A PROMISING TECHNIQUE FOR EVALUATING LIQUEFACTION POTENTIAL, BASED ON A COMPOSITE ANALYSIS OF STATIC AND DYNAMIC CONE PENETRATION TEST RESULTS

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Abstract

There are two recognized methods to evaluate the potential of liquefaction of sandy ground: (1) By determining the dynamic strength characteristics of soil from dynamic tests in the laboratory of undisturbed samples, and (2) the simplified method to determine indirectly the dynamic strength characteristics of soil from N value and grain size characteristics.

Direct determination of dynamic strength from dynamic soil tests in the laboratory has solid theoretical grounding and it can safely be considered reliable. However, it is very difficult to take undisturbed samples from loose, sandy ground. It not only requires considerable expense, but complex equipment and considerable skill on the part of the operator. If effective quality control is not applied throughout the entire process, the reliability of the results will often be questionable. For the simplified method, N value is easily obtained from SPT, but it is not so easy to obtain the other necessary parameters, i.e., average grain size and unit weight from in situ tests only. In addition, the method for determining N value is not internationally standardized, which poses a problem with calibration.

We investigated effective in situ test methods for assessing liquefaction potential by conducting a number of different kinds of dynamic and static penetration tests at four sites in ground in which liquefaction clearly had taken place during past earthquakes and at two sites where liquefaction did not take place during earthquakes, On the basis of these, the following was concluded:

(1) There were marked difference in the results from dynamic and static penetration tests for loose sand apt to undergo liquefaction. Briefly, in sand that has a high liquefaction potential, static penetration resistance is relatively large, while dynamic penetration resistance is relatively small.

(2) It is possible to obtain a comparison of dynamic and static penetration resistance values by conducting both static and dynamic penetration tests at adjacent points. We think it is possible to evaluate liquefaction potential by using this as an index.

(3) A considerable amount of research on the use of in situ tests for evaluating liquefaction potential exists, but it appears difficult to develop an effective evaluation method involving testing using only one kind of in situ test. Rather, it seems very promising that evaluation of liquefaction potential by conducting different kinds of in situ tests at the same site, using a compound analysis, will constitute an effective method.

This paper proposes a new method to evaluate liquefaction potential using as a parameter the ratio of static to dynamic penetration resistance. This ratio is determined by using static cone penetration resistance value, determined from CPT and dynamic cone penetration resistance, determined from the Swedish dynamic cone penetration test (Automatic Ram Sounding).

List of Abbreviations

SPT: Standard penetration test
SDPT: Swedish dynamic cone penetration test (Automatic Ram Sounding)
RTRI-DPT: Japane National Railways Technology Research Institute dynamic cone penetration test
CPT: Dutch double tube cone penetration test
CUPT: Piezo (Pore pressure) cone penetration test
VCPT: Vibratory cone penetration test
WST: Swedish weight sounding test
N: N-value, from SPT
N_J: Measured N value, obtained by using the JIS sampler
N_{US}: Measured N value, obtained by using the U.S. sampler
Ndm: Measured blow count, as obtained by SDPT
Nd: Point penetration resistance value, as obtained by SDPT
$\overline{Nd}$: Average Nd value per 1 m interval, as obtained by SDPT
Nd': Measured blow count, as obtained by RTRI-DPT
qc: Penteration resistance value, as obtained by CPT and CUPT
fs: Side friction of cone in CPT or CUPT
$\overline{qc}$: Average qc value per 1 m interval, as obtained by CUPT
ud: Pore water pressure, measured by CUPT, during penetration
Qcs: Total penetration resistance value during static penetration in VCPT
Qcd: Total penetration resistance value during dynamic penetration in VCPT
Nsw: Number of half turns per 1 m of penetration, as obtained by WST
D_{50}: 50% grain size (mm)
D_{10}: 10% grain size (mm)
F_L: Liquefaction resistance factor

1 A BRIEF REVIEW OF RESEARCH ON EVALUATION OF LIQUEFACTION POTENTIAL USING IN SITU TEST RESULTS

Research on evaluation of liquefaction potential using in situ test results began to progress quickly after information on damage resulting from liquefaction in the 1964 Niigata Earthquake became available. At the time, SPT had already come into general use as a site investigation method. Studies were carried out comparing N value to damage. Many criteria have been proposed for using N value as the basis for evaluating liquefaction potential. In some, in saturated sand layers located below the underground water level, ground in which average grain size (D_{50}) is 0.05 mm to 0.5 mm, and N value is 5 or below, there is a high probability of liquefaction. This fact was made manifest from results from the Niigata earthquake.

Following this, determination of the relationship between dynamic strength of sand and dynamic shear stress from earthquakes came into general use as a method for assessing liquefaction potential. It was proposed that N value, average grain size and effective overburden stress be used to determine dynamic strength of sand. The proposals by B. Seed (1981) and Iwasaki and Tatsuoka (1978) are representative of these. Since they are simpler than the method of laboratory dynamic tests using undisturbed sand samples to determine dynamic strength, they are known as simplified methods.

Because N value from SPT came to be used as an effective parameter for estimating dynamic strength of soil, research began to be conducted on the SPT itself and on the quality of the N value obtained by the SPT. In Japan, the research by Uto, Fuyuki and, in the U. S., by Kovacs and Schmertmann, is most representative among the many research papers that have been contributed. A paper by Schmertmann (1977), demonstrated the following:

(1) N value varies according to the test procedure.
(2) N value varies according to the test equipment.
(3) N value varies according to the method of preparation of the borehole for the SPT.

These results do not of themselves call into question the position that SPT occupies in evaluating liquefaction potential, but at least it does demonstrate that even if the international standardization of the SPT method is unattainable, the establishment of a calibration method is necessary.

The European subcommittee on penetration testing considered this problem from 1975, and in 1977, the Subcommittee published recommendations for a European Standard for the SPT at the Ninth ISSMFE, held in Tokyo. In 1982, the decision was taken at the Second European Symposium on Penetration Testing, held in Amsterdam, to form an ISSMFE Technical Committee on Penetration Testing, and the Technical Committee was assigned the task of completing International Reference Test Procedures by 1985.

Of all the sounding methods, SPT is the only penetration testing method that requires preboring. Since the quality of preboring has a great deal of bearing on the quality of the N value obtained, much research is being conducted on other in situ test methods for evaluating liquefaction potential, and in particular, CPT. In 1975, Wissa developed a piezo cone capable of measuring pore water pressure. Penetration of the cone produced additional shear stress in the soil, which then resulted in negative excess pore water pressure in dense sand, while there was positive excess pore water pressure in loose sand. Since this became clear, a new device capable of measuring pore water pressure in addition to cone resistance was developed. It is called piezo cone or pore pressure cone (CUPT). The development of this apparatus did much to promote research on the evaluation of liquefaction potential. Because the CUPT is capable of continually determining qc and mesuring pore water pressure at the same time, it can be effective in distinguishing soil layers, with special reference to dilatancy characteristics of soil. Unfortunately, however, in sandy ground apt to liquefy, grain size is very uniform, and because permeability coefficient is high, measured excess pore water pressure values are very small, and are controlled by factors other than relative density. Thus, in practice, the research results show that this CUPT apparatus is not an effective method of evaluating liquefaction potential in sandy soil. (For example, see Norton, 1983).

Before CPT can be introduced into the system of methods of determining liquefaction potential, a number of problems must first be solved. The biggest of these problems is that CPT determines static strength coefficient of the soil. Liquefaction strengsh is a dynamic value, and there are basic problems involved with deducing this value from CPT results alone, because qc value represents only staic strength coefficient of the soil.

Sasaki has developed a new cone penetration test using a cone with a new shape and containing electric vibrator, to conduct CPT. With this apparatus, vibration is imparted in the horizontal direction and penetration resistance is measured as the cone penetrates the ground. In addition, power to the vibrator may be turned off and static penetration measured, so the apparatus can be used for both static and dynamic penetration values. Sasaki determined static and dynamic penetration values from two adjacent sites using this apparatus by analyzing both these values, and he found that the ratio of static to dynamic cone penetration resistance is useful in evaluating liquefaction potential.

2 TEST SITES

2.1 Selection of Test Sites

As shown in Figure 1, three sites in the Akita region and three sites in the Niigata region were selected for comparative testing.

Figure 2(A) shows the locations of the comparative test sites in the Akita area. At the time of the 1983 Nihonkai–Chubu earthquake, the Akita region underwent dramatic damage as a result of liquefaction of sandy ground. Comparative testing was conducted at 2 sites where marked damage resulting from liquefaction was observed within a dike surrounding polder land

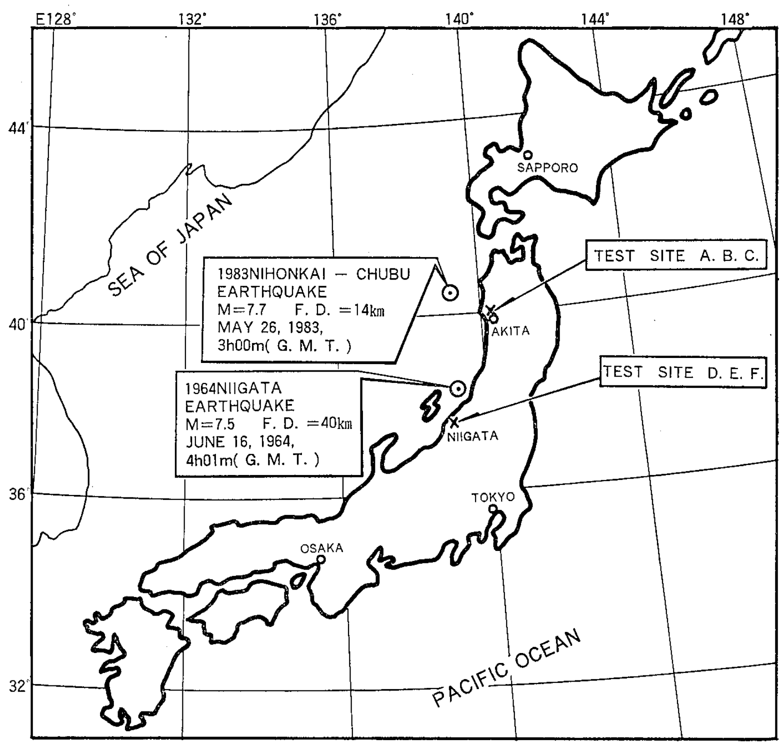
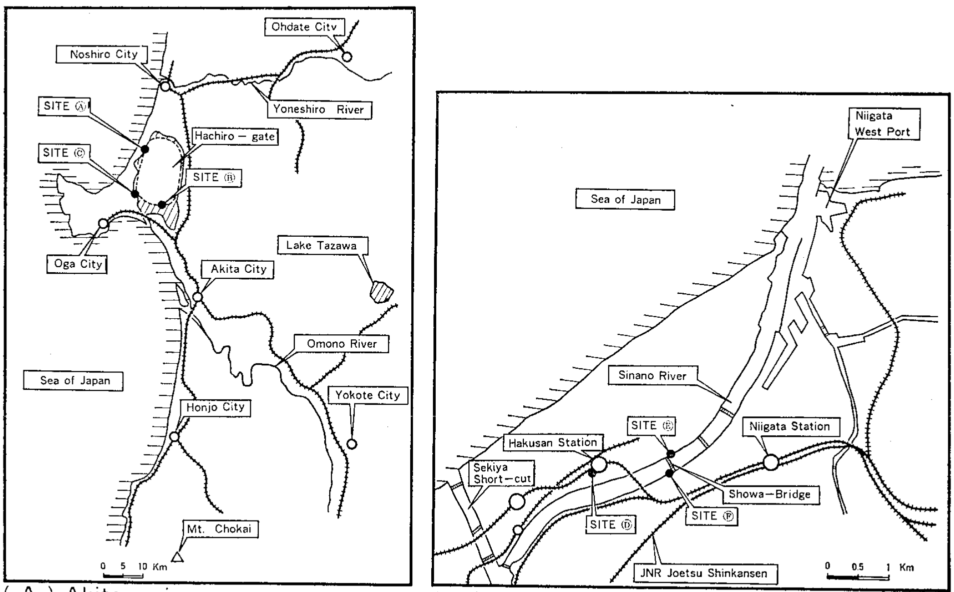


Figure 1 : Location of test sites and the earthquake epicenters



(A) Akita region

(B) Niigata region

Figure 2 : Guide map of Akita and Niigata region

in Hachiro-gata (Sites A and B) and one site (Site C) where damage from liquefaction was not observed.

