Penetration resistance and liquefaction of sands

Résistance à la pénétration et liquéfaction des sables

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SYNOPSIS Some information is given which may be helpful when using the electrical cone penetration test to evaluate the risk of liquefaction of natural sand deposits. The following aspects are examined: the normalization of q_c and N_{SPT} with respect to $\bar{\sigma}_{vo}$; the influence of stress and strain history on q_c and N_{SPT}; the q_c / N_{SPT} ratio for sands.

1. INTRODUCTION

The various available approaches for the evaluation of the liquefaction potential of saturated sands during earthquakes have been thoroughly reviewed in some recent important state-of-the-art reports and many comprehensive papers [Finn (1981), Seed (1983), Seed et al. (1983), Tokimatsu and Yoshimi (1983), ...]. From these works it appears that there are two basic approaches: the first is based on the results of properly performed laboratory tests on sand specimens and seems to be the more rational one, while the other relies on observations of the performance of sand deposits in previous earthquakes correlated empirically with soil characteristics measured by means of in-situ tests. Unfortunatelly, the difficulties which are encountered during undisturbed sampling of sands below ground water level and, to a lesser extent, problems related to laboratory equipment put us in a situation in which the first approach still cannot be considered satisfactory. In these circumstances, methods correlating the observed performance of sand deposits during earthquakes to the Standard Penetration Resistance, N_{SPT}, became an extremely precious tool for the soils engineer.
In the last few years, due to the widely recognized variability and the lack of repeatability of the SPT results, but also because of new developments in in-situ testing techniques, researchers and practing engineers have focused their attention on the possibility of correlating the cyclic resistance of sand deposits to other in-situ tests [Schmertmann (1978), Ohsaki and Iwasaki (1973), Zhou (1980, 1981), Marchetti (1982), Robertson and Campanella (1983), Kok (1983), Norton (1983), Seed (1983), ...].
In this paper, the writers wish to give some complementary information which may be helpful for the use of the electrical Fugro-type CPT tip (of cylindrical shape) and the development of correlations between the liquefaction resistance of the natural sand deposits and cone resistance, q_c.

The present contribution is based on the analysis of the following experimental data:

1a. Extensive electrical CPT tests in fine to coarse clean sands (quarz sands predominant) carried out in the last 12 years under strictly controlled laboratory conditions using large-scale calibration chambers (CC) [Schmertmann (1976), Holden (1971), Lhuer (1976), Harman (1976), Chapman (1979), Parkin et al. (1981), Parkin and Lunne (1982), Baldi et al. (1981), (1981a), (1982), (1983), Bellotti et al. (1983)].
1b. Laboratory investigation (CC) on the SPT performed by Bieganousky and Marcuson (1976, 1977) on four different types of medium to coarse quarz sands.
1c. Results of a large number of in situ SPT and CPT performed in the late seventies in mainly cohesionless deposits of the plane of the river Po.
1d. Cyclic simple shear tests (DSS) performed on pluvially deposited specimens of medium to coarse Ticino sand.

On the basis of the above mentioned data, the following specific aspects of the considered problem are examined here:

- Normalization of q_c and N_{SPT} with respect to the initial effective overburden stress, $\bar{\sigma}_{vo}$.
- Influence of stress and strain histories on q_c and N_{SPT}.
- Relationship between q_c and N_{SPT} for sands.
- Influence of the pre-stressing and pre-straining on the cyclic resistance of pluvially deposited Ticino Sand.

2. NORMALIZED PENETRATION RESISTANCE

To use correlations based on the performance of sand deposits during previous earthquakes, the measured penetration resistances, N_{SPT} and q_c, are generally corrected to an effective overburden pressure of 1 kg/cm², by means of the following relationships:

$$N_1 = C_N \cdot N_{SPT} \text{ (blows/foot)} \qquad (1)$$

$$q_{c1} = C_q \cdot q_c \quad (\text{kg/cm}^2) \qquad (2)$$

where C_N and C_q are functions of the effective overburden pressure at the depth of the penetration test. Values of C_N and C_q may be evaluated in a reliable way by carefully performed laboratory calibrations of the SPT and CPT (see reference under points 1a and 1b).
Referring to these data and considering only normally consolidated sand specimens, it is convenient [for explanation of this procedure, see Harman (1976) and Schmertmann (1976)] to fit the experimental results by the following functions:

$$N_{SPT} = B_o \exp (D_R \cdot B_1) \cdot (\bar{\sigma}_{vo})^{B_2} \text{ blows/foot} \qquad (3)$$

