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Energy Approach for Liquefaction of Sandy and Clayey Silts

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SYNOPSIS: The liquefaction potential of sandy and clayey silts is assessed in this paper using an energy approach original developed for sand. Three series of laboratory tests were conducted to examine the liquefaction resistance of clean silt, sandy silt and clayey silt. The test results have been analyzed to establish a liquefaction failure criterion for silty soils. Based on this criterion, the case records of a new database have been studied and the results suggest that a single criterion can be used for sand, silt, sandy silt and clayey silt. For sandy or silty soils without clay content, the criterion is expressed in terms of the earthquake magnitude, hypocentral distance and the corrected standard penetration resistance. For clayey silt, the same criterion and parameters can be used except the standard penetration resistance has to be modified in terms of the clay content.

INTRODUCTION

Soil liquefaction is a state of soil particle suspension caused by a complete loss of strength when the effective stress drops to zero. Hence liquefaction normally occurs in soils such as sand in which the strength is purely frictional. Indeed, serious study on soil liquefaction was started by the devastating damage due to sand liquefaction failure during the 1964 Niigata earthquake in Japan and the great Alaskan earthquake in the same year in the U.S.A. Since then, a number of investigations both in the laboratory and in the field has been conducted on liquefaction behaviour of clean sand.

As the level of urbanization and energy resource exploration have increased in recent years, more and more sites featuring granular soils other than clean sand have been found to suffer from soil liquefaction failure. The 1976 Tangshan earthquake which claimed a quarter of million lives provided many examples of soil liquefaction failures in silty soils containing different contents of sand and clay (Zhou and Gou, 1985). Research on liquefaction potential of this type of deposit is inadequate in comparison with that on sand.

The work reported in this paper is an attempt to combine laboratory and case record study to evaluate the liquefaction potential of sandy and clayey silt. A newly developed energy approach is used to establish a single criterion applicable for sand, silt, sandy silt and clayey silt. For sandy or silty soils without clay content, the criterion is expressed in terms of the earthquake magnitude, hypocentral distance and the corrected standard penetration resistance. For clayey silt, the same parameters are also used except an equivalent corrected standard penetration resistance is employed.

THE ENERGY METHOD

The essence of the energy method is to use a criterion in terms of energy dissipated in the soil for determining the occurrence of liquefaction failure. Liquefaction failure is defined as the condition when the excess pore pressure is equal to the initial effective confining pressure ($\sigma_o{}'$). Based on laboratory studies on sand by Law et al (1990), the development of excess pore pressure under cyclic loading is closely related to the total energy dissipated in the soil. The total energy consists of two components: one from hysteretic damping and the other from plastic deformation. The energy dissipated through hysteretic damping for one load cycle can be represented by the area of the hysteretic loop of the soil as shown in Figure 1(a).

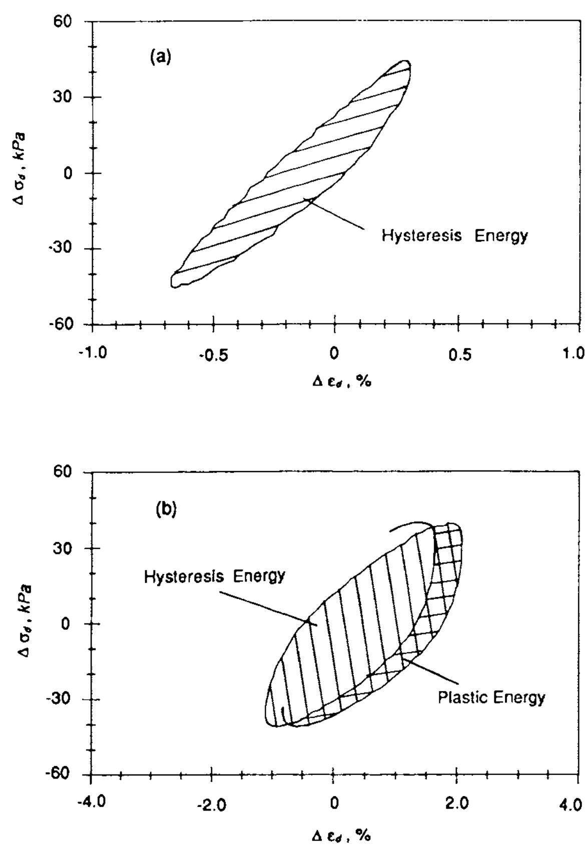


Figure 1. Hysteretic energy loss and plastic energy loss under cyclic loading

The energy dissipated through plastic deformation is the energy causing permanent deformation in the soil after the loading stops. This kind of energy is significant near the failure state of the soil when plastic deformation develops rapidly. The total dissipation energy is the sum of two kinds of energy as shown in Fig. 1(b). The relationship between the excess pore pressure (Δu) and the energy dissipated in the soil is given by:

