Liquefaction Resistance Using CPT and Field Case Histories

By Timothy D. Stark,[1] Member, ASCE, and Scott M. Olson,[2] Associate Member, ASCE

Abstract: Relationships between cone penetration tip resistance and the liquefaction potential of sandy soils are presented to facilitate use of the cone penetration test (CPT) in liquefaction assessments. The relationships are based on 180 liquefaction and nonliquefaction field case histories where CPTs were performed and illustrate the importance of median grain size and fines content on liquefaction resistance. The proposed CPT-based relationships were developed to describe the field case histories where CPT data are available, and eliminate the need to rely on conversions of standard penetration test (SPT) blow counts to CPT tip resistance used by existing CPT liquefaction-potential relationships. A new conversion between CPT tip resistance and SPT blow count is also proposed using the liquefaction-potential relationships developed from CPT data and existing liquefaction-potential relationships developed from SPT data. Finally, tentative CPT-based liquefaction-potential relationships are proposed for clean and silty gravel based on 18 liquefaction and nonliquefaction case histories.

INTRODUCTION

The cone penetration test (CPT) offers a number of advantages over the standard penetration test (SPT) for liquefaction assessments, including the following:

1. It is more economical to perform than the SPT, which allows a more comprehensive subsurface investigation.
2. The test procedure is simpler, more standardized and thus, more reproducible than the SPT.
3. It provides a continuous record of penetration resistance throughout a soil deposit, which provides a better description of soil variability and allows thin (greater than 15 cm in thickness) liquefiable sand or silt seams to be located. This is particularly important in sand and silts because of the natural nonuniformity of these deposits.

Based on these advantages, it is desirable to develop relationships between CPT tip resistance and liquefaction potential, rather than relying on a conversion from the SPT blow count to the CPT tip resistance to develop CPT liquefaction-resistance relationships.

The two main reasons why the CPT has not been used extensively for liquefaction assessment are: (1) The lack of a sample for soil classification and grain size analyses; and (2) limited amount of CPT-based field data pertaining to liquefaction potential was available. The number of field case histories with CPT data has increased significantly. This paper utilizes 180 liquefaction field case histories where CPT data are available to develop empirical liquefaction-potential relationships for sandy soils. In contrast, the liquefaction potential relationships published by Seed et al. (1985) are based on only 125 liquefaction and nonliquefaction case histories. The proposed CPT relationships are compared with existing CPT-based liquefaction-potential relationships and liquefaction field case histories where SPT blow counts are converted to CPT tip resistance to investigate agreement. Finally, the liquefaction potential of clean and silty gravel is estimated from 18 field case histories.

[1]Assoc. Prof., Dept. of Engrg., Univ. of Illinois, Urbana, IL 61801.

[2]Staff Engr., Woodward-Clyde Consultants, Maryland Heights, MO 63043; formerly, Grad. Res. Asst., Dept. of Civ. Engrg., Univ. of Illinois, Urbana, IL.

Note. Discussion open until May 1, 1996. To extend the closing date one month, a written request must be filed with the ASCE Manager of Journals. The manuscript for this paper was submitted for review and possible publication on April 14, 1994. This paper is part of the ***Journal of Geotechnical Engineering***, Vol. 121, No. 12, December, 1995. ©ASCE, ISSN 0733-9410/95/0012-0856-0869/$2.00 + $.25 per page. Paper No. 8255.

ESTIMATING LIQUEFACTION POTENTIAL

Seed et al. (1985) used equivalent cyclic stress ratio (CSR_{eq}) and SPT blow count (N) to develop a procedure for estimating the liquefaction potential of sandy soils. Since CSR_{eq} pertains to a certain number of equivalent laboratory loading cycles corresponding to an earthquake magnitude, it is proposed here to refer to the earthquake loading as the seismic shear-stress ratio (SSR). It is suggested that SSR is more descriptive of field earthquake loading than the equivalent cyclic stress ratio, because liquefaction potential is evaluated based on field-performance data and not on laboratory test results. As a result, the proposed relationships use SSR and CPT tip resistance to estimate the liquefaction potential of sandy soils.

Seed et al. (1985) and Seed and De Alba (1986) proposed boundary lines that separate field conditions causing liquefaction from conditions not causing liquefaction in sandy soils (Fig. 1) for an earthquake magnitude of 7.5. Because the undrained yield strength, s_u(yield), of the soil controls the triggering of liquefaction, Stark and Mesri (1992) concluded that the SSR corresponding to a boundary line in Fig. 1 is equal to the undrained yield-strength ratio of the soil mobilized in the field for a given corrected blow count. The mo-

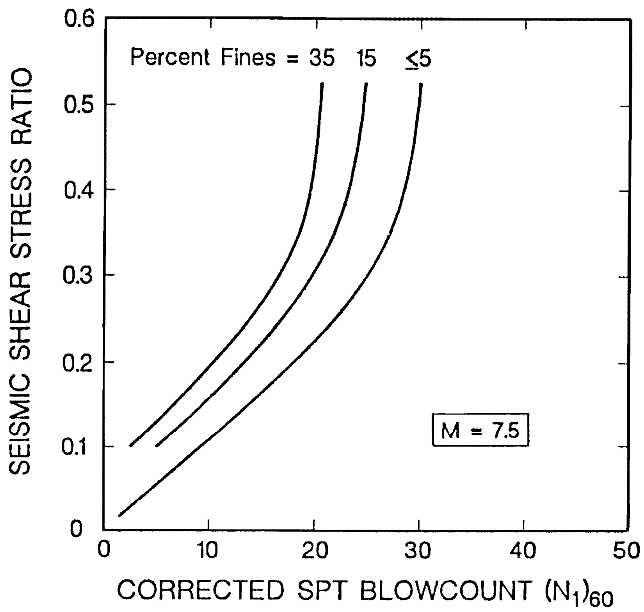

FIG. 1. Relationship between Seismic Shear-Stress Ratio Triggering Liquefaction and $(N_1)_{60}$-Values for Clean and Silty Sand and M = 7.5 Earthquakes [after Seed and De Alba (1986)]

bilized undrained yield-strength ratio is defined as s_u(yield, mob)/σ'_{v0}, where σ'_{v0} is the vertical effective overburden stress. The corrected blow count, $(N_1)_{60}$, is defined as the SPT blow count at a vertical effective overburden stress of 100 kPa and an energy level equal to 60% of the theoretical free-fall hammer energy to the drill stem.

Seed et al. (1985) proposed a standard blow count N_{60}, which corresponds to a transfer of approximately 60% of the theoretical free-fall hammer energy to the drill stem. The following equation was suggested by Seed et al. (1985) to correct various SPT energy ratios to an energy ratio of 60% for use in liquefaction analyses:

$$N_{60} = N \cdot (ER/60\%) \tag{1}$$

where ER = percent of the theoretical free-fall energy; and N = SPT N-value corresponding to the ER. The value of N_{60} is corrected to an effective overburden stress of 100 kPa, i.e., $(N_1)_{60}$, by multiplying N_{60} by the effective stress correction factor, C_N.

The factor of safety against liquefaction is estimated by dividing the undrained yield-strength ratio (Fig. 1) corresponding to the value of $(N_1)_{60}$ at any depth of a potentially liquefiable layer by the SSR generated by the design earthquake at the depth of interest. This factor of safety corresponds to an earthquake magnitude (M) of 7.5, an initial effective overburden stress less than or equal to 100 kPa, and level ground conditions. Seed and Harder (1990) present corrections to the undrained yield-strength ratio for earthquake magnitudes other than 7.5, initial effective overburden stresses other than 100 kPa, and sloping ground conditions.

The penetration resistance from the CPT, similar to a SPT N-value, is influenced by soil density, soil structure, cementation, aging, stress state, and stress history and, thus, can be used to estimate the undrained yield strength of soils (Robertson and Campanella 1985). However, unlike the SPT, a CPT sounding can yield a continuous factor of safety against liquefaction with depth for a potentially liquefiable soil. Further, SPT N-values must be corrected for effective overburden stress, hammer type and release system, sampler configuration, and drill rod length (Seed et al. 1985), while CPT data only needs to be corrected for effective overburden stress.

Correction for Vertical Effective Overburden Stress

Since most field observations of liquefaction have occurred at a vertical effective overburden stress between 50 and 120 kPa, CPT tip resistance values, q_c, and SPT N-values should be corrected to correspond with a vertical effective overburden stress of approximately 100 kPa. The corrected CPT tip resistance q_{c1}, is obtained using the following:

$$q_{c1} = C_q \cdot q_c \tag{2}$$

where C_q = effective overburden stress-correction factor.

Seed et al. (1983) developed an effective overburden stress correction for the CPT, and this correction was later confirmed by Mitchell and Tseng (1990) using cavity expansion theory to predict q_c and q_{c1} from laboratory tests on Monterey No. 0, Tincino, and Hokksund sands. Kayen et al. (1992) proposed the following equation to describe the effective overburden stress-correction factor proposed by Seed et al. (1983):

$$C_q = \frac{1.8}{0.8 + (\sigma'_{v0}/\sigma'_{ref})} \tag{3}$$

where σ'_{ref} = a reference stress equal to one atmosphere (approximately 100 kPa).

Despite the similarity in the shape of existing C_q and C_N (Seed et al. 1983; Liao and Whitman 1985) relationships, values of C_q are larger than C_N-values at vertical effective stresses less than 100 kPA, and slightly lower than C_N values at vertical effective stresses greater than 100 kPa. Therefore, the CPT data used here is corrected to 100 kPa using values of C_q estimated from (3).

Estimating Seismic Shear-Stress Ratio

The seismic shear-stress ratio for each case history was estimated using the simplified method proposed by Seed and Idriss (1971). Using this method, the seismic shear-stress ratio induced by the earthquake at any point in the ground is estimated as

$$SSR = 0.65 \cdot \frac{a_{max}}{g} \cdot \frac{\sigma_{v0}}{\sigma'_{v0}} \cdot r_d \tag{4}$$

where a_{max} = peak acceleration measured or estimated at the ground surface of the site; g = acceleration of gravity (9.81 m/s^2); σ_{v0} = vertical total overburden stress; and r_d = depth reduction factor. The depth reduction factor can be estimated in the upper 10 m of soil as

$$r_d = 1 - (0.012 \cdot z) \tag{5}$$

where z = depth in meters (Kayen et al. 1992). The value of the SSR was then corrected to an earthquake magnitude of 7.5, using the magnitude correction C_m proposed by Seed et al. (1985).

CPT-BASED CASE HISTORIES TO ESTIMATE LIQUEFACTION POTENTIAL

Table 1 presents a compilation of 180 liquefaction and nonliquefaction field case histories for sandy soils where CPT tip resistance data are available. The representative values of q_c for the case histories are the values reported by the investigator(s), or determined by averaging the tip resistance over the interval of sampling where the value of median grain diameter, D_{50}, and fines content were determined. Values of q_{c1} were than calculated as indicated in (2). The occurrence of liquefaction at a site was judged by the investigator(s) from the appearance of sand boils, settlement and/or damage of overlying structures, or lateral ground spreading. The nonoccurrence of liquefaction was assumed by the lack of the aforementioned liquefaction evidence.

Seed et al. (1985) and Seed and De Alba (1986) showed that fines content (percent by weight passing U.S. Standard Sieve No. 200) affects the relationship between SPT penetration resistance and liquefaction potential (Fig. 1). It was anticipated that fines content would have a similar effect on CPT penetration resistance and liquefaction potential. Since gradation and fines content both appear to influence CPT tip resistance, the correlations proposed here utilize both D_{50} and fines content (FC) to describe soil gradation. The CPT field data was divided into three categories based on D_{50} and fines content. The three categories are clean sand [$0.25 < D_{50}$ (mm) < 2.0 and FC (%) ≤ 5], silty sand [$0.10 \leq D_{50}$ (mm) ≤ 0.25 and $5 <$ FC (%) < 35], and silty sand to sandy silt [D_{50} (mm) < 0.10 and FC (%) ≥ 35]. Fines content refers to low to medium plasticity fines with a clay size fraction less than 15%, as suggested by Seed et al. (1983). Clay size fraction is defined as the percent by weight finer than 0.002 mm. Because the fines content is not available for some of the case histories, only median grain size was used to determine the appropriate soil category, e.g., clean sand, silty sand, or silty sand to sandy silt.

