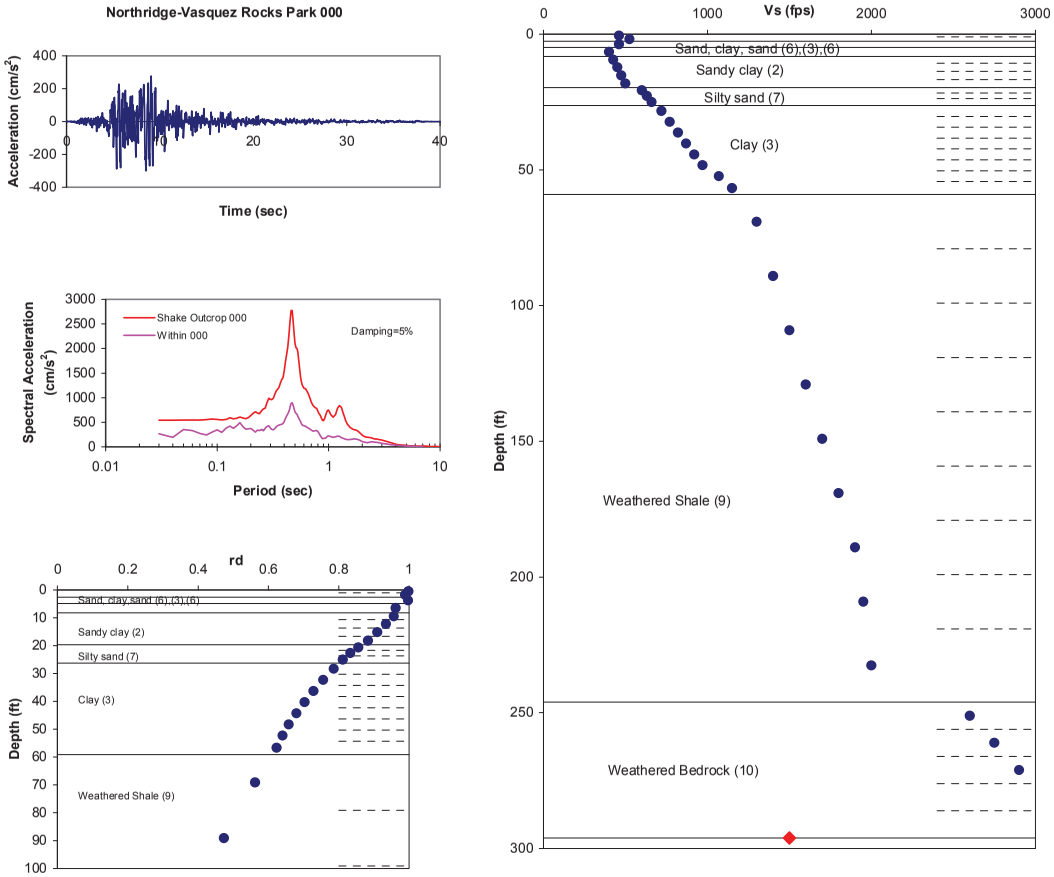




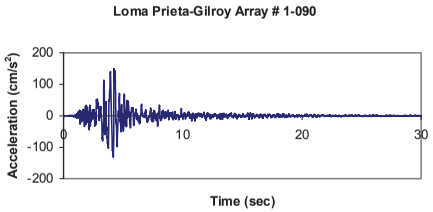


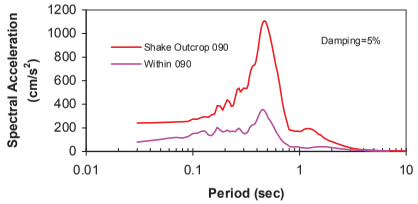


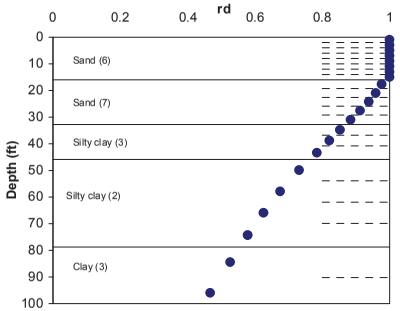
Wynne Avenue-Northridge Earthquake (1994)

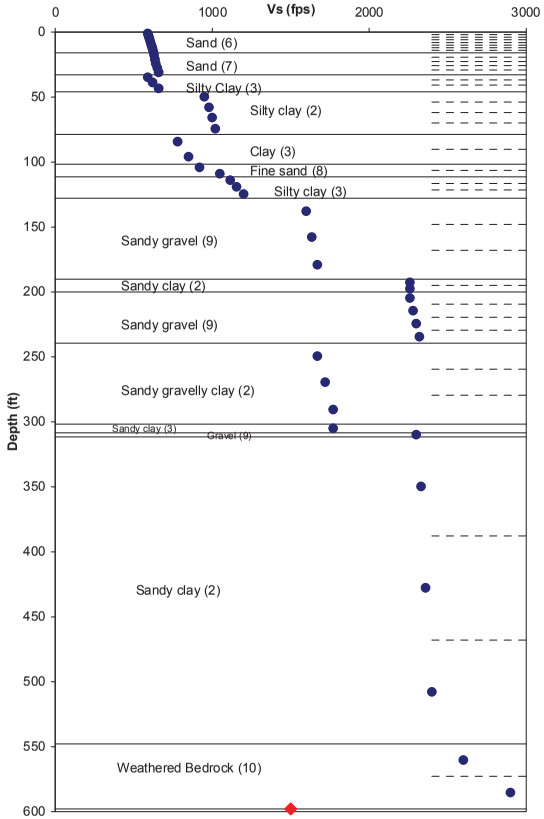


MBARI NO:3 EB-1-Loma Prieta Earthquake (1989)

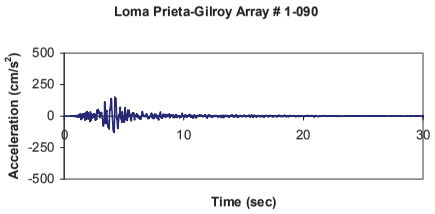


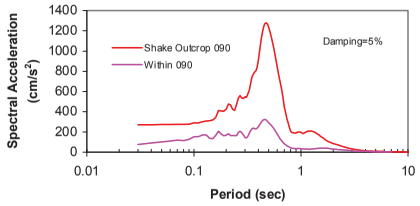


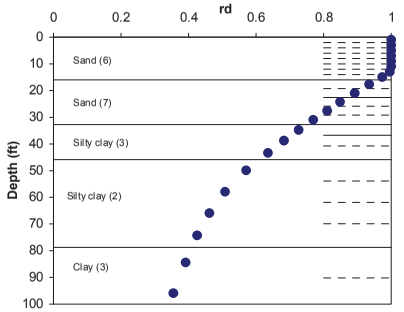


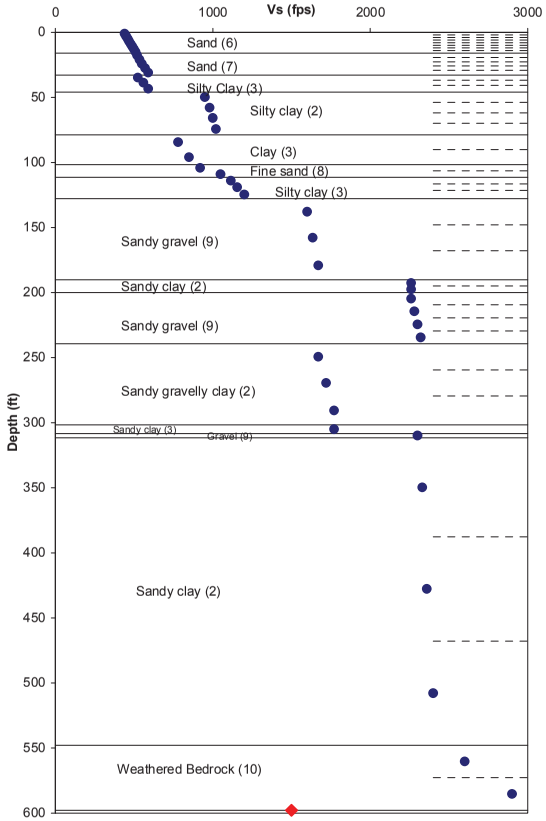


MBARI NO:3 EB-5-Loma Prieta Earthquake (1989)

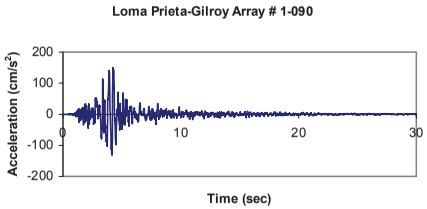


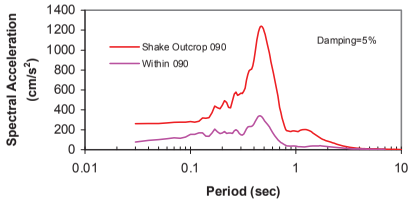


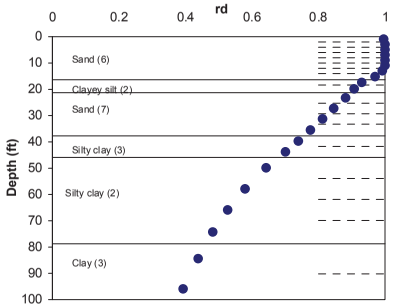


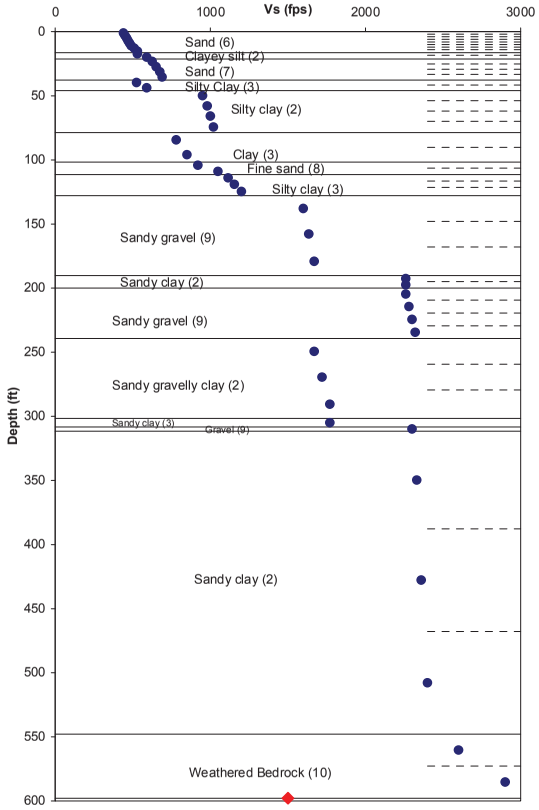


MBARI NO:3 UC-B-10-Loma Prieta Earthquake (1989)

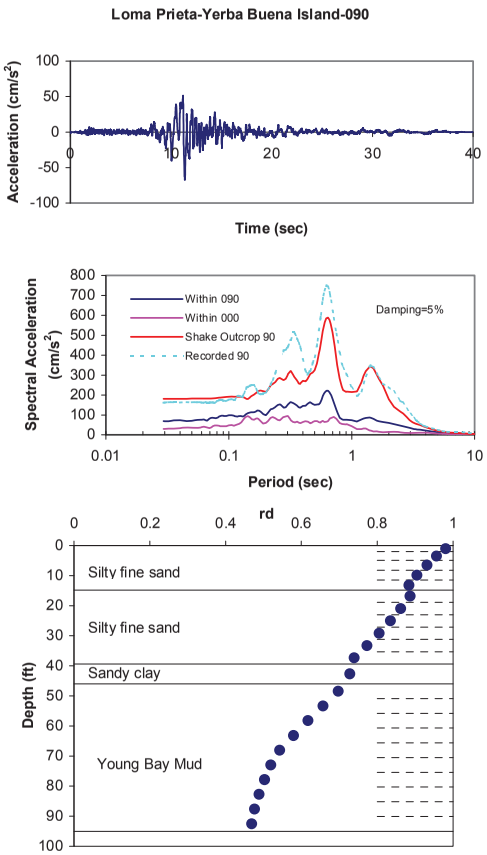


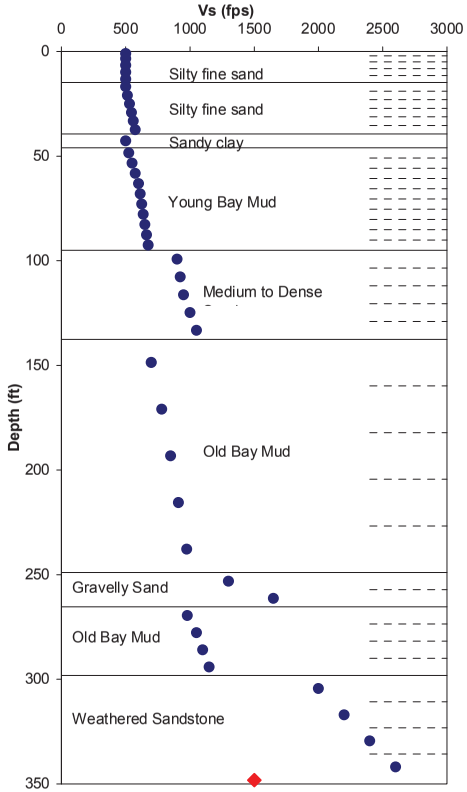




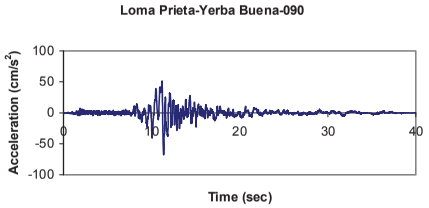


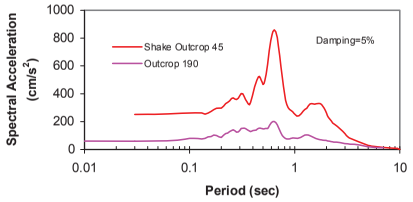
Treasure Island-Loma Prieta Earthquake (1989)

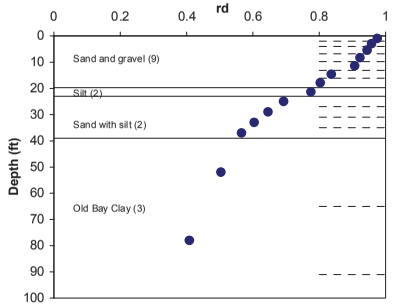


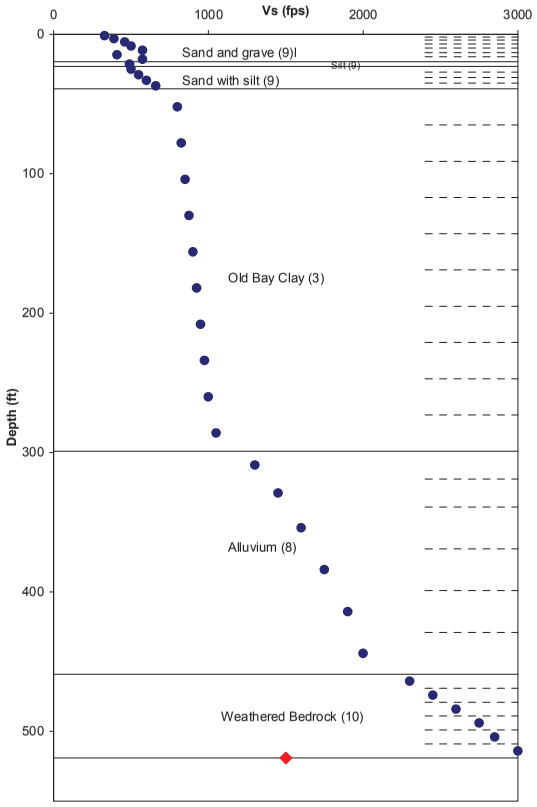


SFOBB-1-Loma Prieta Earthquake (1989)

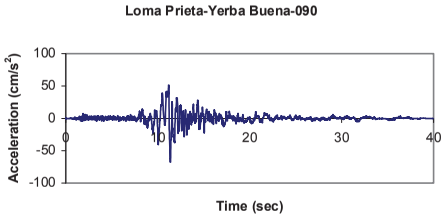


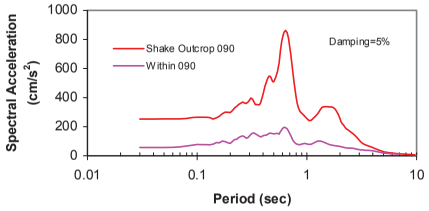


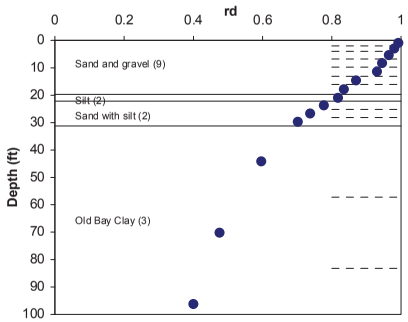


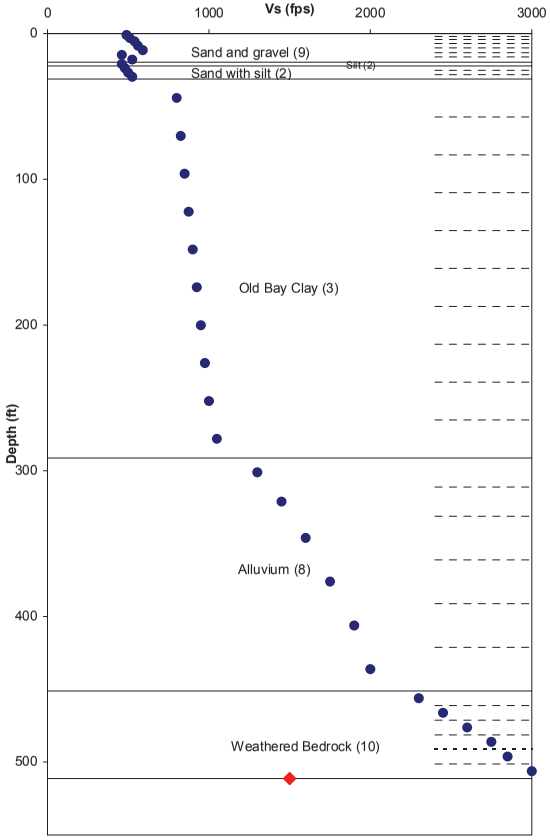


SFOBB-2-Loma Prieta Earthquake (1989)

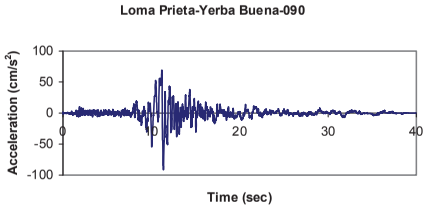


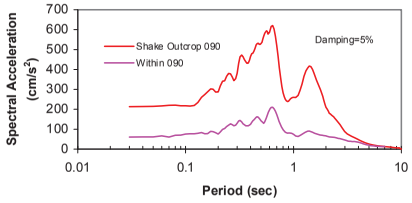


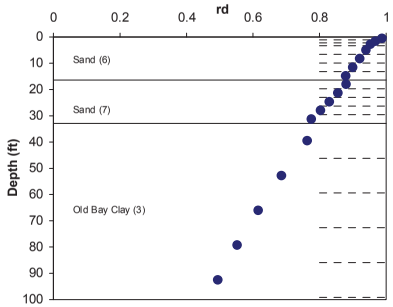


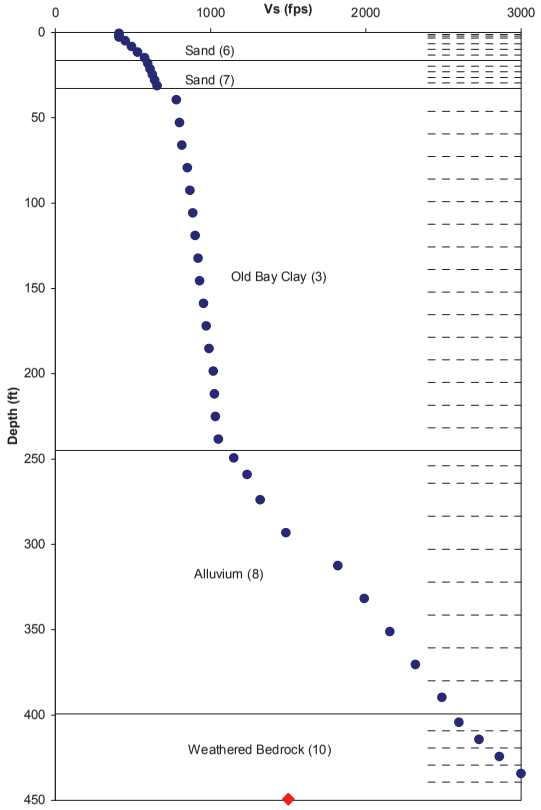


POO7-2-Loma Prieta Earthquake (1989)

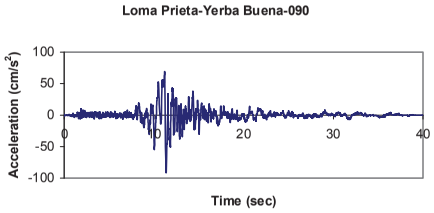


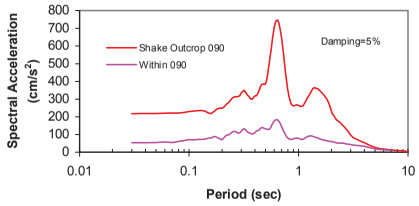


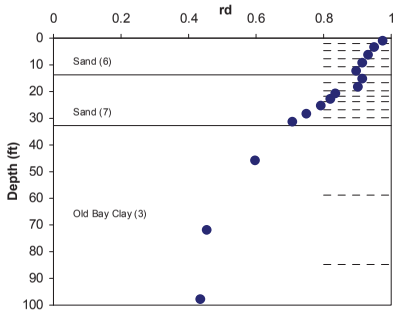


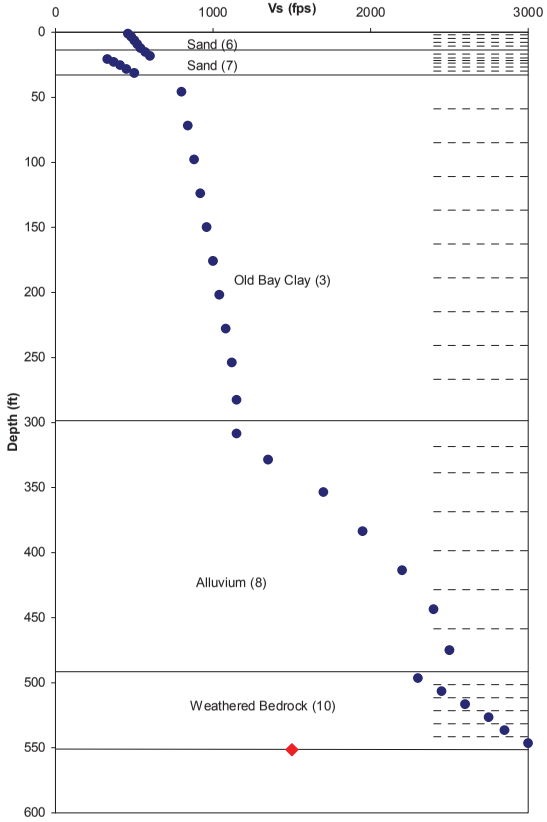


POO7-3-Loma Prieta Earthquake (1989)

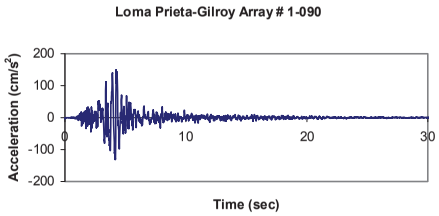


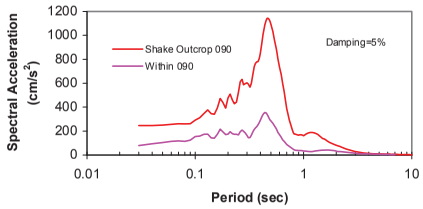


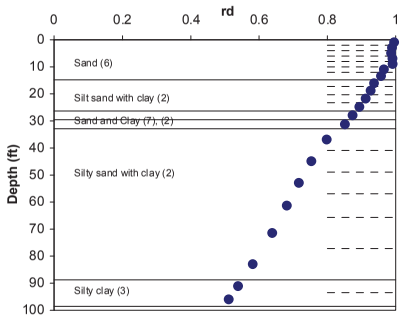


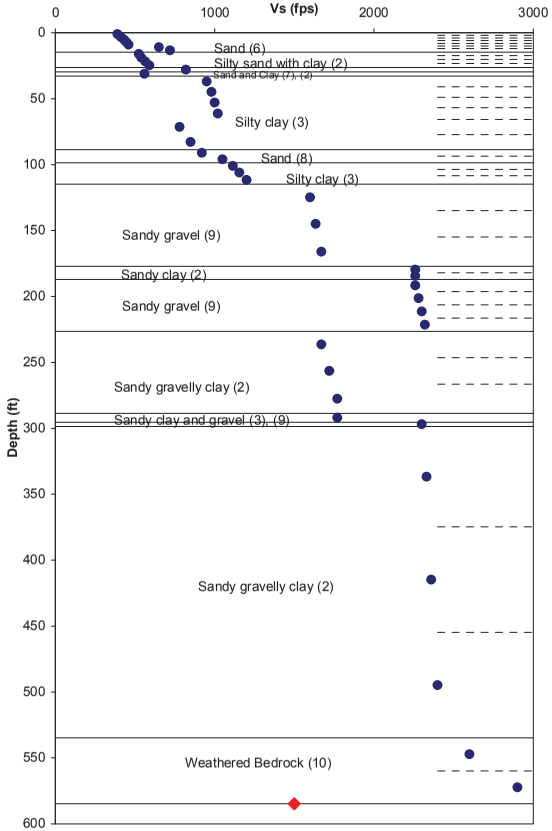


Woodward Marine UC-B4 - Loma Prieta Earthquake (1989)

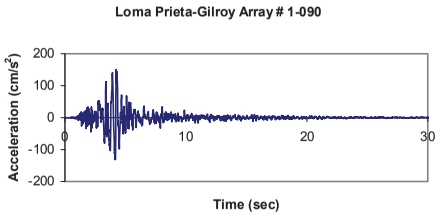


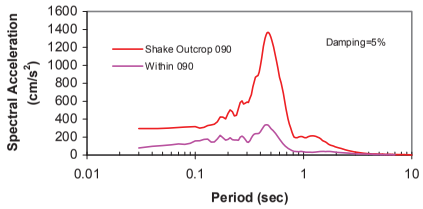


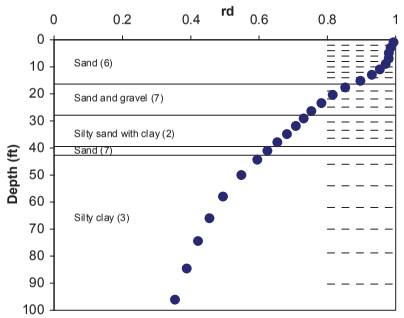


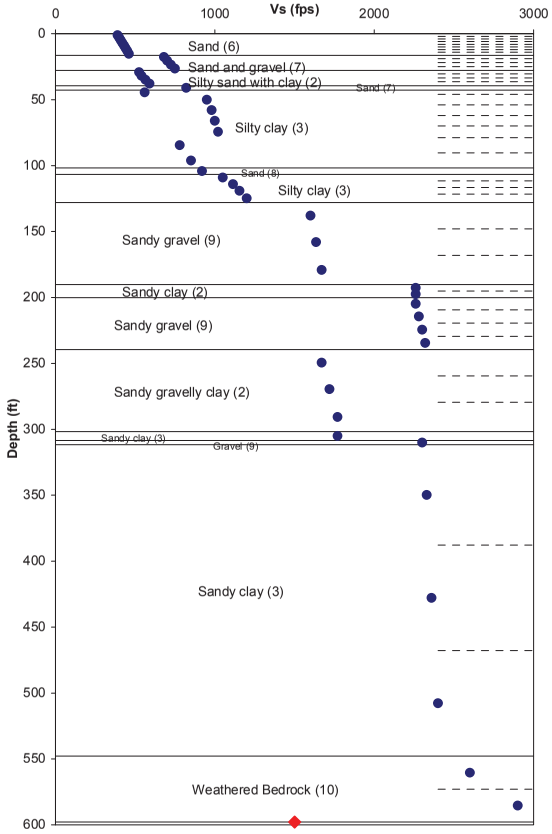


Moss Landing UC-B1 - Loma Prieta Earthquake (1989)

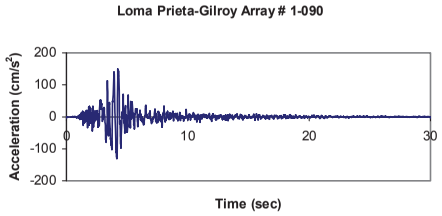


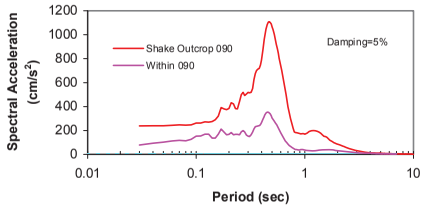


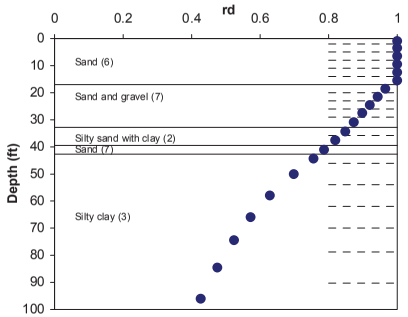


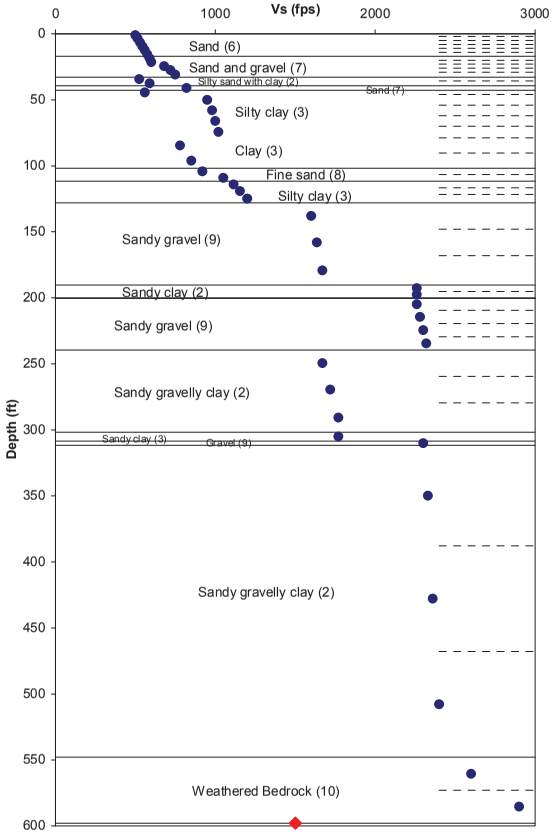


Moss Landing UC-B2 - Loma Prieta Earthquake (1989)

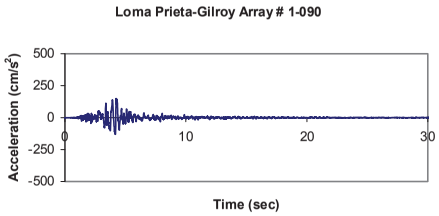


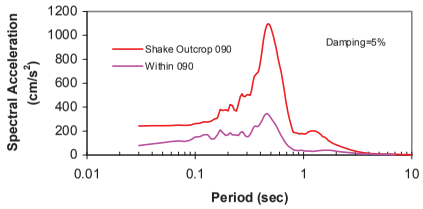


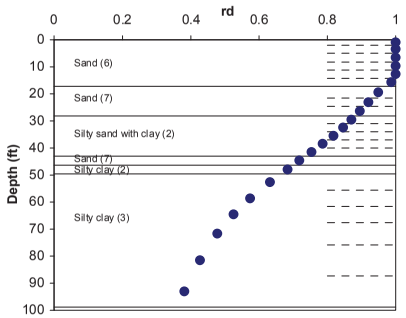


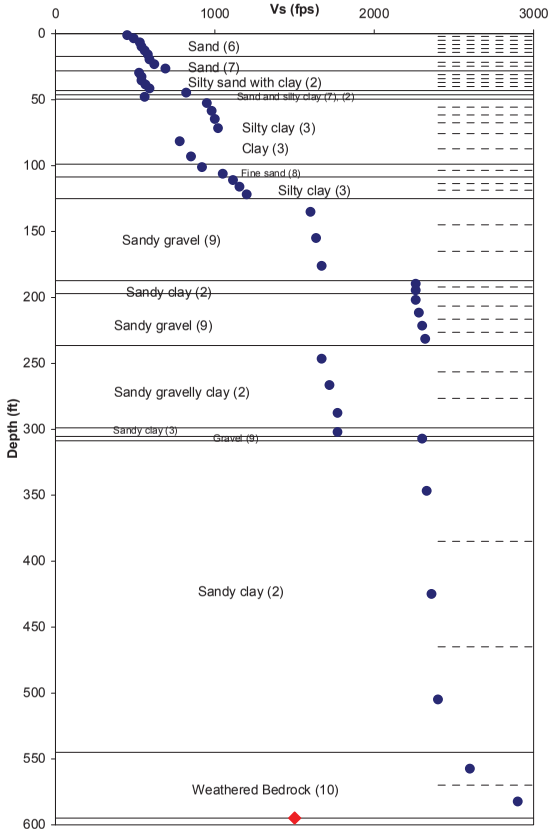


Marine Lab. B1 - Loma Prieta Earthquake (1989)

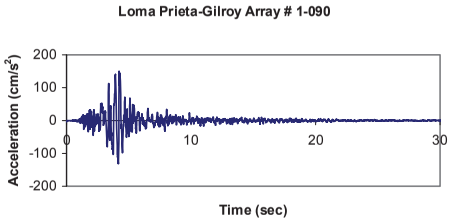


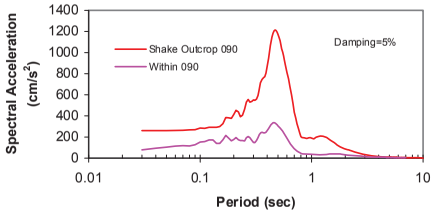


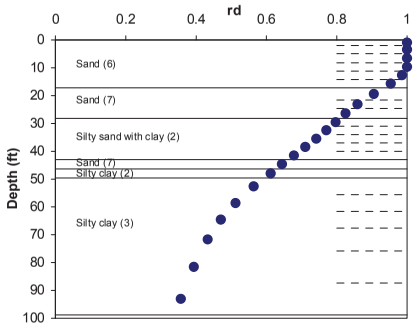


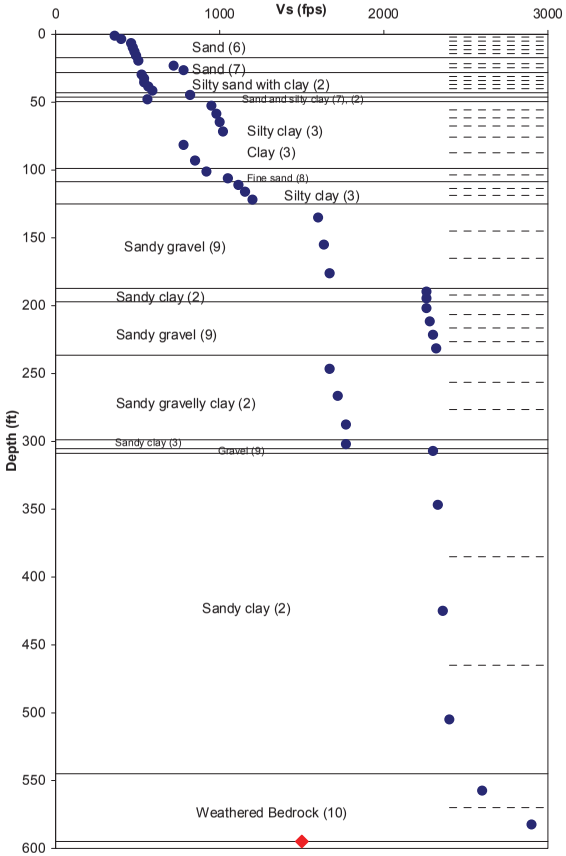


Marine Lab. B2 - Loma Prieta Earthquake (1989)

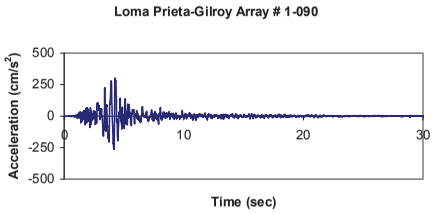


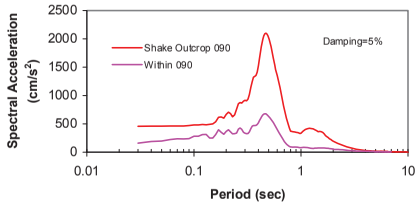


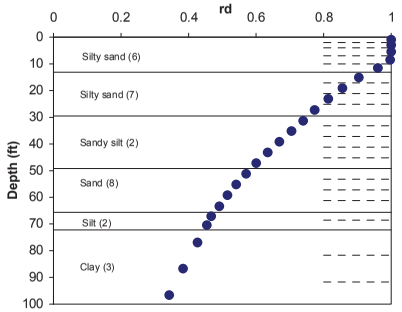


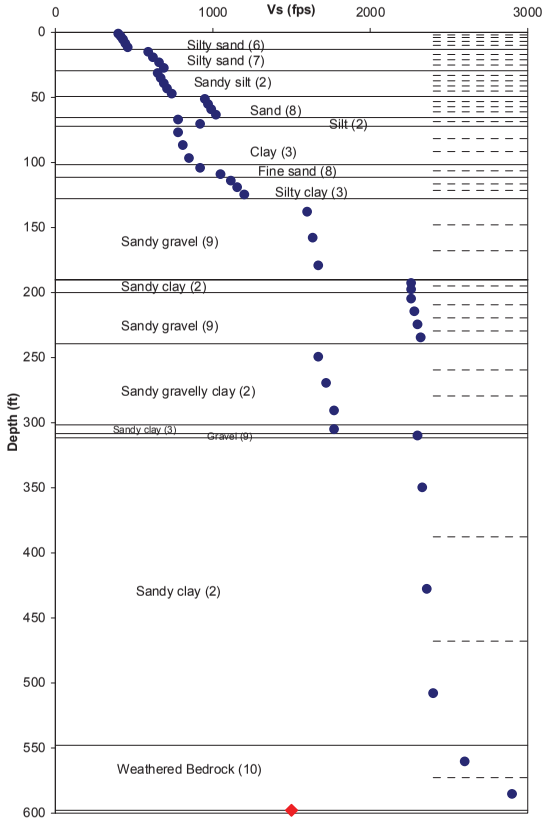


Miller Farm CMF 3 - Loma Prieta Earthquake (1989)

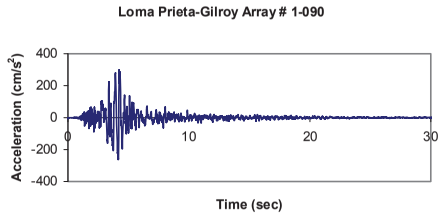


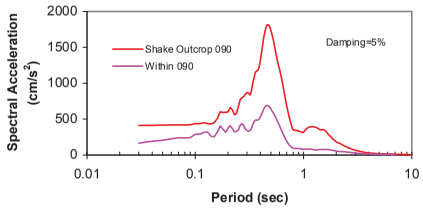


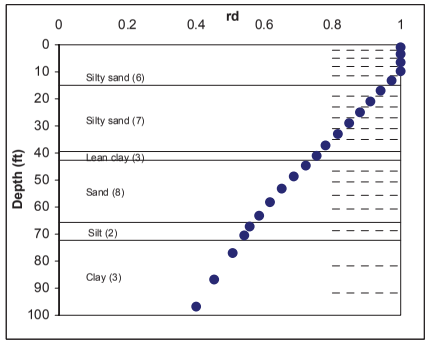


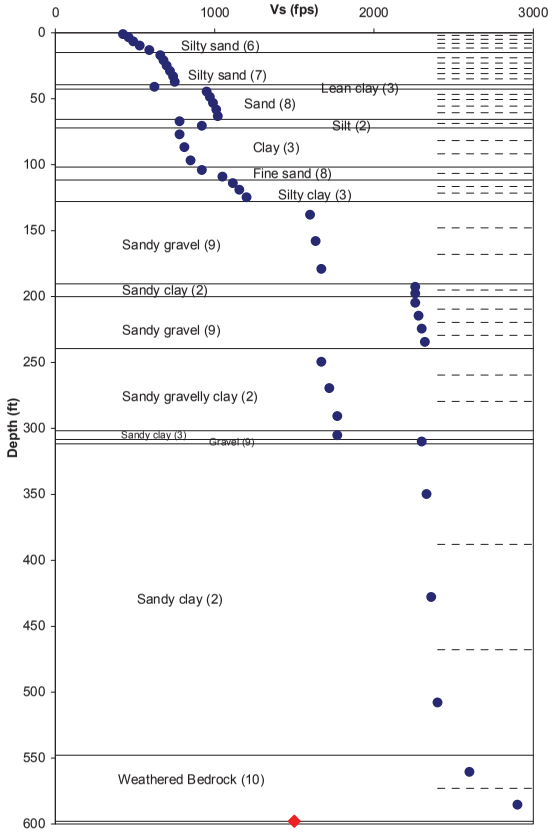


Miller Farm CMF 5 - Loma Prieta Earthquake (1989)

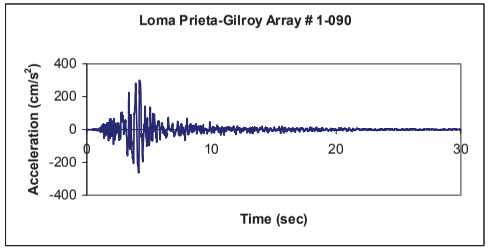


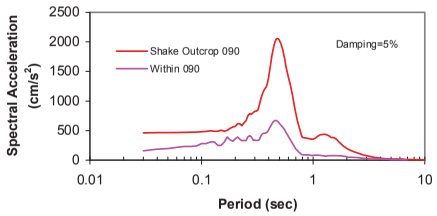


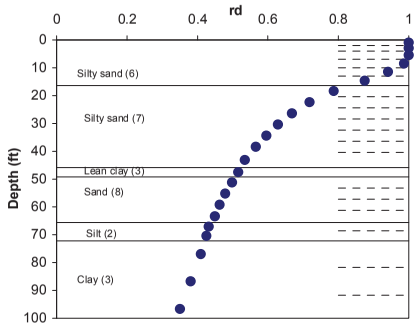


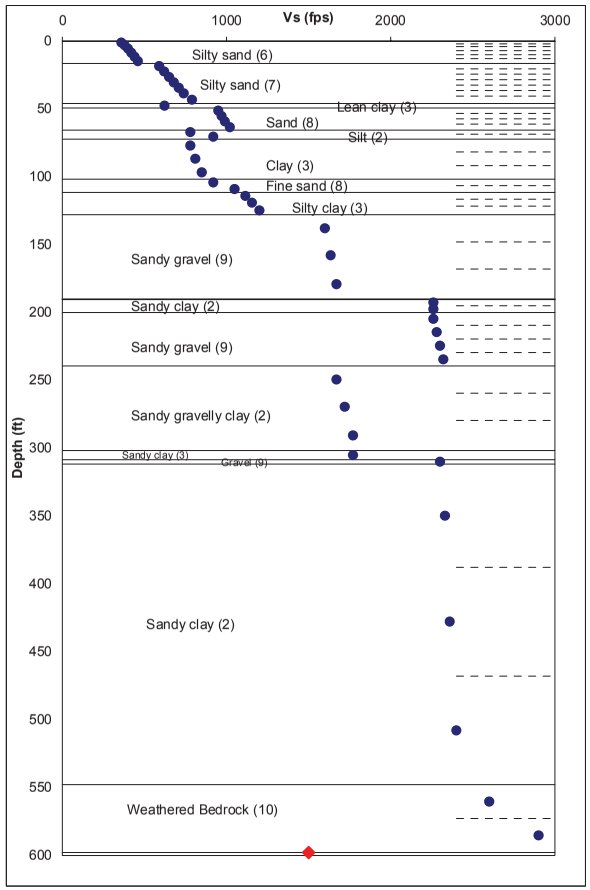


Miller Farm CMF 8 - Loma Prieta Earthquake (1989)

