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SYNOPSIS A simplified method is presented for evaluating liquefaction potential of sand deposits using shear wave velocity. Effectiveness of the proposed method is evaluated through field tests at 17 sites in Niigata city where field performance during the 1964 Niigata earthquake is known. A modified version of steady state Rayleigh wave method is used in which the amplitude ratio between vertical and horizontal ground surface motions can be measured in addition to the phase velocity. Based on the measured phase velocity vs. wavelength relationship, shear wave velocity profile is determined using an inverse analysis. The liquefaction potential of each site is then evaluated using the shear wave velocity. The estimated results are reasonably consistent with the actual field behavior during the earthquake, indicating that the proposed method is effective.

INTRODUCTION

There exists a significant number of simplified procedures for evaluating soil liquefaction potential based on insitu tests such as the standard penetration test (SPT) and cone penetration tests (CPT). They are however basically the same procedures in a sense that they are based more or less on the field correlation between liquefaction resistance and SPT N-value since a sufficient body of field data is only available with SPT N-values.

Since the penetration tests may not always provide a reliable estimate and cannot be performed conveniently at all depths or in all soils, it is desirable to have a different method which is hopefully independent from the SPT based correlation. Shear wave velocity is a possible indicator for this purpose because its value tends to increase with increasing liquefaction resistance.

In addition, shear wave velocity can be measured more rapidly than the SPT if Rayleigh wave method (Stokoe et al., 1988) or seismic cone penetration test are adopted. Further, the Rayleigh wave investigation can simply be performed by placing sensors on the ground surface and without any boreholes. Such rapid and simple site investigation is particularly efficient in characterizing two- and three-dimensional geophysical profile for the determination of liquefaction hazard mapping.

Despite its potential advantages, there seems no reliable procedure to evaluate liquefaction potential of sandy soils using shear wave velocity determined from Rayleigh wave investigation. The object of this paper is to propose a simplified procedure for estimating soil liquefaction potential based on a modified version of Rayleigh wave investigation.

RELATIONSHIP BETWEEN LIQUEFACTION RESISTANCE AND ELASTIC SHEAR MODULUS

Table 1 summarizes major factors that influence liquefaction resistance and shear wave velocity after Tokimatsu et al. (1988, 1989). Most of the factors that increase liquefaction resistance also increase shear wave velocity. This confirms the potential applicability of shear wave velocity for liquefaction evaluations.

There are two possible methods for the evaluation of liquefaction susceptibility using shear wave velocity:

(1) Strain Approach: This was first proposed by Dobry et al. (1982) in which the shear strain to be developed in the ground due to earthquake shaking is compared with the threshold strain at which pore pressures just begin to develop.

(2) Stress Approach: If these is a unique correlation between stress ratio causing liquefaction and shear wave velocity, liquefaction potential can be estimated by comparing the stress ratio to be induced by earthquake shaking with the soil resistance estimated from shear wave velocity.

Table 1 Factors Affecting Liquefaction Resistance and Shear Wave Velocity

Factor	Effect on Stress Ratio Causing Liquefaction	Effect on Shear Wave Velocity
Relative Density	+1)	+
Stability of Fabric	+	+
K_o	+	+
Effective Stress	-2)	+
Other major Factor	Plasticity	Void Ratio

1) significant, 2) insignificant

Since the strain induced in a sand deposit by given earthquake shaking cannot be computed with more accuracy than the stress and since the strain approach results in a considerably conservative estimate, the stress approach appears more preferable than the strain approach.

Probably, Stokoe et al. (1988) is the first to have presented a field correlation in which boundary separating liquefiable from non-liquefiable conditions is defined on a maximum ground surface acceleration vs. shear wave velocity chart. The results can be used to increase data base in the above methods. However, since all the data used in their correlation are for earthquake magnitudes of 5.5 and 6.5, and for the top about 15 ft. of depth, its application to other magnitude and depth appears restricted.

Because of a limited number of field case histories in which shear wave velocity profiles are available, Tokimatsu et al. (1986, 1989) conducted laboratory tests to study the relationship between liquefaction resistance and elastic shear modulus, which is related to shear wave velocity.

They performed cyclic triaxial tests on reconstituted sands with various densities and stress histories, and found that there is a good correlation between the liquefaction resistance and the elastic shear modulus only when soil type and confining pressure are specified. This is mainly due to the effects of material and confining pressure dependence of elastic shear modulus, which must be corrected for if elastic shear modulus or shear wave velocity is used as an indicator for liquefaction potential evaluations.

