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Liquefaction susceptibility and shear wave velocity

Susceptibilité de liquéfaction et la vitesse de l'onde de cisaillement

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ABSTRACT
The influence of age and soil type on Cyclic Resistance Ratio (*CRR*) and normalized small strain shear wave velocity has been examined here using cyclic laboratory tests on frozen and high-quality undisturbed specimens of Pleistocene and Holocene non-cohesive soils comprised of silt to pebble size particles and in-situ shear wave velocity measurements. These data indicate that the relationships between normalized shear wave velocity and the *CRR* for Holocene deposits may not be remarkably different from the corresponding relationships for Pleistocene soils. This inference contradicts the general experience that Pleistocene deposits are remarkably more resistant to earthquake-related liquefaction. However, the relationship between the laboratory-based *CRR* and the normalized shear wave velocity was found to be consistent with observations of field-performance of non cohesive deposits during earthquakes although there was a considerable uncertainty in this regard.

RÉSUMÉ
L'influence de l'âge et du type de terre sur la proportion de resistance cyclique (*CRR*) et normalisé petite tension vitesse de l'onde de cisaillement fut examiné ici en utilisant des epreuves cycliques laboratoires sur examples gelés et paisibles de terre non-cohesive Pleistocene et Holocene, comprise de vase et particules de galets de haute qualité et vitesse de l'onde de cisaillement. Cette information indique que les rapports entre vitesse de l'onde de cisaillement et le *CRR* pour l'arrhes de Holocene ne sont pas tellement differents des rapports correspondants pour Pleistocene terre. Cette inférence contredit l'expérience que l'arrhes Pleistocene est plus resistante à liquefaction reliée au tremblement de terre. Mais, le rapport entre *CRR* (du laboratoire) et la vitesse de l'onde de cisaillement normaliseé fut trouvé d'être compatible avec les observations de champs-performance du L'arrhes non cohésive pendant les tremblement de terre bien que il y ait une incertitude considérable dans ce domaine.

1 INTRODUCTION

Suceptibility of non-cohesive soils to liquefaction is usually assessed using empirical correlations between the critical values of an index of undrained cyclic soil strength that separates observed cases of occurrence and non-occurrence of liquefaction in past earthquakes and the Cyclic Stress Ratio (*CSR*). The following equation is commonly used for estimating the *CSR* for a level site:

$$CSR = 0.65 \times (a_{\max}/g) \times r_d \times (\sigma_v/\sigma_v') \times K_M \quad (1)$$

where $a_{\max}$ is the peak horizontal ground acceleration, g is the acceleration due to gravity, r_d is the stress reduction coefficient that approximately accounts for the flexibility of the soil column, σ_v and σ_v' are the total and effective vertical stresses within the layer in question before the earthquake, respectively, and K_M is the Magnitude Scaling Factor (MSF) that scales the *CSR*s from all earthquakes to a reference Magnitude of 7.5. The correlations between the index measurements and the *CSR* are usually developed by plotting the estimates of the *CSR* against the index measurements from earthquake-affected sites and identifying a threshold that reasonably separates the data from sites that liquefied from the data from sites where liquefaction did not occur.

The threshold identified in this manner represents the relationship between the index measurement and the Critical Stress Ratio. If the *CSR* for the design earthquake for a soil layer exceeds the Critical Stress Ratio, the layer is considered liquefiable. If, on the other hand, the *CSR* for the design earthquake is smaller than the Critical Stress Ratio, the site is assessed as non-liquefiable.

Normalized values of the Standard Penetration Test (SPT) blow count, Cone Penetration Test (CPT) tip resistance and shear wave velocity have been used as indices of undrained cyclic soil strength. The rationale behind their use has generally been that these measurements and liquefaction resistance are affected by relative density, compressibility and geologic age in a qualitatively similar manner (Tokimatsu, 1988).

For using Equation 1, $a_{\max}$ is usually obtained from the attenuation relationship for the earthquake under consideration, and the correction factors, r_d and K_M, are obtained using published relationships (*e.g.*, Youd *et al.*, 2001). Such estimates are not precise. Consequently, the *CSR* values obtained from Equation 1 are of limited precision.

