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Liquefaction Studies on Silty Clays Using Cyclic Triaxial Tests

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ABSTRACT

Liquefaction of saturated soils during earthquake often had been a major cause of damage to structures. Since beginning the liquefaction studies were concentrated on sandy soils, as the sandy soils are known to be more susceptible to liquefaction. However, observations from some sites in China and Loma-Prieta earthquakes show that soils with high plastic fines are also susceptible to liquefaction. Isotropically consolidated undrained cyclic triaxial tests were conducted on undisturbed samples of silty clay soil and results of these tests are used to verify the methods based on SPT data. Slow cyclic tests were performed to investigate the development of pore pressure and cyclic strength, as reliable pore pressure measurement is only possible in slow cyclic triaxial test for clayey soils. The site was than characterized for liquefaction by a computer program developed.
The N-value was obtained for the same site by conducting Standard Penetration Test. Test results were verified using methods reported by Tokimatsu & Yoshimi (1983) and Ishihara (1993). The simplicity of the methods and application of the methods to the fine-grained soils are the main criteria for selection of the field methods. The computer program also provides characterization of site using these methods.

INTRODUCTION

Inspite of so much advancement of science and technology, earthquakes are still the cause of heavy destruction of life and property. Even though, the total duration of earthquake during past century is less than an hour but the damages are extensive. More than 2,00,000 people lost their lives during these earthquakes. Liquefaction of saturated soils during earthquakes often had been a major cause of damage to structures. When a saturated soil is subjected to ground vibrations, due to earthquakes, blasts, repeated loading or heavy tamping, it tends to compact and decrease in volume. If drainage does not occur, this decrease in volume results in increase in pore water pressure and if pore water pressure builds up to the point at which it becomes equal to the overburden pressure, the effective stress become zero. In the case of sands, it loses its strength completely and develops into a liquefied state. This is called liquefaction. Liquefaction phenomenon itself may not be damaging but when liquefaction causes ground failure, it may be disastrous to the structures built on the ground.

Flow failure may occur in loose sand or if the sand is dense, large deformations may occur. In fine-grained soils there may not be pore pressure rise up to overburden pressure but high residual pore water pressure may develop. As the result of liquefaction, excessive ground settlement and sometimes flow failures involving extremely large movement of soil masses occur. Flow failures are generally developed in loose saturated sands or silts on relatively steep slopes. Failure may result in lateral displacement of large masses of liquefied soil or large blocks of soils and rocks sliding over liquefied soil, from few meters to few kilometers down long slopes. The velocity of sliding may be up to tens of kilometers per hour.

Lateral spreads generally develop on gentle slopes and move towards free surface. Horizontal displacements range up to several meters. These types of failure involve lateral displacement of surface soil block due to liquefaction of subsurface soil layer. Lateral spreads are particularly destructive to buried structures like pipelines. If the backfill of a retaining wall liquefies, the lateral pressure on the wall may increase substantially. The wall may be laterally displaced, tilt or structurally fail due to this increased lateral pressure.

During liquefaction phenomena soil looses its strength. If a building or any other structure is supported on this soil, large deformations can occur within the soil mass, which may cause the excessive settlement or tilting of the structure. The buried structure may rise in liquefied soil due to buoyant forces. Often, the weight of structure is not enough to cause large settlements. But, smaller settlements may occur due to dissipation of pore water pressure (which developed during earthquake) after earthquake. This can cause differential settlements, which may destroy facilities or cause structural damage.

The liquefaction studies had been given importance after heavy destruction was observed during Niigata and Alaska Earthquakes. In these earthquakes, all the modern structures were destroyed due to soil liquefaction. Many buildings settled and tilted. During Alaska earthquake (1964) more than 200 bridges were destroyed or damaged by spreading of flood plains deposits towards river channels. Since then, much work had been done to understand the basic mechanism and associated problems. Since beginning, the liquefaction studies were concentrated on sandy soils, as the sandy soils are known to be more susceptible to liquefaction. However, observations from some sites in China and Loma-Prieta show that the soils with high plastic fines are also susceptible to liquefaction and damages to structures may occur. Based on field observations and laboratory test data many empirical correlations had been given between field penetration resistance (SPT, CPT) by different researchers. Some researchers had given correlation between shear wave velocity and liquefaction to evaluate liquefaction potential of a site using these field tests.