Liquefaction Potential of Clean Sand

Fig. 2 presents a compilation of 45 liquefaction and nonliquefaction field case histories involving clean sand [0.25 <

TABLE 1. Database of CPT-Based Liquefaction and Nonliquefaction Case Histories in Sandy Soils

Site (1)	Sounding (2)	Liquefaction observed? (3)	Depth (m) (4)	Ground-water depth (m) (5)	Vertical total stress (kPa) (6)	Vertical effective stress (kPa) (7)	Median grain diameter (mm) (8)	Fines content (%) (9)	CPT q_c (MPa) (10)	C_q (11)	q_{c1} (MPa) (12)	Site a_{max} (g) (13)	r_d (14)	Site seismic shear-stress ratio (15)	M=7.5 seismic shear-stress ratio (16)	Reference (17)
(a) 1964 Niigata Earthquake (M = 7.5)																
Kwagishi-Cho Building		Yes	2.8	1.1	52.0	35.3	0.33	0–5	3.14	1.57	5.02	0.16	0.97	0.15	0.15	Shibata and Teparaksa (1988)
		Yes	4.6	1.1	85.3	51.0	0.33	0–5	1.57	1.38	2.17	0.16	0.94	0.16	0.16	
		Yes	5.2	1.1	97.1	56.9	0.33	0–5	7.06	1.32	9.33	0.16	0.94	0.17	0.17	
		Yes	8.0	1.1	149.1	81.4	0.33	0–5	5.49	1.12	6.16	0.16	0.90	0.17	0.17	
Kwagishi-Cho Building		Yes	4.8	2.0	89.2	61.8	0.33	0–5	5.34	1.28	6.82	0.16	0.94	0.14	0.14	
		Yes	6.7	2.0	124.5	78.5	0.33	0–5	7.80	1.14	8.89	0.16	0.92	0.15	0.15	
		Yes	11.1	2.0	206.9	117.1	0.33	0–5	9.51	0.92	8.73	0.16	0.87	0.16	0.16	
South Bank		No	4.5	0.5	84.3	45.1	0.30	0–5	7.85	1.45	11.34	0.16	0.95	0.18	0.18	
		No	5.0	0.5	93.2	49.0	0.30	0–5	14.27	1.40	20.00	0.16	0.94	0.19	0.19	
(b) 1971 San Fernando Valley Earthquake (M = 6.4)																
Juvenile Hall, California	2-B1	Yes	8.5	8.4	167.6	166.1	0.058	62	6.37	0.74	4.70	0.50	0.90	0.29	0.25	Bennett (1989)
	2-B1	Yes	10.2	8.4	200.5	182.6	0.073	50	6.86	0.69	4.75	0.50	0.88	0.31	0.26	
	2-C	No	13.3	8.4	260.3	212.5	0.400	18	11.77	0.62	7.31	0.50	0.84	0.33	0.28	
	2-C	No	13.9	8.4	272.3	218.5	0.068	52	19.32	0.61	11.76	0.50	0.83	0.34	0.28	
	2-C	No	14.8	8.4	290.3	227.5	0.044	68	21.57	0.59	12.75	0.50	0.82	0.34	0.28	
	4-B1	Yes	6.4	5.8	125.7	119.7	0.052	64	3.14	0.91	2.85	0.50	0.92	0.32	0.26	
	4-B2	Yes	8.4	5.8	164.0	138.9	0.045	71	0.69	0.83	0.57	0.50	0.90	0.35	0.29	
	4-C	No	9.9	5.8	194.5	154.2	0.070	49	1.77	0.78	1.37	0.50	0.88	0.36	0.30	
	4-C	No	10.7	5.8	209.5	161.7	0.160	38	9.81	0.75	7.37	0.50	0.87	0.37	0.31	
	4-C	No	11.6	5.8	227.4	170.7	0.053	49	8.73	0.72	6.32	0.50	0.86	0.37	0.31	
	4-C	No	12.8	5.8	251.4	182.7	0.057	56	5.39	0.69	3.73	0.50	0.85	0.38	0.32	
	4-C	No	14.8	5.8	290.3	202.1	0.072	51	9.32	0.64	6.00	0.50	0.82	0.38	0.32	
	6-B1	Yes	4.6	4.3	89.8	86.8	0.042	74	0.69	1.09	0.75	0.50	0.95	0.32	0.26	
	6-C	No	9.1	4.3	179.6	131.7	0.050	61	7.06	0.86	6.05	0.50	0.89	0.39	0.33	
	6-C	No	10.7	4.3	209.5	146.7	0.095	46	10.79	0.80	8.64	0.50	0.87	0.40	0.34	
	6-C	No	11.3	4.3	221.4	152.7	0.069	52	13.73	0.78	10.71	0.50	0.86	0.41	0.34	
	6-C	No	13.9	4.3	272.3	178.2	0.060	56	8.83	0.70	6.21	0.50	0.83	0.41	0.34	
	6-C	No	15.1	4.3	296.3	190.2	0.082	47	6.86	0.67	4.61	0.50	0.82	0.41	0.35	
	10-B1	Yes	5.0	4.7	98.8	95.8	0.072	52	1.96	1.03	2.02	0.50	0.94	0.31	0.26	
	10-B1	Yes	5.8	4.7	113.7	103.3	0.055	65	0.69	0.99	0.68	0.50	0.93	0.33	0.28	
	10-B1	Yes	6.6	4.7	128.7	110.8	0.038	83	2.94	0.95	2.80	0.50	0.92	0.35	0.29	
	10-C	No	10.2	4.7	200.5	146.7	0.067	52	0.69	0.80	0.55	0.50	0.88	0.39	0.32	
	10-C	No	11.1	4.7	218.5	155.7	0.059	55	1.96	0.77	1.51	0.50	0.87	0.40	0.33	
	10-C	No	12.2	4.7	239.4	166.2	0.130	38	4.90	0.74	3.62	0.50	0.85	0.40	0.33	
	10-C	No	13.1	4.7	257.4	175.2	0.062	54	9.81	0.71	6.98	0.50	0.84	0.40	0.34	
	10-C	No	14.6	4.7	287.3	190.2	0.045	64	15.69	0.67	10.55	0.50	0.82	0.40	0.34	
	11-B1	Yes	6.3	5.9	122.7	119.7	0.051	61	1.96	0.91	1.78	0.50	0.93	0.31	0.26	
	11-B1	Yes	7.3	5.9	143.6	130.2	0.100	43	1.96	0.86	1.69	0.50	0.91	0.33	0.27	
	11-C	No	9.8	5.9	191.5	154.2	0.240	25	20.60	0.78	15.96	0.50	0.88	0.36	0.30	
(c) 1975 Haicheng Earthquake (M = 7.3) (Ying-Kou city)																
Paper mill site		Yes	4.0	1.5	74.6	50.0	0.07	72	0.65	1.43	0.93	0.15	0.95	0.14	0.14	Arulanandan et al. (1986)
Glass fiber site		Yes	3.0	1.5	55.9	41.2	0.08	42	0.53	1.53	0.81	0.15	0.96	0.13	0.13	
Construction building site		Yes	7.0	1.5	130.5	76.5	0.02	83	0.38	1.16	0.44	0.15	0.92	0.15	0.15	
Fishery and shipbuilding site		Yes	3.5	1.5	65.2	45.6	0.016	90	1.30	1.44	1.87	0.15	0.96	0.13	0.13	
Middle school site		No	10.3	1.5	191.0	105.2	0.016	92	0.73	0.98	0.71	0.15	0.88	0.16	0.15	
Chemical fiber site		Yes	7.5	1.5	139.8	80.9	0.035	61	1.20	1.13	1.35	0.15	0.91	0.15	0.15	
(d) 1976 Tangshan Earthquake (M = 7.8)																
Tanghsan area	T-10	Yes	3.0	1.5	55.9	41.2	0.06	—[a]	1.67	1.49	2.49	0.40	0.96	0.34	0.35	Shibata and Teparaksa (1988)
		Yes	6.0	1.5	111.8	67.7	0.25	—[a]	9.22	1.23	11.30	0.40	0.93	0.40	0.41	
		Yes	7.8	1.5	145.1	83.4	0.25	—[a]	5.59	1.11	6.20	0.40	0.91	0.41	0.42	
		Yes	8.5	1.5	158.9	90.2	0.30	—[a]	7.45	1.06	7.94	0.40	0.90	0.41	0.43	
	T-11	Yes	0.9	0.9	16.7	16.7	0.17	—[a]	1.47	1.87	2.75	0.40	0.99	0.26	0.27	
		Yes	1.3	0.9	24.5	20.6	0.17	—[a]	0.98	1.79	1.76	0.40	0.98	0.30	0.32	
		Yes	1.8	0.9	33.3	24.5	0.17	—[a]	4.90	1.73	8.47	0.40	0.98	0.35	0.36	

TABLE 1. (*Continued*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	T-12	Yes	2.0	1.6	37.3	33.3	0.14	—[a]	2.45	1.59	3.91	0.40	0.98	0.28	0.29	
		Yes	3.0	1.6	55.9	42.2	0.14	—[a]	2.55	1.48	3.77	0.40	0.96	0.33	0.34	
		Yes	4.0	1.6	74.5	51.0	0.16	—[a]	3.14	1.38	4.33	0.40	0.95	0.36	0.37	
		Yes	4.7	1.6	87.3	56.9	0.16	—[a]	5.69	1.32	7.52	0.40	0.94	0.38	0.39	
		Yes	6.4	1.6	119.6	72.6	0.16	—[a]	3.43	1.19	4.07	0.40	0.92	0.40	0.41	
		Yes	9.5	1.6	177.5	100.0	0.16	—[a]	8.24	1.01	8.30	0.40	0.89	0.41	0.42	
	T-13	Yes	2.0	1.1	37.3	28.4	0.12	—[a]	1.67	1.67	2.78	0.40	0.98	0.33	0.34	
		Yes	2.1	1.1	39.2	28.4	0.12	—[a]	3.43	1.67	5.72	0.40	0.97	0.35	0.36	
		Yes	2.7	1.1	50.0	34.3	0.12	—[a]	4.02	1.58	6.36	0.40	0.97	0.37	0.38	
	T-14	Yes	1.5	1.3	28.4	26.5	0.17	—[a]	5.39	1.70	9.15	0.40	0.98	0.27	0.28	
		Yes	3.0	1.3	55.9	39.2	0.32	—[a]	8.83	1.52	13.38	0.40	0.96	0.36	0.37	
	T-15	Yes	1.2	1.0	22.6	20.6	0.48	—[a]	6.86	1.79	12.32	0.40	0.99	0.28	0.29	
		Yes	1.8	1.0	33.3	25.5	0.48	—[a]	1.16	1.71	1.98	0.40	0.98	0.33	0.34	
		Yes	2.5	1.0	47.1	32.4	0.48	—[a]	4.16	1.61	6.69	0.40	0.97	0.37	0.38	
	T-16	No	4.0	3.5	74.5	69.6	0.16	—[a]	11.25	1.21	13.61	0.40	0.95	0.26	0.27	
		No	8.4	3.5	156.9	108.9	0.20	—[a]	15.46	0.96	14.84	0.40	0.90	0.34	0.35	
	T-17	No	3.1	2.8	57.9	54.9	0.21	—[a]	11.17	1.34	14.98	0.20	0.96	0.13	0.14	
		No	4.1	2.8	76.5	63.7	0.21	—[a]	11.89	1.26	14.97	0.20	0.95	0.15	0.15	
		No	5.2	2.8	97.1	73.5	0.14	—[a]	17.42	1.18	20.54	0.20	0.94	0.16	0.17	
	T-18	Yes	4.7	3.6	87.3	76.5	0.17	—[a]	1.62	1.16	1.87	0.20	0.94	0.14	0.14	
		Yes	5.2	3.6	97.1	81.4	0.17	—[a]	3.58	1.12	4.02	0.20	0.94	0.15	0.15	
	T-19	Yes	1.5	1.1	28.4	24.5	0.19	—[a]	1.01	1.73	1.74	0.20	0.98	0.15	0.15	
		Yes	2.9	1.1	53.9	36.3	0.31	—[a]	4.90	1.55	7.62	0.20	0.97	0.19	0.19	
		Yes	4.0	1.1	74.5	46.1	0.18	—[a]	2.85	1.43	4.09	0.20	0.95	0.20	0.21	
		Yes	5.5	1.1	103.0	59.8	0.18	—[a]	5.94	1.29	7.69	0.20	0.93	0.21	0.22	
	T-20	No	1.2	1.1	22.6	21.6	0.17	—[a]	12.98	1.78	23.07	0.20	0.99	0.13	0.14	
		No	1.7	1.1	31.4	25.5	0.17	—[a]	12.81	1.71	21.92	0.20	0.98	0.16	0.16	
		No	2.1	1.1	39.2	29.4	0.17	—[a]	16.27	1.65	26.86	0.20	0.97	0.17	0.17	
	T-21	No	3.1	3.1	57.9	57.9	0.26	—[a]	10.39	1.31	13.63	0.20	0.96	0.13	0.13	
		No	3.3	3.1	61.8	59.8	0.26	—[a]	8.94	1.29	11.58	0.20	0.96	0.13	0.13	
		No	4.0	3.1	74.5	65.7	0.26	—[a]	11.07	1.24	13.76	0.20	0.95	0.14	0.15	
	T-22	Yes	3.7	0.8	68.6	40.2	0.16	—[a]	1.90	1.50	2.86	0.20	0.96	0.21	0.22	
		Yes	4.0	0.8	74.5	43.1	0.16	—[a]	4.90	1.47	7.20	0.20	0.95	0.21	0.22	
	T-23	Yes	3.7	1.4	68.6	46.1	0.14	—[a]	2.20	1.43	3.15	0.20	0.96	0.19	0.19	
		Yes	3.9	1.4	72.6	48.1	0.14	—[a]	2.60	1.41	3.67	0.20	0.95	0.19	0.19	
	T-24	Yes	2.8	1.0	52.0	34.3	0.16	—[a]	4.31	1.58	6.82	0.20	0.97	0.19	0.20	
		Yes	3.2	1.0	59.8	38.2	0.16	—[a]	2.94	1.53	4.50	0.20	0.96	0.20	0.20	
	T-25	Yes	8.2	0.7	153.0	79.4	0.08	—[a]	8.83	1.14	10.03	0.20	0.90	0.23	0.23	
	T-26	Yes	5.2	0.8	97.1	53.9	0.14	—[a]	1.96	1.35	2.65	0.10	0.94	0.11	0.11	
	T-27	Yes	5.0	0.7	93.2	51.0	0.07	—[a]	1.08	1.38	1.49	0.20	0.94	0.22	0.23	
	T-28	No	11.0	0.7	205.0	103.9	0.08	—[a]	15.20	0.99	14.98	0.10	0.87	0.11	0.12	
		No	11.4	0.7	212.8	107.9	0.08	—[a]	6.37	0.97	6.15	0.10	0.86	0.11	0.11	
	T-29	No	4.8	1.0	89.2	52.0	0.10	—[a]	8.83	1.37	12.10	0.10	0.94	0.11	0.11	
		No	5.3	1.0	99.0	56.9	0.10	—[a]	2.45	1.32	3.24	0.10	0.94	0.11	0.11	
		No	5.9	1.0	109.8	61.8	0.10	—[a]	16.18	1.28	20.66	0.10	0.93	0.11	0.11	
	T-30	No	4.8	2.5	89.2	66.7	0.25	—[a]	13.39	1.23	16.52	0.10	0.94	0.08	0.08	
		No	6.0	2.5	111.8	77.5	0.25	—[a]	13.85	1.15	15.93	0.10	0.93	0.09	0.09	
		No	8.5	2.5	158.9	100.0	0.28	—[a]	18.57	1.01	18.70	0.10	0.90	0.09	0.10	
	T-31	Yes	2.3	2.3	43.1	43.1	0.16	—[a]	3.45	1.47	5.07	0.20	0.97	0.13	0.13	
		Yes	3.1	2.3	57.9	50.0	0.16	—[a]	2.68	1.39	3.72	0.20	0.96	0.14	0.15	
	T-32	Yes	3.0	2.3	55.9	49.0	0.21	—[a]	3.23	1.40	4.52	0.20	0.96	0.14	0.15	
		Yes	3.2	2.3	59.8	51.0	0.21	—[a]	4.04	1.38	5.58	0.20	0.96	0.15	0.15	
		Yes	3.8	2.3	70.6	55.9	0.21	—[a]	2.88	1.33	3.84	0.20	0.95	0.16	0.16	
	T-33	Yes	3.2	2.3	59.8	51.0	0.15	—[a]	2.94	1.38	4.06	0.20	0.96	0.15	0.15	
		Yes	5.0	2.3	93.2	66.7	0.32	—[a]	5.74	1.23	7.08	0.20	0.94	0.17	0.18	
		Yes	5.6	2.3	103.9	77.6	0.32	—[a]	8.83	1.19	10.54	0.20	0.93	0.18	0.18	