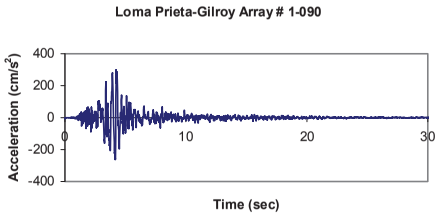


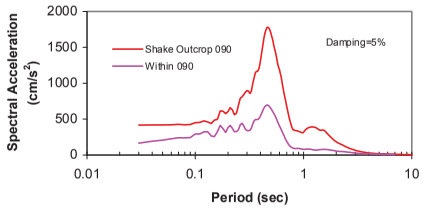


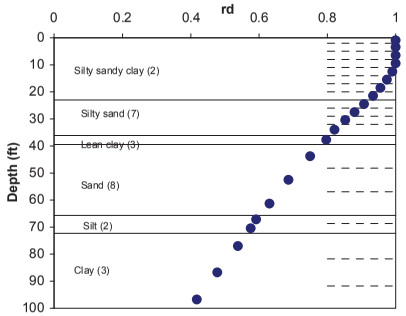


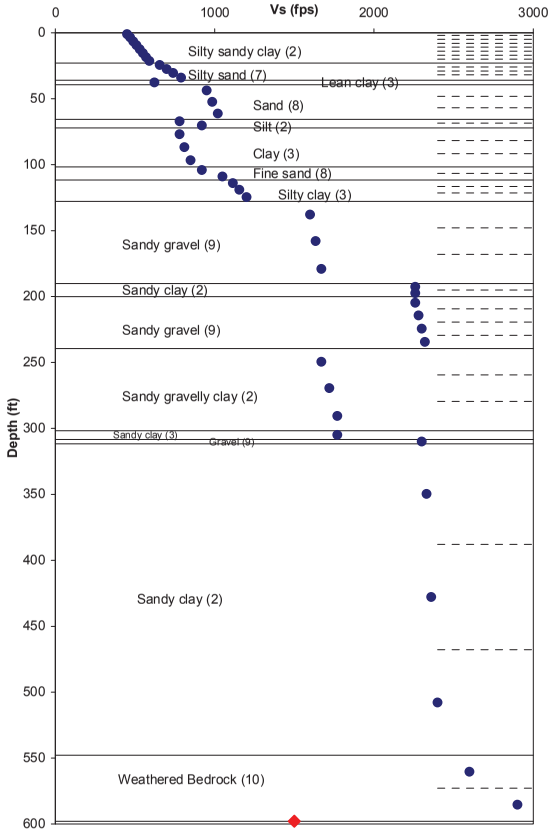


Miller Farm CMF 10 - Loma Prieta Earthquake (1989)

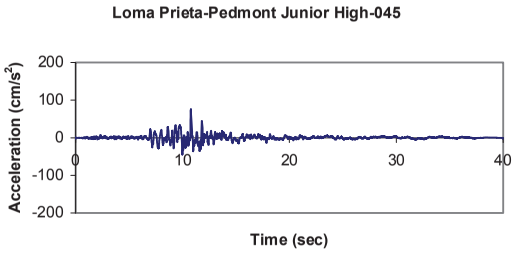


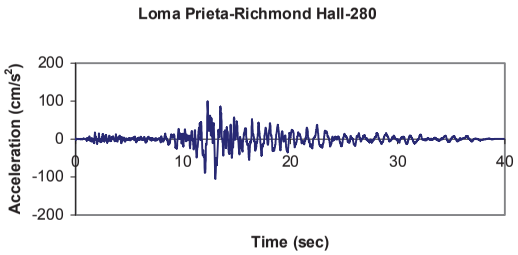


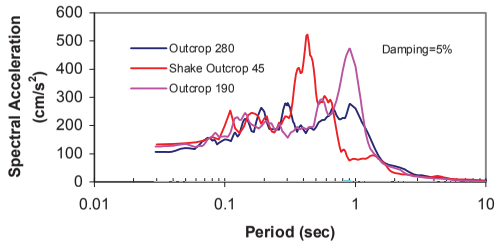


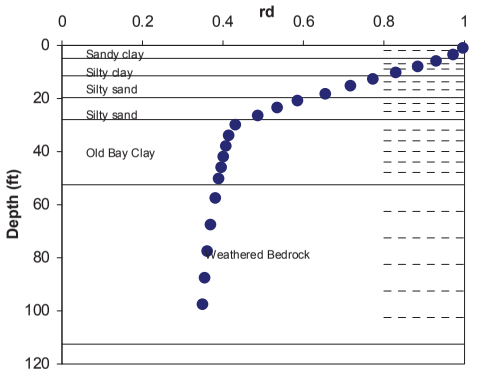


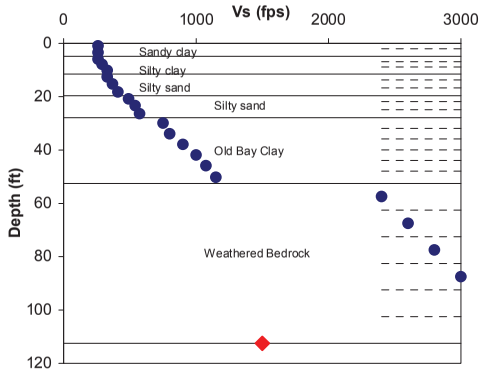
Richmond City Hall - Loma Prieta Earthquake (1989)



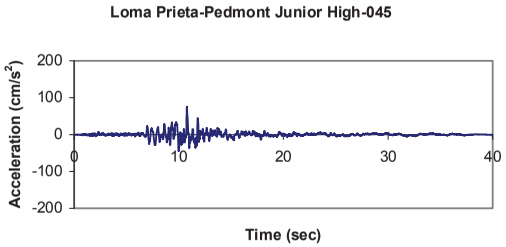


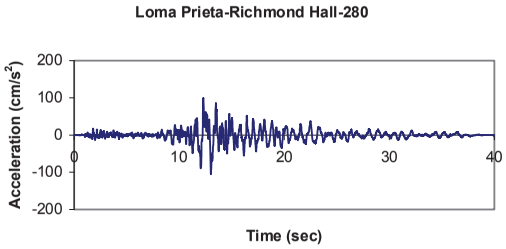


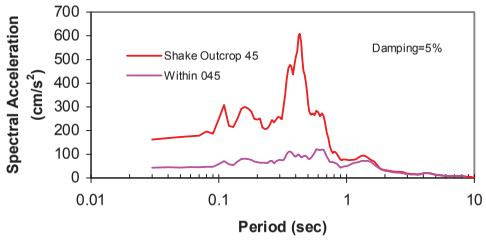


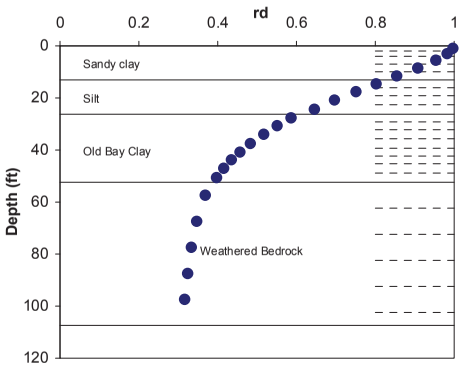


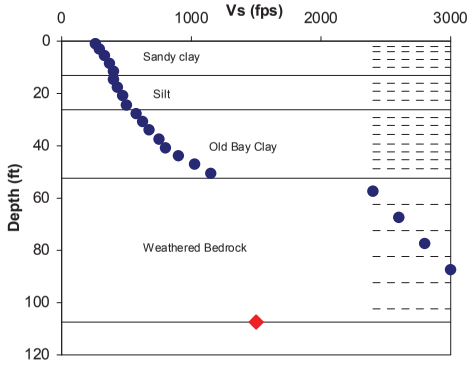
Port of Richmond POR-2 - Loma Prieta Earthquake (1989)



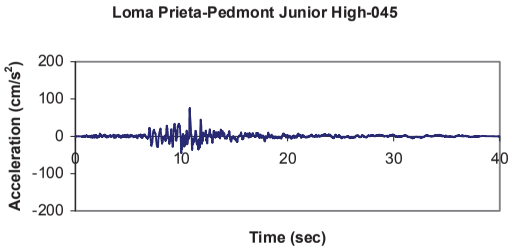


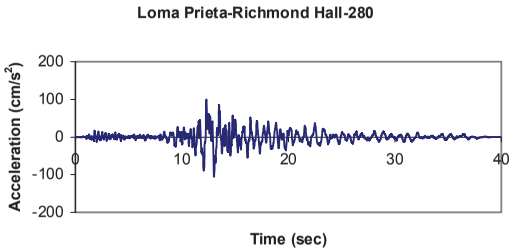


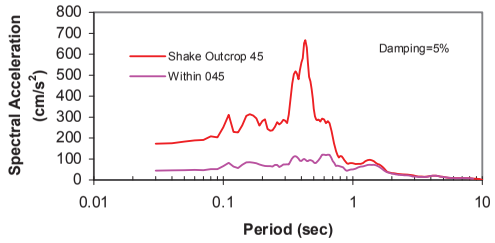


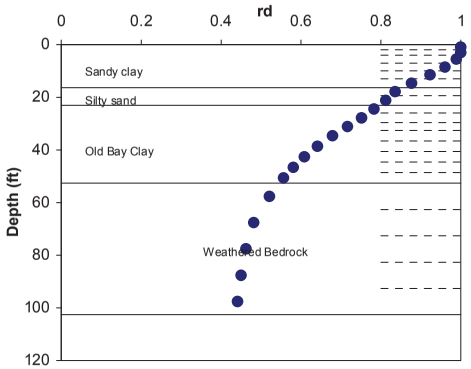


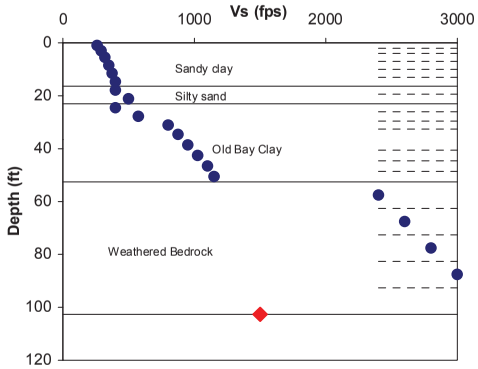
Port of Richmond POR-3 - Loma Prieta Earthquake (1989)



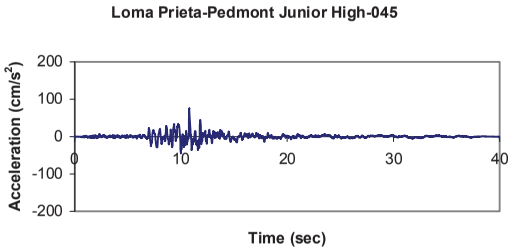


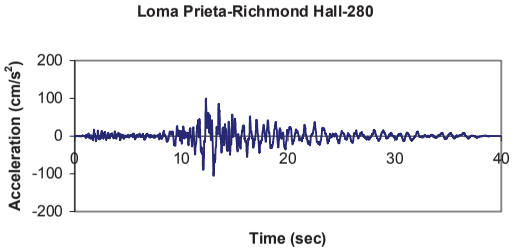


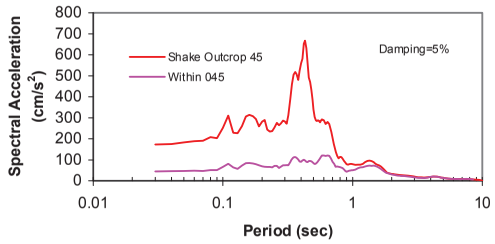


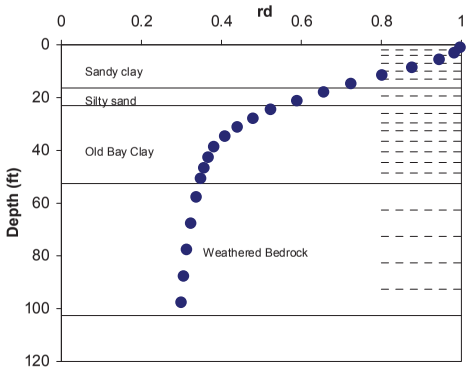


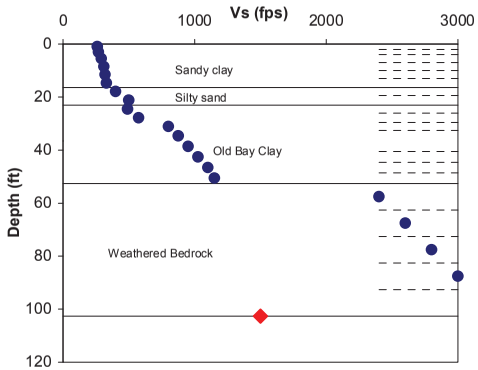
Port of Richmond POR-4 - Loma Prieta Earthquake (1989)



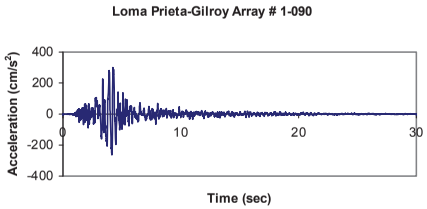


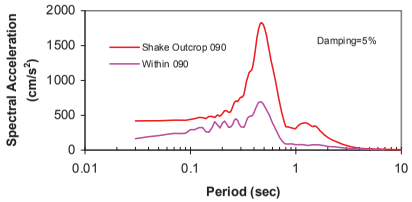


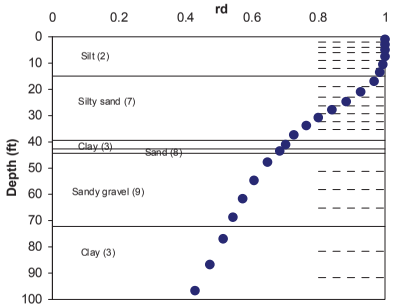


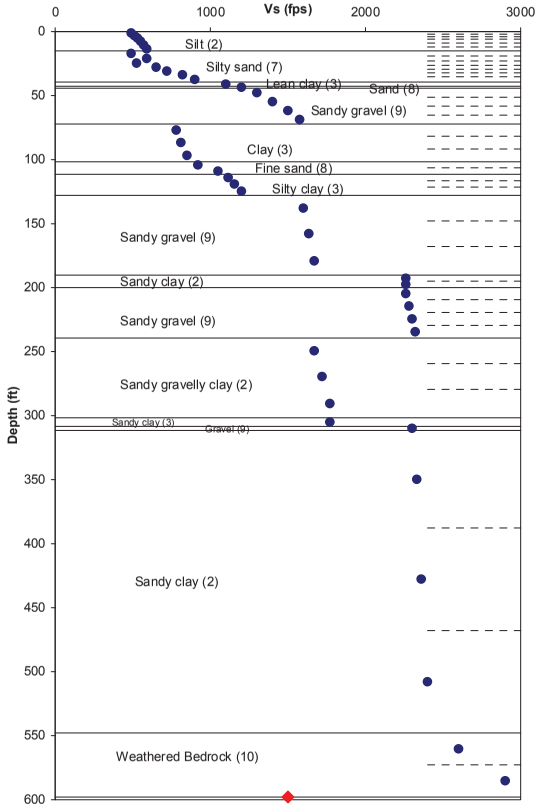


Miller Farm - Loma Prieta Earthquake (1989)

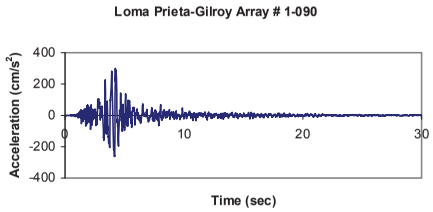


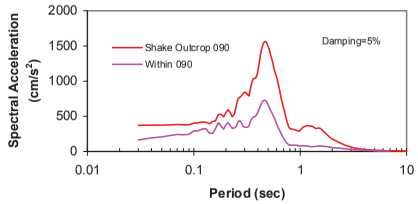


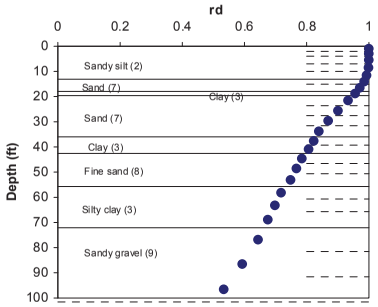


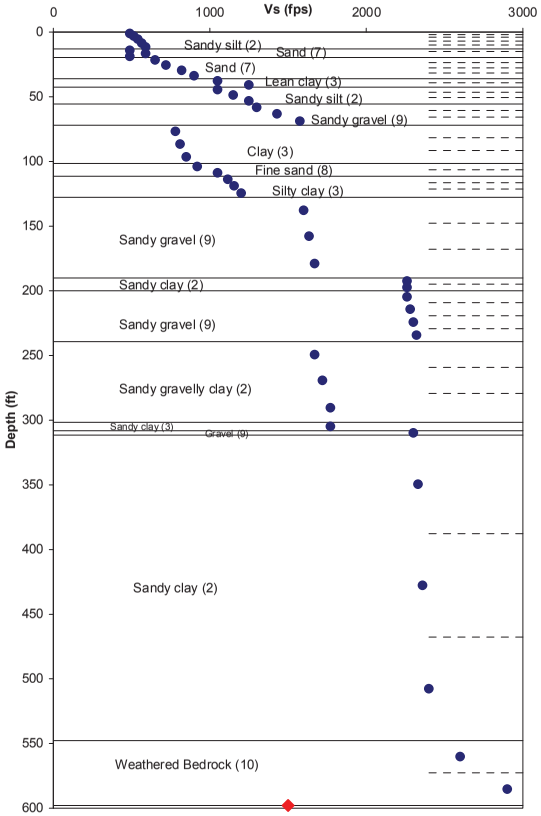


Farris Farm - Loma Prieta Earthquake (1989)

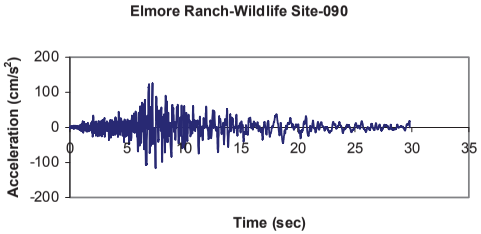


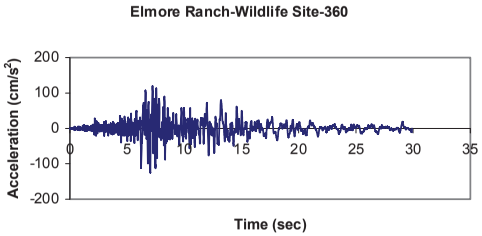


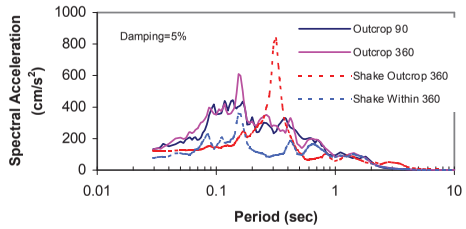


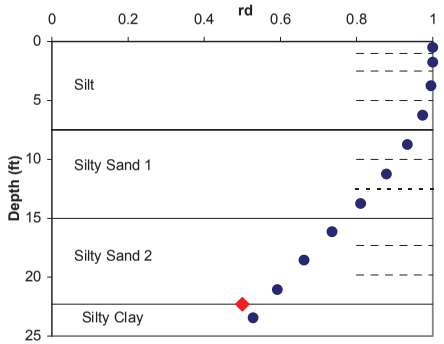


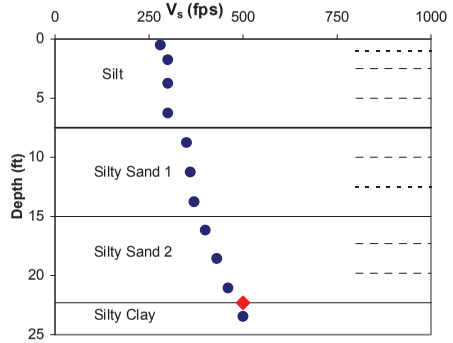
Wildlife Site – Elmore Ranch (1987)



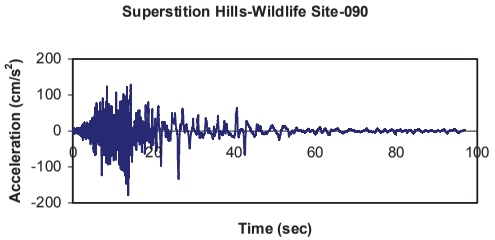


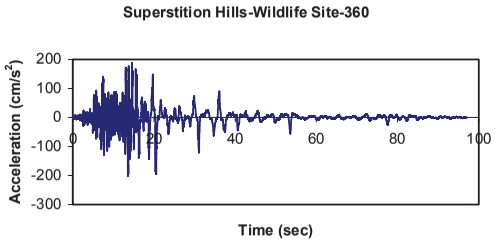


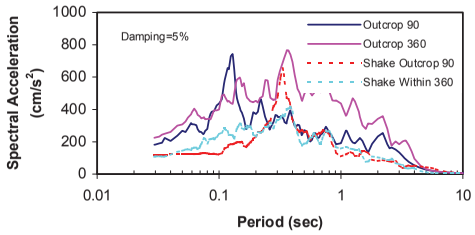


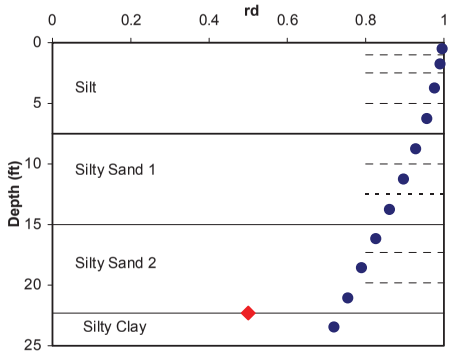


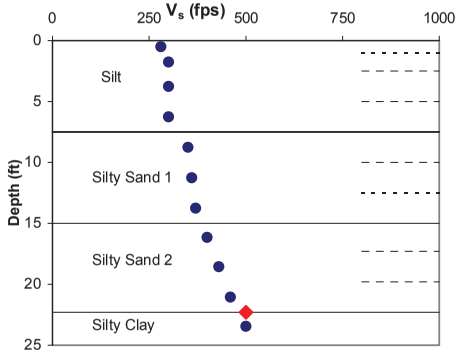
Wildlife Site – Superstition Hills (1987)



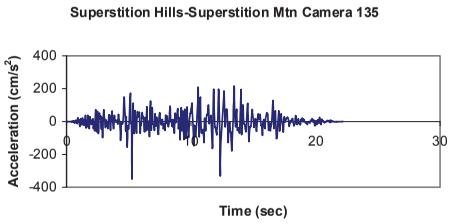


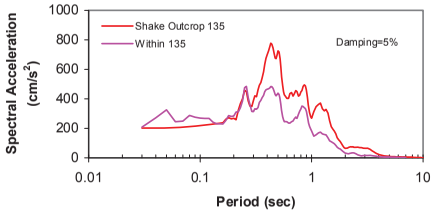


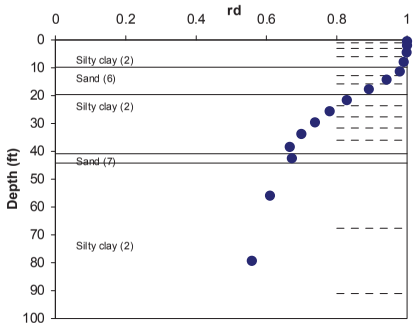


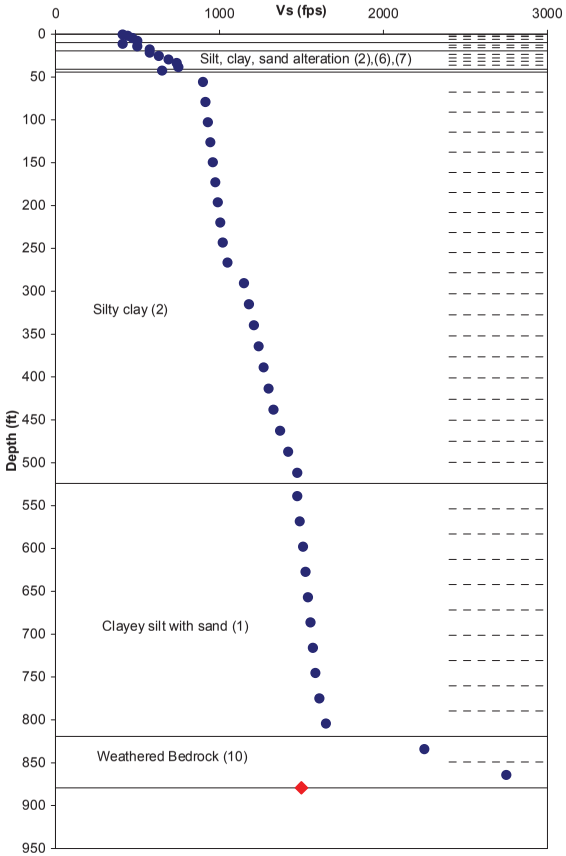


Radio Tower B1 – Superstition Hills (1987)

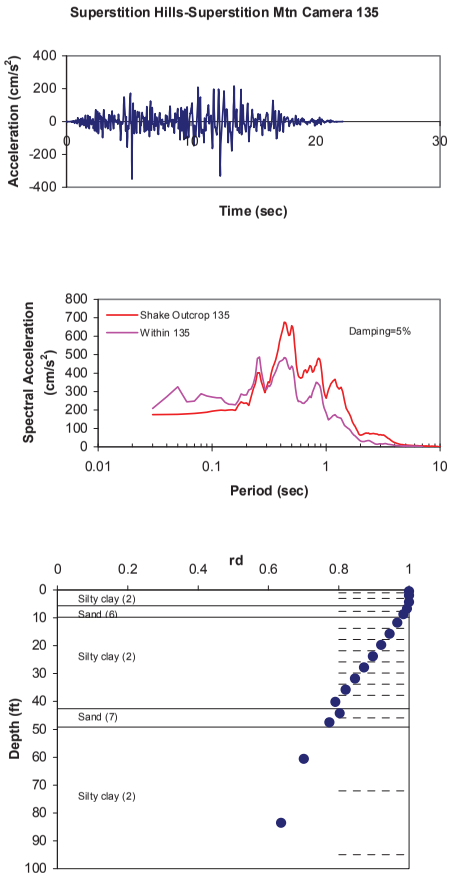


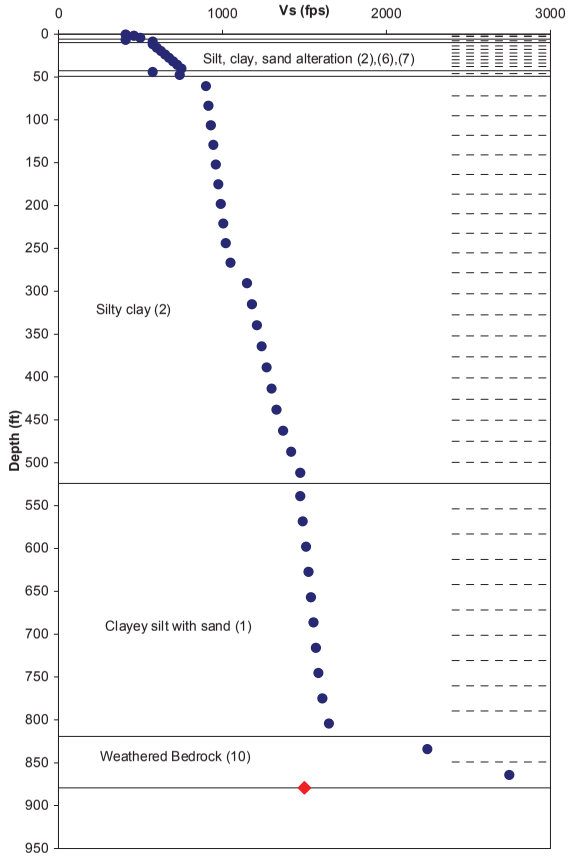




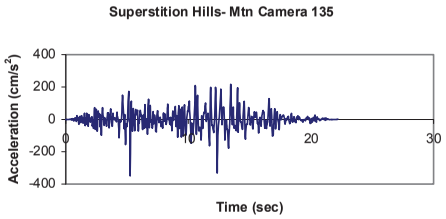


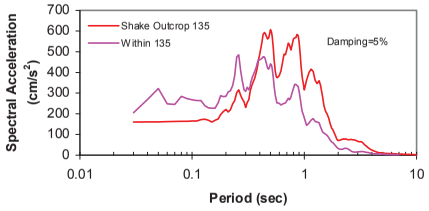
Radio Tower B2 – Superstition Hills (1987)

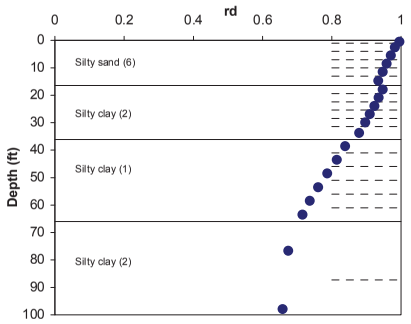


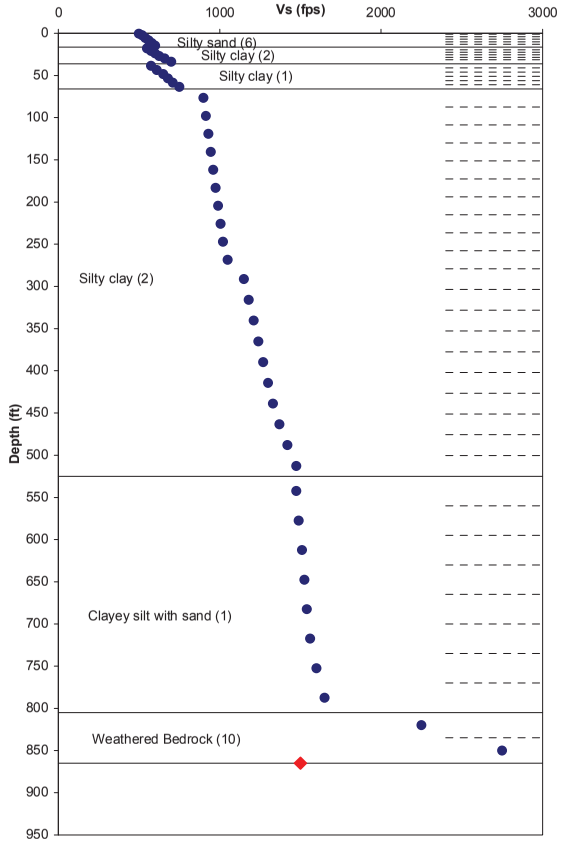


McKim Ranch A – Superstition Hills (1987)

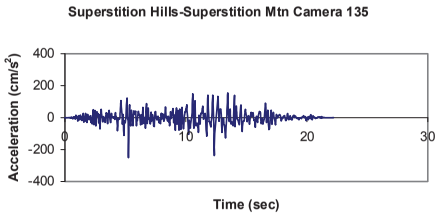


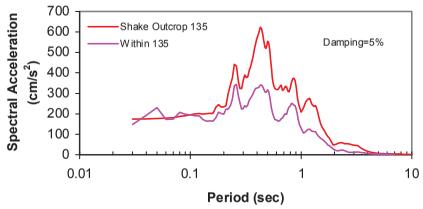


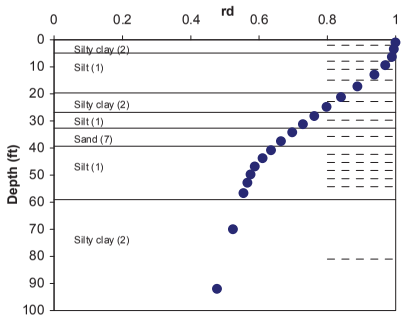


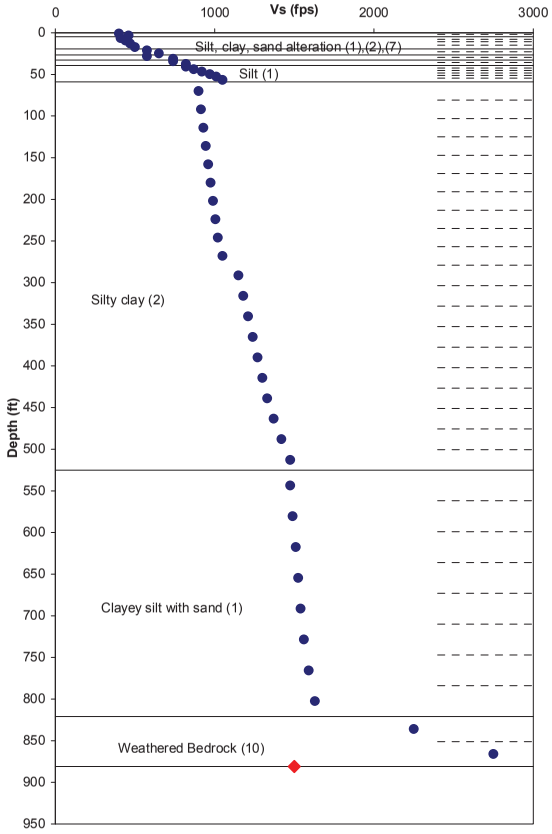


Kornbloom – Superstition Hills (1987)

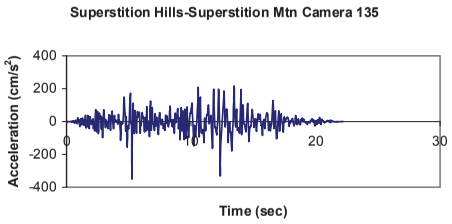


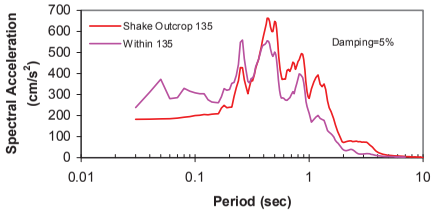


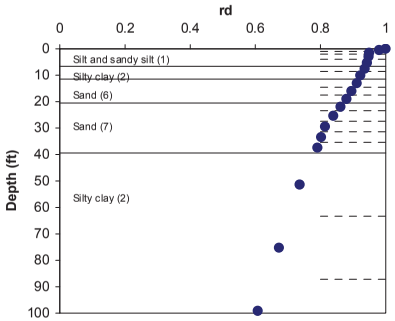


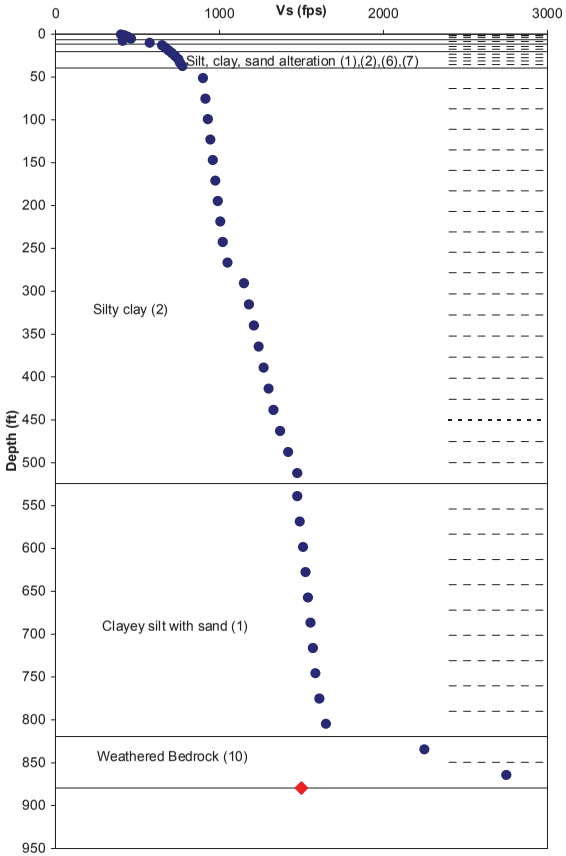


River Park A & C – Superstition Hills (1987)

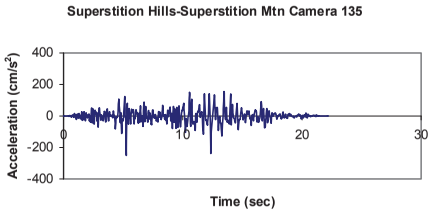


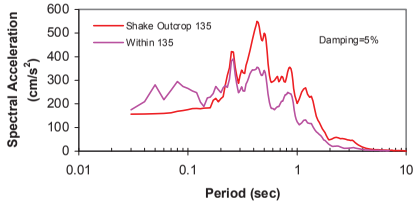


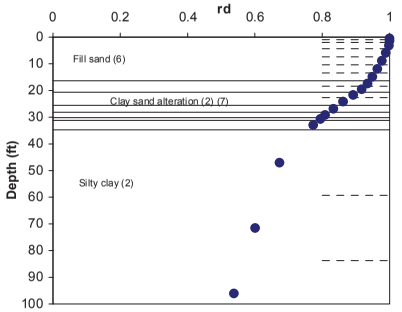


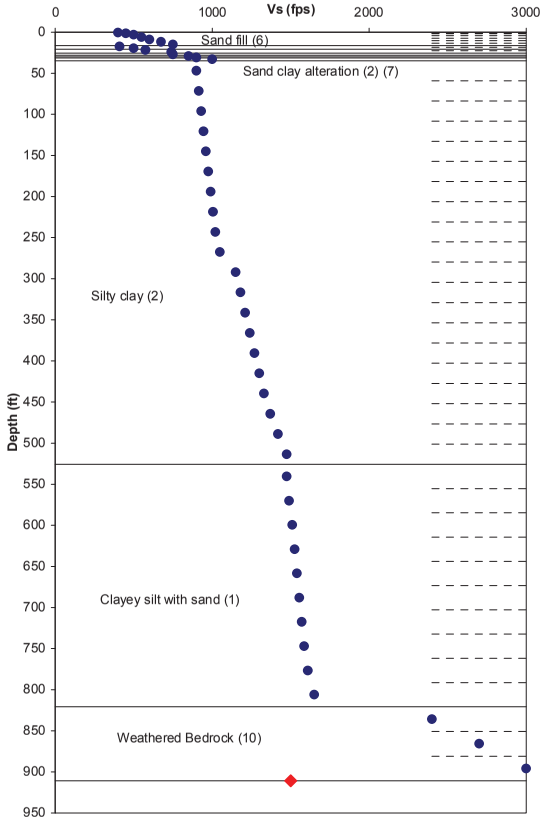


Heber Road A1 – Superstition Hills (1987)

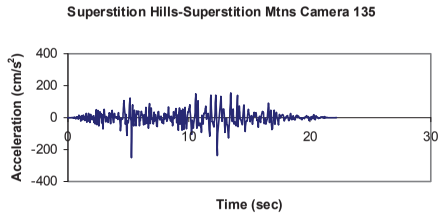


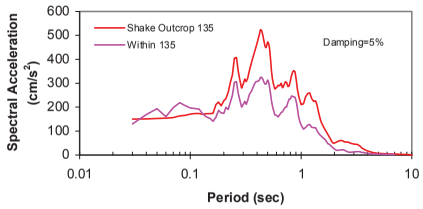


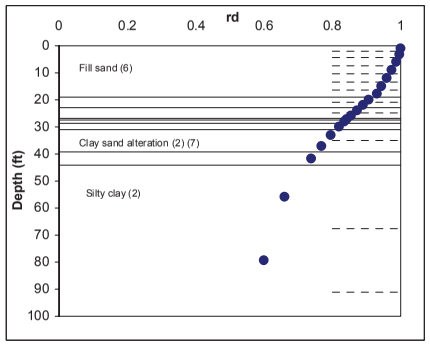


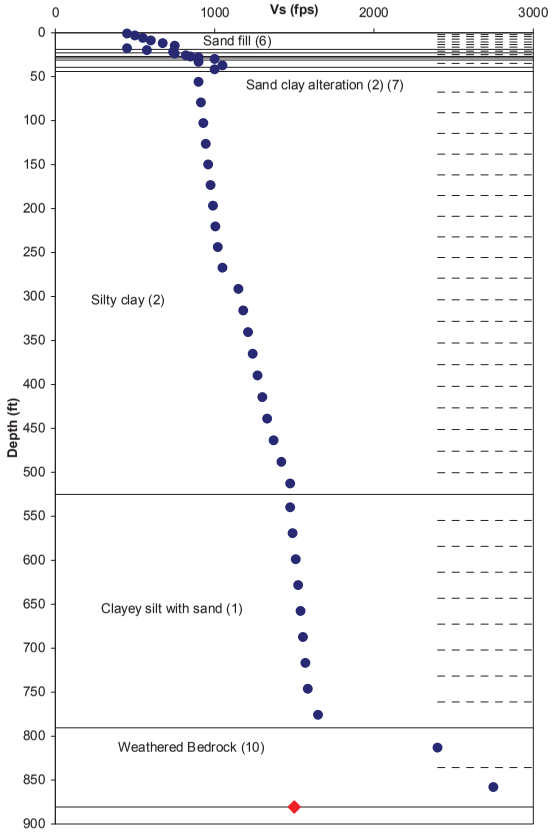


Heber Road A2 – Superstition Hills (1987)

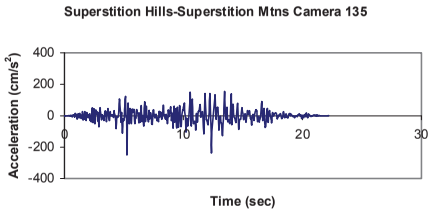


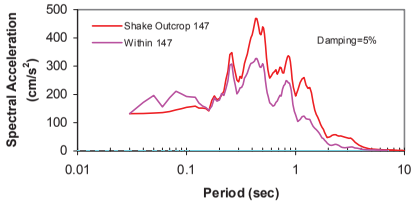


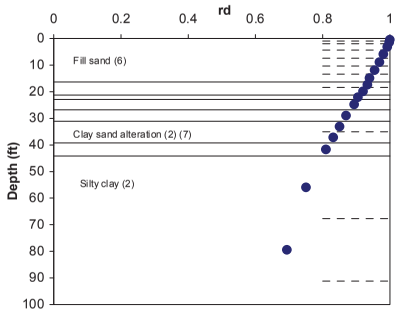


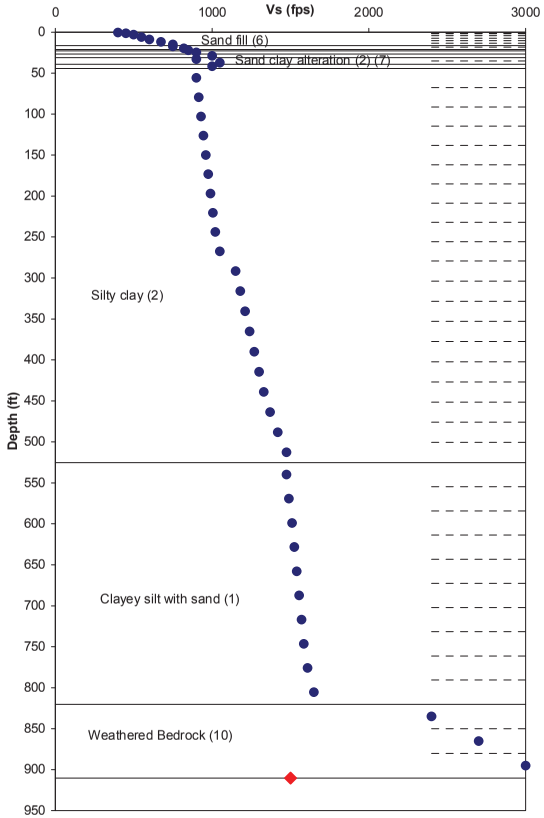


Heber Road A3 – Superstition Hills (1987)

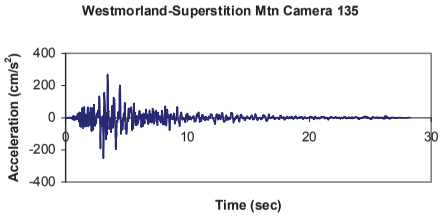


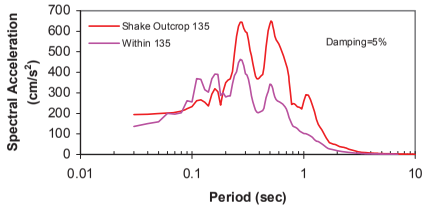


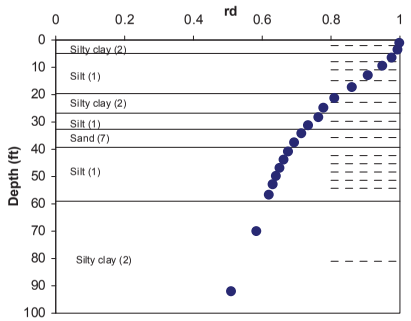


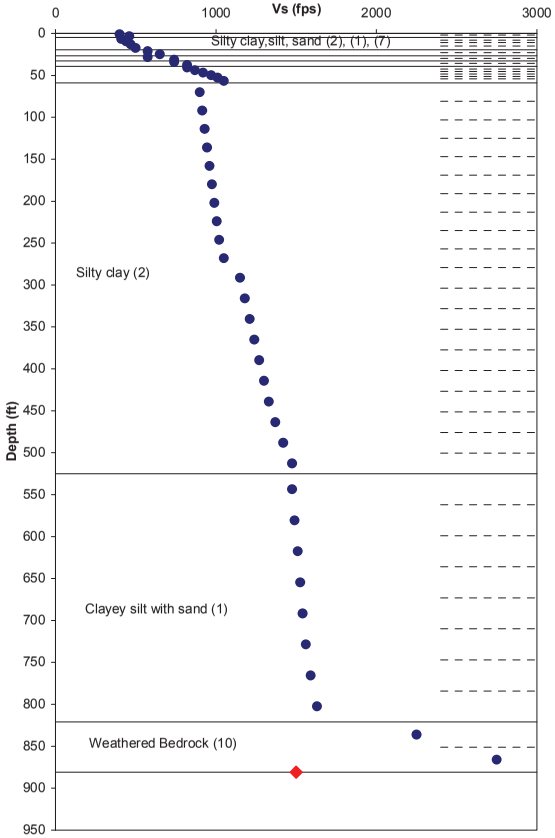


Kornbloom – Westmorland (1981)

