Earthquake Engineering, Tenth World Conference © 1992 Balkema, Rotterdam. ISBN 90 5410 060 5

Uniform strain series equivalent to seismic strain

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ABSTRACT: To predict seismic settlement it is necessary to transform the irregular strain-time history imposed in the soil during an earthquake by an equivalent uniform series of strain cycles. The objective of this paper is to present such a transformation procedure. The method is based on the results of dynamic simple shear tests on undrained normally consolidated clay specimens subjected to five different wave patterns of cyclic loading. After completion of cyclic loading excess pore water pressures were allowed to drain from the tops of the samples and the settlement recorded. To check the accuracy of the proposed method predictions are compared with results from special simple shear tests in which the imposed strain time histories based on those observed in two reported earthquakes.

1 INTRODUCTION

Seismic settlement of clay layers was observed in the 1978 Miyagiken-oki earthquake (Suzuki, 1984) and the 1957 Mexico earthquake (Zeevaert 1972) and the 1985 Mexico earthquake (Jaime, Miguel and Mario 1987). Often such settlements are considerably larger than those resulting from secondary consolidation and consequently predicting their magnitude is of practical importance. For example, for a kaolinite clay, settlement strains produced by cyclic loading in simple shear tests can be of the order of 5 % (O-hara and Matsuda 1988). The magnitude depends on the cyclic shear strain amplitude, number of cycles and the overconsolidation ratio of the clay.

When using cyclic simple shear tests to estimate the seismic settlement it is necessary to transform the irregular strain-time history imposed in the soil during an earthquake by an equivalent uniform strain series. Several procedures for performing such a transformation are currently employed in liquefaction potential analyses.(Seed and Idriss 1971, Ishihara 1977, Annaki and Lee 1977) However these are only concerned with estimating the onset of liquefaction when effective confining stresses become very low. They are not applicable for seismic settlement analysis as such settlement may occur pre-liquefaction. The objective of the study described in this paper was to develop such a transformation process for estimating seismic settlement.

Based on experimental simple shear test results, a new method to transform irregular strain-time histories into equivalent uniform shear strain histories is described. The accuracy of the method is checked by using simple shear tests with constant cyclic strain histories to predict the behaviour of special tests in which irregular strain-time histories based on those observed in real earthquakes were applied.

2 CALCULATION PROCEDURE FOR EARTHQUAKE INDUCED SETTLEMENT

A procedure for calculating seismic settlement of clay layers has been presented by Matsuda and O-hara (1990). This consists of the following six steps:

1. Divide the soil layer thickness H into N_ℓ layers of thickness H_i.
2. Determine the shear moduli G_i, damping ratio h_i and densities ρ_i for each layer.
3. Calculate the irregular shear strain histories for each soil layer from a response calculation.
4. Evaluate the equivalent uniform strain amplitude, γ_{dyn} and corresponding number of strain cycles n for the above irregular strain histories using a transformation process.
5. Using the above γ_{dyn} and n and an empirical database, estimate the permanent seismic strain ϵ_i for each layer.
6. Obtain the total settlement of the soil layer using the following equation

$$\Delta H = \sum_{i=1}^{N_\ell} (\epsilon_i \times H_i) \quad (1)$$

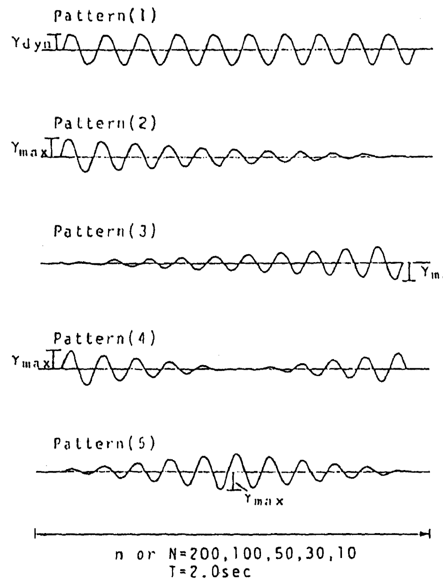

Figure 1. Cyclic strain patterns.