TABLE 1. *(Continued)*

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
	T-34	Yes	2.6	2.5	48.1	47.1	0.13	—[a]	1.84	1.42	2.62	0.20	0.97	0.13	0.13	
	T-35	Yes	3.9	2.9	72.6	62.8	0.17	—[a]	2.50	1.27	3.17	0.20	0.95	0.14	0.15	
		Yes	4.0	2.9	74.5	63.7	0.17	—[a]	4.41	1.26	5.56	0.20	0.95	0.14	0.15	
		Yes	5.6	2.9	103.9	77.5	0.17	—[a]	4.16	1.15	4.78	0.20	0.93	0.16	0.17	
	T-36	No	6.0	2.3	111.8	75.5	0.22	—[a]	7.85	1.16	9.14	0.20	0.93	0.18	0.18	
Lutai area	L-1	No	6.9	0.4	111.8	57.2	0.062	—[a]	8.31	1.32	10.95	0.20	0.92	0.23	0.24	
		No	12.0	0.4	223.6	110.6	0.067	—[a]	4.46	0.95	4.25	0.20	0.86	0.23	0.23	
		No	13.1	0.4	244.2	120.4	0.067	—[a]	5.68	0.91	5.14	0.20	0.84	0.22	0.23	
	L-2	Yes	5.9	0.2	111.8	54.3	0.062	—[a]	2.43	1.35	3.28	0.20	0.93	0.25	0.26	
		Yes	6.0	0.2	118.7	55.3	0.062	—[a]	1.54	1.34	2.06	0.20	0.93	0.26	0.27	
		Yes	11.2	0.2	208.9	101.8	0.067	—[a]	1.42	1.00	1.42	0.20	0.87	0.23	0.24	
		Yes	11.6	0.2	215.7	104.6	0.067	—[a]	2.11	0.98	2.07	0.20	0.86	0.23	0.24	
		Yes	12.1	0.2	225.5	109.7	0.067	—[a]	2.55	0.96	2.44	0.20	0.85	0.23	0.24	
	L-3	Yes	11.2	0.4	208.9	101.8	0.067	—[a]	2.68	1.00	2.67	0.20	0.87	0.23	0.24	
		Yes	11.5	0.4	214.8	104.6	0.067	—[a]	1.75	0.98	1.71	0.20	0.86	0.23	0.24	
	L-4	No	11.1	0.8	206.9	106.5	0.067	—[a]	7.49	0.97	7.28	0.20	0.87	0.22	0.23	
(*e*) 1977 Vrancea Earthquake (M = 7.2)																
Dimbovitza (Site 1)		Yes	4.2	1.0	78.5	47.1	0.20	—[a]	5.12	1.42	7.29	0.22	0.95	0.23	0.22	Shibata and Teparaksa (1988)
		Yes	5.0	1.0	93.2	53.9	0.20	—[a]	3.66	1.35	4.94	0.22	0.94	0.23	0.22	
		Yes	6.0	1.0	111.8	62.8	0.20	—[a]	3.05	1.27	3.87	0.22	0.93	0.24	0.23	
		Yes	7.0	1.0	130.4	71.6	0.20	—[a]	1.29	1.19	1.55	0.22	0.92	0.24	0.23	
		Yes	8.0	1.0	149.1	80.4	0.20	—[a]	5.12	1.13	5.78	0.22	0.90	0.24	0.23	
(*f*) 1979 Imperial Valley Earthquake (M = 6.6)																
Heber Road	A2	No	4.0	2.1	62.8	44.5	0.11	15–20	19.90	1.45	28.91	0.60	0.95	0.52	0.46	Youd and Bennett (1983)
	A2	Yes	4.0	2.1	62.8	44.5	0.11	15–20	1.80	1.45	2.56	0.60	0.95	0.52	0.46	
	A3	No	4.0	2.1	62.8	44.5	0.08	40	7.00	1.45	10.11	0.60	0.95	0.52	0.46	
River Park	Unit A	Yes	2.0	0.2	31.4	13.9	0.07	50	2.00	1.92	3.77	0.20	0.98	0.29	0.25	
	Unit C	Yes	5.0	0.2	78.5	31.6	0.15	20	4.90	1.62	7.94	0.20	0.94	0.30	0.26	
(*g*) 1983 Nihonkai-Cho Earthquake (M = 7.7)																
Noshiro-Cho		No	3.1	2.0	56.9	47.1	0.32	—[a]	9.81	1.42	13.96	0.23	0.96	0.17	0.18	Shibata and Teparaksa (1968)
		No	3.8	2.0	71.6	53.0	0.32	—[a]	15.69	1.36	21.35	0.23	0.95	0.19	0.20	
		No	5.0	2.0	94.1	63.7	0.32	—[a]	15.08	1.26	19.00	0.23	0.94	0.21	0.21	
		Yes	2.8	2.1	53.0	45.1	0.32	—[a]	1.76	1.45	2.54	0.23	0.97	0.17	0.17	
		Yes	3.4	2.1	62.8	51.0	0.32	—[a]	4.02	1.38	5.55	0.23	0.96	0.18	0.18	
		Yes	5.1	2.1	94.1	65.7	0.32	—[a]	7.80	1.24	9.69	0.23	0.94	0.20	0.21	
		Yes	6.0	2.1	111.8	73.5	0.32	—[a]	8.80	1.18	10.38	0.23	0.93	0.21	0.22	
(*h*) 1988 Sanguenay Earthquake (M = 5.9)																
Ferland, Quebec, Canada		No	2.5	1.8	50.8	43.1	0.10	15	4.26	1.47	6.25	0.25	0.97	0.19	0.14	Tuttle et al. (1990)
		No	3.5	1.8	70.4	53.1	0.10	15	4.91	1.36	6.68	0.25	0.96	0.21	0.15	
		Yes	4.5	1.8	90.0	63.0	0.10	15	2.76	1.27	3.49	0.25	0.95	0.22	0.16	
		No	5.5	1.8	109.6	72.8	0.10	15	5.71	1.19	6.77	0.25	0.93	0.23	0.17	
		No	6.5	1.8	129.3	82.6	0.10	15	6.51	1.11	7.26	0.25	0.92	0.23	0.18	
		No	7.5	1.8	148.9	92.4	0.10	15	7.77	1.05	8.16	0.25	0.91	0.24	0.18	
		No	8.5	1.8	168.5	102.2	0.10	15	7.77	0.99	7.73	0.25	0.90	0.24	0.18	
(*i*) 1989 Loma Prieta Earthquake (M = 7.1)																
San Francisco Marina District	MAR1	No	5.8	2.3	118.4	84.0	0.303	5	16.75	1.10	18.51	0.24	0.93	0.20	0.19	Bennett (1990)
	MAR2	No	3.4	2.7	69.4	63.0	0.239	3	9.75	1.27	12.34	0.24	0.96	0.16	0.16	
		No	5.8	2.7	118.4	88.5	0.253	2	19.00	1.08	20.44	0.24	0.93	0.19	0.19	
	MAR3	No	3.8	2.7	77.6	67.2	0.275	4	13.94	1.23	17.14	0.24	0.95	0.17	0.16	
		No	4.9	2.7	100.0	78.9	0.361	3	18.00	1.14	20.52	0.24	0.94	0.19	0.18	
		No	6.9	2.7	140.9	100.1	0.350	4	13.00	1.01	13.08	0.24	0.92	0.20	0.19	
	MAR4	Yes	3.4	2.9	64.1	59.1	0.178	5	3.35	1.30	4.36	0.24	0.96	0.16	0.15	
		Yes	6.1	2.9	115.0	83.6	0.160	21	0.75	1.11	0.83	0.24	0.93	0.20	0.19	
	MAR5	Yes	6.4	2.4	120.6	81.8	0.197	3	1.20	1.12	1.34	0.24	0.92	0.21	0.20	
	MAR6	No	7.0	5.5	131.9	117.1	0.244	6	5.50	0.92	5.06	0.24	0.92	0.16	0.15	
Leonardini Farm	39	Yes	2.3	1.4	45.6	36.4	0.10	20–25	1.30	1.55	2.02	0.14	0.97	0.11	0.10	Charlie et al. (1994)
	38	Yes	2.2	1.7	44.1	39.5	0.10	20–25	1.50	1.51	2.27	0.14	0.97	0.10	0.09	
	37	No	3.0	2.1	60.4	51.8	0.12	20–25	2.50	1.37	3.43	0.14	0.96	0.10	0.10	
Port of Richmond	POR2	Yes	5–7	2.5	108.9	66.2	0.07	57	1.7	1.24	2.11	0.16	0.93	0.15	0.14	Kayen et al. (1992) and Mitchell et al. (1994)
	POR3	Yes	5–7	2.5	108.9	66.2	0.07	57	1.9	1.24	2.35	0.16	0.93	0.15	0.14	
	POR4	Yes	5–7	2.5	108.9	66.2	0.07	57	1.5	1.24	1.86	0.16	0.93	0.15	0.14	
San Francisco-Oakland Bay Bridge	SFOBB1	Yes	5–7.5	2.0	128.8	87.1	0.27	7	4.7	1.08	5.10	0.29	0.93	0.26	0.24	
	SFOBB2	Yes	6–9	2.0	154.5	100.6	0.26	12	10.0	1.00	10.04	0.29	0.91	0.26	0.25	
	SFOBB3	Yes	6–8	2.0	154.5	100.6	>0.25	—[a]	9.0	1.00	9.00	0.29	0.91	0.26	0.25	
	SFOBB4	Yes	6–8	2.0	154.5	100.6	>0.25	—[a]	5.0	1.00	5.00	0.29	0.91	0.26	0.25	
	SFOBB5	Yes	6–8	2.0	154.5	100.6	>0.25	—[a]	9.4	1.00	9.40	0.29	0.91	0.26	0.25	

TABLE 1. (*Continued*)

(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)	(15)	(16)	(17)
Port of Oakland	POO7-1	Yes	5–8	3.0	130.5	91.2	>0.25	0–5	11.7	1.06	12.38	0.29	0.92	0.25	0.23	
	POO7-2	Yes	5–7	3.0	111.8	82.4	0.30	3	8.7	1.12	9.71	0.29	0.93	0.24	0.22	
	POO7-3	Yes	4–7	3.0	116.5	84.6	0.30	5	6.5	1.10	7.15	0.29	0.93	0.24	0.22	
	POO7-4	No	7–12	3.0	177.1	113.3	>0.25	0–5	—[a]	0.94	17.00	0.29	0.89	0.26	0.24	
	POO7-5	Yes	4–6	3.0	93.2	73.6	>0.25	0–5	—[a]	1.18	12.00	0.29	0.94	0.22	0.22	
	POO7-6	Yes	4–7	3.0	102.5	78.0	>0.25	0–5	—[a]	1.15	10.00	0.29	0.93	0.23	0.22	
Oakland Airport	ACPT3	Yes	2–5	2.0	65.2	50.5	0.22	10	—[a]	1.39	10.00	0.27	0.96	0.22	0.20	
	ACPT4	Yes	2–5	2.0	65.2	50.5	0.22	10	—[a]	1.39	5.00	0.27	0.96	0.22	0.20	
	ACPT7	Yes	2–5	2.0	65.2	50.5	0.22	10	5.3	1.39	7.35	0.27	0.96	0.22	0.20	
Bay Farm Island	BFI-P6	Yes	2–5	2.0	65.2	50.5	0.22	10	6.1	1.39	8.45	0.30	0.96	0.24	0.23	
	DFI-DIKE	No	3–5	2.0	74.6	54.9	0.22	20	26.0	1.34	34.87	0.30	0.95	0.25	0.24	
	BFI-CPT1	Yes	2–4	2.0	55.9	46.1	0.22	—[a]	—[a]	1.43	10.00	0.30	0.96	0.23	0.21	

Note: MAR = Marina, and POR = Port of Richmond.
[a]Not available.