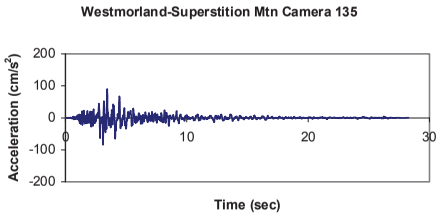


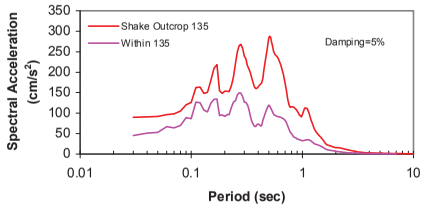


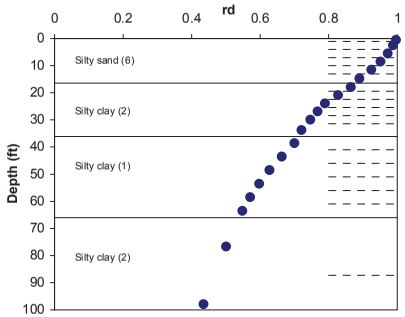


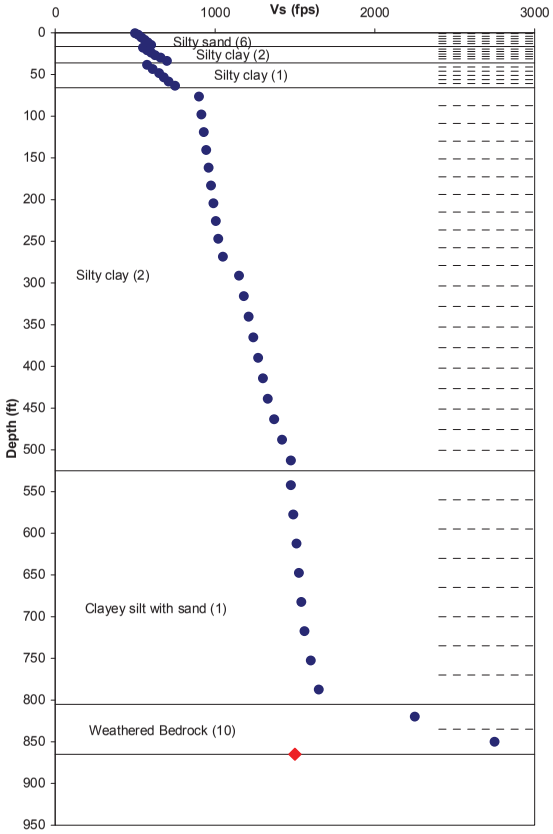


McKim Ranch – Westmorland (1981)

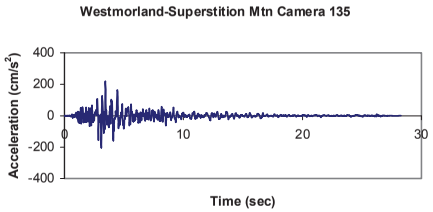


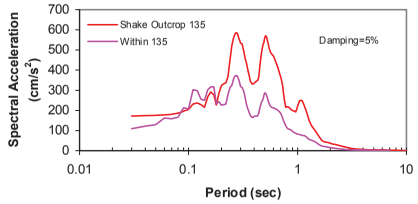


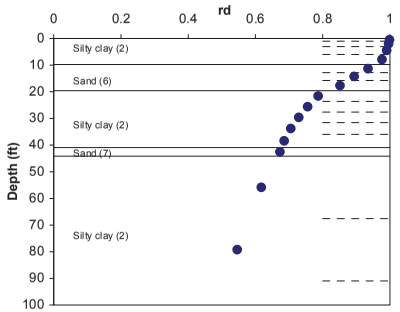


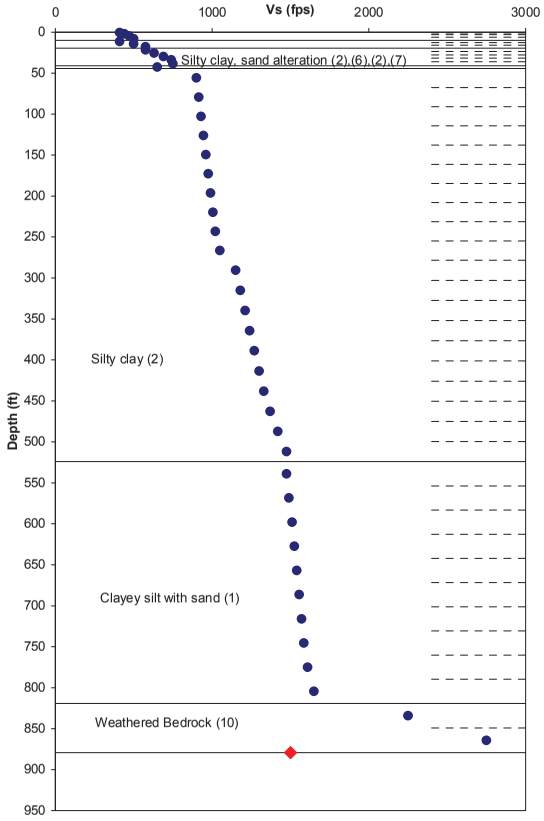


Radio Tower B1 – Westmorland (1981)

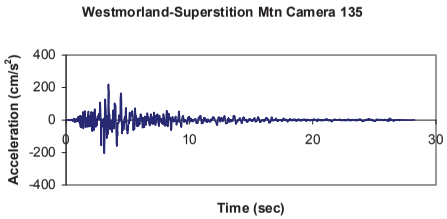


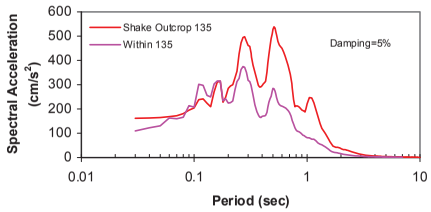


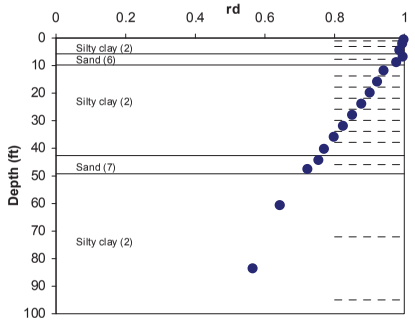


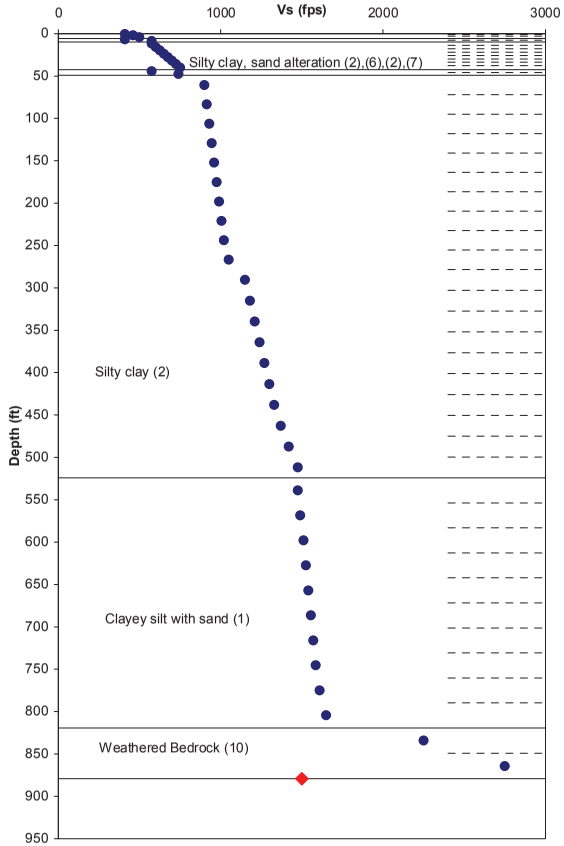


Radio Tower B2 – Westmorland (1981)

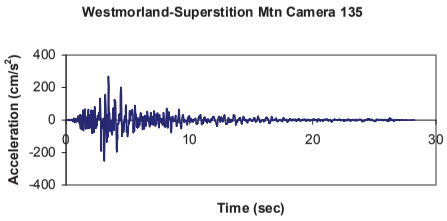


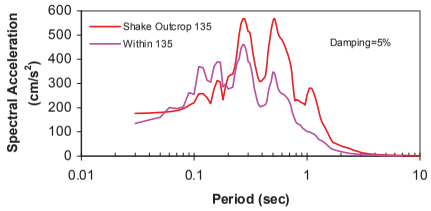


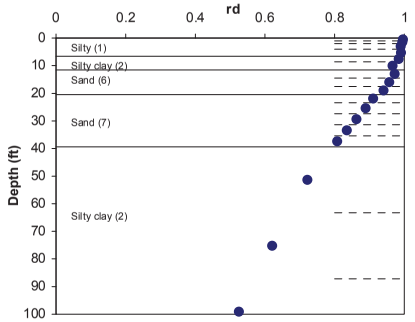


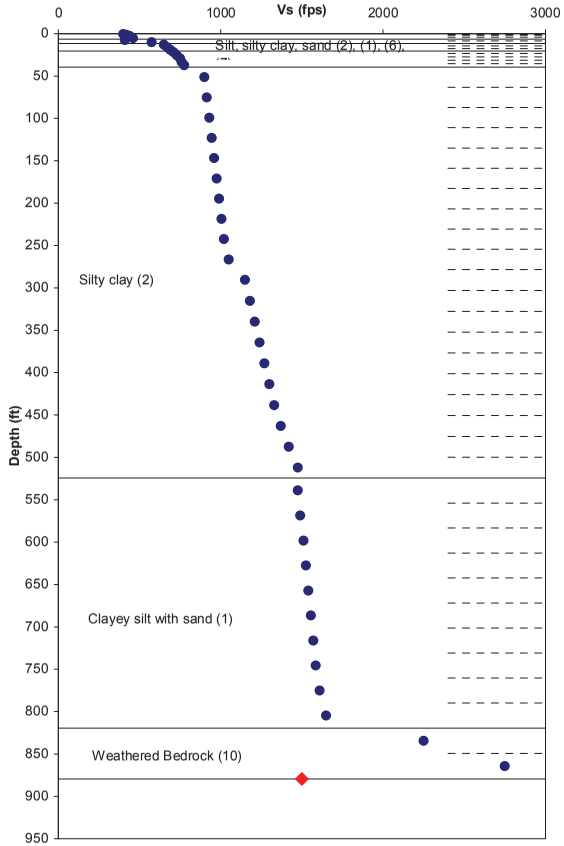


River Park A & C – Westmorland (1981)

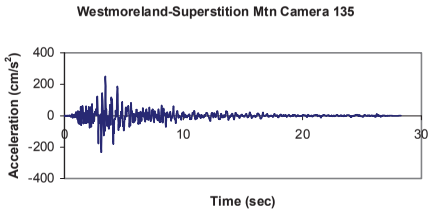


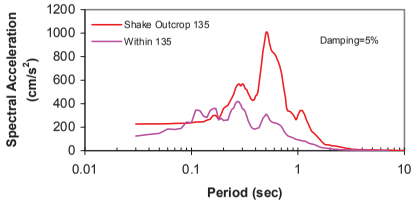


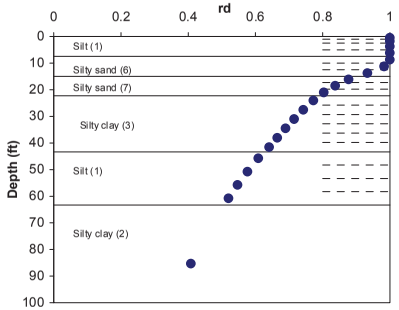


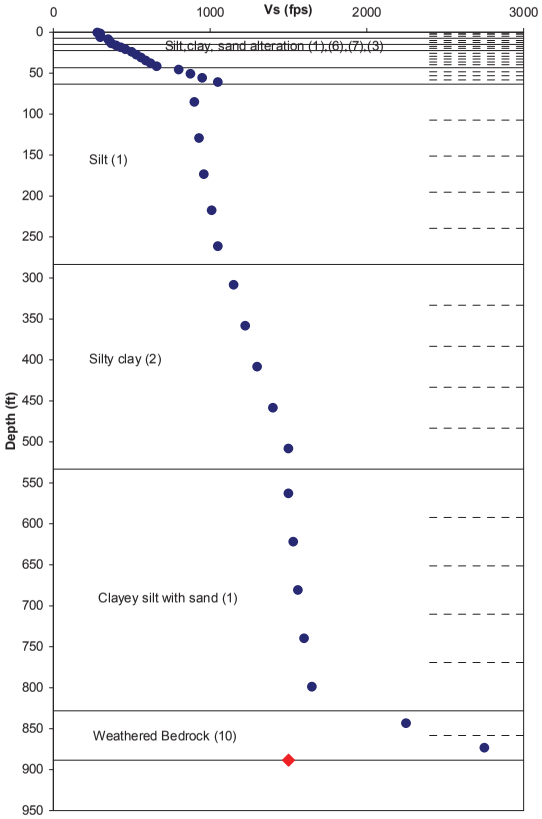


Wildlife – Westmorland (1981)

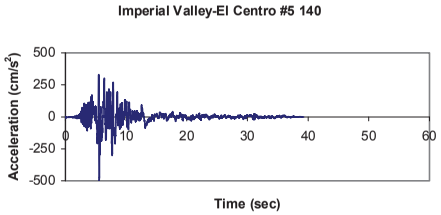


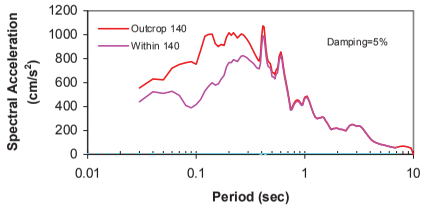


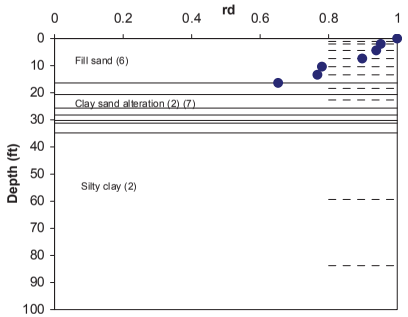


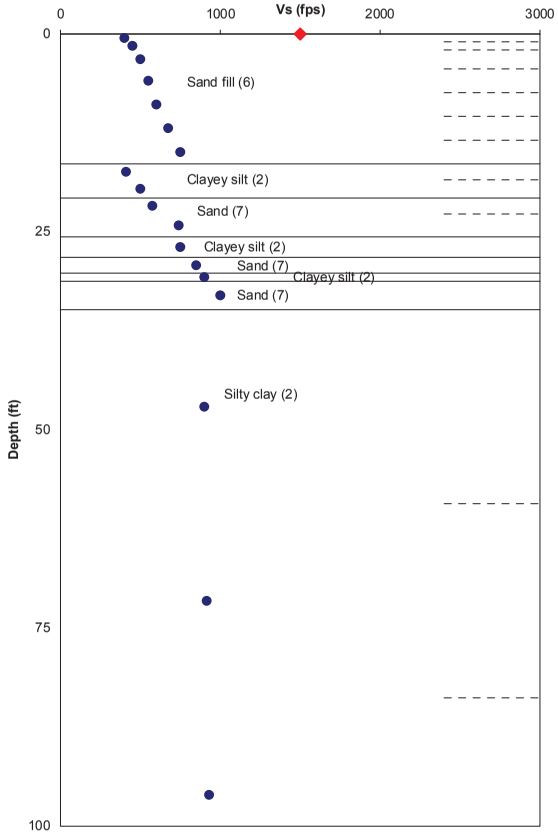


Heber Road A1 – Imperial Valley (1979)

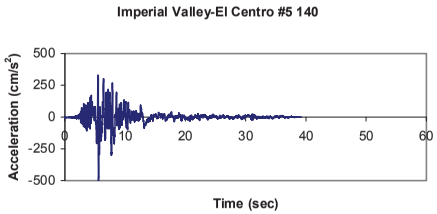


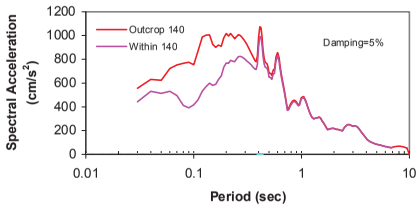


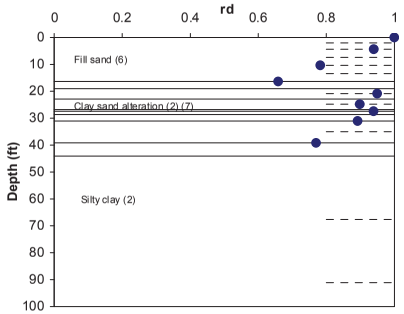


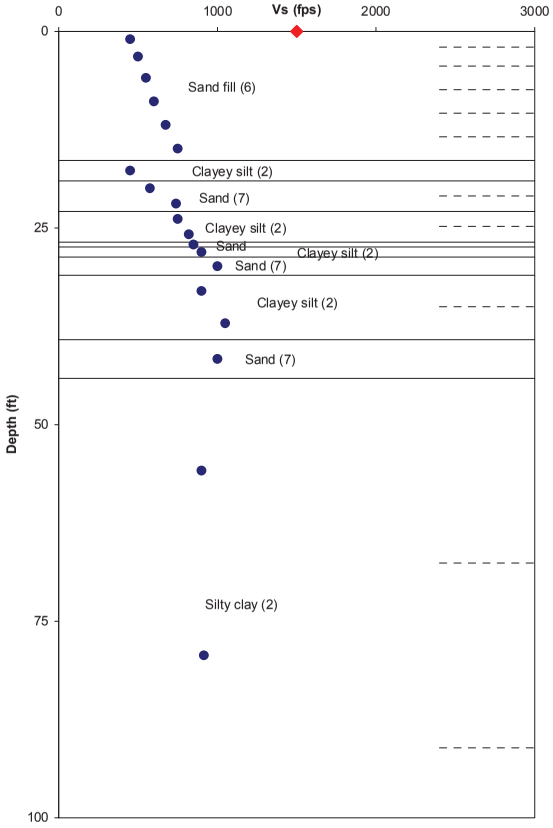


Heber Road A2 – Imperial Valley (1979)

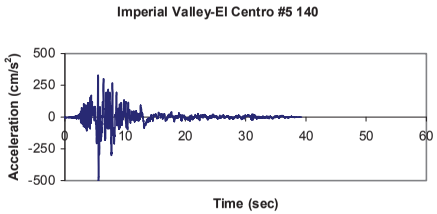


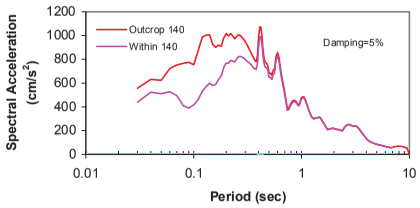


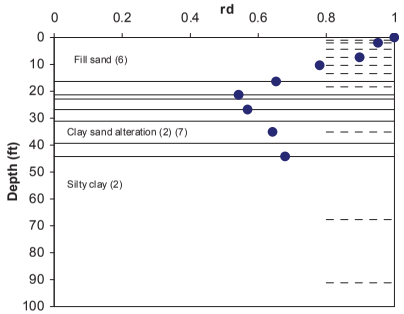


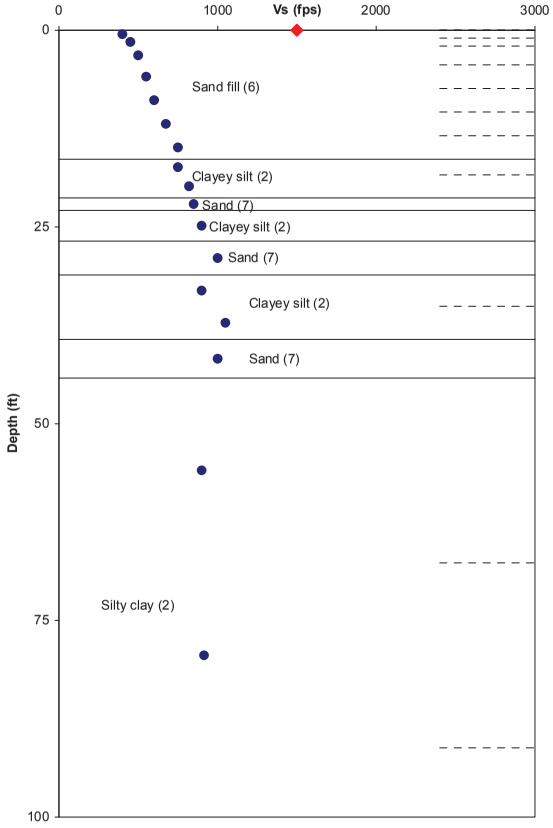


Heber Road A3 – Imperial Valley (1979)

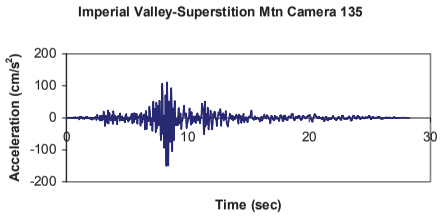


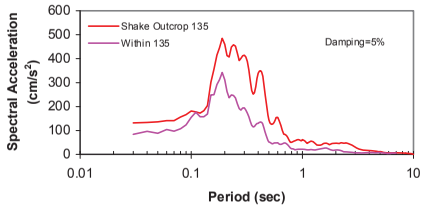


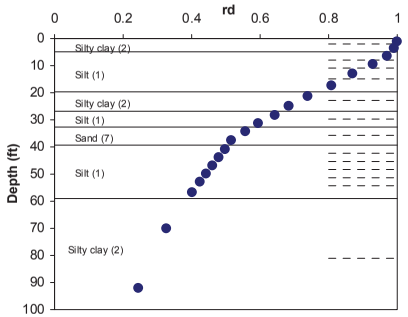


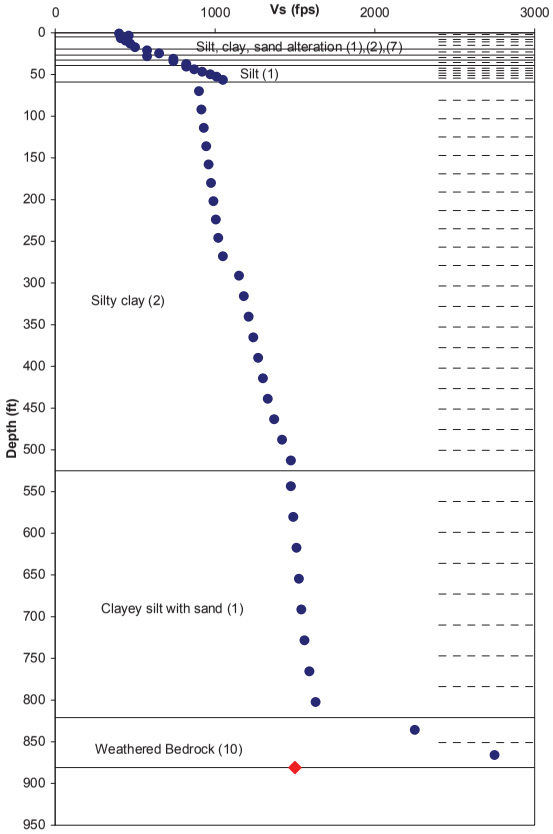


Kornbloom – Imperial Valley (1979)

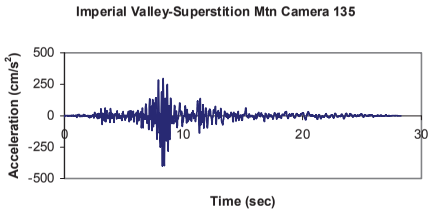


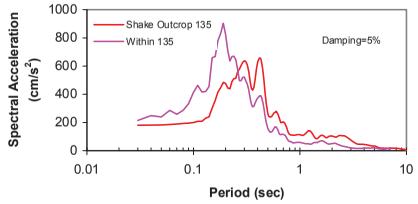


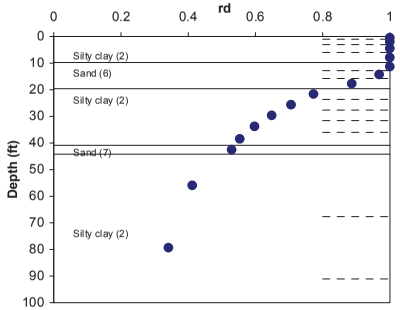


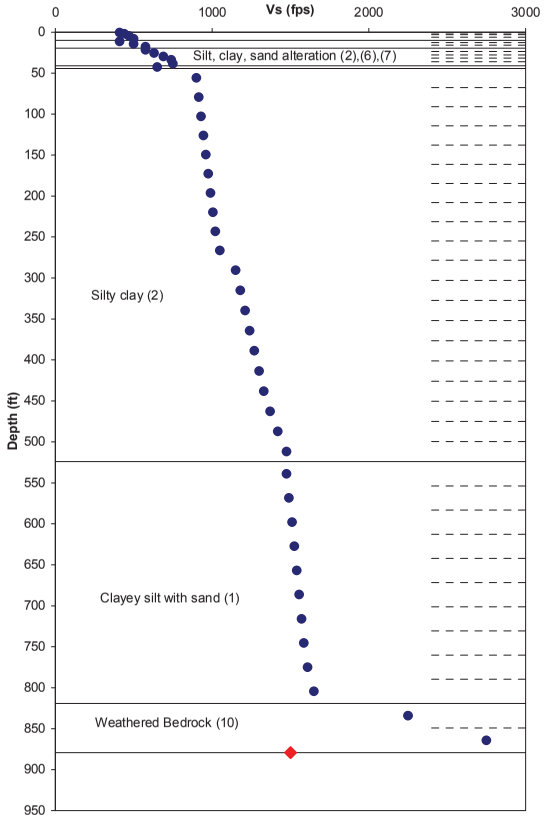


Radio Tower B1 – Imperial Valley (1979)

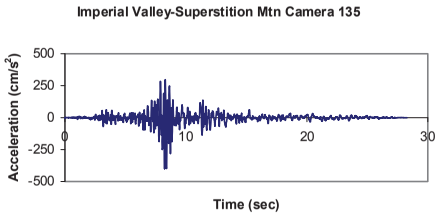


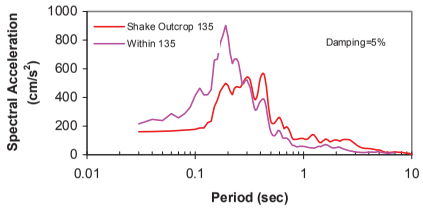


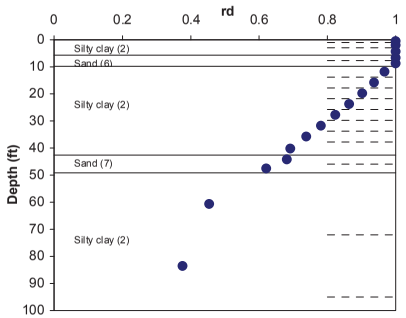


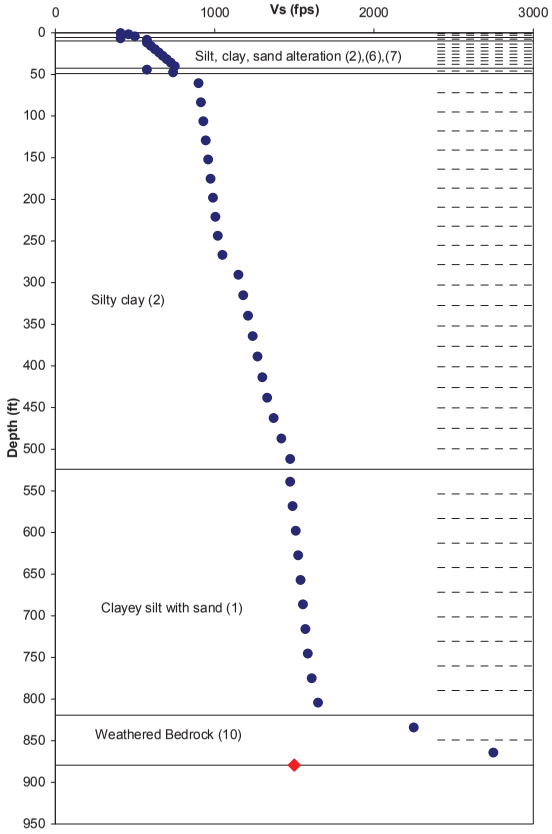


Radio Tower B2 – Imperial Valley (1979)

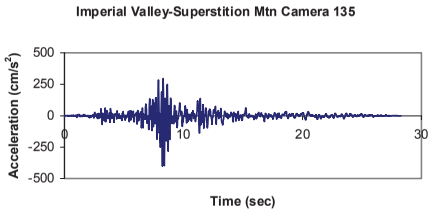


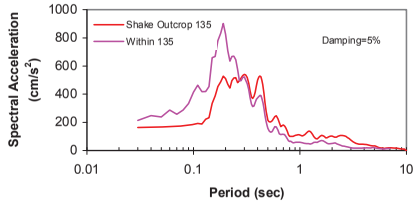


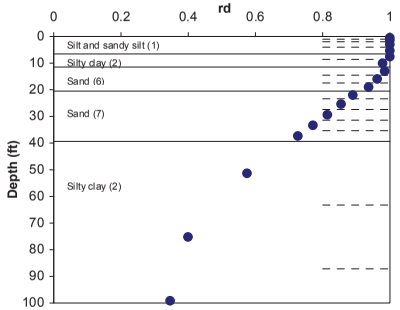


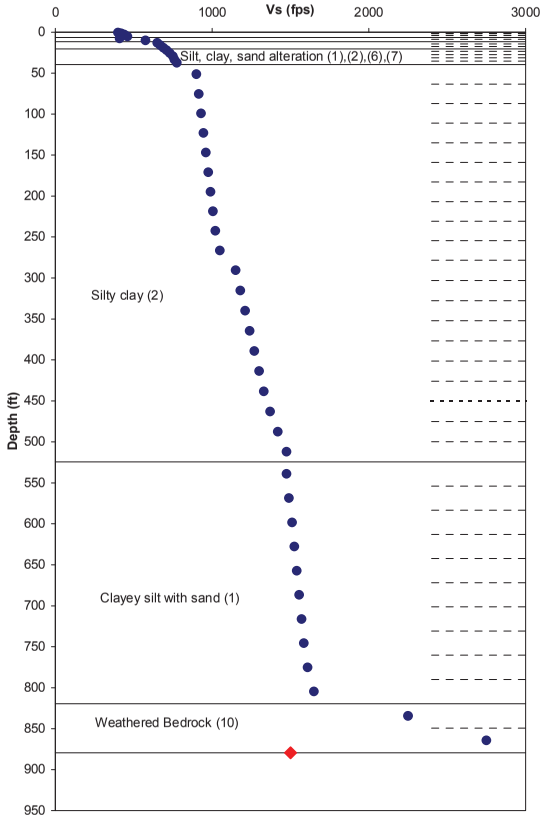


River Park A & C – Imperial Valley (1979)

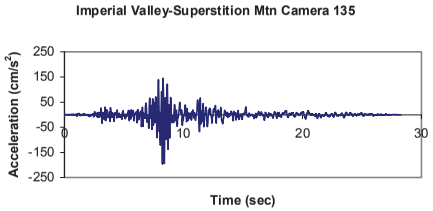


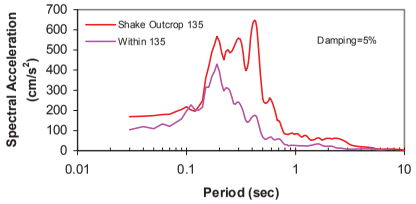


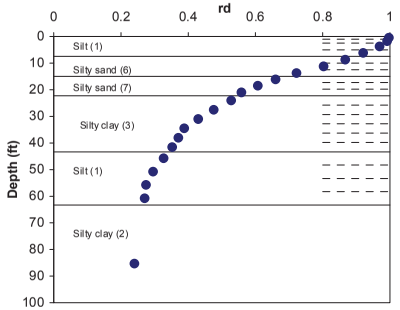


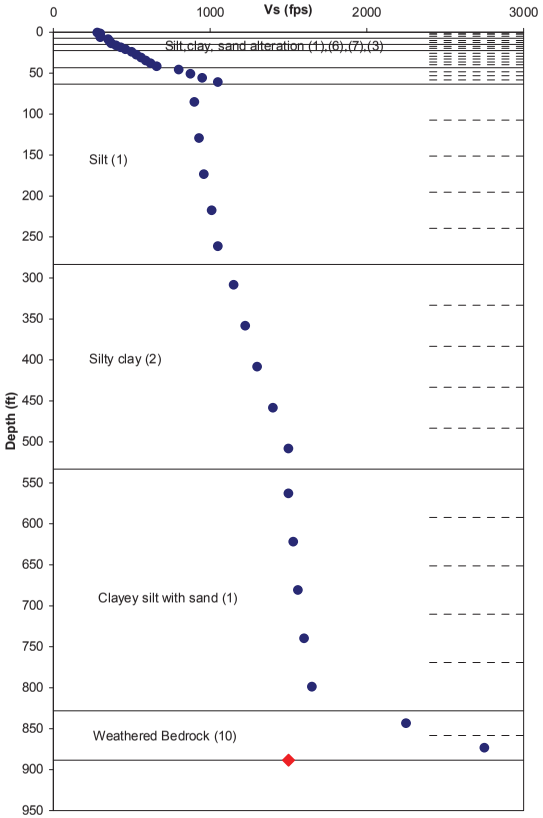


Wildlife Site – Imperial Valley (1979)

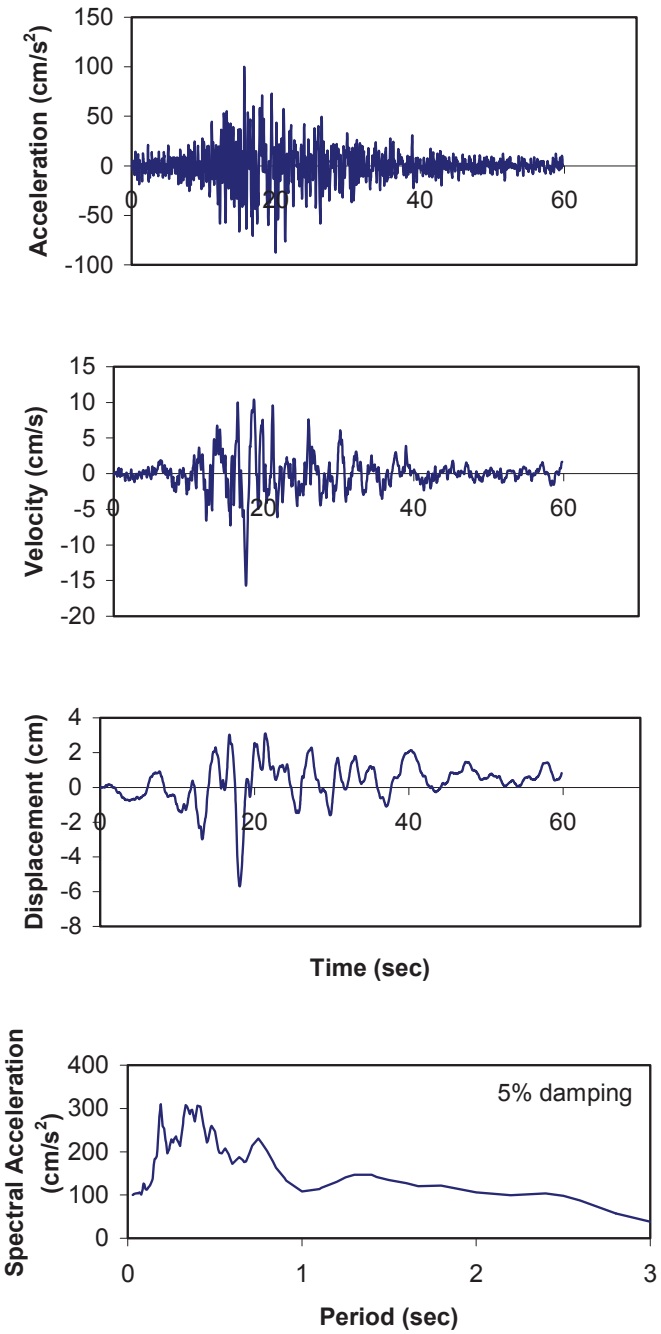




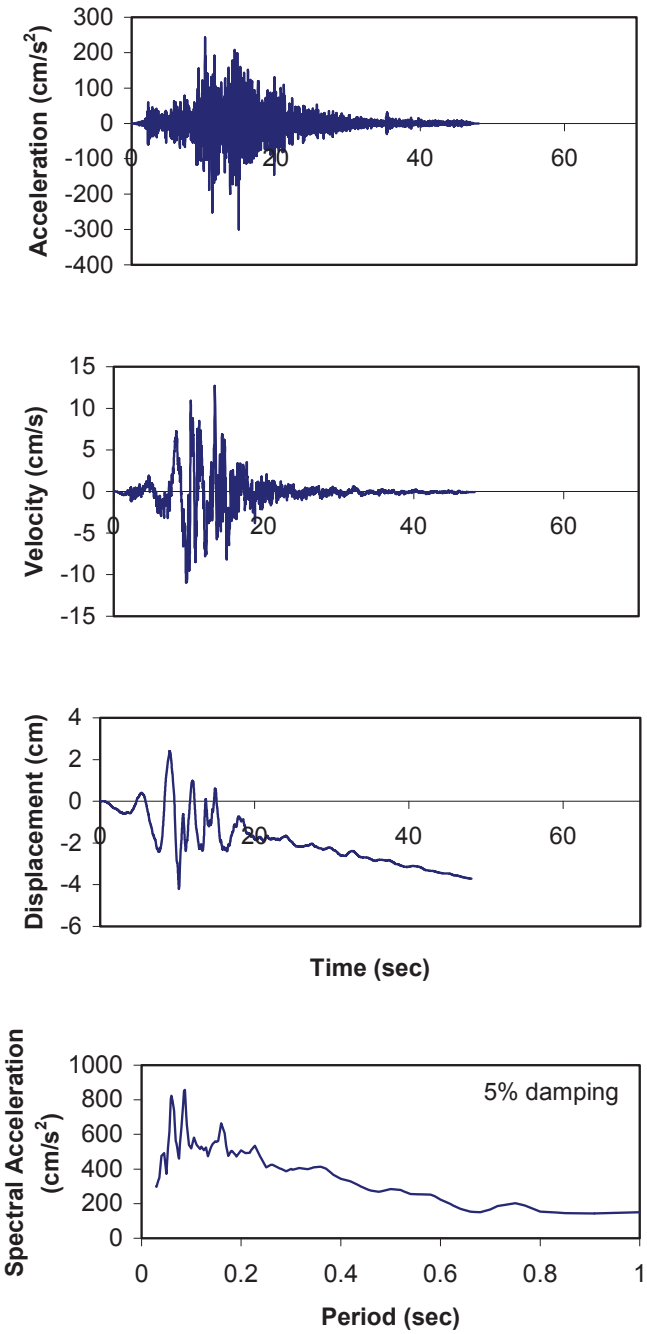




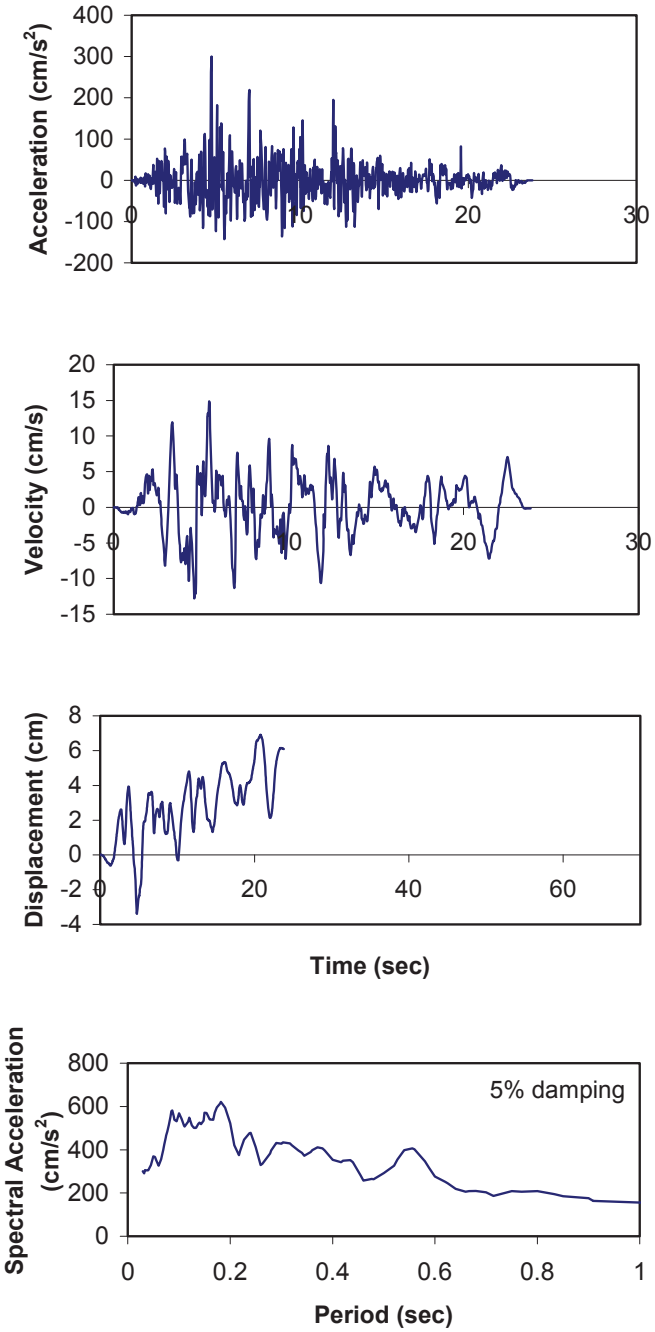
1985 Michoacan Earthquake - Ocotito Station (S00E)



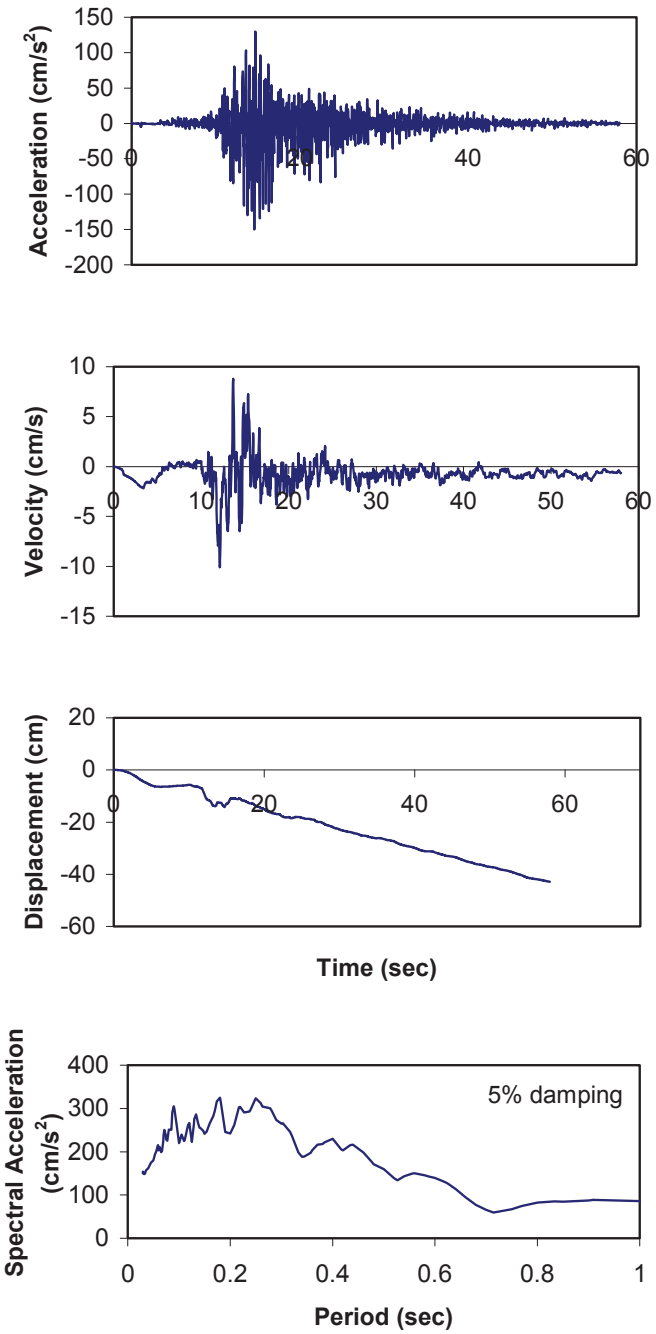
Synthetic Seismogram - Landers Earthquake (Lucerne-260)



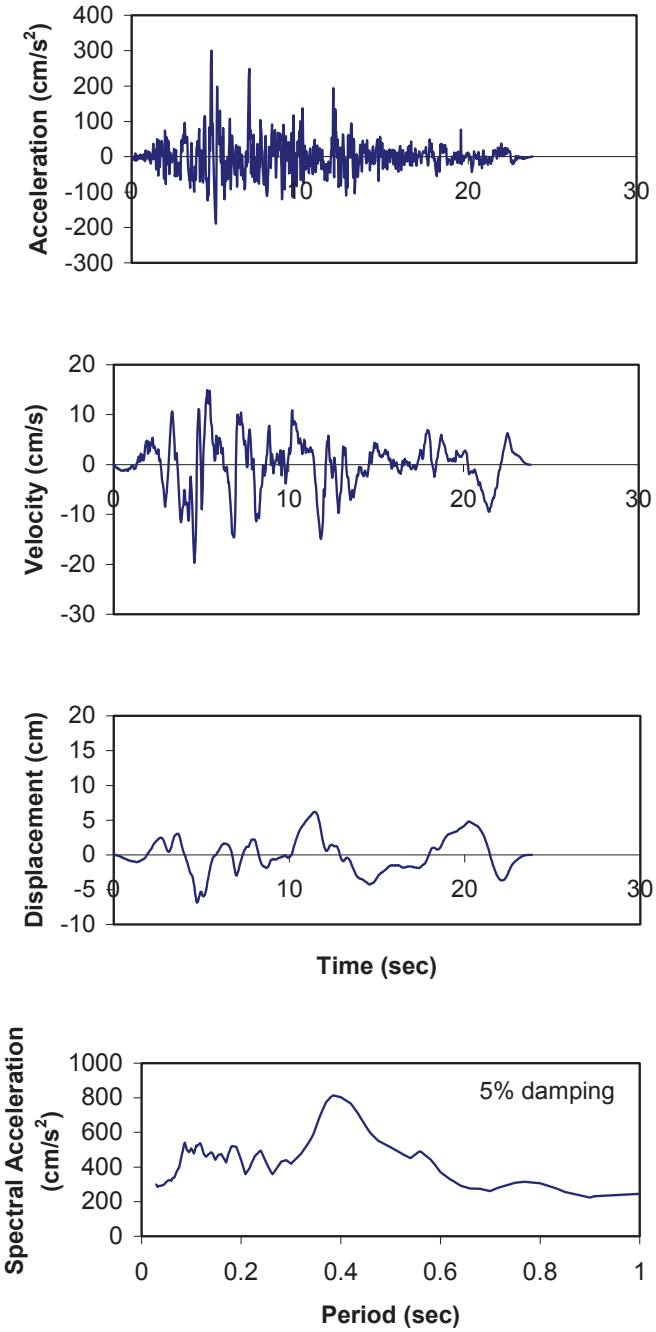
Synthetic Seismogram - Tabas Earthquake (Dayhook)