$$\frac{\Delta u}{\sigma_o'} = \alpha W_N^{\beta} \tag{1}$$

where α and β are coefficients to be determined from the test results and W_N is the dimensionless energy term obtained by normalizing the dissipated energy per unit soil volume by the initial effective confining pressure, ratio of vertical to horizontal consolidation pressures, and relative density.

Liquefaction failure will occur when

$$\frac{\Delta u}{\sigma_o'} = \alpha W_N^{\beta} \geq 1.0 \tag{2}$$

For an actual site, the energy dissipated in the soil is proportional to the seismic energy arriving at the site and characteristics of the soil. The seismic energy arriving at a site depends on the magnitude of the earthquake M, the hypocentral central distance R, and the energy attenuation of bedrock material characterized by the attenuation coefficient B. Taking these factors into consideration, Law et al (1990) proposed the following criterion for liquefaction failure:

$$\frac{T(M,R)}{\eta_L(N_1)} \geq 1.0 \tag{3}$$

where

$\eta_L(N_1)$ = the liquefaction resistance function that characterizes the soil in terms of the corrected standard penetration resistance (N_1)

T (M, R) = seismic energy function given by:

$$T(M,R) = \frac{10^{1.5M}}{R^B} \tag{4}$$

As of this date, there is no direct measurement of the coefficient B. An assumption was made by Law et al (1990) that the energy attenuation is proportional to earthquake intensity attenuation for which some information is available. Thus a value of $B = 4.3$ has been taken, based on the work of Hasegawa et al (1981) for the American west coast, China and Japan, where highly fractured rock exists.

There are a number of coefficients in T and in η_L. They have been evaluated by means of a regression analysis of a database with 103 sand sites and 31 silty soil sites in North America, South America, China and Japan. The resulting criterion for liquefaction failure in sand is given by:

$$\frac{10^{1.5M}}{2.28\, N_1^{11.5} \times 10^{-10} R^{4.3}} \geq 1.0 \tag{5}$$

A NEW DATABASE

A new database was developed for the present study by adding information gathered by Liao and Whitman (1986). The new database now contains 354 different cases of which 244 are known to involve sandy soils (Table 1). All these sandy sites were analyzed on the basis of Expression (5) and the results are shown in Figure 2. The rate of success, i.e. correctly predicting if liquefaction occurred or not, is 82.4%. If only the liquefied sites were considered, the success rate is 97%. The results therefore suggest that Expression (5) has a larger applicability than for the original 103 cases.

Table 1. Summary of cases records involving clean sand in the new data base

No.	Earthquake	Year	Magnitude M	No. of liquefied sites	No. of non-liquefied sites
1	Niigata	1802	6.6	0	2
2	Niigata	1887	6.1	0	2
3	Mino-Owari	1891	8.4	6	0
4	Tokyo	1894	7.5	2	2
5	San Francisco	1906	8.3	5	1
6	Gono	1909	6.9	0	2
7	Kanto	1923	7.9	3	3
8	Santa Barbara	1926	6.3	1	0
9	Nishi-Saitama	1931	7.0	2	2
10	El Centro	1940	7.0	3	0
11	Tonankai	1944	8.3	2	0
12	Fukui	1948	7.2	5	1
13	San Francisco	1955	5.4	0	1
14	San Francisco	1957	5.3	1	31
15	Chile	1960	8.4	2	3
16	Alaska	1964	8.3	2	1
17	Niigata	1964	7.5	5	6
18	San Francisco	1965	4.9	0	1
19	Caracus	1967	6.3	1	0
20	Tokachi-Oki	1968	7.9	1	4
21	Saitama	1968	6.1	0	5
22	Santa Rosa	1969	5.7	0	2
23	Gediz, Turkey	1970	7.1	0	1
24	Haicheng, China	1975	7.3	3	2
25	Guatemala	1976	7.5	1	3
26	Tangshan, China	1976	7.8	37	28
27	Argentina	1977	7.4	4	2
28	Miyagiken-Oki	1978	6.7	1	20
29	Miyagiken-Oki	1978	7.4	13	8
30	Guerrero	1979	7.6	1	1
31	Montenegro	1979	6.9	1	0
32	Imperial Valley	1979	6.6	1	3
33	Westmoreland	1981	5.6	1	3
Total				104	140

It is also of interest to note in Figure 2 that liquefaction seldom occurs when the value of energy function $T(M, R)$ drops below 500. For this low value, either the earthquake magnitude is small or the hypocentral distance of the site is large. If cases with $T(M,R) \geq 500$ are considered, the success rate rises to 86.5%.