Figure 2(B) shows the locations of the test sites in the Niigata area. The same as with the Akita region, the 1964 Niigata earthquake caused liquefaction, resulting in a great deal of damage. Comparative testing was conducted at two sites (Sites D and E) in which a representative amount of damage was observed, and one site (Site F) where there was almost no damage. Following are descriptions of conditions at each site.

(a) Akita Area

Site A: This site was located on an access road to the Gomyoko Bridge, along the enclosing embankment dike in the Hachiro-gata area. There was a great deal of settlement of the embankment, and flow slide type damage. Also, numerous cracks appeared. Along the toe of the

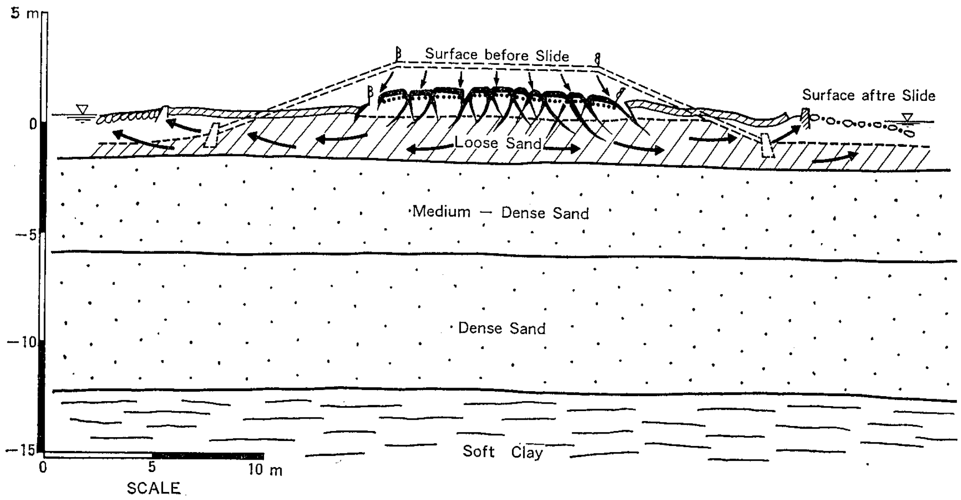


Figure 3 : Flow slide of road embankment at Site A

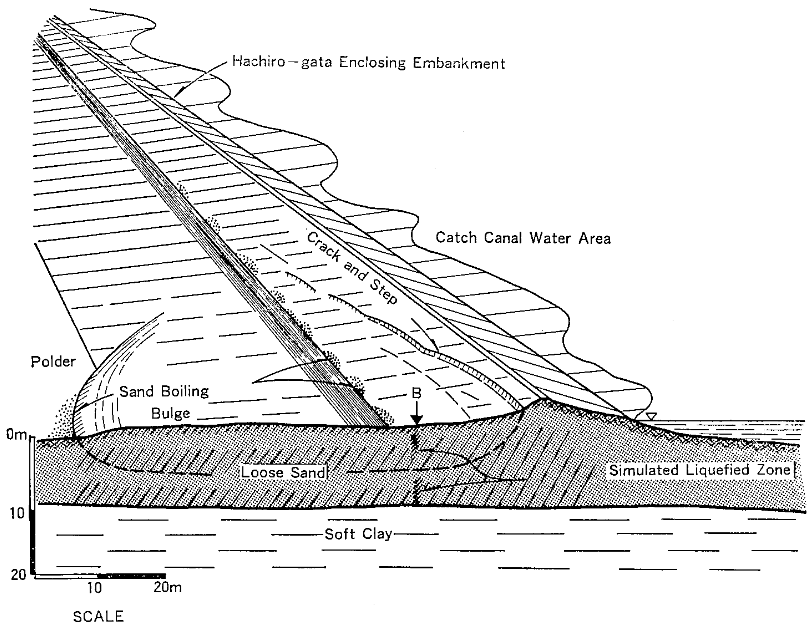


Figure 4 : Slope failure of the Hachiro-gata enclosing embankment at Site B

slope, boiling of sand occurred. Differential settlement of the embankment for the bridge structure was 50 to 100 cm. Figure 3 is a sketch of the damage immediately after the disaster. It also shows subsoil conditions from the investigation, and deformation of an access road, produced by the earthquake. Testing was conducted around the toe of the slope of the road.

Site B: Site B is located at the front of the enclosing dike in Hachiro-gata. Here, a large scale slope failure occurred, as a result of reduction in shear strength of the foundation layer, accompanying liquefaction. Figure 4 is a sketch showing the changes thought to be wrought by the earthquake in the dike. The breadth of the slide extends from 30 to 40 m. At the top of the embankment dike, approximately 60 cm of settlement occurred. The maximum dislocation at the scarp was 1.02 m. Boiling of sand occurred along the outer edge of the road, around the inside of the dike. As Figure 4 shows, testing was conducted in the area of the toe of the slope on the inside of the dike.

Site C: Damage resulting from liquefaction was not seen here. Testing was conducted at the top of the embankment.

(b) Niigata Area

Site D: This site is located in Kawagishi-cho. It includes the place where the well-known disaster occurred; bearing capacity of the foundation soil was lost as a result of liquefaction, and several four story apartment buildings either fell over or almost fell over. Kawagishi-cho is located along an old river bed of the Shinano River. It consists of ground that was reclaimed from old river bed relatively recently (from 1945—55). It is not recorded how reclamation work was carried out, but it seems likely that adequated measures were not taken for compacting the ground, with sandy soil simply being dumped in place. (Ishihara et al, 1981).

Site E: This site located on the embankment around the left bank of the Showa Bridge. The same as the Kawagishi-cho area, it is located on an old river bed, in which there was

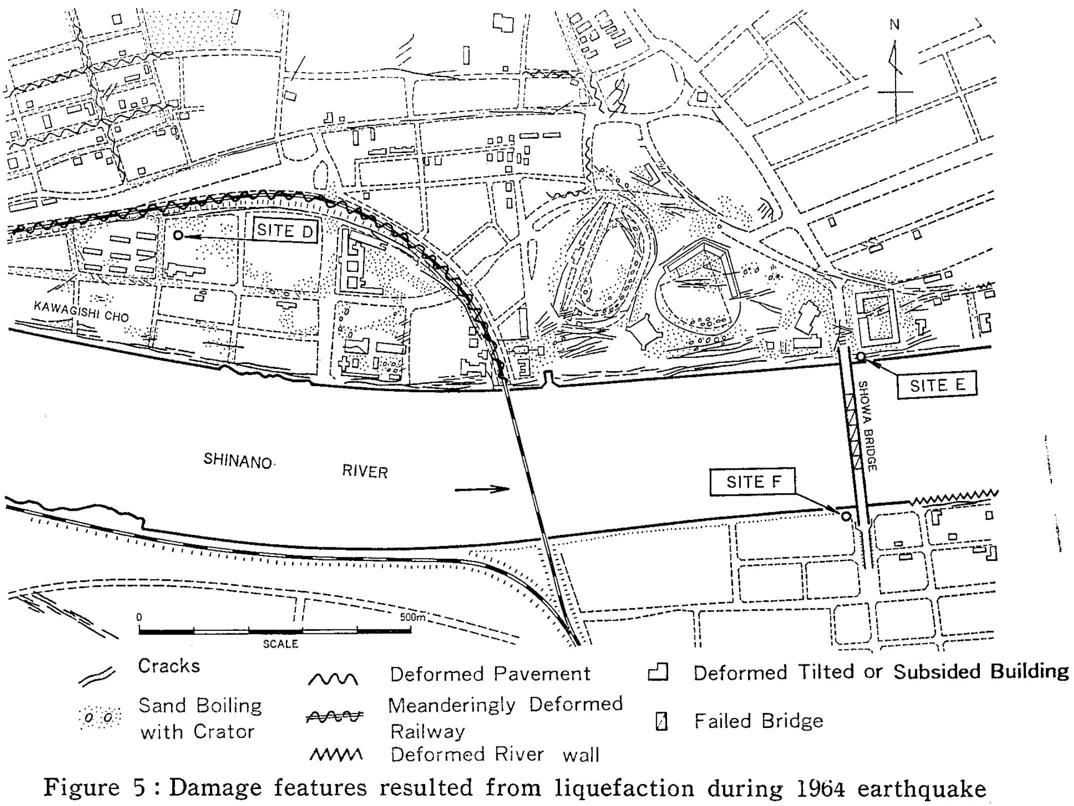


Figure 5 : Damage features resulted from liquefaction during 1964 earthquake (Niigata Univ. and Fukada Geological Institute, 1964)

great deal of damage from liquefaction. There was extensive cracking, sand boiling, craters on the surface and evidevce of deformation of man-made structures. In addition, there was damage from falling girders from the bridge that was built several months before the earthquake.

Site F: This site is located on the embankment along the right bank of the Shinano River, close to the Showa Bridge. Unlike the left banke, there was almost no damage from liquefaction in this area.

OYO Corporation had sent an investigation team to survey the damage from the Niigata earthquake immediately afer it occurred. The survey results were compiled on a map of the damage, on a scale of 1/3, 000. This map of damage was published by Niigata University and Fukada Geological Institute (1964). Figure 5 was prepared from the map, showing the damage features of the surrounding area of Site D, E and F.

OYO Corporation also sent a survey team to assess damage in the Hachiro-gata area as well, following the Nihonkai-Chubu earthquake there. A great many investigations were carried out in connection with repair work. Figures 3—4, which show the damage situation at Sites A and B, were prepared by compiling the results of the investigations and testing results obtained at this time.

2.2 Records of the Earthquake

The 1983 Nihonkai-Chubu earthquake and the 1964 Niigata earthquake both had their epicenters in the Sea of Japan area (Figure 1). Their magnitudes were 7.7 and 7.5, respectively. According to the records of strong motion seismographs, acceleration was 168 gal in the Akita area (near Sites A, B and C) and 157 gal at the Niigata area (near Sites D, E and F). Thus, magnitude of the earthquake and acceleration intensities in the ground during the earthquakes were similar for each of the sites.

Records of the earthquake and simplified summaries of damage at each of the sites are shown in Table 1.

Table 1 : Earthquake records and damage features of the site by the earthquakes

Earthquake records	Site	Damage features of the site resulted by the earthquake
1983 Nihonkai—Chubu Earthquake Date of occurence : May 26. 1983. 3h00m(G. M. T.) Magnitude : 7.7 Epicenter : N40°21', E139°05' Focal depth : 14 km Distance from epicenter to the test sites : 90 ± 5 km Surface acceleration recorded : 168 gal	A	Heavily damaged : Flow slide of embankment with fractures
	B	Heavily damaged : Slope failure of enclosing embankment sand boilings
	C	No damage
1964 Niigata Earthquake Date of occurence : June 16. 1964. 4h01m(G. M. T.) Magnitude : 7.5 Epicenter : N38°21', E139°11' Focal depth : 40 km Distance from epicenter to the test sites : 100 ± 3 km Surface acceleration recorded : 157 gal	D	Heavily damaged : Overturn of 4 stories apartment buildings, sand boilings with crators
	E	Heavily damaged : Sand boilings, deformation of buildings, settlement and failure of embankment
	F	No damage

3 SOIL CONDITIONS OF THE TEST SITES AND PRESUMED LIQUEFIED ZONES

3.1 Soil Conditions of the Test Sites

As will be explained below, eight types of penetration tests, including SPT, were conducted at each test site. Before giving the results of these comparative tests, let us first give a brief description of the soil conditions of each site and attempt to deduce the depth distribution of soil layers in which liquefaction occurred.