3 TRANSFORMATION OF AN IRREGULAR STRAIN-TIME HISTORY TO AN EQUIVALENT UNIFORM STRAIN HISTORY

3.1 Apparatus and test procedure

The apparatus used in this study was a servo-controlled electro-hydraulic dynamic simple shear test apparatus with a Kjellman type of shear box enabling soil samples 75 mm in diameter and 20 mm in height to be tested. (O-hara and Matsuda 1988) Kaolinite clay powder with a specific gravity G_s=2.70 was mixed with water to form a slurry and then one dimensionally consolidated to obtain soil samples. These had liquid and plastic limit of 53.5% and 28.5% respectively.

Undrained cyclic shear tests on normally consolidated clay specimens were performed using different cyclic strain-time histories (see below). After completion of undrained cyclic shear, excess pore water pressures were allowed to dissipate by allowing drainage from the top of the specimen. During the tests, horizontal displacement of the top of the sample and the pore water pressure at the bottom of the sample were measured.

Five different cyclic strain-time histories were used and these are shown in figure 1. Pattern(1) consists of a series of uniform (constant amplitude) shear strain, γ_{dyn}, cycles. (O-hara and Matsuda 1988) Pattern (2) consists of a series of shear strain cycles in which the amplitude decreases linearly with number of cycles. The maximum shear strain amplitude is γ_{max} which reduces to zero over N cycles. Pattern (3) is the reverse of pattern (2) with the strain amplitude increasing linearly from zero to γ_{max} over N cycles. Patterns (4) and (5) consist of a combination of patterns (2) and (3). In the experiments wave pattern (1) was generated using a function generator whereas patterns (2) to (5) were obtained using a data recorder. Tests were performed for each wave pattern and for each pattern tests were carried out in which γ_{max} or γ_{dyn} varied between 0.05 and 2.0% and the number of cycles n or N was either 10, 30, 50, 100 or 200.

In all tests the wave period was kept constant at 2.0 seconds. Evidence suggests that seismic settlement of clay layers is independent of wave period. (Matsuda, O-hara and Sano 1988)

3.2 Accumulated excess pore water pressure

The variation of the accumulated excess pore water pressure, u_{dyn}, at the end of a test normalized by the original vertical effective stress, σ_{vo}', with shear strain amplitude, γ_{dyn}, for tests with wave pattern (1) are shown in figure 2. Data for tests with different values of γ_{dyn} and number of cycles, n, are given.

The variation of the ratio u_{dyn}/σ_{vo}' with number of cycles for tests with wave patterns (2), (3), (4) and (5) are given in figure 3. All these tests had a total number of cycles N of 50 and approximately the same maximum shear strain, γ_{max}, of 1%. It is evident that while the different wave patterns effect the development of the excess pore pressure during the tests the final values of u_{dyn}/σ_{vo}' are approximately the same.

It was observed that for cycles with a shear strain amplitude below 0.05%, no excess pore water pressure was generated. Consequently any cycles with strain amplitudes below this threshold value did not contribute to the final accumulated excess pore water pressure.

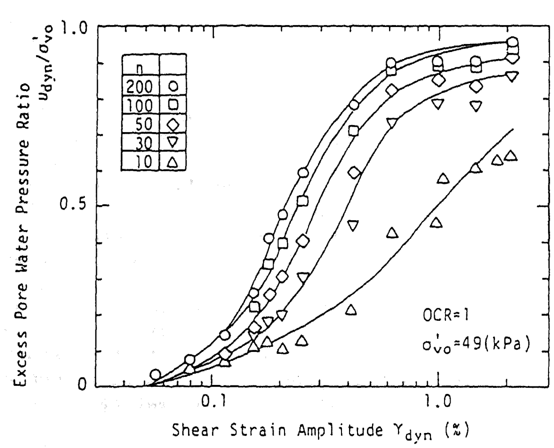

Figure 2. Relationships between γ_{dyn} and u_{dyn}/σ_{vo}' (by wave pattern (1)).

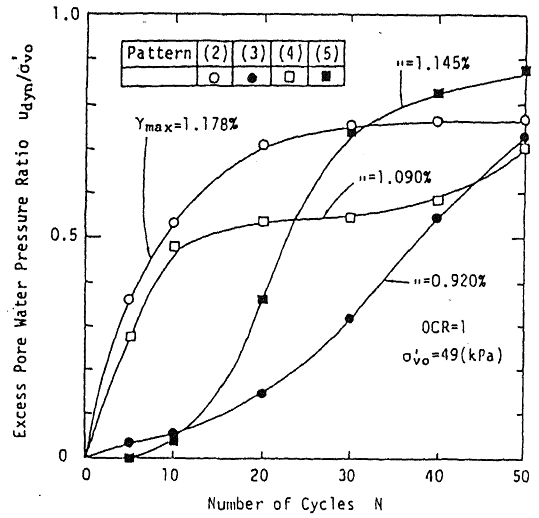

Figure 3. Relationships between the number of irregular strain cycles N and the accumulated excess pore water pressure ratio u_{dyn}/σ_{vo}'.