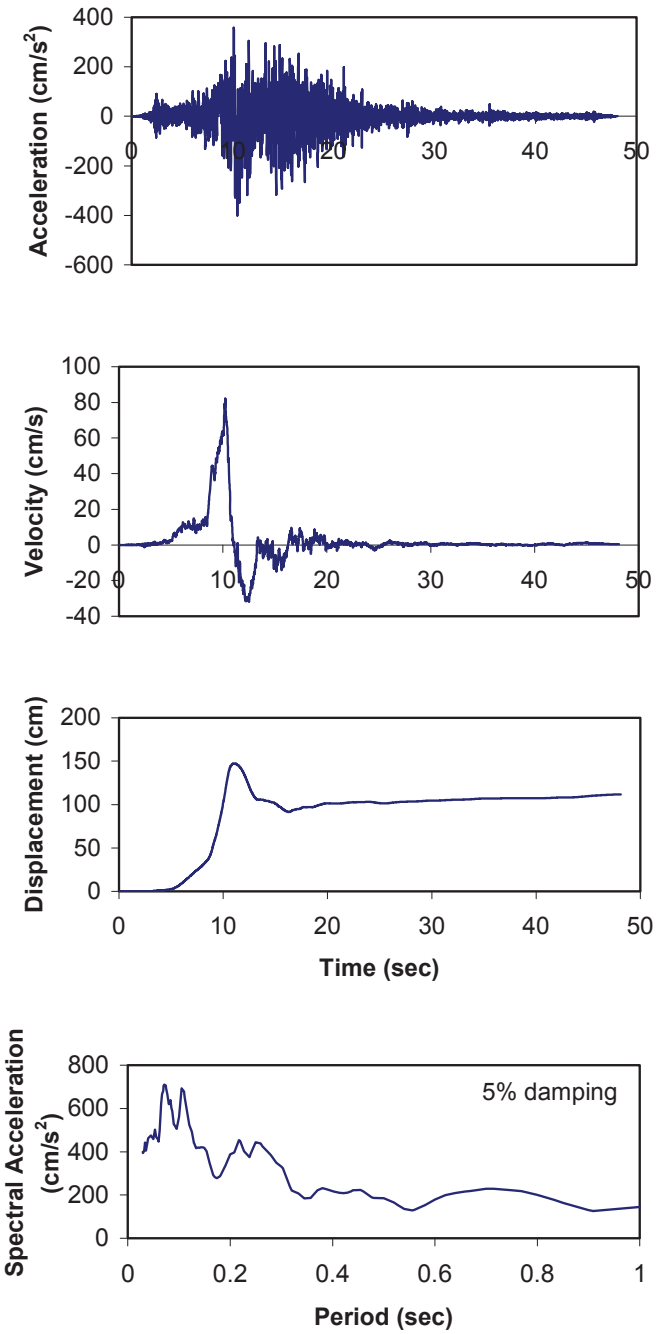
1978 Miyagioki Earthquake - Ofunato Bochi Station



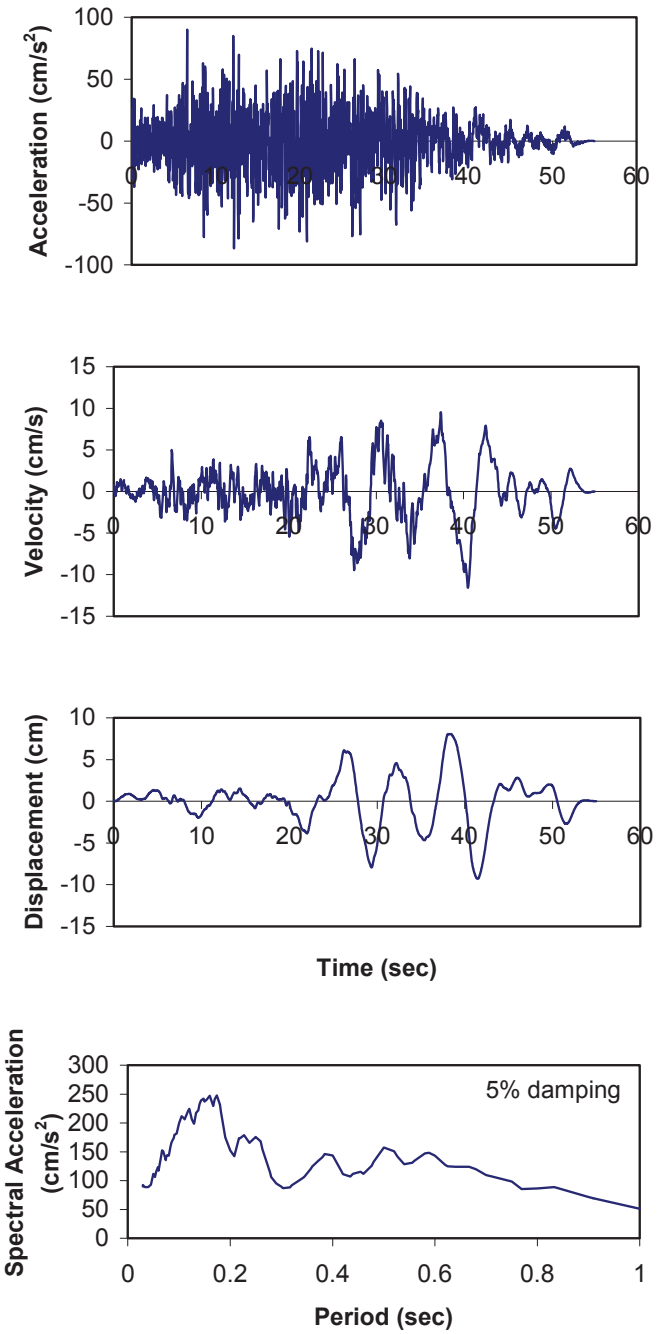
1978 Tabas Earthquake – Dayhook Station



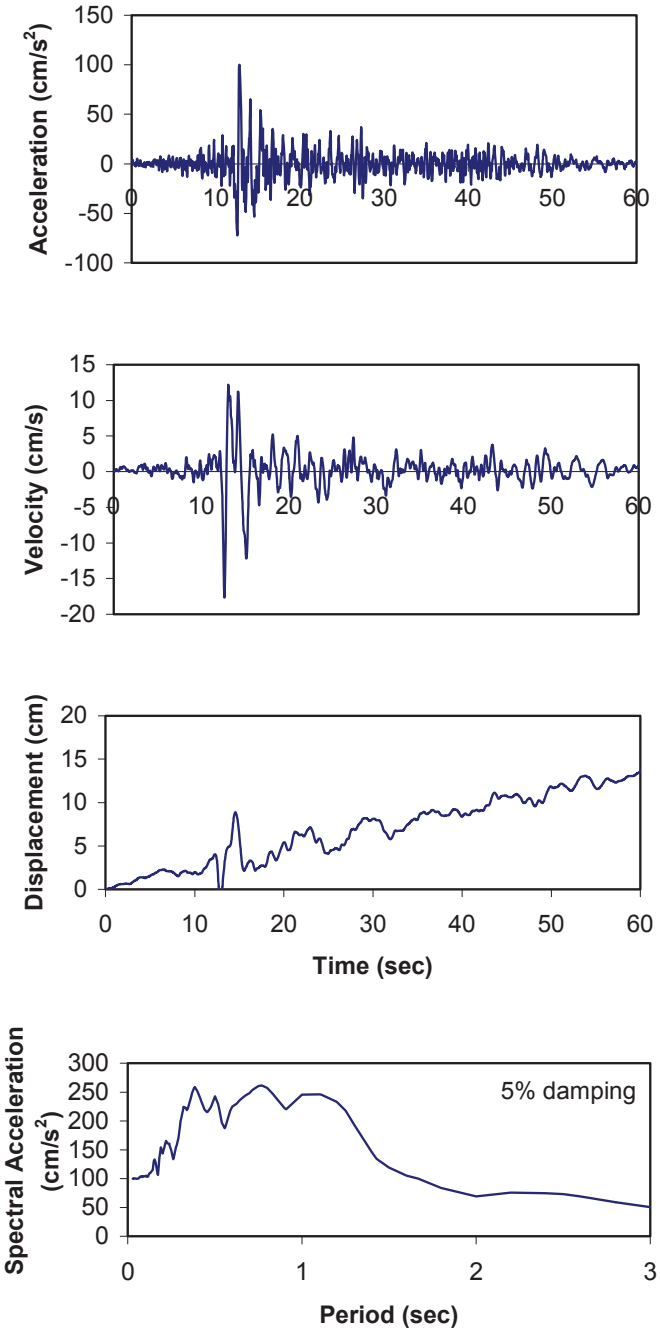
1992 Landers Earthquake – Lucerne Station (090)



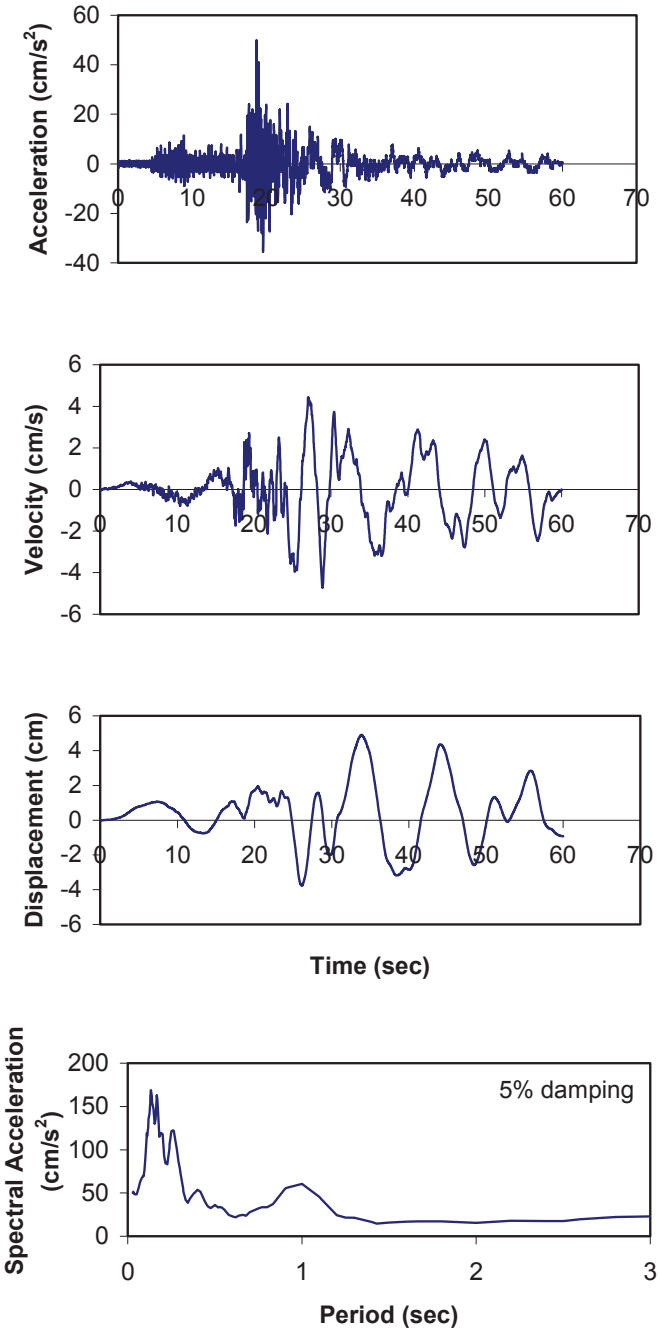
1992 Landers Earthquake – Silent Valley Station (090)



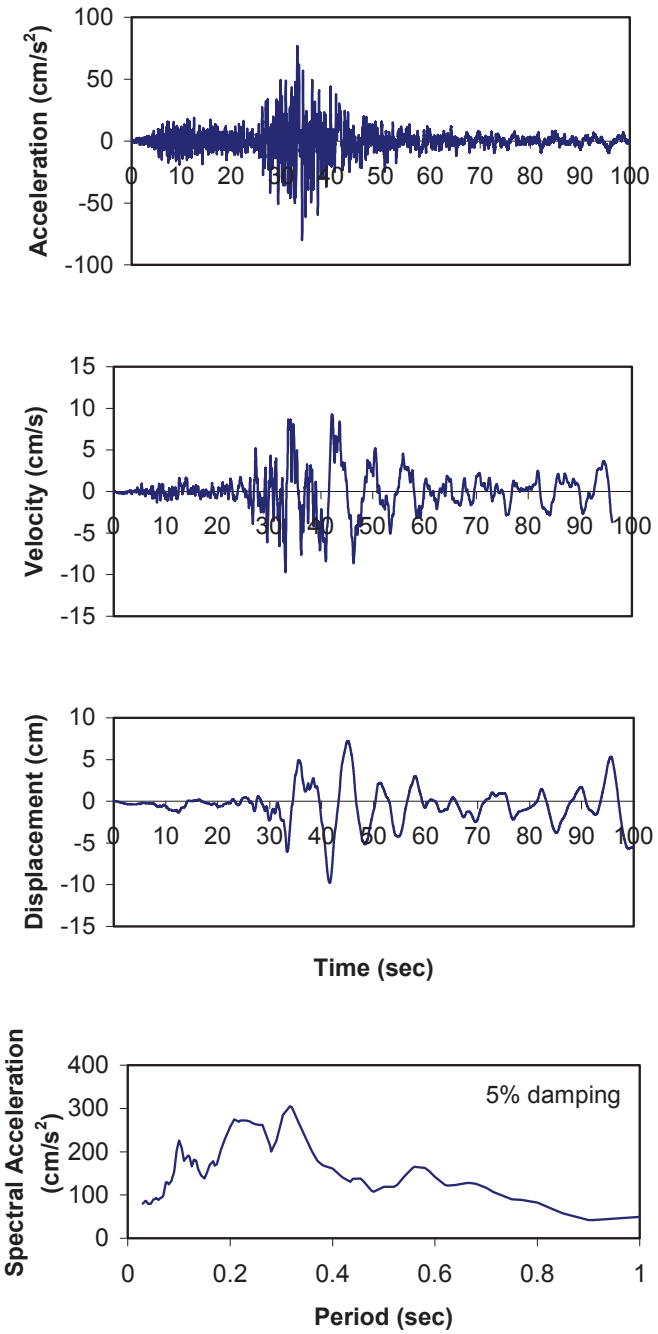
1979 Alaska Earthquake – Munday Creek Station



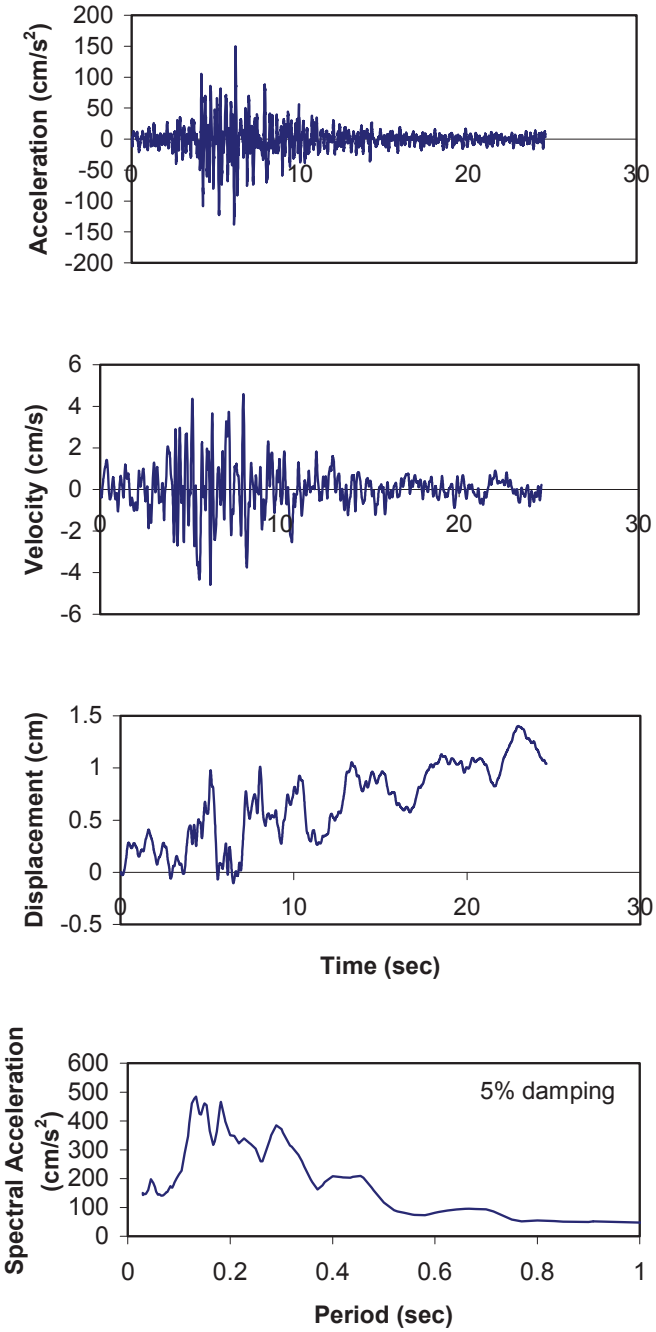
1994 Euroka Earthquake – Cape Mendocino Station



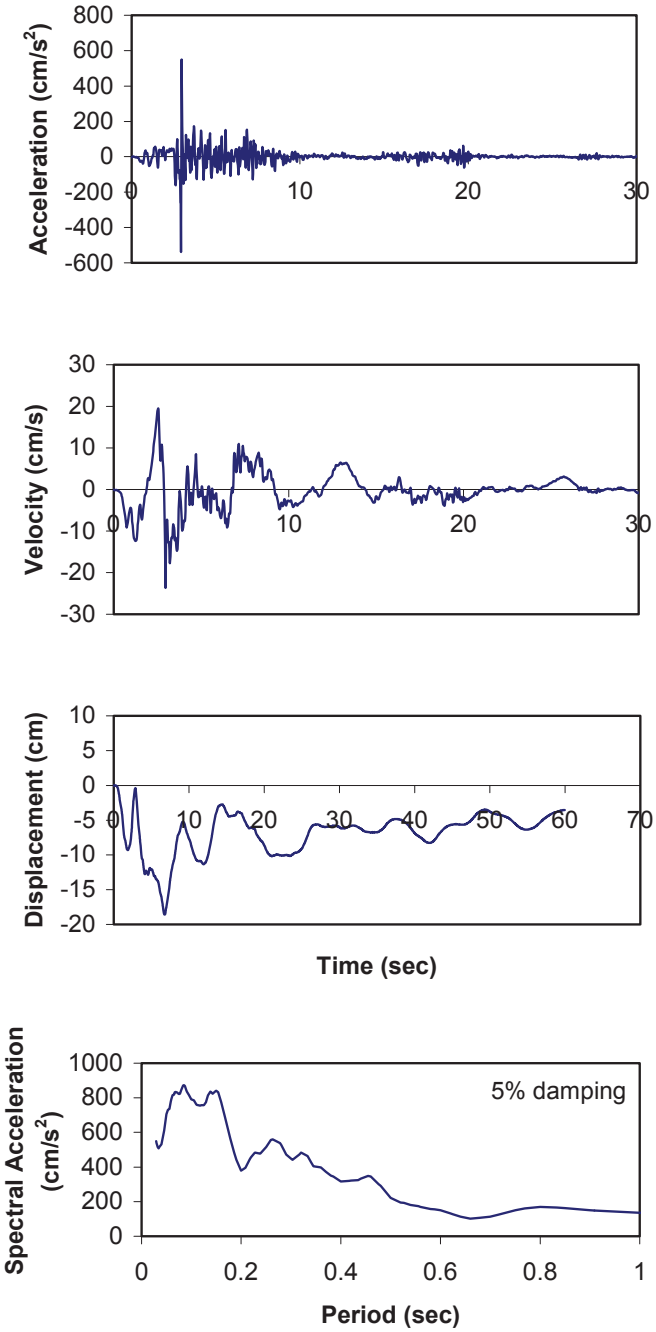
1999 Hector Mines Earthquake – LA City Terrace Station



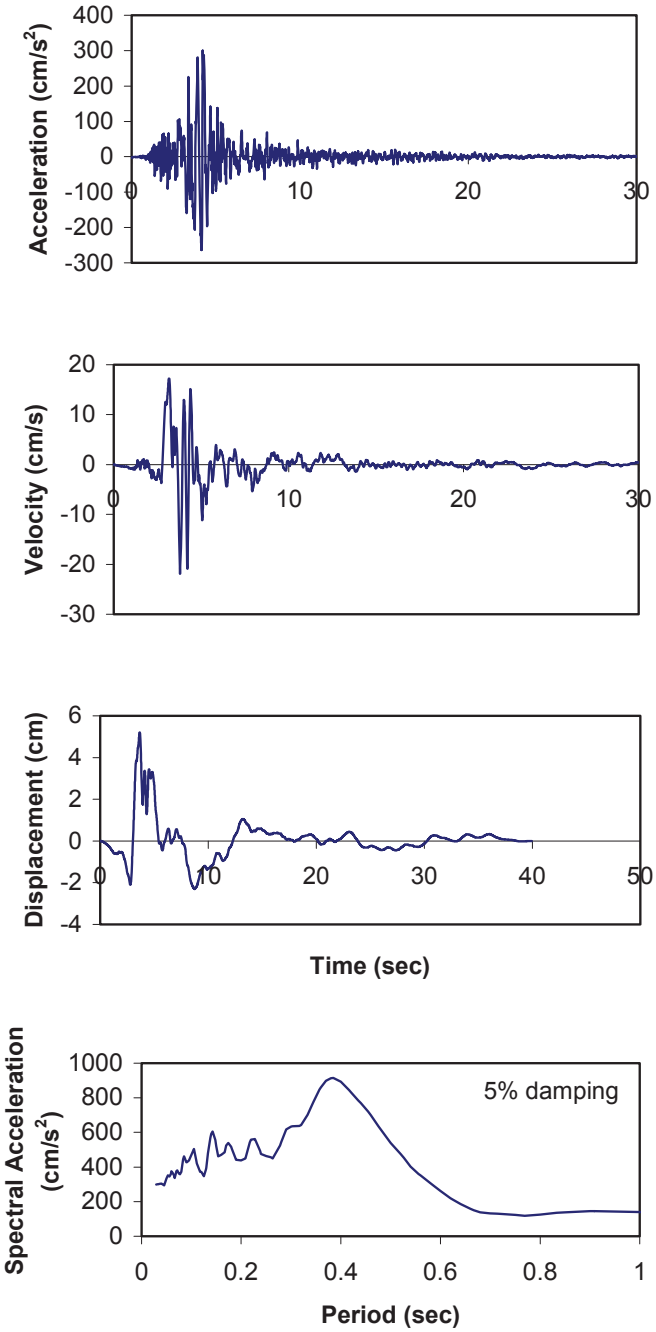
1971 Adak Alaska Earthquake – Naval Base Station



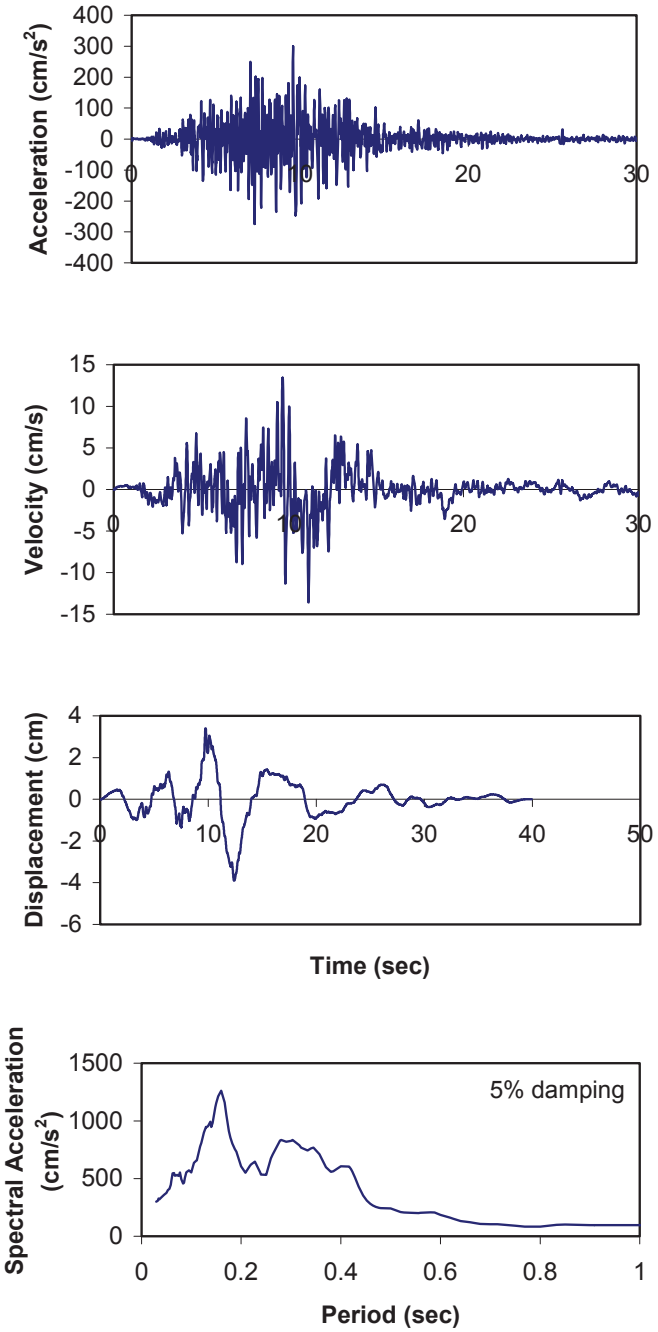
1992 Cape Mendocino Earthquake – Cape Mendocino Station



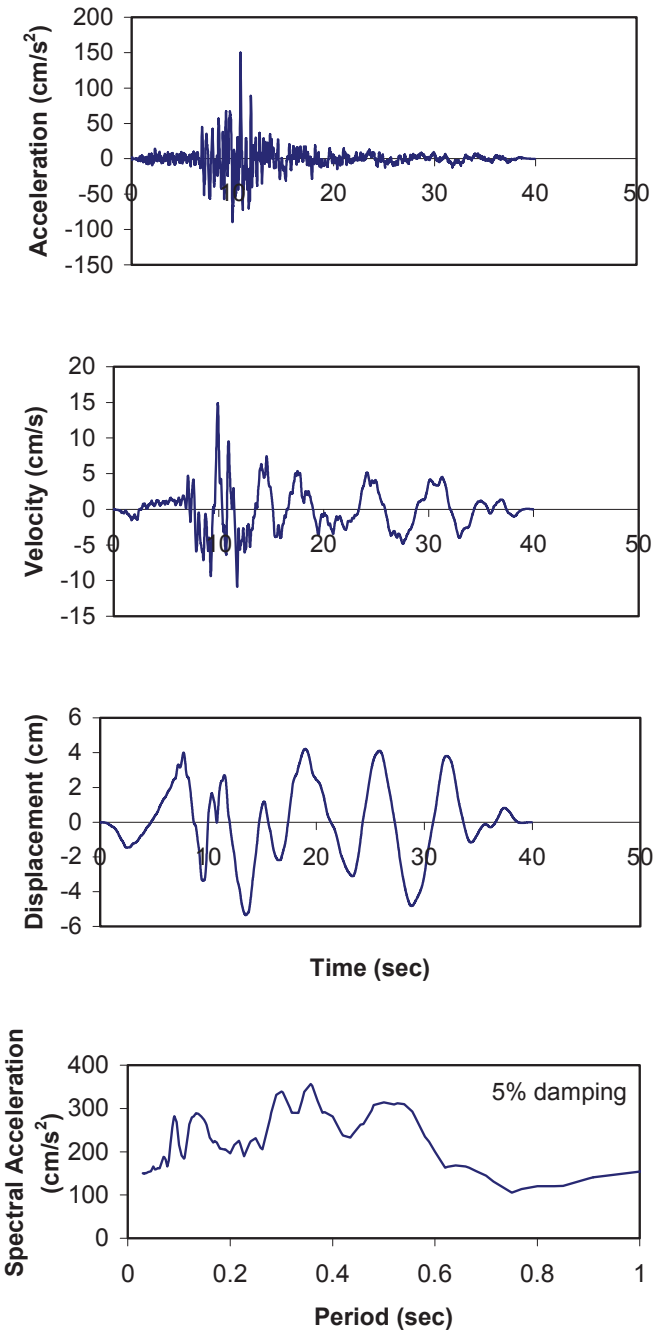
1989 Loma Prieta Earthquake – Gilroy # 1 Station (090)



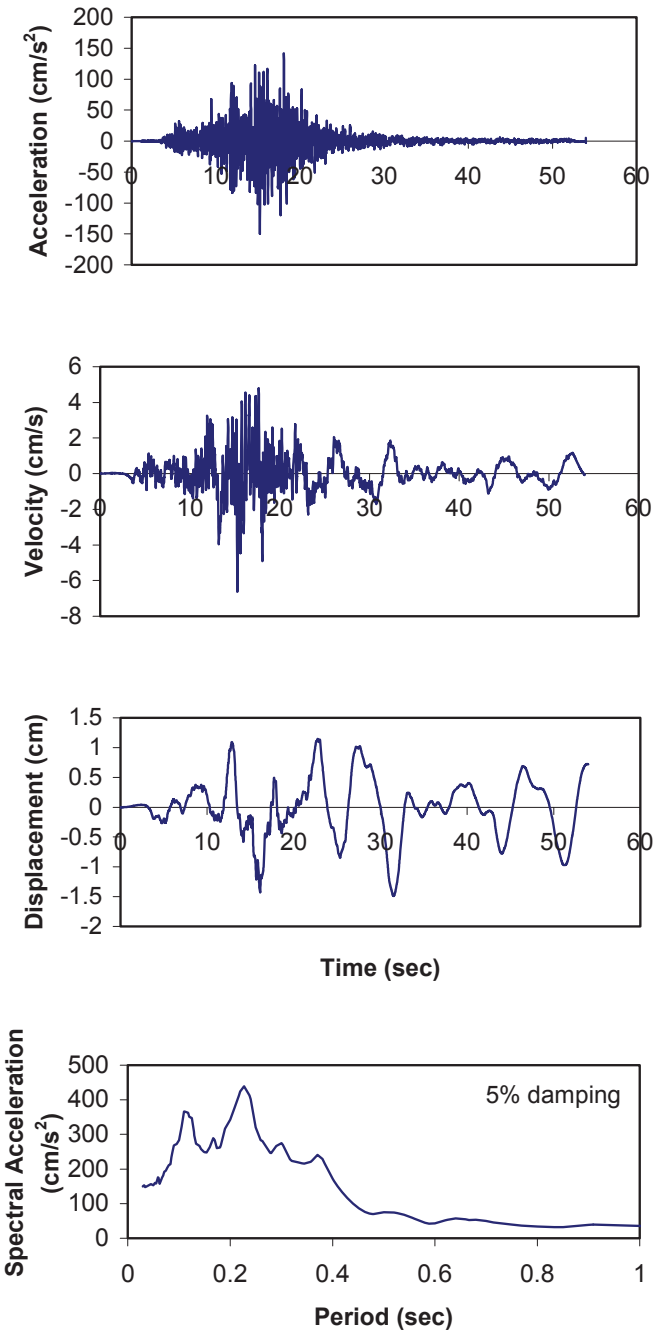
1989 Loma Prieta Earthquake – Lick Observatory Station (090)



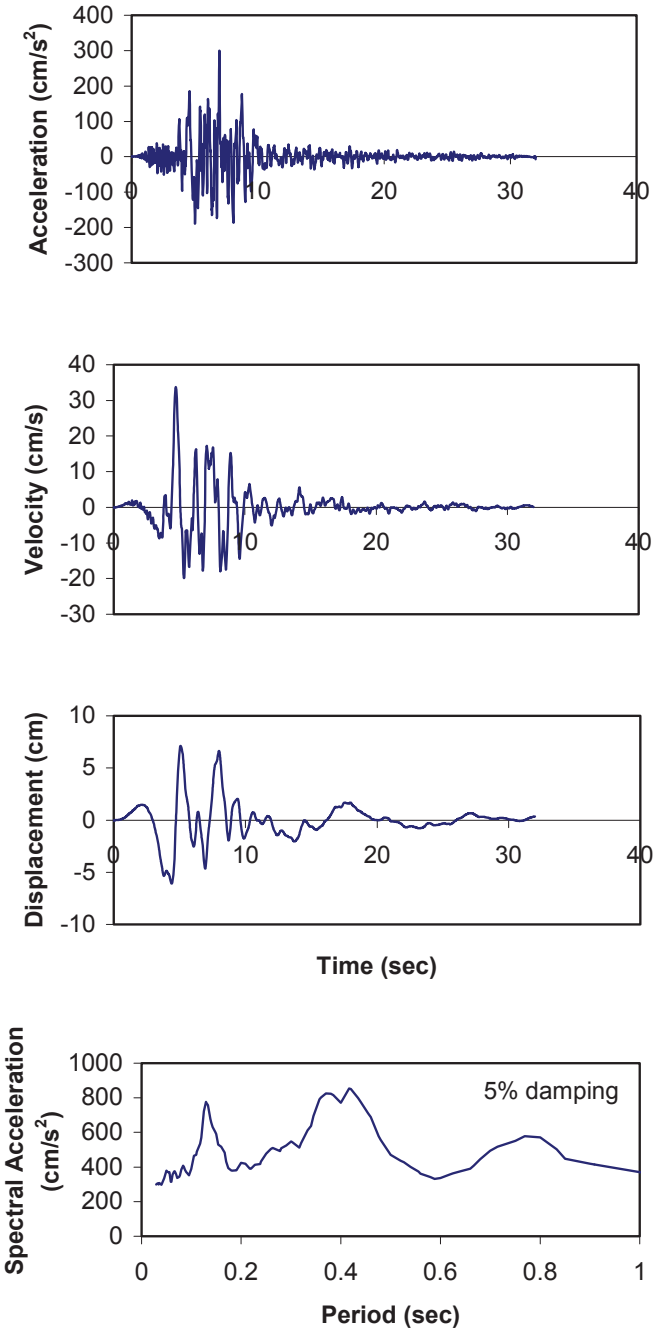
1989 Loma Prieta Earthquake – Piedmont Junior High School (045)



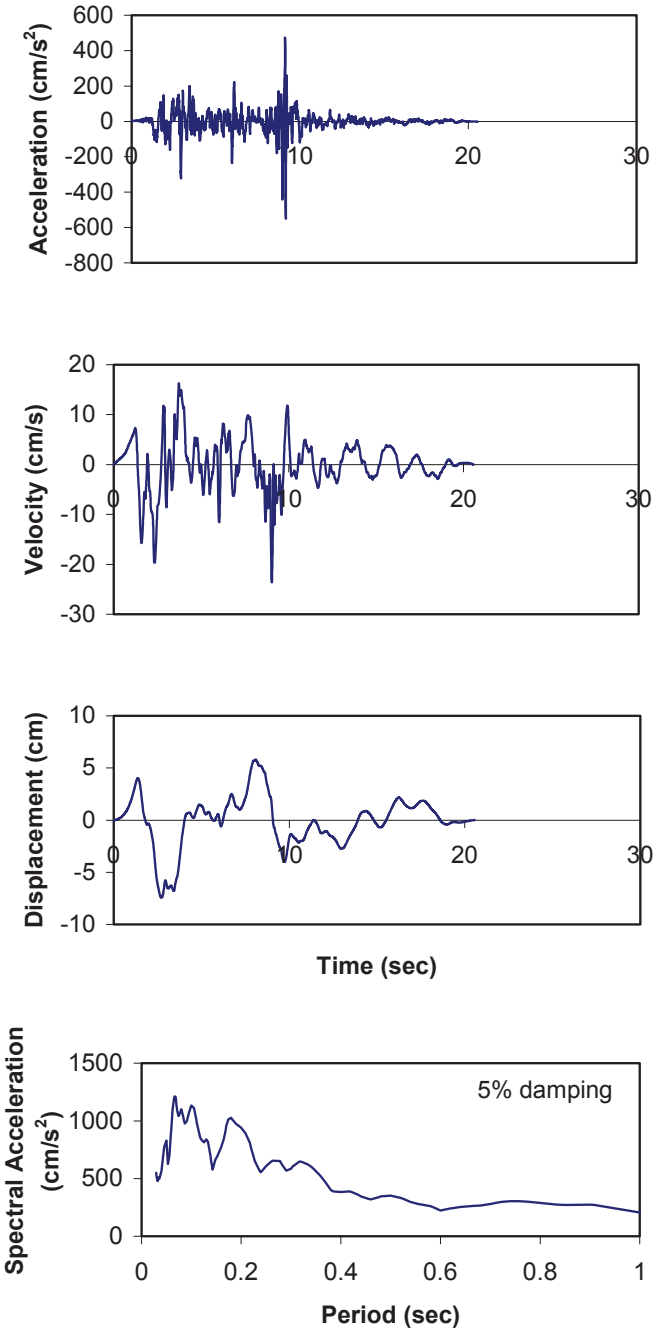
1995 Hyogoken-Nambu Earthquake – Chihaya Station (090)



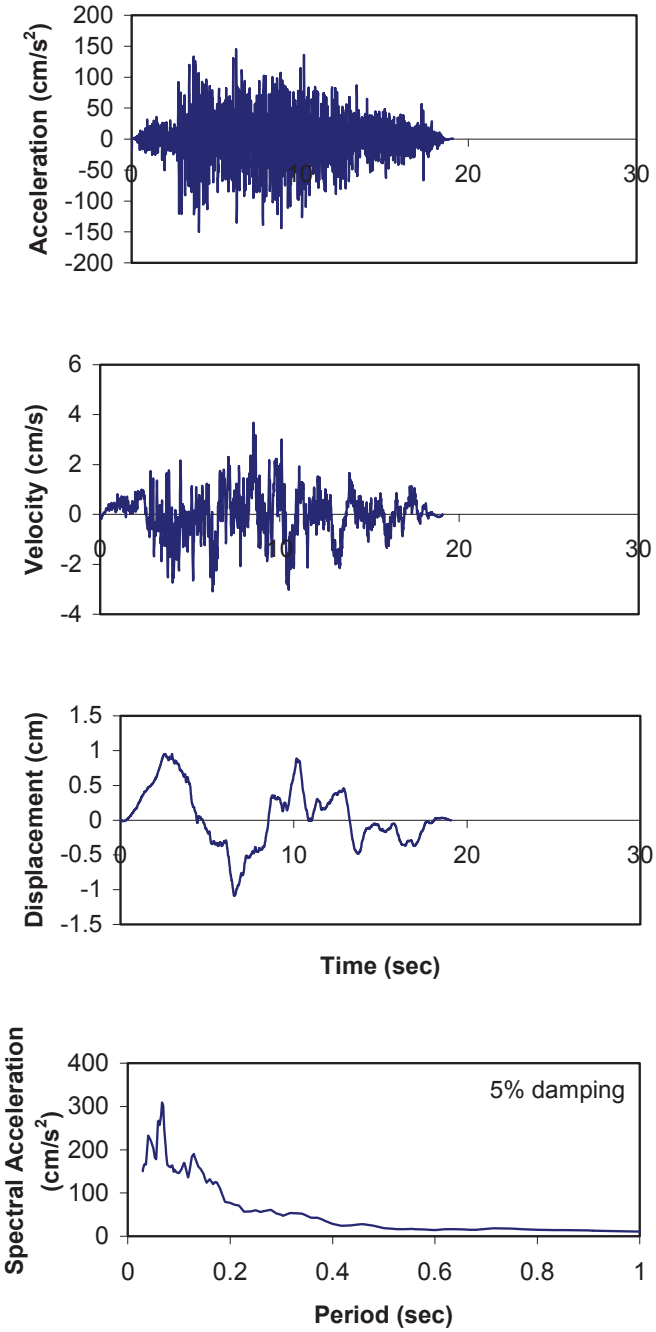
1995 Hyogoken-Nambu Earthquake – Kobe University Station (090)



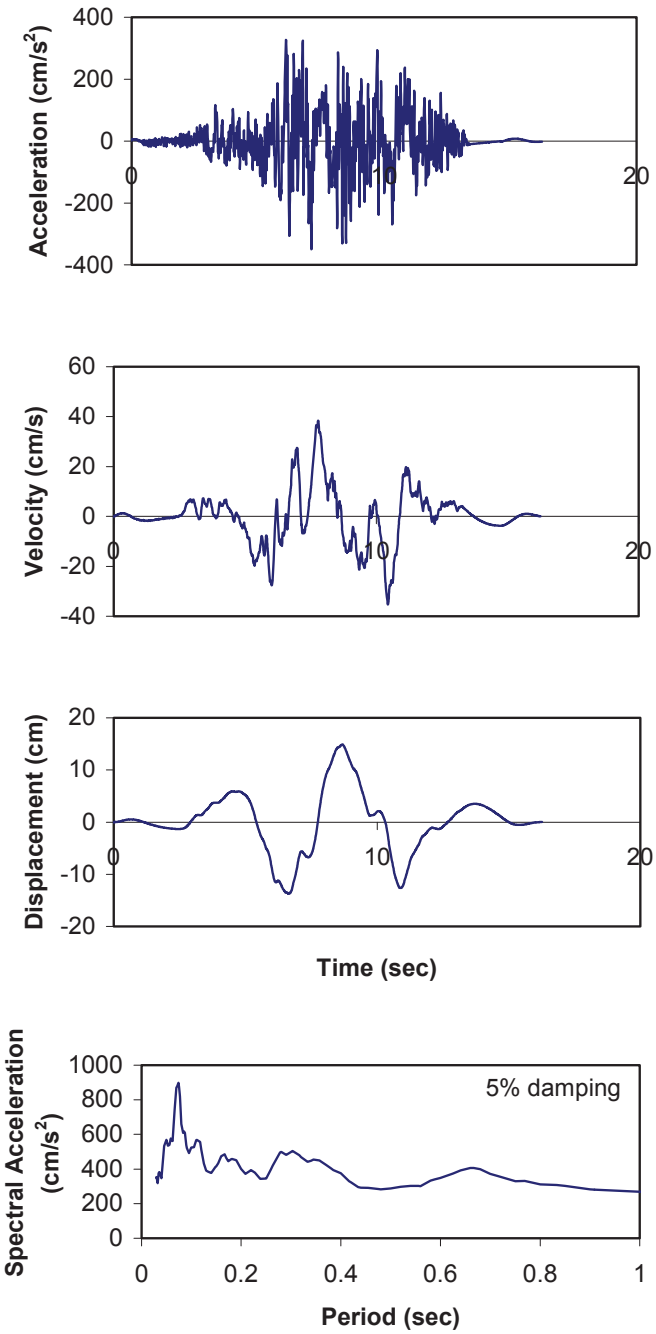
1985 Nahanni Earthquake – Site 1 Station (280)



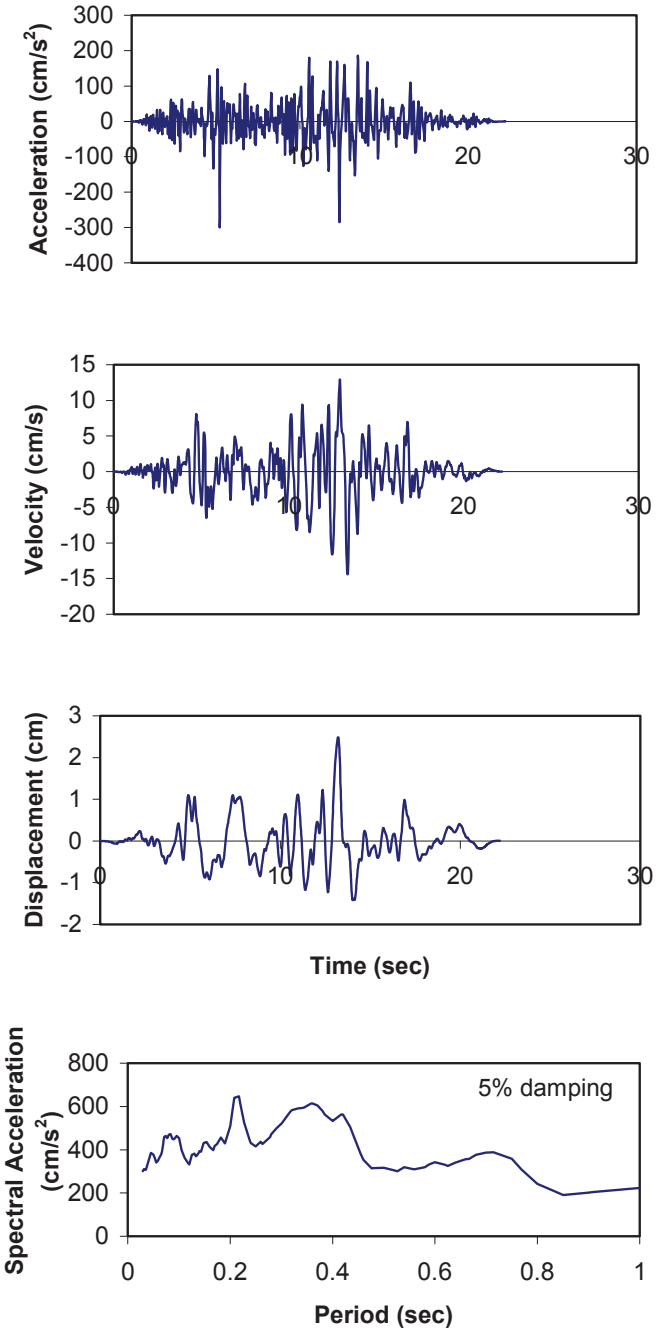
1985 Nahanni Earthquake – Site 3 Station (360)