A more precise approach for assessing liquefaction susceptibility involves development of a relationship between an appropriate index measurement and the cyclic soil strength obtained from cyclic undrained simple shear (or triaxial) tests conducted in the laboratory on undisturbed soil samples (Ishihara 1985). Such an approach has been adopted in this study. The cyclic soil strength determined in the laboratory is usually referred to as the Cyclic Resistance Ratio (*CRR*), which is identical to the *CSR* for barely liquefiable deposits.

Out of the indices of liquefaction resistance listed above, the normalized shear wave velocity, V_{s1}, is of particular interest because in many sites it is difficult to undertake any in-situ testing other than non-intrusive shear wave velocity profiling. Although frameworks for assessing liquefaction potential from shear wave velocity measurements have been proposed by many investigators (see Youd *et al.*, 2001 for a review), conceptual difficulty arises from the fact that liquefaction phenomenon is a manifestation of the plastic behavior of soils whereas shear wave velocity represents the elastic behavior (Roy *et al.*, 1996).

The objective of this study is to assess the uncertainty involved in the use of normalized shear wave velocity in assessing liquefaction susceptibility. Towards this, the uncertainty in the laboratory-based *CRR* - V_{s1} relationships is first examined. How precisely they "predict" triggering and non-triggering of liquefaction is then considered based on field performance case-histories during past earthquakes.

2 DATABASE

A database comprised of laboratory and in-situ test data and field performance case histories has been used in this study. The database includes soils of with a wide range of grain sizes (from silt to pebble) and grain compressibility (from rounded river sands to compressible pumice sands). The geologic age of the Holocene soils of this database ranges from 500 years to about 10,000 years. The geologic age of the Pleistocene soils of this database ranges from about 10,000 years to about 100,000 years.

A part of the database comprises published undrained cyclic laboratory test data from testing of undisturbed (frozen) or high quality soil samples from 24 sites in Canada, Italy, Japan, Taiwan and the USA, and in-situ test data from near the sampling locations (Table 1).

The other portion of the database comprises published post-earthquake field observation records of whether or not liquefaction was triggered at a site, the estimates of the *CSR* and shear wave velocity measurements from these locations (Table 2). These data were obtained from 66 locations in China, Japan, Taiwan and the USA following 19 earthquakes.

3 DATA ANALYSIS

The following relationship is used in this study to normalize the shear wave velocity, V_s, measured in the field:

$$V_{s1} = V_s \times \left(\sigma'_v / P_a\right)^{-0.25} \text{ subjected to} \quad V_{s1} \le 1.3 \times V_s \qquad (2)$$

where P_a is the atmospheric pressure.

Further, a correction was applied to the normalized shear wave velocity to account for the fact that in gravel sites V_{s1} is usually larger than that in a sand site with a comparable liquefaction resistance (see, *e.g.*, Rollins *et al.* 1998). For the soils types considered in this study, the gravel correction essentially meant multiplication of the uncorrected V_{s1} by 0.8 irrespective of the geologic age of the deposit. Youd *et al.* (2001) recommendations were adhered to for estimating the values of *CSR* in past earthquakes.

4 *CRR* – V_{s1} RELATIONSHIP

*CRR*s from undrained laboratory triaxial and simple shear testing of undisturbed (frozen) samples have been plotted in Figure 1 against V_{s1} from near the sampling locations. It is apparent from these data that for soils with fines content (F.C) of up to 15 %, the *CRR* relates approximately to the normalized shear wave velocity. Additionally, it is of interest that Pleistocene and Holocene soils are characterized with similar values of V_{s1} exhibit similar *CRR* and liquefaction resistance. This observation appears to contradict the notion that Pleistocene soils are more resistant to liquefaction.