TABLE I SPT - CALIBRATION CHAMBER TESTS

SAND TYPE	N (a)	STATE	CU (b)	D_{50} (mm)	γ_{max} (t/m^3)	γ_{min} (t/m^3)	PASSING SIEVE N° 200 ASTM (%)	C_o (*)	C_1 (*)	C_2 (*)	D_R RANGE	R^2 (-)	REFERENCE
REID BEDFORD	89	S	1.6	0.25	1.716	1.421	≅ 1	1.54	2.94	0.595	$24\% < D_R < 75\%$	0.85	(c)
OTTAWA	8	S	1.5	0.21	1.748	1.490	≅ 1	0.055	8.02	0.868	$57\% < D_R < 63\%$	0.98	(c)
STANDARD CONCRETE	20	S	2.1	0.5	1.932	1.661	≅ 3	1.70	3.28	0.664	$20\% < D_R < 96\%$	0.96	(d)
PLATTE RIVER	20	S	5.3	2.0	1.965	1.647	≅ 3	1.76	3.71	0.395	$19\% < D_R < 92\%$	0.99	(d)
FOR ALL CONSIDERED NC SPECIMENS: N = 137; B_o = 1.40; B_1 = 3.35; B_2 = 0.56; R^2 = 0.89													

TABLE II CPT - CALIBRATION CHAMBER TESTS

SAND TYPE	N (a)	STATE	CU (b)	D_{50} (mm)	γ_{max} (t/m^3)	γ_{min} (t/m^3)	PASSING SIEVE N° 200 ASTM (%)	C_o (*)	C_1 (*)	C_2 (*)	D_R RANGE	R^2 (-)	REFERENCE
EDGAR 70-140	10	D+S	1.4	0.16	1.650	1.311	≅ 1	11.9	3.22	0.685	$31\% < D_R < 70\%$	0.97	(e)
EDGAR 30-65	21	D	1.8	0.48	1.750	1.410	≅ 1	11.3	2.39	0.824	$48\% < D_R < 99\%$	0.98	(f)
OTTAWA 90	23	D	1.8	0.21	1.823	1.515	≅ 1	10.5	3.57	0.729	$20\% < D_R < 83\%$	0.97	(g)
OTTAWA 90	11	S	1.8	0.21	1.823	1.515	≅ 1	10.3	3.26	0.737	$28\% < D_R < 80\%$	0.97	(g)
REID BEDFORD	17	D+S	1.7	0.24	1.748	1.448	≅ 1	12.3	2.79	0.788	$24\% < D_R < 81\%$	0.98	(e)
HILTON-MINE	15	D	2.2	0.20	1.893	1.497	≅ 3	12.1	3.05	0.603	$30\% < D_R < 84\%$	0.96	(g)
HILTON-MINE	5	S	2.2	0.20	1.893	1.497	≅ 3	11.5	2.61	0.600	$30\% < D_R < 84\%$	0.97	(g)
TICINO	66	D	1.5	0.60	1.700	1.391	≅ 1	13.5	2.84	0.584	$11\% < D_R < 95\%$	0.98	(h)
HOKKSUND	20	D	2.2	0.44	1.750	1.414	≅ 1	11.3	3.31	0.736	$28\% < D_R < 95\%$	0.99	(i)
MELBOURNE	18	D	2.1	0.32	1.832	1.526	≅ 1	13.6	2.19	0.855	$52\% < D_R < 100\%$	0.97	(l)
FOR ALL CONSIDERED NC SPECIMENS: N = 206; C_o = 11.79; C_1 = 2.93; C_2 = 0.72; R^2 = 0.92													

(*) Referred to NC sands only; S = submerged; D = drained; (a) Number of Tests;
(b) Coefficient of uniformity; (c) Bieganousky e Marcuson (1976); (d) Bieganousky e Marcuson (1977):
(e) Lhuer (1976); (f) Holden (1971); (g) Harman (1976);
(h) Joint Research of Enel Cris (Milan), Ismes (Bergamo) and Technical University (Turin);
(i) Parkin et al. (1981); (l) Chapman (1979)

$$q_c = C_o \exp (D_R \cdot C_1) \cdot (\bar{\sigma}_{vo})^{C_2} \text{ kg/cm}^2 \qquad (4)$$

where:

D_R = relative density as fraction of unity

$\bar{\sigma}_{vo}$ = effective overburden stress, in kg/cm^2

B_o, B_1 and B_2; C_o, C_1 and C_2 } experimental coefficients

Values of these coefficients, derived from almost all presently available results of calibration chambers SPT and CPT tests, are given in Tables I and II. In these sands, the rate of cone penetration, 2 cm/sec, assures virtually drained test conditions: the results of the CC tests show that the q_c values of measured on saturated specimens are very close to the values obtained on dry specimens with the same D_R and stress history (Baldi et al., 1983; Bellotti et al. 1983). Fig. 1 shows C_N and C_q values computed from B_2 and C_2 obtained from the best fit of the available data from NC sands. In addition, Fig. 2 shows a possible range of the variation of C_q computed for the sands listed in Table II.

The above mentioned C_N and C_q values may be taken as representative of NC uncemented, unaged quarz clean sands.

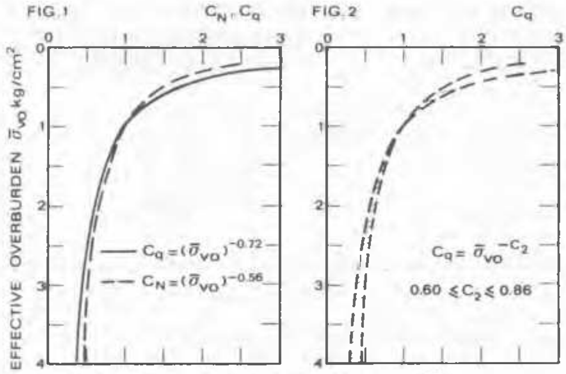
3. INFLUENCE OF STRESS AND STRAIN HISTORY ON PENETRATION RESISTANCE

The current developments in the methods which correlate in-situ test results to the observed behaviour of sand deposits during earthquakes require a better understanding of how these tests are influenced by the stress and strain history of the soil.

In fact, field evidence and laboratory test results show clearly that the age and past stress and strain history have a great influence on the cyclic resistance of cohesionless soils [Seed and Peacock (1971); Seed et al. (1975)].

On the other hand, very little is known as far as the influence of these factors on the in-situ tests is concerned. The present view of this important aspect of the problem may be summarized as follows:

a. Bieganousky and Marcuson (1976) presented the results of twelve SPT tests carried out in the calibration chamber on Reid Bedford Model sand specimens with an OCR equal to 3.
Table III compares N_{SPT} resistances measured on OC specimens with N_{SPT} resistances obtained on NC specimens of the same sand: it is possible to observe that the SPT seems to be quite insensiti



Figs. 1 and 2. C_N and C_q Versus $\bar{\sigma}_{vo}$

ve to soil stress history. Probably the number of available experiments and the value of the overconsolidation ratio are too limited to allow any definitive conclusions.

b. Schmertmann (1975) suggested the following empirical correlation between q_c and OCR on the basis of an extensive laboratory calibration of the CPT:

$$\frac{q_c^{OC}}{q_c^{NC}} = 1 + \chi\ (OCR^{\beta} - 1) \qquad (5)$$

where:

q_c^{NC} = cone resistance on NC sand

q_c^{OC} = cone resistance on OC sand

χ = experimental coefficient

β = exponent in the well-known empirical formula $K^{RB}/K_o^{NC} = OCR^{\beta}$

K_o^{NC} = coefficient of earth pressure at rest for NC sands

K^{RB} = ratio of lateral to vertical effective stress during one-dimensional rebound

$\chi \cong 0.75$ and $\beta \cong 0.42$ are the values suggested by Schmertmann (1975)

c. Lambrecht and Leonards (1978) postulated, on the basis of small-scale laboratory tests, that q_c is completely insensitive to past strain history.
Marchetti (1982), Baldi et al. (1982) and Bellotti et al. (1983) documented by means of numerous large-scale laboratory CC tests on Ticino sand, that q_c is only slightly influenced by the strain history of the soil specimen. By contrast, the DMT, in virtue of its geometry which causes much smaller shear strains during penetration, appeared more sensitive to past strain history.