Boreholes were drilled at each site and SPT conducted at 1 m intervals. Grain size analysis was conducted on all of the samples obtained by the SPT sampler. In addition, the liquid limit

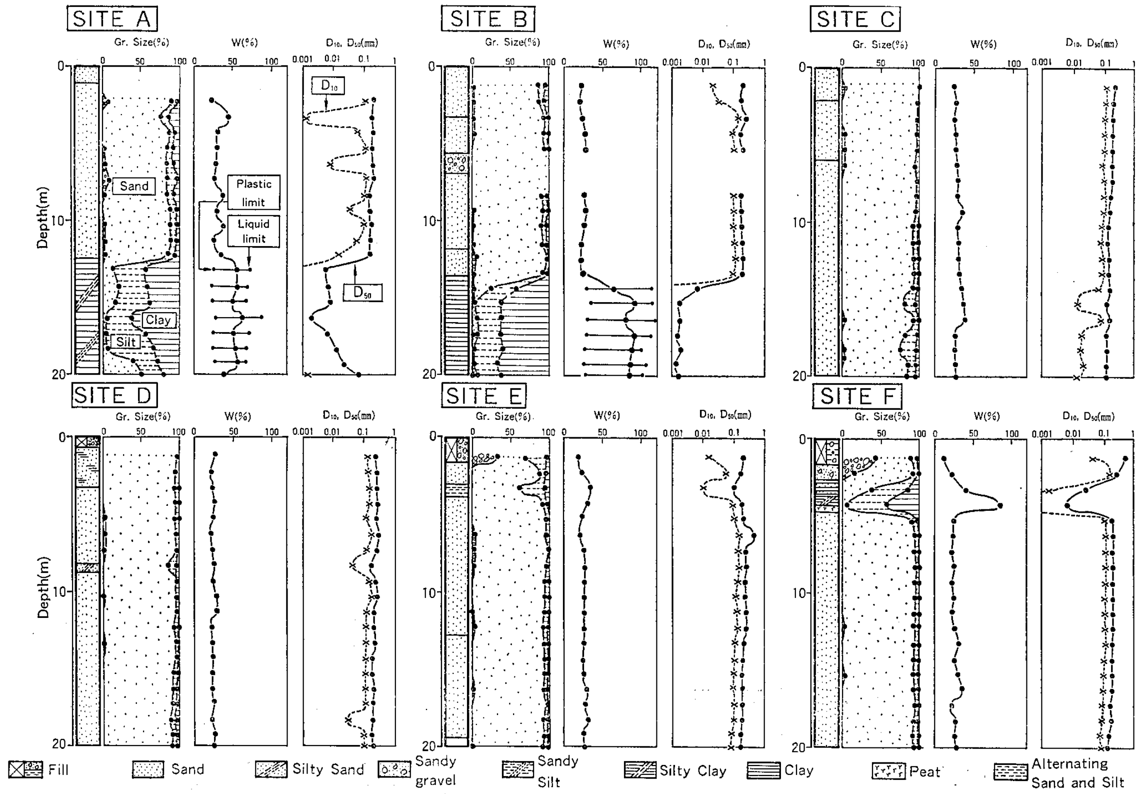


Figure 6 : Soil properties and soil logs at test sites

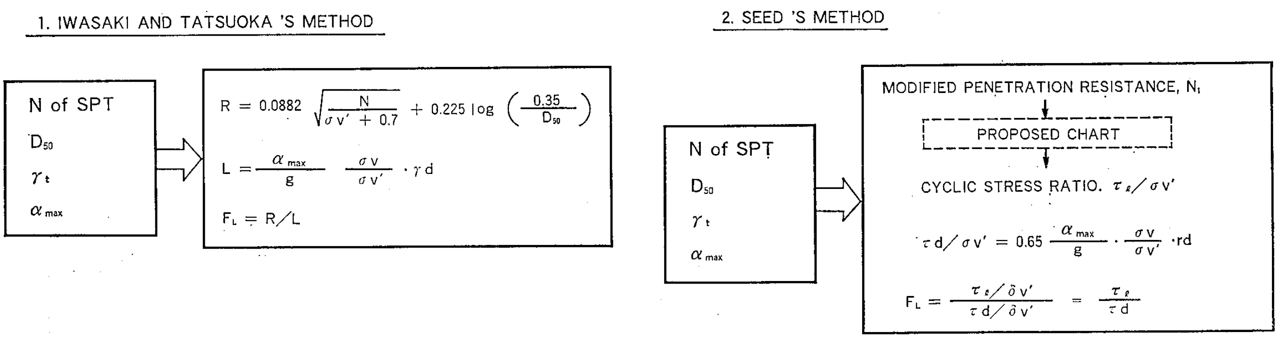


Figure 7 : Simplified method for evaluating liquefaction potential

and plastic limit tests were conducted on clay samples. Figure 6 is a compilation of these results, in the form of colums, and grain size analysis results and moisture content distribution. In general, the subsoil consists of fine sand, with average grain sizes of 0.1 to 0.3 mm and uniformity coefficient of 2 to 4.

3.2 Presumed Liquefaction Zones

As mentioned above, at sites A, B, D and E, a dramatic amount of damage due to liquefaction occured. However, damage is a phenomenon that is observed on the ground surface. It is not clear what part of the subsoil underwent liquefaction. Accordingly, the recognized simplified method for evaluating liquefaction potential was applied to the SPT results to aid our evaluation of the extent of liquefaction.

The methods used were Seed's method (1981) and that of Iwasaki and Tatsuoka (1978). We will not here go into the details of these methods, but each requires such soil Parameters as unit weight, N value, average grain size, effective overburden stress and acceleration intensity at the surface. Figure 7 is a simplified version of the calculation procedure. Each determines the unique dynamic strength value of the ground and dynamic shear stress produced during earthquakes, from which surface acceleration is in turn calculated. Finally, liquefaction resistance factor is determined from the ratio of the two values. In this paper, this factor is referred to as F_L:

$$F_L = \frac{\dfrac{\tau_\ell}{\sigma_v'}}{\dfrac{\tau_d}{\sigma_v'}} = \frac{\tau_\ell}{\tau_d}$$

where F_L: Liquefaction resistance factor

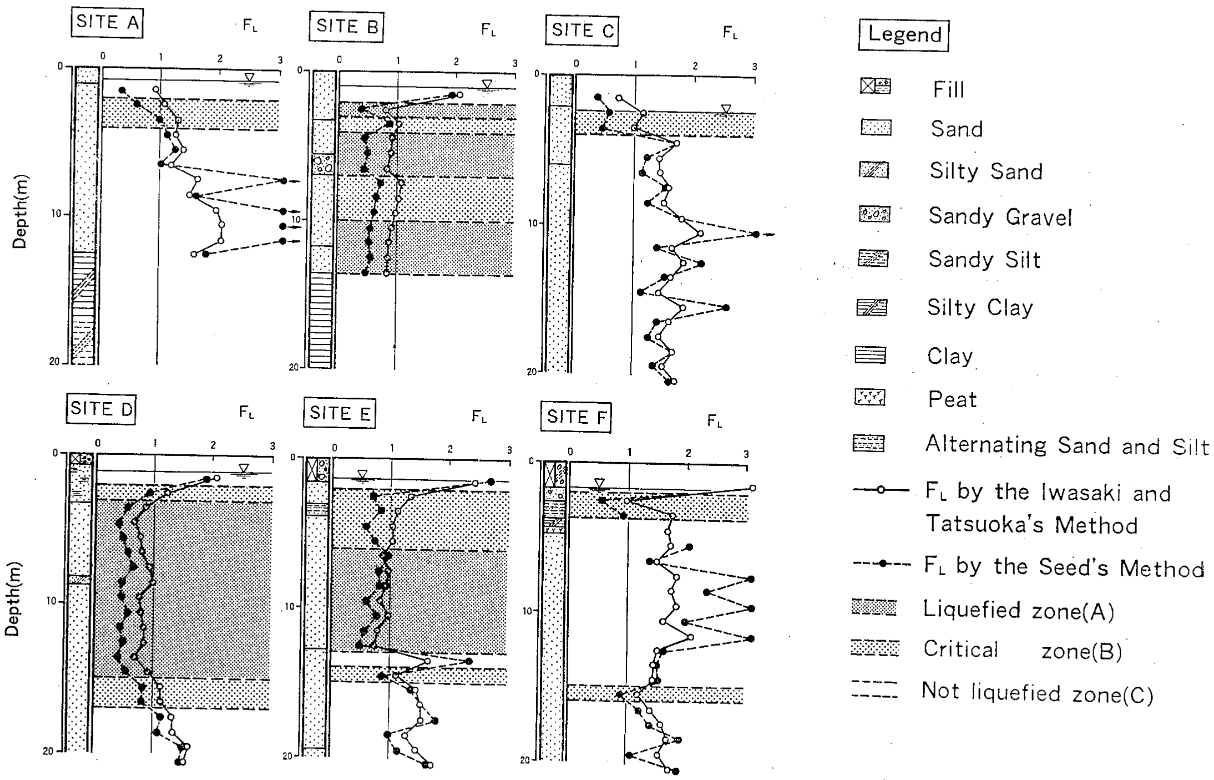


Figure 8 : Comparison of F_L distribution by the Iwasaki and Tatsuoka method and by the Seed's method

τ_l/σ_v': Cyclic stress ratio for liquefaction of a given soil type

τ_d/σ_v': Cyclic stress ratio resulting from calcuated earthquake shaking

Figure 8 shows the liquefaction resistance ratios determined in this way for each site. These results conform well with the actual state of damage due to liquefaction observed on the surface. The figure divides the ground into three types, where it was deduced that liquefaction occurred (A), where there was a possibility that weak liquefaction occrred (B), and where liquefaction did not occur (C).

Soil located above the underground water level and clay layers were assumed not to have liquefied. In this way, zoning was conducted for each of the sites.

For sites B, D and E, where marked damage was observed, F_L shows low values of less than 1. It may be concluded that the supposed liquefaction zone and that occurred at each site agree well.

For site A, where flow slide of the road embankment was observed, F_L shows low values of less than 1 only for a limited zone near the surface. All other points show a higher liquefaction resistance factor than 1. Careful consideration of the site has convinced the authors that the flow slide resulted from liquefaction of the man-made embankment, and not from liquefaction of the foundation subsoil. (See Figure 3.)

For sites C and F, where no damage was observed, F_L shows higher values than 1. This finding confirms the reliability of F_L

Generally speaking, we can see good agreement between hypothesized liquefaction potential and the actually observed damage at each site.

4 COMPARATIVE TESTS

4.1 Types of Tests Conducted

The following is a list of the types of sounding used in the comparative tests.