Table 1. Laboratory CRR and field V_{s1}

Site	F.C	V_{s1}	*CRR*
CEORKA Stn., PI, Kobe	≤ 5%	141.92	0.11
CEORKA Stn., PI, Kobe	≤ 5%	150.47	0.13
Chiba Gravel	*≤ 5%*	*253.22*	*0.97*
Gioia Tauro Harbor	≤ 5%	196.69	0.14
Kagawa Gravel	≤ 5%	248.92	0.40
KIDD # 2	≤ 5%	182.87	0.09
Massey Tunnel	≤ 5%	171.46	0.09
Nagoya diluvial sand	*≤ 5%*	*215.92*	*0.21*
Showa Bridge, Niigata Sand	≤ 5%	206.03	0.17
Ogishima Sand	≤ 5%	170.60	0.06
Ogishima Sand	≤ 5%	156.84	0.07
Ogishima Sand	≤ 5%	160.91	0.06
Ogishima Sand	≤ 5%	172.40	0.08
Ogishima Sand	≤ 5%	177.28	0.07
Packing Factory, P.I, Kobe	≤ 5%	171.53	0.16
Packing Factory, P.I, Kobe	≤ 5%	156.50	0.19
Packing Factory, P.I, Kobe	≤ 5%	173.46	0.19
CEORKA Stn., P.I., Kobe	≤ 15%	164.84	0.10
Edogawa	*≤ 15%*	*312.35*	*0.40*
Edogawa	*≤ 15%*	*374.34*	*0.57*
Highmont Dam	≤ 15%	128.40	0.14
Japanese diluvial sand: FS3	*≤ 15%*	*238.18*	*0.28*
LL Dam	≤ 15%	151.99	0.05
Natorigawa	≤ 15%	148.46	0.28
Shirasu Soil, Kagoshima	≤ 15%	138.87	0.25
Shirasu Soil, Kagoshima	≤ 15%	139.87	0.21
Syncrude J-pit	≤ 15%	130.08	0.08
Tokyo Gravel	≤ 15%	*313.69*	0.33
Tonegawa	≤ 15%	237.71	0.38
Tonegawa	≤ 15%	209.55	0.54
Tonegawa	≤ 15%	200.30	0.24
Savannah River	≤ 15%	*153.46*	0.15
Edogawa	≤ 35%	*304.62*	0.45
Edogawa	≤ 35%	*336.48*	0.36
Japanese diluvial sand: FS1	≤ 35%	*227.11*	0.32
Japanese diluvial sand: FS2	≤ 35%	*236.96*	0.35
Natorigawa	≤ 35%	201.79	0.34
Natorigawa	≤ 35%	192.57	0.29
Niigata Sand	≤ 35%	200.06	0.44
Yuanlin	≤ 35%	190.60	0.19
Natorigawa	> 35%	221.11	0.50

Note 1. Italicized fonts have been used to indicate Pleistocene deposits and all other data are for Holocene soils.

Table 2. Laboratory *CSR* and field V_{s1}

Site	F.C	V_{s1}	*CSR*
Wufeng Fu Tin Bridge	≤ 5%	151.34	0.56
Gordon Farm GDN001	≤ 5%	157.98	0.52
Gordon Farm GDN002	≤ 5%	205.11	0.37†
James Loop JSL007	≤ 5%	164.37	0.29
Keir Farm KER001	≤ 5%	168.17	0.28
Morris Farm MRS001	≤ 5%	177.26	0.39
Morris Farm MRS003	≤ 5%	182.49	0.36†
Whakatane Board Mill WBM001-2	≤ 5%	156.50	0.26†
Whakatane Hospital HSP001	≤ 5%	166.74	0.14†
Sapanca Hotel	≤ 5%	216.41	0.40
Jefferson 148	≤ 5%	161.41	0.13
Jefferson Ranch 32	*≤ 5%*	*199.92*	*0.12*
Leonardini Farm	≤ 5%	197.94	0.10
Miller Farm	≤ 5%	142.67	0.28
Port of Oakland: P007-3	≤ 5%	260.51	0.22
Treasure Isl.: Pier 1 Improved Area	≤ 5%	198.76	0.13†
Treasure island: UM-05	≤ 5%	184.90	0.12

Notes 1. See Note 1 of Table 1.
2. † indicates non-liquefaction. Other data: liquefaction

Table 2 Continued.

