Practical Method of Predicting Sand Liquefaction

La Méthode Pratique pour la Prévision de la Liquéfaction des Sables

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SYNOPSIS A simplified and practical method of predicting sand liquefaction is presented. The method consists of primary and secondary predictions. The primary prediction is performed with special attentions to soil characteristics such as density, depth and etc., obtained in general soil surveying. In this prediction, it will be made clear whether or not liquefaction may occur, or that further response analysis may be required for the assurance. For the last case, the secondary prediction, that is, analytical method is to be taken. The simplified method proposed by Seed et al. is adopted with some modifications in the procedures by comparing with accurately performed earthquake analysis on actual models of different geological and input wave conditions. The results may be put in forms of liquefaction, no liquefaction or possibility of liquefaction. The above prediction procedures have been applied to actual examples of liquefaction with high accuracy.

INTRODUCTION

Many attentions have been paid on sand liquefaction problems especially after the Niigata earthquake in 1964 when severe hazards were caused due to liquefaction and many studies have been conducted on how to predict sand liquefaction during earthquakes. The mechanism and behaviour of sand liquefaction are so complicated that there remain many uncertainties, despite of multi-processed investigations in the past. From the plactical view point, however, simplified and reasonable prediction method, if possible, is preferable based on experiential and analytical information.

In the bridge design specification (1971) of Japan Road Association (JRA), criteria of liquefaction is first employed based on field observations during earthquakes and laboratory experiments. The principal parameters adopted in the specification are N-value of standard penetration test, particle sizes of sand and depth of the layer in question. The design standard (1974) of Architectural Institute of Japan (AIJ) also describes the requirements of soil properties and geological conditions with respect to liquefaction problems.

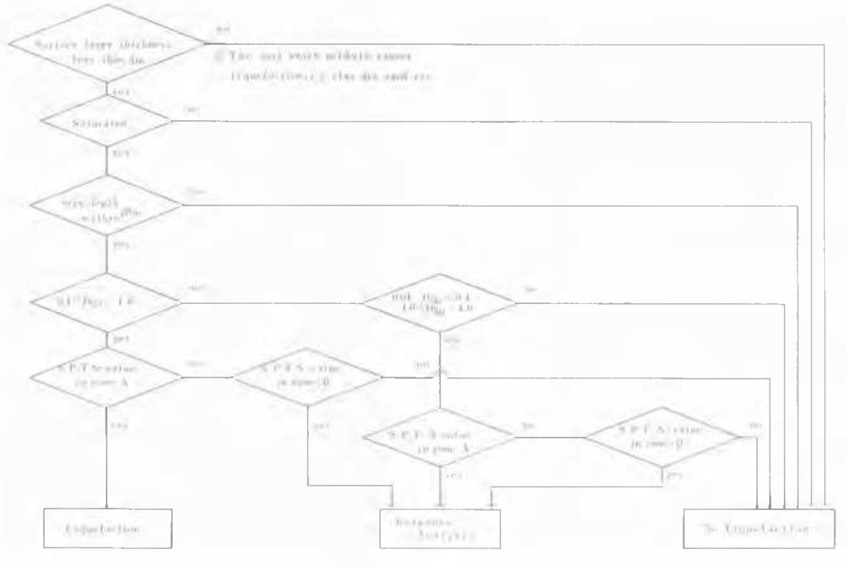


Fig. 1 Primary prediction by geological survey

In the present report, a newly proposed method of predicting liquefaction is presented. The method consists of two steps; primary prediction by geological survey and secondary prediction by response analysis through which the accuracy of liquefaction prediction is improved. The present method is believed to be significant, since those experiential and analytical information on liquefaction problems ever reported are taken into account and are reorganized to construct a simplified predicting method for practical purpose.

PREDICTION BY GEOLOGICAL INVESTIGATION

The first step of prediction is illustrated in Fig. 1 where required data are all known from usual soil surveying. The term, surface layer in the chart indicates a layer of clay and dry sand which is not subject to liquefaction. Ishihara (1973) studies data of liquefaction and concluded that in the case of surface layer with more than 3 meters, no liquefaction was observed during earthquake. This observations are taken into account in the chart.

Many reports indicate the critical depth to be approximately 15 to 20 meters below the ground surface. The greater the depth is, the larger becomes the confining stress, and consequently the less susceptible is the possibility of liquefaction. In the chart 20 meters is assumed as the critical depth.