On the basis of extensive CC tests performed by the writers on Ticino and Hokksund sand, the following information may be added:

d. Equation (5) can be used to express the relationship between q_c and OCR, but for the above two tested sands the values of both χ and β are different from the ones suggested by Schmertmann.

e. For both sands β appears to increase with increasing D_R [Baldi et al. (1983)]. The relationship between β and D_R may be expressed, on first approximation, as follows:
for Ticino sand: $\beta \cong 0.30 + 0.27\ D_R$
for Hokksund sand: $\beta \cong 0.25 + 0.25\ D_R$

f. The available experimental data, obtained from CC tests performed on Ticino and Hokksund sands, show that χ varies approximately between 0.50 (OCR = 2) and 0.25 (OCR = 15), decreasing with increasing D_R.

TABLE III

Influence of Overconsolidation on Laboratory N_{SPT}
Reid Bedford Model Sand
Data from Bieganousky and Marcuson (1976)

D_R (%)	$\bar{\sigma}_v$ (kg/cm^2)	N_{SPT} blows/foot	
		NC	OCR = 3
58	0.7	11 to 13	12
58	2.8	22 to 23	25 to 27
58	5.6	-	35 to 38
40	0.7	6 to 9	4 to 5
40	2.8	7 to 15	16
40	5.6	25 to 31	27 to 30

g. For OC sands, no unique relationship between q_c, D_R and $\bar{\sigma}_{vo}$ exists. The experimental data for both NC and OC calibration chamber specimens may still be fitted by means of eqn.(4), replacing $\bar{\sigma}_{vo}$ by the mean effective stress, $\bar{\sigma}_o$ [see Baldi et al. (1983)].
In the case of Ticino sand, for both NC and OC specimens, one obtains:
C_o=17.1; C_1=2.96; C_2=0.59; R^2=0.96; N=112.

h. The laboratory calibration of the CPT in Ticino and Hokksund sand indicates that the use of correlations $C_q = f\ (\bar{\sigma}_{vo})$ and $C_N = f\ (\bar{\sigma}_{vo})$ as the ones shown in Fig. 1 should be restricted to NC deposits only.
If one wants to use this type of correlation also for OC sands, one has to refer to the $\bar{\sigma}_o$ rather than $\bar{\sigma}_{vo}$.
Unfortunately, a reliable assessment of OCR and/or K_o values of natural sand deposits is still an unsolved problem.

i. The experimental evidence summarized in Fig.3 and in Table IV shows that q_c in Ticino sand is almost independent of strain history when the

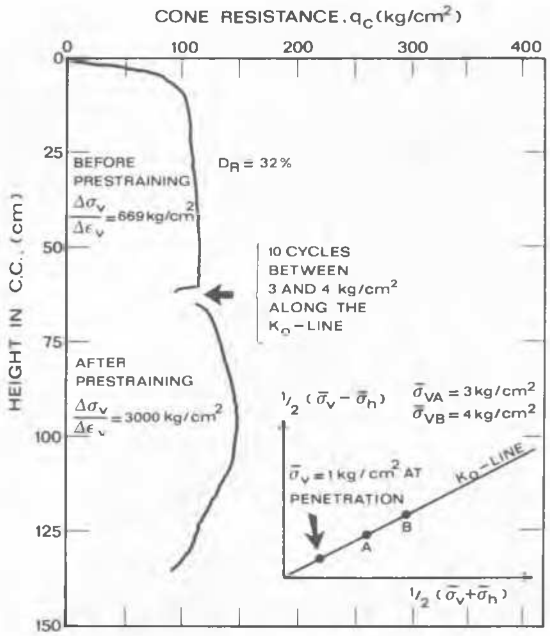


Fig. 3. Example of the Influence of Strain History on q_c.