Site	F.C	V_{s1}	*CSR*
Treasure island: UM-06	≤ 5%	189.93	0.14
Akita Port Accelerograph Station	≤ 5%	173.72	0.22†
Akita Port: Ohama No. 3 Wharf	≤ 5%	156.94	0.20
Hachiro-Gata A	≤ 5%	157.14	0.17
Hachiro-Gata B	≤ 5%	143.08	0.16
Hachiro-Gata C	≤ 5%	148.02	0.12†
Pence Ranch	*≤ 15%*	*145.80*	*0.23*
Chang-Bin Ind. Park LW-C1	≤ 15%	150.42	0.12
Wufeng, Site C	≤ 15%	300.54	0.46
Yuanlin C-24	≤ 15%	177.30	0.16
Yuanlin YL2	≤ 15%	172.02	0.16
Yalova Harbor	≤ 15%	200.54	0.12
Bay Bridge Toll Plaza: SFOBB1	≤ 15%	154.08	0.17
Salinas River South	≤ 15%	150.36	0.19
Treasure island: UM-09	≤ 15%	160.29	0.11
Niigata Site F	≤ 15%	142.92	0.11†
Niigata: Shinano Estuary	≤ 15%	137.42	0.13
Wufeng CPT-7	≤ 35%	184.15	0.14†
Whiskey Springs	≤ 35%	*206.93*	0.48
Nantou CPT-02	≤ 35%	205.34	0.23
Nantou CPT-03	≤ 35%	167.95	0.21
Nantou CPT-15	≤ 35%	185.78	0.37
Wufeng CPT-7	≤ 35%	184.15	0.35
Wufeng, Site B, WAC-2	≤ 35%	204.36	0.58†
Yuanlin C-16	≤ 35%	128.97	0.14†
Awaroa Farm	≤ 35%	188.89	0.34
McKim Ranch A	≤ 35%	161.30	0.39
Adapazari Site B1	≤ 35%	386.75	0.31
Adapazari Site C2	≤ 35%	203.46	0.36
Soccar Field	≤ 35%	102.10	0.25
Treasure Isl.: Pier 1 Loosened Soil	≤ 35%	195.40	0.13
Heber Road, CA: Point Bar	≤ 35%	207.08	0.13†
Nantou CPT-07	> 35%	184.00	0.23
Nantou CPT-11	> 35%	136.20	0.28
Nantou NT1	> 35%	135.54	0.37
Wufeng CPT-10	> 35%	153.13	0.60
Wufeng CPT-8	> 35%	126.51	0.72†
Wufeng, Site B, WAC-6	> 35%	189.47	0.61
Yuanlin C-32	> 35%	160.48	0.18
Wildlife Array, Alamo River, CA	> 35%	167.93	0.10†
Construction Committee Building	> 35%	109.64	0.13
Kornbloom	> 35%	121.97	0.10†
Adapazari Site D2	> 35%	165.08	0.28
Adapazari Site G1	> 35%	153.48	0.44
Adapazari Site J2	> 35%	138.65	0.45
Degirmendere Nose	> 35%	238.52	0.28
Port of Richmond: POR-2	> 35%	181.85	0.12
Wayne Avenue, LA	> 35%	173.40	0.33
Radio Tower	> 35%	103.51	0.15†
Wildlife Array, Alamo River, CA	> 35%	167.93	0.17
Tienstsin Y21	> 35%	143.80	0.08
Tienstsin Y24	> 35%	225.33	0.11
Tienstsin Y29	> 35%	155.47	0.08
Kornbloom	> 35%	120.42	0.19

In order to develop a correlation between *CRR* and V_{s1} for soils with F.C ≤ 5 %, the *CRR*s are first transformed using:

$$CRR^* = \frac{-36.092 + 309.357 \times CRR^{0.472}}{0.077 + CRR^{0.472}} \quad (3)$$

and subsequently a straight line was fitted between *CRR** and V_{s1}, where *CRR** is the transformed cyclic resistance ratio. The *CRR* - V_{s1} relationship obtained in this manner is plotted on Figure 1. The r^2 value for this relationship was 0.53. The relationship is therefore not very precise.