TEST PROGRAM AND RESULTS

The silt used in this study comes from near Armstrong, northern Ontario, Canada. It was obtained by screening out particles exceeding 0.075 mm in size. It is a non-plastic fine-grained soil with the grain size distribution as shown in Figure (3). This material is called clean silt in this paper. In real situations silt exists with different amounts of sand and/or clay. In this study, therefore, known amounts of sand or clay have been added to different samples for studying the influence of different soil contents. The sand used is a uniformly graded fine sand with grain size distribution also shown in Figure (3) while the clay is a highly plastic clay (P.I. = 40) from New Liskeard, Ontario.

Tests were conducted by means of a cyclic traixial apparatus. Samples were prepared using the moist tamping method in ap-

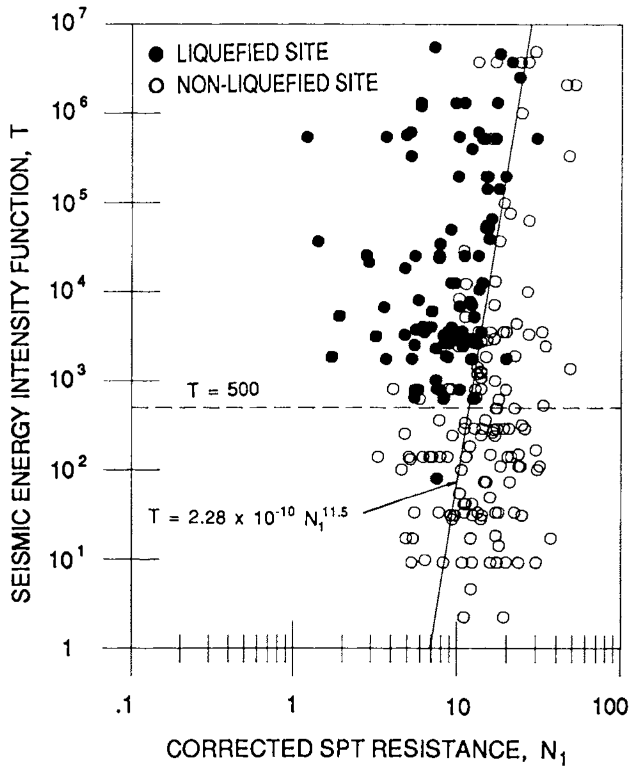


Figure 2. Correlation of Seismic Energy Intensity Function, T, with Corrected Standard Penetration Test (SPT) Resistance from the New Database

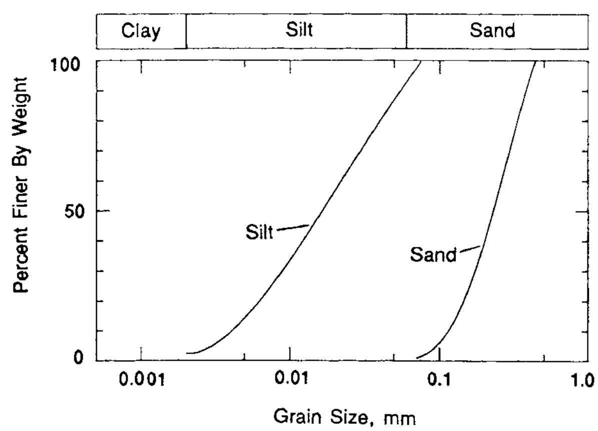


Figure 3. Grain Size Distribution of the Silt and Sand used in the Tests

propriate moulds. The saturation process consisted of passing carbon dioxide, followed by distilled water, through the sample. The initial effective confining pressure was applied in an isotropical manner. The cyclic loading was applied by an electro-pneumatic system operating at 1 Hz. During the undrained cyclic loading stage, readings were taken with an IBM PC AT compatible data acquisition system. The system uses an integrated circuit board which provides up to 8 analogue-to-digital channels. The computer was programmed to take 100 readings per second for each of the channels monitoring the axial loading, excess pore pressure and axial deformation. This large number of readings enable the precise calculation of the area of the hysteretic loop and other data analysis. Typical time response curves of cyclic stress, strain, excess pore pressure and dissipation energy are shown in Figure 4.