RELATIONSHIP BETWEEN LIQUEFACTION RESISTANCE AND NORMALIZED SHEAR MODULUS

Based on laboratory test results, Hardin and Drnevich (1972) have found that the elastic shear modulus of sands, G_o, can be expressed by:

$$G_o = A\ F(e)\ (\sigma_m')^n \tag{1}$$

in which A is a constant reflecting soil fabric, and is assigned a value ranging from 500 to 900 (Tokimatsu et al., 1986), σ_m' is mean effective

Table 2 Physical Properties of Soils

	G_s	D_{10} (mm)	U_c	e_{min}	σ_o' (kgf/cm^2)
Niigata #10	2.69	0.18	1.8	0.77	1.00
Niigata #11	2.69	0.18	1.6	0.78	0.5-2.0
Toyoura Sand	2.64	0.12	1.5	0.64	0.5-2.0
Meike #6	2.73	0.16	1.6	0.66	0.5-0.8
Meike #7	2.72	0.15	1.7	0.68	0.8-0.9
Meike #8	2.76	0.18	1.5	0.67	0.7-1.0
Ohgishima #8	2.75	0.12	2.0	0.91	0.79
Ohgishima #9	2.71	0.13	2.2	0.72	0.85
Ohgishima #10	2.70	0.12	2.0	0.71	0.92
Silica Sand	2.68	0.16	1.8	0.73	0.37
Makuhari #1	2.71	0.076	2.2	0.73	0.48
Makuhari #3	2.70	0.13	1.9	0.73	1.22

confining pressure in kgf/cm^2, n is a constant approximately equal to 0.5. F(e) is a function of void ratio, e, and may be given by:

$$F(e) = (2.17-e)^2/(1+e) \tag{2}$$

In order to correct for the effects of soil type and confining pressure on the liquefaction resistance vs. elastic shear modulus relationship, Tokimatsu and Uchida (1990) have proposed normalized shear modulus defined by:

$$G_N = G_o/[F(e_{min})(\sigma_m')^n] \tag{3}$$

in which n=2/3. In order to verify the applicability of the normalized shear modulus for various conditions in terms of soil type and confining pressure, Tokimatsu and Uchida (1990) have complied laboratory liquefaction tests including those of in-situ frozen samples (FS). Their results are summarized in Figs. 1 and 2. The liquefaction resistance in this case is

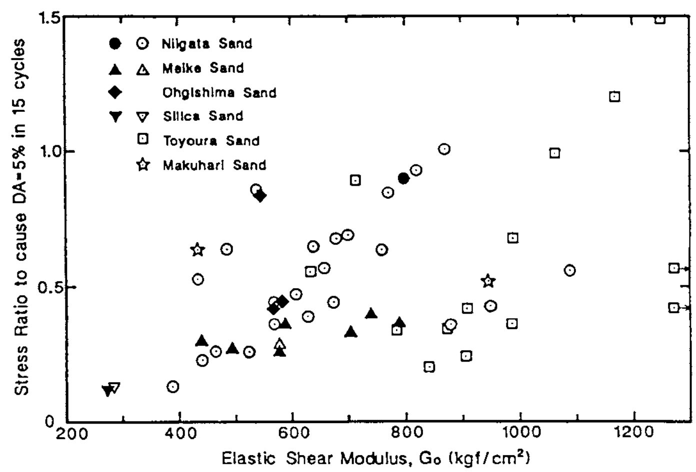


Fig. 1 Relationship between liquefaction resistance and shear modulus for various sands

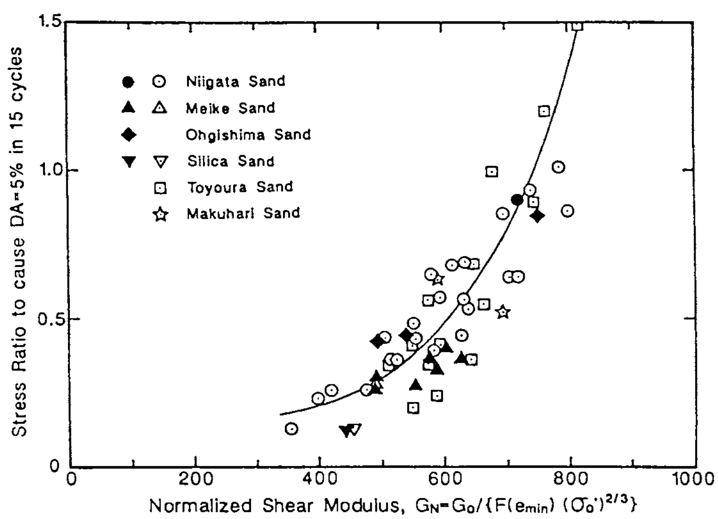


Fig. 2 Relationship between liquefaction resistance and normalized shear modulus for various sands