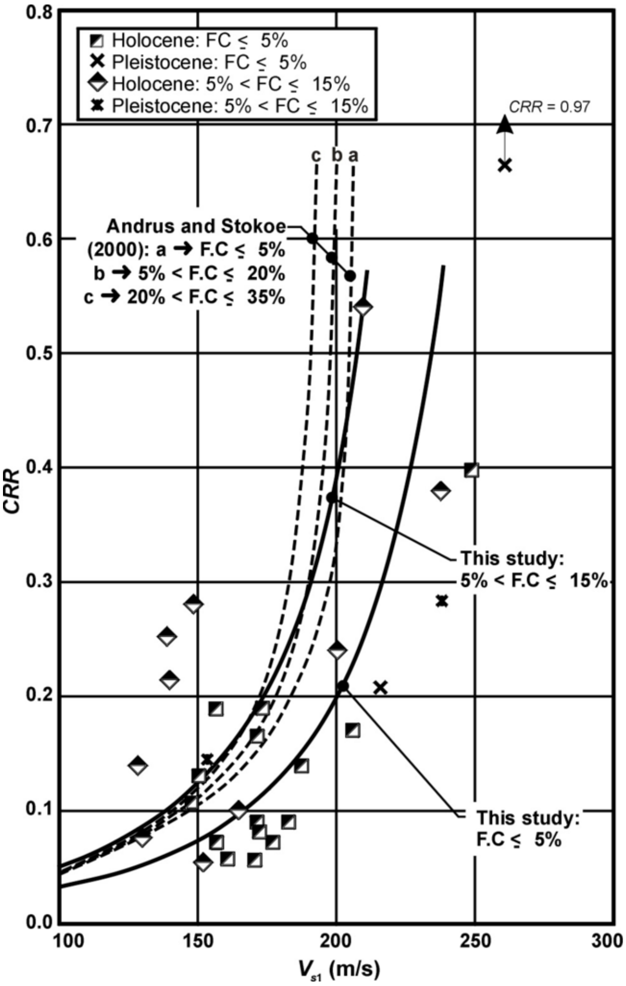

Figure 1. Laboratory-based *CRR*-V_{s1} relationships

A similar approach could not be used for soils with fines content of more than 5 % and up to 15 % because of a more significant scatter in the data. As a result, a best-fit correlation could not be developed between *CRR* and V_{s1} for these soils. An approximate *CRR* - V_{s1} relationship was developed for these soils by scaling the corresponding relationship for soils with F.C ≤ 5 %. This *CRR* - V_{s1} relationship is plotted on Figure 1 as well.

Considering that the *CRR* depends primarily on the plastic behavior of the soil, whereas V_{s1} represents the elastic behavior, nonexistence of a precise relationship between *CRR* and V_{s1} is not altogether unexpected.

Comparison of the Andrus and Stokoe (2000) relationships between *CSR* and V_{s1} with those identified above indicates that for soils with F.C ≤ 5 %, the *CRR*s based on laboratory testing of undisturbed soil samples are generally smaller than those predicted by Andrus and Stokoe (Figure 1). For soils containing fines of more than 5 % and up to 15 %, the Andrus and Stokoe (2000) correlation appears to be in agreement with the corresponding relationship developed in this study for V_{s1} < 180 m/s before becoming slightly unconservative for larger values of V_{s1}.

5 COMPARISON WITH FIELD CASE-HISTORIES

Estimates of *CSR* from 66 sites are plotted against the corresponding values of V_{s1} in Figure 2. Also presented on Figure 2 are the *CRR* - V_{s1} relationships developed in the preceding section and those developed by Andrus and Stokoe (2000).