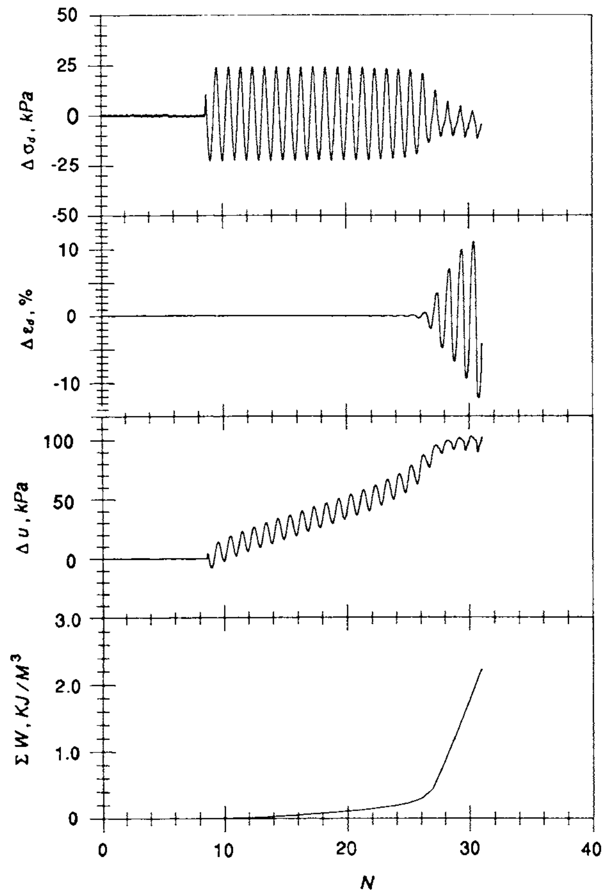
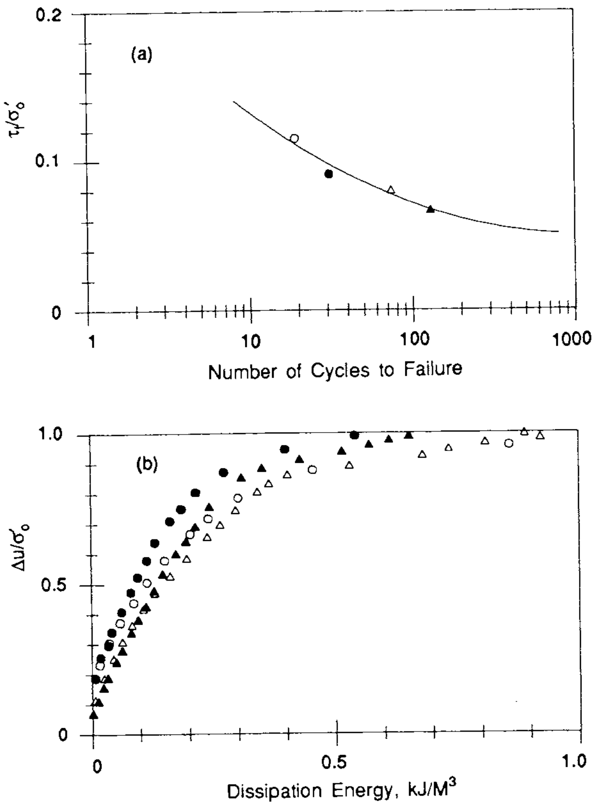


Figure 4. Typical Time Response Curves of Cyclic Stress ($\Delta\sigma_d$), Strain ($\Delta\varepsilon_d$), Pore Pressure (Δu), and Dissipation Energy (ΣW)