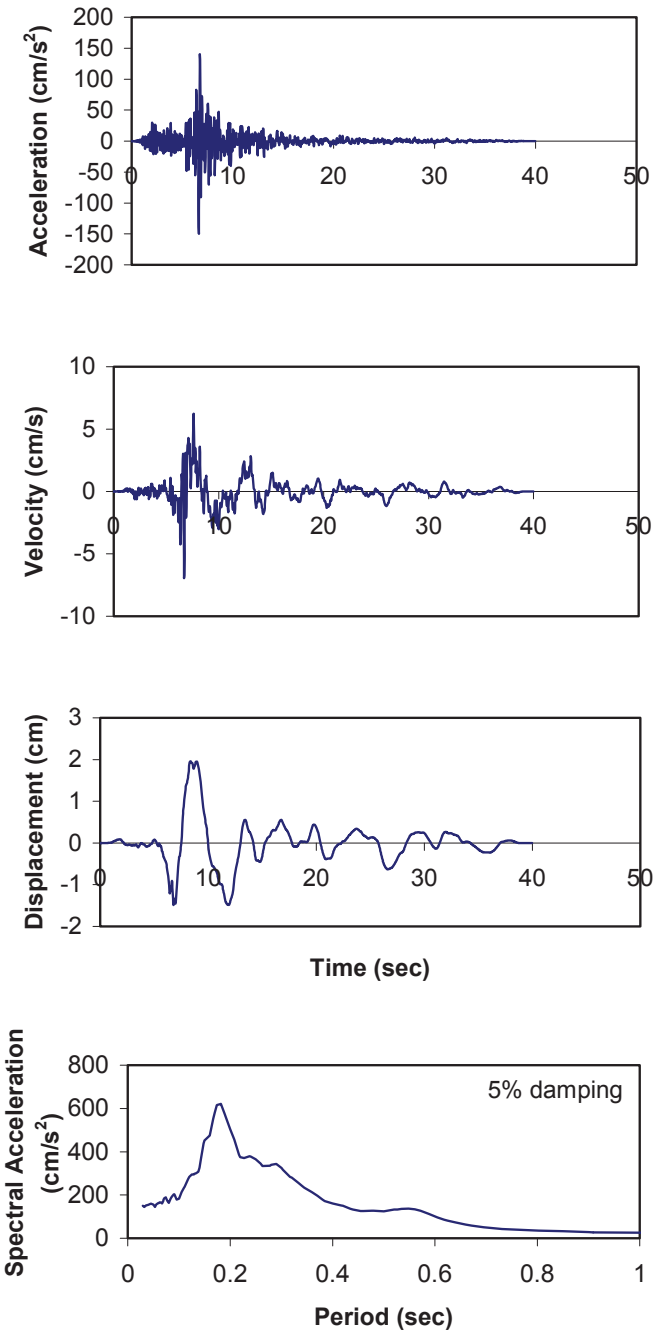
1976 Gazli Earthquake – Karakyr Station (000)



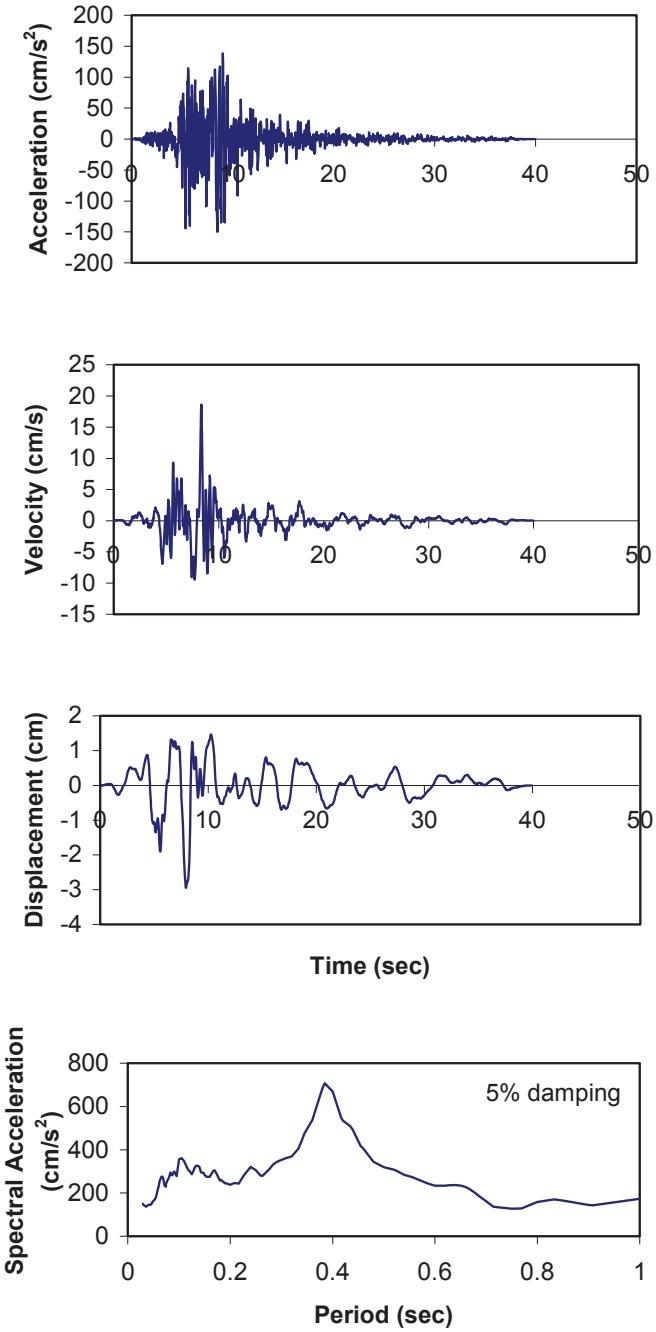
1987 Superstition Hills Earthquake – Superstition Mountain Station (135)



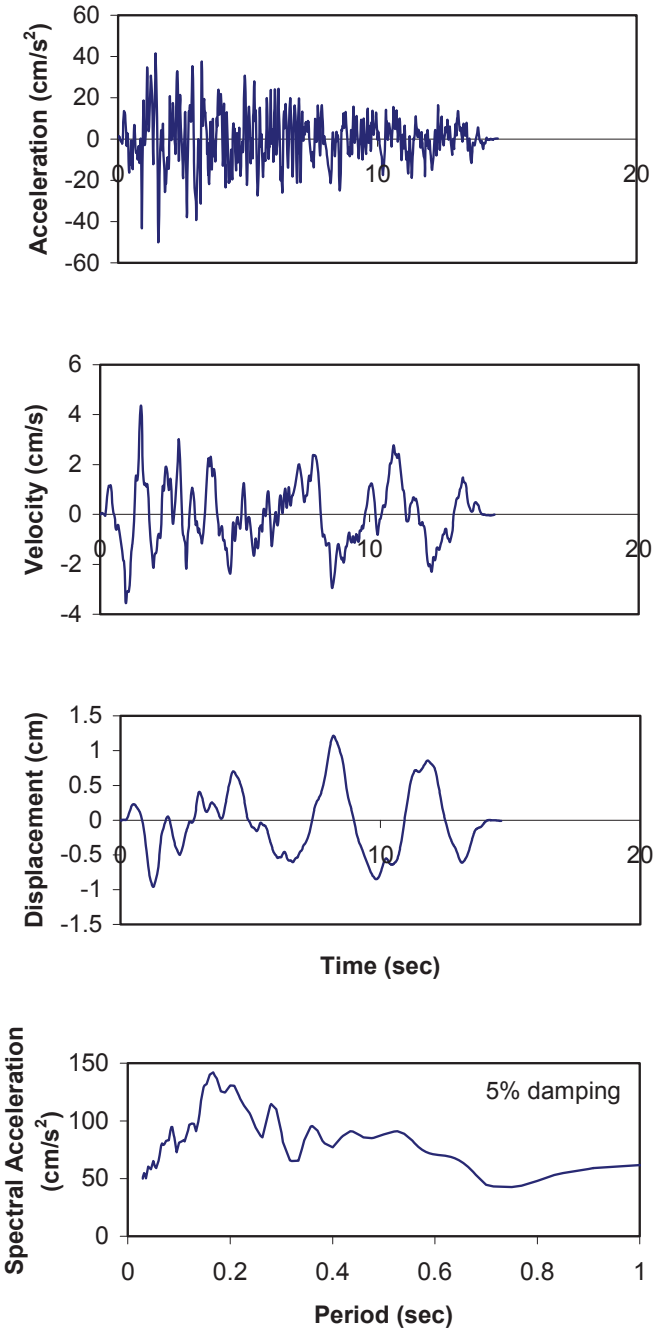
1994 Northridge Earthquake – Lake Hughes # 9 Station (090)



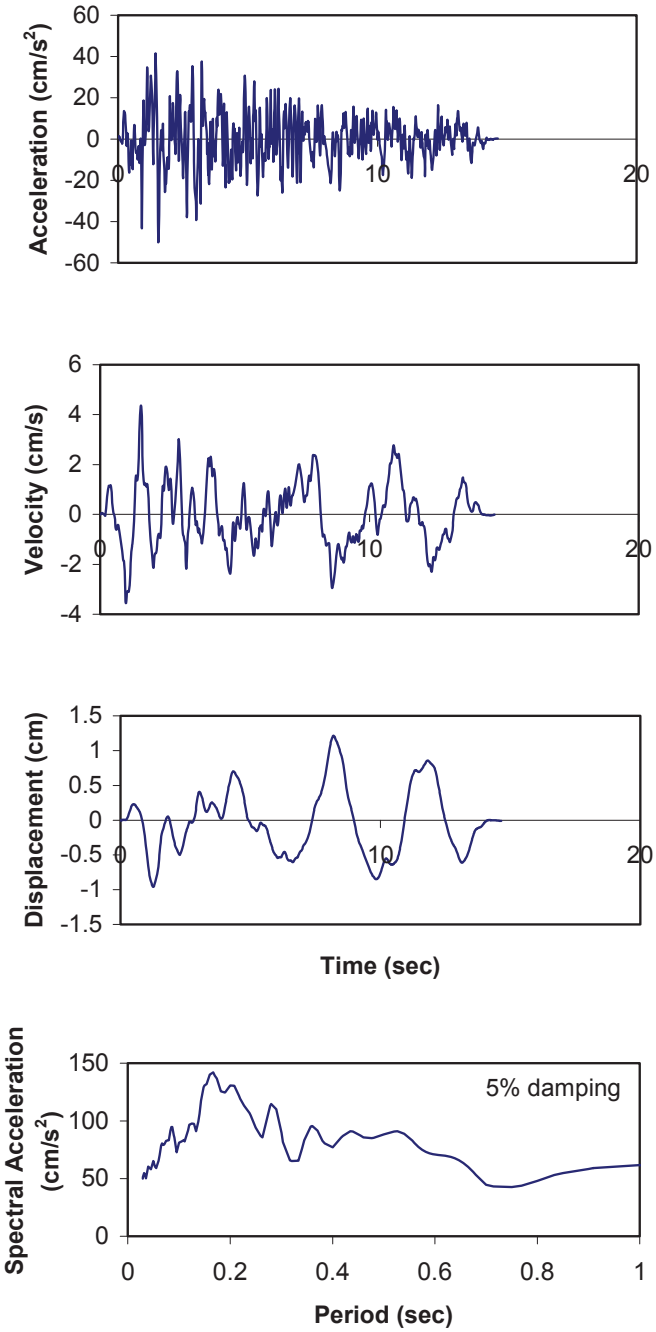
1994 Northridge Earthquake – Vasquez Rocks Station (000)



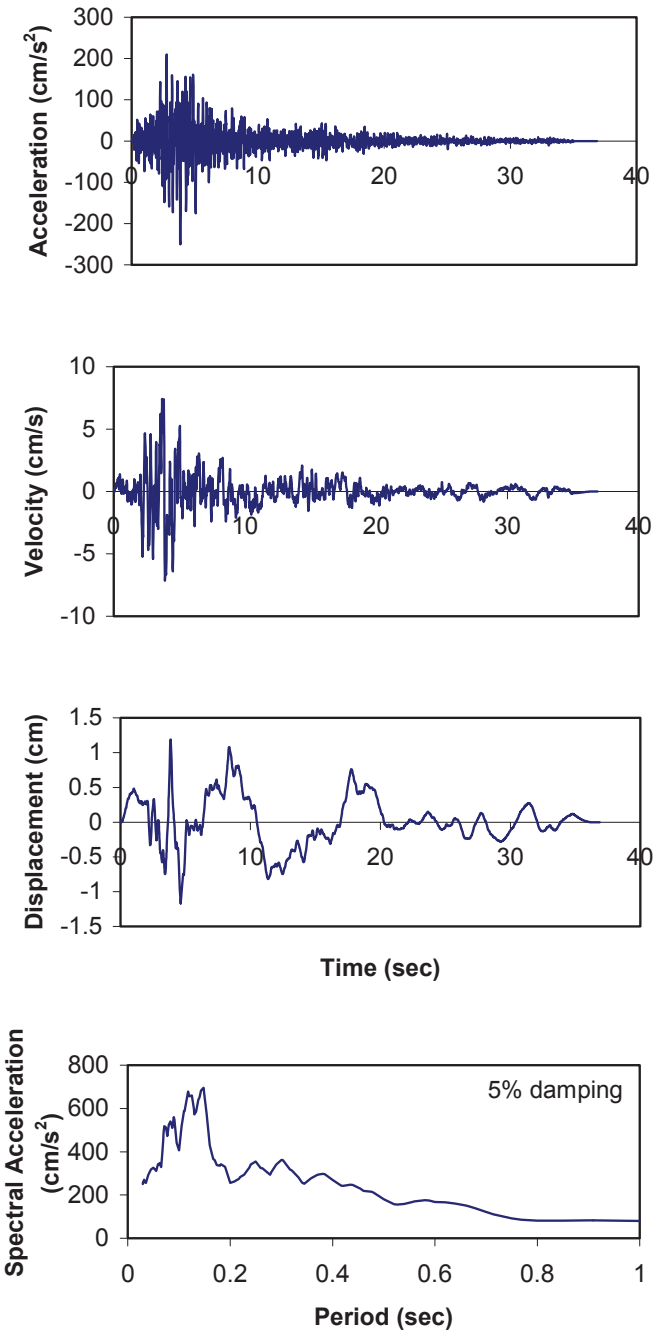
1971 San Fernando Earthquake – Cedar Springs Station (095)



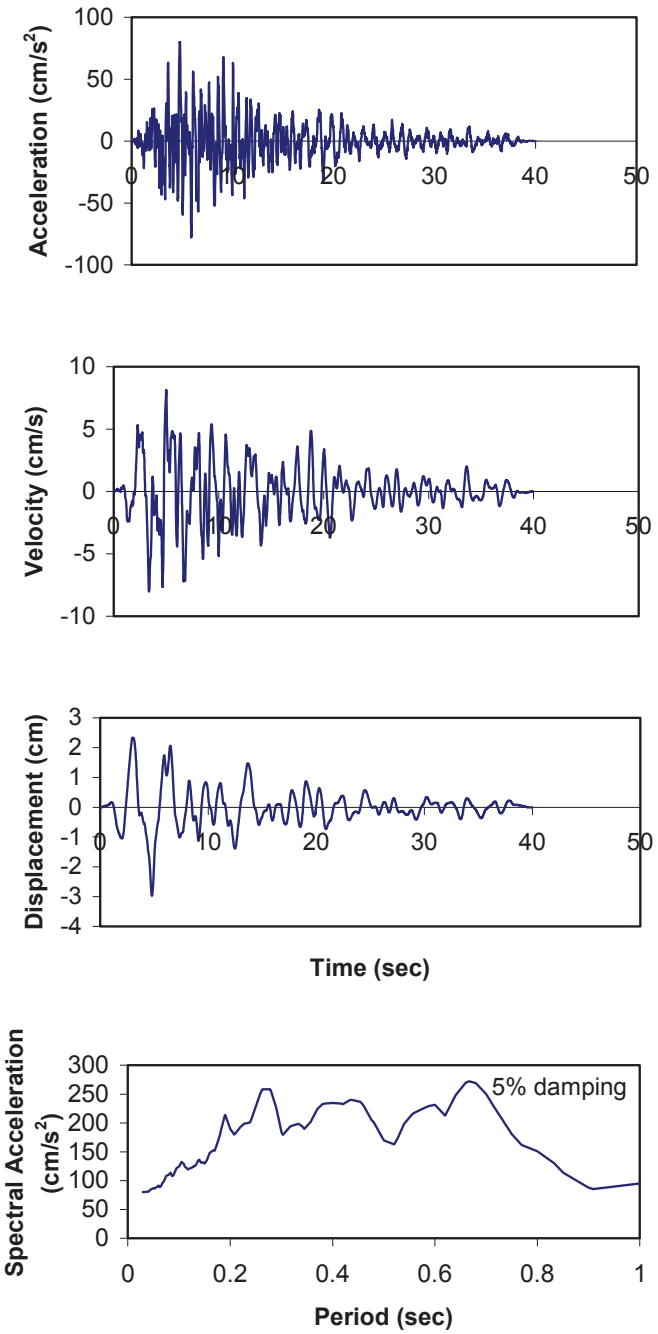
1971 San Fernando Earthquake – Carbon Canyon Station (220)



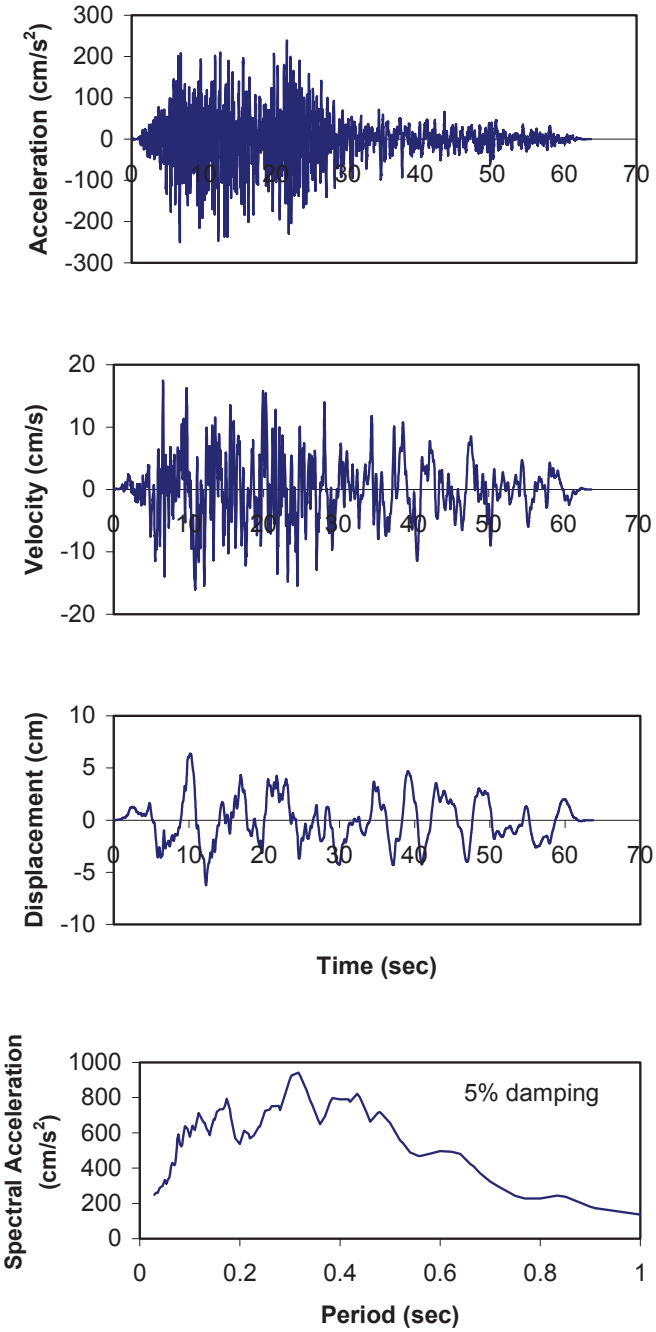
1971 San Fernando Earthquake – Lake Hughes # 4 Station (111)



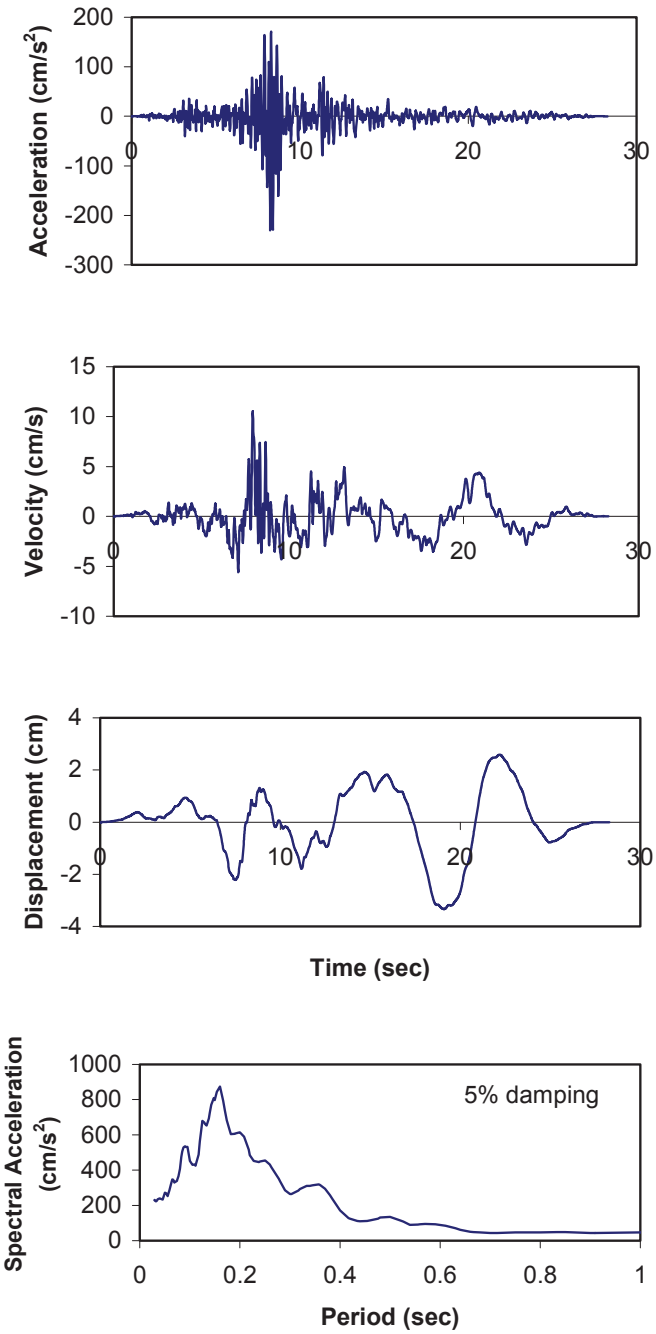
1983 Coalinga Earthquake – Parkfield Cholame 3E Station (000)



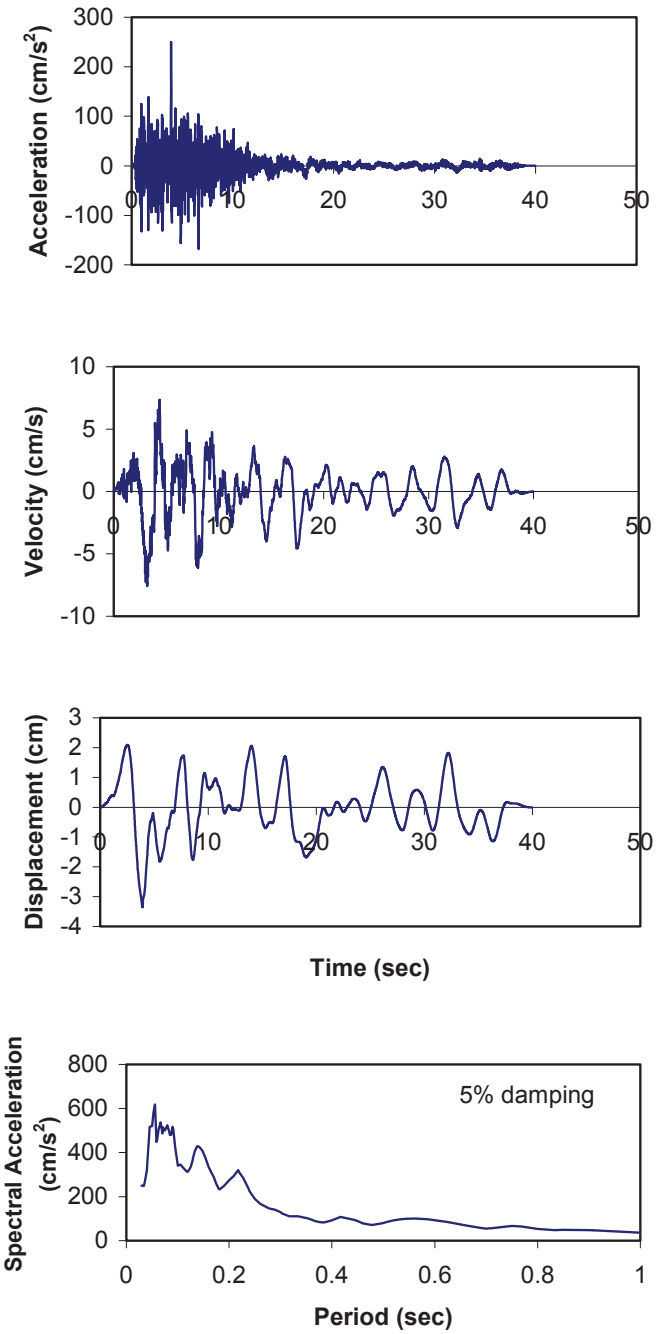
1979 Imperial Valley Earthquake – Cerro Prieto Station (147)



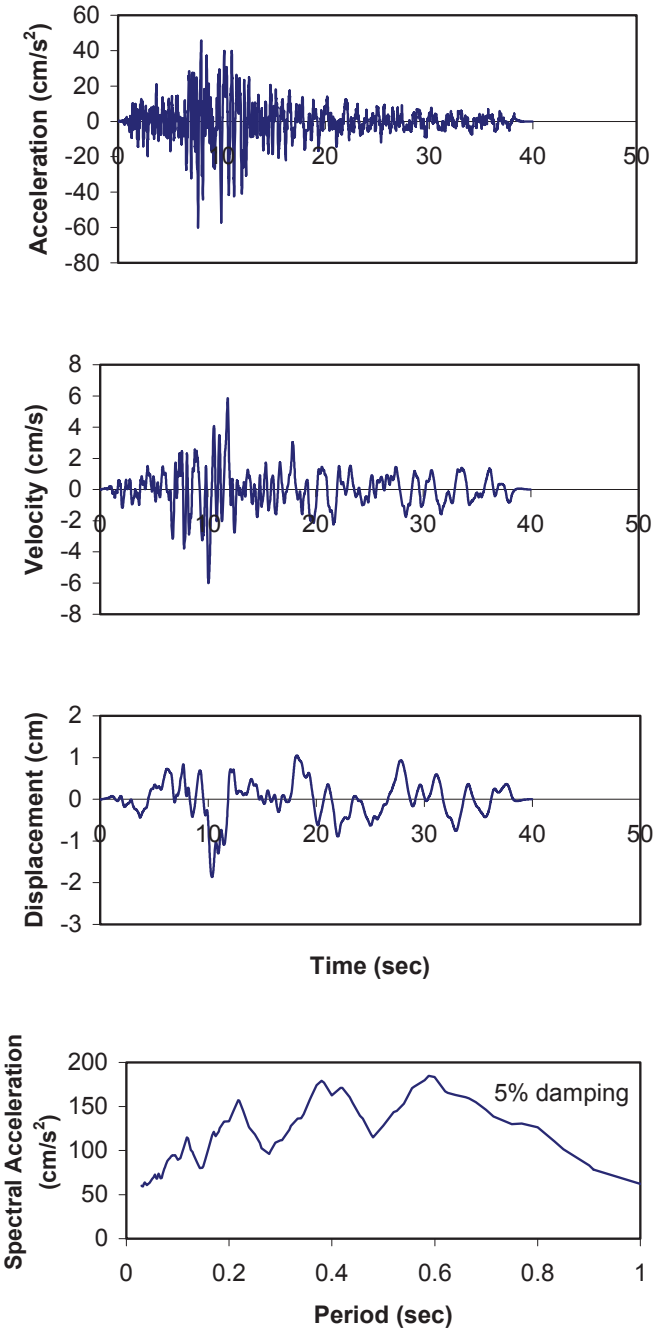
1979 Imperial Valley Earthquake – Superstition Mt. Camera Station (135)



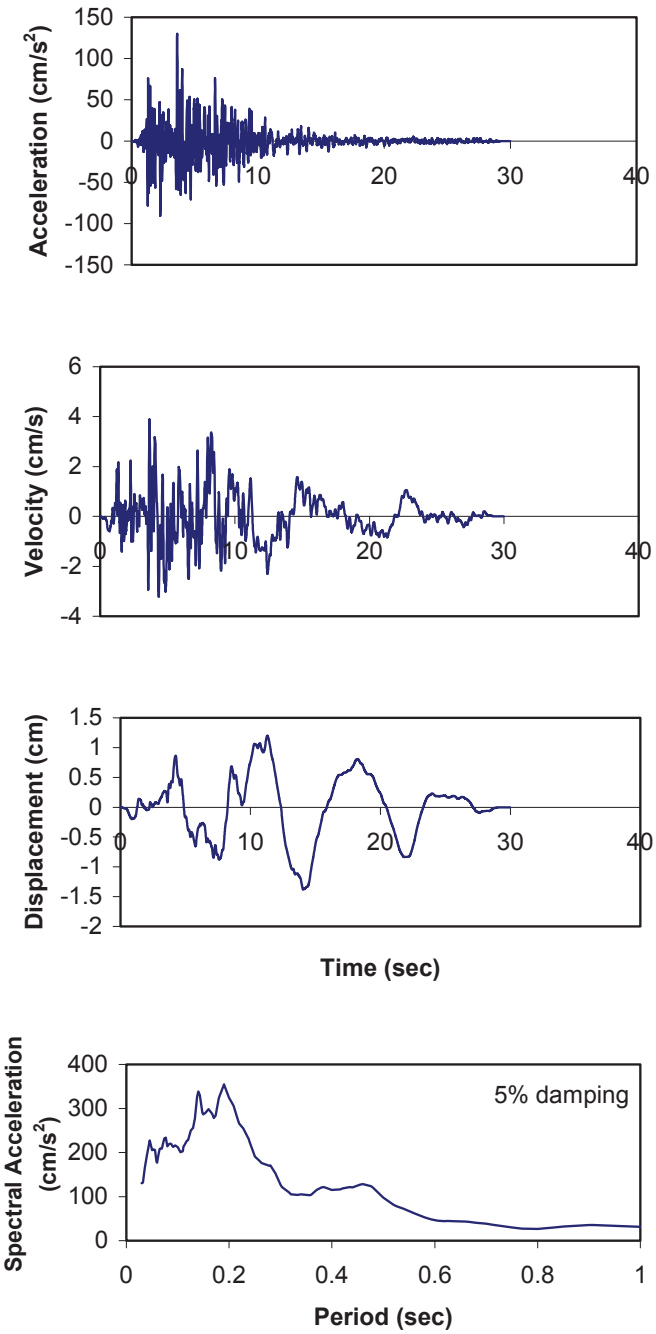
1986 Chalfant Valley Earthquake – Paradise Lodge Station (070)



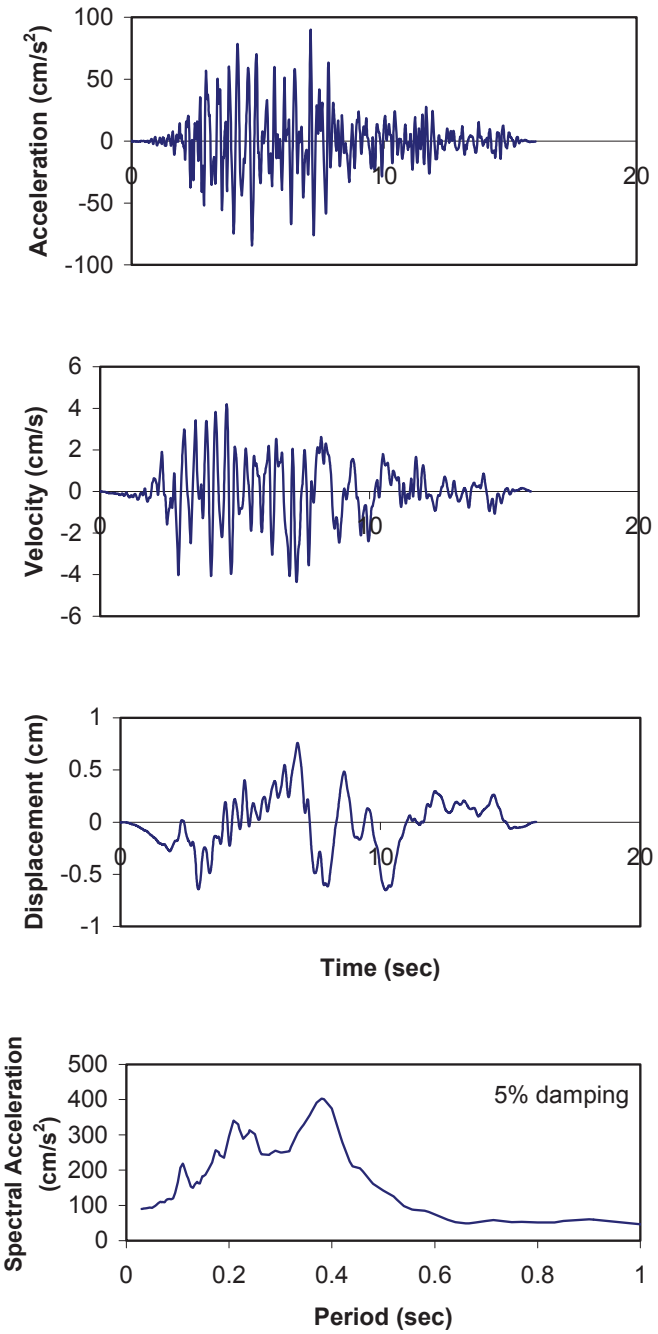
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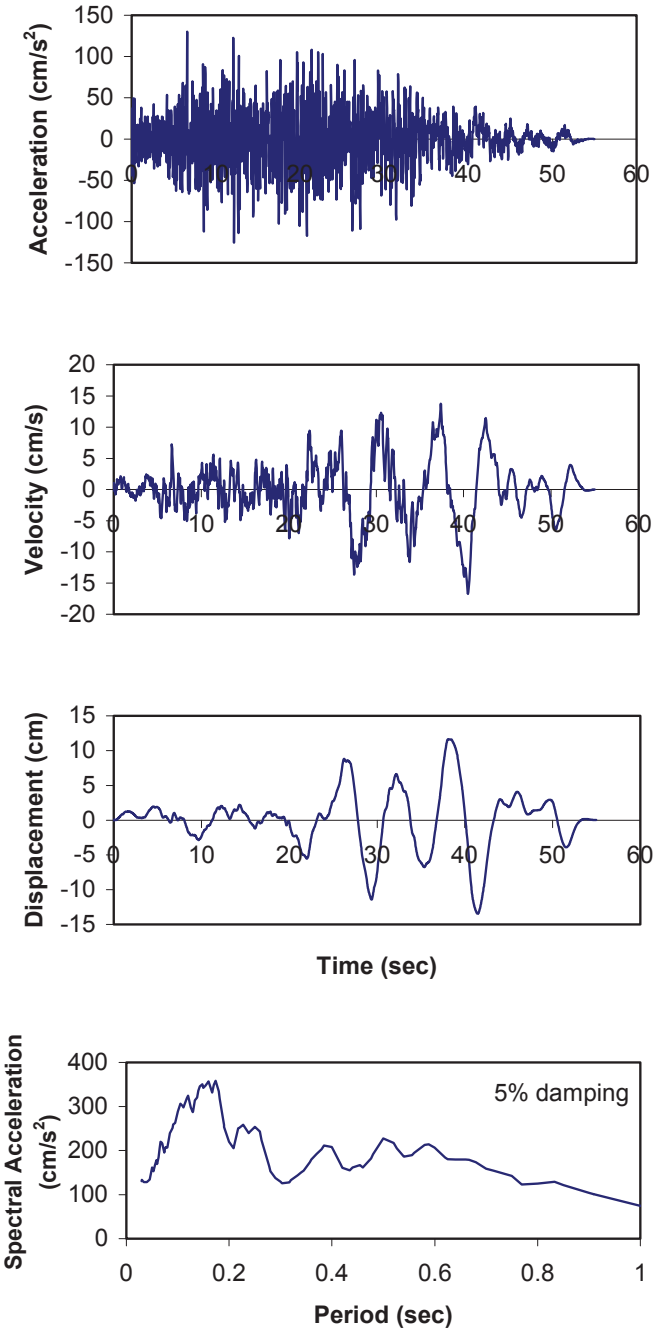
1984 Morgan Hill Earthquake – Gilroy #1 Station (320)



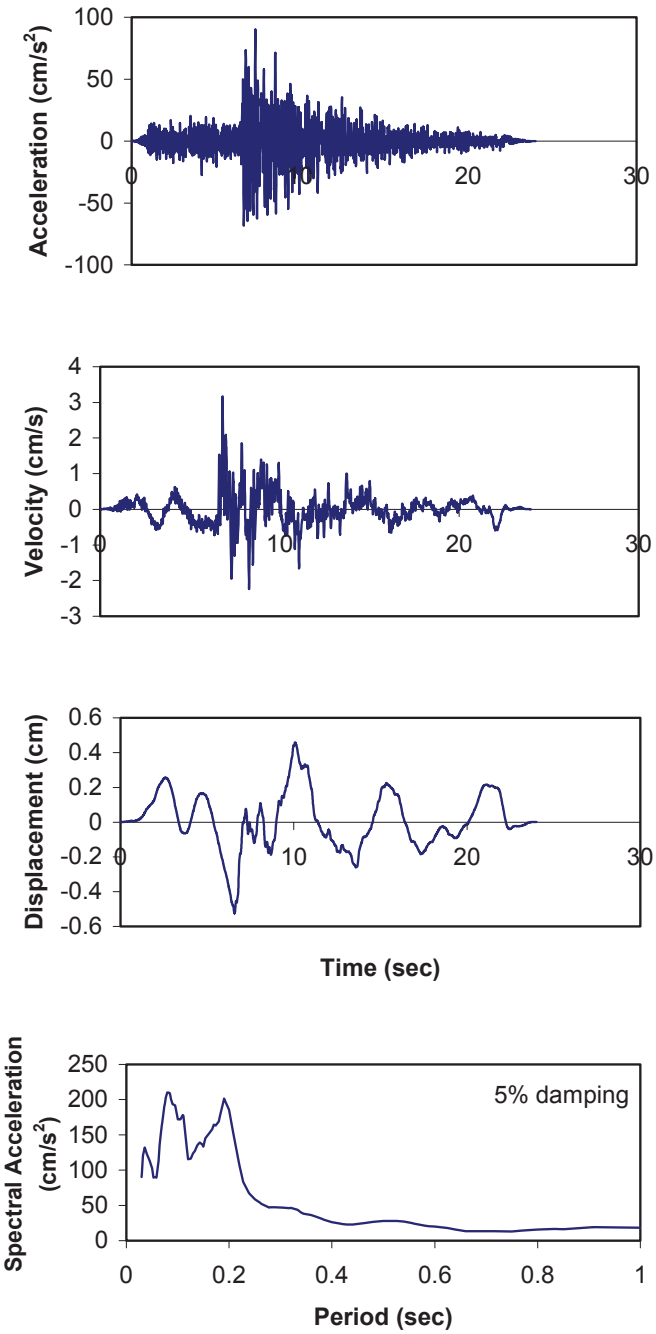
1984 Morgan Hill Earthquake – Lick Observatory Station (320)



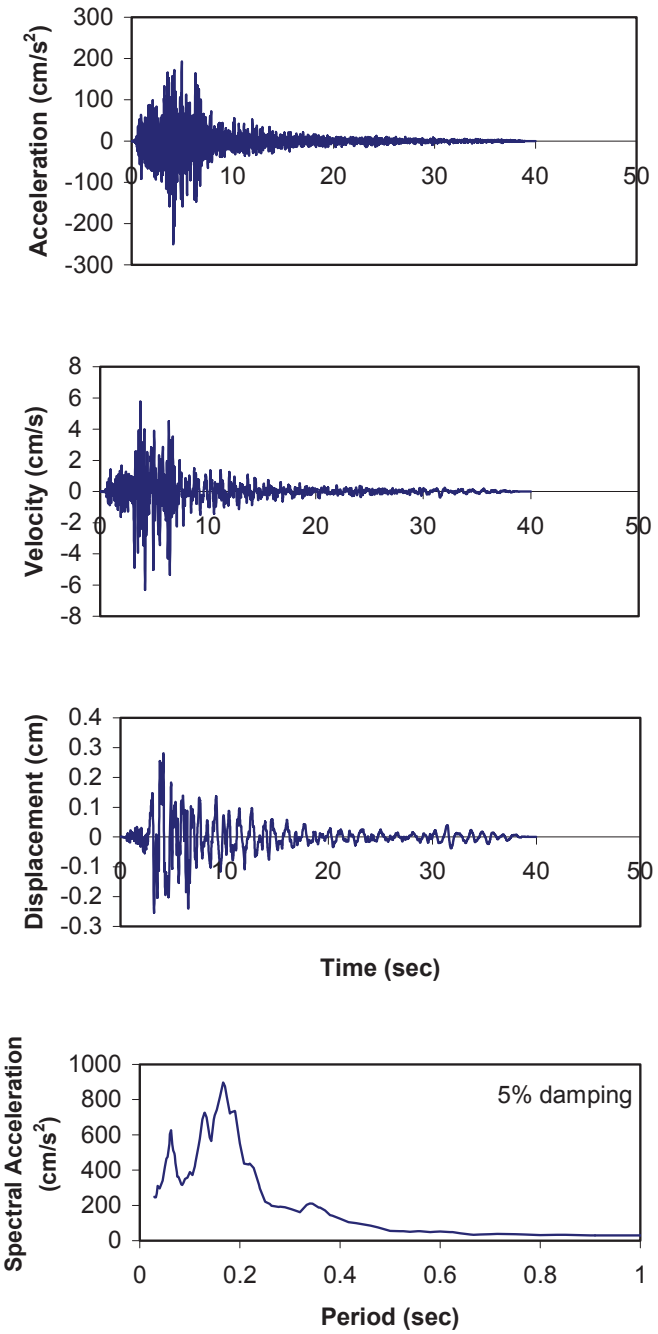
1986 N. Palm Springs – Silent Valley Station (090)



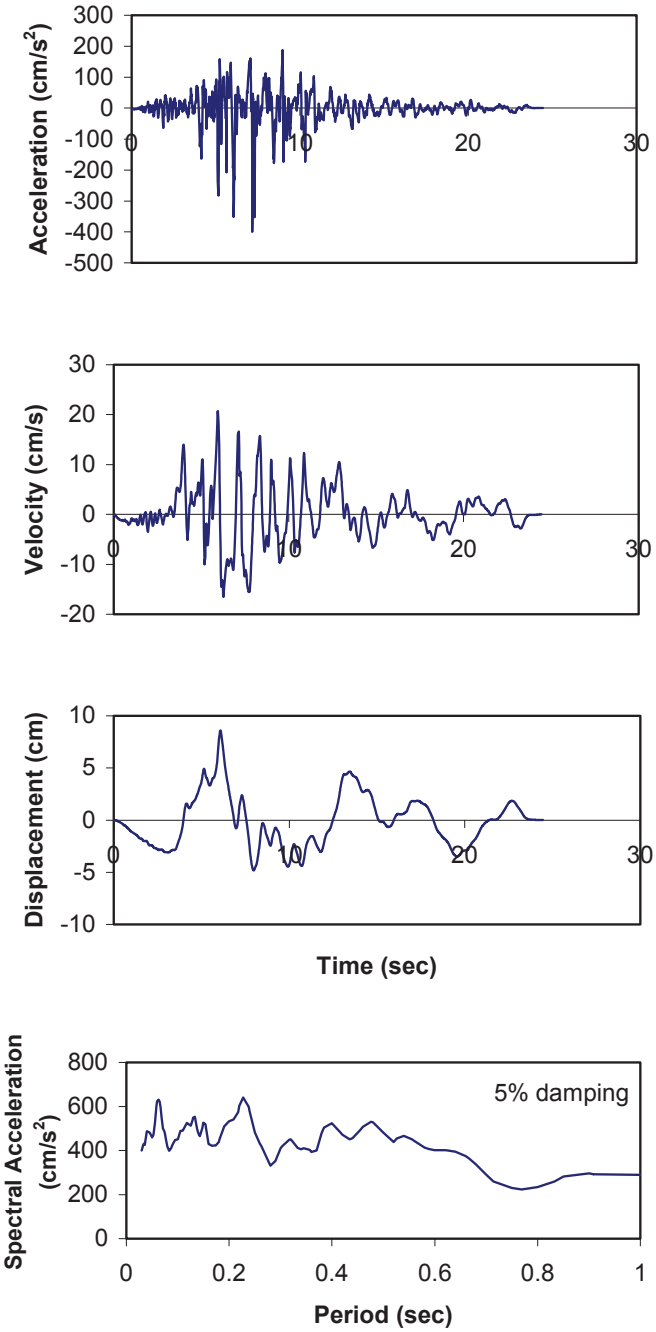
1986 N. Palm Springs – Murieta Hot Springs Station (000)