defined as the stress ratio to cause DA=5% at 15 cycles. The physical properties of these soils are listed in Table 2. The minimum void ratio was determined by the JSSMFE Standard Method of Testing for the Maximum and Minimum Densities of Sand, JSF Standard T26-81T (JSSMFE, 1979). The minimum void ratios range from 0.61 to 0.91, and the confining pressure from 0.37 to 2.0 kgf/cm^2.

Because of the material and confining pressure dependence of elastic shear modulus, the liquefaction resistance has a poor correlation with the elastic shear modulus in Fig 1. However, when the shear modulus is normalized as shown in Fig. 2, there is a good correlation in which the liquefaction resistance increases with increasing normalized shear modulus. The curve drawn in the figure is a representative relation to define this trend.

Fig. 3 summarizes a set of representative relations in terms of the number of loading cycles. As expected the liquefaction resistance for a given G_N increases with decreasing number of loading cycles. Since shear modulus or shear wave velocity can be measured both in the field and the laboratory, the correlation established in the laboratory could readily be applied to the field problem.

RELATIONSHIP BETWEEN LIQUEFACTION RESISTANCE AND NORMALIZED SHEAR WAVE VELOCITY

More conveniently, the correlation shown in Fig. 3 can be converted into the correlation between liquefaction resistance and normalized shear wave velocity, V_{s1}, as shown in Fig. 4 by using the following relationship:

$$V_{s1} = \sqrt{G_N F(e_{min})/\rho} \tag{4}$$

in which ρ is mass density and V_{s1} can also be defined by:

$$V_{s1} = V_s/(\sigma_m')^{1/3} \tag{5}$$

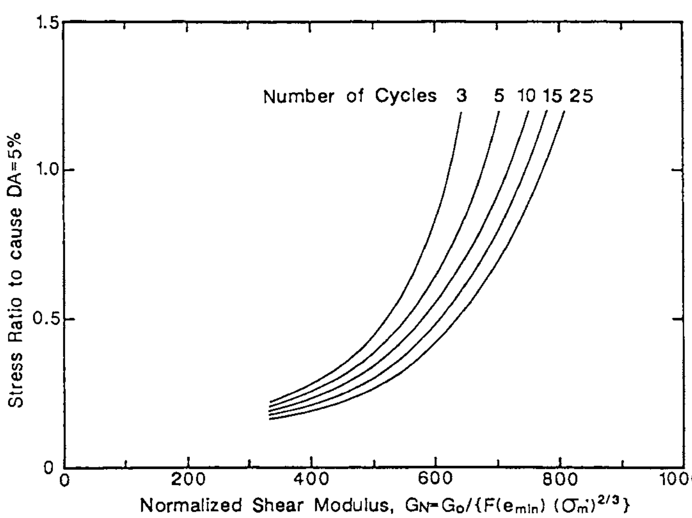
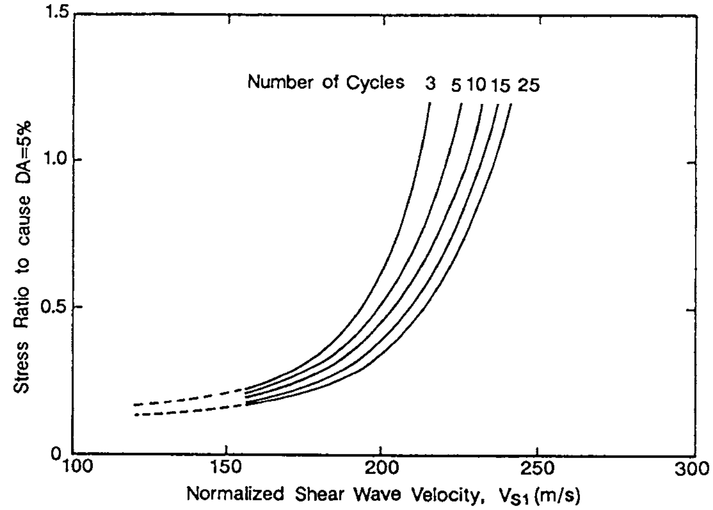
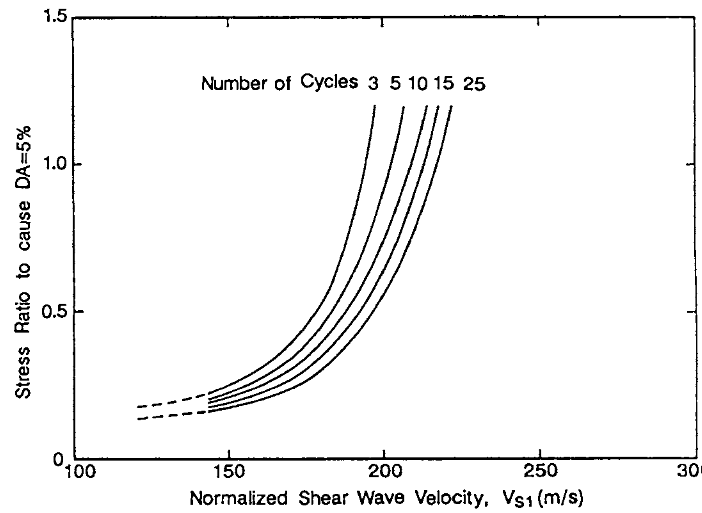


