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A NEW REALISTIC APPROACH FOR LIQUEFACTION ANALYSIS

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ABSTRACT

Liquefaction of saturated sands has often been the main cause of catastrophic damage to structures resulting in loss of life and property. If a foundation soil is liable to liquefy during an earthquake, additional strengthening of the superstructure is not of any help in the event of an earthquake. This has been amply demonstrated during Niigata Earthquake. of 1964 and earlier in Nepal Bihar earthquake of 1934. It is therefore extremely important that the possibility of liquefaction of sand during an anticipated earthquake must be determined and a suitable treatment of foundation soil if necessary be carried out before construction. Since these measures are often costly and time consuming it is very necessary to have a realistic approach of liquefaction analysis.

An investigation has been carried out by the authors on horizontal vibration table to determine if a particular soil deposit would liquefy under an anticipated ground motion and a new method of analysis has been developed. This has been shown to be quite a realistic and reliable approach. Two sites has been analysed. At one site the sand had liquefied while at the other site, the sand had not liquefied during the 1964 Niigata earthquake. The predicted results tally extremely well with the observed behaviour. Such a prediction has been possible for the first time and the method may replace the Seed and Idriss (1971) approach which is at best approximate.

Key Words : Analysis, Earthquakes, Liquefaction, Pore Water Pressure, Sands, Vibrations

INTRODUCTION

Liquefaction of saturated sand has often been the the main cause of catastrophic damage to structures resulting in loss of life and properry. This has been amply demonstrated during the Niigata Earthquake of 1964 and the Nepal Bihar earthquake of 1934. In saturated sandy soils mud fountains originating from certain depths below the ground have been observed with a time delay of several minutes after the main shock during an earthquake. In the Niigata earthquake of 1964 many structures settled more than Im and the settlement was often accompanied by severe tilting. During Nepal Bihar earthquake of 1934, bridge piers and abutments subsided by about 3 metres. Since the liquefied soil mass behaves like a viscous fluid, light structures like timber piles, septic tanks and manholes are floated up.

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Large landslides also have been observed during the earthquakes on account of liquefaction of foundation soil. All these observations bring out that the soil had lost its bearing capacity and behaved like a viscous fluid.

Therefore in a seismic area, if a super structure is designed for seismic forces, the additional safety in its design is not of any help in the event of liquefaction of soil during an earthquake. Hence it is extremely important that the possibility of liquefaction of sand during an anticipated earthquake must be determined and a suitable treatment of foundation soil if necessary be carried out before construction. Since these measures are often costly and time consuming it is very necessary to have a realistic approach of liquefaction analysis.

The shear strength of sandy soil is due to internal friction and effective stresses only. In saturated sand under dynamic loads the pore water pressures increase and the effective stresses are reduced due to transfer of ntergranular stress to pore water. This results in reduced strength. If ithis transfer is complete, there is complete loss of strength and complete liquefaction occurs. If only partial transfer of stress takes place there is only partial liquefaction. The above definition of liquefaction has been used in the present investigation.

An investigation has been carried out by the authors on horizontal vibration table to determine if a particular soil deposit would liquefy under an anticipated ground motion and a new method of analysis has been developed. This has been shown to be quite a realistic and reliable approach. The soils data of Niigata Earthquake from two sites has been analysed using the proposed method. At one site the sand had liquefied while at the other site, the sand had not liquefied during the 1964 earthquake. The predicted results tally extremely well with the observed behaviour at both the sites. Such a prediction has been possible for the first time and the method may replace the Seed and Idriss (1971) approach which is at best approximate.

It is further shown that the Horizontal Vibration Table Test has quite a potential to be used as a standard test and may be adopted by the Standards Institutions in countries where seismic activity is considerable.

TEST SETUP AND TEST PROCEDURE

General

The stresses induced in the ground by an earthquake are generally considered to be primarily due to the upward propagation of shear waves in the deposit although other forms of wave motion are also expected to occur and an element of sand is subjected to horizontal shear stresses which reverse in direction many times during an earthquake. Therefore horizontal vibration in laboratory are considered to represent the field conditions to a better degree in comparison to vertical vibrations and the present investigations were carried out on horizontal vibration table.

The study carried out is not considered as a model study and no model laws are considered. The test has been considered to represent a sample at some depth in the field and is required to be subjected to vibration conditions as expected in the field. The predominent frequencies in many earthquakes in alluvial deposits have been observed to be of the order of 2 to 5 cycles per second. The present investigations were carried out at a frequency of 5 cps. Two types of tests were performed (i) Tests with zero surcharge, (ii) Tests with initial surcharge.

Test Set up

The test sample of saturated sand deposit was prepared in a test tank of size 1.05m × 0.4m high by depositing dry sand under water The tank is mounted on a horizontal vibration table which can be excited at known accelerations under independently controlled conditions of amplitude and frequency. The amplitude could be varied from 0 to 1,0 mm in this table. The tests were carried out at accelerations in the range of 0 to 0.5g which are most likely to occur in field. The whole set up is shown in Figure 1 for carrying out tests under zero surcharge condition which represented a sample near surface.

For tests under initial surcharge condition to repesent a sample at depth, the surcharge was applied with precast concrete blocks attached rigidly to a steel plate (Fig 2). The device permitted partial drainage on the sides in the test.

For measuring the pore pressure in the deposit, simple glass tube piezometers were used Since there is a time lag the observation were corrected with the help of an automatic pore pressure transducer developed for this purpose. The pore pressures were measured in the centre of the sample at 6 cm, 15 5 cm and 25 cm depth from the surface of the deposit. Height of the sample was about 26 cm.