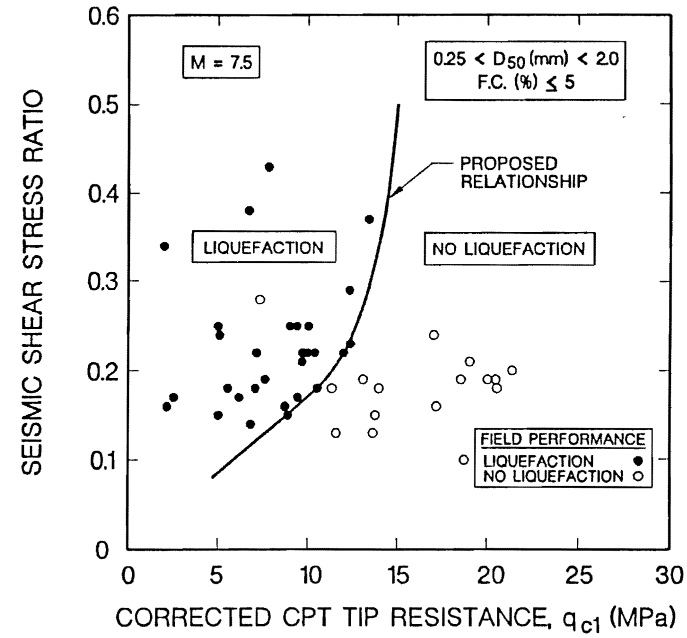

FIG. 2. Relationship between Seismic Shear-Stress Ratio Triggering Liquefaction and q_{c1}-Values for Clean Sand and M = 7.5 Earthquakes

D_{50} (mm) < 2.0 and FC (%) $\leq$ 5] for which CPT data are available. From the field data, a boundary line was drawn between liquefied sites and nonliquefied sites. This boundary defines a relationship between the mobilized undrained yield-strength ratio and CPT q_{c1}-values for clean sand and magnitude 7.5 earthquakes. This boundary represents a reasonable lower bound of the liquefied data, instead of attempting to encompass all the data, to be consistent with the concept of the mobilized undrained yield-strength ratio. Fig. 2 shows that the proposed liquefaction-potential relationship for clean sand is in good agreement with the field-case-history data.

Only one of the 29 field case histories where liquefaction was observed lies on the outside edge of the proposed relationship. This case history is from the 1989 Loma Prieta Earthquake (Kayen et al. 1992). The representative q_c-value corresponds to the average tip resistance in the depth range indicated by Kayen et al. (1992) as probably having liquefied. Within this depth range there is a zone of looser sand (lower q_c-values), which may correspond more precisely to the zone that liquefied initially. Therefore, the reported q_c-value may be slightly larger than the q_c-value in the looser sand. However, since the data point lies on the outside edge of the boundary, reinterpretation of the q_c-value would not alter the proposed relationship.

The one nonliquefaction case history that plots above the proposed relationship is from the 1971 San Fernando Valley Earthquake (Bennett 1989). This case involved a clean sand surrounded by sandy silt with significantly higher fines content. Therefore, the reported q_c-value may be considerably lower than a typical clean sand would exhibit.

The proposed relationship for clean sand is limited to values of D_{50} less than 2.0 mm because: (1) Liquefaction field case histories with CPT data and values of D_{50} greater than 2.0 mm are limited; and (2) the use of a standard cone penetrometer ("Standard" 1994) in coarse sand and gravel (gravel content as low as 5%) may result in artificially large values of q_c. These large values of q_c may, therefore, lead to an overestimation of liquefaction resistance (Seed and De Alba 1986).

Fig. 3 compares the proposed liquefaction-potential relationship with several existing correlations of liquefaction potential for clean sand and an earthquake magnitude of 7.5. The relationships proposed by Mitchell and Tseng (1990) for D_{50} = 0.40 mm and D_{50} = 0.20 mm are not included in Fig. 3, because they are nearly coincident with the relationships proposed by Shibata and Teparaksa (1988) for $D_{50} \geq 0.25$ mm and Seed and De Alba (1986) for D_{50} = 0.8 and FC < 5%, respectively. The relationship developed during this study is in agreement with the relationship proposed by Robertson and Campanella (1985) for SSR values between 0.13 and 0.25.

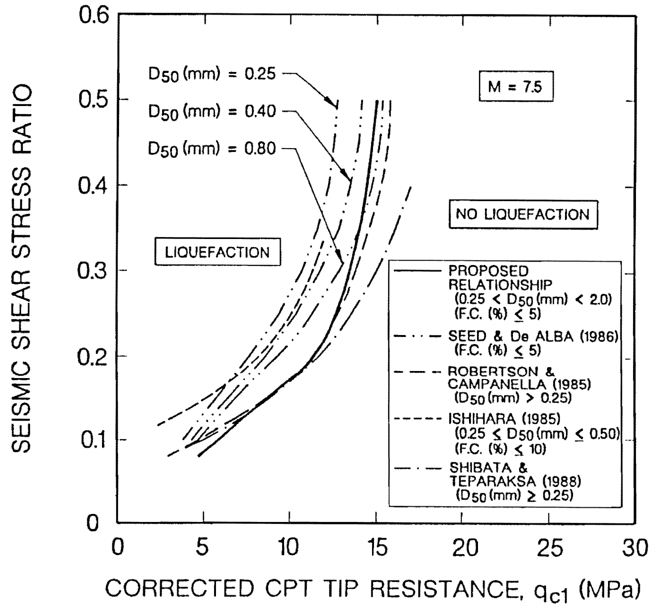

FIG. 3. Comparison of CPT Liquefaction-Potential Relationships for Clean Sand and M = 7.5 Earthquakes

At values of SSR less than 0.13 and greater than 0.25, the proposed relationship differs from the Robertson and Campanella (1985) relationship. At values of SSR less than 0.13, the proposed relationship can be extended to the origin as indicated by the SPT- and CPT-based clean-sand liquefaction-potential relationships proposed by Seed and De Alba (1986). At values of SSR greater than 0.25, the proposed relationship is less conservative than the Robertson and Campanella (1985) relationship.

The Seed and De Alba (1986) relationships were developed by converting the SPT $(N_1)_{60}$-values corresponding to the clean-sand liquefaction-potential relationship (Seed et al. 1985) to CPT q_{c1}-values for various values of D_{50}, rather than utilizing case histories in which CPT data are available. Seed and De Alba (1986) converted the SPT $(N_1)_{60}$-values on the clean-sand liquefaction-potential boundary to CPT q_{c1}-values using the q_c/N_{60} relationship that they proposed. This relationship is shown in Fig. 4 and will be discussed subsequently.

Robertson and Campanella (1985) also used the SPT field database presented by Seed et al. (1984) to develop CPT-based liquefaction-potential relationships for clean sand and silty sand. The SPT N-values from the case histories presented by Seed et al. (1984) were converted to CPT q_c-values using the Robertson and Campanella (1985) SPT-CPT conversion (also shown in Fig. 4). This differs from the Seed and De Alba (1986) conversion for values of D_{50} greater than approximately 0.02 mm and, thus, explains the difference in these liquefaction-potential relationships.

Ishihara (1985) used data in which field CPT q_c-values are available at the site of soil sampling, and the corresponding cyclic shear strengths were determined from laboratory cyclic triaxial tests to develop the liquefaction-potential relationship for clean sand in Fig. 3. In summary, Ishihara (1985) did not utilize field case histories to develop a liquefaction-potential relationship for clean sand, and the resulting relationship is less conservative than the proposed relationship.

Shibata and Teparaksa (1988) utilized field case histories in which CPT q_c-values and field SSRs are available to develop liquefaction-potential relationships. A grain size correction was developed to correct or calibrate the q_c-values of soils with D_{50} less than 0.25 mm to correspond to q_c-values obtained in clean sands ($D_{50} \geq 0.25$ mm). Shibata and Teparaksa (1988) assumed that the boundary between liquefied and nonliquefied sites is hyperbolic. This led to the development of a hyperbolic equation relating q_{c1} to SSR with a correction for $D_{50} < 0.25$ mm. The equation was used to estimate liquefaction potential for soils with $D_{50} < 0.25$ mm. By inserting various values of $D_{50} < 0.25$ mm into the equation, Shibata and Teparaksa (1988) calculated liquefaction-potential relationships for silty sand ($D_{50} = 0.20$ mm and $D_{50} = 0.15$ mm) and silty sand to sandy silt ($D_{50} = 0.10$ mm and $D_{50} = 0.05$ mm), which will be presented later in this paper.

Mitchell and Tseng (1990) developed two theoretical liquefaction-potential curves for clean sand ($D_{50} = 0.40$ mm and $D_{50} = 0.20$ mm), based on laboratory measured values of cyclic shear strength and theoretical values of CPT tip resistance predicted using the cavity expansion theory. As mentioned earlier, the relationship for $D_{50} = 0.40$ mm is in agreement with the clean-sand liquefaction-potential relationship proposed here for SSR values less than 0.25. The relationship for $D_{50} = 0.20$ mm is less conservative than the relationship proposed here, except for SSR values greater than 0.35.

In summary, the liquefaction-potential relationship presented in Fig. 2 is generally in agreement with existing relationships. However, earlier studies relied on a grain size correction (Shibata and Teparaksa 1988), a conversion of SPT blow count to CPT tip resistance (Seed and De Alba 1986; and Robertson and Campanella 1985), or laboratory cyclic triaxial data with estimated (Mitchell and Tseng 1990) or measured (Ishihara 1985) values of CPT tip resistance to estimate the liquefaction potential of clean sand. The proposed relationship is based solely on CPT-based liquefaction and nonliquefaction case histories and utilizes 45 clean-sand case histories to predict liquefaction potential. Therefore, the liquefaction-potential relationship proposed here represents the best estimate of the field behavior of clean sand during earthquakes from CPT data.