Grain size and its distribution have been considered important factors on this problem; sand of poor grading with particular grain size shows high possibility of liquefaction. JRA specification for example rules that the sand of coefficient of uniformity less than 5 and D_{20} between 0.04 and 0.5 mm will liquefy if other conditions are satisfied. The authors studied dynamic compression test data on the effect of grading of which results are given in Fig. 2, where the data are compared under the identical testing condition.

First, the effect of uniformity coefficient is investigated by noting that the coefficient is one of the indication of grain size distribution. As seen from the figure, no clear effect of grain size distribution in terms of coefficient of uniformity upon liquefaction can be observed. In some cases, soils of high coefficient of uniformity show remarkably lower liquefaction stress as is contrary to our intuitions.

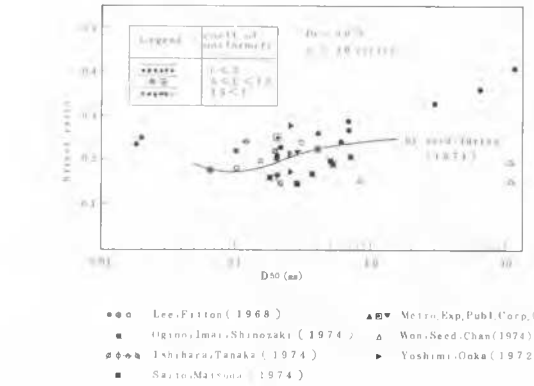


Fig. 2 Stress ratio and D_{50}

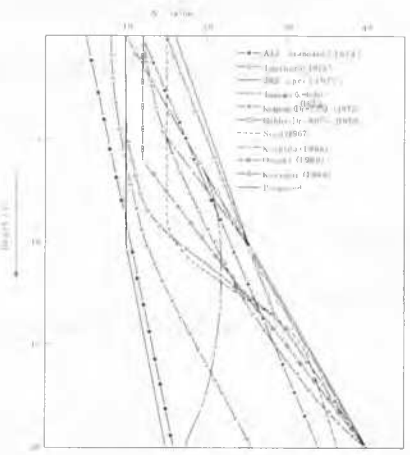


Fig. 3 S.P.T. N-value and depth

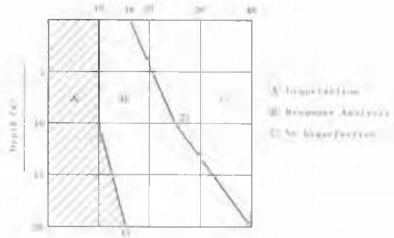


Fig. 4 Proposed critical N-value

As for the particle size, soil of medium grain size seems to be subject to liquefaction as seen from the figure. In this respect, the proposal by Seed and Idriss on mean grain size and stress ratio is quite reasonable.

After all, it may be concluded that the liquefaction potential can be better represented by mean grain size than by coefficient of uniformity. The appropriate range of mean grain size should be estimated both from laboratory tests and field observations in liquefied sites during earthquakes.

However one should keep in mind that a question still remains on the effect of fine grain content in sand. Generally sand in alluvial deposit contains significant amount of fines, but only few samples have ever been tested. Therefore, no conclusion has been obtained up to date.

The most important parameter with respect to soil conditions is N-value by the standard penetration test. Some reports are available on the relations between the critical N-value and depth which were obtained in experiential and analytical ways (Fig. 3). However, since the proposed values are rather widely distributed, one can hardly draw an average line. Therefore three zones are assumed as in Fig. 4; the zone-A implies that liquefaction will definitely take place if other conditions are satisfied while zone-C will not. The zone-B holds uncertainties and reserve further considerations.

PREDICTION BY RESPONSE ANALYSIS

Response analysis is conducted on the case for which the prediction was reserved in the preceeding section in order to make further detailed considerations with the aid of stress parameters in addition to the geological properties. The analytical procedure by Seed et al. is adopted in principle, and shear stress τ_E in soil element is compared with stress τ_L which causes liquefaction. In order to evaluate the stress τ_E, response analysis such as finite element method or one-dimensional wave propagation theory are employed in general. However, for the simplicity the equation by Seed and Idriss (1971) is herein employed.