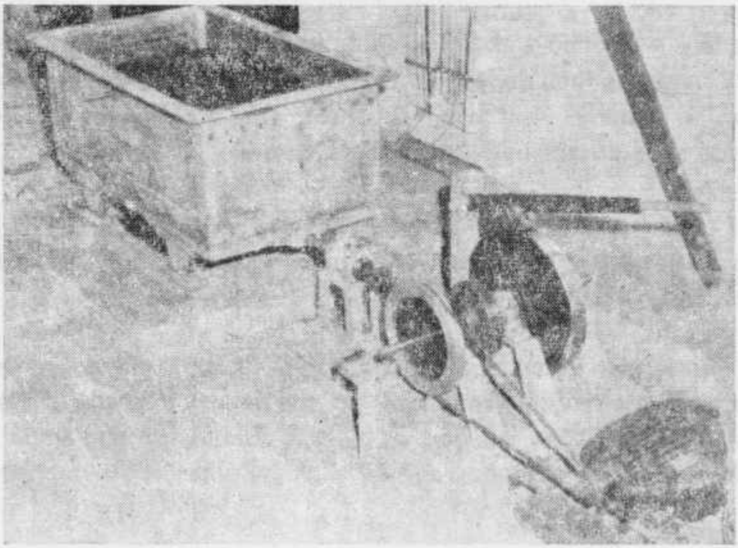

Fig. 1—Test set up

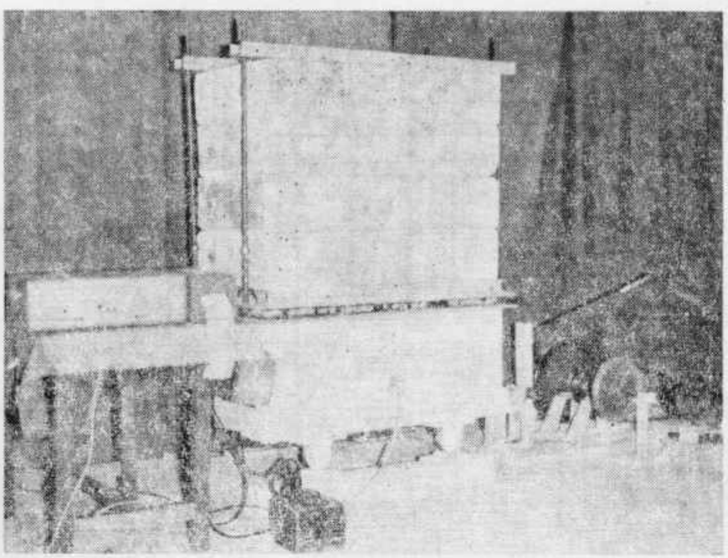

Fig. 2—Dead weight surcharge test

Test Procedure

The tank was filled with a known quantity of water and known weight of air dry sand was then deposited to obtain uniform density and desired height of the sample. Any water overlying the deposit was removed and weighed to compute the initial relative density.

(i) Tests with zero surcharge : The tank with saturated soil sample was excited at desired amplitude and frequency and predetermined cycles of motion were imparted and pore pressure increase was observed.

(ii) Tests with initial surcharge-After the sample was prepared in the tank the surcharge device was placed on the deposit and the desired dead weight applied. It was vibrated at desired frequency and amplitude for predetermined cycles of motion and excess pore pressures were measured.

Soils Tested

The tests were carried out on a fine sand which is locally available from the Solani River. Mean grain size of the sand is 0.15mm and its uniformity coefficient is 1.9. According to Indian Standard Classification the sand belong to S.P. Group. The sand was particularly selected in this study since the grain size characteristics are very close to those of Niigata (Fig. 3).

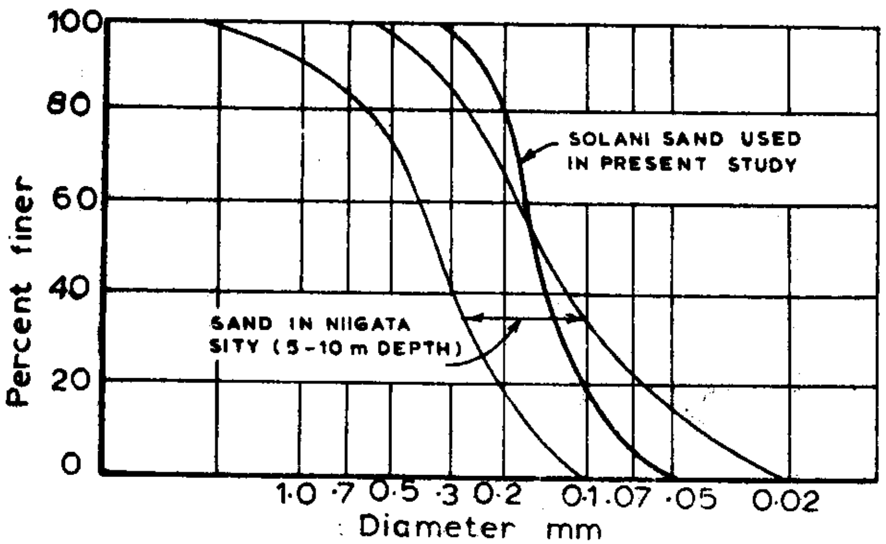

Fig. 3—Comparison of grain size distribution

TEST RESULTS AND INTERPRETATION

Behaviour of Loose Sand

(a) Test without Surcharge

Figure 4 shows the time wise increase and dissipation of excess pore water pressure during vibrations. It can be observed that in first three cycles there was no change in pore water pressure and it increased to the maximum in about 10 cycles of motion. The maximum pore pressure for Solani sand (a fine sand) remained constant for about 35 seconds (175 cycles) and then started dissipating slowly. Since this sample represents a condition near the surface in the field, the pore pressures may be expected to remain constant for about 35 seconds before they start dissipating.