To compare the results from tests using wave patterns (2), (3), (4) and (5) with each other and with tests with uniform strain amplitude, pattern (1), it is necessary to define N' as the number of significant cycles in which the strain amplitude exceeds the threshold value. (Matsuda and O-hara 1989) For tests with wave pattern (1) all cycles had a strain amplitude greater than the threshold value and N'=n.

The accumulated excess pore water pressure for tests with wave patterns (2), (3), (4) and (5) are plotted against maximum shear strain amplitude, γ_{max}, in figure 4 for N' of approximately 40 (i.e. 40±10). This figure confirms that the accumulated excessive pore water pressure during N'=40±10 cycles of strain is

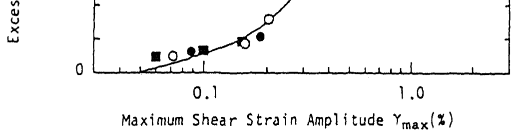

Figure 4. Relationships between γ_{max} and u_{dyn}/σ_{vo}' for different strain wave patterns.

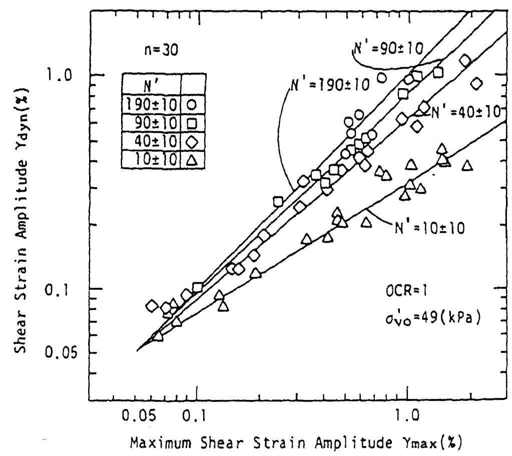

Figure 5. γ_{max} and γ_{dyn} for n=30.

approximately independent of the wave pattern and increases with magnitude of the maximum shear strain amplitude. Results from tests with N'=10±10, 90±10 and 190±10 also confirm the above findings.

3.3 Transformation of irregular strain time history to an equivalent constant cyclic strain series

The induced settlement of a clay layer due to cyclic shearing depends on the accumulated excess pore water pressure (O-hara and Matsuda 1988). This implies that a transformation procedure related to the magnitude of the accumulated excess pore water pressure is appropriate for estimating seismic settlement.

Such a transformation procedure can be derived from figures 2 and 4. For example by matching the u_{dyn}/σ_{vo}' values on figure 4 with those given on figure 2 the equivalent uniform strain amplitude, γ_{dyn}, for selected number of uniform cycles, n, and for different maximum shear strain amplitudes can be estimated.

This procedure has been used to obtain the relationship between γ_{max} and the equivalent γ_{dyn} for a uniform strain series with n=30 in figure 5 (symbols). As an example the data for N'=40±10 presented in this figure were obtained as follows. For each maximum shear strain amplitude, γ_{max}, the excess pore water pressure ratio u_{dyn}/σ_{vo}' was read from figure 4. The equivalent uniform strain amplitude, γ_{dyn}, for this value of u_{dyn}/σ_{vo}' and for n=30 was then obtained from figure 2. The data in figure 5 for other N' values, were obtained using a similar procedure but employing plots similar to figure 4 (for the appropriate N' value).