Fig. 3 Representative correlations between normalized shear modulus and stress ratio causing DA=5% at different number of cycles (after Tokimatsu and Uchida, 1990)



(a) Clean Sand



(b) Silty Sand

Fig. 4 Representative correlations between normalized shear wave velocity and stress ratio causing DA=5% at different number of cycles

In the above conversion, the information on the minimum void ratio and the unit weight of soil is required. The minimum void ratio may be evaluated from Fig. 5 in which its relation to fines content is given. On the average the minimum void ratio is 0.65 for clean sands without significant fines content, 0.75 for silty sands with significant fines content. These values were assumed as the first approximation. The unit weights of 1.9tf/m^3 for clean sands and 1.85tf/m^3 for silty sands were also assumed.

Fig. 4 indicates that any sand with a normalized shear wave velocity less than about 150 m/s can have a low liquefaction resistance, and that sands with a normalized shear wave velocity more than about 180 to 200 m/s could hardly liquefy during moderate to strong earthquake.

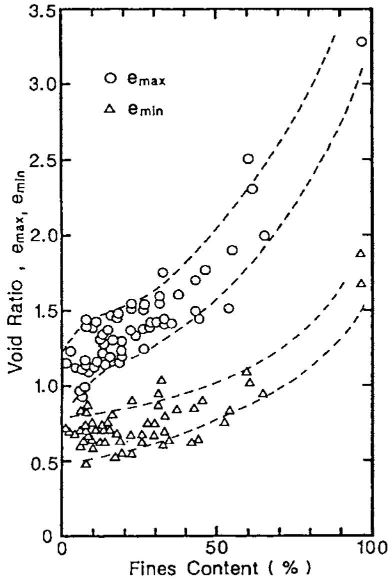


Fig. 5 Relationship between maximum and minimum void ratios and fines content (after Sakai and Yasuda, 1979)

EVALUATION OF LIQUEFACTION POTENTIAL FROM NORMALIZED SHEAR WAVE VELOCITY

Based on the correlation shown in Fig. 4, a simplified procedure for liquefaction potential evaluations using shear wave velocity can be developed as follows (See Fig. 6):

(1) Determination of the induced shear stress ratio, τ_d/σ_v', at a depth during an earthquake by:

$$\tau_d/\sigma_v' = 0.65\frac{\alpha_{max}}{g}\frac{\sigma_v}{\sigma_v'}r_d \tag{6}$$

in which α_{max} = maximum horizontal ground surface acceleration, σ_v = total vertical stress, σ_v' = effective vertical stress, and r_d = reduction coefficient with a value less than 1.

(2) Determination of the liquefaction resistance of soil, τ_ℓ/σ_v', at the same depth based on shear wave velocity as described later.

(3) Evaluation of the liquefaction potential, i. e., the factor of safety against liquefaction, FL, based on the comparison of the values obtained in Step (1) and (2).

The above procedure excluding Step (2) is essentially the same as the conventional procedure using SPT N-values. Thus only the details in Step (2) will be described hereafter.

(2-1) Determination of the shear wave velocity profile of the site. This may be made using Rayleigh wave investigation.

(2-2) Determination of the normalized shear wave velocity by Eq. (5).