Liquefaction Potential of Silty Sand

Fig. 5 presents a compilation of 84 liquefaction and nonliquefaction field case histories involving silty sand [0.10 ≤

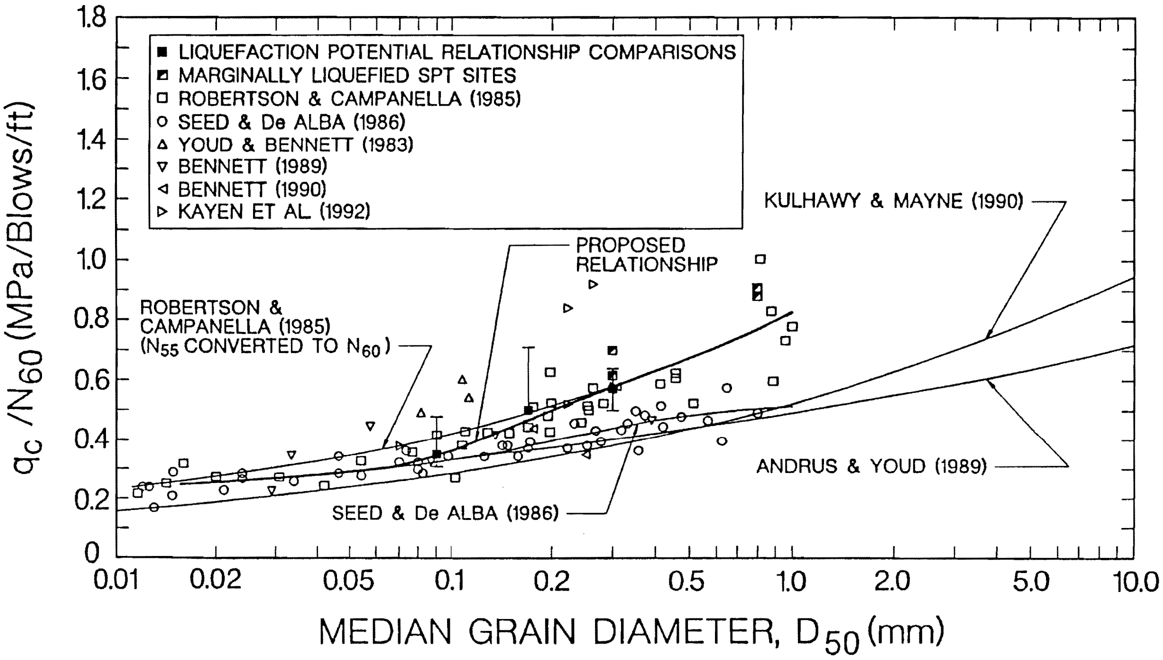

FIG. 4. Conversion of SPT N-Values to CPT q_c-Values Using Median Grain Diameter

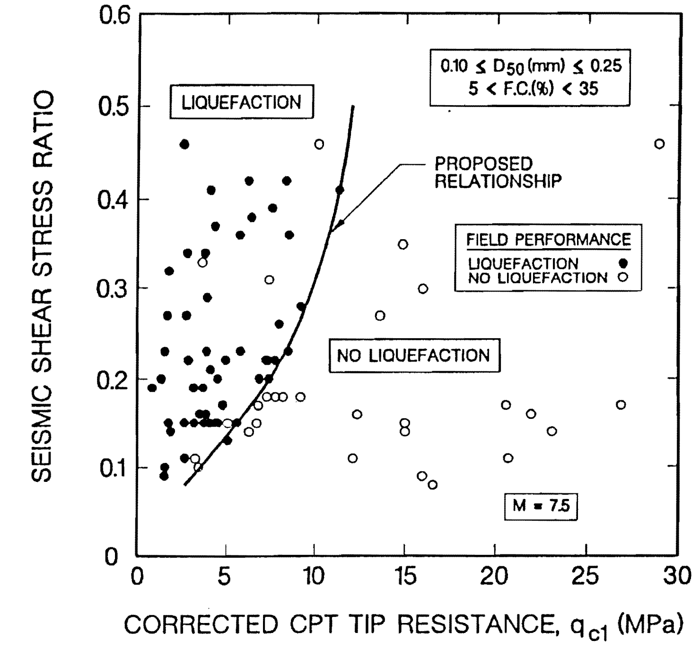

FIG. 5. Relationship between Seismic Shear-Stress Ratio Triggering Liquefaction and q_{c1}-Values for Silty Sand and M = 7.5 Earthquakes

D_{50} (mm) $\leq$ 0.25 and 5 < FC (%) < 35] where CPT data are available. From the field data, a boundary line between liquefied sites and nonliquefied sites was established. Similar to the boundary for clean sand, the boundary in Fig. 5 defines a relationship between the mobilized undrained yield-strength ratio and CPT q_{c1}-values for silty sand and magnitude 7.5 earthquakes. The relationship for silty sand plots to the left of the relationship for clean sand. It is anticipated that the plasticity of the fines reduces the potential for liquefaction during earthquake shaking because the fines reduce soil particle movement and pore-water pressure generation during shaking. Thus, a higher SSR is required to cause liquefaction in a silty sand than in a clean sand of equal relative density. In addition, the fines may cause a partially undrained condition during penetration, which can lead to a decrease in CPT tip resistance as compared with a clean sand of equal relative density. These two factors result in a silty sand appearing more resistant to liquefaction than a clean sand of equal relative density.

Only one of the 53 case histories where liquefaction occurred plots on the outside edge of the proposed boundary (Fig. 5). The case history not bounded is sounding T-31 from the 1976 Tangshan Earthquake (Shibata and Teparaksa 1988). It was not possible to obtain the original CPT log for interpretation; therefore, further scrutiny of the reported q_c-value was not possible for this case history. However, revising the measured q_c-value for this case history would not affect the proposed boundary.

It is seen that several nonliquefaction case histories plot above the proposed silty sand liquefaction-potential relationship, and thus in the liquefaction zone. The three cases below a SSR value of 0.2 are near the boundary and probably represent the transition from liquefiable to nonliquefiable conditions. The two anomalous cases with SSR values near or slightly above 0.3 are from the 1971 San Fernando Valley Earthquake and involve silty sand surrounded by soil with significantly higher fines content. Therefore, the reported q_{c1}-values may be lower than a typical silty sand would exhibit.

The final anomalous case with a SSR of 0.46 corresponds to the Heber Road site in the 1979 Imperial Valley Earthquake (Youd and Bennett 1983). Youd and Bennett (1983) indicated that it is possible that pore-water pressures increased and liquefaction occurred in this silty sand. However, Youd and Bennett (1983) found no surficial evidence of liquefaction from that soil unit. Therefore, this case was judged as a "no liquefaction" case history. As a result, this case was not weighted as heavily as cases where liquefaction was or was not clearly observed for the determination of the proposed boundary.

Fig. 6 compares the proposed liquefaction-potential relationship for silty sand with existing correlations of liquefaction potential for silty sand and an earthquake magnitude of 7.5. The proposed relationship is in agreement with the relationship proposed by Robertson and Campanella (1985) for silty sand (D_{50} < 0.15 mm), except for SSR values less than approximately 0.2. The proposed relationship also shows good agreement with the relationship proposed by Seed and De Alba (1986) for silty sand (D_{50} = 0.25 mm and fines content $\approx$ 10%). However, poor agreement is found with the Seed and De Alba (1986) relationship for silty sand (D_{50} = 0.20 mm and fines content $\approx$ 15%). The proposed relationship is in between the relationships proposed by Shibata and Teparaksa (1988) for D_{50} values of 0.15 mm and 0.20 mm.

In summary, previous silty sand liquefaction-potential relationships are sensitive to changes in D_{50}. This uncertainty is attributed to a lack of CPT-based case histories to clarify the effect of D_{50}. The proposed relationship (Fig. 5) encompasses the range of D_{50} [0.10 $\leq$ D_{50} (mm) $\leq$ 0.25] of existing relationships. As a result, the proposed relationship appears to clarify the effect of D_{50} on the liquefaction potential of silty sand and provides an encompassing relationship.

Liquefaction Potential of Silty Sand to Sandy Silt

Fig. 7 presents a compilation of 51 liquefaction and nonliquefaction field case histories for silty sand to sandy silt [D_{50} (mm) < 0.10 and FC (%) $\leq$ 35] where CPT data are available. From the field data, a boundary separating liquefied sites from nonliquefied sites was established. Similar to the clean-sand and silty-sand relationships, this boundary defines a relationship between the mobilized undrained yield-strength ratio and CPT q_{c1}-values for silty sand to sandy silty and magnitude 7.5 earthquakes. The proposed relationship is a slight

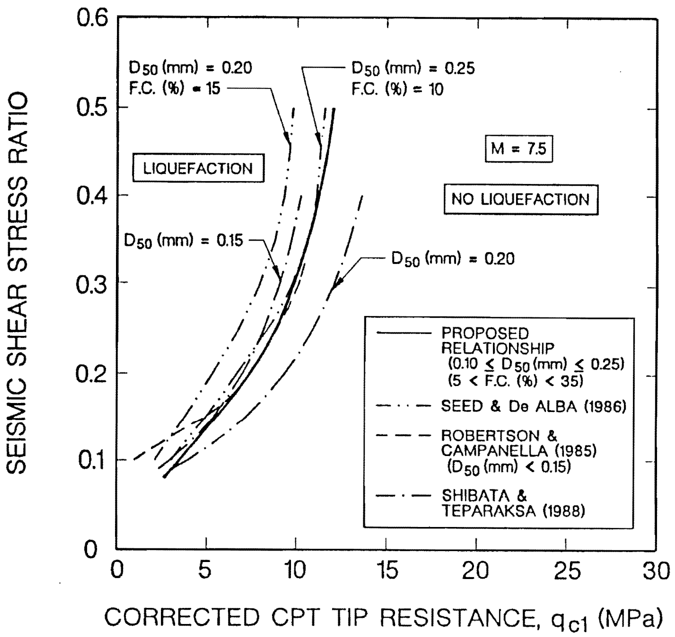

FIG. 6. Comparison of CPT Liquefaction-Potential Relationships for Silty Sand and M = 7.5 Earthquakes

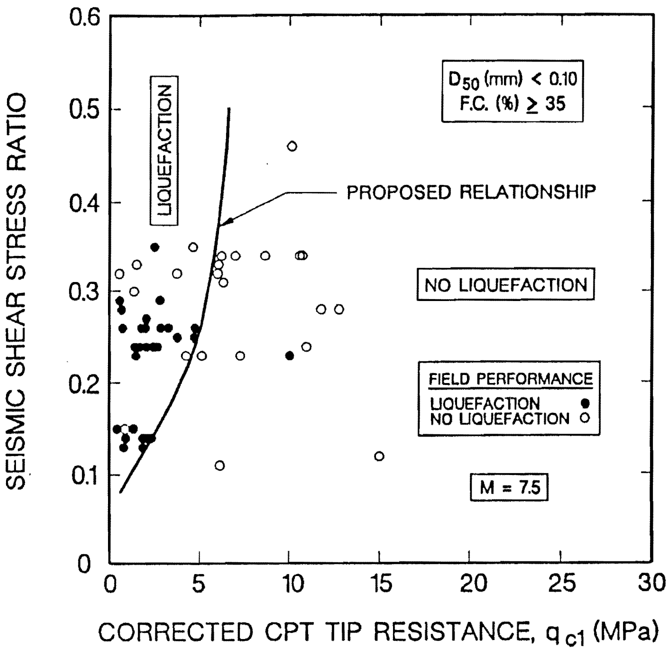

FIG. 7. Relationship between Seismic Shear-Stress Ratio Triggering Liquefaction and q_{c1}-Values for Silty Sand to Sandy Silt and M = 7.5 Earthquakes

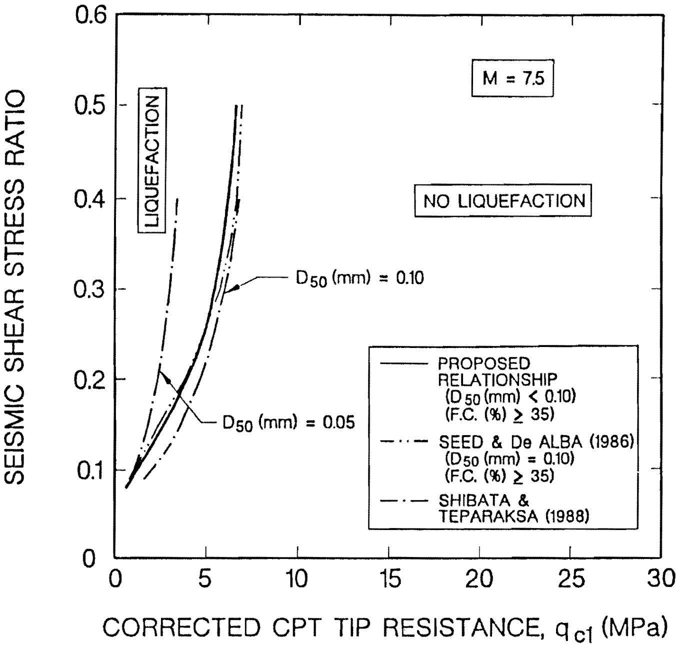

FIG. 8. Comparison of CPT Liquefaction-Potential Relationships for Silty Sand to Sandy Silt and M = 7.5 Earthquakes

modification of the relationship proposed by Seed and De Alba (1986) for silty sand (D_{50} = 0.10 mm and FC ≤ 35%) to describe the recently obtained CPT data.

Only one of the 28 cases where liquefaction was observed lies outside of the proposed boundary. This case history corresponds to the T-25 sounding from the 1976 Tangshan Earthquake (Shibata and Teparaksa 1988). The anomalously large q_c-value was reported by the investigators without explanation, and no further scrutiny was possible.

Several silty sand to sandy silt nonliquefaction cases plot above the proposed boundary. These cases generally involve soils with a fines content of 50% or greater. It is anticipated that the large fines content caused an undrained or partially drained condition during the CPT, which probably resulted in an underestimation of the q_c-value. It is possible that another boundary may need to be developed for sandy silt with a fines content of 50% or greater. However, at present there is insufficient data to develop such a relationship. Therefore, the proposed relationship for silty sand to sandy silt may underestimate, or conservatively estimate, the liquefaction resistance of a soil containing more than 50% fines.

The nonliquefaction case history with a SSR equal to 0.15, that plots above the proposed relationship, is the Middle School site from the 1975 Haicheng Earthquake (Arulanandan et al. 1986). In this case, the soil layer that was reported to have liquefied had a clay size fraction of more than 20%. This large clay size fraction probably accounts for the low q_c-value. Further, the liquefaction depth was reported as more than 10 m. At this depth, surface evidence of liquefaction may not be readily visible.