(a) Standard penetration test (SPT)
(b) Swedish dynamic cone penetration test (SDPT)
(c) Japan National Railways Technology Research Institute dynamic cone penetration test (RTRI-DPT)
(d) Dutch cone penetration test (CPT)
(e) Piezo cone penetration test (CUPT)
(f) Vibratory cone penetration test (VCPT), with and without vibration
(g) Swedish weight sounding test (WST)

CPT and WST are well known sounding methods. Both are standardized by the Japanese Industrial Standards (JIS). There is no significant difference between the JIS standards and the standards established by the Delft Soil Mechanics Laboratry for CPT and by the Swedish Geotechnical Society for the WST. Therefore, we will dispense with explaining them here. In addition, a detailed account of VCPT is given by Sasaki et al (1984), and, therefore, also will not be explained here. Following are brief explanatoin of other types of penetration tests.

i) SPT

Drilling is conducted using a rotary hand feed type drilling machine. Bentonite mud water and casing provide for the stability of the borehole wall. Cuttings are removed from the borehole by circulating bentonite mud water, which has a specific density of approximately 1.05g/cm^3. To allow enough space between the bore hole wall and the SPT sampler, borehole of 66 to 86 mm diameter is prepared.

Before the location of the ground water level is confirmed, waterless drilling is conducted. After the groundwater level is confirmed, the borehole is filled with bentonite mud and the mud water level in the borehole kept higher than the ground water level throughout all drilling and testing procedures.

In principle, SPT was carried out accrding to JIS A 1219, but, as there were some variations on fine points, an outline of the equipment and method used is given below.

Samplers: Both the JIS type sampler and the conventional type sampler used in the U.S. were used. The shoe of the U.S. type sampler has an inner diameter of 1 3/8", and the inner diameter of the split barrel is 1/2". Thus, there is a clearance of 1/8". Both the shoe and split barrel of the JIS type sampler have inner diameters of 35 mm, so there is no inner clearance.

Rods: Rods with a nominal diameter of 40.5 mm, as specified in JIS M 1409, and rod couplings with a nominal diameter of 40.5 mm, as specified in JIS M 1410, were used.

Knocking Heads: Knocking heads shaped in accordance with the example shown in JIS A 1219 were used. However, slight changes were introduced in outer diameters and heights of the knocking heads.

Hammers: Hammers shaped in accordance with the example shown in JIS A 1219 were used. However, slight changes were introduced in dimensions of parts.

Hammering: The rope and cathead method, the most commonly used method in Japan, with one and three-fourths of a turn, was applied in most cases. A manila rope, with 15 mm in diameter was used. Even thonugh care was taken to minimize fricton between the rope and cathead, the hammer still did not fall freely.

ii) SDPT

This test is also called the Swedish Ram Sounding test. The most commonly used equipment for the Swedish DPT is manufactured by the firm, Borro AB, of Stockholm. The technique involves the use of 32 mm sounding rods and a cone point, The sounding rods are driven down mechanically by a hammer weighing 63.5 kg (140 lb) which is allowed to fall freely 50 cm, The number of blows required to drive the penetrometer down 20 cm is recorded as Ndm. The technique has the advantage of speed and gives a continuous record of the penetration resistance as a function of depth. An unfortunate point in common with many dynamic penetration tests is that it not possible to distinguish between point resistance and side friction resistance.

Bergdahl and Dahlberg have proposed an improved method which allows the instrument to determine side friction resistance so that point resistance may be obtained with reasonable accuracy. This improved method measures the torque required to rotate the rods at the ground surface per 1 meter of penetration. The only additional tool needed is a torque wrench.

Point resistance Nd is determined according to the following procedure:

a) Measured resistance Ndm is dependent on point resistance Nd and side friction resistance Nf, where $Ndm = Nd + Nf$.
b) Side friction resistance is measured with a torque wrench, applied to the sounding rods at the ground surface.
c) If there appears to be a relationship between Nf and measured torque (T), Nf may be determined by the equation, $Nf = k \times T$, where k is a correction coefficient. When torque T is measured in units of kg×cm, correction coefficient $k = 0.004$.
d) Now, point resistance Nd can be calculated from the following equation:

$$\begin{aligned} Nd &= Ndm - Nf = Ndm - kT \\ &= Ndm - 0.004T \end{aligned}$$

Borro AB has developed a new instrument, the "Automatic Ram Soundig" machine. This instrument automatically controls the hammer's free fall of 50 cm and counts the number of blows. Also the side friction resistance required to rotate the rods is recorded every 1 meter. This is done during the interval when the next rod is connected and consequently does not interfere with the measurement of the penetration test.

The "Automatic Ram Sounding" machine was used and the improved testing method was

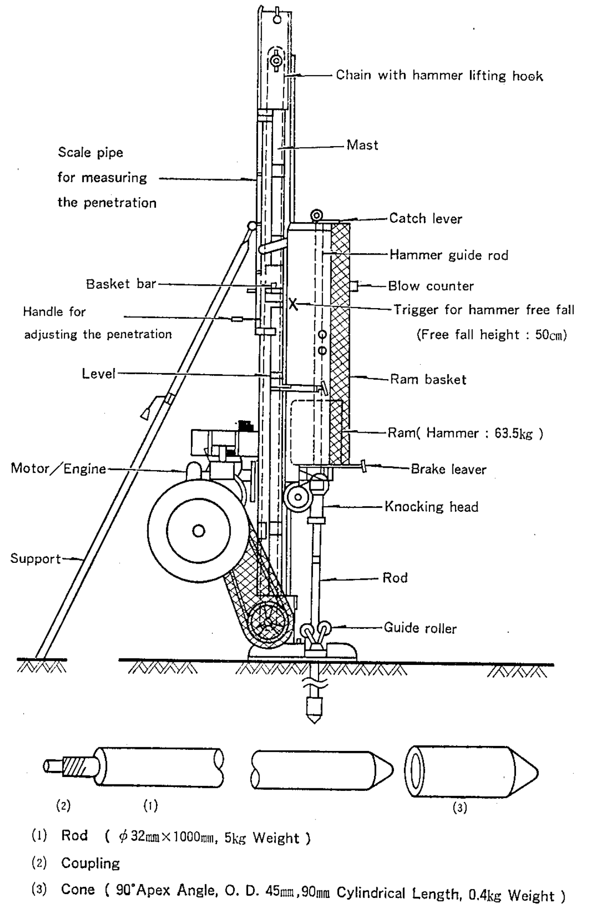


Figure 9 : Schematic diagram of Swedish dynamic cone penetration apparatus (Automatic ram sounding)

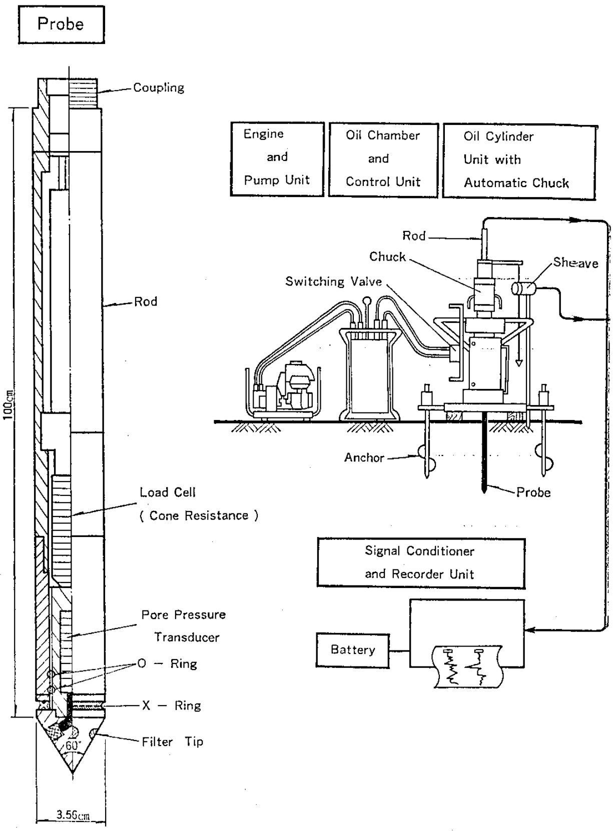


Figure 10 : Schematic diagram of OYO piezo cone penetration apparatus

Table 2 : Comparison of SPT with RTRI-DPT

test	end	rod hammer	dropping distance	penetra－tion	measure－ment	term for no. of blows	pre－dril－ling
SPT	tube sampler	40.5 m／m 63.5 kgf	75cm	30cm	non－continuous	N	yes
RTRI DPT	60 end angle cone	40.5 m／m 63.5 kgf	75cm	30cm	continuous	Nd'	no

applied for the comparative tests conducted by the authors. Figure 9 outlines the testing apparatus.

iii) CUPT

There are many types of piezo cone. In the comparative testing, an electric CUPT device that was developed by OYO Corporation was applied. In this test, the end cone is penetrated at a speed of 1 cm/sec, automatically measuriug point penetration resistance, qc and pore water pressure ud. Figure 10 outlines the apparatus. It uses standard dimensions for the end cone, with an outer diameter of 35 mm and apex angle of 60°. Maximum penetration force is 4 tons.

iv) RTRI-DPT

Drilling is not required for this test. It was developed by the Japan National Railways

Technology Research Institute, as a simple method for determining Nd values of the ground. The method involves the use of 40.5 mm regular boring rods and a cone point. The rods are driven down mechanically by a hammer weighing 63.5 kg (140 lb) freely falling 75 cm. The number of blows required to drive the cone 30 cm is recorded as Nd'. Table 2 compares the SPT and RTRI-DPT methods.

4.2 Results of Comparative Tests

Figures 11 through 13 give the results from the comparative sounding tests. As mentioned, the objective of the tests was to use sounding apparatus that sensitively reflects liquefaction potential as a way of investigating their applicability. With this objective in mind, the following is an outline of the results of each of the soundings.

i) Results of the SPT (N_J—N_{US})

Both the JIS and U.S. samplers were used in the SPT to determine N. Two boreholes each were drilled at sites A, B, C and D. In the first borehole, the JIS sampler was used to measure N value at odd numbered depth levels, i.e., 1 m, 3 m and so on up to 19 m. The semeasureed values were designated N_J. Then, in the same borehole, the U.S. sampler was used to measure N value at even numbered depth levels, i.e., 2 m, 4 m and so on up to 20 m. These measured values were designated N_{US}. In the second borehole, SPT was conducted with the JIS sampler at even numbered depth levels and the U.S. sampler was used at odd numbered depth levels. N values measured by the two methods at similar depths, but in mutually different soil layers were not included. A total of 49 pair of N_J and N_{US} data were accumulated. Figure 14, taken from these results, shows the relationship between those values. In Figure 14, N_J tends to be greater than N_{US}. The average figures are as follows:

$$N_{US} = 0.88\, N_J$$

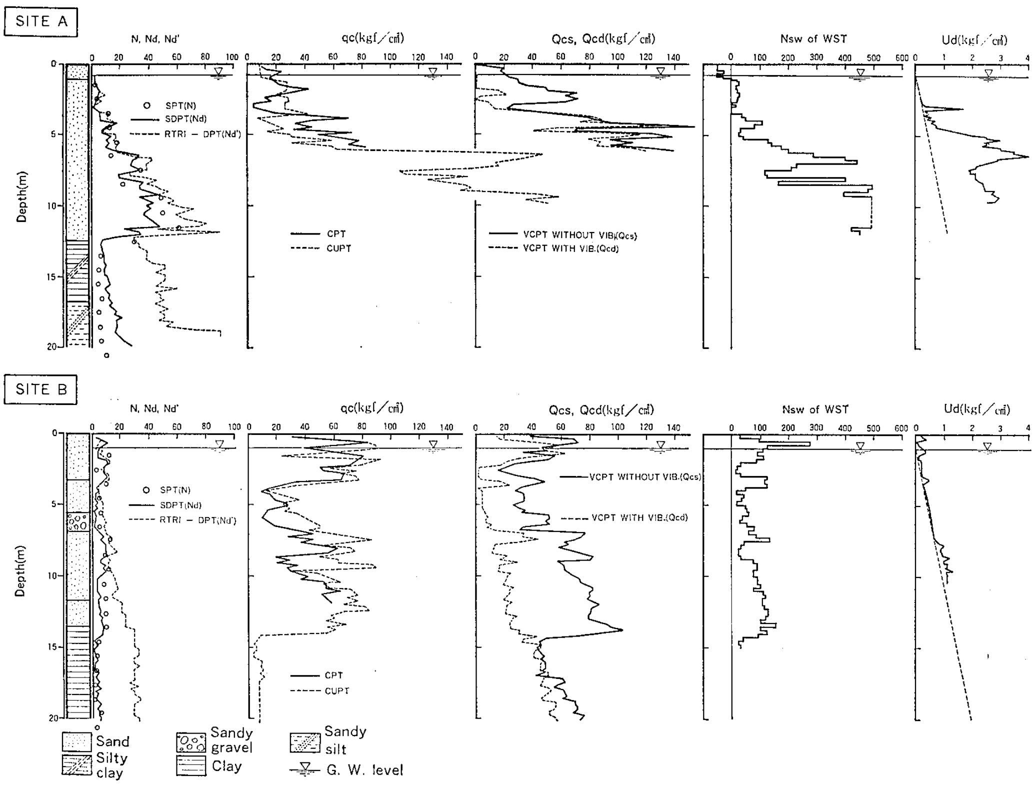


Figure 11 : Compiled results of sounding in Akita liquefaction region

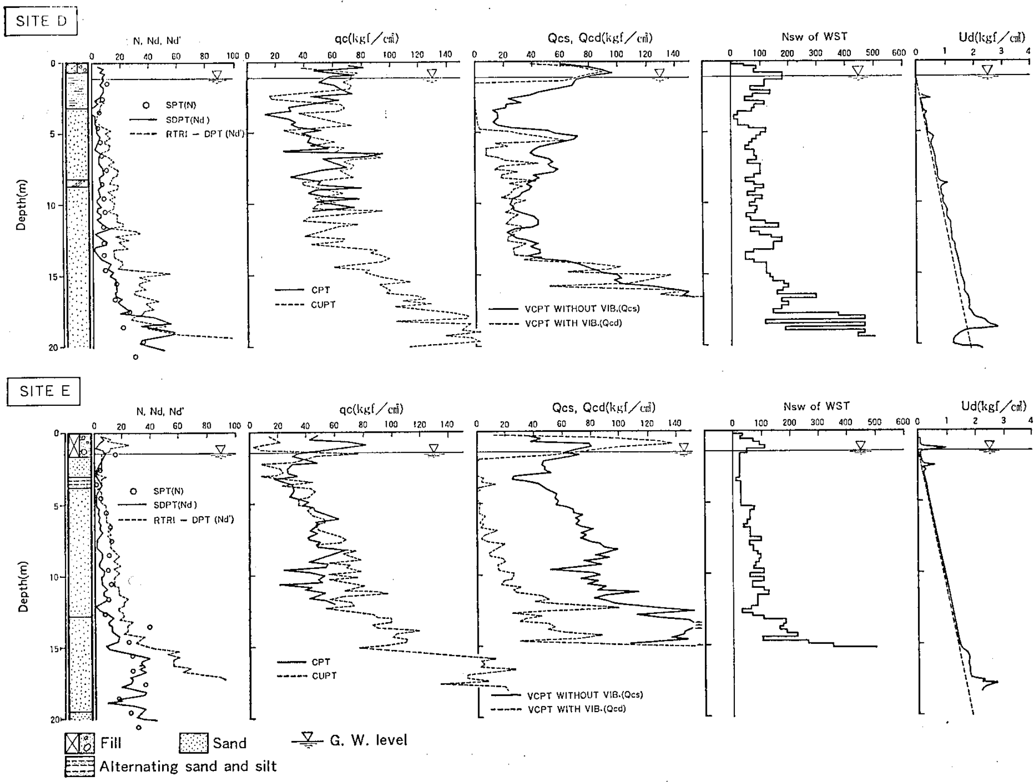


Figure 12 : Compiled results of sounding in Niigata liquefaction region

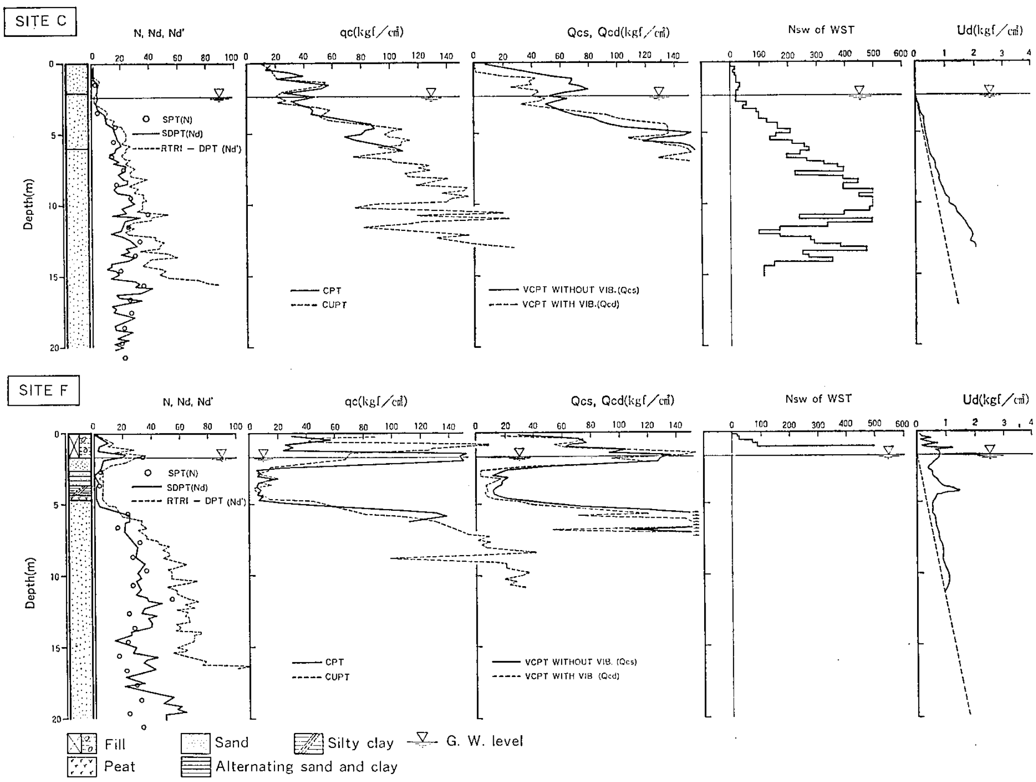


Figure 13 : Compiled results of sounding in Akita and Niigata non-liquefaction region

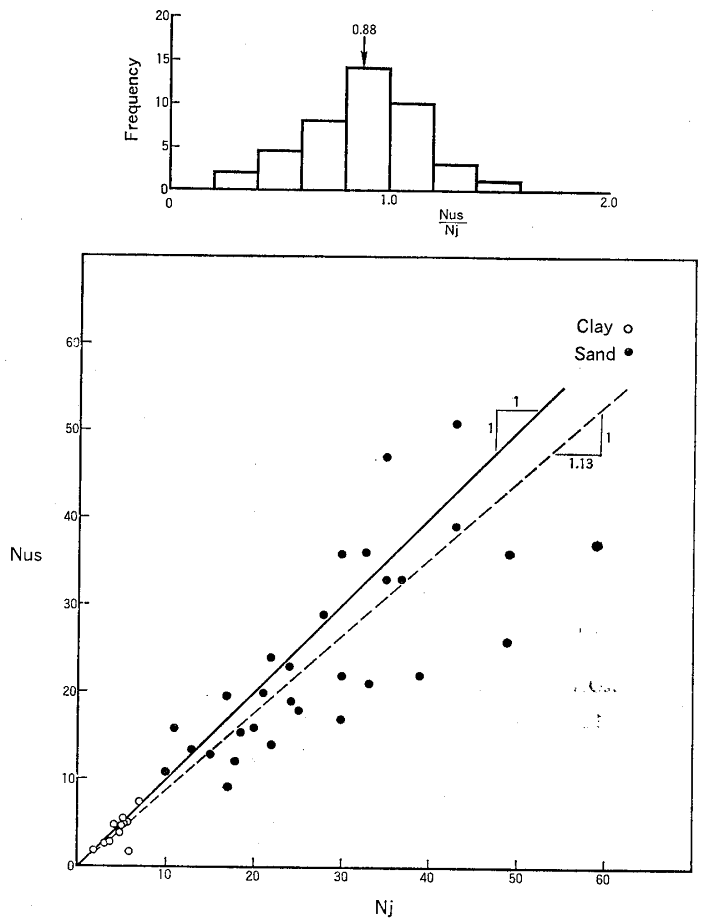


Figure 14 : Relationship between N_J and N_{US}

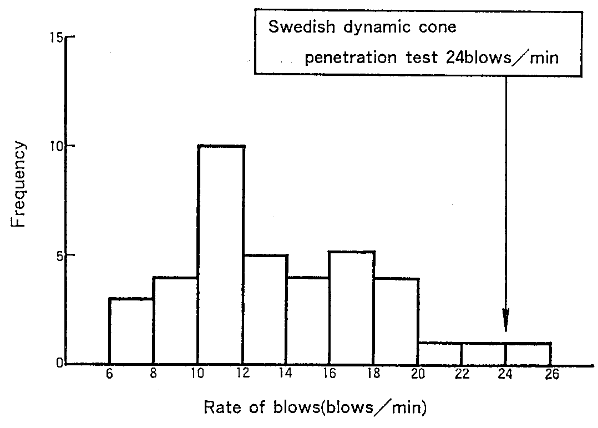


Figure 15 : Distribution of rate of blows in SPT (W. D. Kovacs et al. 1984)

The reason for the difference is clearly due to the difference in dimensions of the JIS and U. S. samplers. Therefore, it is possible that the JIS sampler has the larger inside friction.

The N value plotted in the figure have not been corrected for the ratio of energy transmitted to the rod (ERi). The ratio of energy transmitted to the rod by this test has been extensively studied by Kovacs et al. (1984). According to the study, the average ratio of energy transmitted into the rod is 68 percent of the theoretical energy level.

ii) Dynamic penetration tests (SPT, SDPT and RTRI-DPT)

The depth distribution of all three of these penetration resistance values shows similar tendencies. In particular, N value from SPT and Nd from SDPT, follow practically the same pattern. However, in areas where N value is 15 or below, average Nd value is about 75% that of N value. The reason for this is probably that in SDPT, the rate of hammer blows is relatively fast, compared to SPT (Figure 15). In the SPT, there is more excess pore water pressure produced by dynamic penetration remaining, so Nd value more sensitively reflects the reduction in strength with dynamic penetration in layers of loose sand, where N value is low. The speed of hammer blows in SPT may vary greatly with environmental conditions and the person conducting the testing. Because speed of hammer blows is completely automatic in SDPT, this test measures dynamic penetration resistance without error due to the human factor.