TABLE IV: INFLUENCE OF PRESTRAINING ALONG K_0-LINE ON CONE RESISTANCE AND DEFORMABILITY OF TICINO SAND (a)

TEST N°	$\bar{\sigma}_{VA}$ (kg/cm^2)	$\bar{\sigma}_{VB}$ (kg/cm^2)	D_R (%)	N_c (-)	BEFORE PRESTRAINING q_c (kg/cm^2)	$\Delta\bar{\sigma}_v/\Delta\varepsilon_v$ (kg/cm^2)	K_o (-)	AFTER PRESTRAINING q_c (kg/cm^2)	$\Delta\bar{\sigma}_v/\Delta\varepsilon_v$ (kg/cm^2)	K_o (-)
4	1.17	1.65	44	1	84.4	559	0.467	91.3	6240	0.465
6	1.15	1.65	42	7	96.0	470	0.484	112	2232	0.499
7	1.13	3.15	44	7	107.0	661	0.482	116.8	2820	0.487
8	1.14	1.67	44	7	102.0	565	0.508	115.6	2437	0.503
70(b)	1.13	4.19	36	10	61.8	476	0.469	91.8	4692	0.469
71	1.13	1.64	36	10	60.2	474	0.487	72.7	2947	0.492

(a) Tests performed in the calibration chamber [see Bellotti et al. (1983)] on dry pluvially deposited sand; (b) Usually penetration was performed at $\bar{\sigma}_v=\bar{\sigma}_{VA}$; in test n°70 the specimen was subjected to cyclic loading between 3.17 and 4.19 kg/cmq; then unloaded along the K_o-line to $\bar{\sigma}_v$=1.13 kg/cm^2 before penetration was resumed. The results shown in Fig. 3 refer to a test carried out under very similar conditions.
N_c=number of loading-unloading cycles; $\Delta\varepsilon_v$=change in vertical strain; $\Delta\sigma_v$=change in vertical stress;

specimen is subjected to cyclic loading along the K_o-line, while the stiffness of the sand increases appreciably.

It appears, that both N_{SPT} and q_c are influenced probably only to a limited extent by past stress-strain history of the soil which, on the other hand, has an appreciable influence on sand stiffness. Even if the lack of q_c response to prestraining along other stress paths still has to be proved, it is reasonable to aspect that CPT tests will not be able to detect the past strain history of a natural sand deposit. Moreover, the results of CC tests carried out on Ticino and Hokksund sand show that q_c reflects only to some extent changes in the horizontal effective stress, $\bar{\sigma}_h$, due to mechanical overconsolidation.
This situation is very different from the one which is encountered when dealing with the cyclic resistance of saturated sands where the strain and stress history is important.
In particular, it must be pointed out that strain history, without being reflected by the measured q_c, may lead to an increase of the resistance of sands against liquefaction.
Table V and Fig.4 show the results of cyclic direct simple shear (DSS) tests carried out on pluvially deposited Ticino sand from which, qualitatively, the influence of both mechanical overconsolidation and prestraining along the K_o-line may be inferred. It may be interesting to compare the values of the cyclic stress ratio SR_ℓ at liquefaction of NC specimens with those of OC specimens (with OCR = 3.5). Considering for example N=15 cycles, SR_ℓ increases from ≅ 0.15 ÷ 0.16 up to ≅0.28 - 0.29, while the corresponding q_c value, evaluated by means of eqn.(4), increases only from ≅ 100 to ≅ 125 kg/cm^2.

TABLE V

Example of the Influence of Stress and Strain Histories on the Resistance of Pluvially Deposited - Ticino Sand in DSS Cyclic Tests

HISTORY	SR_ℓ (-)	N_ℓ (-)	D_R (%)
OCR=1	0.244	11	71 ± 1
OCR=1.6	0.225	13	
OCR=2.5	0.242	17	
OCR=3.5	0.233	26	
OCR=3.5	0.243	20	
PSR=3.5	0.220	30	
PSR=3.5	0.270	16	

SR_ℓ=stress ratio causing initial liquefaction, γ (double amplitude) equal to 7.5% and/or 95% of the pore pressure response
N_ℓ =number of stress cycles at liquefaction
PSR=prestraining ratio=$\bar{\sigma}_{VB}/\bar{\sigma}_{VA}$, see Fig.3 and Table IV

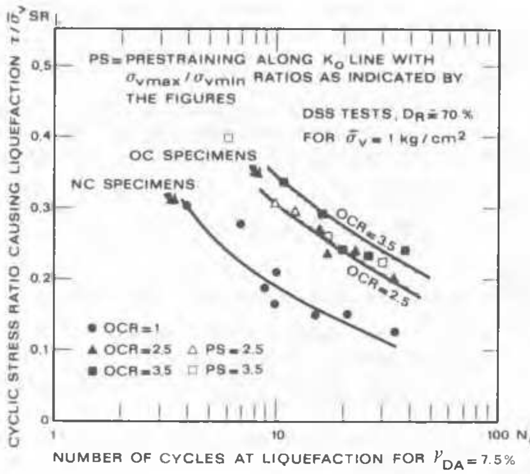


Fig. 4. Example of the Influence of Stress-Strain History on SR_ℓ.