The liquefaction phenomena were observed in India during Assam Earthquakes (1897, 1954 and 1971), Bihar-Nepal Earthquake in Jan. 1934 and Aug. 1988. Liquefaction studies in India have been carried out since 1960. The studies were done on sands. After that, not much work has been done to understand the behavior of undisturbed soils of this region, as the seismic activities in this region were not much. Three physiographic regions of India are Himalayas, Indo-Gangetic plains and the southern plateau. Himalayas are located along northward moving Indian tectonic plate, which is going down the Eurasian tectonic plate. The Indo-Gangetic plain, with a total of about, 850,000 sq. Km. forms the surface cover of Indo-Gangetic basin. Ganges and Brhamputra rivers have contributed sediments in this region. The alluvial deposits of Indo-Gangetic basin have been estimated to have a thickness of the order of thousands of meters (Rao, 1973). The region lies in tectonically active zone. From 1988, seismic activities in this region have increased considerably. The alluvium is made of sand, sandy silt to clayey silt or silty clay. Such types of strata are likely to liquefy during earthquake. Common practice for evaluating liquefaction potential in India is to use field correlations given by researchers for the soils not belonging to this region without checking the applicability of these correlations to the soils of this region. Therefore, an attempt has been made to test at least one type of Indian soil (locally available) in laboratory using cyclic triaxial test. The correlations (N- value from SPT and liquefaction potential) given by Tokimatsu et al. (1983) and Ishihara (1993) were also verified.

A C –code has been developed to characterize the sites against liquefaction. The interfacing of the code has been done using Visual Basic.

LABORATORY TESTING

The field conditions during earthquake can be best reproduced in simple shear test or centrifuge tests. But, because of availability and the greater simplicity in testing procedure, cyclic triaxial testing method is used to evaluate liquefaction potential of the undisturbed samples obtained from the ground in this study.

Cyclic triaxial tests were performed on undisturbed samples of silty clay, obtained from the campus of I.I.T. Kanpur. A pit of size 2m× 2m×2m was excavated. The undisturbed samples were extracted by pushing thin wall tube sampler (38mm internal diameter) into the ground. The samples were carefully transported and stored. 38mm diameter and 76mm high samples were prepared from these tube samples by pushing the soil in the mould of the same dimensions. These samples were then tested in computer controlled triaxial testing apparatus (GDS Triaxial Testing System). A personal computer is linked to a hydraulic triaxial cell via three microprocessor controlled hydraulic actuators called "digital controllers". The controlling computer was an IBM PC. The computer controls the test parameters, data logging and data presentation in the form of tables and graphs (Anubhav, 1999).

The samples tested in cyclic triaxial tests were carefully removed from the triaxial cell. The index properties of these samples were found out to classify the soil. Particle size distribution of soil sample has been determined by washed sieve analysis and hydrometer test. The consistency limits (liquid limit and plastic limit) were determined as per standard procedures in laboratory. Results of sieve analysis and hydrometer analysis are shown in Fig. 1. The particle size distribution curve shows that the soil contains predominantly

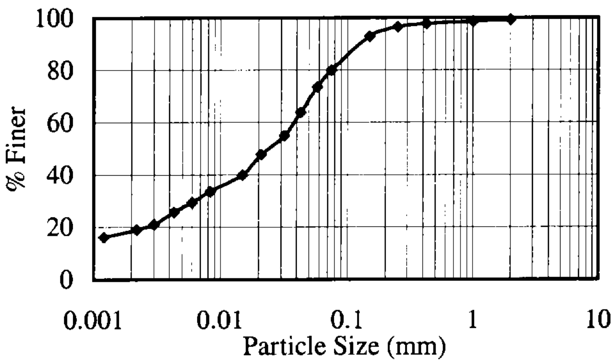

Fig. 1. Particle Size Distribution Curve

silt size particles along with clay and sand size particles. The samples contain homogeneous silty clay soil. The consistency limits of the soil (LL – 34, PI – 16) show that the soil falls just above the A – line in plasticity chart. The soil is a low plasticity clayey silt or silty clay. From index properties, the soil can be classified as CL – ML. The natural density of the soil is found to be 18kN/m^3. The saturated density of the undisturbed samples was found to be 20 kN/m^3. Natural density is very close to saturated density because the samples were collected during rainy season.

Consolidation Stress History

The consolidation test result (e – log p curve) is shown in Fig. 3. The initial void ratio of the sample (e_0) was found as

0.65. The compression index (c_c) of the sample was obtained as 0.19. Pre-consolidation pressure was determined using empirical construction method proposed by Casagrande. Pre-consolidation pressure from this test indicates that soil is lightly over-consolidated (over-consolidation ratio of 1.4).

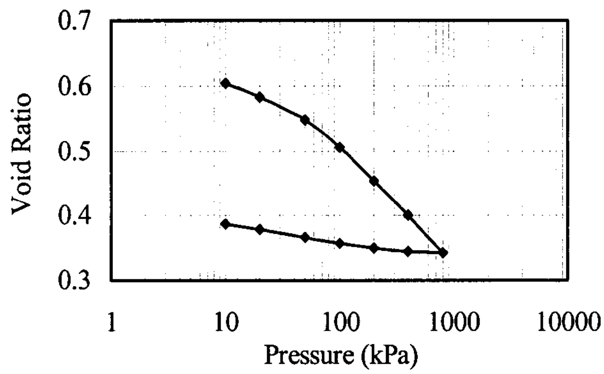

Fig. 2. Consolidation Test Results

Cyclic Triaxial Tests

Six isotropically consolidated undrained cyclic triaxial tests were performed on silty clay samples. Sample of diameter 38mm and height 76mm was placed on a porous disc on the pedestal of triaxial apparatus. Filter paper strips were cut and soaked in water. Soaked filter drains were wrapped around the specimen to increase the rate of consolidation. The plain-ended top cap was placed on top of the sample. The sample was covered with very thin membrane. O – rings, under tension, was used to seal the membrane to the pedestal and top cap. Extension device was fitted on the top cap to perform compression-extension cyclic triaxial tests.