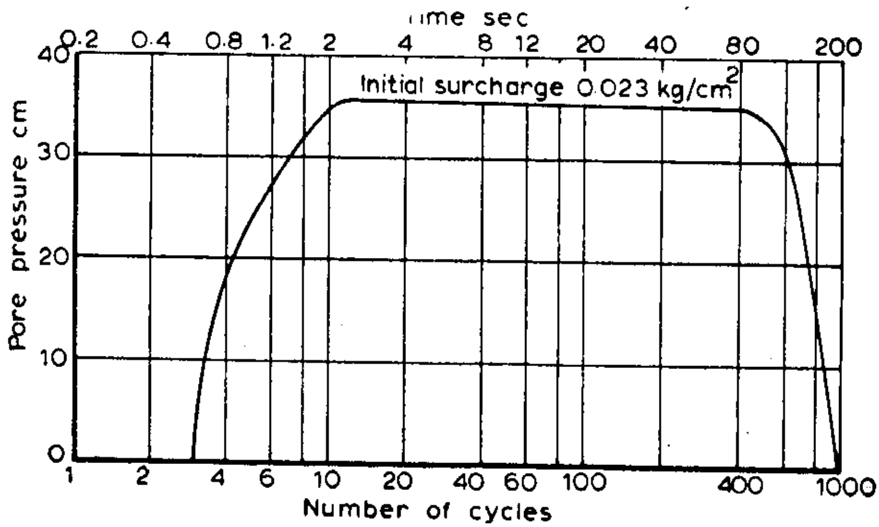

Fig. 4—Pore pressure vs number of cycles

Figure 5 shows the maximum pore pressure versus acceleration at relative density of 20% and at a depth of 25 cm, 15.5 cm and 6 cm from the surface. The values of ratio of excess pore pressure ($u = \gamma_w h$) to initial effective over burden pressure $\bar{\sigma_1}$ are also plotted. A value of unity of this ratio represents complete liquefaction.

It is seen from this figure that the pore pressure increases with increase in acceleration initially and becomes constant with further increase in acceleration at all the three depths. Also the values of $\frac{u}{\bar{\sigma_1}}$ at all the three

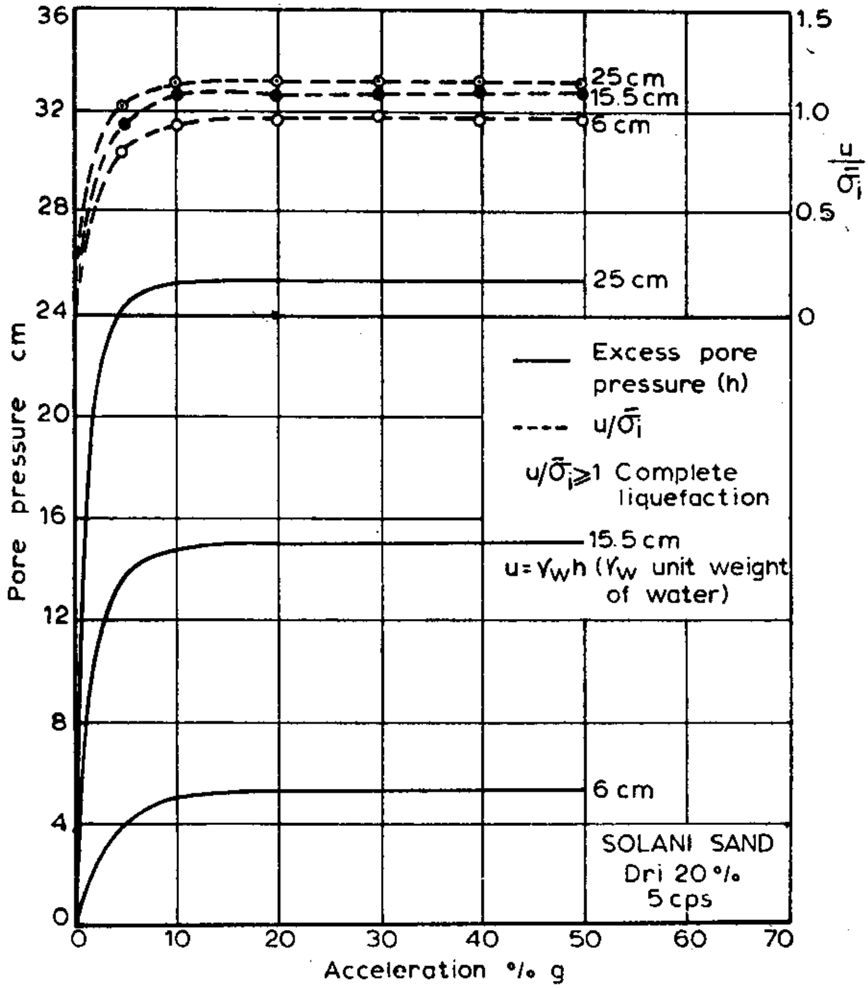

Fig. 5—Pore pressure vs acceleration

depths at accelerations larger than about 5% g is close to unity. This shows that complete liquefaction had occured in the soil mass. It was also observed that the sand water mixture started ejecting out in the form of a fountain upto a height of about a centimeter above the top surface of the sample. Therefore the horizontal vibration table test depicts the field behaviour to a reasonably good extent.