Nd', from RTRl-DTP, is considered to include side friction on the rod. Comparing results from SPT and SDPT, it appears that the applicable depth is limited to 5 m. Consequently, at depths below 5 m, Nd' cannot be considered to be an index that reflects dynamic strength of the end cone. In consideration of this, RTRl-DPT is considered to be unsuitable for liquefaction potential investigations, unless it uses a method of correcting for friction.

iii) Static Penetration Tests (CPT, CUPT and VCPT)

In all static penetration tests (CPT, CUPT and VCPT), penetration resistance and depth distribution can be considered to be basically similar during static penetration. However, CPT can be used to measure fs and CUPT, to measure ud, as a means of determining soil layers. In addition, because the location of the ground water level can be determined from the depth distribution of ud, CUPT is useful in that it provides basic information for determining liquefaction potential. VCPT is a complex penetration test, consisting of a combination of dynamic and static methods. In test sites where liquefaction occurred, there is a marked difference between Qcs and Qcd, obtained from VCPT. Sasaki showed that it is possible to evaluate liquefaction potential by analyzing both these values (Sasaki, et al. 1984). However, in this testing, penetration resistance is measured by using a load cell on the surface, accord-

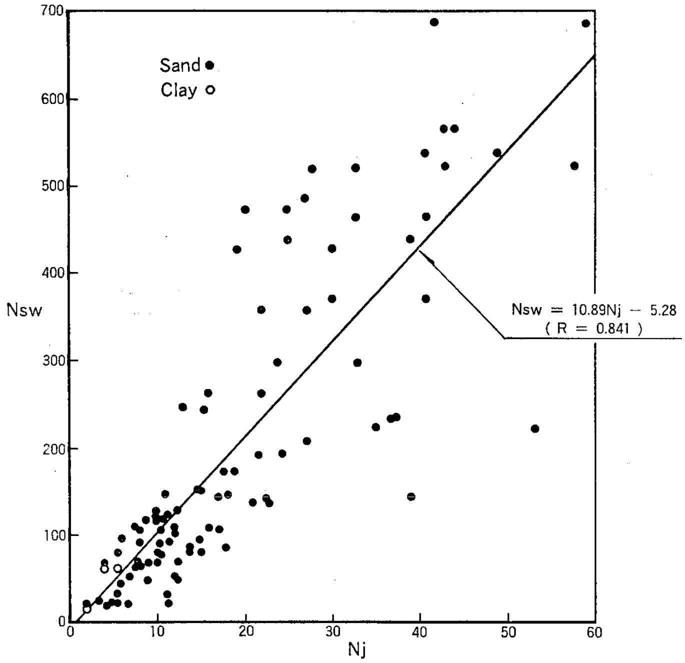
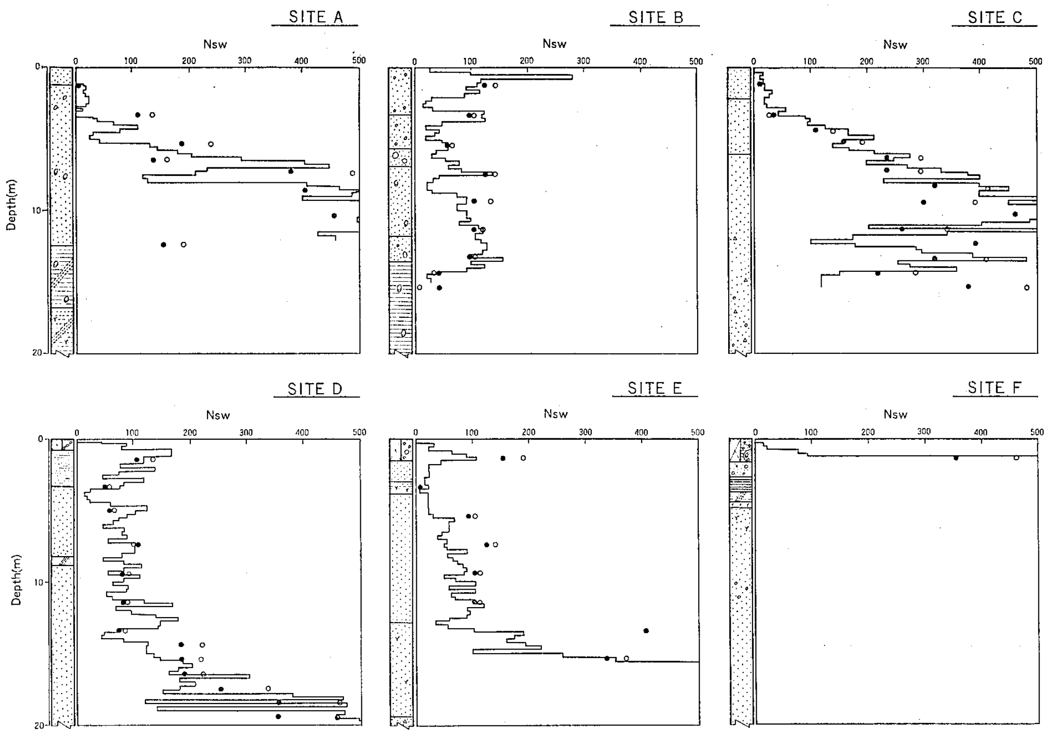


Figure 16 : Relationship between N_J and N_{SW}



o : Converted Nsw by using Inada's formula
• : Converted Nsw by using formula " Nsw = 10.89N − 5.28 "

Figure 17 : Comparison of N_{SW}(from WST) with converted N_{SW} (from N of SPT)

ingly, it is clear that the effects of side friction exert an influence on Qcs and Qcd, as well as on the ratio between Qcs and Qcd. This is particularly clear in the results from site B.

iv) Swedish Weight Sounding Test (WST)

There is a relatively good correlation between WST and SPT (Figure 16).

The following equation have been proposed to express the relationship between N_{SW}, obtained from WST, and N, obtained from SPT:

$$N = 0.318 N_{SW}^{0.755} \quad \cdots\cdots (\text{Ueda})$$

$$\left.\begin{array}{l} N = (1/12) N_{SW} \ (\text{for sand}) \\ N = (1/9) N_{SW} \ (\text{for loam}) \end{array}\right] \cdots\cdots (\text{Miki})$$

$$\left.\begin{array}{l} N = 2 + 0.067 N_{SW} \ (\text{for sandy soil}) \\ N = 3 + 0.050 N_{SW} \ (\text{for clayey soil}) \end{array}\right] \cdots\cdots (\text{Inada})$$

Of these, the equation proposed by Inada has the most data available, and is most widely used. Figure 17 shows the results from the tests carried out for this paper using this formula. From this, the following can be said:

(1) Nsw corresponds well with N value, overall.

(2) In sandy soil, for penetration of 20 m or more, there is no need for correction for side friction. This is due to the fact that the WST test apparatus rotates the rod, which makes it possible to minimize side friction.

(3) When evaluating liquefaction potential from N_{SW} it is easy to hypothesize N value, using Inada's equation and use the standard simplified method for determining F_L.

(4) In ground where liquefaction was believed to have occurred, N_{SW} is generally 150 or less. Where Nsw is 100 or less, liquefaction always occurs.

(5) From this, it may be concluded that liquefaction potential is very high in sandy ground

where N_{SW} is less than 100.

(6) Using Inada's equation ($N=2+0.067\,N_{SW}$) to determine N value, F_L was then determined by applying the simplified method. The results were compared with F_L values determined using N values measured in the comparative tests. It was found that F_L from Inada's equation consistantly shows the more conservative values.

(7) WST has the advantage of providing continuous data.

(8) Since samples cannot be obtained with the WST, information on grain size cannot be obtained with this test.

(9) In sandy soil, which has heigh liquefaction potential, it is not clear what the effects are on Nsw of the rate of the rotation of the screw point. Consequently, when the objective is to evaluate liquefaction potential, it is desirable that the rotation velocity be controlled. However, since this apparatus normally manually operated, it is hard to control.

4.3 Summary

On the basis of the above, the following conclusions are drawn:

(a) It is difficult to directly determine the liquefaction potential of the ground on the basis of results from a single in situ test.

(b) By combining static and dynamic tests, there is a possibility of directly determining liquefaction potential. VCPT is an example of such a composite sounding method.

(c) It is necessary to minimize error in measuring cone point penetration resistance values as much as possible. Therefore, it is desirable to take continuous records from an apparatus that automatically conducts penetration. Also, correction for side friction resistance must be made. In consideration of this, CPT, CUPT (static) and SDPT (dynamic) methods may be regarded as appropriate methods.

(d) ud, obtained from CUPT, makes it possible to locate the ground water level. Also, it may be used to determine soil layers at a site.

5 ATTEMPT TO DETERMINE LIQUEFACTION POTENTIAL BY COMBINING STATIC AND DYNAMIC PENETRATION TEST RESULTS

Let us investigate whether it is possible to estimate liquefaction potential by analyzing the results from a combination of CUPT and SDPT.

Figure 18 combines together the qc/Nd depth distribution values for each site, showing liquefied zones and critical zones. Figure 19 takes into consideration the fact that there are very abrupt changes from layer to layer. It shows the depth distribution of qc/Nd, on the basis of the average values per 1 m interval (the average of 5 measurements taken every 20 cm).

These figures make clear, the following conditions concerning ground in which it was deduced that liquefaction occurred:

(a) $qc/Nd > 10$ Corresponds to places where it was deduced liquefaction occurred.

(b) $7 \leqq qc/Nd \leqq 10$ Corresponds to places where it was deduced that the borderline between liquefaction and non-liquefaction was reached.

(c) $qc/Nd < 7$ Corresponds to places where it was deduced liquefaction did not occur.

For all data, the ratio of qc to Nd is around 6. As Nd gets smaller, this ratio has a tendency to get larger (Figure 20). This shows that it is possible for qc/Nd to serve as a clear index of probability of liquefaction in loose sand.

Figure 21 shows the relationship between qc/Nd and D_{50}. In the figure, the symbols A, B and C represent places where liquefaction occurred (A), places where the liquefaction boundary was reached (B) and where liquefaction did not occur (C). Though it is clear in the figure, the value qc/Nd corresponds well with liquefaction potential.

The relationships shown in Figure 21 may be summarized as follows:

(a) For the entire range of D_{50} values, if qc/Nd <7, liquefaction does not occur.

(b) If $D_{50} > 0.04$ mm, when qc/Nd >10, liquefaction for the most part does occur.