4. CORRELATIONS BETWEEN N_{SPT} AND q_c

In the last few years, the interest in assessing the ratio q_c/N_{SPT} has increased because of attempts to include also CPT data in the procedures for evaluating the liquefaction potential from SPT data (Seed, 1983; Robertson and Campanella 1983).
A comprehensive review of such correlations has been presented by Robertson et al. (1983). These authors have taken into account the influence on the q_c/N_{SPT} ratio of both soil grain size composition and SPT energy input; the resulting correlation curve is shown in Fig.5. This curve refers mainly to USA and Canadian SPT data where the N_{SPT} resistance is usually obtained using a driving method in which a cathead with two turns of a Manila rope is generally employed and a sampling spoon without liners. The writers, on the basis of a large number of SPT and CPT (only electrical Fugro type CPT tip has been used) tests carried out in predominantly cohesionless deposits of the western and central part of the Po valley [Cerutti (1979), Fuoco (1984)], have worked out the q_c vs. N_{SPT} correlations shown in Figs. 5 and 6.
In this case, all N_{SPT} values were obtained using a trip hammer delivering an average ≅ 64% of the theoretical driving energy to the top of the driving roads and sampling spoon with liners.

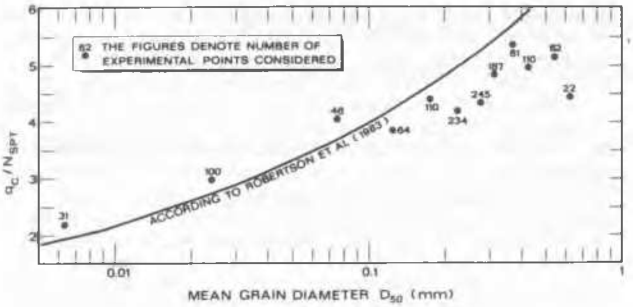


Fig.5 q_c/N_{SPT} Ratio Versus D_{50}

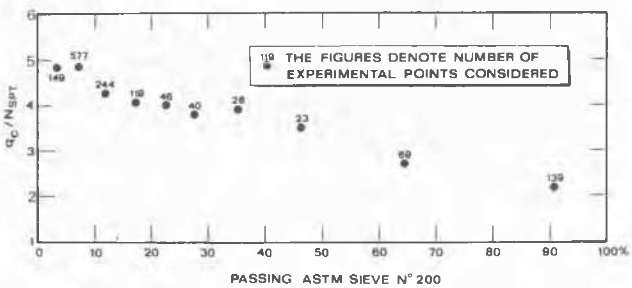


Fig. 6 q_c/N_{SPT} Ratio Versus Fines Content

The comparison of the Italian q_c/N_{SPT} ratios with correlation curve of Robertson et al. (1983) is shown in Fig.5. In spite of the differencies in soil types, applied energies and used sampler, the agreement between the two correlations appears to be surprisingly good.

Fig. 6 shows a similar comparison, referring instead of D_{50} to the percentage of fines passing ASTM sieve No. 200 ASTM. However, the writers wish to point out that a lot of caution should be used when applying this type of correlation in design problems.

Firstly, one has to keep in mind that the q_c/N_{SPT} ratio may be strongly influenced by local soil conditions and test procedures. Secondly, it must be pointed out that mean values of q_c/N_{SPT} ratios like the ones shown in Figs.5 and 6 are generally obtained from experimental data which are widely scattered (see Fig. 7).

FINAL REMARKS

In the present work some aspects of the calibration of SPT and CPT tests carried out under well-controlled laboratory conditions are reviewed and possible consequences are discussed with respects to the methods used for evaluating the resistance against liquefaction of a natural sand deposit. The experimental evidence described allows the following comments.