Figure 6 shows the effect of relative density on increase in pore pressure at 10%g. It is observed that with increase in initial relative density the excess pore pressure under vibrations decreases. In this case

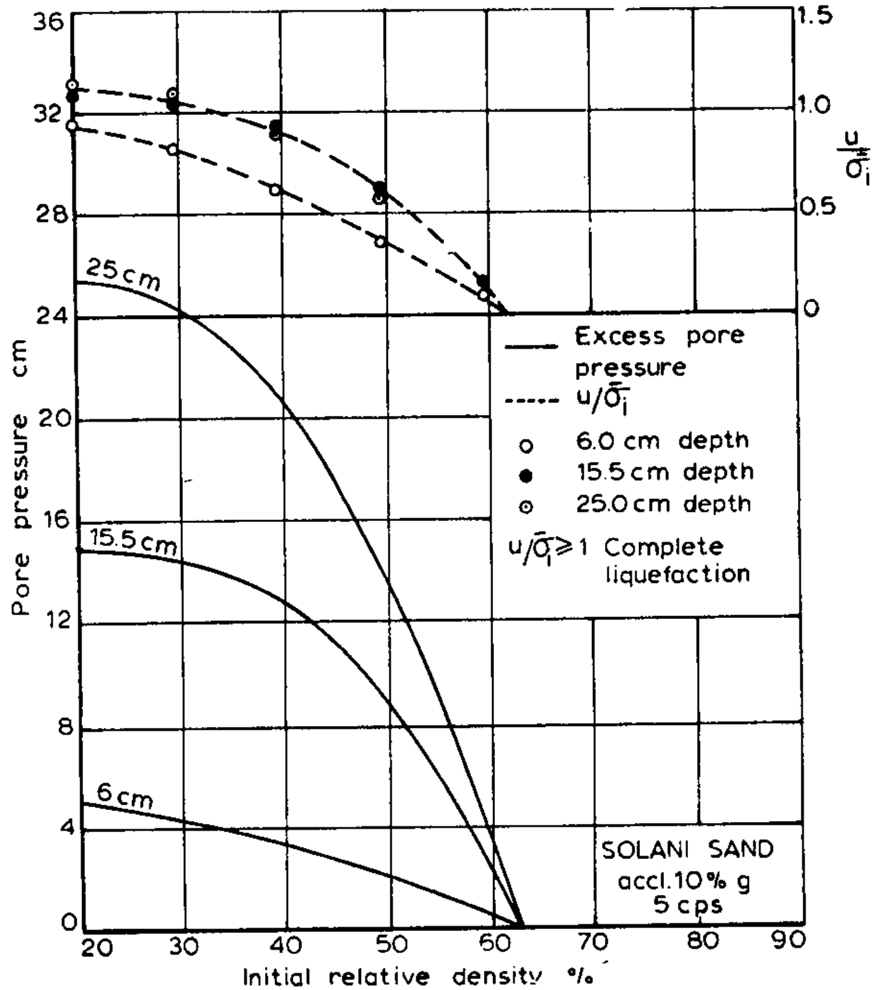

Fig. 6-Pore pressure vs initial relative density

no pore pressure increase was observed if the initial relative density was equal to 62%. The corresponding values of initial relative density for acceleration 20%g 40%g and 50%g were 62.5%, 66% and 66.5% respectively (Gupta 1977). Thus the chances of liquefaction reduce with increase in initial relative density and it may not liquefy if sand is at relative density of more than 65%.

(b) Tests with initial surcharge

The development and dissipation of pore pressure with initial effective surcharge of 0.023 kg per cm^2 is shown in Figure 7. This surcharge corresponds to a thickness of soil cover of 23 cm above the top of the specimen

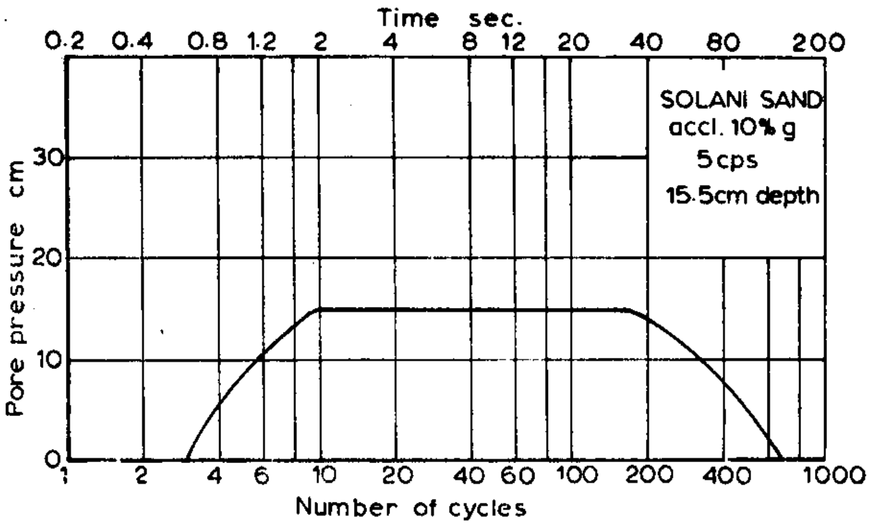

Fig 7—Pore pressure vs number of cycles

in the tank. Thus the conditions at 25cm depth in tank correspond to a depth of 0 48m in the field if submerged unit weight of deposit is considered to be unity.

The maximum pore pressure developed in 10 to 12 cycle and remained constant for about 90 second (450 cycles) before it started dissipating. In laboratory it provided partial drainage but in field undrained conditions might prevail. Therefore it is likely that the pore pressures may remain constant for more than 90 seconds in field if the vibrations were continued for that time. The corresponding time was 35 seconds in tests without surcharge. In most cases however the duration of earthquakes in alluvial deposits has been observed to be 20 to 40 seconds. Therefore it is expected that the excess pore water pressure developed will remain constant during the entire duration of the earthquake

Similar tests were performed with different values of initial effective surcharges (Gupta 1977). From this data, Figure 8, which shows the effect of overburden pressure on the increase in pore pressure, has been prepared. The excess pore pressure is plotted against effective overburden pressure in this figure. It is observed that pore pressure increases with increase in over burden pressure upto about 200 g/cm but with further increase in effectiv eoverburden it started decreasing. The line A B represents "complete liquefaction".