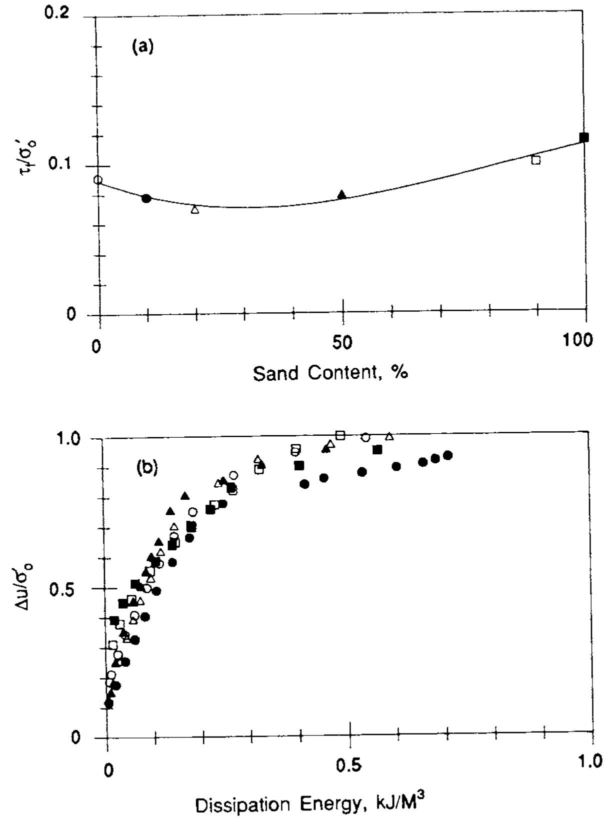
Results of tests on clean silt at the same void ratio of 0.72 and same consolidation pressure are shown in Figure 5. Figure 5(a) shows the conventional way of expressing liquefaction strength (τ_f) in terms of stress ratio $\left(\frac{\tau_f}{\sigma_o'}\right)$ as a function of the number of cycles to failure. As usual the higher the stress ratio, the fewer number of cycles would be required to reach liquefaction failure. On the other hand, if the test results are expressed in terms of excess pore pressure and the dissipation energy, a functional relationship appears to exist as shown in Figure 5(b). Similar relationships have also been observed for sand (Law et al., 1990) and for clay (Cao and Law, 1990).

A second series of tests have been conducted on silt samples mixed with different amounts of sand. These samples were cohesionless as no clay particles were added. They were all prepared at the same void ratio of 0.72. Consequently these samples were at different relative densities. No attempt was made to determine the relative density because such determination is not possible when the sand content is low and the silt content is high. The test results are shown in Figure 6. In Figure 6(a), the liquefaction resistance at 30-35 load



Note: The same test is represented by the same symbol in (a) & (b)

Figure 5. Results of Tests on Clean Silt



Note: The same test is represented by the same symbol in (a) & (b)

Figure 6. Results of Tests on Silt with Different Sand Contents

cycles to failure is plotted against sand content. There is a definite relationship between sand content and liquefaction resistance. A minimum strength is observed when the sand content is around 30 to 40%. Figure 6(b) shows the relationship between the pore pressure response $\left(\frac{\Delta u}{\sigma_o'}\right)$ and dissipation energy. At $\frac{\Delta u}{\sigma_o'}$ below 80%, the experimental scatter is relatively small and the pore pressure response is not dependent on the sand content. At $\frac{\Delta u}{\sigma_o'} > 80\%$, the sample rapidly approaches failure with a sharp increase in energy as shown in Figure 4. Hence the experimental scatter is large at this stage, yet the pore pressure is again not dependent on the sand content. Combining the observations from Figures 6(a) and 6(b), one may note that while the sand content affects the liquefaction strength it has no effect on the amount of energy needed to reach failure. This is because when the sample is stronger in strength, it is also higher in rigidity with smaller deformation upon loading. The energy dissipated in the sample is proportional to the stress applied and inversely proportional to the resulting deformation. When a sample is stronger, therefore, it does not necessarily require more energy to reach failure.

A third test series was conducted on silt samples mixed with different quantities of clay material. Again the samples were prepared at the same void ratio of 0.72. They are designated clayey silt samples here, to distinguish them from the cohesionless silt sample in the second test series. The results of this test series, summarized in Figure 7, shows that clayey silt behaves differently from cohesionless silt. The liquefaction strength increases fairly linearly with clay content (Figure 7(a)) and the pore pressure response is significantly influenced by the clay content. In fact as clay content increases, the conditions of failure change. At low clay content, failure fits the conventional definition of liquefaction failure with the excess pore pressure equal to the initial effective confining pressure $\left(\frac{\Delta u}{\sigma_o'} = 1.0\right)$. At higher clay contents (20% or more), failure is dominated by a shear failure with $\frac{\Delta u}{\sigma_o'} < 1.0$.