Fig. 8 compares the proposed liquefaction-potential relationship for silty sand to sandy silt with existing relationships for silty sand and an earthquake magnitude of 7.5. The proposed relationship is a modification of the Seed and De Alba (1986) relationship for silty sand to sandy silt (D_{50} = 0.10 mm and FC ≥ 35%). Modifications to the Seed and De Alba (1986) relationship were made to encompass the liquefaction case histories near a SSR value of 0.13, and to exclude the nonliquefaction case histories near SSR value of 0.32 (Fig. 7). The relationship proposed here is in between the relationships proposed by Shibata and Teparaksa (1988) for D_{50} = 0.10 mm and D_{50} = 0.05 mm. Robertson and Campanella (1985) and Ishihara (1985) did not present liquefaction relationships for silty sand to sandy silt.

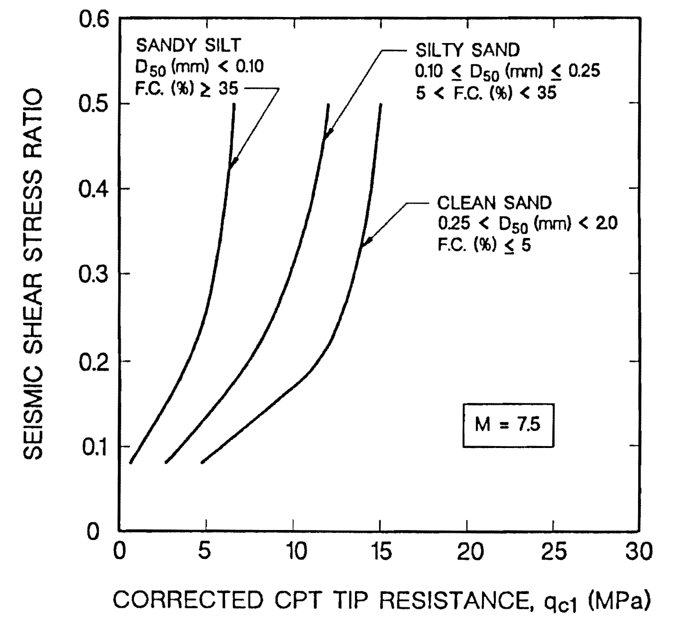

FIG. 9. CPT-Based Liquefaction-Potential Relationships for Sandy Soils and M = 7.5 Earthquakes

In summary, fines content and median grain diameter influence the liquefaction resistance of soils. As a result, different empirical relationships are presented for the liquefaction potential of clean sand, silty sand, and silty sand to sandy silt that are based on values of corrected CPT tip resistance (Fig. 9). The proposed liquefaction-potential relationships in Fig. 9 are obtained from Figs. 2, 5, and 7, and constitute a design assessment chart that can be used to estimate the factor of safety against liquefaction for an earthquake magnitude of 7.5, a vertical effective overburden stress equal to 100 kPa, and level ground conditions. Corrections described by Seed and Harder (1990) should be used to adjust the undrained yield-strength ratio estimated in Fig. 9 for other earthquake magnitudes, effective overburden stresses, and sloping ground conditions.

As recognized by investigators, the main disadvantage of the liquefaction relationships in Fig. 9, and thus, the use of the CPT in liquefaction assessments, is that an estimate of fines content and D_{50} is required. It is possible to estimate fines content from soil classification charts, e.g., Olsen and Farr (1986) and Robertson (1990), based on CPT and/or piezocone values of tip resistance and friction ratio. However, because of the uncertainties in estimating D_{50} from CPT results, it is recommended that the CPT be used to delineate zones and/or seams of potentially liquefiable soils. In zones of potential liquefaction, a sample and blow count(s) should be obtained to determine D_{50}, fines content, and to verify the liquefaction potential. This combination of CPTs and one or more borings has been used for many years and, thus, should not significantly increase the cost of a site investigation. Further, the proposed CPT-based liquefaction-potential relationships would allow the use of CPT data directly and should increase the effectiveness of liquefaction assessments because of the continuous profile of tip resistance versus depth. This profile allows the natural variability of sandy deposits to be characterized.

COMPARISON OF PROPOSED CPT RELATIONSHIPS AND SPT CASE HISTORIES

To utilize the large database of SPT-based liquefaction and nonliquefaction case histories (Seed et al. 1984) for comparison with the proposed CPT-based relationships, the SPT N-values must be converted to CPT q_c-values. Because of the large variation in the q_c/N_{60} conversion ratio for a given value of D_{50}, several conversions have been proposed (Fig. 4). The conversions developed for use in liquefaction analyses are presented by Seed and De Alba (1986), Robertson and Campanella (1985), and Andrus and Youd (1989). Several conversions over a larger range of D_{50} have also been proposed for general use, e.g., Kulhawy and Mayne (1990).

Clarification of SPT-CPT Conversion

Fig. 4 presents existing SPT-CPT conversions and the proposed SPT-CPT conversion. Fig. 4 also includes q_c/N_{60} data presented by Seed and De Alba (1986), Robertson and Campanella (1985), and additional data from field investigations conducted by Youd and Bennett (1983), Bennett (1989), Bennett (1990), and Kayen et al. (1992). The additional data exhibit a large variation in the ratio of q_c/N_{60} for a particular value of D_{50}. All SPT data was corrected to a SPT hammer energy of 60% as described earlier, and the data in Fig. 4 are average values of q_c/N_{60} reported by the investigators for subsurface layers where CPTs and SPTs were conducted adjacent to one another. The subsurface layers where adjacent CPT and SPT data are available did not necessarily liquefy.

The SPT-CPT conversion suggested by Seed and De Alba (1986) is based on median grain size and remains approximately constant for D_{50} values greater than approximately 0.5 mm (Fig. 4). As a result, Seed and De Alba (1986) used a value of q_c/N_{60} between 0.42 and 0.51 (MPa/blows/ft), which corresponds to a D_{50} between 0.25 mm and 0.8 mm, to convert the SPT $(N_1)_{60}$-values that correspond with the clean-sand liquefaction-potential relationship (Seed et al. 1985) to q_{c1}-values. These q_{c1}-values were used to develop their CPT-based liquefaction-potential relationships for clean sand (Seed and De Alba 1986), and are presented in Fig. 3.

Robertson and Campanella (1985) also proposed a SPT-CPT conversion relationship based on median grain size, but used an average energy ratio of 55% for the SPT N-values. For consistency, the Robertson and Campanella (1985) SPT-CPT conversion and data were corrected to an energy ratio of 60% using (1), and are presented in Fig. 4. Their SPT-CPT conversion indicates that the value of q_c/N_{60} should increase for all values of D_{50}.

Andrus and Youd (1989) developed a SPT-CPT conversion by extending the Seed and De Alba (1986) conversion to account for values of D_{50} up to 40–45 mm. The case histories used to extend the SPT-CPT conversion involve the 1983 Borah Peak Earthquake and gravelly soils. Andrus and Youd (1989) found no correlation between q_c and N_{60} when values of D_{50} were obtained from SPT samples. The investigators assumed this lack of agreement resulted from the diameter of the split spoon sampler being too small to obtain a representative sample of the gravelly soil. However, values of D_{50} obtained from 127-mm auger samples produced a logical correlation between q_c and N_{60} because a more representative value of D_{50} was obtained. Therefore, the values of D_{50} from the 127-mm auger samples were used to extend the SPT-CPT conversion. The values of CPT tip resistance used by Andrus and Youd (1989) were obtained using an electric cone with a conical tip area of 0.0015 m^2. This conical tip area is larger than the standard cone ("Standard" 1994), which is 0.001 m^2. The extended conversion developed by Andrus and Youd (1989) is considerably lower than the conversion proposed here. This may be caused by the N-values obtained from the SPT being slightly higher than would be expected for a clean sand. If the SPT sampler encountered large soil particles, the N-value could be artificially high. The overestimated N-values would result in lower values of q_c/N_{60}.

The conversion proposed by Kulhawy and Mayne (1990) is based on statistical analysis of q_c/N_{60} data from 197 cases, with values of D_{50} ranging from 0.001 mm to 10 mm. This database included the data from Robertson and Campanella (1985) and Seed and De Alba (1986). For values of D_{50} greater than 1 mm, however, the data used by Kulhawy and Mayne (1990) is limited and does not include several of the cases from Andrus and Youd (1986).

The SPT-CPT conversion proposed here was developed by determining the q_c/N_{60} ratios that yielded the best agreement between SPT liquefaction case histories and the proposed CPT-based liquefaction-potential relationships in Fig. 9. The proposed SPT-CPT conversion is intermediate to the Seed and De Alba (1986) and Robertson and Campanella (1985) conversions (Fig. 4). As expected from the agreement between the liquefaction-potential relationships for clean sand proposed here and by Robertson and Campanella (1985) in Fig. 3, the proposed SPT-CPT conversion is coincident with the Robertson and Campanella (1985) conversion for values of D_{50} greater than 0.3 mm. Similarly, the proposed SPT-CPT conversion is coincident with that developed by Seed and De Alba (1986) for values of D_{50} less than 0.08 mm. However, there is a lack of agreement between the new SPT-CPT conversion and existing conversions in Fig. 4 for values of D_{50} between 0.08 mm and 0.3 mm. The proposed SPT-CPT conversion deviates from the Seed and De Alba (1986) relationship to the Robertson and Campanella (1985) relationship for values of D_{50} between 0.08 and 0.3 mm.

Ratios of q_c/N_{60} used to determine the proposed SPT-CPT conversion were estimated by comparing the proposed CPT-based liquefaction-potential relationships in Fig. 9 with the SPT-based liquefaction-potential relationships proposed by Seed and De Alba (1986) in Fig. 1. For example, the CPT-based clean-sand liquefaction-potential relationship (Fig. 9) yields a value of q_{c1} of 12.5 MPa for a SSR of 0.25. In the SPT-based clean-sand liquefaction-potential relationship (Fig. 1), the value of $(N_1)_{60}$ corresponding to a SSR of 0.25 is 21.8 blows/ft. Therefore, the value of $q_{c1}/(N_1)_{60}$ is 12.5 MPa divided by 21.8 blows/ft, which equals 0.57 for a SSR of 0.25.

TABLE 2. Additional SPT-Based Liquefaction and Nonliquefaction Case Histories

Site (1)	Boring (2)	Liquefaction observed? (3)	Depth (m) (4)	Ground-water depth (m) (5)	Vertical total stress (kPa) (6)	Vertical effective stress (kPa) (7)	SPT $(N_1)_{60}$ (blows/ft) (8)	Median grain diameter (mm) (9)	Fines content (%) (10)	q_c/N_{60} (11)	q_{c1} (mPa) (12)	M = 7.5 seismic shear-stress ratio (13)	Reference (14)
						(*a*) 1987 Seismic Exploration							
Lake Ackermann, Michigan		Yes	4.7	3.05	90.7	74.6	3.2	0.40	0–5	0.628	2.01	0.12	Hryciw et al. (1990)
						(*b*) 1990 Luzon, Philippines Earthquake (M = 7.8)							
Luzon Area, A.B. Fernandez Avenue	13	No	9.4–10.7	0.9	187.3[a]	97.6[a]	22.9	0.13	8	0.414	9.48	0.203	Tokimatsu et al. (1994) and Ishihara et al. (1993)
	4	Yes	2.4–6.1	0.9	79.2[a]	102.0[a]	15.3	0.13	0	0.414	6.33	0.203	
	5	Yes	3.7–5.2	0.9	82.9[a]	48.1[a]	13.9	0.13	2	0.414	5.75	0.197	
	10	No	4.6–6.5	0.9	103.4[a]	57.8[a]	12.0	0.13	15	0.414	4.97	0.200	
	15	No	10.0–11.5	0.9	200.4[a]	103.7[a]	31.4	0.13	9	0.414	13.00	0.210	
Luzon Area, Perez Boulevard	12	No	6.5–8.0	0.9	135.1[a]	72.8[a]	25.2	0.15	15	0.436	10.99	0.208	
	2	Yes	1.2–8.0	0.9	85.7[a]	49.4[a]	5.8	0.15	10	0.436	2.53	0.224	
	3	Yes	7.6–9.3	0.9	157.5[a]	84.2[a]	15.1	0.15	10	0.436	6.58	0.213	
	1	Yes	4.3–10.0	0.9	133.3[a]	72.0[a]	14.2	0.15	10	0.436	6.19	0.206	
	11	Yes	7.7–10.7	0.9	171.5[a]	90.1[a]	12.3	0.15	22	0.436	5.36	0.211	
	16	Yes	4	0.9	74.6[a]	44.2[a]	20.0	0.15	9	0.436	8.72	0.191	

[a]Values estimated from available information; not used for calculations.