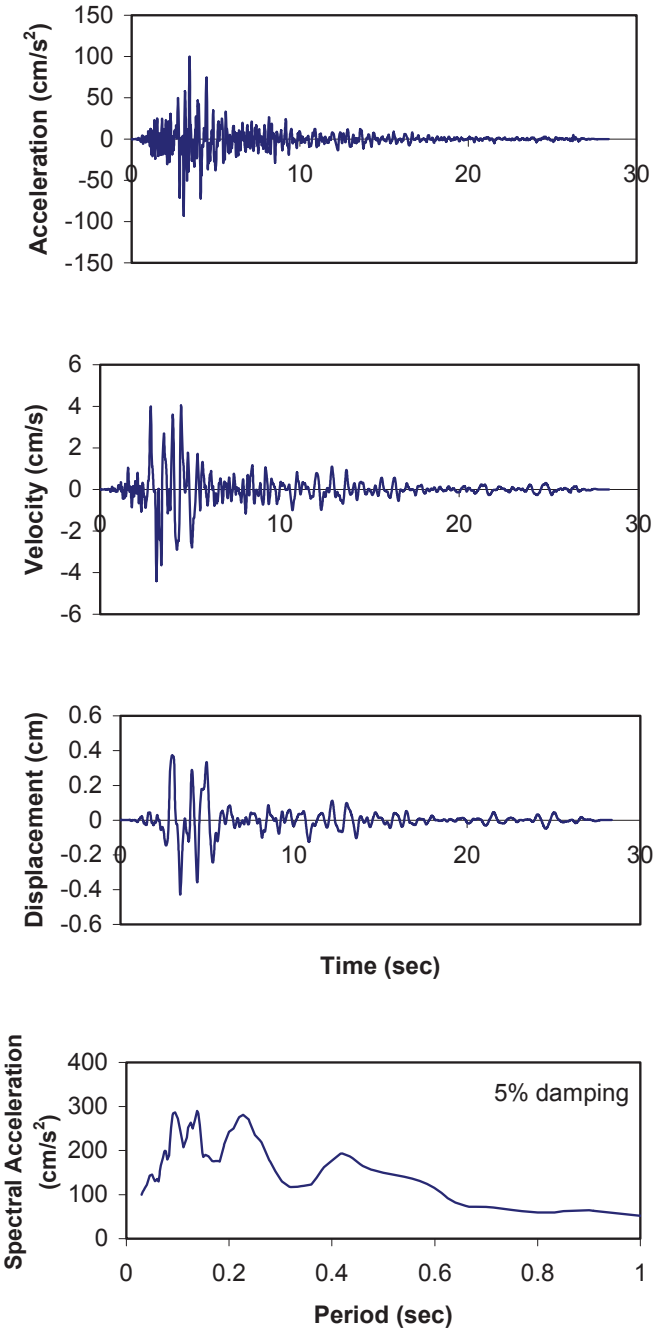
1987 Whittier Narrows Earthquake – Mnt. Wilson Station (090)



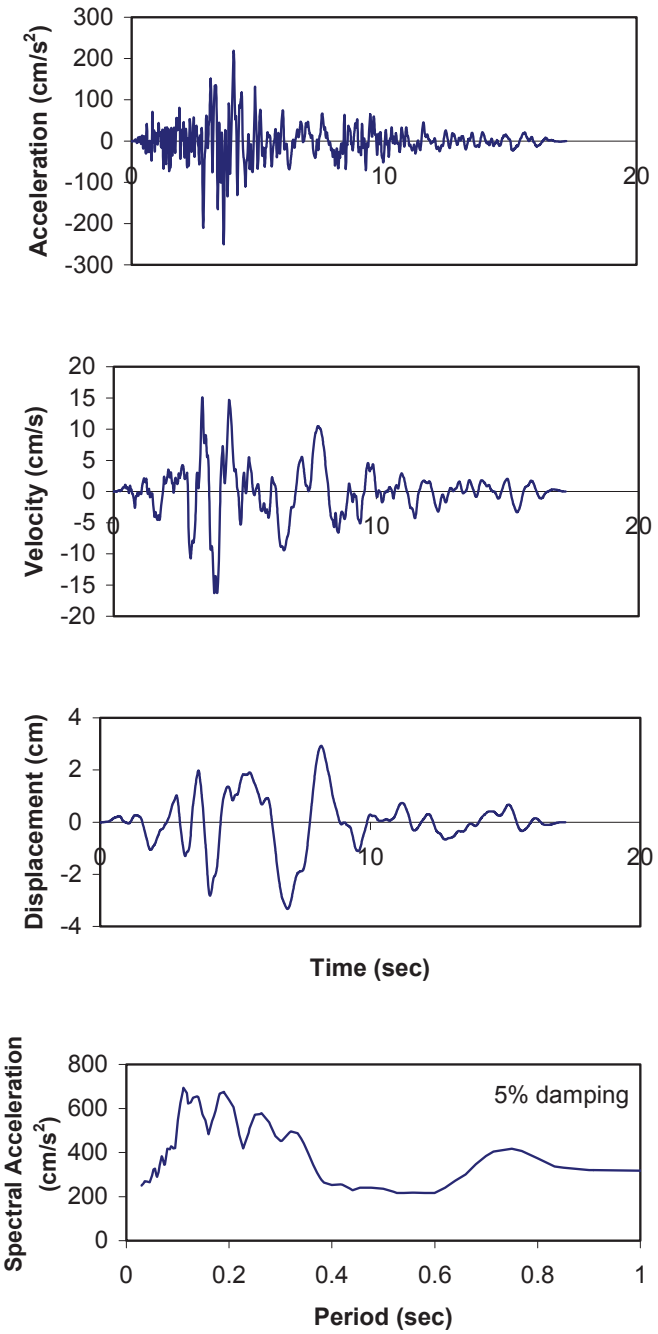
1980 Vistoria Earthquake – Cerro Prieto Station (045)



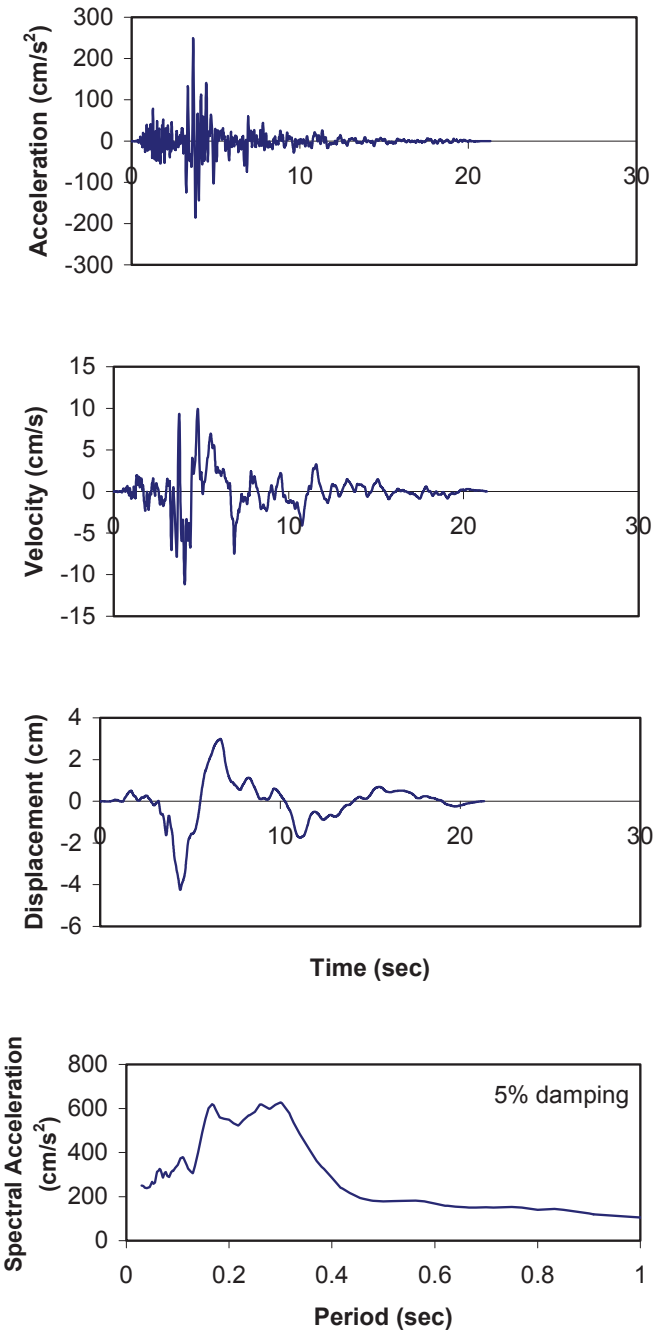
1981 Westmorland Earthquake – Supestition Camera Station (320)



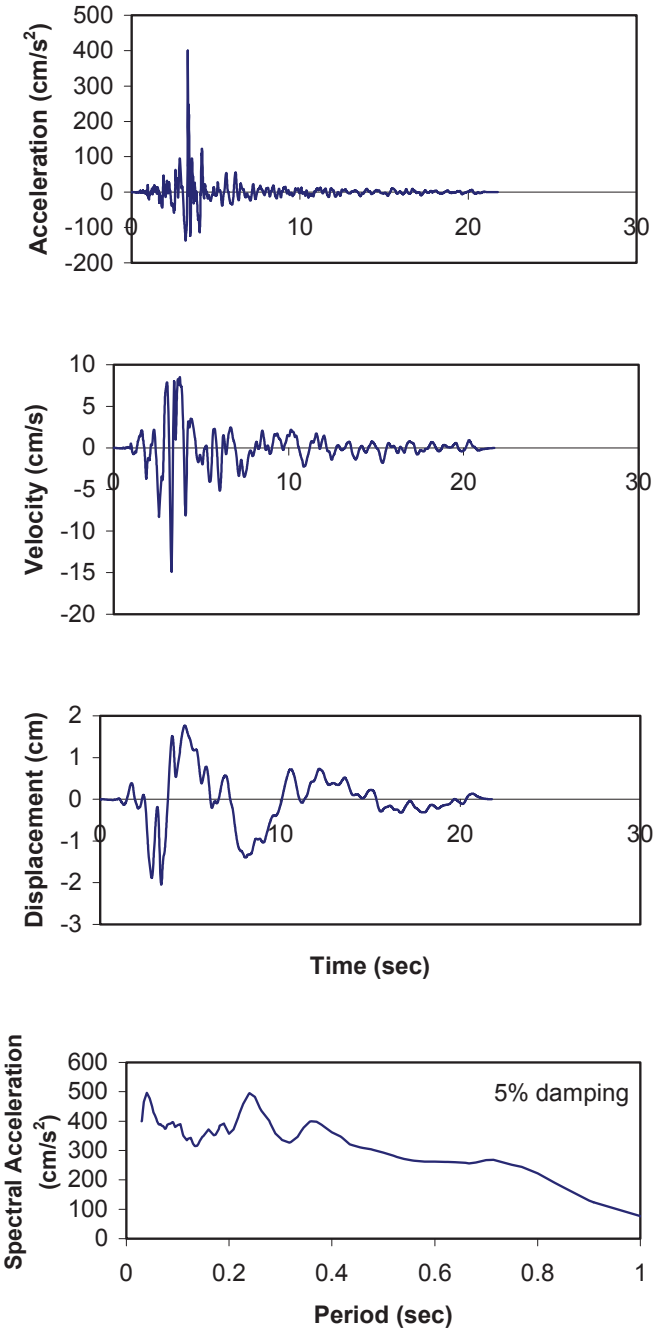
1983 Coalinga Earthquake – Oil Fields Fire Station (270)



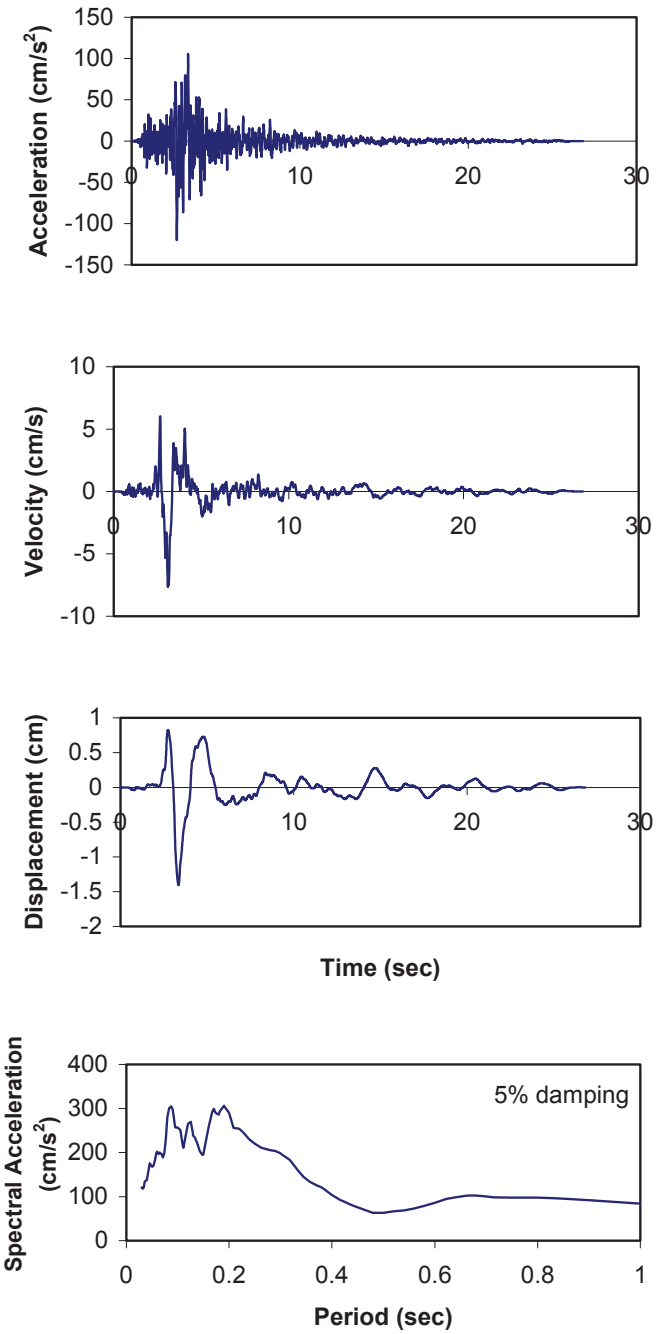
1983 Coalinga Earthquake – Skunk Hollow Station (270)



1983 Coalinga Earthquake – Oil Transmitter Station (360)



1979 Cayote Earthquake – Gilroy Array # 1 Station (320)



APPENDIX C

CETIN AND SEED (2004)

(Due to copyright laws, the full paper could not be published electronically.)

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Nonlinear shear mass participation factor (r_d) for cyclic shear stress ratio evaluation

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Accepted 14 October 2003

Abstract

Most methods for assessment of in situ seismic soil liquefaction potential require evaluation of the earthquake-induced cyclic shear stress ratio (CSR). Estimates of the in situ CSR can be developed directly, using dynamic response analyses, but it is common in 'simplified' analysis methods to develop estimates of the in situ CSR using empirical relationships. Unfortunately, the most widely used existing empirical relationships are based on limited response analyses and do not take full advantage of current knowledge of factors affecting this response problem. As a result, they are both biased and unnecessarily imprecise. This paper presents the results of a relatively comprehensive suite of site response studies (2153 site response analyses), performed using carefully selected suites of site conditions and input time histories, to provide an improved basis for development of estimates of in situ CSR using the r_d-approach. The resulting empirical correlations, developed using the Bayesian updating method, provide a much improved basis for simplified empirical evaluation of CSR as a function of (1) depth; (2) earthquake magnitude; (3) intensity of shaking; and (4) site stiffness.
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Keywords: Nonlinear shear mass; Cyclic shear stress ratio; Soil liquefaction; Dynamic response analysis

1. Introduction

Evaluation of the in situ cyclic shear stress time history induced within any soil element (or stratum) is a key component of any well-based method for assessment of the likelihood of 'triggering' (or initiating) seismically induced soil liquefaction. The seismically induced cyclic shear stress time history is, in most analysis methods, normalized by some measure of the initial normal effective stress in the soil, to give rise to the earthquake-induced cyclic shear stress ratio (CSR).

As the earthquake-induced cyclic shear stress history is a nonlinear function of the interaction between the 'input' strong motion characteristics and the characteristics affecting site response (e.g. geometry, stratigraphy, dynamic soil properties, etc.), earthquake-induced CSR time history within a soil element is a site-specific and motion-specific issue. Accordingly, it is generally most accurately assessed *directly*, based on a dynamic response analysis.

In many cases, however, direct response analyses are not performed either (1) because time and budget do not permit; (2) because necessary information/data for full response analyses are not available; or (3) it is desired to use 'simplified' empirical estimates of CSR in order to be fully compatible with the basis employed to develop a given empirical correlation for estimation of in situ liquefaction resistance. Simplified, empirical estimation of CSR is common, both for engineering design studies as well as for back-evaluation of earthquake field performance case histories for purposes of development of empirical estimates of in situ liquefaction resistance. In both applications, accuracy, lack of systematic error (or bias), and an understanding of uncertainty (or variance) are important.

2. Simplified method

At most soil sites, the cyclic shear stresses acting on horizontal planes due to seismic loading are largely dominated by cyclic shear stresses induced by vertically propagating, or nearly vertically propagating, shear waves.

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APPENDIX D

TECHNICAL LETTER SUBMITTED TO NRC COMMITTEE

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28/06/2014

Ms. Sammantha Magsino
Director,
Committee on Geological and Geotechnical Engineering
National Research Council
500 Fifth Street, NW
Washington, DC 20001

Dear Ms. Magsino:

Based upon your letter and e-mail dated May 28, 2014, and at the request of the members of the NRC Committee on the State of the Art and Practice in Earthquake Induced Soil Liquefaction Assessment, we have prepared the attached technical response letter. The co-authors of the original work and paper by Cetin et al. (2004) have been informed of these works and responses, but not all have had the opportunity to contribute due to the short time frame. As indicated by Prof. Raymond Seed of U.C. Berkeley, an additional response will be transmitted separately.

We have made a significant effort to make our response comprehensive enough to clarify unresolved issues, and have also attempted to incorporate as complete a list of our updated findings as the short time response window permitted.

Please do not hesitate to contact me if you have any questions, or if the Committee needs any further of our input.

Regards,

Prof. Dr. K. Onder Cetin

cc: Professors R.B. Seed, A. Der Kiureghian, K. Tokimatsu, R. Kayen, R. E. S. Moss, Dr. L. Harder

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Technical Response Letter

On the basis of the "info request letter" dated May 28 (Enclosure 1), within this letter/response we have prepared our written input on the subject matters listed. Due to time limitations, and with the intent of making our response available to Committee Members before their meeting on June 30th, we have followed an informal but hopefully adequately technically comprehensive approach. Our work on these issues is still in progress, and a more complete and further detailed response is intended to be included in a technical report, and one or more technical papers to be submitted in the Fall of 2014 to a journal for proper review and publication. As part of these efforts, currently, we have completed the initial re-processing of case history data, and are in the process of assessing the updated liquefaction triggering boundary curves, some of the early results of which are presented herein.

Within the scope of this technical response letter, we have attempted to follow the same order of questions addressed by the Committee members, and listed the questions raised by the Committee members in black font, followed by our response in blue.

There are a number of calculations performed to assess the impacts of proposed changes, and so we pause occasionally to present these and the findings that result. Due to the sequence of questions, this can appear a little bit chaotic and difficult to follow. As a guide to understanding, we begin by making a number of changes in the processing and/or interpretation of selected field case histories, as proposed by Professors Idriss and Boulanger, in order to test their hypotheses. We should note that we do not agree with all of their proposed changes, and that there exist additional case histories, where we disagree with their back-analyses and processing of the cases. However, due to the limited time window, we will not analytically discuss these additional cases herein. Instead, we simply present the results on the basis of the full set of revisions exactly as proposed by Professors Idriss and Boulanger, including the ones with which we disagree, in order

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to help answer the question whether the processing of these cases is the principal cause of differences between the correlations.

Next, in response to subsequent questions, we continue making additional modifications and/or changes in order to investigate other potential reasons for differences between the two correlations and procedures.

Finally, we present additional information and suggestions regarding sources of differences between the two correlations, and between the methods by which they were developed.

Question # 1: There are important differences between the Cetin et al. (2004) and Idriss and Boulanger (2004) CRR-SPT relationships. Idriss and Boulanger (2012) identified issues/questions regarding your interpretations and calculations associated with some case histories in your database, as summarized below.

Please respond to these comments, summarized below:

Question # 1(a): Re-classification of the Miller Farm CMF-10, Malden Street Unit D, Kobe #6, and ShuangTai Zi River sites as liquefied.

Miller Farm CMF-10 : As shown in Figure 14, page 192 of Holzer 1998 (USGS Professional Paper 1551-B), CMF-10 is located within the proximity of a sand boil and ground cracks. We have observed that the N values of the critical "silty sand layer" do not change significantly along a cross section extended from CMF 5 to CMF 10 (N value of 20 blows/30 cm at CMF 5 and N values of 12 and 25 blows/30cm at CMF 10). Note that CMF 5 is located (and was also classified as a liquefied site) within a similar proximity to ground cracks. Additionally, on page 186 of Charlie et al. (1998), the following information was presented:

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"We conducted three piezovane tests (CSU 3,8,9, Figure 14) at the study site in soils in which extensive lateral spreading had occurred during the earthquake, and two piezovane tests, CSU 1 and 10, in soils in which no lateral spreading had occurred."

As stated by Charlie et al. (1998), CSU 10 was performed clearly in the non-liquefied zone, which was 80 meters south of CMF 10, which was also about 80 meters south of sand boils and ground cracks. Hence, CSU10 (a CPT sounding) was observed to be located clearly in the non-liquefied zone; whereas, CMF 10 was judged to be located at midway between liquefied and non-liquefied zones, as shown in Figure 1.

On the basis of the foregoing, at the time of our initial work, we classified this case to be a liquefied site. However, now we understand that the field investigation team specifically intended CMF 10 to be located in the non-liquefied zone. Given the relative continuity from CMF 5 to CMF 10, this was difficult to infer from their reports. We accept the field investigation team's expert judgment on this case history, and we have updated the CMF 10 case history to a non-liquefied one.

For the purpose of responding to the argument that the disagreement in the processing of these selected cases histories is the principal reason behind the differences in liquefaction boundary curves, we have made this change and developed (regressed) new liquefaction triggering boundary curves by modeling the Miller Farm CMF-10 as a non-liquefied case history site.

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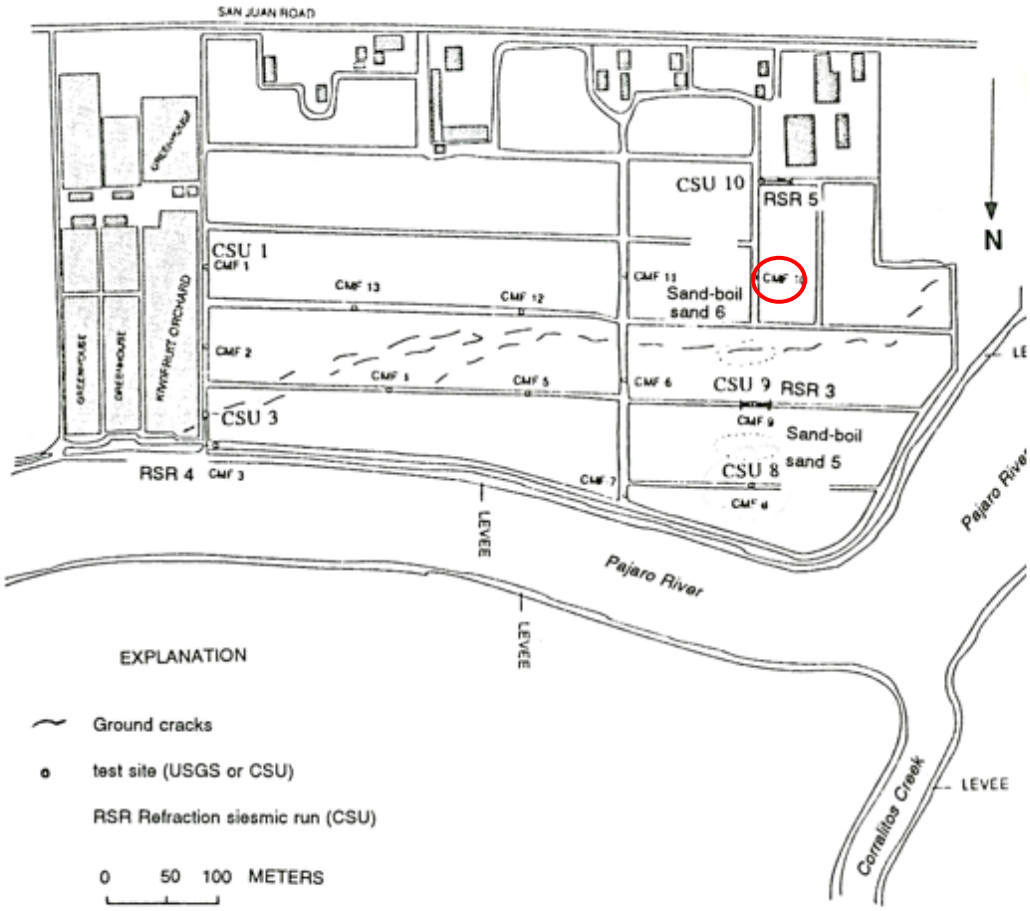


Figure 14.—Miller Farm study site (fig. 7), showing locations of liquefaction features and tests by Colorado State University (CSU) an Geological Survey (USGS).

Figure 1. Plan view of Miller Farm Site (Holzer et al.,1998).

Malden Street Unit D: The 1994 Northridge Earthquake, Malden Street Unit D case history site is composed of 3 distinct soil layers (Figure 2): Unit A is a compacted fill, located mostly above the water table, and is judged to be non-liquefiable; Unit B is a fine grained soil layer with FC>70 %, average PI = 18%, and average clay content of 31 % (Holzer et al. 1999; page 5). Clayey soils with PI=18 %, were categorically judged to be potentially non-liquefiable back when our original

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work was performed. Unit D is Pleistocene silty sand; hence it was concluded to be suspect for liquefaction-induced ground deformations. Unit D was identified as the critical layer. BH 3 and BH 5 were used to characterize the SPT N values in this critical layer. Also note that BH 3 is located within the "permanent ground deformation" zone (Figure 4 of Holzer et. al. 1999).

However, after having revisited this case history with the extended state of knowledge today, it has been tentatively decided that the Malden Street Unit D case may be excluded from Cetin et al (2014) database. We note that Malden Street Unit D was classified as a non-liquefied site in Idriss and Boulanger (2010) database. Contrary to this choice, we believe that Unit D may not be used in the updated liquefaction case history database as a non-liquefied soil layer due to the potential bias introduced in CSR estimations after cyclic softening of the upper soft lean clay layer (Unit B). The presence of these overlying soft soil layers, potentially susceptible to cyclic softening, can significantly reduce the induced cyclic shear stresses due to their significantly reduced inertial mass participation effects. This response makes the estimation of CSR or r_d values a difficult task, which is beyond the fully reliable limits of either simplified procedures or total - stress based site response analyses.

Even though, we may eventually prefer to exclude this case history site from the final Cetin et al (2014) liquefaction case history database, for the purpose of responding to the questions asked by the Committee, and responding to Professor Idriss' and Boulanger's argument that the disagreement in the processing of these cases histories is the principal reason for the differences between the liquefaction boundary curves, later in this letter, we develop boundary curves by assuming that Malden Street Site Unit D is a non-liquefied case history, consistent with the recommendations and the execution of Idriss and Boulanger (2010).

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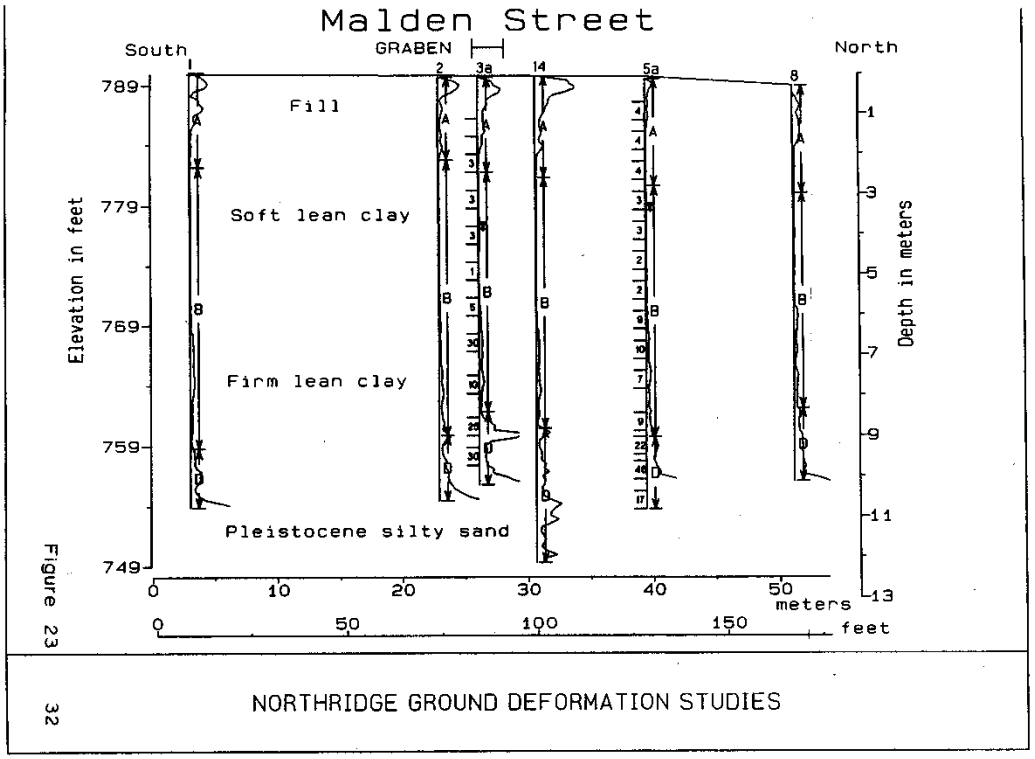


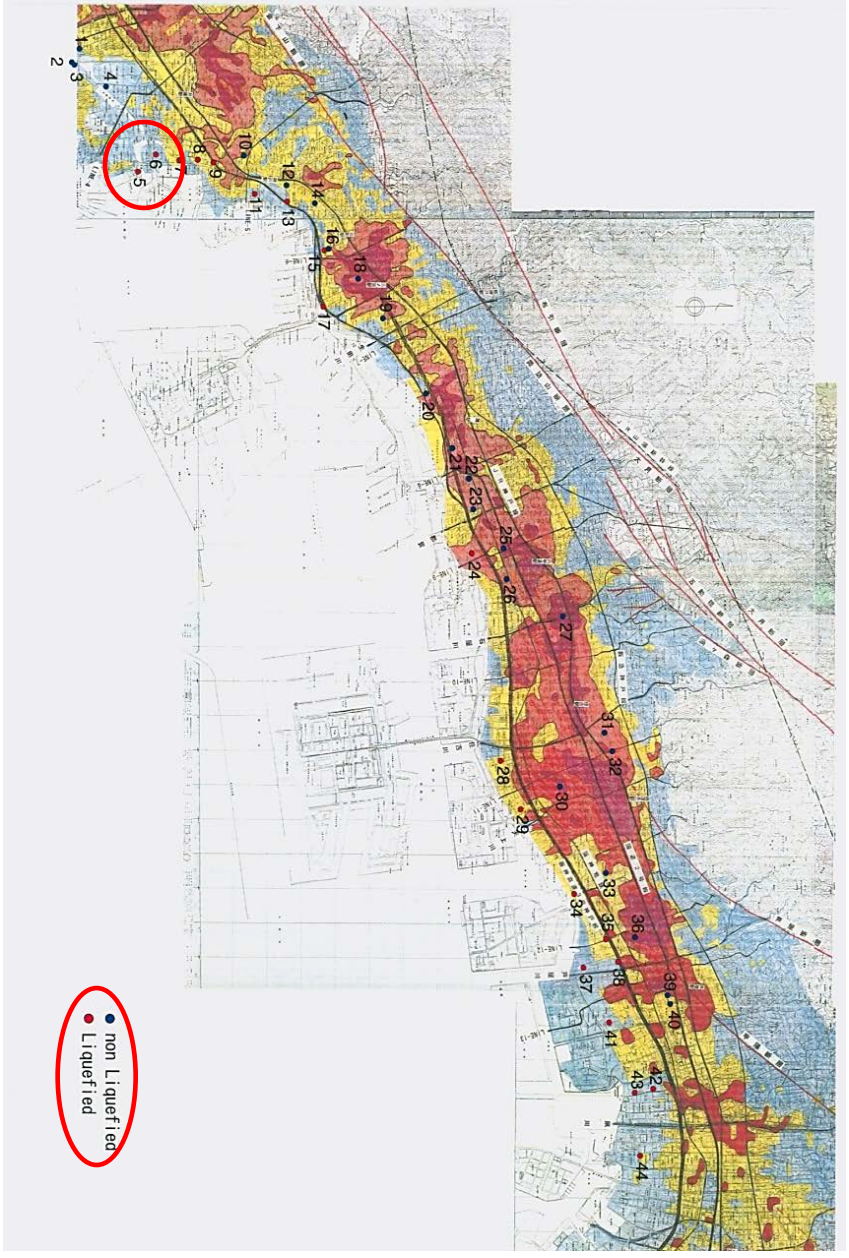
Figure 2. Malden Street Soil Profile

Kobe # 6 : As shown in Figure 3, Kobe # 6 site was shown to be a liquefied site on the overall summary map, as originally provided by Prof. Tokimatsu. However, conflicting with the map legend, the same site was listed as a non-liquefied site on the accompanying summary table, also provided by Prof. Tokimatsu. When compiling the original Cetin (2000) and Cetin et al. (2004) databases, the site was listed as "Liquefied". After recent personal communication with Prof. Tokimatsu, we have resolved this conflict, and the site was updated as a "Non-liquefied" site in the database of Cetin et al. (2014).

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Site No.	Liquefied	corrected N	Reference
1	No	51.17	2
2	No	38.47	2
3	No	49.66	2
4	No	36.03	2
5	Yes	5.97	2
6	No	21.81	2
7	Yes	24.66	2
8	Yes	21.23	1
9	Yes	10.61	2
10	No	25.22	1
11	Yes	7.59	1
12	No	24.45	2
13	Yes	11.71	1
14	No	19.85	2
15	Yes	17.65	2
16	No	21.82	1
17	Yes	19.29	1
18	No	36.48	1
19	No	20.16	1
20	No	60.94	1
21	No	29.31	1
22	No	35.38	1
23	No	20.56	1
24	Yes	20.10	1
25	No	30.98	1
26	No	33.95	1
27	No	36.99	1
28	Yes	21.93	1
29	Yes	15.38	1
30	No	40.02	1
31	No	48.01	1
32	No	24.57	1
33	No	27.38	1
34	Yes	22.50	1
35	Yes	15.48	1
36	No	28.20	1
37	Yes	18.67	1
38	Yes	17.88	1
39	No	56.20	1
40	No	33.96	1
41	Yes	12.56	1
42	Yes	10.18	1
43	Yes	13.82	1
44	Yes	6.42	1

Figure 3. Hyogoken-Nanbu case history map and summary table as originally provided by Prof. Tokimatsu

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ShuangTai Zi River: The 1975 Haicheng Earthquake, Shuang Tai Zi River Site was originally correctly classified as a non-liquefaction case history site in the original work of Cetin et al. (please refer to page 319, Appendix 2 of Cetin 2000; "Nature of Failure: Non-liquefied site"), Hence, it was also processed and assessed correctly as a non-liquefied site. (Note in the original dissertation that the critical layer is shaded by blue (lighter) shading, an indication of a non-liquefaction case history site). Because this site was originally processed and modeled as non-liquefied site, as part of the Cetin (2000) and Cetin et al. (2004) studies, there is no controversy, and no change to be made here. Unfortunately, in the final tables, it was erroneously listed as "liquefied" due to a typographical error. In the Cetin 2014 database summary tables, this typo will be corrected.

Question # 1.b): Calculation of CSR_{eq} using r_d values smaller than predicted by the Cetin et al. (2004) r_d relationship. This issue influences the location of CRR curve predominantly through six sites:

Kobe #6, Kobe #7, Rail Road #2, Panjin Chem Fertilizer Plant, Shuang Tai Zi River, and San Juan B-3. If the r_d values from the Cetin et al. (2004) relationship are used, these data points shift up from your published CRR curve.

These six case histories were re-visited to investigate the reason(s) behind not being able to re-produce the reported r_d values at Cetin et al. (2004). For these cases, r_d value was originally calculated by using Cetin and Seed (2004) r_d relationship. A typo at the third decimal point in one of the model parameters (in Excel execution of the r_d formula) was identified, and this typo was concluded to be the main reason for this problem. The possible influence of this typo was also investigated, and it was found out that it produced 5.5% difference, on the average, in the estimated r_d values. Differences between calculated and intended r_d values were smaller than this at shallow depths, and larger at greater depths. This error was not caught in the review and checking of the original work, as cases checked closely at the end were mainly shallow cases located in informative positions. During re-examination, it was also noted that some of the $V^*_{s,40}$

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values (Tables 6 and 8 of Cetin et al., 2004) were also shifted down (or up) one row during production and type-setting of the Cetin et al. (2004) manuscript. The use of incorrect $V^*_{s,40}$ values as a result of this also contributed to unsuccessful attempts to re-produce the r_d values. All these and more were shared with Professors Idriss and Boulanger as part of a personal communication dated June 29, 2010 (Enclosure 2).

For the purpose of responding to the view that:

(1) the disagreements in the processing of the cases histories (i.e.: re-classification of the Miller Farm CMF-10, Malden Street Unit D, Kobe #6, and ShuangTai Zi River sites as liquefied) are the reason behind the differences in liquefaction triggering boundary curves, and

(2) r_d values smaller than the values predicted by Cetin et al. (2004) r_d relationship significantly further influence the location of CRR curve predominantly through six sites: Kobe #6, Kobe #7, Rail Road #2, Panjin Chem Fertilizer Plant, Shuang Tai Zi River, and San Juan B-3;

we have taken the following actions:

(1) Even though we have expressed our disagreements with some of the Idriss and Boulanger (2010) choices of site classifications and assessments, as noted earlier, we have updated the (a) Miller Farm CMF 10, (b) Malden Street Site Unit D, and (c) Kobe # 6 sites exactly as they propose, and (d) we have verified but not changed the already correct ShuangTai Zi River site classification, modeling all four of these sites as non-liquefied, exactly as suggested by Idriss and Boulanger (2010).

(2) We have updated the r_d values (and corresponding CSR values as well) with the typo-free version in summary Excel files of the Cetin et al (2002) r_d relationship for the following sites:

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Kobe #6, Kobe #7, Rail Road #2, Panjin Chem Fertilizer Plant, Shuang Tai Zi River, and San Juan B-3 sites.

(3) Moreover, even though we have noted our disagreement, as will be described later in this letter, we have also updated the Kobe # 7 case history site's critical soil layer SPT-N value, again to exactly the same value as suggested by Idriss and Boulanger (2010) (i.e.: critical depth was adopted as 3.3 m and a single SPT blowcount of N=8 blows/30 cm was used to characterize the critical stratum).

(4) As will be discussed later in this letter, we have also updated our unit weights for Railroad # 2 site, exactly as suggested by Idriss and Boulanger (2010).

(5) And finally, we have re-assessed the liquefaction boundary curves after having incorporated the above listed modifications/updates into the Cetin et al. (2004) database.

In Figure 4 (a), the solid curve represents the original 50 % probability of liquefaction triggering (P_L=50 %) boundary curve of Cetin et al. (2004), and the dashed red curve represents the updated boundary curve for P_L=50 % after having made exactly the changes proposed by Professors Idriss and Boulanger as listed above, even though for some of which we have noted our disagreement. Figure 4(b) presents P_L=15 % curve for the original regressed relationship of Cetin et al. (2004) and newly assessed P_L=15 % curve with the same changes implemented. When updating the boundary curves, for the purpose of eliminating the effects of K_σ, MSF and fines corrections, maximum likelihood assessment was performed on the intercept term (θ_6) of Cetin et al. (2002) likelihood function (i.e.: Eqn 16). The updated (θ_6) was estimated as 17.25 as compared to its earlier version of 16.85. As clearly addressed by these figures, contrary to Idriss and Boulanger (2010 and 2012) conclusions, Cetin et al. liquefaction triggering boundary curves are not significantly affected by the listed differences.

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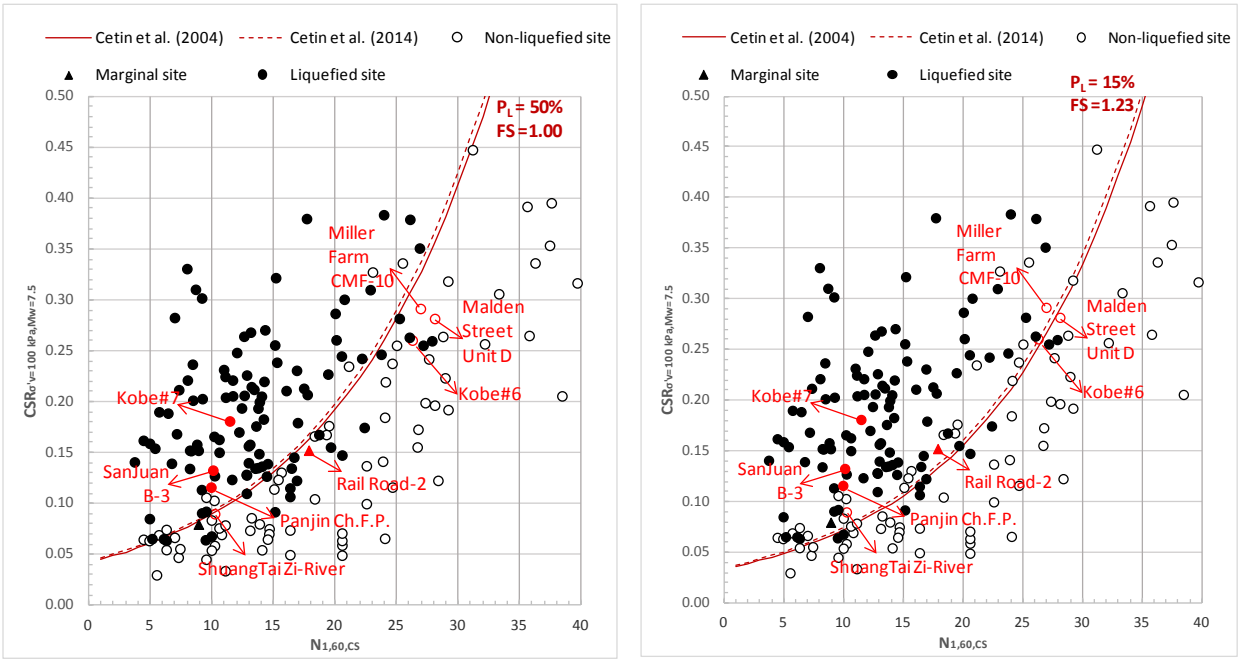


Figure 4. Liquefaction triggering curves after introducing recommendations by Idriss and Boulanger (2010 and 2012) as compared with Cetin et al. (2004) boundary curves.

This conclusion was no surprise. Please note that Cetin (2000) studied the effects of sampling bias on the liquefaction triggering boundary curves through weighting of the case history data. Through a weighting scheme to address field case history sampling bias (please refer to Cetin (2000) and Cetin et al. (2002) for a complete discussion of this sampling bias adjustment), the number of non-liquefied sites was increased by 20 % and the number of liquefied sites was reduced by 20 %, which led to an overall weighting factor of 1.5. In simpler terms, the weighting scheme increased roughly the number of non-liquefied sites from 91 to 109, and decreased the number of liquefied sites from 109 to 87. As shown in Figure 5 (adopted from Cetin (2000)), "adding" 18 (=109-91) and "deleting" 22 (109-87) case history sites through the outlined weighting scheme produced only a modest shift in the liquefaction triggering curves. Hence, it was not surprising that no major shift in liquefaction boundary curves resulted, after incorporating

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even all of the recommendations of Idriss and Boulanger (2010 and 2012), which involve only 8 case history sites, only seven of which actually changed. However, it should be also noted that we appreciate all the suggestions and efforts aiming to improve case history data quality and processing details, independent of their impact on the overall liquefaction triggering boundary curves.