The energy dissipated in the soil to reach failure, W_F, is plotted against clay contents (R_c) in Figure 8. For liquefaction type of failure, the relationship can be approximated by a straight line:

$$W_{FC} = (1 + \xi * R_c)\, W_{FO} \qquad (6)$$

where

W_{FC}, W_{FO} = dissipation energy to failure for clay content = Rc and 0, respectively;

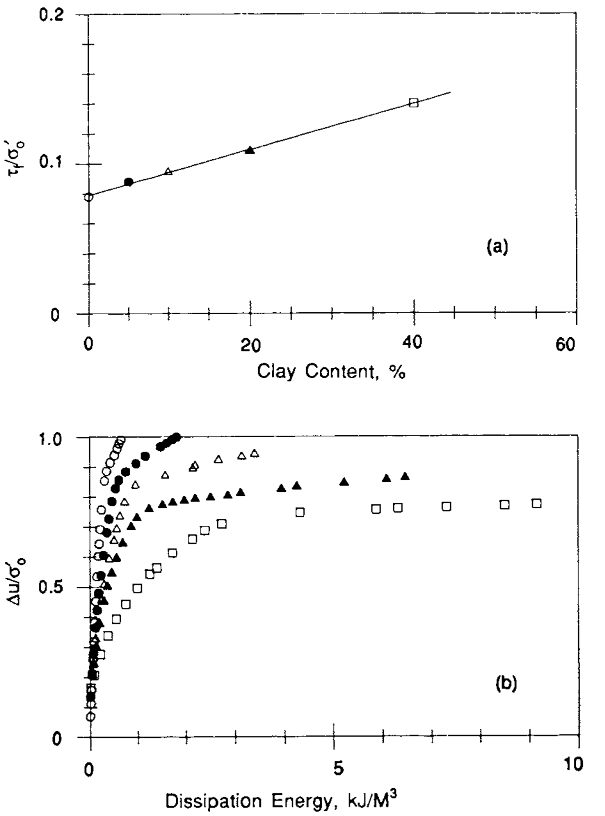
ξ = slope of the straight line (= 0.4 for this test series).

The application of the above test results is shown in the following.

LIQUEFACTION POTENTIAL

Cohesionless silty soil

There is one important implication from the observation that the



Note: The same test is represented by the same symbol in (a) & (b)

Figure 7. Results of Tests on Silt with Different Clay Contents

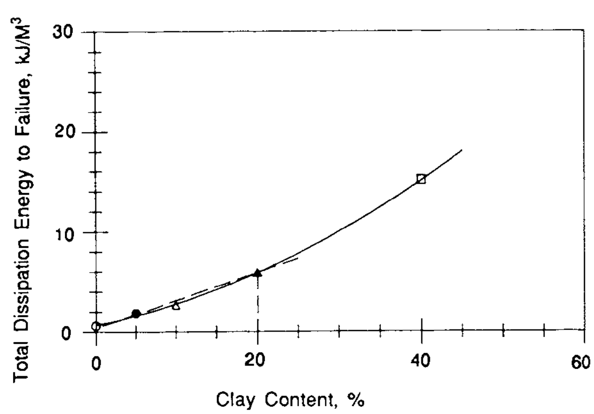


Figure 8. Total Dissipation Energy to Failure vs Clay Content for Clayey Silt Sample.

dissipation energy required to reach liquefaction failure for cohesionless silt is not significantly affected by the sand content. The implication is, since the soil is free of clay material, that the same liquefaction criterion (Expression (5)) can be used irrespective of the relative content of sand or silt. In other words, the liquefaction potential of sand, silty sand, sandy silt or silt can be examined by this single expression. Consequently, for the same seismic energy, i.e. constant M and R, sand or silt without clay content will have the same liquefaction potential if they have the same N_1. This finding is in apparent contrast to the observations of many researchers (e.g. Robertson and Campanella 1985, Seed and DeAlba 1986 and Shabita and Teparaksa 1988). Close examination of these observations show that most of the silty soils reported by these researchers actually contain clay material. As discussed later, this clay content enhances the liquefaction resistance of the soil and thus the silty soil with the same N_1 as sand will have a lower potential to liquefy.

In order to support the finding of the present study, the new database is consulted. There are 123 silty sites recorded in the database, but only 11 of them involve clean silt or cohesionless silty sand. The occurrence or non-occurrence of liquefaction failure of these 11 records was assessed based on Expression (5) and the results are summarized in Table 2. The success rate of this exercise is 81.8%, showing that Expression (5) originally developped for sand is also acceptable for cohesionless silty soil.