The straight lines in figure 5 were obtained using a least squares fit to the data obtained using the above procedure. As noted the threshold strain for excess pore pressure build up

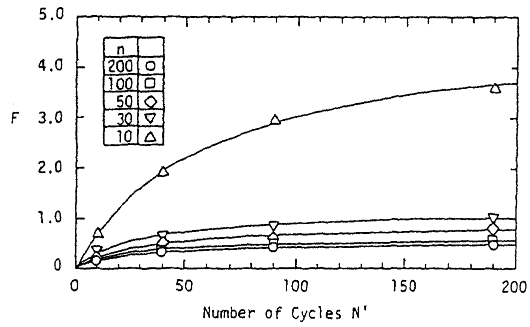

Figure 6. Parameter F and the effective number of cycles N'.

is 0.05%. Consequently the lines in figure 5 pass through the point (γ_{dyn}, γ_{max}) = (0.05, 0.05). Although there is some scatter in the results the logarithm of (γ_{dyn}) is to a first approximation proportional to the logarithm of γ_{max}. This gives the following equation.

$$\gamma_{dyn} = F \times (\gamma_{max})^G \quad (2)$$

where parameters F and G are evaluated as follows. Since Eq.(2) is satisfied at (γ_{dyn}, γ_{max}) = (0.05, 0.05), G is defined as follows;

$$G = 1 - \log F / \log 0.05 \quad (3)$$

The relationships between F and the effective number of cycles N' are shown in Fig.6; from which the following equation is derived.

$$F = N' / (H + I \times N') \quad (4)$$

where H and I are the constants and the values for current tests are shown in Table 1.

By using Eqs.(2), (3) and (4), the uniform shear strain amplitude γ_{dyn} equivalent to the irregular strain time history is evaluated.

3.4 Reliability of the transformation method

To investigate the reliability of the transformation method additional strain controlled cyclic simple shear tests were performed. In these tests irregular strain time histories based on either the Tokachi-oki Earthquake of 1968 or the El Centro Earthquake of 1940 were used. These histories were obtained by integrating the observed accelerograms twice. Several tests were performed using each strain-time history pattern but with the magnitude of the strain amplitudes scaled. The

Table 1. Constants H and I.

n	H	I
200	45.00	1.81
100	39.24	1.60
50	34.94	1.09
30	25.83	0.84
10	12.25	0.21

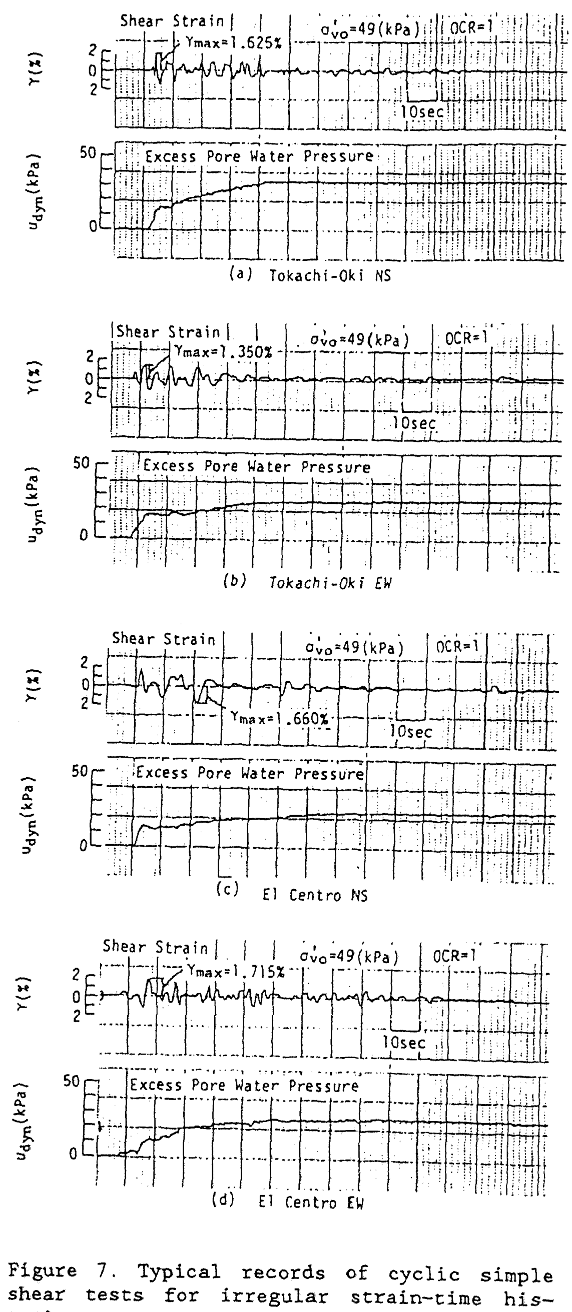

Figure 7. Typical records of cyclic simple shear tests for irregular strain-time histories.