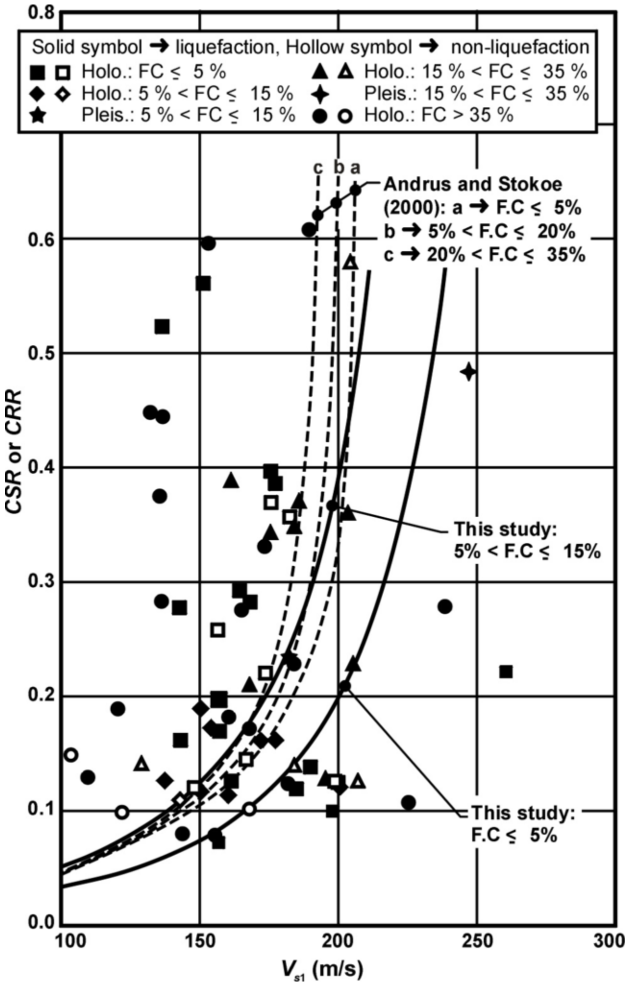

Figure 2. Performance of CRR-V_{s1} relationships

Examination of Figure 2 indicates that the CRR - V_{s1} relationships of this study approximately delineates the threshold for triggering of liquefaction albeit with a considerable uncertainty. For instance, for soils with F.C ≤ 5 % there are 12 cases of misclassification (*i.e.*, non-triggering of liquefaction when the data point representing a case history plots above the CRR - V_{s1} relationship or triggering of liquefaction when the data point plots below CRR - V_{s1} relationship) out of a total of 24 case histories. Such a frequency of misclassification is statistically significant at 95 % confidence level. Five instances of misclassification is also noticed for soils with fines contents of more than 5 % and up to 15 % out of a total of 10 case histories. Performance of Andrus and Stokoe (2000) relationships is statistically similar.

6 CONCLUSIONS

A database of undisturbed cyclic undrained laboratory test data and shear wave velocity measurements has been examined in this study in an effort to check whether the CRRs measured in the laboratory relate to normalized shear wave velocities with reasonable precision. The exercise resulted in approximate (imprecise) CRR - V_{s1} relationships.

As expected, the CRR - V_{s1} relationships were found to depend on the fines content of the soils. Although the laboratory-based CRR - V_{s1} relationships were found to be insensitive to the geologic age of the deposit, this inference should be considered tentative because only a small proportion of available laboratory test data pertains Pleistocene soils.

The approximate CRR - V_{s1} relationships identified in this study were found to differ from the corresponding Andrus and Stokoe (2000) relationships. Compared to Andrus and Stokoe, the CSR - V_{s1}, the relationships of this study are, in general, conservative.

The laboratory-based CRR - V_{s1} relationships were used to check whether they are in agreement with observed occurrences and non-occurrences of liquefaction during past earthquakes using 66 field performance records. This exercise led to approximately 50 % misclassification.

These results indicate that liquefaction potential can only be assessed approximately from normalized shear wave velocity. Lack of precision in this regard can be explained by the fact that the resistance of soils against liquefaction depends primarily on the plastic behavior of the soil whereas V_{s1} represents the elastic behavior. The procedures for assessing liquefaction potential from V_{s1} should therefore only be used cautiously and with adequate conservatism.

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