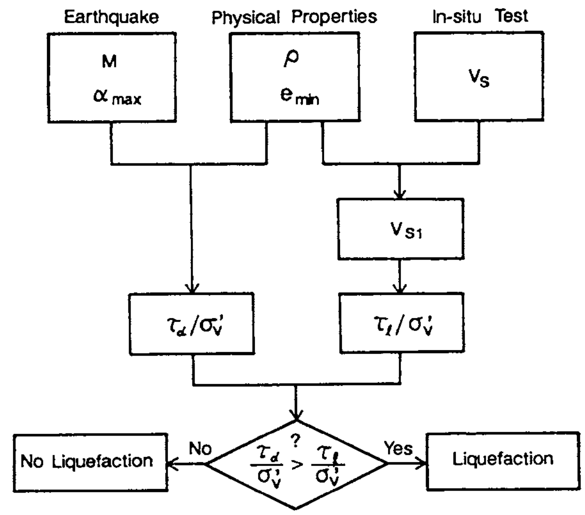


Fig. 6 Outline of the proposed method

(2-3) Evaluation of the stress ratio to cause liquefaction in triaxial test conditions, $(\sigma_d/2\sigma_o')$, from Fig. 4 with the normalized shear modulus for an appropriate loading cycles representing the effects of given earth quake magnitude (Seed et al., 1985).

(2-4) Conversion of the stress ratio to cause liquefaction for field K_o conditions, (τ_ℓ/σ_v'), according to the studies by Seed (1979) and Yoshimi et al. (1989) by:

$$(\tau_\ell/\sigma_v') = r_c\,\frac{1+2K_o}{3}(\sigma_d/2\sigma_o') \tag{7}$$

in which r_c is a constant to account for the effects of multidirectional shaking, with a value between 0.9 and 1.0.

In the above evaluations, the information on the earth pressure coefficient at rest, K_o, is required. However, since the procedures involved in Steps (2-2) and (2-4) almost cancel out the effects of K_o on the factor of safety against liquefaction, any value between 0.5 and 1 can be assumed for all practical purposes.

METHOD OF RAYLEIGH WAVE INVESTIGATION

Test Apparatus and Test Arrangements

The test apparatus and test arrangements used in this study are basically the same as those reported by Tokimatsu et al. (1991). Thus only the outline will be described herein. The test system consists of a vertical exciter, two pairs of sensors, amplifiers, and a personal computer. The exciter used has a maximum driving force of either 20 kgf or 250 kgf over the frequency range 5 Hz to 200 Hz. The sensors are velocity transduces with a natural frequency of 1 Hz.

As shown in Fig. 7, the exciter, and two pairs of sensors are placed in a line in such a way that the midpoint of the two pairs of sensors is located at the exact point under which V_s-profile is to be determined. The distances between the two pairs of sensors and between the exciter and the midpoint are defined by D and L. These values should be changed with measured wavelength, λ_i, so as to satisfy the following requirements (Tokimatsu et al., 1991):

$$\lambda_i/4 \leq L \tag{8}$$

$$\lambda_i/16 \leq D < \lambda_i \tag{9}$$

Each pair of sensors is set in such a way that the vertical and radial ground surface motions induced by the exciter can be measured at two different points. This arrangement can yield not only the phase velocity but also the parti-

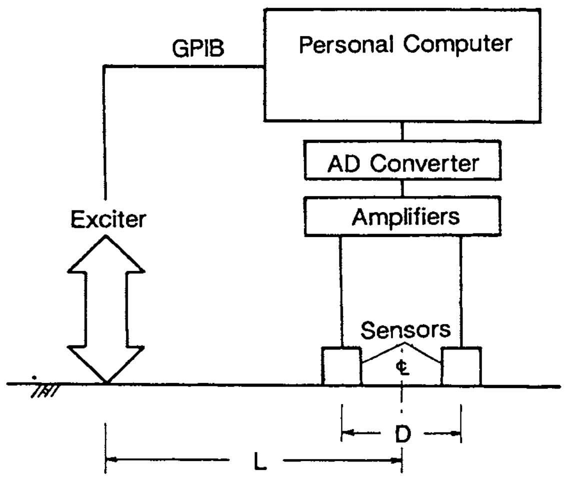


Fig. 7 Schematic diagram of test system

cle orbits of ground surface motions. The latter information can be used to identify which mode of Rayleigh wave is dominant and whether or not the measured motion is Rayleigh wave, since the particle motion of Rayleigh wave is elliptical in the vertical plane containing the direction of propagation of its wave.

<u>Test Procedure and Field Analysis</u>

The exciter oscillates with a simple vertical harmonic motion at a given frequency of f_i. The ground surface motions measured with the sensors are amplified and converted into digitized form through the AD converter installed in the computer.