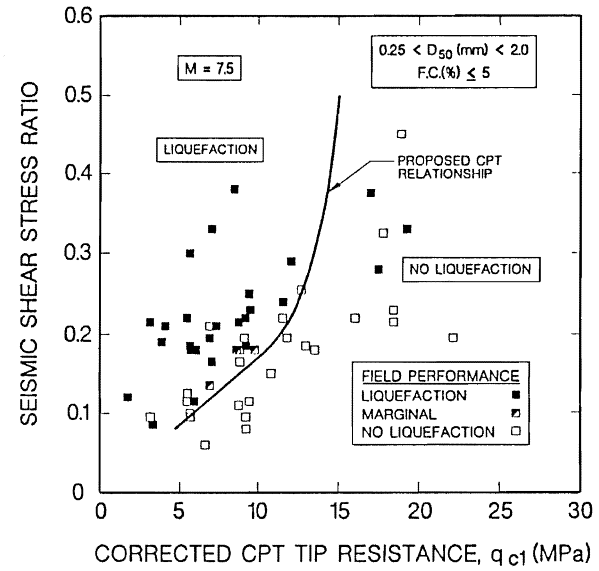

FIG. 10. Comparison of Clean Sand CPT Liquefaction-Potential Relationship and Converted SPT Field Case Histories

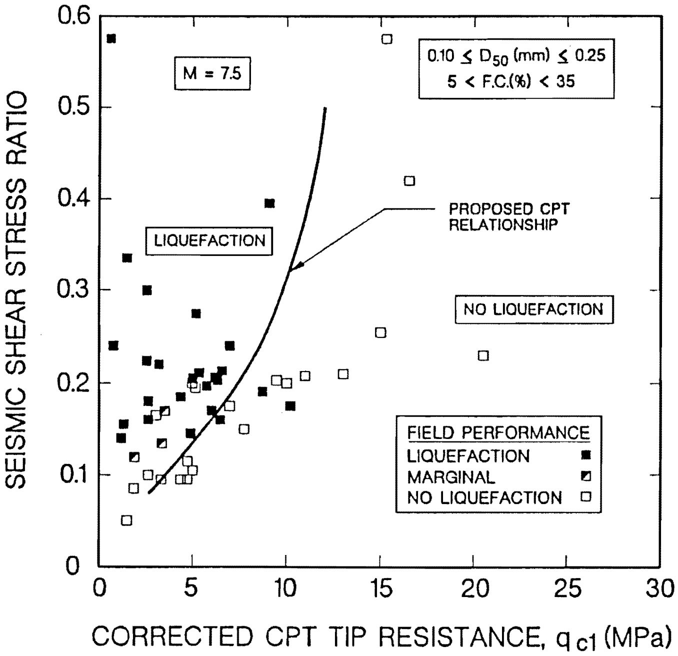

FIG. 11. Comparison of Silty Sand and CPT Liquefaction-Potential Relationship and Converted SPT Field Case Histories

No correction is necessary to convert $q_{c1}/(N_1)_{60}$ to q_c/N_{60} because C_q is equal to C_N at a vertical effective overburden stress of 100 kPa. The ratio of q_{c1}, obtained from the proposed CPT-based clean-sand liquefaction relationship in Fig. 9, to $(N_1)_{60}$, obtained from the SPT-based clean-sand liquefaction relationship in Fig. 1, ranges from 0.49 to 0.64 for all values of SSR. The weighted average value of $q_{c1}/(N_1)_{60}$ for clean sands is 0.57, which is plotted in Fig. 4 with the corresponding range at an average D_{50} of 0.30 mm. This average value of $q_{c1}/(N_1)_{60}$ was used to develop the proposed SPT-CPT conversion. This ratio is near the upper boundary of the data in Fig. 4 for a value of D_{50} between 0.2 and 0.3 mm. This suggests that the trend line in Fig. 4 should increase for values of D_{50} greater than 0.25 mm instead of remaining constant as proposed by Seed and De Alba (1986). This is also in agreement with the trend of the SPT-CPT conversion proposed by Robertson and Campanella (1985).

This process was repeated for the silty sand (average D_{50} of 0.17 mm) and silty sand to sandy silt (average D_{50} of 0.09 mm) liquefaction-potential relationships in Figs. 1 and 9. The range and weighted average values of $q_{c1}/(N_1)_{60}$ for these liquefaction-potential relationships are shown in Fig. 4. Additional support for the proposed SPT-CPT conversion was obtained by determining the q_c/N_{60} ratio required for the marginally liquefied SPT clean sand case histories to coincide with the CPT-based clean-sand liquefaction-potential curve. As shown in Fig. 4, these data plot slightly above the proposed SPT-CPT conversion and also suggest that the conversion should increase with increasing values of D_{50}.

These data guided the development of the proposed SPT-CPT conversion. Prior to this compilation of CPT liquefaction case histories and the comparison with SPT-based liquefaction-potential relationships, an estimate of the accuracy of SPT-CPT conversions for liquefaction analyses was not available. The proposed SPT-CPT conversion can be used for liquefaction-potential assessments because it is based on field liquefaction performance and not just on adjacent SPT and CPT data. However, the proposed SPT-CPT conversion is an average trend line, and there is considerable variance in the data used to develop this conversion. In summary, the proposed SPT-CPT conversion is more representative than

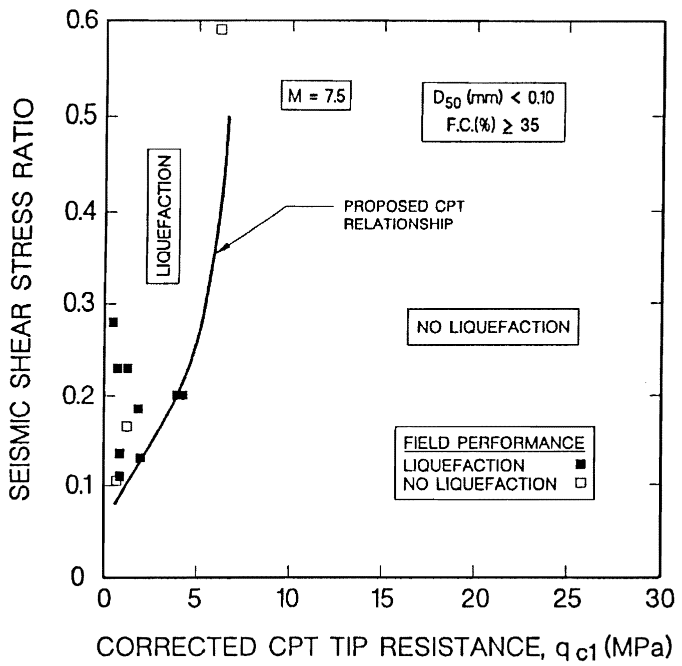

FIG. 12. Comparison of Silty Sand to Sandy Silt CPT Liquefaction-Potential Relationship and Converted SPT Field Case Histories

existing conversions, but site-specific conversions are still more desirable.

Comparison of CPT-Based Liquefaction-Potential Relationships and SPT-Based Field Data

Table 2 presents an augmentation of the SPT-based liquefaction-case-history database for sandy soils presented by Seed et al. (1984). The SPT N-values reported by Seed et al. (1984) and in Table 2 for liquefaction case histories were converted to CPT q_c-values using the proposed SPT-CPT conversion shown in Fig. 4. Figs. 10, 11, and 12 present the converted SPT-based case histories and the corresponding CPT-based liquefaction-potential relationships developed here for clean sand, silty sand, and silty sand to sandy silt, respectively.

The converted SPT data in Figs. 10 and 11 are in agreement with the proposed CPT-based liquefaction-potential relationships. The converted SPT data in Fig. 12 are in excellent agreement with the CPT-based liquefaction-potential relationship for silty sand to sandy silt. Disparity between the SPT data and CPT-based relationships is typically caused by the inability of the average SPT-CPT conversion to account for all case histories. The conversion proposed in Fig. 4 represents an average of the variable data. As shown in Fig. 4, a large variation from the proposed trend line can exist for individual case histories. This reinforces the need for site-specific SPT-CPT conversions.

LIQUEFACTION POTENTIAL OF GRAVELLY SOILS

Although far less common than cases of liquefaction in sandy soils, several case histories involving the liquefaction potential of gravelly soils have been documented. These case histories include the 1948 Fukui Earthquake (Ishihara et al. 1974), 1964 Alaskan Earthquake (Ishihara et al. 1989), 1975 Haicheng Earthquake (Wang 1984), 1976 Tangshan Earthquake (Wang 1984), 1983 Borah Peak Earthquake (Andrus and Youd 1989), and 1988 Armenia Earthquake (Yegian et al. 1994). Of these case histories, only the 1983 Borah Peak Earthquake yielded near-level ground liquefaction and non-liquefaction case histories where CPT tip-resistance data are available. Yegian et al. (1994) documented case histories in which a low permeability layer located directly above the gravelly layer was believed to have impeded drainage and led to a liquefaction flow failure.

The documented cases of liquefaction during the 1983 Borah Peak Earthquake include both clean and silty gravels. The Pence Ranch, Idaho site is underlain by a clean gravel, with a fines content ranging from 1 to 5%. The Whiskey Springs, Idaho site is underlain by a silty gravel, with a fines content

TABLE 3. Database of CPT-Based Liquefaction and Nonliquefaction Case Histories in Gravelly Soils

1983 Borah Peak Earthquake (M = 7.3)																
Site (1)	Sounding (2)	Liquefaction observed? (3)	Depth (m) (4)	Ground-water depth (m) (5)	Vertical total stress (kPa) (6)	Vertical effective stress (kPa) (7)	Median grain diameter (mm) (8)	Fines content (%) (9)	CPT q_c (MPa) (10)	C_q (11)	q_{c1} (MPa) (12)	Site a_{max} (g) (13)	r_d (14)	Site seismic shear-stress ratio (15)	M=7.5 seismic shear-stress ratio (16)	Reference (17)
Pence Ranch, Idaho	HY1-C	Yes	1.8–3.6	1.65	47.9	38.6	5.4	2	4.6	1.52	7.01	0.3	0.97	0.23	0.23	Andrus and Youd (1987, 1989) and Stokoe et al. (1988)
	HY1-D	No	3.6–5.0	1.65	77.9	51.8	12.0	2	15.2	1.37	20.86	0.3	0.95	0.28	0.27	
	HY2-C	Yes	0.9–4.1	1.45	42.9	32.9	9.0	3	5.3	1.60	8.48	0.3	0.97	0.25	0.24	
	HY2-D	No	4.1–5.0	1.45	79.3	49.1	4.0	5	15.2	1.40	21.29	0.3	0.95	0.30	0.29	
	HY3-C	Yes	0.8–3.1	1.35	33.7	28.0	—[a]	—[a]	5.6	1.67	9.36	0.3	0.98	0.23	0.22	
	HY3-D	No	3.1–5.2	1.35	73.6	46.3	—[a]	—[a]	17.1	1.43	24.49	0.3	0.95	0.29	0.29	
	BR1-C	Yes	2.1–5.3	1.85	65.7	47.6	2.5	1	7.3	1.42	10.34	0.3	0.96	0.26	0.25	
	BR1-D	No	5.3–7.0	1.85	109.1	67.1	—[a]	—[a]	17.0	1.23	20.92	0.3	0.93	0.29	0.29	
	PH1-C	Yes	0.9–2.6	1.1	30.0	23.6	5.6	1	6.0	1.74	10.46	0.3	0.98	0.24	0.24	
	PH1-D	No	2.6–5.2	1.1	69.4	41.7	12.0	3	18.5	1.49	27.47	0.3	0.95	0.31	0.30	
Whiskey Springs, Idaho	WS1B-C1	Yes	1.8–4.0	0.8	58.3	37.4	10.0	21	5.65	1.54	8.70	0.5	0.97	0.49	0.48	
	WS1B-D	No	5.9–6.2	0.8	122.4	70.7	34.0	15	23.65	1.20	28.42	0.5	0.93	0.52	0.51	
	WS2-C1	Yes	2.4–4.3	2.4	63.2	54.2	2.0	30	4.69	1.35	6.32	0.5	0.96	0.36	0.35	
	WS2-C3	No	4.3–6.0	2.4	100.5	74.1	>2.0	30	12.54	1.18	14.75	0.5	0.94	0.41	0.40	
	WS2-D	No	6.0–9.2	2.4	154.1	103.6	16.0	20	16.28	0.99	16.08	0.5	0.91	0.44	0.43	
	WS3-C1	Yes	6.7–7.8	6.7	136.2	131.0	13.0	21	6.89	0.86	5.93	0.5	0.91	0.31	0.30	
	WS3-C3	No	7.8–9.3	6.7	163.8	145.9	3.5	23	13.69	0.80	11.00	0.5	0.90	0.33	0.32	
	WS3-D	No	9.3–12.5	6.7	215.7	174.5	3.5	17	21.35	0.71	15.24	0.5	0.87	0.35	0.34	

[a]Not available.