1° The response of both SPT and CPT tests to the effects of a mechanical overconsolidation of sands is quite small.

2° The results of calibration chamber tests show that q_c is almost independent of past straining along the K_o-line.

3° On the other hand, both overconsolidation and past straining along the K_o-line tend to increase the cyclic resistance and stiffness of a cohesionless soil.

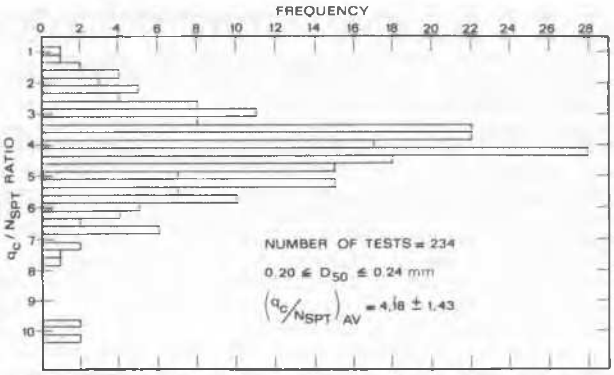


Fig. 7 Example of Histogram of q_c/N_{SPT} Ratio

4° Therefore, the use of N_{SPT} and/or q_c values for the evaluation of the cyclic stress resistance of natural sand deposits subjected to a complex stress-strain history might appear questionable.

5° The possibility of using q_c instead of N_{SPT} in field performance charts like the ones presented by Seed (1983), Robertson and Campanella (1983) is presently limited because of a lack of adequate data bases which correlate q_c and sand liquefaction characteristics. The transformation of N_{SPT} to an equivalent q_c on the basis of an assumed q_c/N_{SPT} ratio [see Seed (1983), Robertson and Campanella (1983)] may lead to unrealistic answers because the charts refer to mean values deduced from widely scattered experimental data.

6° The presently available information on correlations between q_c and SR_ℓ for magnitude 7½ - earthquakes is summarized in Fig.8 in which the

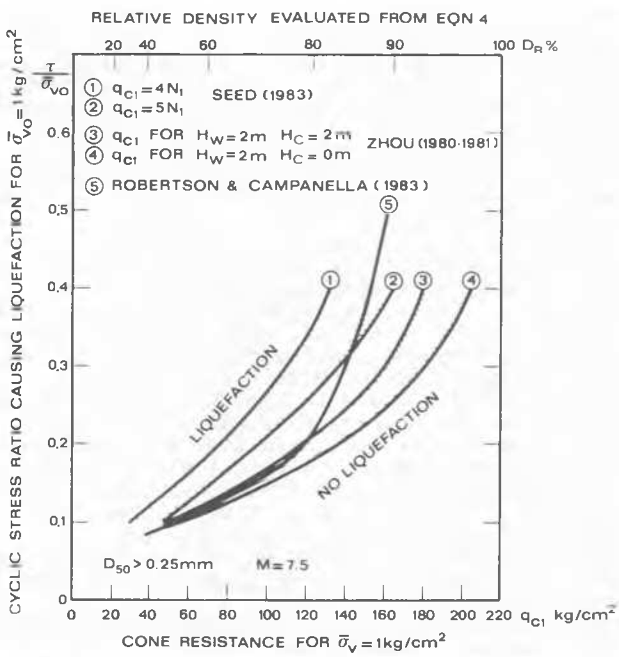


Fig. 8 Evaluation of Liquefaction Potential from q_c Values

values of D_R(=f (q_c)) are also indicated. The figure refers to eqn.(4), using C_0, C_1 and C_2 values which are representative for the NC clean, unaged quarz sands tested in the calibration chamber. The curves referring to Chinese experience [Zhou (1980)] have been obtained assuming the GWL at a depth of H_w=2 m below G.L. and supposing that the thickness, H_c, of the top most cohesive layer is 0 and 2 meters, respectively. It may be postulated that this type of correlation, because of its inherent limitations [Whitman (1984)], should be referred only to young NC deposits. Its use when dealing with deposits which have a complex stress and/or strain history requires a knowledge of the initial, lateral in situ effective stress and the development of some new in situ device which is much more sensitive to the effects of past stress and strain histories.

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