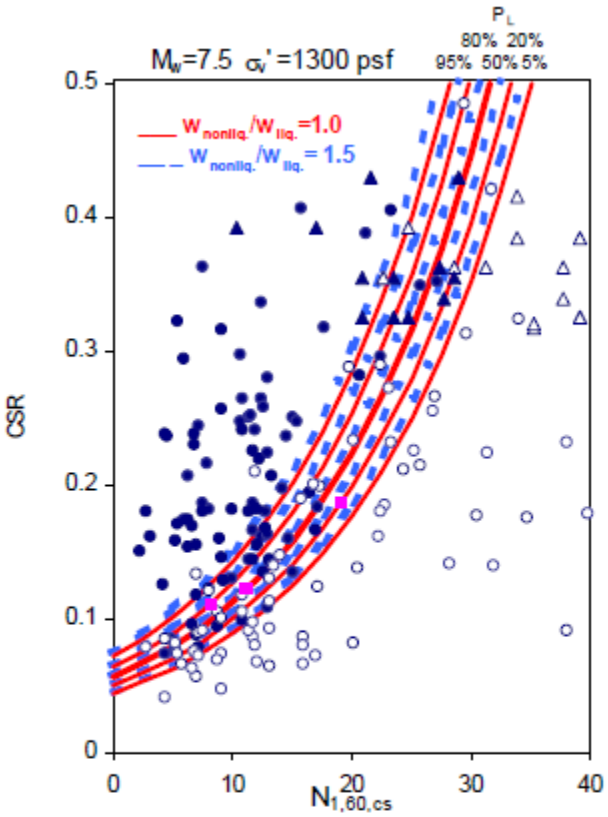


Figure 5. Effects of weighting of liquefied and non-liquefied field case histories to address sampling disparity (adopted from Cetin 2000)

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Question # 1(c): Selection of a representative blow count for the Kobe#7 site from the measured values. Consideration of the liquefaction manifestation at this site on the calculation of a representative blow count.

Kobe # 7 case history data input, as provided by Prof. Tokimatsu, is shown in Figure 6. On the basis of the summary table and the borehole log data provided, the N values corresponding to depth range of 4.3 to 8.3 m was judged to be critical. The SPT blow count at 6.3 m depth was excluded in the averaging scheme by Prof. Tokimatsu, due to its anomalously high N value, which suggested potential gravel interference. However, for this site, Idriss and Boulanger (2010) adopted 3.3 m depth as their critical depth and used the single N value of 8 blows/30 cm. We note that 3.3 m depth is very close to the reported ground water table depth of 3.16 m. We respectfully disagree with the interpretation of Professors Idriss and Boulanger on this case. We believe that N-values are locally variable, sometimes due to localized variability in soil conditions, and sometimes due to SPT procedures (e.g. variable hammer energy, etc.). As a result, it is our practice in evaluating these types of field case histories to use multiple N-values within a stratum such as this one, so long as there are no excessive and systematic variations suggesting a change in stratum. After normalization for effective overburden stress effects (C_N), the resulting N_1 values for the five SPT tests in this depth range are, in our opinion, sufficiently similar (with relatively typical, but non-systematic variability) so as to represent a continuous stratum. If we were all to select the single lowest N-value at any case history site, then all of the correlations would change significantly, and engineers desiring to use those correlations for forward analyses of projects would also have to select the single lowest N-value among their data sets for compatibility. That would result in a very volatile forward analysis situation, in which a single low N-value (which can occur for a variety of reasons, and not be “representative”) would skew the triggering analysis results. That is why we disagree with the interpretation of Professors Idriss and Boulanger in this case. After recent personal communication with Prof. Tokimatsu, the singular lower SPT N-value of 8 blows/30 cm was decided to be averaged in with the rest of the

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N values, and the critical depth was extended to cover the depth range of 3.3 m to 8.3 m. in Cetin et al (2014).

However, again for the purpose of investigating the potential impact of the alternate interpretation of this case by Professors Idriss and Boulanger, the regression shown previously in Figures 4a and 4b was performed with an N-value of 8 blows/30 cm, again exactly as proposed by Professors Idriss and Boulanger.

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No.	7		kt00991									
ACC=	400		liquefied							STDEV of corrected N		1.86
W.T=	3.16									Average of corrected N		24.68
	tf/m3	tf/m2	tf/m2									
DEPTH(m)	Density	SV	SVD	Nvalue	C_R	C_S	C_B	C_E	C_N	Corrected N	F.C.	Soil quality
4.3		8.40	7.26	21	0.85	1	1	1.22	1.17	25.56	0.00	
6.3		12.60	9.46	32	0.95	1	1	1.22	1.03	38.14	0.12	
7.3		14.70	10.56	23	0.95	1	1	1.22	0.97	25.94	0.00	
8.3		16.80	11.66	21	0.95	1	1	1.22	0.93	22.54	0.00	

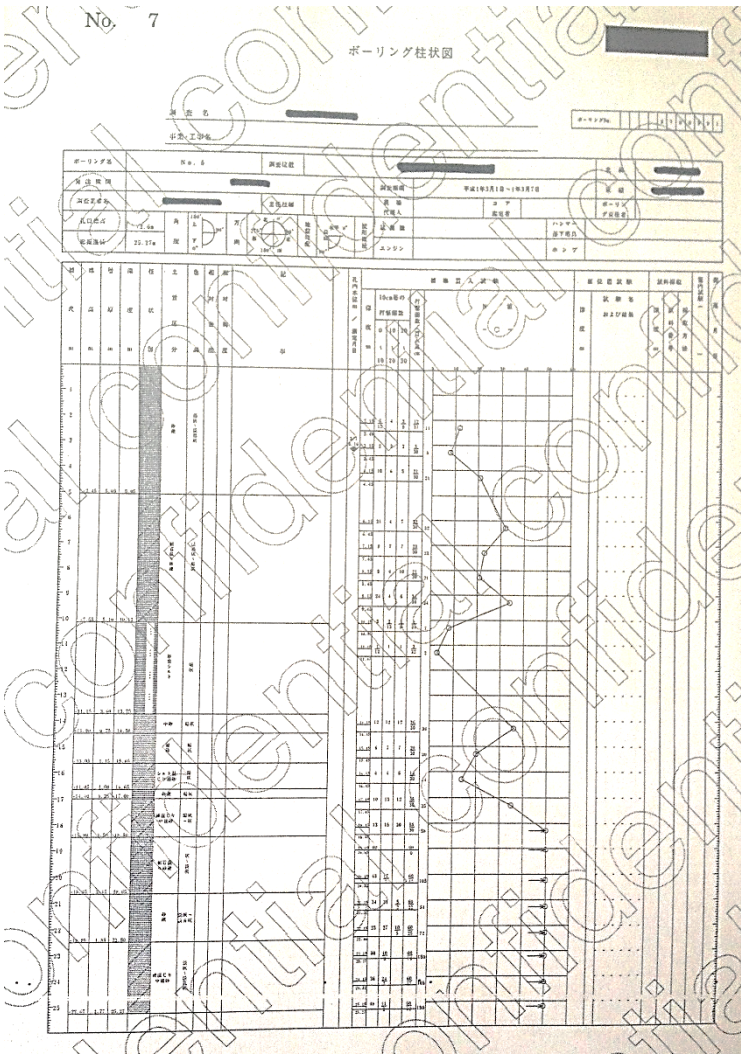


Figure 6. Kobe #7 case history details (as provided by Prof. Tokimatsu)

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Question # 1(d): The influence of using small total unit weights (~104 pcf) for the liquefiable soils. These unit weights are used to compute the effective stresses for the blow count stress correction and for CSR_{eq}. This issue influences the location of CRR curve predominantly through site Rail Road #2.

We have updated the unit weights for the RailRoad # 2 Site, using exactly the same unit weights proposed by Dr's Idriss and Boulanger, and this change was also incorporated in the regression presented in Figures 4(a) and 4(b).

As discussed earlier, all of these changes (some of which we agree with, and some of which we do not) have been implemented to test the statement by Prof's. Idriss and Boulanger that these are the principal reasons for the differences between the two correlations, and that our assessed correlation's and boundary curves will collapse to agree with theirs if these changes are implemented. As shown in Figures 4(a) and 4(b), these changes instead resulted in only moderate shifts in the regressed correlation. This conclusion does not support the argument that the differences are due primarily to the processing of eight (actually seven) case history sites. As a result, it is necessary to look for further explanations for the differences between the two correlations, which will be discussed next.

Question # 1(e): If you believe there are additional reasons for the differences between the CRR curves, please comment.

Inspired by these ongoing technical discussions, we have extended our attention to all of the remaining case histories, and we have re-processed them by taking advantage of updates in the current state of knowledge and comments and suggestions made by others. Changes made here include the following:

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(1) For every case history, r_d values were re-calculated with the typo-free Excel version of Cetin and Seed (2004) formulation (except for the 43 case histories, for which site-specific seismic site response analyses were performed to directly calculate CSRs (and r_d values)). When doing so, $V_{s,12}$ values were also re-evaluated for consistency and robustness, comparing the values developed originally by Cetin (2000) with values based on averaging of Vs estimates from suites of empirical estimation approaches. This resulted in no major changes in any individual $V_{s,12}$ value, but it produced a 5 % increase in $V_{s,12}$ values, in the overall average. That, in turn, had no significant overall effect on CSR's calculated.

(2) Similarly, unit weights for each case history site were updated consistent with the information summarized in Table 1, unless case specific information states otherwise. These unit weights serve to provide overall values of total and effective stresses for the compiled case histories that are, on average , very close to (and slightly higher than) the average values of the common case histories as modeled by Professors Idriss and Boulanger. It should be noted here that the unit weights used by Idriss and Boulanger do not vary systematically as a function of density or N-value, or depth, for either cohesionless or cohesive soils. The unit weight scheme of Table 1 will not be our final unit weights, as we will slightly tailor unit weights by using the types of estimates produced with Table 1, in conjunction with additional information, for cases where such information is available. However, it appears that our overall average unit weights will be very close to the averages used herein, but we are still completing that work and it will have to be carefully reviewed before it is final.

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Table 1. Unit weights as used in Cetin et al. (2014)

For granular soil layers		
SPT-N_{60} (blows / ft)	γ_{wet} (lb/ft^3)	γ_{sat} (lb/ft^3)
0 - 4	90	110
5 - 10	110	120
11 - 30	120	125
30 - 50	125	135
For fine grained soil layers		
0 - 4	100	110
5 - 8	110	120
9 - 16	115	125

These were all the corrections common to every case history site. Additional modifications were then made to selected individual case histories as follow:

(1) The following three sites were modeled as non-liquefied: (a) Miller Farm CMF-10, (b) Malden Street Unit D, and (c) Kobe #6. As discussed previously, we consider back-analysis of the Malden Street Unit D site to be challenging due to the soft overlying cohesive layer of moderate plasticity, and this case will be deleted in our final assessments. But to be responsive to the questions asked, we are making even the changes proposed by Professors Idriss and Boulanger that we do not agree with within the scope of this short-term iteration. The depth range for the critical stratum depth for the Kobe #7 case was extended upwards slightly to encompass depths of between 3.3 to 8.3 meters, which had the effect of newly including the SPT N-value measured at a depth of 3.3 m among the four N-values used to characterize this stratum.

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(2) Based upon improved understanding and recent developments in strong ground motion seismology, and findings of the NGA program, over the past 14 years, some of the historical earthquake magnitudes and peak ground acceleration levels were also updated. These changes affected only a limited sub-set of cases, and none of these changes were judged to be significant.

(3) Similarly, even though the number is limited, for a few cases, average fines contents as well as average critical depth and average N values were recalculated. These changes, too, were of little overall significance and were made only for completeness.

Table 2 summarizes a complete list of the impacts of all of these changes, expressed as the overall averages of key input parameters in the Cetin et al. (2004) and (2014) databases.

Table 2. A summary of **non-weighted** average input parameters

Parameter	Cetin et al. 2004		Cetin et al. 2014	
	Mean	St. Dev.	Mean	St. Dev.
a_{max} (g)	0.30	0.16	0.30	0.16
M_w	7.05	0.48	7.01	0.48
FC (%)	16.77	20.40	16.26	19.53
d_{cr} (ft)	17.57	8.44	17.27	7.70
$(N_1)_{60}$	17.64	12.42	16.61	11.67
γ_{above_GWT} (pcf)	98.23	7.92	104.85	10.76
γ_{below_GWT} (pcf)	108.28	8.06	120.60	6.31
r_d	0.85	0.12	0.90	0.09
σ'_v (psf)	1203	624	1352	626
σ_v (psf)	1860	980	1990	931
$V_{s,40}$ ft (ft/s)	591.73	69.37	632.01	170.42
CSR	0.250	0.127	0.254	0.128

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Figure 7 further illustrates the effects of these changes, and their impacts on individual case histories, as well as on the overall case history database. In this figure, the black symbols represent the values from the Cetin (2002, 2004) database, and the red symbols represent the modified values for the analysis that is performed herein. Note from Table 2 and Figure 7 that the average changes in the input parameters are generally minor and usually non-systematic, except for (a) r_d, (b) total vertical stress and (c) effective vertical stress terms, which increased by about 6%, 7 % and 12% (on average), respectively, due to elimination of the typo in the original excel spreadsheet execution of r_d formulation, and the modifications of unit weights.

More importantly, it should be noted that:

(1) The overall, average shift in $N_{1,60}$ values is a decrease of 1.03 blows/ft, due mainly to the increased effective overburden stress (and the resulting decrease in C_N values); and

(2) The overall, average change in CSR values is an increase of 1.6%. This 1.6 % increase is the result of the largely offsetting effects of (a) increased unit weights (which served to decrease CSR values), and (b) increased r_d values as the spread sheet typo was corrected (as presented previously), which tended to increase CSR values.

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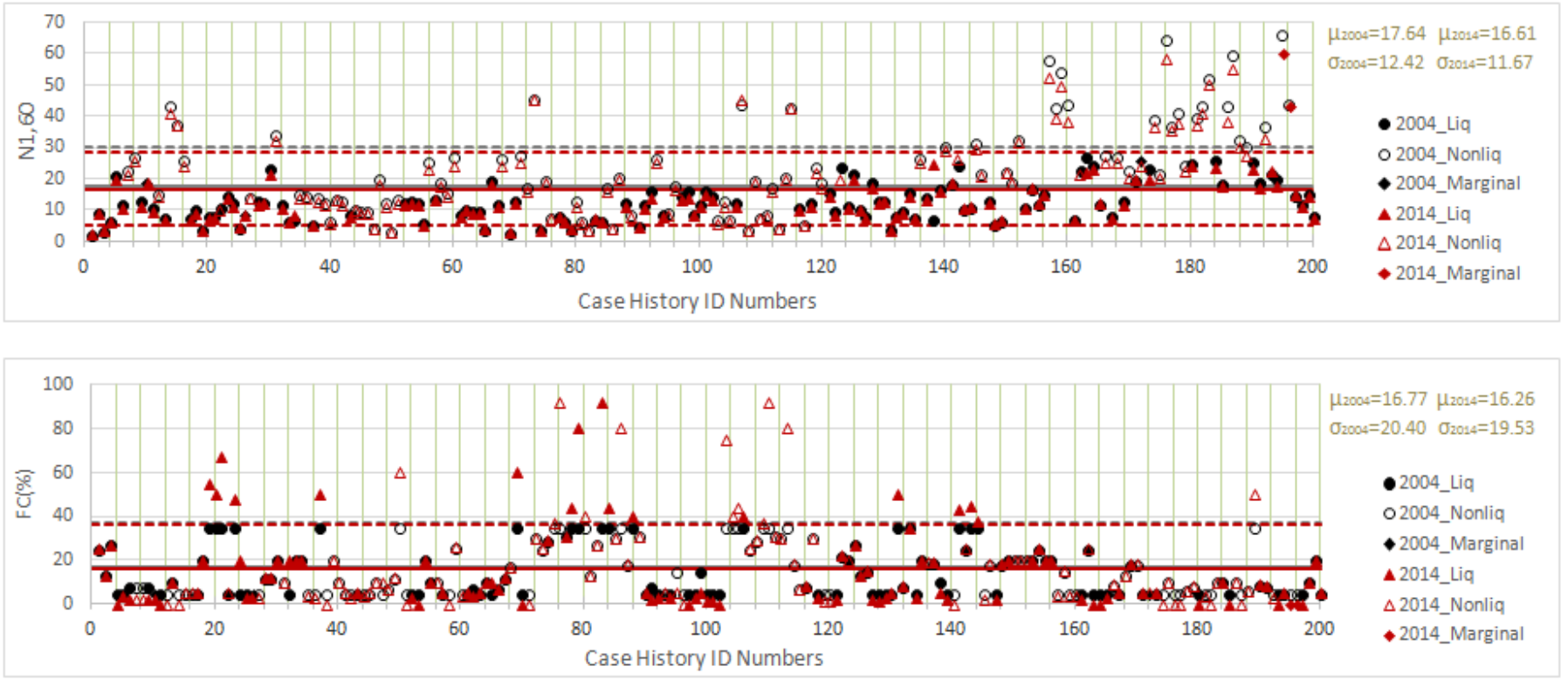


Figure 7. A summary of changes in input parameters

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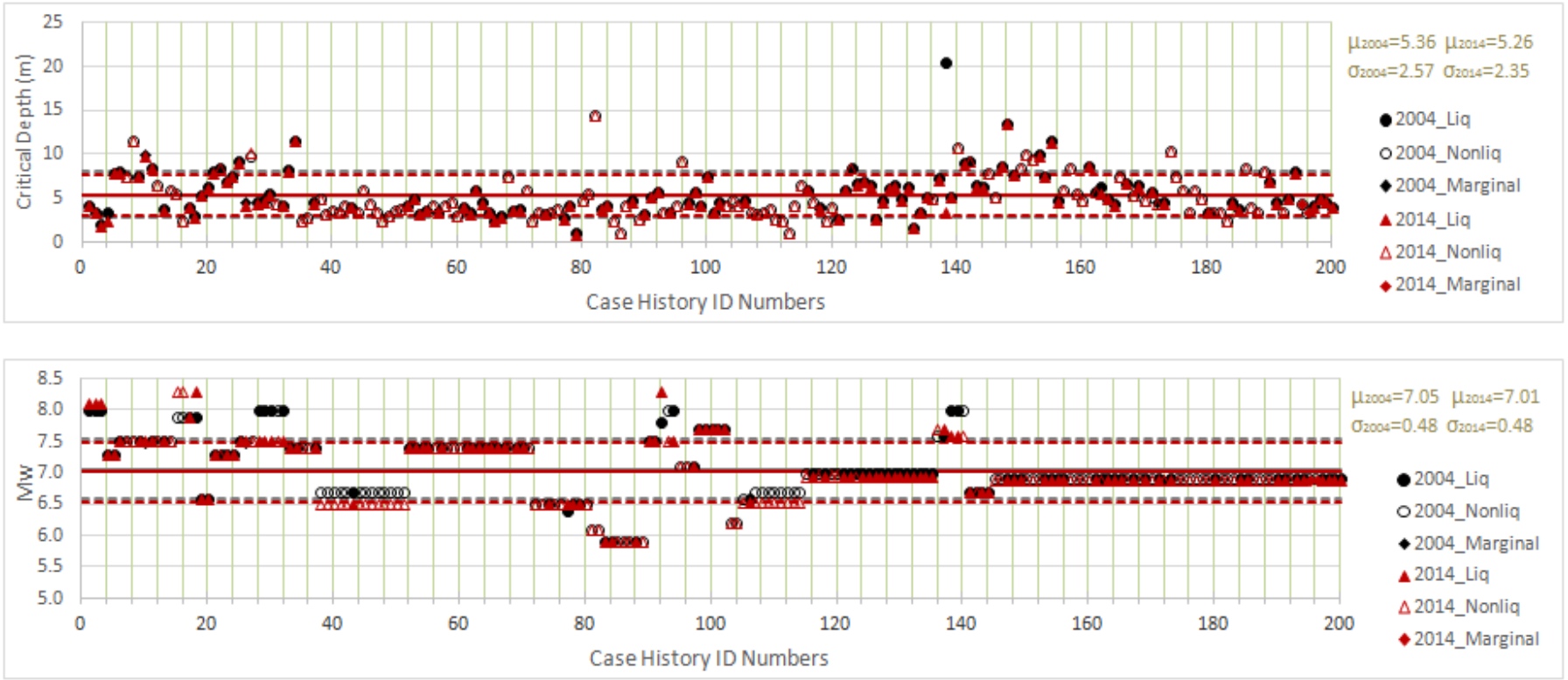


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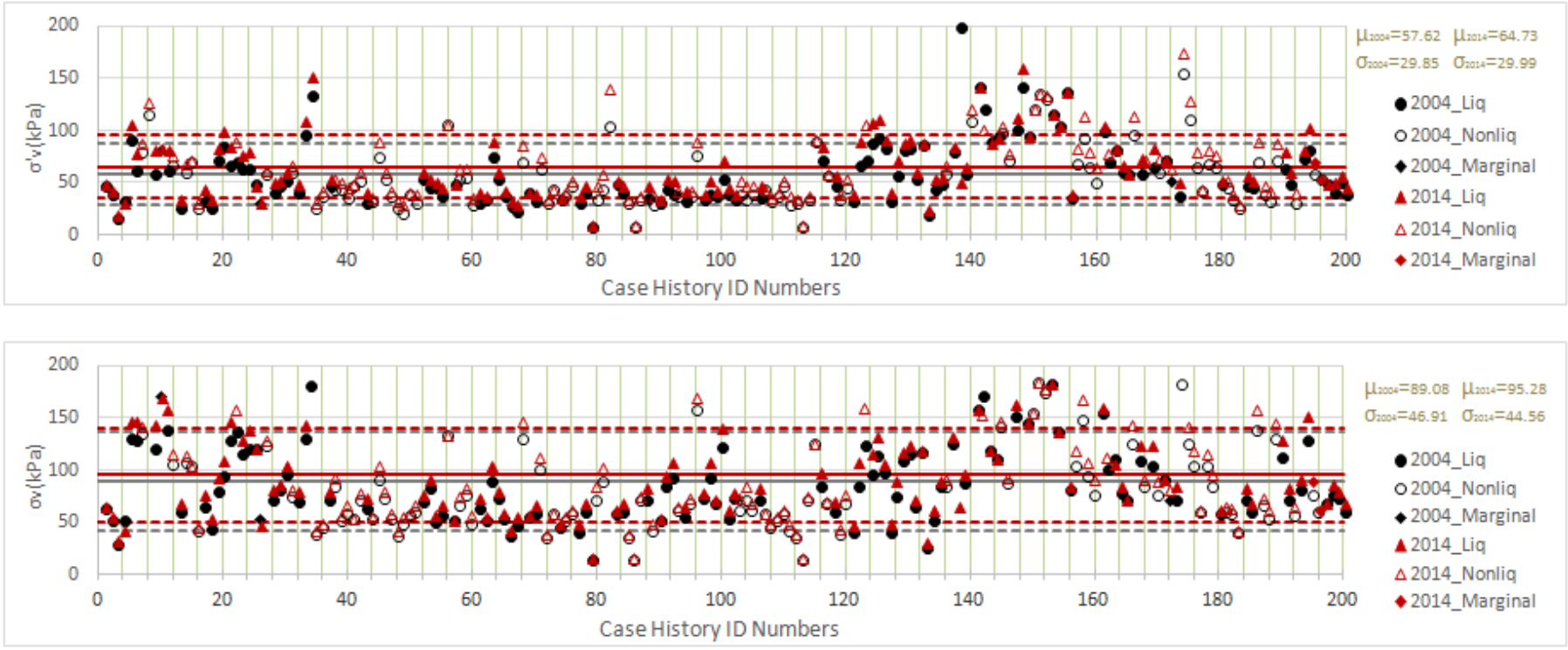


Figure 7. A summary of changes in input parameters

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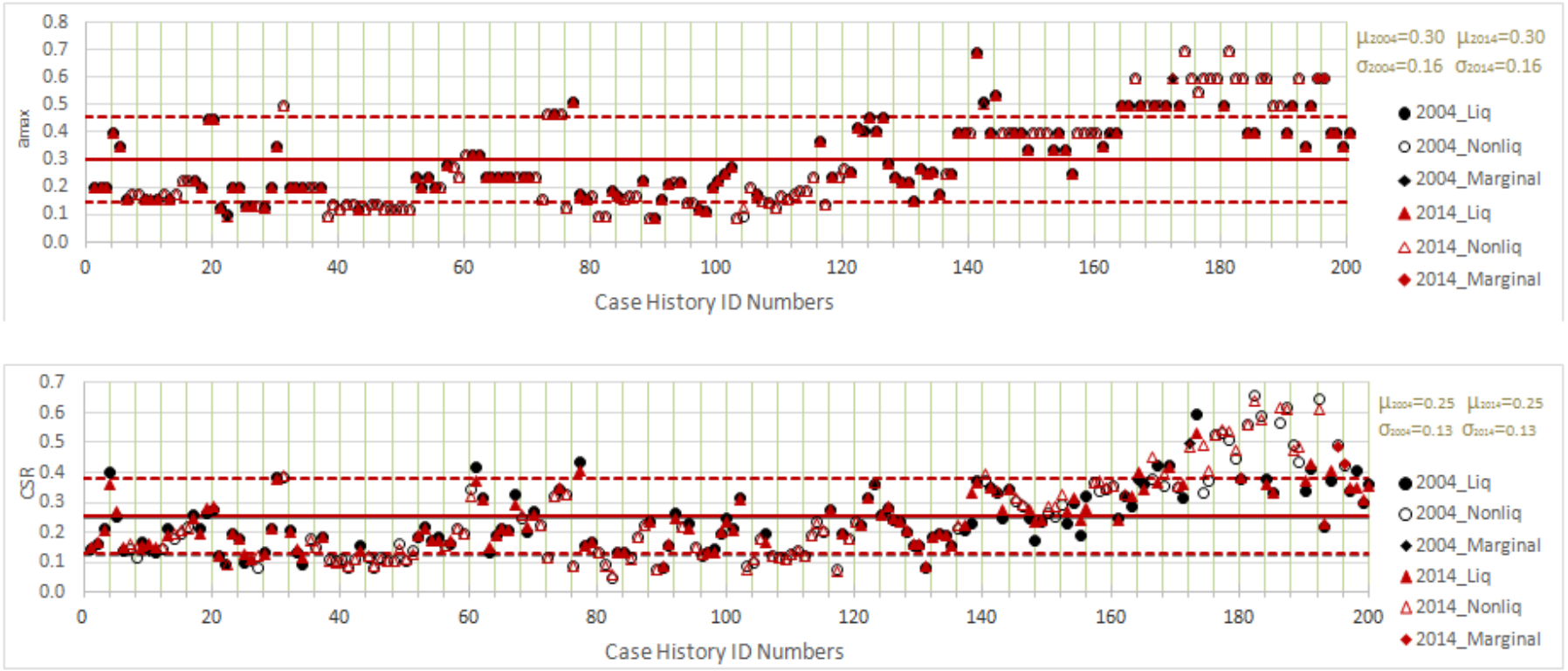


Figure 7. A summary of changes in input parameters

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After having incorporated all these updates, new liquefaction triggering boundary curves were then re-assessed by using the maximum likelihood methodology, as described in Cetin (2000) and Cetin et al. (2004) on two alternative limit state models:

Alternative Model 1: only the intercept (θ_6) term of the Cetin et al. (2002) functional form is assessed. This choice enabled the liquefaction triggering boundary curve to shift left or right, without changing the K_σ, MSF, fines corrections and the model uncertainty.

Alternative Model 2: K_σ, MSF correction parameters along with the intercept parameter (i.e: θ_2, θ_3, θ_4, θ_6) are assessed. Fines correction parameters were excluded from the update again along with the model uncertainty term to achieve a stable and comparable solution.

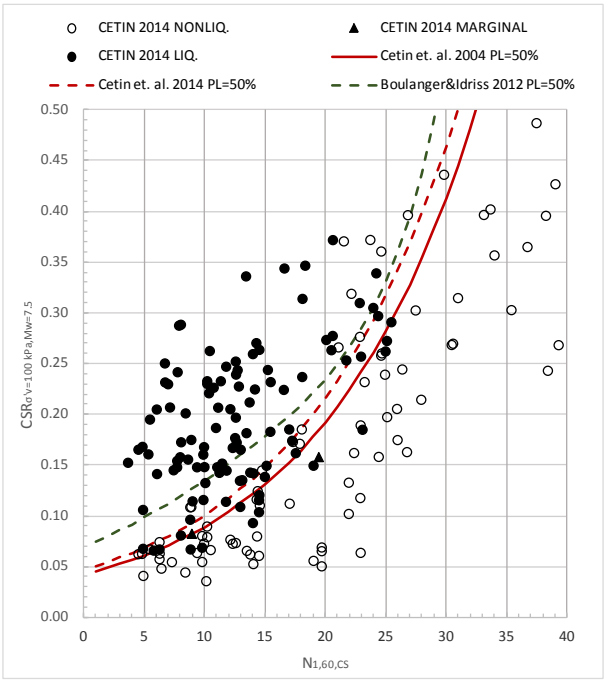
Further adjustments will be made, for completeness and for consistency, in the next couple of months, as the final updates are performed for the eventual revised correlation. But this current iteration will serve well to illustrate the effects of these clear and understandable modifications to this database, and based on the progress of our work to date, it appears that the resulting updated correlation presented here can be expected to be very close to the final relationship.

The resulting new (modified) boundary curves on the basis of Alternative 1, are shown in Figure 8(a), along with the corresponding Cetin et al. (2004), and Boulanger and Idriss (2012) probabilistic boundary curves, for P_L = 50 and 15%, respectively. Figure 8(b) also shows the newly regressed (modified) boundary curves by using the limit state function defined in Alternative 2, again for P_L = 50 and 15 % along with the deterministic boundary curve of Seed et al. (1985).

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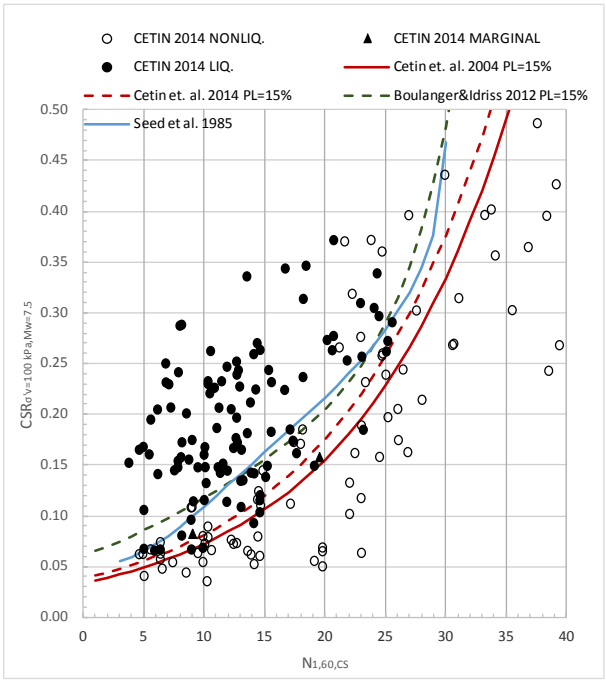


Figure 8(a). Updated Liquefaction triggering curves, and comparisons with previous triggering curves proposed by Cetin et al. and by others (Alternative 1)

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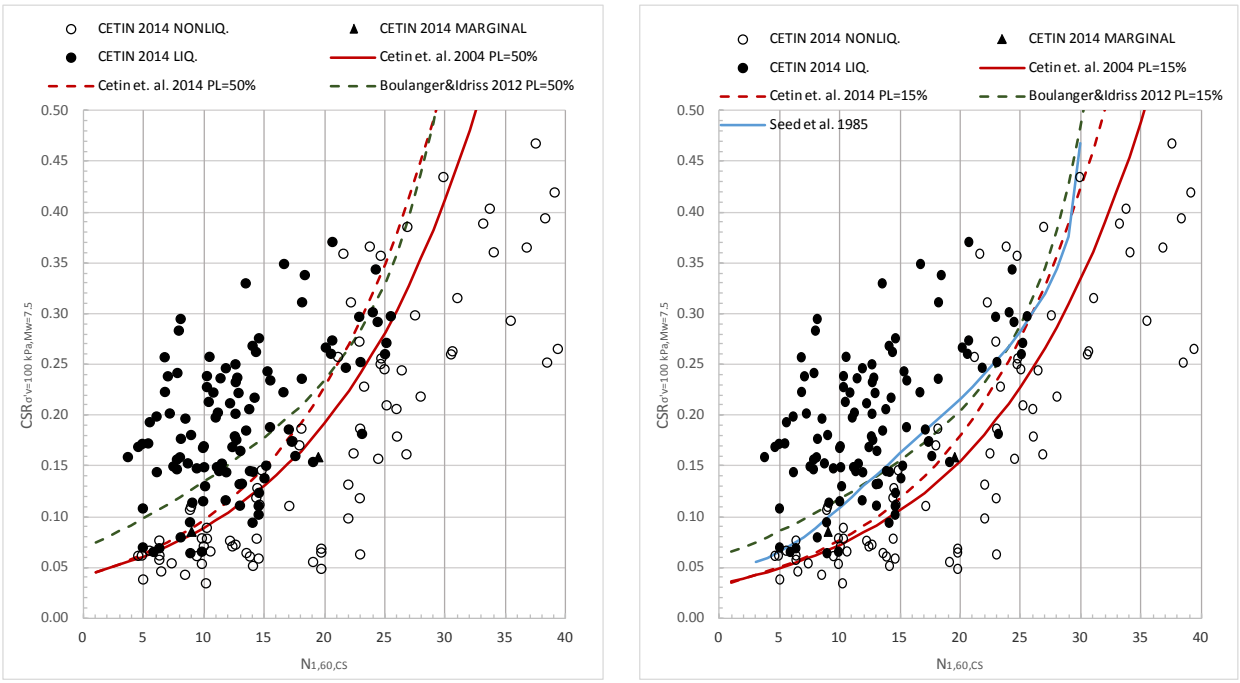


Figure 8b. Updated Liquefaction triggering curves, and comparisons with previous triggering curves proposed by Cetin et al. and by others (Alternative 2)

In these figures, the data points, and the boundary curves, are all adjusted (normalized) to a reference set of values corresponding to σ'_v=100 kPa, M_w = 7.5 and FC≤5%.

As shown clearly by these figures, the newly updated (draft) Cetin et al. (2014) curve for P_L = 15% (the recommended "deterministic" boundary curve), is located in a position between the original Cetin et al (2004) and the curve of Idriss and Boulanger (2008). These new curves have shifted to the left at the upper right hand border ($N_{1,60,CS} \geq 25$ blows/ft) and at the top is now very close to the boundary proposed by Professors Idriss and Boulanger. But at the lower "belly" portion of the curves ($N_{1,60,CS} \leq 25$ blows/ft) the triggering curves have not risen much, and the separation between these new triggering curves and the curves proposed by Idriss and Boulanger (2008) continues to be significant. As compared to Cetin et al. (2014) P_L=15 % liquefaction

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triggering CRR curve, the corresponding CRR values of Idriss and Boulanger (2012) are observed to be 60 to 80 % higher in the very low SPT blow count range (i.e.: $N_{1,60,CS} < 5$). These differences are reduced to roughly 50 % and 15 % at $N_{1,60,CS}$ values of 10 and 20 blows/30 cm, respectively.

As suggested by our recent analyses, it is our opinion that the "correct" liquefaction boundary lies between the two currently proposed relationships of Cetin et al. (2004) and Idriss and Boulanger (2008, 2012).

This suggests that the differences in the resulting liquefaction triggering boundary curves cannot be explained by the differences in the data processing details (of eight (actually seven) case history sites or more). Actually, for over 200 case histories, disagreements on only eight case history data processing details should be judged as a major agreement rather than disagreement that reflects the success of original data processing quality. In conclusion, we believe that in both quality and quantity, the liquefaction case history database has reached a sufficient level of maturity, that modifications of a limited number of case histories, due to different expert opinions, will not produce major shifts in liquefaction triggering boundary curves.

This brings us to the discussion of additional reasons for the differences between the CRR curves. As presented clearly during the GeoCongress-2014 liquefaction panel session, the reasons behind the disagreements lie in the details of how CSR and $N_{1,60,CS}$ terms are estimated and how they are normalized or corrected. Starting with the most important to least, these issues will be addressed next. Most of the plots and figures were adopted from our GeoCongress-2014 presentation, which was also provided electronically, as an attachment to this letter (Enclosure 3).

When processing case history data, cyclically-induced shear stresses (estimated by either simplified procedures with the use of r_d values, or directly by site response analyses) need to be normalized with vertical effective stress to estimate in-situ CSR value; which will be further

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corrected through K_σ, MSF and K_α effects to estimate the reference CSR value at 100 kPa vertical effective stress, M_w=7.5 and α=0 (i.e.: no shear stresses acting on the horizontal plane, valid for perfectly level sites) as given in Equation 1. From our point of view, the major differences between the Cetin et al. (2004 or 2014) and Idriss and Boulanger (2012) boundary curves are due to the execution of r_d, K_σ, and to a lesser extent MSF and fines correction. Fortunately, all of the case history sites are compiled from "almost" level sites, so K_α is a non-issue at the moment. However, it can be an important issue when extrapolating liquefaction triggering boundary curves to cover non-level sites.

$$CSR_{\sigma_v'=100,\alpha=0,M_w=7.5} = \frac{\tau_{av}}{\sigma_v'} = 0.65 \cdot \frac{a_{\max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \cdot \frac{1}{K_\sigma} \cdot \frac{1}{MSF} \cdot \frac{1}{K_\alpha} \qquad \text{Eqn (1).}$$

Before elaborating further, we need to define a "typical" potentially liquefiable site. On the basis of the weighted averages of the liquefaction case history input parameters from Cetin et al (2004), a typical site is defined as shown in Figure 9. As shown by this figure, the median values of the effective vertical stress and M_w in the Cetin et al. (2004) case history database are different than the reference values of 100 kPa and 7.5, and this makes the application of corrections to reach to the reference state even more critical.

Please note that the typical depth of a critical layer is about 5 to 6 m, or approximately 15 feet. As shown in Figure 10, at this depth the median r_d values estimated by Cetin and Seed (2004) are approximately 0.85, those of Seed and Idriss (1971) are 0.96, and those of Idriss (1999) are 0.95. Idriss and Boulanger (2010) adopted Idriss' (1999) r_d values. Hence, Idriss and Boulanger's estimated CSR values, just due to the differences in the adopted r_d values are on average about 12 % (=0.95/0.85) higher than those of Cetin et al. (2004).

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Figure 10 is illustrative of these sources of differences. It is illustrative only, because the left-hand figure (from Golesorkhi, 1989) is only one of several similar figures from that reference that would have to be averaged together to produce a representative value for median overall case history relative density and for M = 7.1.

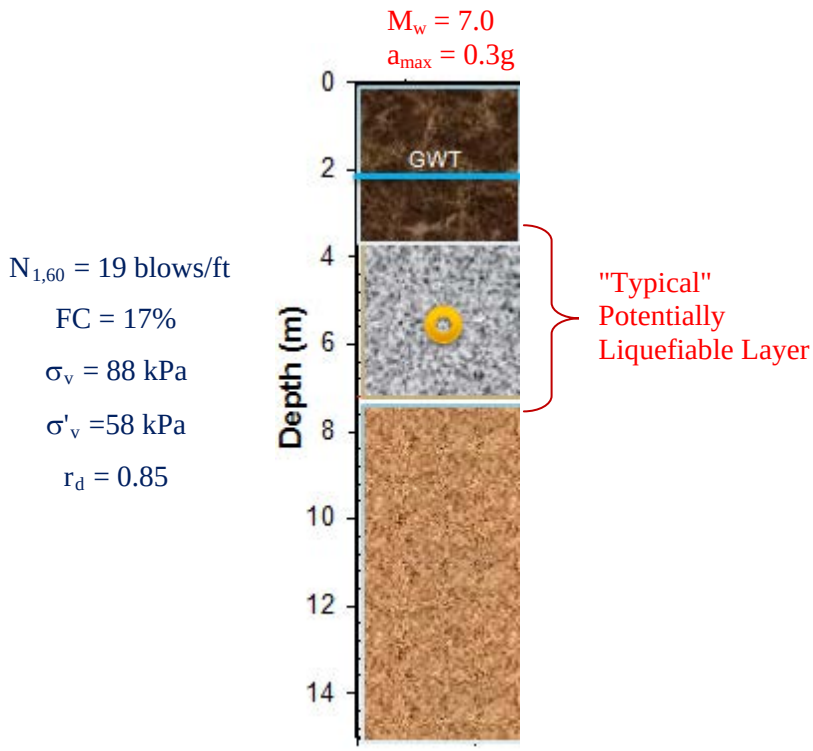
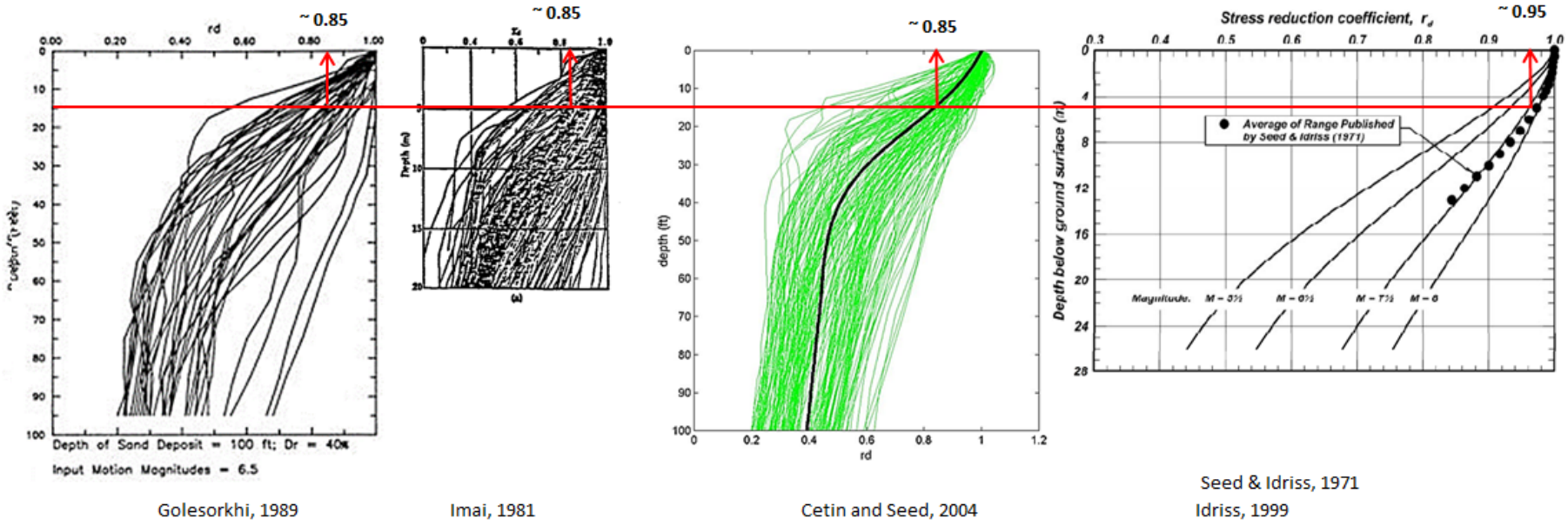


Figure 9. A "typical" potentially liquefiable site.

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Golesorkhi, 1989 Imai, 1981 Cetin and Seed, 2004 Seed & Idriss, 1971 Idriss, 1999

Figure 10. r_d values estimated at the approximate median critical depth of the databases.

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Figure 11 shows the full set of probabilistic contours for a recent regression of the dataset with modeling details very nearly the same as those used for the "interim" regression presented previously in Figure 8. As illustrated schematically by the arrows in Figure 11, a higher r_d value increases all of the estimated CSR values and forces the case histories to shift up, which in turn results in boundary curves shifted upwards (and to the left at any given value of CRR), leading to larger CRR values. It is difficult to precisely evaluate the quantitative effect this has on the CRR curves, because (a) the locations of individual data points (both liquefied and non-liquefied) are somewhat a matter of chance so that the overall "average" shift in CSRs (and r_d) is not necessarily precisely equal to the actual shift in the boundary curves, and (b) 43 of our case histories were back-analyzed by means of site-specific, and event-specific, seismic site response analyses, so that the CSR's calculated for those cases (representing slightly more than 20% of the overall data set) were even more "correct" and made no use of the simplified r_d curves. Overall, it appears that the average vertical shift due to use of non-representative r_d curves in the development of the relationship of Idriss and Boulanger (2008) is on the order of about 10% to 15%. Because Seed et al. (1985) r_d curves were also developed for site conditions not representative of the field case history data, and because their r_d curves were developed based on site conditions that were similarly overly "stiff and monolithic", their r_d curves also overestimated CSR and by approximately the same magnitudes.