The digitized motions are then transformed from the time domain to the frequency domain by the Fast Fourier Transform. The phase lag of the vertical motions between the two observed points, ϕ_i, is then determined based on their cross power spectrum.

The time lag of motions between the two points, Δt, is given by:

$$\Delta t = \phi_i/2\pi f_i \tag{10}$$

The phase velocity, c_i, can be determined from:

$$c_i = D/\Delta t \tag{11}$$

The corresponding wavelength, λ_i, can be given by:

$$\lambda_i = c_i/f_i \tag{12}$$

The particle orbit at each observed point can be obtained by plotting its horizontal and vertical motions on a x-z plane. The characteristics of elliptical particle motions can simply be defined by using the amplitude ratio between horizontal and radial motions, u/w. Positive values of u/w corresponds to prograde elliptical motions, and negative values to retrograde elliptical motions. The phase velocity and the amplitude ratio for the given frequency is display in the CRT of the computer, and stored with the basic data in a disk for in-house analysis.

The aforementioned measurements and analyses are repeated by changing frequency of the exciter. Owing to good performance of the computer, it takes about 20 to 30 minutes to measure and compute a dispersion curve with a maximum wavelength of about 50 m.

DETERMINATION OF V_S-PROFILES FROM DISPERSION CURVE

Haskell (1953) has developed an algorithm to determine both the fundamental and higher modes of Rayleigh wave dispersion curves for a horizontally stratified soil deposit consisting of N layers as shown in Fig. 8. The soil properties required to determine the dispersion curves are the thickness, H, mass density, ρ, P-wave velocity, and S-wave velocity of each layer. The total number of the properties is 4N-1, since the Nth layer is a halfspace.

Thus, Rayleigh wave method requires an inverse analysis on the measured dispersion curve for the determination of V_s-profiles. In the inversion, the effects of higher modes of Rayleigh waves which are dominant in high frequency range are taken into account according to the study by Harkrider (1964).

If the phase velocity, c_{ei}, are measured for I different frequency, f_i, from field observation, the inversion is to find soil properties that minimize the following:

Layer No.	Thickness	Density	P-Wave Velocity	S-Wave Velocity
1	H_1	ρ_1	V_{P1}	V_{S1}
2	H_2	ρ_2	V_{P2}	V_{S2}
⋮	⋮	⋮	⋮	⋮
N-1	H_{N-1}	ρ_{N-1}	V_{PN-1}	V_{SN-1}
N	∞	ρ_N	V_{PN}	V_{SN}

Fig. 8 One-dimensional soil layer model

$$S = \sum_{i=1}^{I} (c_{ei} - c_i)^2 \quad (13)$$

in which c_i can be computed based on the theory by Haskell (1953) and Harkrider (1964). Since the effects of the difference in density and P-wave velocity on the final results are negligibly small, only the thickness and S-wave velocity are the variables to be determined in the inversion. Thus the total number of layer properties to be determined is 2N-1.

The minimization of Eq. (13) may be achieved by first assuming appropriate values of soil properties and then updating them by using a modified version of nonlinear optimizing method originally proposed by Dorman and Ewing (1962) until S becomes practically zero, i. e., the theoretical dispersion curve matches with the observed one. Finally, the theoretical particle orbits computed for the updated model are compared with the measured ones to check whether or not the inversion is successfully conducted. If the computed particle orbits are consistent with the observed ones, the inverted model is considered as the actual soil profile. With this comparison, the reliability of the solution can be enhanced.

COMPARISON OF LIQUEFACTION POTENTIAL EVALUATIONS WITH FIELD PERFORMANCE DURING THE 1964 NIIGATA EARTHQUAKE

Liquefaction potential is evaluated using the proposed procedure at 17 sites in Niigata City where field performance during the 1964 Niigata earthquake is known. The earthquake has a Magnitude of 7.5, and its epicenter is about 50 km from the city. The maximum horizontal ground surface acceleration recorded at Kawagishi-cho is about 0.16 g.

Fig. 9 shows the locations of the test sites. Also shown in the figure is the zoning of building damage during the 1964 earthquake after Ohsaki (1966). Zone A corresponds to little or no damage, Zone B small damage, and Zone C heavy damage.