Table 2. Summary of case records involving cohesionless silty soils

No.	Earthquake	Year	M	R	D50	N1	Liquefied ?
1	Alaska	1964	8.3	99	.35	9.4	Yes
2				99	.35	5.8	Yes
3	Argentina	1977	7.4	80.6	.11	14.7	Yes
4				80.6	.10	5.2	Yes
5	Izu-Oshima	1978	7.0	40.3	.25	1.2	Yes
6	Miyagiken-Oki	1978	6.7	129.7	.25	19.0	No
7	Miyagiken-Oki	1978	7.4	118.8	.15	19.0	No
8	Imperial Valley	1979	6.6	14.1	.06	39.2	No
9				51.0	.03	8.0	No
10	Westmoreland	1981	5.6	8.6	.10	8.0	Yes
11				50.5		39.2	No

<u>Clayey Silt</u>

The linear relationship (Equation (6)) between dissipation energy to reach liquefaction failure and clay contents can be applied directly to Expression (5) as follows:

$$\frac{T(M,R)}{(1+\beta * R_C)\,\eta_L(N_1)} \geq 1 \qquad (7)$$

$T(M, R)$ in Expression (7) is the earthquake energy intensity for a given site. Expression (7) states that more energy, proportional to $(1+\beta * R_C)$, is required to liquefy a clayey soil with clay content R_C. At $R_C = 0$, Expression (7) is reduced to Expression (5) for application to cohesionless silty soil. Expression (7) can also be interpreted as for the same energy arriving at a site, a clayey soil with N_1 will be stronger than a cohesionless silty soil with the same N_1.

Expression (7) can be rewritten as

$$\frac{10^{1.5M}}{2.28 \times 10^{-10} \times (1+0.4\,R_C) \times N_1^{11.5} \times R^{4.3}} \geq 1 \qquad (8)$$

One can use an equivalent standard penetration resistance N_{1C} in order to reduce Expression (8) into the same form as Expression (5). N_{1C} can be defined by:

$$N_{1C} = (1 + 0.4\,R_C)^{1/11.5}\,N_1 \qquad (9)$$

Hence Expression (8) becomes:

$$\frac{10^{1.5M}}{2.28 \times 10^{-10} \times N_{1C}^{11.5}\,R^{4.3}} \geq 1 \qquad (10)$$

which is the same form as Expression (5) for use with cohesionless sandy or silty soils. Typical values of N_{1C} expressed in terms of N_1 are shown in Table (3).

Table 3. Typical N_{1C} / N_C values with clay contents

Clay Content %	0	5	10	15	20
N_{1C} / N_1	1.00	1.10	1.15	1.18	1.21

Assessment of relevant case records in the new database has been conducted with Expression (10). There are 66 sites with clayey silt as shown in Table 4 and the results are illustrated in Figure 9. The success rate of the assessment is 90.9%, indicating the high applicability of Expression (10).

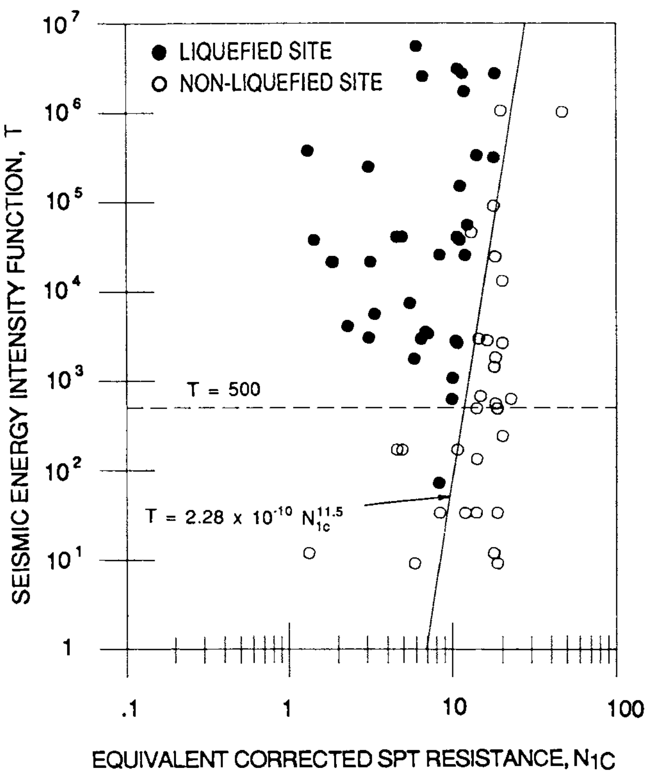


Figure 9. Correlation of Seismic Energy Intensity Function, T, with Equivalent Corrected SPT Resistance, N_{1c}, for Clayey Silt Sites