The accuracy of the equation highly depends on the determination of the coefficient r_d. The best way to evaluate appropriate value of r_d is to perform the dynamic response analysis on ground models by FEM or wave propagation approach, and to recalculate r_d so that simplified equation can be adjusted to the theoretically calculated results. Then the authors conducted the dynamic response analysis on ground models by one-wave propagation theory. The ground models in which authors are interested are alluvial plain ground. The very soft clayey soil with thickness of several tens meters rest on the base rock or gravel bed, and loose sand layer which are in question for liquefaction are seen on the soft clay strata. This type of ground as alluvial plain is very common not only in Tokyo area but all over in Japan. Six ground models are picked up noting the bed rock depth, thickness and density of sand layer etc. as seen in Fig. 6.

Generally speaking, the calculated results of dynamic response analysis are greatly affected by assumed soil properties and in-put external forces. Some of the important factors in this respect are shear modulus and damping factors which are known to be dependent on not only strain level but also confining stress level. Iwasaki and Tatsuoka (1975) worked out this problems, the results of which are used in the analysis. In-put wave characteristics are also dominant factors. Two earthquake waves are used; Kushiro-S-634 and Tokyo 101, the former as a cyclic type of long period wave contents and the latter as a shock type of short period wave contents. As for the acceleration level, the maximum values are taken respectively 50, 100 and 150 gal at the top of bed layer.

Taking above factors into accounts, the response analysis were performed to recalculate r_d. The results are shown in Fig. 8. Differently from the result by Seed et al., the range obtained herein becomes slightly lower and narrower as far as the ground models employed here are concerned, and the average curve is obtained as in the figure.

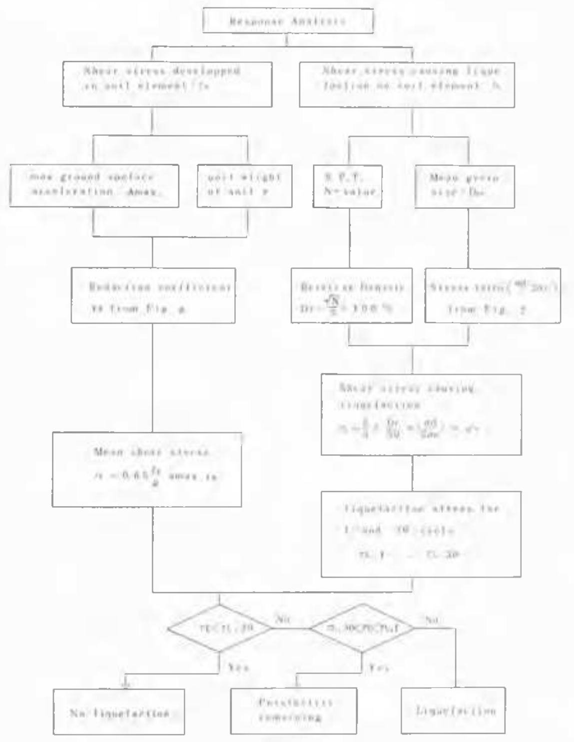


Fig. 5 Secondary prediction by response analysis

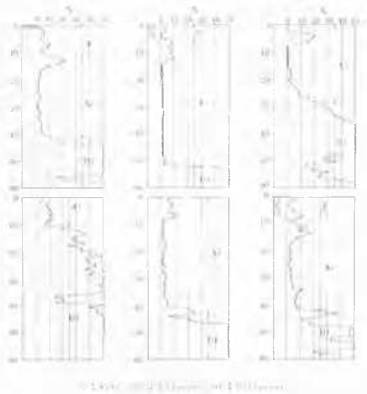


Fig. 6 Soil profile of ground models

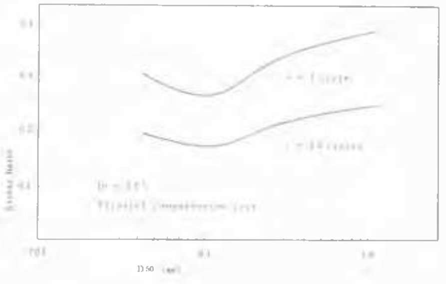


Fig. 7 Proposed stress ratio and D_{50}

The stress τ_L to cause liquefaction is easily determined from the relationship between the stress ratio and mean grain size in Fig. 8 and from N-value by which relative density is estimated.