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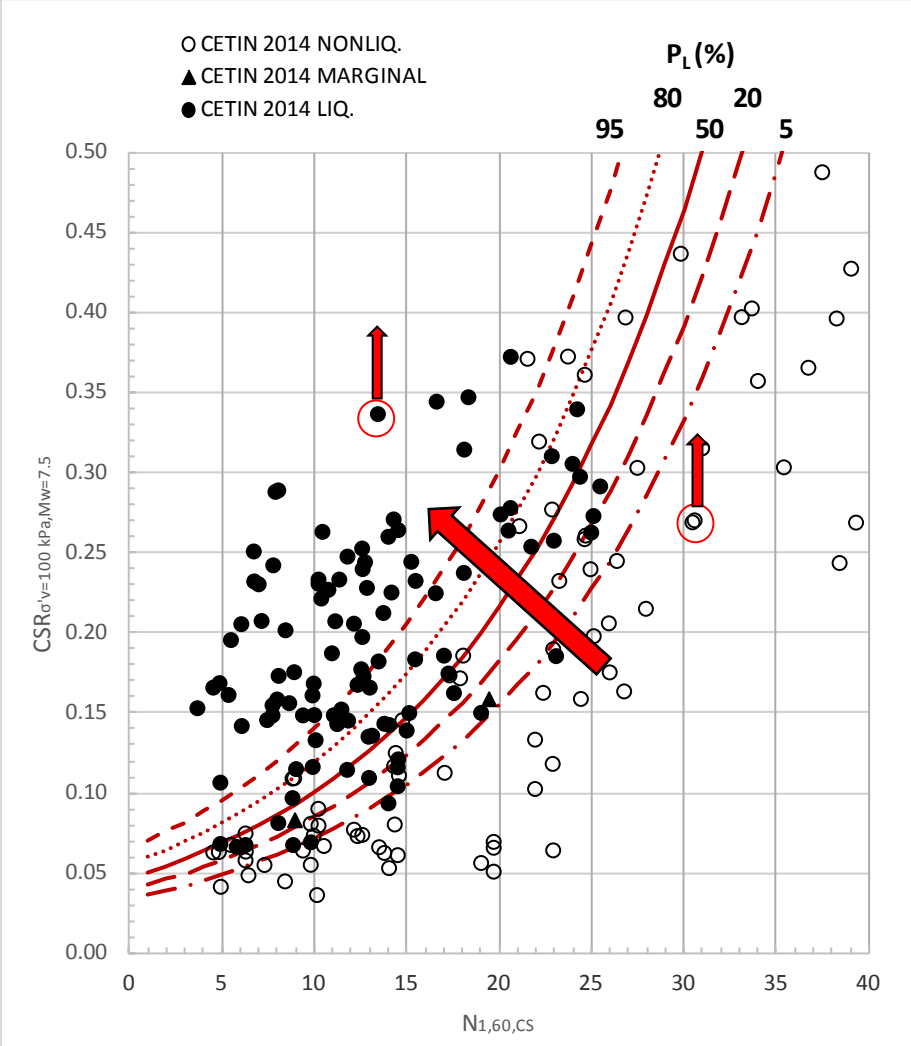


Figure 11. The effect of using higher r_d values on liquefaction triggering boundary curves

Similarly, Figure 12 presents the proposed K_σ corrections by Cetin et al (2004) and Idriss and Boulanger (2004 and 2008).

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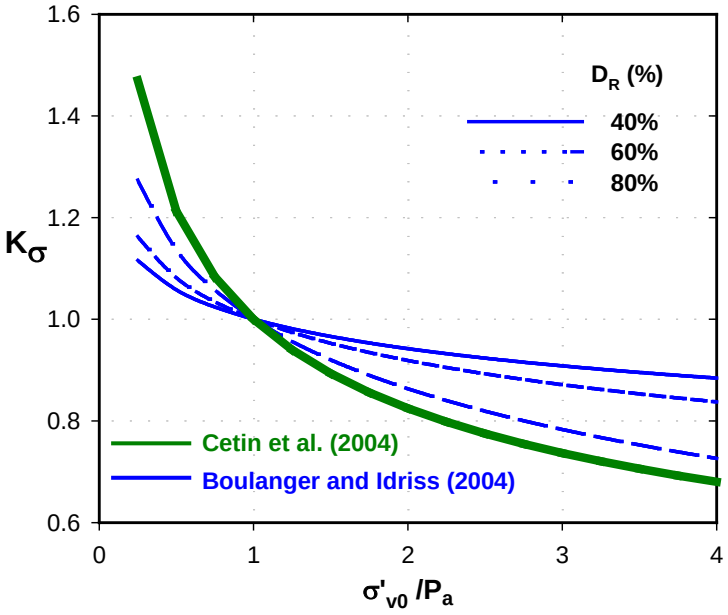


Figure 12. Proposed K_σ corrections

As documented below, Idriss and Boulanger (2006) originally preferred to use a K_σ upper cap of 1.0 for vertical effective stresses less than 100 kPa. It was also stated in the same reference that capping K_σ at a value of less than or equal to one "only affects a few of the case history data points".

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The K_σ factor is normally applied to the 'capacity' side of the analysis during design [Eq. (5)], but it must also be used to convert the site CSR to a common σ'_{vo} value for the empirical derivation of a CRR $-(N_1)_{60}$ curve. This is accomplished as:

$$(\mathrm{CSR})_{M=7.5,\,\sigma=1} = 0.65\left(\frac{\sigma_{vo}a_{max}}{\sigma'_{vo}}\right)\frac{r_d}{\mathrm{MSF}}\frac{1}{K_\sigma} \qquad (23)$$

such that the values of CSR correspond to an equivalent σ'_{vo} of 1 atm, and thus the liquefaction correlation also corresponds to an equivalent σ'_{vo} of 1 atm. Since K_σ has been restricted to ≤ 1 [Eq. (13)], this only affects a few of the case history points. Note that in applying the liquefaction correlation in design, the K_σ factor is still applied to the capacity side as indicated in Eq. (5).

We believe that this statement needs to be rephrased. As shown in Figure 13, the majority (~80 %) of case histories are compiled from sites with vertical effective stress of less than 100 kPa and executing a K_σ cap of 1.0 affects the CSR estimations for more than 150 of the case history sites. Later, the cap on K_σ was increased to 1.10, as stated in Idriss and Boulanger (2008). To begin to approximately quantify the effects of the different K_σ curves used, K_σ values were estimated for the typical liquefaction soil site, as shown in Figure 13. The median K_σ value recommended by Cetin (2004) is estimated as 1.15 as compared to 1.06 by Idriss and Boulanger (2004). Please note that K_σ corrections were executed within the likelihood function intrinsically as part of Cetin et al. studies. The difference in estimated K_σ values produce CSR values approximately 8.5 % higher than those estimated for Idriss and Boulanger (2010) database. This does not mean, again, that it causes exactly an 8.5% difference between the two correlations, because it again also makes a difference where individual data points are located and the amount by which they are affected. Note that a smaller K_σ value used in back analyses produces larger CSR values for case histories with vertical effective stress less than 1 atm., which in turn will lead to an upward shift in the liquefaction triggering boundary curves.

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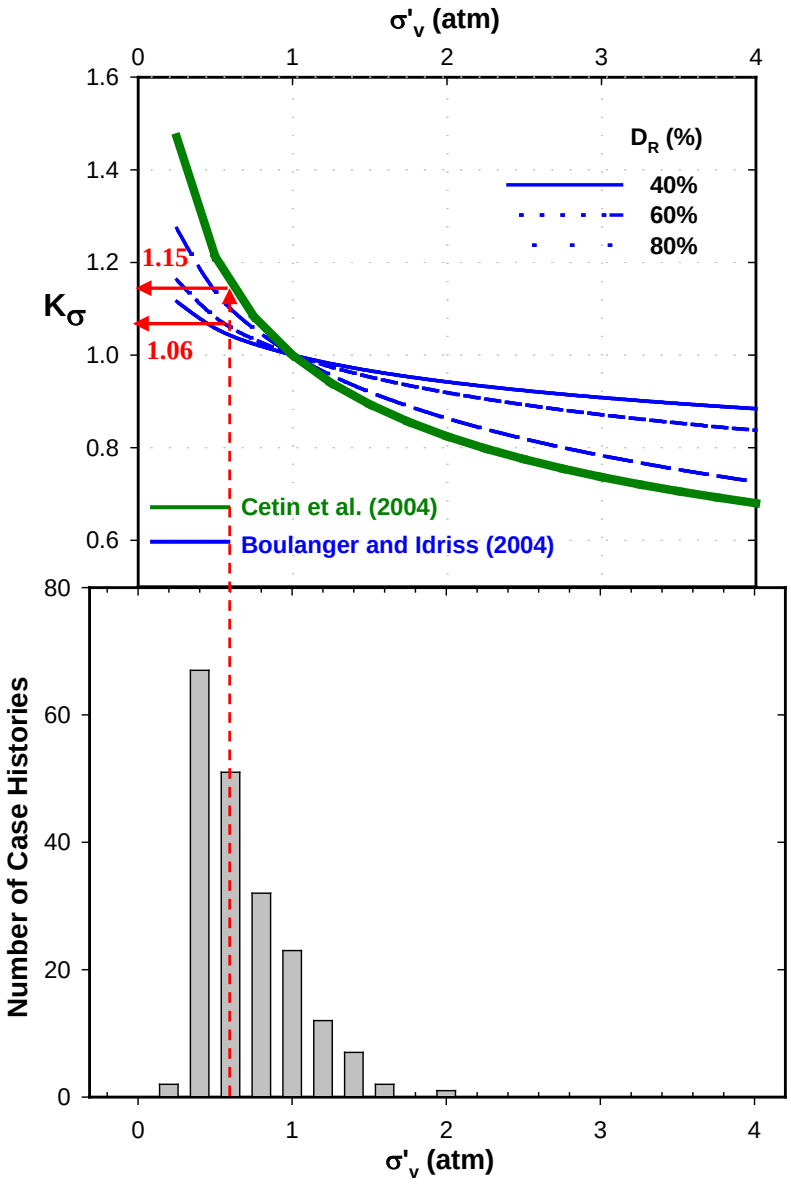


Figure 13. Histogram showing the variation of σ'_v at critical depths (Cetin et al database)

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Magnitude scaling is another correction, which may affect liquefaction triggering assessments especially for small magnitude cases. As shown Figure 14, there exist major differences in MSF recommendations by various research teams at very low magnitudes (e.g. M_w=5.5), and differences can also be significant at very high magnitudes (e.g. M_w=8.0 and greater). Fortunately, even though this difference can be quite significant for forward (design) assessments of sites subjected to very low or very high magnitude events, due to the fact that the median M_w of case histories is about 7.1, the average difference in the calculated MSF values by Cetin et al and Idriss (1999) is relatively small at this magnitude. Hence "average" differences in the application of MSF effects on case history processing is judged to be a relatively minor issue. For M_w =7, Cetin et al. estimates a MSF value of 1.165 as opposed to 1.14 by Idriss (1999). This only causes the estimated median CSR values to differ by 2% on the average. Differences can be significant, when using differing MSF factors for forward analyses and design problems for very small and very large magnitude events.

There is a large amount of new data available from recent earthquakes, and from laboratory investigations, and as these data are developed, this MSF issue can be expected to continue to evolve. Our own recent work, for example, suggests that MSF might be better modeled in a strain compatible manner, as will be briefly discussed later.

Overall, it must be concluded that MSF's are only a relatively minor cause of differences between the different relationships at this time.

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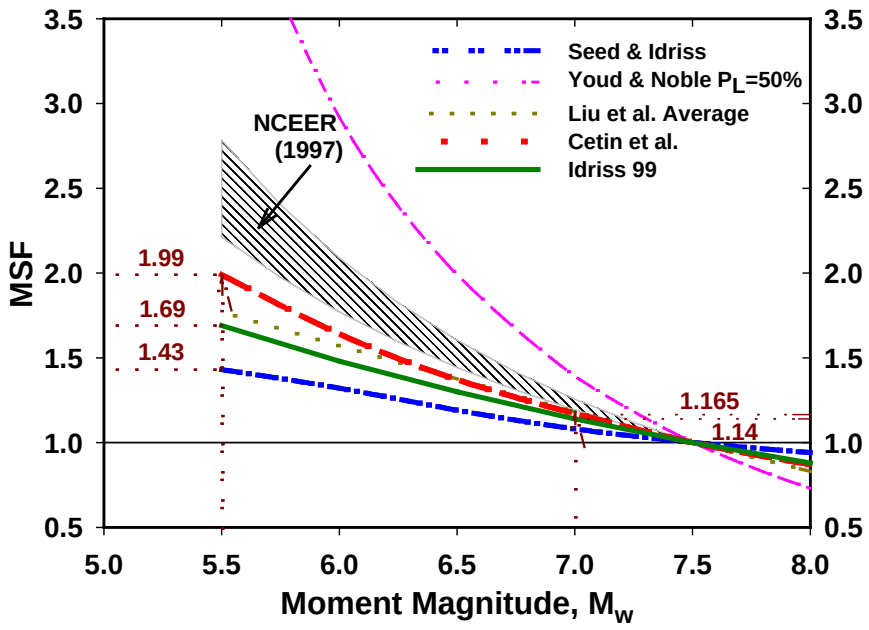


Figure 14. MSF as a function of M_w

As shown in Table 3, the multiplied differences in the representative r_d, K_σ and MSF values produce an average ratio of 1.25 (0.79/0.63) in the estimated CSR values by Idriss and Boulanger (2010) relative to those of Cetin et al. (2004), with the effects of truncation of K_σ neglected. Hence, due to differences in the adopted r_d values and K_σ and MSF correction schemes, Idriss and Boulanger curves are concluded to produce CRR values that will be, on average, more than 1.25 times higher than the Cetin et al. values. Similarly, and again initially neglecting the truncation of K_σ, the multiplied ratio for Seed et al. (1985) relative to Cetin et al. (2004) is approximately 1.39. Given the more severe "truncation" that was inadvertently (and implicitly) applied by Seed et al.(1985) it is concluded that the CSR values of Seed et al. will on average be greater than those of Cetin et al by a ratio of more than 1.4 based on these three factors.

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Looking next at Figure 8, it appears that the magnitudes of the differences between the vertical locations of the plotted data points, and the vertical locations of the three different triggering relationships, are generally compatible with these ranges.

The actual effects of these issues on the triggering boundary curves will, of course, be somewhat a function of "chance" as it will also depend on which data points were affected and to what degree, and on the locations of those points and their contributions (or impact) on the overall boundary curves. Nevertheless, it appears that a majority of the differences in the vertical locations of the boundary curves would likely be explained principally by r_d and K_σ differences, and that MSW is also a factor, but of less significance.

Table 3. A summary of average r_d, K_σ and MSF factors

$CSR_{\sigma_v'=100,\alpha=0,M_w=7.5} = \frac{\tau_{av}}{\sigma_v'} = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_v}{\sigma_v'} \cdot r_d \cdot \frac{1}{K_\sigma} \cdot \frac{1}{MSF}$				
Parameters		Seed et al. 1985	Idriss & Boulanger 2008	Cetin et al. 2004
d_{cr}=15feet	r_d	0.96	0.95	0.85
σ'_v=50-60kPa	K_σ	1.0	1.06	1.15
M_w=7.0	MSF	1.1	1.140	1.165
$r_d \cdot \frac{1}{K_\sigma} \cdot \frac{1}{MSF}$		0.87	0.79	0.63
$\frac{CRR_{Model}}{CRR_{CETIN}}$		1.39	1.25	1.00

After having discussed the differences in the factors concerning CSR (i.e: r_d, K_σ and MSF), next the potential differences on factors involving SPT-N values, will be discussed. Seed et al. (1985),

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Cetin et al. (2004) and Idriss and Boulanger (2008, 2010), "capped" C_N at some chosen maximum or limiting value as shown in Table 4.

Idriss and Boulanger (2010) adopted an upper cap of 1.7 on overburden correction term of C_N. The upper limit on C_N was selected as 2.0 by Seed et al, and Cetin et al., consistent with the recommendations of NCEER (1998). Table 4 presents a summary of C_N normalization values recommended by different research teams. The choice of "cap" values had only minor overall impact on differences between the three relationships, as the lower cap of $C_N \leq 1.7$ used by Idriss and Boulanger (2008, 2010) affects roughly 14 cases (the number was estimated by using Cetin et al database). The use of different C_N relationships by different research teams is more difficult to quantitatively assess, and like the other factors discussed it is a bit a matter of chance as to which data points are affected ,and to what degree, and what their locations (and impacts of the boundary curves developed) are. Nonetheless, it is noted that the approximate "median" C_N curves used by all three research teams were largely similar to each other, especially in the depth (and overburden stress) ranges where most of the case histories resided. And so we suggest that C_N likely has a relatively minor, but as yet unquantified, effect on the differences between the three methods.

It should be noted that in our next iteration (development in progress, targeted at late summer) we are implementing a scheme of C_N curves that vary with soil conditions, as do the curves of Idriss and Boulanger. But our curves will differ somewhat from theirs for soils with significant fines (e.g. SM and ML materials). It does not appear that this will have a major overall impact on the boundary curves relative to those shown as "draft" examples in Figure 8, but it must be expected that some differences may result.

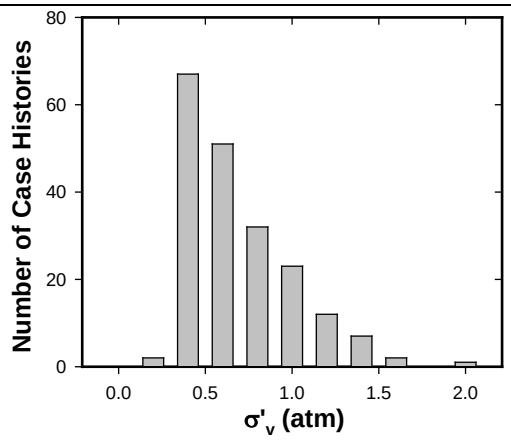
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Table 4. C_N normalization cap and its effects

C_N limit	Adopted By	Effect on Database	
≤2.0	Seed et al., 1985; NCEER, 1997; Robertson & Wride, 1998; Cetin et al., 2004	4 out of 200 data	
≤1.7	Moss et al., 2006; Idriss & Boulanger, 2008	14 out of 200 data	
Difference in $N_{1,60}$ by a factor of 1.06 in 10 cases			



Additionally, the fines corrections adopted by Idriss and Boulanger, Cetin et al, and Seed et al. were comparatively illustrated in Figure 15. For an $N_{1,60}$ value of 20 and FC=17 % (median of the liquefaction case history database), ΔN was estimated as 2.2 and 3.9 by Cetin et al, and Idriss and Boulanger relationships. The difference is about 7% in overall "median" fines corrected $N_{1,60,CS}$ values and Idriss and Boulanger clean sand correction produces slightly larger overall ΔN values. Also note that Cetin et al fines correction is relative density (or $N_{1,60}$) dependent and the denser the layer, the larger are the effects of fines on $N_{1,60}$. The Idriss and Boulanger(2008) fines correction is only a function of FC. Even though there exist differences in the fines corrections, considering the complexity of the effects of fines, such differences are judged to be relatively small, and understandable.

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$N_{1,60,CS} = N_{1,60} + \Delta N_{1,60}$

$N_{1,60}$= 10 and FC= 17 %

$\Delta N_{1,60}$= 3.6 Seed et al

= 1.5 Cetin et al.

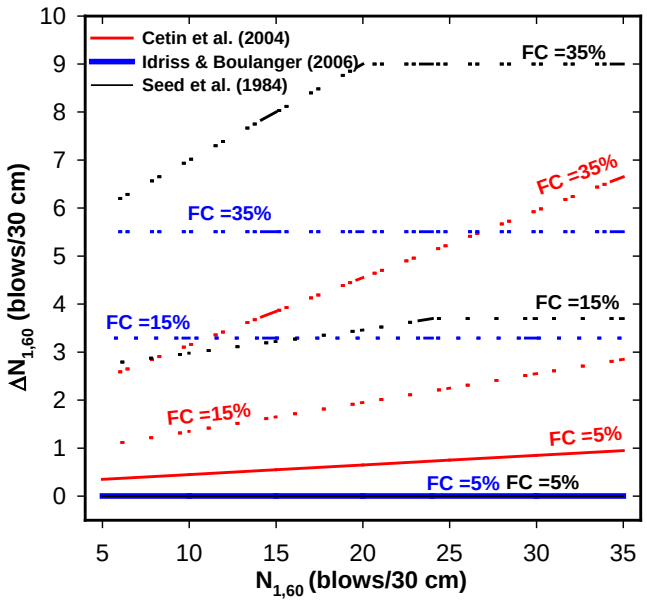
= 3.9 Idriss and Boulanger

$N_{1,60}$= 20 and FC= 35 %

$\Delta N_{1,60}$= 4.2 Seed et al

= 2.2 Cetin et al.

= 3.9 Idriss and Boulanger



For FC = 17%		
Seed et al.	$N_{1,60,CS} = 10 + 3.6 = 13.6$	$N_{1,60,CS} = 20 + 4.2 = 24.2$
Idriss and Boulanger	$N_{1,60,CS} = 10 + 3.9 = 13.9$	$N_{1,60,CS} = 20 + 3.9 = 23.9$
Cetin et al.	$N_{1,60,CS} = 10 + 1.5 = 11.5$	$N_{1,60,CS} = 20 + 2.2 = 22.2$

Figure 15. Fines correction

In conclusion, previously, we have shown that there exists a difference by factors of approximately 1.12, 1.09 (+) and 1.02 due to r_d, K_σ and MSF, expressed by the ratio of CSR values of Idriss and Boulanger (2010) and Cetin et al. (2004). These variations would produce a CRR ratio of greater than approximately 1.25 as clearly illustrated in Table 2, with the MSF factor being a relatively minor contributor to these differences, and the K_σ effects actually being significantly larger than 1.09 (and thus the "+") due to the additional issue of truncation of K_σ.

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Additional sources of differences between the three triggering correlations are due to differences in C_N relationships used, and due to different "caps" on maximum values of C_N. These move processed $N_{1,60,CS}$ values laterally, and so these move the boundary curves laterally. The overall systematic effects of these differences in approach appear likely to be relatively minor contributors to differences between the different relationships, but this is difficult to evaluate explicitly because (1) it is to some degree a function of "chance" with regard to the locations of individual data points, and their individual processing differences, and thus their impact on the overall triggering boundary curves, and (2) this is made even harder to differentiate due to the final sets of sources of differences addressed below.

The final set of sources of differences between the various relationships are: (1) the selection (and de-selection) of individual case histories to be included in the different databases, (2) engineering judgments made in back-analyses of the cases selected for inclusion (e.g. selecting the N-values that will be included as representing the "critical" stratum, etc.), and (3) the levels of conservatism with which the eventual overall triggering curves are then "set" once the data has all been processed. These issues can be very important, but it is difficult to rigorously assess the impacts of these types of differences on the overall correlations, partly due to difficulties associated with lack of documentation, and partly because in the end these are inter-dependent on the other issues discussed above.

From this conclusion, the question arises which boundary curve should be used for triggering assessments. We note that whichever method is preferred, engineers should be very careful not to mix-match the r_d, K_σ, MSF, C_N and fines corrections proposed by alternative methods. After having said that, it is necessary to emphasize that (1) the method which adopts all these normalizations/corrections in a manner most consistent with the current state of knowledge, is most likely to produce more reliable (i.e.: unbiased) liquefaction triggering boundary curves, and (2) that the different triggering relationships should be used in a manner compatible with the basis of their development.

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Question 2) Please explain your preference for using approximate equations for r_d versus site response analyses when (a) plotting case histories on your triggering plot and (b) performing a liquefaction triggering analysis.

It should be noted that we performed as many site-specific, and event-specific seismic site response analyses as possible for back analyses of the field case histories in order to directly calculate CSR's. Then, we went to great effort to develop r_d curves that would at least be suitably representative of the ranges of conditions in the field case history data set, so that the resulting r_d curves could at least be used to develop statistically unbiased estimates of CSR's (at least on the average) for the rest of the field cases, for which site- and event-specific response analyses could not be performed.

As a reminder, site response analyses were performed for 43 of the case histories, where one or more nearby (event-specific) ground motion recordings were available. The remaining cases were back-analyzed using r_d relationships. When developing our r_d relationships, fifty of the actual sites from the case history data set were modeled, and multiple input ground motions were analyzed for each of these sites. The empirical r_d model by Cetin and Seed (2004), was developed probabilistically (i.e. considers model uncertainty) and was based on the results of these 2,153 site response analyses performed for actual sites within the field case history database. The resulting r_d behavior is defined as a function of not only depth and earthquake magnitude, but also of site stiffness and intensity of shaking. As a result, the developed r_d curves are concluded to be specifically well-suited to represent the range of site conditions in the data set, and hence their functional form is slightly more complicated than the other two sets of r_d curves.

The principal motivation for proceeding as above was to develop the first triggering correlation that could be used, largely without bias, in conjunction with forward, project-specific engineering analyses performed either by using simplified r_d curves, or by using seismic response analyses. The resulting Cetin et al. correlations are believed to be compatible with direct project-specific

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seismic response analyses (either 1-D, or 2-D, and of varying levels of advancement and complexity). But that does not mean that it is the recommended first approach either for back-analyses of case histories, nor for forward analyses for all engineering projects. In conclusion, if necessary geotechnical and seismic input parameters are available, site response analyses by utilizing a large number of available acceleration time histories should be the preferred approach for both back-analyses and forward engineering assessments. If project challenges do not warrant site-specific response analyses, then the use of "simplified" procedures employing r_d curves is the next alternative. Here, too, the forward analyses performed should be compatible with the procedures used to generate the overall triggering relationship(s) being used.

Also equally importantly, we believe that if a positively biased r_d model (biased with larger values of r_d and also biased with larger values of CSR) was used in the development of the liquefaction triggering boundary curves (i.e.: back analyses), then engineers will tend to produce unconservatively-biased overall assessments of triggering hazard (statistically), if they then carefully perform site response analyses for a given project and thus estimate more "correct" but smaller values of CSR's, on the average. Unfortunately, because the site conditions analyzed to create the r_d curves used to generate the other two triggering relationships modeled sites that were, on average much stiffer and more monolithic than sites in the case history databases, the two r_d relationships used to create those triggering relationships produced back-calculated values of CSR's that were, on average biased high. In simpler terms, Seed et al. (1985) and Idriss and Boulanger (2008, 2010) r_d values are judged to be non-representative for the field case history database, and to produce higher r_d values than the values suggested by the results of site response analyses of Imai (1981), Golesorkhi (1989) and Cetin and Seed (2004), as shown earlier in Figure 10. Hence, pairing their CRR values (estimated by using relatively larger r_d values as compared with site response analyses results, given in Figure 16) with relatively smaller seismically-induced shear stresses estimated by site response analyses, may produce an unconservatively-biased conclusion. Cetin and Seed r_d values are shown to be consistent with Imai (1981) and

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Golesorkhi (1989) values (as given earlier in Figure 10) ,and on the average match well with site response analyses results.

The situation gets more complicated when the triggering correlations are used in the "simplified" manner (in conjunction with forward analyses based on simplified r_d curves). It is not necessarily true that using the same procedures going forward as were used going backward to create the triggering relationships, succeeds in all cases in creating fully offsetting (or mutually cancelling) errors.

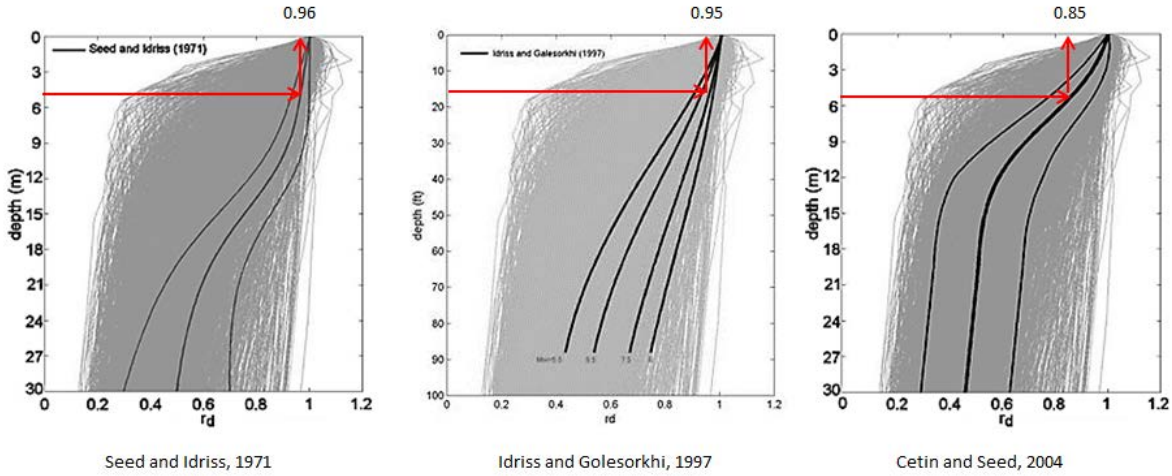


Figure 16.Comparison of site response analyses and r_d model predictions

3. Please share any unpublished work/thoughts related to the following topics:

(a) Calculation of earthquake-induced shear stresses (r_d issue)

We have already addressed all the important issues related to r_d in this letter, also in Cetin and Seed (2004). However, it is worthwhile to address a few additional items for the sake of completeness:

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1.) The presence of a liquefied layer above or below a non-liquefied critical layer makes it very difficult to assess the induced shear stresses. Hence, a non-liquefied case history data may not be adopted from a soil profile, which includes a liquefied (cyclically mobilized) soil layer.

2.) For consistency purposes, total stress based site response analyses should generally be preferred, if the intent is to use estimate induced shear stresses along with liquefaction triggering boundary curves. But suitably carefully performed effective stress-based analysis can also be used here. In our experience, engineers using effective stress based analytical models can run the risk of over-softening, and overdamping, and thus of underestimating cyclic shear stresses. This potential problem can be avoided by careful analyses, and careful review.

(b) Stress-normalization for penetration resistance (C_N issue)

We believe that C_N corrections should not be capped, at least for back-analyses, which is consistent both with the available field data and with the principles of critical state. To apply such a cap in back-analyses may unconservatively bias triggering relationships. For forward (design) assessments, engineers may prefer to be conservative, and can apply an upper cap on C_N.

Additionally, different sets of normalization schemes should be used for clean sands and sands with significant fines. Also it should be noted that stress normalization should be assessed differently for stability and deformation problems. Unfortunately, liquefaction case histories are often composed of both stability and deformation types of problems, and defining a stress normalization valid for both problems can be more difficult than it may appear.

For a detailed discussion on relative density and soil type dependent stress normalization scheme, please refer to Cetin and Isik (2007) (Enclosure 4).

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(c) Effect of fines content/soil type on penetration resistance and cyclic resistance.

The effects of fines on penetration resistance and cyclic resistance should ideally be separately assessed. Presence of fines, due to their so called "lubrication" effects, can reduce penetration resistance in cohesionless soils. Similarly, it is also known that fine grained soils can be somewhat more resistant to pore pressure buildup and cyclically induced strength and stiffness losses. These two effects can be jointly handled through correction factors jointly regressed and applied. We believe that presence of fines may be overly corrected in Seed et al. (1985) triggering relationships, especially at low N-values, and that a somewhat lower correction might be preferred. This observation is judged to be consistent with the recent literature. Case history based probabilistic assessments also support this judgment, and the results of some may even lead to extreme conclusion that the presence of non-plastic fines has almost no effect on CRR. Overall, the fines adjustments of the three relationships do not differ strongly at the median fines content value of the databases. It is important to use the fines corrections recommended with the individual relationship for which it was developed.

(d) Stress-correction for cyclic resistance (K_σ issue). Has high pressure laboratory testing, and possibly centrifuge testing, informed your thinking about the potential for liquefaction at high over-burden stress?

Our revised assessments have revealed that K_σ is one of the key contributors to the differences between Cetin et al. and Idriss and Boulanger models. It is noted, however, that “capping” or “truncation” of K_σ also contributes to these differences.

Please refer to Cetin and Bilge (2013, ICEGE-Istanbul 2013, Proceedings; Enclosure 5), for our interpretation of the influence of effective vertical stress on the cyclic resistance. As also

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presented in Figure 17, K_σ correction was shown to be shear strain level dependent. A shear strain level of 4 to 6 % matches well with Cetin et al. (2004 and 2014) K_σ correction factors. The use of different sets of K_σ corrections may be necessary for sites expected to experience different levels of cyclic deformation. We are currently in the process of developing a strain dependent combined K_σ, K_α SRR (stress reversal ratio) correction framework on the basis of large number of cyclic simple shear test results. The manuscript is expected to be released before the Fall of 2014.

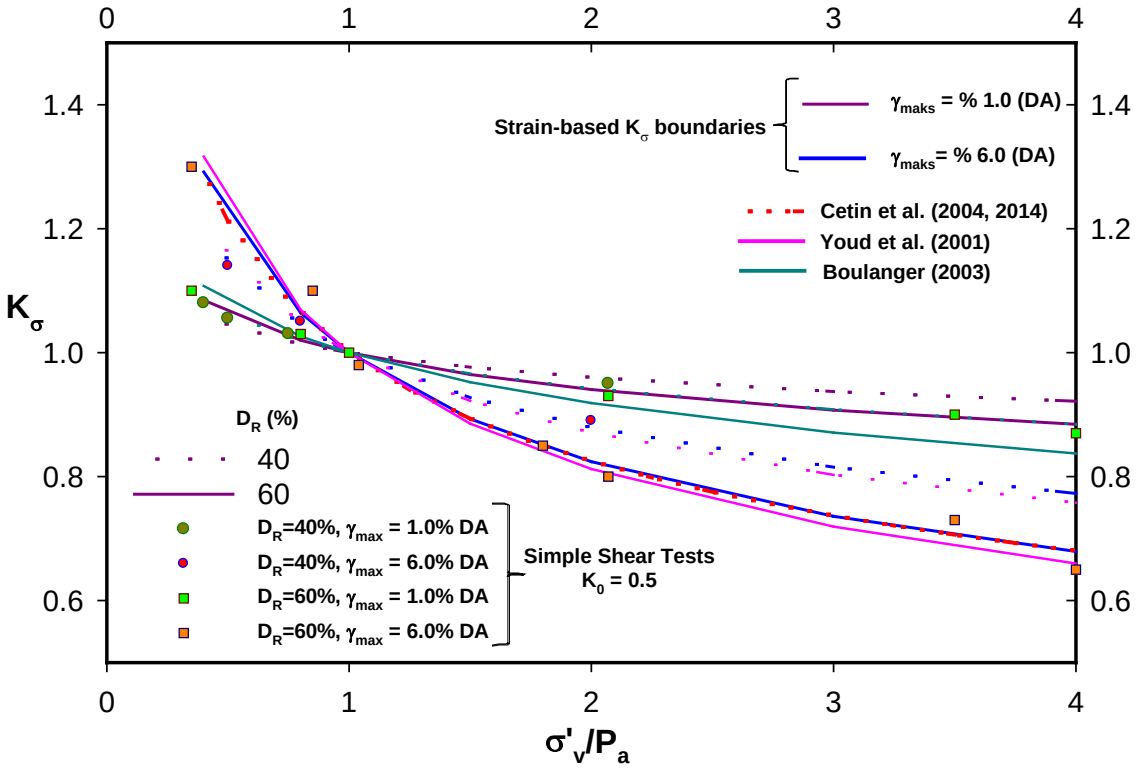


Figure 17. Shear strain-based K_σ factors

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(e) Magnitude scaling factors.

We have published a recent manuscript (Cetin and Bilge (2012a), Enclosure 6), where we have clearly addressed that magnitude (or duration) correction scheme should also be shear strain dependent. Figure 18 presents our proposed, strain dependent MSF values. This plot also partially explains the reasons behind such large scatter in the proposed MSF values developed by other research teams. Depending on the expected shear strain levels (or if the site is liquefied or not) a different set of MSF values should ideally be used.

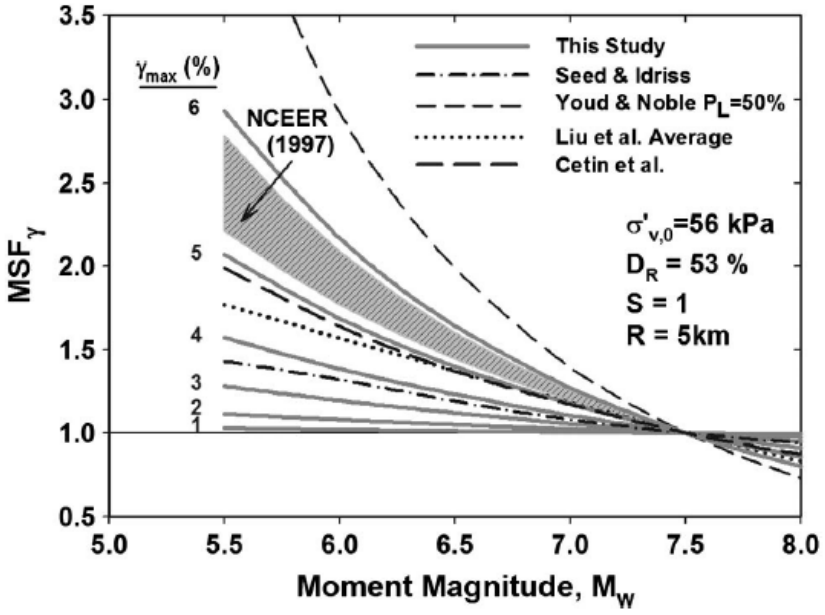


Figure 18. Strain-based MSFs

4. It is known that static shear stress influences the cyclic resistance ratio. However, consensus has not been achieved regarding its inclusion in the analysis of liquefaction triggering. Please share any work/thoughts on this topic beyond that available in the published literature.

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It should be noted that both Cetin et al.(2004) and Idriss and Boulanger (2010) developed their models based on free field case histories. It is known that the existence of static shear stresses may alter cyclic response dramatically. The contractive or dilative behavior depending on the stress and density state of soils defines the overall K_α response. Similar to K_σ and MSF corrections, we do believe that the selected performance level (i.e.: shear strain level) directly affects the K_α value. As a part of our on-going research, Cetin et al. have focused on the development of new unified CSR correction factors. Please refer to Cetin and Bilge (2013, Enclosure 5) for their preliminary results. As referred to earlier and shown in Figure 19, Cetin and Bilge (2013) propose a shear strain dependent, jointly executed, K_σ , K_α correction factors, designated as $K_{\sigma,\alpha}$on the basis of cyclic simple shear test results.

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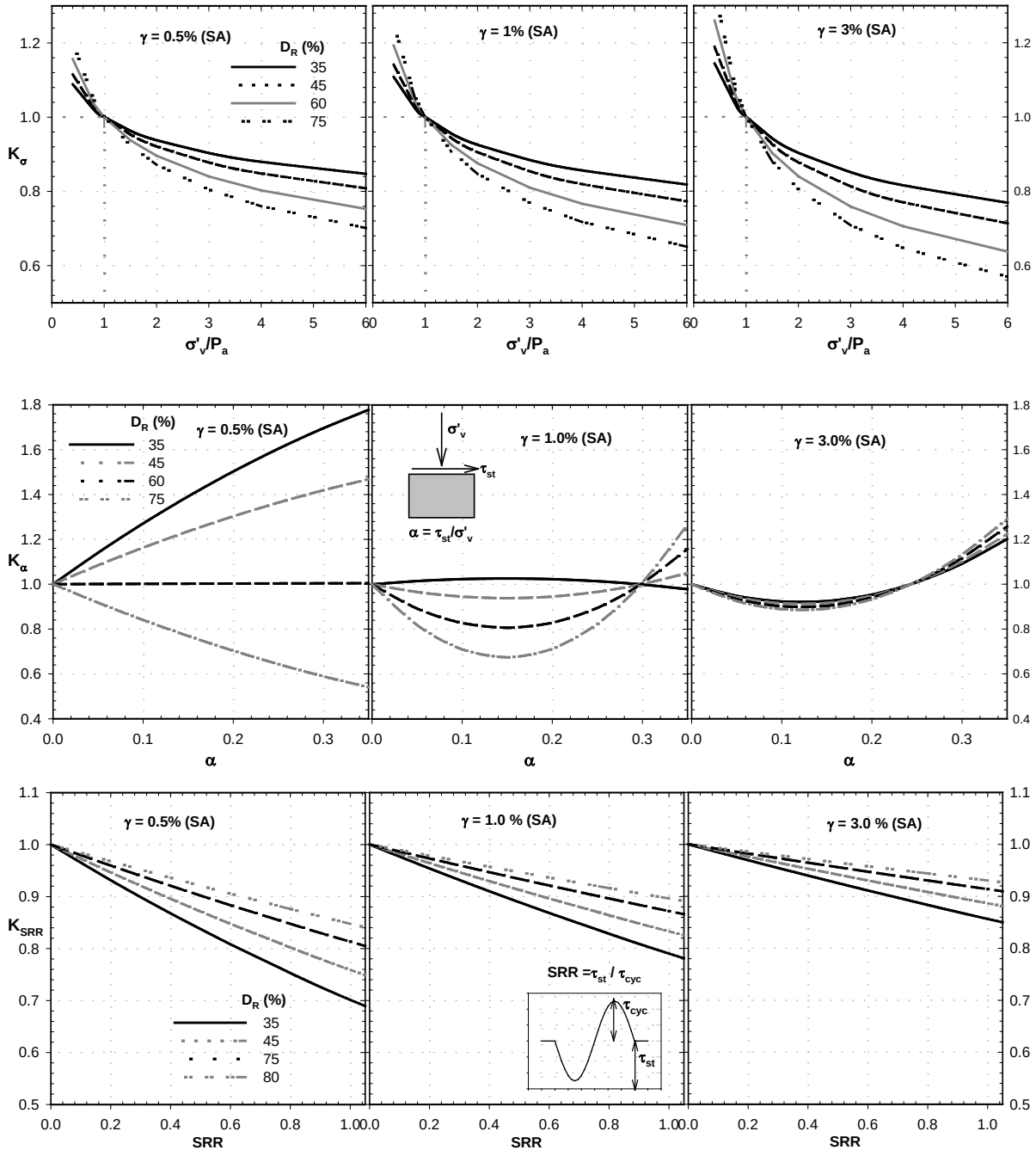


Figure 19. Proposed correction factors for different shear strain levels

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5. Please discuss your thoughts on triggering model development with respect to identification of critical layers from field data, particularly when multiple layers in the profile appear to be liquefiable

At first glance, identifying critical layers may seem to be difficult. However, it is our experience that for a majority of case history sites, it is not such a difficult task. There exist only a few cases in the large data sets currently assembled where it has been relatively difficult. More difficult, often, is the assessment as to whether or not a site "triggered" or did not liquefy, especially when the most potentially critical layer or stratum has relatively high $N_{1,60,CS}$ values (on the order of 25 to 35 blows/ft). In such cases shear strain potential is limited (limiting observations of ground deformation), and volumetric reconsolidation is also relatively small (limiting boiling and ejecta). Ejecta, on its own is not necessarily conclusive evidence of liquefaction triggering. Similarly, for soils with moderately high penetration resistances, the absence of ejecta is not conclusive evidence of the absence of triggering, especially if the most potentially susceptible stratum is not very near to the surface. In some of the most difficult cases, uncertainties can be high, and cases can be unsuitable for the databases as a result.

(b) Experimental/analytical constraints on parameters

We believe that applying strict constraints on input parameters may eliminate the chance of observing the true response of case history data. We strongly believe that in nature, there usually exist relatively smooth transitions rather than sharp constraints. One can benefit from both analytical and experimental findings; however constraining them in a stiff manner through a series of strict corrections may produce compliance effects. Laboratory testing, and laboratory-scale model testing (e.g. centrifuges, etc) all have challenges with regard to boundary conditions and scaling effects. Those types of tests and experiments can usefully inform us as to types of mechanisms, and in some cases can give us a sense of scale or magnitude. But it is in the field that nature sets the rules, and field data, when adequately available, and when adequately

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assessable, is to be preferred. That is why we generally prefer to update fines correction, K_σ, and MSF effects as part of the liquefaction triggering boundary curve assessment, as opposed to externally defining and executing them.

(c) Range of validity and approach to extrapolation outside the bounds of the data.

If unbiased K_σ, MSF, and fines correction schemes are used, then CRR values can be extrapolated outside the bounds of the data. However, if these correction schemes were biased, then extrapolating beyond the limits of the database may produce biased estimates of the true performance. Extrapolation needs to be done with great care, and it should usually be subject to expert peer review.

6. Liquefaction case histories are identified based on the manifestation of liquefaction—i.e., liquefaction is identified based on its observable consequences. It is possible that some of the non-liquefaction case histories liquefied without manifestation. Alternatively, liquefaction may be identified but the subsurface soils may not have fully liquefied. Should this issue be handled in liquefaction triggering models? If so, how?

Field case history based probabilistic liquefaction triggering relationships present to some extent the probability of having a surface manifestation of soil liquefaction in the form of sand boils, ground cracks and/or excessive settlements, etc. They do not necessarily estimate directly the probability of liquefaction triggering, as they are largely based on its observable manifestation at the ground surface. Naturally with such definition the question arises what if some sites liquefied without surface manifestation (i.e.: "blind" liquefaction cases), and what if we treat them as non-liquefied sites. Note that this is primarily a one way (biased) trend and that the opposite cannot be as commonly true (i.e.: the presence of sand boils accompanied by evidence of ground deformations ground cracks at non-liquefied sites). On the other hand, as discussed above, it is also possible for experienced field experts to disagree as to whether the observable manifestations

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are sufficient as to warrant classification of a site as “liquefied’, especially when the most critical soil strata present are of moderate or higher density and penetration resistance. This conclusion suggests that liquefaction triggering boundary curves developed based on field case history data may be partially unconservatively-biased.

We also note that the number of liquefied case histories are available far outnumber by the number of non-liquefied cases (i.e.: sampling bias) due to researchers unbalanced desire to investigate failures rather than non-failures. Hence these two opposite effects may be mutually compensating to some extent. Also note that probabilistic frameworks can involve model error terms, which can be adopted to take into account these sort of incidents and issues regarding the model imperfection (i.e. missing descriptive parameters with influence, and the possibility of a better functional forms, etc.).

In the end, field observational judgments, engineering analytical choices and judgments, and issues like sampling bias, etc. are simply among the many interesting challenges involved in this type of work.

7. Please share information, beyond that available in the published literature, regarding current research on residual strength, in particular:

The cyclically-induced pore pressure accumulation response as a function of induced shear strain levels are assessed as part of Cetin and Bilge (2012b) (Enclosure 7). These and similar studies could be used to assess the degradation of shear strength in the effective stress domain. However after initial liquefaction, we still need relationships to assess quantitatively the residual strength values. Currently, we are working on a probabilistically-based residual strength relationship. Unfortunately, our findings have not reached a level of maturity, which permit sharing at the moment.

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