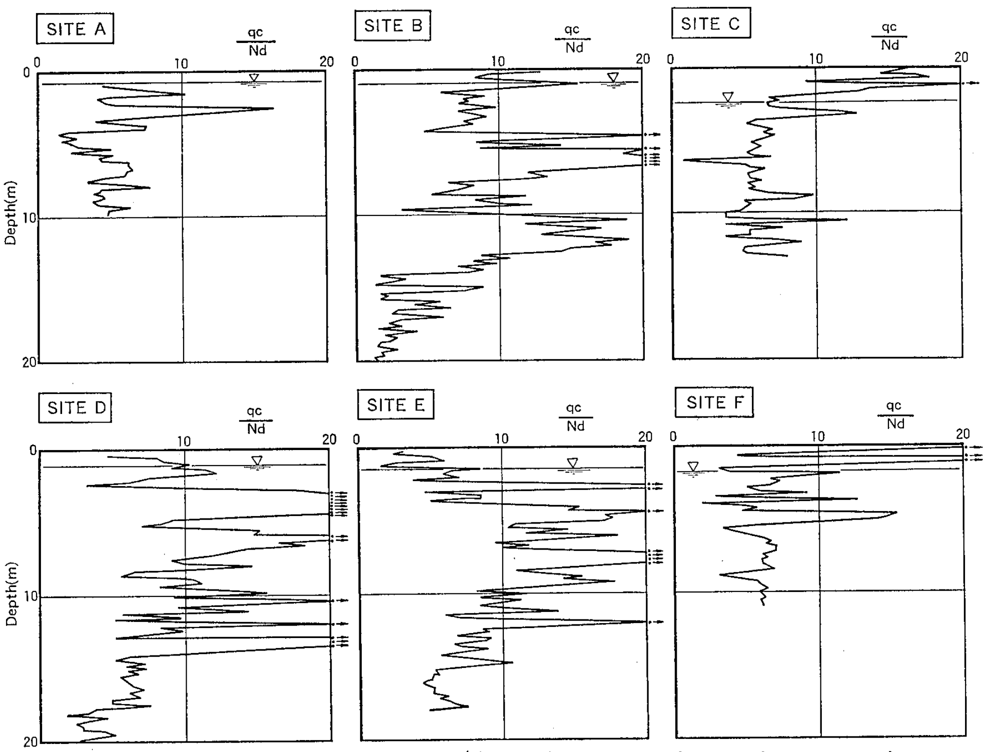


Figure 18 : Distribution of static/dynamic penetration resistance ratio

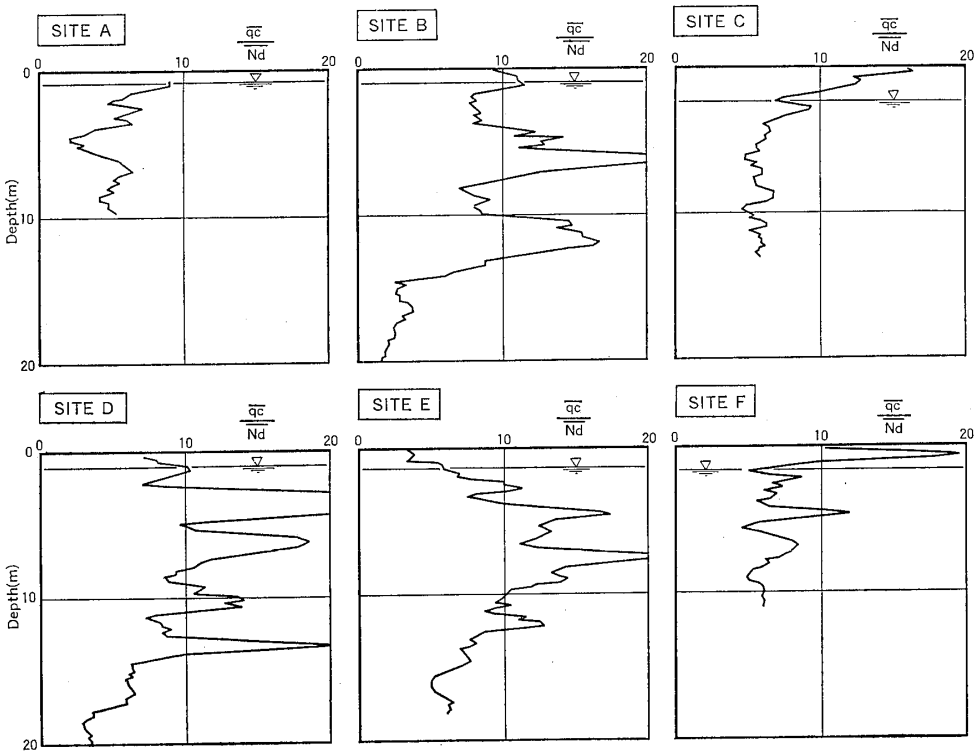


Figure 19 : Distribution of average static/dynamic penetration resistance ratio

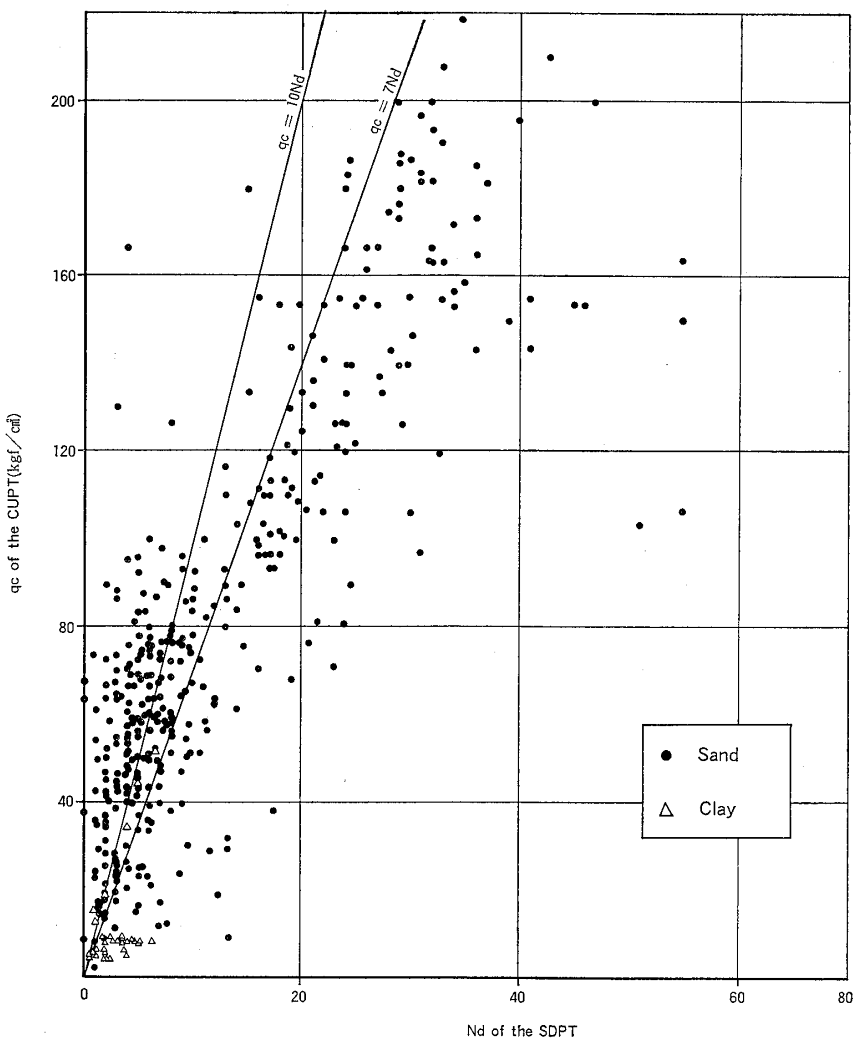


Figure 20 : Relationship between qc of the CUPT and Nd of the SDPT

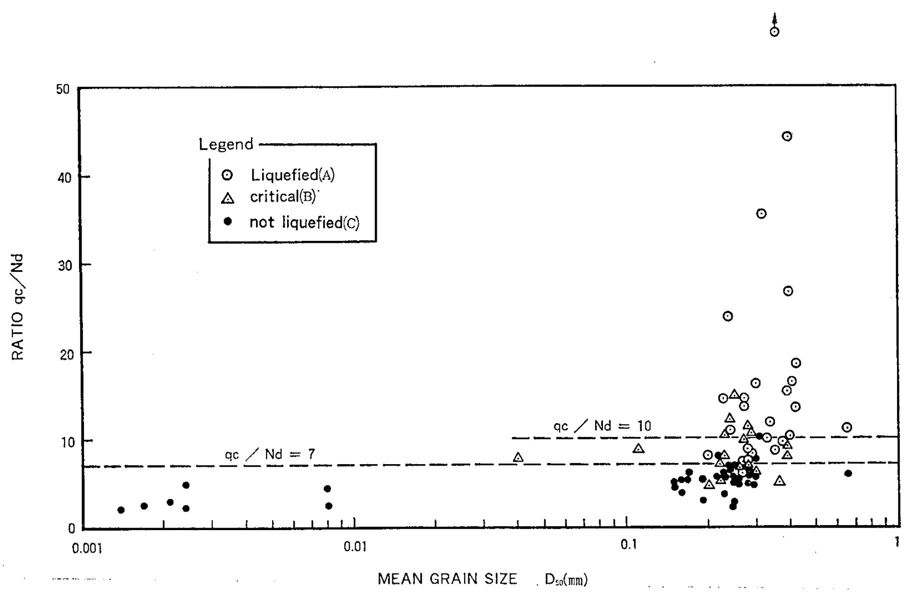


Figure 21 : Relationship between qc/Nd and D_{50}

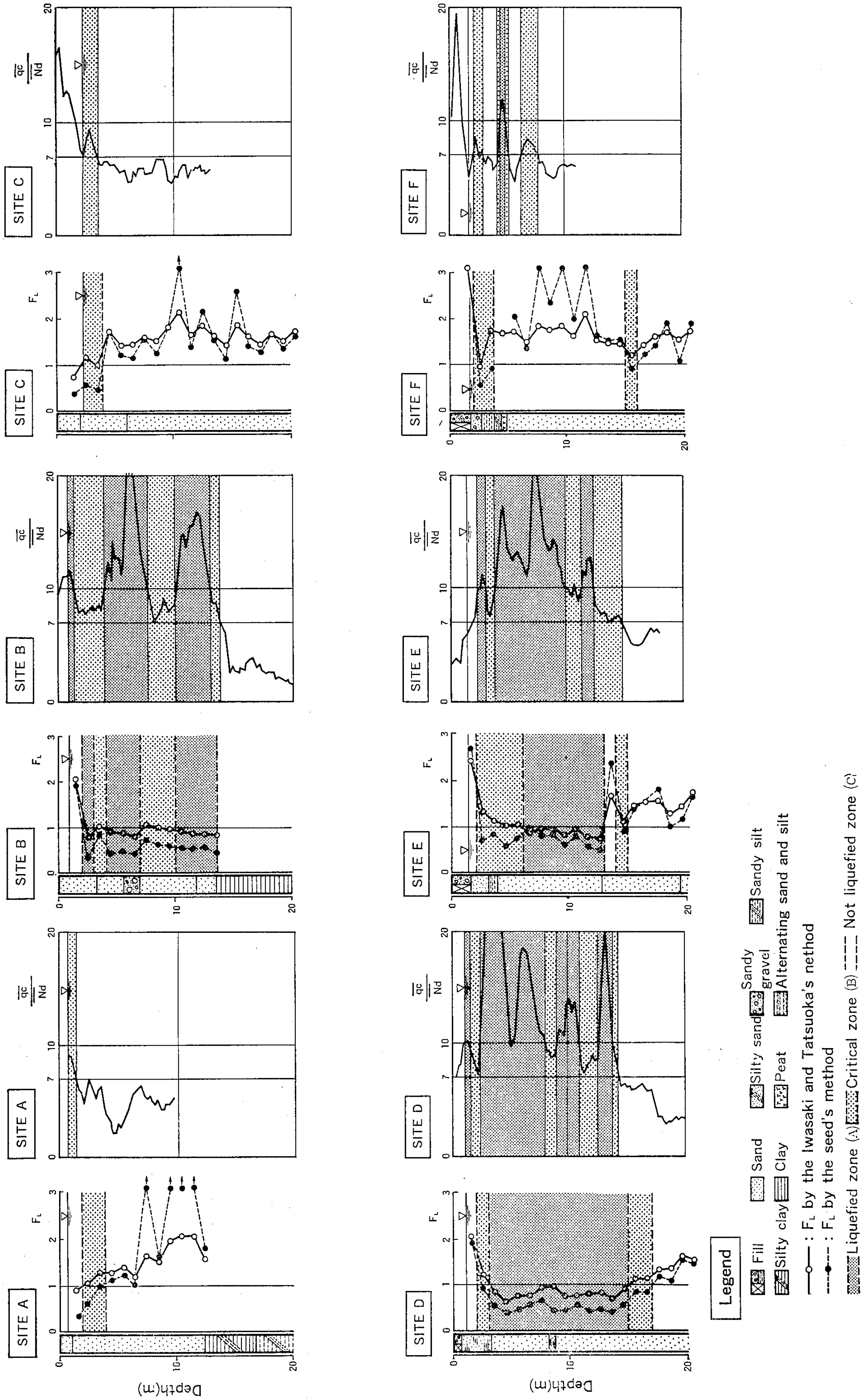


Figure 22 : Comparison of liquefaction potential by qc/Nd and simplified method using N value

(c) If $D_{50} \geqq 0.04$ mm, when qc/Nd = 7 to 10, there are places where liquefaction does and does not occur and where the borderline is reached.

(d) In clay, if qc/Nd = 5 or less, liquefaction does not occur.

We can see a good agreement in Figure 22 between the evaluated zoning of liquefaction potential based on N from SPT, and the criteria established from the ratio qc/Nd.