During the Niigata earthquake of 1964 in heavily damaged zone, large

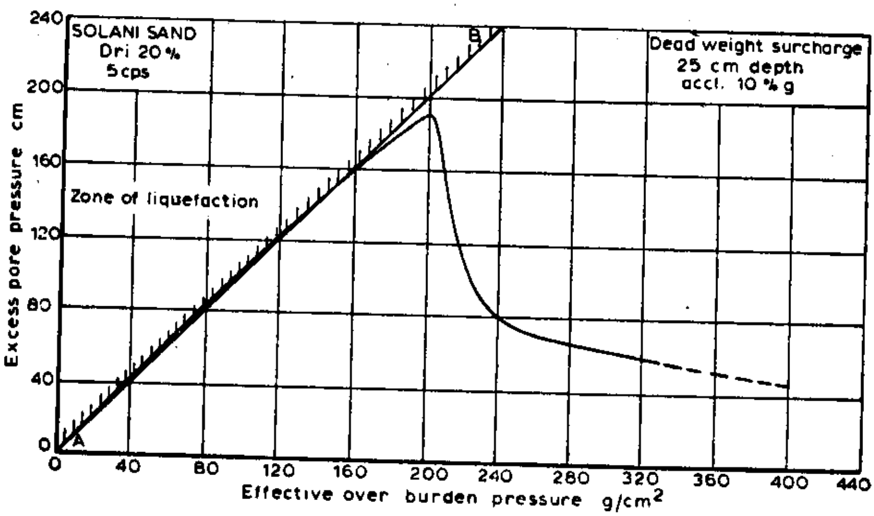

Fig. 8—Excess pore pressure vs effective overburden pressure

amount of liquefaction was observed throughout except in a location where there was a dry fill of about 2.75m high. There were no mud fountains or any type of damage to the structures. This indicates that a stress intensity of the order of 400 g/cm² prevented liquefaction completely in field. In Figure 8 the pore pressure at an overburden of 400 g/cm² is about 45 cm. This means that the transfer of intergranular stress to the pore water is only about 10% which is af no great practical significance. Hence the laboratory data obtained depicts the field bahaviour remarkably well.

The fact that the horizontal vibration table tests have given very encouraging results, it was thought proper to develop a rational methodology to predict possibility of liquefaction of a deposit based upon such tests, both with and without surcharge. It may be of interest to note at this stage that no effort has so far been made by investigators who had carried out studies on vibration table in previous years in this direction (Maslov 1957, Florin and Ivanov 1961, Bazant 1965, Finn et. al 1971).

METHOD OF LIQUEFACTION ANALYSIS

The following physical phenomenon constitute the basis of analysis of liquefaction,

(i) If a pressure P_0 is applied on top of a confined column of water the water pressure is increased equally by P_0 throughout the column height.

Therefore an increase in porewater pressure during vibrations at a depth in a saturated soil mass causes an equal increase in pore water pressure throughout the deposit below this depth. This phenomenon has been observed in the laboratory by Florin and Ivanov (1961) in horizontal vibration table tests.

(ii) It has been explained before that pore pressure developed during an earthquake remains constant during its entire duration. Therefore if some pore pressures are developed in first few cycles of ground motion at a depth these will be transmitted equally in all the deeper layers and the effective overburden pressure in these deeper layers is reduced. In the next few cycles and under reduced effective overburden pressures further pore water pressure may develop in deeper layer resulting in still further reduction in effective overburden pressure on still deeper layers. In this process, if the pore water pressures become equal to the initial effective over burden pressures, progressive liquefaction may occur starting at some depth and moving to deeper layers.

(iii) The pore water pressure developed during the earthquake dissipate only by upward movement of water thus setting up a water flow. During this process a hydraulic gradient will be set up in the deposit and may develop quick conditions in the upper layers which may not have liquefied earlier. In many cases for all practical purpose they have been seen to liquefy during tnis vertical flow of water forming vertical pipes in the deposit and bringing out the sand and water mixture.

Based on these physical concept a method of analysis is developed. The information required and the method of analysis is given below.

INFORMATION REQUIRED FOR THE ANALYSIS

The following information is required for carrying out the analysis.

1. Depths of alluvial deposits : The depths of deposits overlying the firm base can be determined by carrying out geophysical exploration and making a geologic assesment and more precisely by soil exploration to the base rock if possible.

2. Relative densities and position of water table in field : This information can be obtained by obtaining a bore log data at the site and by correlating the N values (S.P.T.) with relative density.

3 Effective overburden pressure with depth : This can be computed by assuming a reasonable value of density or from engineering properties

obtained on sand samples in laboratory for the purpose of this analysis.

4. Accelerogram of the anticipated earthquake : Estimation of an appropriate accelerogram for any site is difficult problem. However a suitable accelerogram for a site can be assigned by considering the upward propagation of shear waves from an underlying firm base or by directly modifying an accelerogram of an existing earthquake if possible. The method of analysis has been presented else where (Gupta 1977).

5. Laboratory data : After the relative densities and effective overburden pressure with depth and accelerogram for the site are established, the laboratory test are required to be performed on the sand sample obtained from site on vibration table described earlier. The tests are required to be carried out at predominent frequency of the accelerogram at different relative densities estimated for field and different surcharges in the range of 0 to 0.4 kg/cm^2 using the dead weight surcharge device. Plots of excess pore water pressure at 25 cm depth versus effective overburden pressure for corresponding field acceleration and relative densities and a plot of time wise response of pore water pressure are obtained. This will serve as laboratory data for making the analysis.