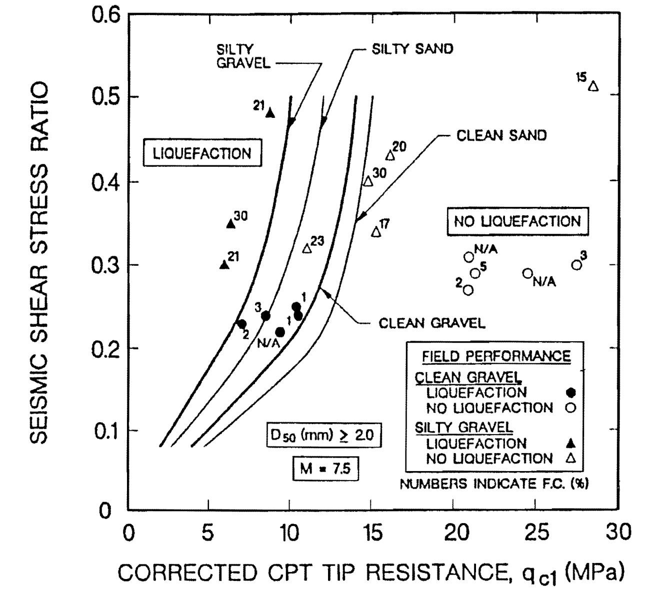

FIG. 13. Relationship between Seismic Shear-Stress Ratio Triggering Liquefaction and q_{c1}-Values for Gravel and M = 7.5 Earthquakes

ranging from 15% to 30%. Table 3 presents the pertinent data from both Idaho sites.

At both sites, a cone with a conical tip area of 0.0015 m^2 rather than the standard tip area of 0.001 m^2 was used to ensure penetration into the gravelly soils. The SSR values for the sites were estimated using (4), which was used for the sandy soil case histories. No correction was employed for gravel content.

Fig. 13 presents the available case histories for gravelly soils [D_{50} (mm) $\leq$ 2.0]. The fines content of each of the case histories is displayed next to the data point. Tentative liquefaction-potential relationships are presented for clean gravel (fines content less than 5%) and silty gravel (fines content approximately 20%), based on the separation of sites that experienced liquefaction and those that did not experience liquefaction during the 1983 Borah Peak Earthquake. For comparison, the CPT-based clean sand and silty sand liquefaction relationships are included in Fig. 13. The liquefaction-potential relationships for both the clean gravel and silty gravel plot above the liquefaction-potential relationships for clean sand and silty sand, respectively. This indicates that gravelly soil exhibits greater liquefaction resistance than sandy soil. Unfortunately, the data supporting this hypothesis are rather limited. As more data becomes available on the field behavior of gravelly soil during earthquakes, the liquefaction-potential relationships presented here may need to be reevaluated.

CONCLUSIONS

The CPT appears to be better suited to liquefaction assessments than the SPT because it is more standardized, reproducible, cost-effective and, most importantly, yields a continuous penetration record with depth. The continuous profile is important in sandy soils because these deposits are inherently nonuniform. Therefore, a number of CPTs can be quickly and economically conducted to identify thick and thin layers of liquefiable soil, which may be cost-prohibitive with SPT.

This paper presented 180 field case histories where liquefaction was and was not observed in sandy soils and values of CPT tip resistance are available. These data are used to develop relationships between soil resistance to liquefaction and corrected CPT tip resistance for clean sand, silty sand, and silty sand to sandy silt and an earthquake magnitude of 7.5. The proposed CPT-based relationships were developed to describe the field case histories where CPT data are available, and to eliminate the need to convert SPT blow counts to CPT resistance.

Tentative liquefaction-potential relationships were presented for clean gravel and silty gravel and for an earthquake magnitude of 7.5 based on 18 liquefaction and nonliquefaction field case histories. An electrical cone with a conical tip area of 0.0015 m^2 instead of the standard conical tip area of 0.001 m^2 was used to estimate the CPT tip resistance of the gravelly soils. These relationships indicate that the liquefaction resistance of gravelly soil is greater than the liquefaction resistance of sandy soil.

The main disadvantage of the CPT is the lack of a sample for soil classification and grain size analyses. Since liquefaction resistance depends on fines content and median grain size, it is recommended that a sample and blow counts be obtained in the liquefiable soil to determine D_{50}, fines content, and verify the liquefaction potential. The combination of CPTs and SPTs has been used for many years and, thus, should not significantly increase the cost of a site investigation. However, the CPT-based liquefaction-potential relationships will allow the CPT data to be directly used in liquefaction assessments instead of relying on a conversion of CPT tip resistance to SPT blow count.

ACKNOWLEDGMENTS

This study was performed as a part of the National Science Foundation (NSF), Grant Number BCS-93-00043. The support from the NSF is gratefully acknowledged. The first writer also acknowledges the support provided by the William J. and Elaine F. Hall Scholar Award.

APPENDIX. REFERENCES

Andrus, R. D., and Youd, T. L. (1987). "Subsurface investigation of a liquefaction-induced lateral spread, Thousand Springs Valley, Idaho." *Geotech. Lab. Miscellaneous Paper GL-87-8*, U.S. Army Corps. of Engrs., Wtrwy. Experiment Station, Vicksburg, Miss.

Andrus, R. D., and Youd, T. L. (1989). "Penetration tests in liquefiable gravels." *Proc., 12th Int. Conf. on Soil Mech. and Found. Engrg.*, A. A. Balkema, Rotterdam, The Netherlands, 679–682.

Arulanandan, K., Yogachandran, C., Meegoda, N. J., Ying, L., and Zhauji, S. (1986). "Comparison of the SPT, CPT, SV and electrical methods of evaluating earthquake induced liquefaction susceptibility in Ying Kou City during the Haicheng Earthquake." *Proc., Use of In Situ Tests in Geotech. Engrg., Geotech. Spec. Publ. No. 6*, ASCE, New York, N.Y., 389–415.

Bennett, M. J. (1989). "Liquefaction analysis of the 1971 ground failure at the San Fernando Valley Juvenile Hall, California." *Bull. Assoc. of Engrg. Geologists*, 26(2), 209–226.

Bennett, M. J. (1990). "Ground deformation and liquefaction of soil in the Marina District." *Effects of the Loma Prieta Earthquake on the Marina District, San Francisco, California; Open File Rep. 90-253*, Dept. of the Interior, U.S. Geological Survey, Denver, Colo., 44–79.

Charlie, W. A. Doehring, D. O., Brislawn, J. P., Scott, C. E., and Butler, L. W. (1994). "Liquefaction evaluation with the CSU piezovane." *Proc., 13th Int. Conf. on Soil Mech and Found. Engrg., Vol. 1*, 197–200.

Hryciw, R. D., Vitton, S., and Thomann, T. G. (1990). "Liquefaction flow failure during seismic exploration." *J. Geotech. Engrg.*, ASCE, 116(12), 1881–1899.

Ishihara, K. (1985). "Stability of natural deposits during earthquakes." *Proc., 11th Int. Conf. on Soil Mech. and Found Engrg.*, A. A. Balkema, Rotterdam, The Netherlands, *Vol. 1*, 321–376.

Ishihara, K., Acacio A. A., and Towhata, I. (1993). "Liquefaction-induced ground damage in Dagupan in the July 16, 1990 Luzon Earthquake." *Soils and Found.*, Tokyo, Japan, 33(1), 133–154.

Ishihara, K., Kokusho, T., and Silver, M. L. (1989). "General report/discussion session 27: earthquakes: influence of local conditions of seismic response—state-of-the-art report: recent developments in evaluating liquefaction characteristics of local soils." *Proc., 12th Int. Conf. on Soil Mech. and Foundation Engrg.*, A. A. Balkema, Rotterdam, The Netherlands, 2719–2734.

Kayan, R. E., Mitchell, J. K., Seed, R. B., Lodge, A., Nishio, S., and

Coutinho, R. (1992). "Evaluation of SPT-, CPT-, and shear wave-based methods for liquefaction potential assessments using Loma Prieta data." *Proc., 4th Japan-U.S. Workshop on Earthquake Resistant Des. of Lifeline Facilities and Countermeasures for Soil Liquefaction; NCEER-92-0019*, Nat. Ctr. for Earthquake Engrg., Buffalo, N.Y., 177–192.
Kulhawy, F. H., and Mayne, P. W. (1990). "Manual on estimating soil properties for foundation design." *Electric Power Res. Inst. EL-6800; Prof. 1493-6*, Electric Power Res. Inst., Palo Alto, Calif., 2–38.
Liao, S. C., and Whitman, R. V. (1985). "Overburden correction factors for SPT in sand." *J. Geotech. Engrg.*, ASCE, 112(3), 373–377.
Marcuson, W. F., and Bieganousky, W. A. (1977). "Laboratory standard penetration tests on fine sands." *J. Geotech. Engrg. Div.*, ASCE, 103(6), 565–588.
Mitchell, J. K., and Tseng, D. J. (1990), "Assessment of liquefaction potential by cone penetration resistance." *Proc., H. B. Seed Memorial Symp., Vol. 2*, BiTech Publishing, Vancouver, B.C., Canada, 335–350.
Mitchell, J. K. et al. (1994). "Insitu test results from four Loma Prieta Earthquake liquefaction sites: SPT, CPT, DMT, and shear wave velocity." *Rep. No. UCB/EERC-94/04*, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley, Calif.
Olsen, R. S., and Farr, J. V. (1986). "Site characterization using the cone penetrometer test." *Proc., INSITU '86, ASCE Spec. Conf. on Use of In Situ Testing in Geotech. Engrg., Geotech. Spec. Publ. No. 6*, 854–868.
Robertson, P. K. (1990). "Seismic cone penetration testing for evaluating liquefaction potential." *Proc., Symp. on Recent Advances in Earthquake Des. Using Lab. and In Situ Tests*, ConeTec Investigations Ltd., Burnaby, B. C., Canada.
Robertson, P. K., and Campanella, R. G. (1985). "Liquefaction potential of sands using the CPT." *J. Geotech. Engrg.*, ASCE, 111(3), 384–403.
Seed, H. B., and De Alba (1986). "Use of SPT and CPT tests for evaluating the liquefaction resistance of sands." *Proc., INSITU '86, ASCE Spec. Conf. on Use of In Situ Testing in Geotech. Engrg., Spec. Publ. No. 6*, ASCE, New York, N.Y.
Seed, H. B., and Idriss, I. M. (1971). "Simplified procedure for evaluating soil liquefaction potential." *J. Geotech. Engrg. Div.*, ASCE, 97(9), 1249–1273.
Seed, H. B., Idriss, I. M., and Arango, I. (1983). "Evaluation of liquefaction potential using field performance data." *J. Geotech. Engrg.*, ASCE, 109(3), 458–482.
Seed, H. B., Tokamatsu, K., Harder, L. F., and Chung, R. (1984). "The influence of SPT procedure on soil liquefaction resistance evaluations." *Rep. No. UCB/EERC-84/15*, Earthquake Engrg. Res. Ctr., Univ. of California, Berkeley, Calif.
Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. (1985). "Influence of SPT procedures in soil liquefaction resistance evaluations." *J. Geotech. Engrg.*, ASCE, 111(12), 861–878.
Seed, R. B., and Harder Jr., L. F. (1990). "SPT-based analysis of cyclic pore pressure generation and undrained residual strength." *Proc., H. B. Seed Memorial Symp., Vol. 2*, BiTech Publishing, Vancouver, B. C., Canada, 351–376.
Shibata, T., and Teparaksa, W. (1988). "Evaluation of liquefaction potentials of soils using cone penetration tests." *Soils and Found.*, Tokyo, Japan, 28(2), 49–60.
"Standard test for deep, quasi-static, cone and friction-cone penetration test of soil—D 3441-86." (1994). *Annual book of standards; Vol. 04.08, Section 4*, ASTM, Philadelphia, Pa., 338–343.
Stark, T. D., and Mesri, G. (1992). "Undrained shear strength of liquefied sands for stability analysis." *J. Geotech. Engrg.*, ASCE, 118(11), 1727–1747.
Stokoe II, K. H., Andrus, R. D., Rix, G. J., Sanchez-Salinero, I., Sheu, J. C., and Mok, Y. J. (1988). "Field investigations of gravelly soils which did and did not liquefy during the 1983 Borah Peak, Idaho, Earthquake." *Geotech. Engrg. Rep. GR87-1*, Civ. Engrg. Dept., Univ. of Texas, Austin, Tex.
Tokimatsu, K., Kojima, H., Kuwayama, S., Alie, A., and Midorikawa, S. (1994). "Liquefaction-induced damage to buildings in 1990 Luzon Earthquake." *J. Geotech. Engrg.*, ASCE, 120(2), 290–307.
Tuttle, M., Law, K. T., Seeber, L., and Jacob, K. (1990). "Liquefaction and ground failure induced by the 1988 Saguenay, Quebec, Earthquake." *Can. Geotech. J.*, Ottawa, Canada, 27(5), 580–589.
Wang, W. (1984). "Earthquake damages to earth dams and levees in relation to soil liquefaction." *Proc., Int. Conf. on Case Histories in Geotech. Engrg.*, Univ. of Missouri, Rolla, MO., 512–522.
Yegian, M. K., Ghahraman, V. G., and Harutiunyan, R. N. (1994). "Liquefaction and embankment failure case histories, 1988 Armenia Earthquake." *J. Geotech. Engrg.*, ASCE, 120(3), 581–596.
Youd, T. L., and Bennett, M. J. (1983). "Liquefaction sites, Imperial Valley, California." *J. Geotech. Engrg.*, ASCE, 109(3), 440–457.