Fig. 7 is the reproduction of Fig. 2 and the 30-cycle stress line is identical to that of Seed et al. but the 1-cycle line is newly introduced here. As commented before, these average lines are obtained from triaxial test data. However, recent researches on the problem show that torsional testing method is better fitted rather than triaxial test. Since available data of torsional tests are limited, study on this kind of data remains as a future problem.

As for the relative density and N-value, Tsuchida proposed an approximate relation as $N=25\times D_r^2$ which is more simple than Gibbs' proposal and, noting the accuracy of relative density itself, is to be used for the relative density estimation.

The final judgement is conducted comparing the average dynamic stress τ_E with both τ_L, 1 and τ_L, 30 as seen in Fig. 9. Since many uncertainties are involved in the process of prediction, it is less productive to discuss the equivalent numbers of loading corresponding to the random wave of earthquakes. Therefore 1- and 30-cycles are taken into account as the critical numbers and it is assumed that the sand in question is left uncertain for liquefaction if the shear stress τ_E falls between the two stress range.

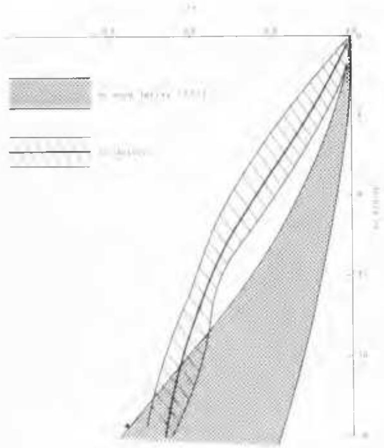


Fig. 8 Reduction factor r_d

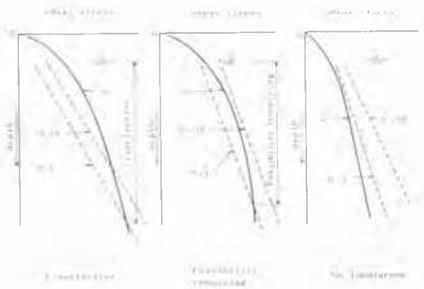
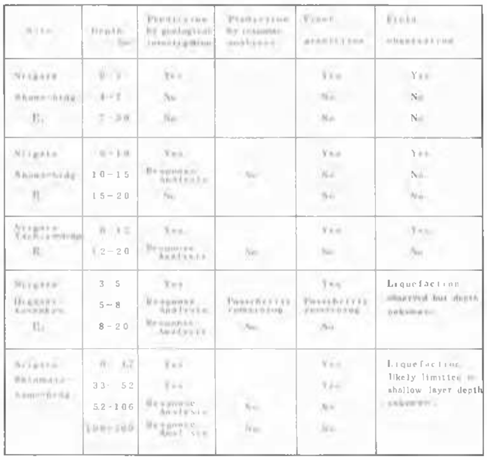


Fig. 9 Final judgement by response analysis

Table I Application of the method



Site	Depth (m)	Prediction by geological investigation	Prediction by response analysis	Final prediction	Field observation
Niigata [illegible]	0-3	Yes		Yes	Yes
	4-7	No		No	No
	7-20	No		No	No
Niigata [illegible]	0-10	Yes		Yes	Yes
	10-15	Response Analysis	No	No	No
	15-20	No		No	No
Niigata [illegible]	0-12	Yes		Yes	Yes
	12-20	Response Analysis	No	No	No
Niigata [illegible]	3-5	Yes		Yes	Liquefaction observed but depth unknown.
	5-8	Response Analysis	Possibility remaining	Possibility remaining	
	8-20	Response Analysis	No	No	
Niigata [illegible]	0-33	Yes		Yes	Liquefaction likely limited to shallow layer depth unknown.
	33-52	Yes		Yes	
	52-106	Response Analysis	No	No	
	106-200	Response Analysis	No	No	

APPLICATION OF THE METHOD TO ACTUAL EXAMPLE

The method mentioned above is applied to actual earthquake examples, on which soil properties and field observations in detail are provided by Public Works Research Institute (1965). The results are shown in Table I showing satisfied accuracy of prediction not only for the site but also even for the depth of liquefaction.

CONCLUSIONS

The predicting method for sand liquefaction presented in this paper is easy to be performed and has satisfactorily high accuracy. This present method is significant because it takes into account both geological conditions and analytical parameters through which the accuracy is improved. The method is successfully applicable especially to soft alluvial ground with deep bed layer.

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