LIQUEFACTION ANALYSIS

After obtaining the necessary information the analysis is performed step by step. The steps involved are described below. (Fig. 9)

1. Mark the thickness of different strata of different relative densities.

2. Plot the effective overburden pressure versus depth as shown by line OA.

3. Consider the accelerogram to consist of suitable different number of cycles of different intensities of accelerations shown below keeping the sequence of vibrations the same as in the accelerogram

	(1)	(2)	(3)	(n)
Acceleration intensity	a_1	a_2	a_3	a_n
Number of cycles	n_1	n_2	n_3	n_n

4. For first set of n_1 cycles of acceleration (a_1) the pore pressure with depth is plotted with the help of correspondig laboratory data till maximum pore presure at particular depth D_1 is obtained. The pore

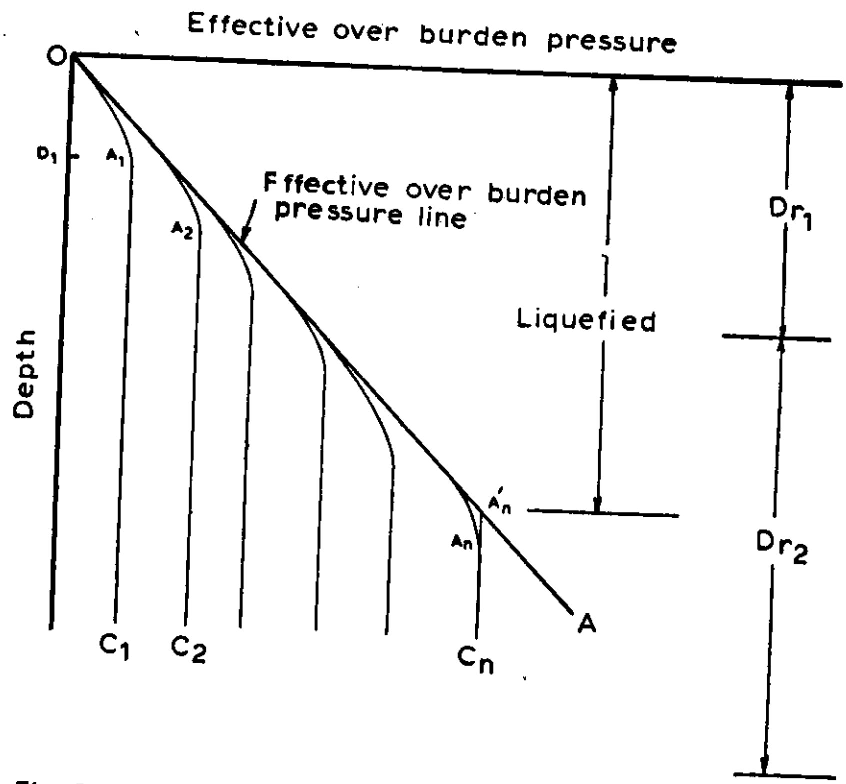

Fig. 9—Excess pore water pressure developed during earthquake

pressure below this depth D_1 equals this maximum value as explained earlier. Thus pore pressure line in first n_1 cycles is given by OA_1C_1. Hence the horizontal ordinate between OA and OA_1C_1 on any particular depth represents the effective over burden pressure immediately after n_1 cycles of attendant motion.

5. Repeat step 4 for second set of number of cycles n_2 of intensity a_2 and obtain further loss in effective overburden given by OA_2C_2.

6. Repeat the process step by step for entire accelerogram finally obtaining a loss in effective over burden given by OA_nC_n.

7. Extend A_nC_n to meet OA at A'_n. Depth to A'_n has been considered to have liquefied in this analysis.

Prediciion of liquefaction possibility in field is possible in a more rational manner in the above analysis. The method considers the progressive failure of ground on account of liquefaction as expected during ground shaking. The case of the Niigata earthquake of 1964 was studied as given below for two sites.

LIQUEFACTION ANALYSIS OF NIIGATA SITE

Two sites were selected for analysis from Niigata. In site I there was heavy damage on account of liquefaction during the earthquake of June 1964 which has been estimated to be of 7.5 magnitude at an epicentral distance of about 50 km. Site II also lies in this area but there was a 2.75m high dry fill and no damage occurred in this zone.

For analysis of liquefaction a suitable accelerogram needs be selected. For this purpose the koyna earthquake accelerogram which was recorded on rock, was modified to account for (i) Magnitude of earthquake, (ii) Epicentral distance from Niigata, (iii) Duration at Niigata. There was alluvial deposits of about 80 metre above the firm base in Niigata. A time wise shear beam reponse of the deposit was determined on IBM 360/44 computer to obtain the response near ground surface when the firm base below was subjected to the modified koyna eartquake. This response near surface constitutes artificial earthquake for analysis of liquefaction Fig. 10. The details are presented elsewhere (Gupta 1977). This accelerogram indicates a maximum ground acceleration of 0.2g which compares well with the maximum recorded acceleration of about 0.16g near the surface. When the soil below the ground level is undergoing liquefaction, the surface motion are expected to attenuate and a lower value of accelerations may have been recorded. Therefore the artificial earthquake of Figure 10 seems to be reasonably correct for the site under consideration. Since the grain size curves of Solani Sand lie fairly close to the sand deposits of Niigata area, the laboratory result on this sand could be used to predict the behaviour of that site.

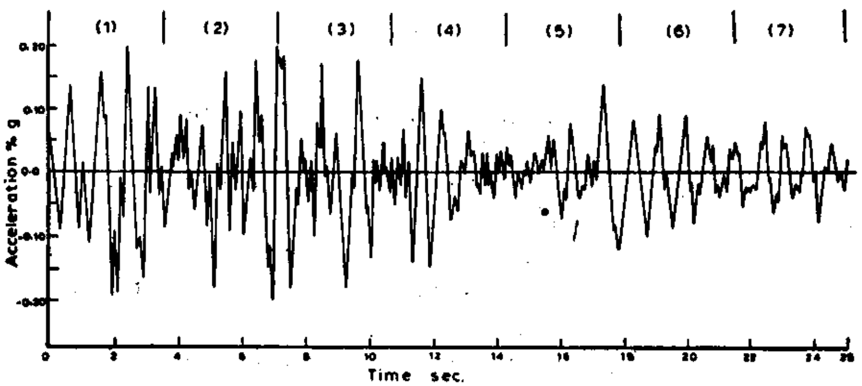

Fig. 10—Artificial accelerogram for Niigata site

Laboratory test results on Solani Sand show that it liquefies completely at all accelerations of more than about 8 %g both with and without surcharge. Therefore all the peaks higher then 10%g in the earthquake could be considered of 10%g intensity for purposes of anaysis. Neglecting one or two smaller or higher peaks in between the earthquake can be considered to consist of following combinations of accelerations and number of cycles without much error.