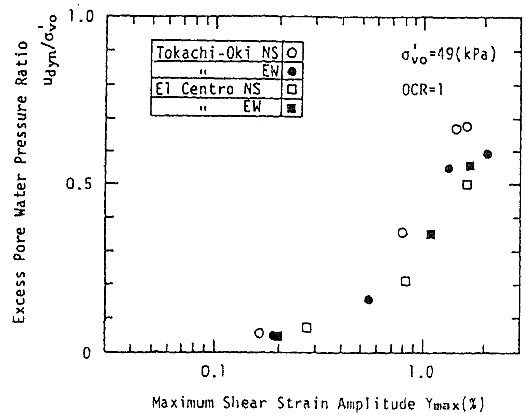

Figure 8. Relationships between γ_{max} and u_{dyn}/σ_{vo}' for irregular strain-time histories.

maximum strain amplitude ranged from 0.1% to 2%. Typical results from theses tests are given in figure 7 in the form of accumulated excess pore water pressures and strain amplitude versus time.

The accumulated excess pore water pressure at the end of these tests plotted against the maximum shear strain are shown in figure 8. Inspection of this figure indicates that the results are independent of the strain-time history pattern and are therefore in agreement with the results presented in figure 4.

The relationship between the stress reduction ratio SRR=$1/(1-u_{dyn}/\sigma_{vo}')$ and the change of void ratio for these tests is shown in figure 9. The solid line in this figure shows the average trend of the results. The settlement induced by the irregular strain-time history apparently depends on the excess pore water pressure accumulated during the undrained cyclic shear.

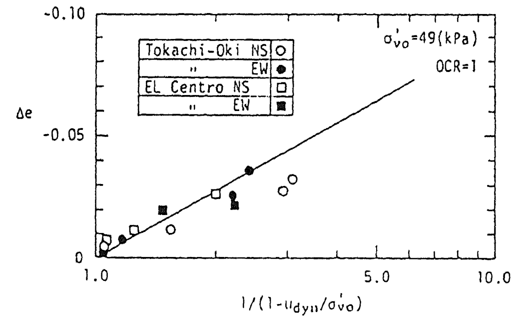

Figure 9. Change of void ratio and the stress reduction ratio.

The reliability of the proposed transformation method was checked by comparing the observed settlement of the sample of these special tests with those estimated using tests with a uniform strain wave pattern and the method outlined above. The estimated accumulated excess pore water pressure ratio u_{dc}/σ_{vo}' and settlement strain ϵ_{vc} are compared in Table 2 with those observed from the special tests. There later quantities are labelled u_{dyn}/σ_{vo}' and ϵ_v.

The errors in the estimated excess pore water pressure, u_{error}, and in the settlement ϵ_{error} are also included in Table 2. These are defined as;

$$u_{error}=(u_{dc}/\sigma_{vo}'-u_{dyn}/\sigma_{vo}')/(u_{dyn}/\sigma_{vo}') \times 100 \tag{5}$$

$$\epsilon_{error}=(\epsilon_{vc}-\epsilon_v)/\epsilon_v \times 100 \tag{6}$$

The results shown in Table 2 indicate that when γ_{max} is greater than 0.5% both u_{error} and

Table 2. Comparisons of the estimated result with observed one (Tokachi-oki EW).

γ_{max}(%)	N'	u_{dyn}/σ'_{vo}	ϵ_v(%)	n	γ_{dyn}(%)	u_{dc}/σ'_{vo}	u_{error}(%)	ϵ_{vc}(%)	ϵ_{error}(%)
2.055	28.5	0.588	1.656	200	0.452	0.750	27.6	2.055	24.1
				100	0.532	0.730	24.1	1.938	17.0
				50	0.724	0.720	22.5	1.886	13.9
				30	1.026	0.730	24.1	1.936	16.9
				10	3.605	0.834	41.8	2.659	60.6
1.350	23.5	0.549	1.198	200	0.318	0.653	18.9	1.566	30.7
				100	0.367	0.627	14.1	1.458	21.7
				50	0.476	0.603	9.9	1.370	14.3
				30	0.650	0.604	10.1	1.372	14.6
				10	1.917	0.691	25.8	1.738	45.1
0.550	12.0	0.151	0.365	200	0.139	0.325	115.4	0.583	59.6
				100	0.155	0.322	113.2	0.575	57.6
				50	0.181	0.297	96.9	0.522	43.1
				30	0.228	0.290	91.8	0.506	38.7
				10	0.467	0.281	85.9	0.488	33.7
0.190	3.5	0.048	0.138	200	0.057	0.020	-58.3	0.030	-78.3
				100	0.061	0.025	-47.9	0.037	-72.8
				50	0.065	0.020	-58.3	0.030	-78.3
				30	0.074	0.030	-37.5	0.045	-67.3
				10	0.106	0.051	6.3	0.078	-43.8