6 DISCUSSION

6.1 Nd, from SDPT

N and Nd have very similar tendencies, but in the low regions of N values, in sandy ground, where liquefaction potential is high, Nd < N. Figure 23 uses distribution of N/Nd, using N value as a parameter to designate zones of liquefaction potential, in order from A through C. The figure shows that in zone A, where dynamic sensitivity is high, N/Nd is generally more than 1, and extends over a range of 1 to 2. 3, with an average value of about 1. 5. In zone B, N/Nd values are above 1. In zone C, the ratio is in the range of 0. 4 to 1. 4, with an average value of 0. 9. The reason for this, as mentioned above, may be that in SDPT, the rate of hammer blows is faster than in SPT, with the consequent difference in dissipation of excess pore water pressure.

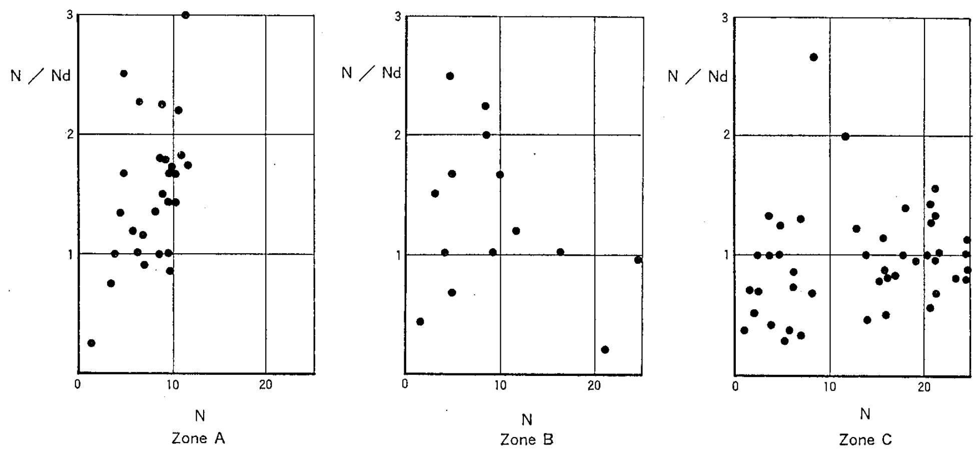


Figure 23 : Distribution of N/Nd to N value at Zones A, B and C

Figure 20 shows the relationships between Nd and qc. There is a notable difference in the values of qc/Nd in the ranges of Nd values of 10 and below and 10 and above. Nd was determined for a hammering rate of 24 blows per minute, and qc, for a penetration velocity of 1 cm/sec. Table 3 shows penetration velocities corresponding to various Nd values.

When Nd is 8 or less, speed of penetratoion is fater in SDPT than in CUPT. In general, when speed of penetration is faster, penetration resistance becomes greater, but in this case, even though SDPT, has the faster penetration velocity, penetration resistance is relatively low. Consequently, it must be assumed that with dynamic penetration tests, excess pore water pressure is produced in the sand around the end of the cone, which reduces penetration. On the basis of the above results, it can be concluded that with dynamic penetration testing, especially for loose sand, which has high porosity, hammering rate should be an extremely important element. With SPT, it cannot be said that there has really been adequate research on the matter. In using N value to evaluate liquefacion, it is necessary that a standard rate of blows be taken into account as a reference method.

If penetration resistance is considered to be controlled by the hammering rate, by con-

Table 3 : Comparison of penetrating speed of SDPT and CUPT

Nd	Pene. Speed of SDPT(cm/s)	Pene. Speed of CUPT(cm/s)	Pene. Speed ratio of SDPT/CUPT
2	4.0	1.0	4.0
5	1.6	1.0	1.6
10	0.8	1.0	0.8
20	0.4	1.0	0.4
40	0.2	1.0	0.2

ducting SDPT twice at the same point and changing hammering rate, it should be possible to determine different Nd values. Therefore, by conducting, for example, a test of 24 hammer blows/min and a another test with 10 hammer blows per minute, it is be possible to evaluate liquefaction potential from the ratio between the respective Nd values obtained (Nd_{20}/Nd_{10}).

6.2 Significance of the Comparative Tests

Eight types of penetration tests were conducted at sites both where liquefaction had and had not occurred. The penetration resistance values obtained for the different types of ground showed a general tendency to agree. However, there were clear differences in the results from dynamic and static penetration tests. With the dynamic penetration tests, penetraiton resistance value Nd varied according to the rate of hammer blows per minute. From this, qc/Nd showed itself useful in evaluation of liquefaction potential. It appeared that liquefaction potential can be evaluated by conducting different test methods. This finding made it possible to conduct different kinds of sounding in the same ground and compare the results. This is regarded as a very significant finding.

Liquefaction may be defined as loss of shear strength as a result of excess pore water pressure, caused by dynamic stress applied to the ground. Therefore it is appropriate to use dynamic in situ tests to determine how sensitively penetration resistance changes as a method of evaluating liquefaction potential. In other words, the sensitivity of the ground in response to dynamic force should be determined. The main result of loss of strength of the ground being excess pore water pressure, future research should use apparatus for conducting the SDPT, in which changes in pore water pressure as the result of changes in hammer blow rate or penetration speed are measured. By increasing our knowledge of the mechanism of penetration in the SDPT, a more reliable evaluation of liquefaction potential can be expected.

7 CONCLUSION

The following points were determined concerning the evaluation of liquefaction potential of sandy ground from in situ tests:

(a) It is difficult to evaluate liquefaction potential from a single in situ test results, whether they be static or dynamic penetraton tests.

(b) SDPT is the most appropriate dynamic test to obtain more detailed, continuous data. The reasons for this are: (1) Rate of hammering is uniform; (2) free fall of the hammer can be completely controlled and (3) side friction can be adequately compensated for.

(c) Unstable, sandy ground prone to liquefaction has different static and dynamic penetration resistance ratios than normal ground.

(d) This fact may be used to evaluate liquefaction potential from results of combined static and dynamic penetration tests.

(e) To do so, it is necessary to correctly obtain continuous penetration resistance values for the end cone in both the static and dynamic tests. Of the various test apparatus used in these tests, the most appropriate seem to be the following:

Static: CPT or CUPT (manual/electric)

Dynamic: SDPT

(f) Using qc/Nd from CPT (CUPT) and SDPT as a parameter, the following can be proposed as criteria for evaluating liquefaction potential:

$qc/Nd > 10$	Very high probability of liquefaction.
$7 \leqq qc/Nd \leqq 10$	Possibility of liquefaction.
$qc/Nd < 7$	No possibility of liquefaction.

(g) These criteria correspond to the situation where maximum acceleration on the surface is 150 to 170 gal. The problem of how to correct for different maximum surface acceleration values remains for future research.

(h) Information concerning ground water level and distribution of layers can serve as essential correction factors for determinig liquefaction potential. In this way, it is recommended that local friction, pore water pressure, etc. be measured. In particular, measurement of pore waterr pressure is extremely effective in determining groundwater conditions and in distinguishing different soil layers.

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In conducting these field comparative tests, nationally owned land in Akita and Niigata was used. This was made possible through the good offices of Dr. Toshio Iwasaki and others, of the Public Works Research Institute of the Ministry of Construction.

We wish to extend our warmest appreciation to him.

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静的・動的貫入抵抗比を用いた液状化ポテンシャルの評価法について

大矢　暁・岩崎恒明・若松幹男

概　要

地震多発国である日本では，飽和した砂地盤の液状化ポテンシャルの評価は，建築基礎，土構造物基礎の設計にとって重要な検討項目となっている。現在，実用的に用いられている液状化ポテンシャルの評価法を分類すると次のようになる。

(1) 乱さない試料の室内動的試験の結果から，土の動的強度を求める直接的方法。

(2) N値および土の粒度特性から経験的に土の動的強度を求める間接的方法。

室内における動的土質試験の結果から，直接的に土の動的強度を求める方法は，理論的な根拠を持ち，信頼のおけるものと考えられがちである。しかし，ゆるい砂地盤の試料を乱さないで採取することは大変難しく，費用がかかるばかりでなく，複雑な装置やそれを操作するためのオペレーターに高度な技能を必要とする。したがって，全体のプロセスを通して品質管理が行われない限り，結果の信頼性に疑問の残る場合が多い。

一方，標準貫入試験（ＳＰＴ）によって求められるＮ

値による方法は，簡便であるように考えられているが，粒度試験の結果に基づく平均粒径や単位体積重量をパラメーターとし必要とし，データーの処理方法も必ずしも簡易でない。また，SPTの試験方法そのものも国際的に統一されていないので，試験方法間のキャリブレーションの問題も残っている。

そこで，図―1に示した過去に発生した地震の際に，明瞭な液状化を生じた地盤4ヶ所，これらの箇所に近接した箇所で，液状化の被害のみられなかった地盤2ヶ所において8種類の動的・静的貫入試験を行い，液状化ポテンシャルを評価するための有効な原位置試験方法について検討した。

まず第一に，比較試験を行なった6サイトについての液状化による被害の詳細な観察を行った。その結果は表―1および図―1～図―5に示すとおりである。さらに，従来の液状化の簡易評価法による F_L によって，液状化を生じたと考えられるゾーンを推定し，図―7に示すような結果を得た。これらのことから，図―8に示すように，F_L の値によって実際の液状化の被害状況を比較的正確に評価し得ることが明らかになった。

次に，各サイトで実施した動的・静的サウンディングの比較試験の結果は，図―11～図―17に示すとおりである。これらの結果について，地盤の液状化ポテンシャルを適切に評価することができる原位置試験法という観点からの評価を行った。その結果，

① 単一の原位置試験結果だけからでは，地盤の液状化ポテンシャルを評価しにくいこと。

② 地盤の貫入抵抗を連続的に正しく求めることができる静的・動的貫入試験は，ダッチコーンタイプの貫入試験（CPT），または間隙水圧を測定することができるエレクトリックタイプのピエゾコーン貫入試験（CUPT，図―10参照）とスウェーデン式動的コーン貫入試験（SDPT，図―9参照）であること。

が明らかになった。

また，図―18および図―19に示すように，液状化を起こすような動的外力に対する鋭敏度の高い地盤では，上記の貫入試験結果から得られるqc/Nd値が，極めて特異な値を示すことが明らかになった。

qc/Nd値は．平均粒径（D_{50}）によって変化するものではあるが，液状化を生じる可能性のある砂質地盤の粒径の領域においては，一般に，5～6付近の値を示すことが多いと考えられている。

図―21は，今回の比較試験の結果に基づいて，qc/Nd と D_{50} の関係を示したものである。この図には，液状化が生じたと考えられる領域，液状化を生じなかったと考えられる領域およびこれらの領域の間にある境界領域を区別して示してある。この結果からは，おおむね，次のように考えることができる。

$qc/Nd > 10$	：液状化が生じる可能性が高い
$7 \leqq qc/Nd \leqq 10$	：液状化が生じる可能性がある
$qc/Nd < 7$	：液状化が生じる可能性は低い

上記の区分に基づいて，今回の比較試験を行つた6サイトの液状化ポテンシャルを評価し，従来の F_L 法によって判定したものと対比を行うと，図―22に示すように極めて良い一致をみた。

以上のことから，同一箇所において，静的貫入試験（CPTまたはCUPT）と動的貫入試験（SDPT）の両方の試験を実施し，この結果得られるqc/Ndを用いて砂質地盤の液状化の可能性を評価し得ることが明らかになった。ただし，今回得られた区分基準は，液状化を生じた地盤の地震時の地表最大加速度が 150gal 程度，震央距離が 100km 前後という状況のもので得られたものであることを付記しておく。

（この論文は，1985年サンフランシスコで行なわれた「液状化判定のための原位置試験に関する第1回ワークショップ」において発表したものに加筆したものです。）