SUMMARY AND CONCLUSIONS

An energy approach originally developed for sandy soils (Law et al. 1990) has been extended to silt with or without clay material. The original data have been expanded with the addition of new cases from the work of Liao and Whitman (1986). Three series of tests were carried out to examine the effects of sand content and clay content in the silt. The analysis of the test results in light of the information in the new database leads to the following conclusions:

(1) The information in the new database strongly supports the criterion for liquefaction failure of sandy soils originally developed by Law et al (1990) and represented by Expression (5) in this paper.

(2) For a cohesionless soil, i.e. one without clay content, Expression (5) can be applied as is irrespective of the sand or silt contents.

(3) The clay content in silt has a beneficial effect on the liquefaction resistance. The higher the clay content, the stronger is the liquefaction resistance or the higher is the energy required to reach failure.

(4) A new formula for assessing liquefaction potential for clayey silt (Expression (10)) is suggested. This formula is strongly supported by the information in the new database.

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Table 4. Summary of case records involving granular soils with known clay content

No.	Earthquake	Year	Magnitude M	Hypocentral Distance km	N_1	Liquefied ?	Clay content %
1	Kanto	1923	7.9	82.4	2.0	Yes	9
2	Tonankai	1944	8.3	60.8	1.6	Yes	10
3				60.8	1.6	Yes	12
4				60.8	2.9	Yes	4
5	Niigata	1964	7.5	61.9	6.3	Yes	4
6	Tokachi-Oki	1968	7.9	211.0	7.4	Yes	6
7	Miyagiken-Oki	1978	6.7	96.0	10.6	No	7
8				96.0	12.3	No	8
9				96.0	17.1	No	4
10				96.0	7.7	No	4
11				129.7	5.3	No	5
12				129.7	17.4	No	3
13	Miyagiken-Oki	1978	7.4	90.1	12.3	No	8
14				90.1	17.1	No	4
15				90.1	17.4	No	3
16				36.1	10.6	Yes	7
17				67.1	5.3	Yes	5
18				36.1	7.7	Yes	4
19	Imperial Valley	1979	6.6	14.1	17.2	No	1
20				55.9	19.5	No	1
21				60.8	3.9	No	12
22				60.8	4.3	No	9
23				60.8	9.4	No	9
24				46.1	17.6	No	1
25				10.1	1.2	Yes	5
26				10.6	16.4	Yes	4
27				11.2	2.8	Yes	5
28				10.4	12.2	Yes	10
29	Westmoreland	1981	5.6	50.5	1.2	No	5
30				50.5	17.2	No	1
31				16.6	16.4	No	4
32				9.9	19.5	No	1
33				15.7	17.6	No	1
34				28.9	12.2	No	10
35				7.6	3.9	Yes	12
36				7.6	4.3	Yes	9
37				7.6	9.4	Yes	9
38				13.9	2.8	Yes	5
39	Tangshan	1976	7.8	82.7	16.9	No	10 *
40				81.9	15	No	10 *
41				84.0	20.9	No	10 *
42				50.2	18.9	No	10 *
43				43.3	13.6	No	10 *
44				115.5	15.4	No	10 *
45				117.5	23.5	No	10 *
46				21.1	48.9	No	10 *
47				20.9	20.5	No	10 *
48				70.9	3.5	Yes	10 *
49				84.0	11.2	Yes	10 *
50				79.4	7.4	Yes	10 *
51				82.9	11.0	Yes	10 *
52				81.9	6.7	Yes	10 *
53				103.6	10.5	Yes	10 *
54				117.5	10.4	Yes	10 *
55				66.2	5.7	Yes	10 *
56				45.4	11.6	Yes	10 *
57				41.5	12.9	Yes	10 *
58				32.9	11.7	Yes	10 *
59				45.4	1.5	Yes	10 *
60				16.3	11.3	Yes	10 *
61				18.6	12.4	Yes	10 *
62				16.7	12.1	Yes	10 *
63				16.7	18.9	Yes	10 *
64				17.0	6.9	Yes	10 *
65				14.2	6.3	Yes	10 *

* Estimated based on Zhao and Guo (1985) and from information by J.G. Wang (personal communication, 1990)