ϵ_{error} are between 10% and 60%. The errors are smallest when the value of n approaches the effective number of cycles N'. In this case the estimated settlement is greater than that observed. When the maximum shear strain, γ_{max}, is smaller than 0.5% the errors for both excess pore water pressure and settlement exceeds 100%. However it should be noted that in these cases the observed settlement is very small, being in the range 0.1% to 0.2%. Such small seismic settlements are unlikely to have any great engineering significance.

4 CONCLUSIONS

In this paper a new method to transform irregular strain histories into equivalent uniform strain series has been presented. This method was based on simple shear tests using five different wave patterns. The reliability of the method was checked by comparing predictions with results of special simple shear tests using strain-time histories based on real earthquake accelerograms. The following conclusions are made:

1. When a specimen is subjected to cyclic shear strains with the same maximum shear strain amplitude and with the same effective number of cycles, the accumulated excess pore water pressure during cyclic shear converges to the same value irrespective of the shear strain wave patterns.
2. There is a linear relationship between the logarithm of the maximum shear strain amplitude and the logarithm of the equivalent uniform shear strain amplitude.
3. The accumulated excess pore water pressure and the settlement in strain induced by the irregular strain time history can be estimated by using the maximum shear strain amplitude and the effective number of cycles.
4. By bringing the value n closer to the effective number of cycles N', any errors in the settlement estimation decrease to a minimum value.
5. The estimated settlement in strain is somewhat greater than the observed and therefore the method is conservative.

ACKNOWLEDGEMENTS

The authors express their gratitude to Emeritus Prof. Sukeo O-hara and Dr. Dave M. Potts for their valuable comments.

REFERENCES

Annaki, M. & K. L. Lee. 1977. Equivalent uniform cycle concept for soil dynamics. Journal of the Soil Mechanics and Foundation Div., ASCE, 13, GT6: 549-563.

Ishihara, K. 1977. Pore-water pressure response and liquefaction of sand deposits during earthquakes. Proceedings, International Symposium on Dynamic Methods in Soil and Rock Mechanics, Plastic and Long-term Effects in Soils: 161-193: Balkema.

Jaime, A., P. R. Miguel, & J. R. Mario. 1987. Seismic induced settlement in a building. 8th Pan-American Congress on Soil Mechanics and Foundation Engineering: 257-275.

Matsuda, H., S. O-hara, & S. Sano. 1989. Settlement and excess pore water pressure of saturated clay induced by cyclic shear with different periods. Proceedings, 9th WCEE, III: 83-88.

Matsuda, H. & S. O-hara. 1989. Threshold strain of clay for pore pressure buildup. Earthquake Geotechnical Engineering, 12th ICSMFE: 127-130.

Matsuda H. & S. O-hara. 1990. Geotechnical aspects of earthquake-induced settlement of clay layer. Marine Geotechnology, 9: 179-206.

O-hara, S & H. Matsuda. 1988. Study on the settlement of saturated clay layer induced by cyclic shear. Soils and Foundations, 28, 3: 103-113.

Potts, D. M. 1985. Behaviour of clay during cyclic loading. Developments in Soil Mechanics and Foundation Engineering 2:105-138.

Seed, H. B. & I. M. Idriss. 1971. Simplified procedure for evaluating soil liquefaction potential. Journal of the Soil Mechanics and Foundation Div. ASCE 97 SM9: 1249-1273.

Suzuki, T. 1984. Settlement of saturated clays under dynamic stress history. Journal of the Japan Society of Engineering Geology, 25, 3: 21-31. (in Japanese)

Zeevaert, L. 1972. Foundation Engineering for Difficult Subsoil Conditions. 522-523: New York: Van Nostrand Reinhold.