Sl. No.	(1)	(2)	(3)	(4)	(5)	(6)	(7)
Acceleration %g	10	10	10	5	5	10	5
Number of cycles	10	10	10	10	10	10	10

Analysis

(a) Site I

The liquefaction analysis is shown in Figure 11. The effective overbu-

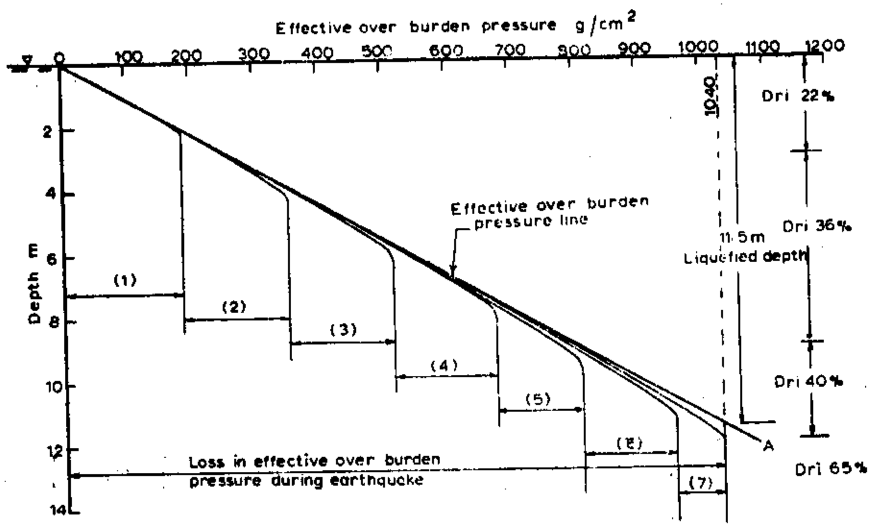

Fig. 11—Liquefaction analysis of Niigata site

rden is plotted with depth in this figure. The loss in overburden pressure is estimated for the above combinations of accelerations and number of cycles step by step as explained earlier. It is seen that during the earthquake a loss of 1040 g/cm² in overburden pressure takes place. This means, that the effective overburden upto a depth of about 11.5m has been completely transferred to pore water. The reported depth of liquefaction at this site is 10m to 15m (Kawasumi 1968). This shows that the method proposed above predicts the liquefaction of a soil in field reasonably well.

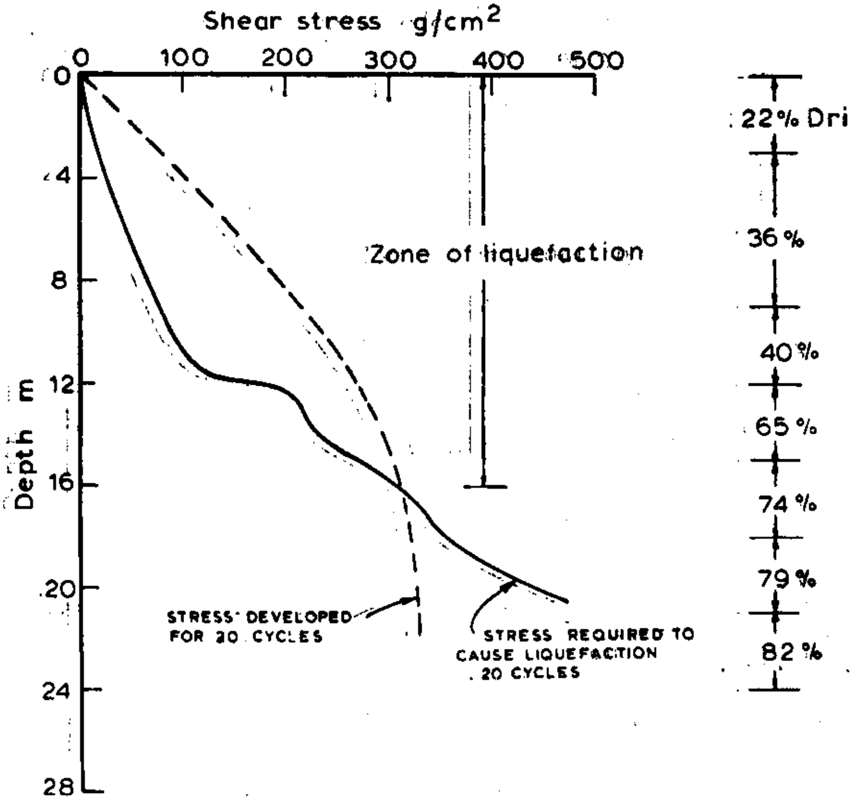

Fig. 12—Liquefaction analysis of Niigata site (Seed and Idriss method 1971)

The analysis was carried out for this site by Seed and Idriss (1971) approach also (Fig. 12) with the ground motions as in Fig. 10. Liquefaction has been found to occur upto a depth of about 16m. This show that the accelerogram chosen in this analysis is quite in order. These results are in good agreement with the observed behaviour both by the method of Seed and Idriss (1971) and by the proposed method.