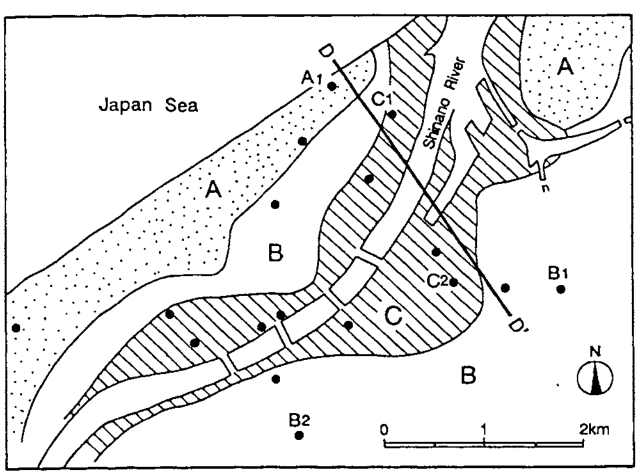


Fig. 9 Location of test sites with zoning map of building damage during the 1964 Niigata earthquake

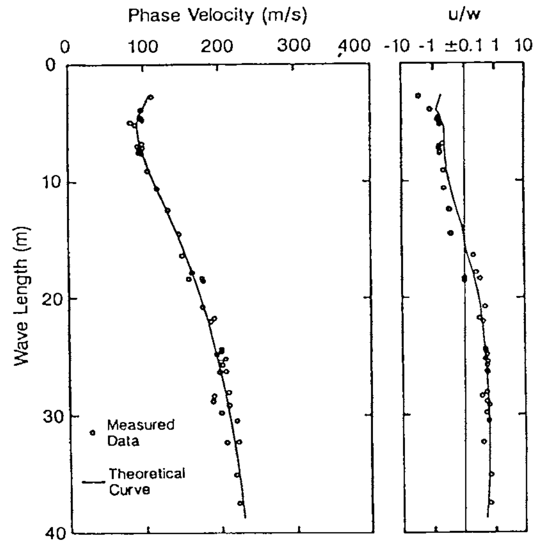


Fig. 10 Observed and Computed dispersion curves and u/w for Site B2 (Tokimatsu et al., 1991)

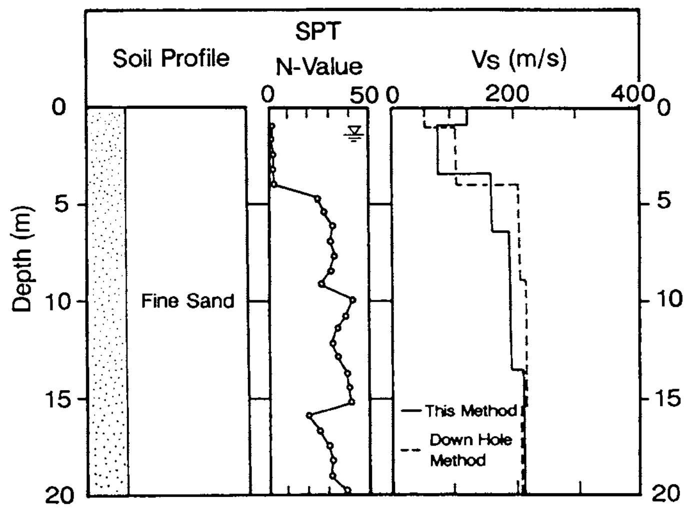


Fig. 11 Comparison of V_S-profiles determined by Rayleigh wave and downhole investigation at Site B2

Typical Rayleigh wave dispersion curve measured at Site B2 is shown in Fig. 10. The computed dispersion curve and amplitude ratio with wavelength for the inverted model is also shown in the figure for comparison. A good agreement between the computed and observed ones suggests that the inversion is successfully conducted.

The inverted shear wave velocity profile is shown in Fig. 11. For comparison, the shear wave and borehole logs of the site determined by conventional methods are also shown in the figure. The good agreement in shear wave velocity profiles obtained by different methods suggests that the proposed method is effective.