(b) Site II

Liquefaction analysis for this site is shown in Fig. 13. The analysis shows that residual effective overburden pressure at the level of the water table is about 108g/cm^2 i.e. 23% of initial effective overburden of 480g/cm^2 and a pore pressure of about 372 g/cm^2 has built up during the earthquake. When this will try to dissipate it can cause only a small temporary rise in water table and no damage is expected because of dry soil. This result is in exact confirmity with the field behaviour.

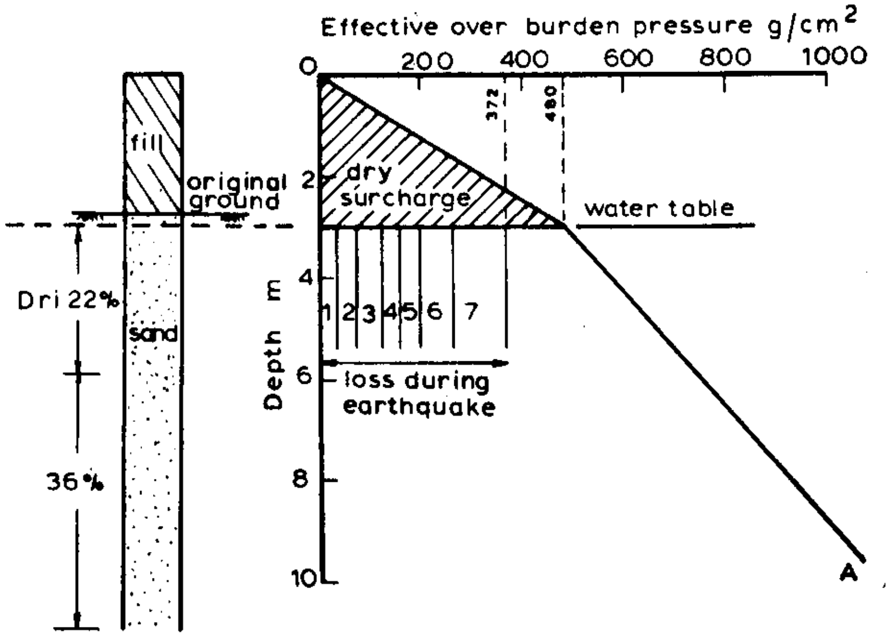

Fig. 13-Liquefaction analysis of fill area

The analysis for this site was also carried out by Seed and Idriss (1971) approach (Fig. 14) where it predicts liquefaction to a depth of about 17m. While no traces of liquefaction were observed at this site. Thus the method proposed by Seed and Idriss (1971) failed completely to predict the behaviour where liquefaction did not occur. The present method predicted it well.

Their method suffers from the following limitations

1. Representation of field behaviour by laboratory test data on small samples : In this respect it has been brought out that these tests do not represent the field conditions Castro and Polous (1977).

2. Consideration of progressive failure in the deposit. : Since the method is based on concept of total equivalent number of significant cycles, the

progressive nature of failure in field can not be taken care of in their analysis.

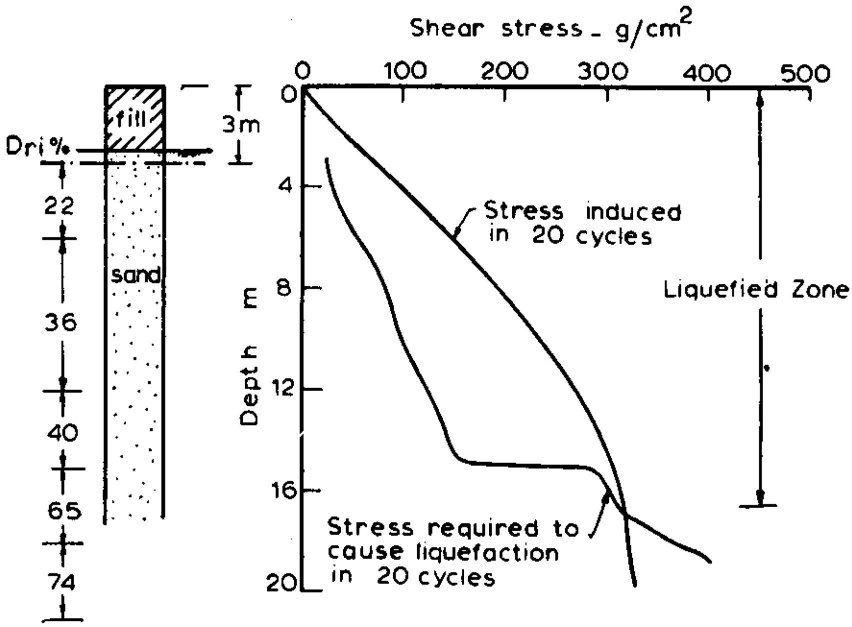

Fig. 14 Liquefaction analysis of fill area
(Seed and Idriss method 1971)

The vibration table test depicts the field behaviour to a reasonably good extent. The sample is prepared and consolidated under anisotropic condition similar to field. Also the deformations occur under plane strain conditions. The method also takes care of progressive nature of failure. The method using the vibrations table test data has predicted field behaviour for both places where liquefaction had occurred and where it had not occurred. This reflects sufficient confidence in this method of analysis and in the attandent vibration table tests which are much simpler to perform and monitor compared to the triaxial tests. Therefore the proposed method has quite a potential to be used as a standard test and may replace the Seed and Idriss (1971) approach which is at best approximate.

CONCLUSIONS

1. In field excess pore water pressure developed remain constant during the duration of the earthquake.

2. The sand with relative density of more than 65% would be safe against liquefacton.

3. Vibration table tests can be considered to represent the field condition with sufficient degree of confidence. These are much simpler to perform and monitor.

4. A method has been developed for the first time for making the analysis using vibration table test data in a more rational manner and has been shown to be more realistic and reliable.

5. The proposed method has quite a potential to be used as a standard for professional use of field engineer.

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