The liquefaction potential of the deposit is computed for each site, assuming the maximum horizontal acceleration of 0.16g. The safety factors obtained with depth at sites along the

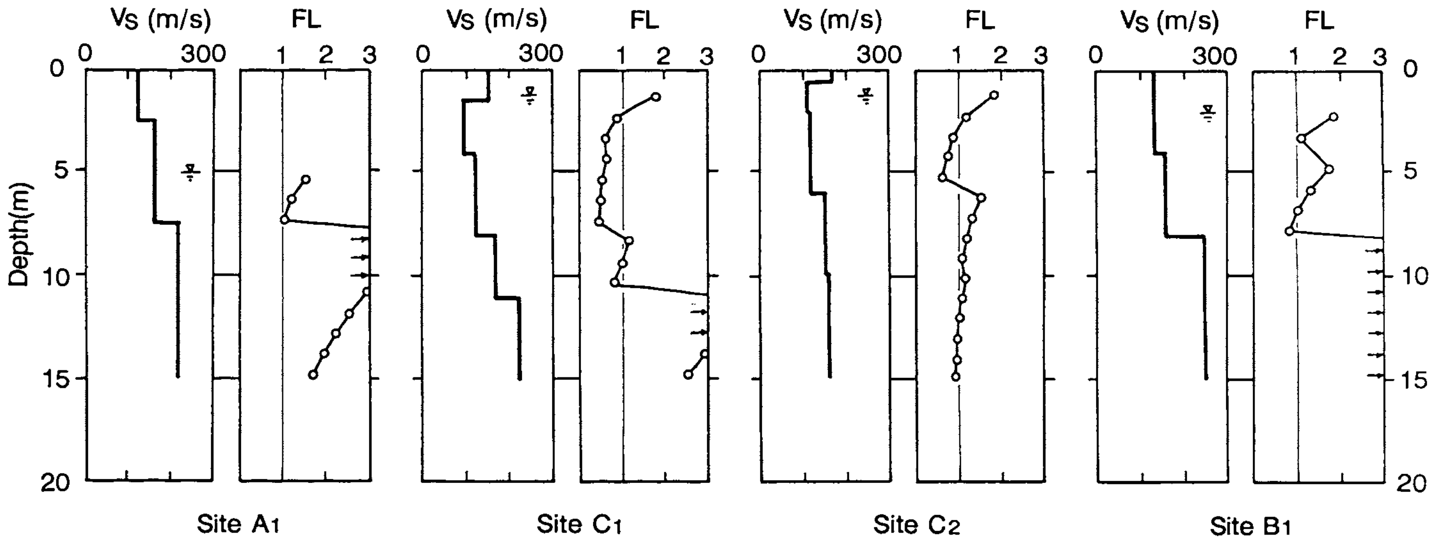


Fig. 12 Shear wave velocity profiles and factors of safety against liquefaction at sites along D-D' line

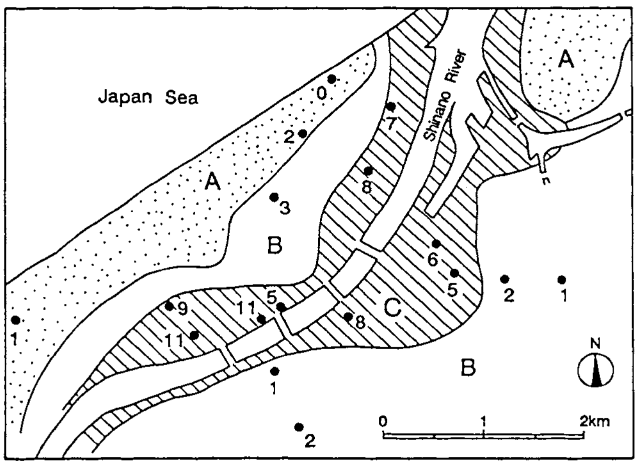


Fig. 13 Estimated thickness of liquefied layer in Niigata City superimposed on the zoning map of building damage during the 1964 Niigata earthquake

line D-D' in Fig. 9 are shown in Fig. 12 together with the shear wave velocity profiles. The safety factors at sites in Zone C at depth than shallower than about 10 m are significantly less than or about equal to unity, whereas the safety factors at sites in Zones A and B are slightly below unity only at a shallow depth and generally higher than unity.

The computed thickness of liquefied layer at each site is superimposed on the zoning map of building damage, and shown in Fig. 13. The estimated thicknesses of the liquefied layer are more than 5 m in Zone C, but less than a couple of meters in Zones A and B. These results appear consistent with the damage patterns of buildings during the earthquake.

CONCLUSIONS

A review of previous studies indicated that the liquefaction resistance of sands is uniquely related with the shear wave velocity which is normalized with respect to minimum void ratio and confining pressure. Based on the above findings, a simplified procedure was presented for estimating liquefaction potential. In the proposed method, Rayleigh wave investigation is used for determining shear wave velocity of the deposit. The applicability of the method was studied at 17 sites in Niigata city where field performance during the 1964 Niigata earthquake is known. The estimated results were reasonably consistent with the actual field behavior during the earthquake, indicating that the proposed method is effective.

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