Skempton's pore pressure parameter (B parameter) was checked before testing. If B parameter was found to be less than the desirable value i.e. 0.97, back pressure was applied to saturate the sample. B parameter was checked at several stages during saturation process. In all cases, B parameter equal or greater than 0.97 was achieved, indicating satisfactory saturation. The back pressure required to obtain this B value was about 200kPa. After ensuring saturation, the sample was allowed to consolidate isotropically under the effective confining pressure of 200 kPa, which corresponds to normally consolidated state of soil in the field. After 24 hours of consolidation, the drainage valves were closed. Tests were carried out in undrained conditions, using uniform cyclic load in the sinusoidal waveform at a frequency of 0.02Hz. Zergoun and Vaid (1994) demonstrated that the pore pressure measurements on clay samples can only be obtained reliably in slow cyclic triaxial tests. Therefore, slow cyclic tests were performed to investigate the development of pore pressure and cyclic strength.

Tests were run until the pore pressure become equal to the effective confining pressure or axial strain reaches 5% double amplitude or cumulative axial strain of 10% or 10 cycles of loading. If the sample withstood 10 cycles of loading, the sample was immediately subjected to another series of loading. This was repeated till failure was reached. After completion of cyclic triaxial tests samples were removed and water content was determined. These samples were kept for testing the index properties of soil

The tests were conducted at different cyclic stress ratios (CSR). Cyclic stress ratio is the ratio of deviator stress (σ_d) and twice the initial effective confining pressure (σ_0). The tests were conducted with the cyclic stress ratios equal to 0.35, 0.33, 0.3, 0.25, 0.23 and 0.18. Figure 3 shows the plot of cyclic stress ratio vs. number of cycles to cause 5% double amplitude axial strain or 10% cumulative strain. The relationship between pore pressure ratio (ratio of pore pressure, u and initial effective confining pressure, σ_0) and cycle ratio (N/N_l) is presented in Fig. 4. Where, N is the number of cycle to increase pore pressure by Δu and N_l is number of cycles to cause liquefaction.

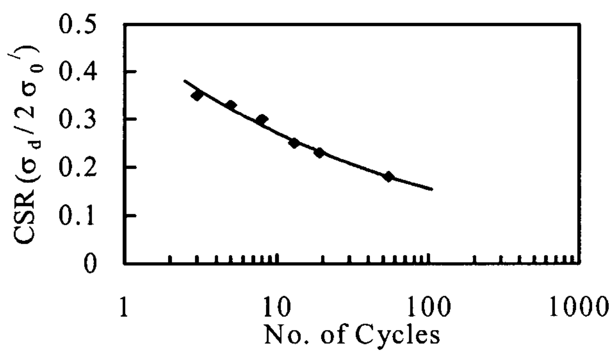

Fig. 3. Cyclic Stress Ratio vs. No. of Cycles

Consolidated undrained triaxial compression tests were also performed on similar samples of soil to see the pore pressure response during monotonic loading. The results of consolidated undrained triaxial test show that maximum pore pressure generation during compression loading was about 50% of initial effective confining pressure, whereas, test results of all cyclic triaxial tests show that the pore pressure increases with number of cycles. The pore water pressure during cyclic load application on the sample was fully built up to the initial effective confining pressure, for the cyclic stress ratios above 0.33. For the cyclic stress ratios lower than this value pore pressure does not build up to the initial effective confining pressure. The maximum residual pore pressure during the tests (CSR < 0.33) were 90 - 95%.

The pore pressure increases during compression loading and reduces significantly during extension loading. From the relation between pore pressure ratio and cycle ratio it can be seen that the excess pore pressure increases rapidly at the beginning of the cyclic load application on the silty clay specimen. The increase of pore pressure was rapid up to a cycle ratio of 0.5. Pore pressure ratio reaches up to around

0.85 at this cycle ratio (Fig. 4). Beyond a cycle ratio of 0.5 rate of increase in residual pore pressure was observed to be very slow (usually 0.85 – 0.95 for CSR < 0.33 and 0.9 – 1.00 for CSR >0.33) (Fig. 4). Figure 4 also shows the curves between pore pressure ratio and cycle ratio for different sands given by Lee et al. (1974). The data obtained from cyclic triaxial tests for different sands falls within the region shown. By comparing curves in Fig. 4, it can be clearly concluded that the increase in pore water pressure in silty sand is quicker than that reported for sand.

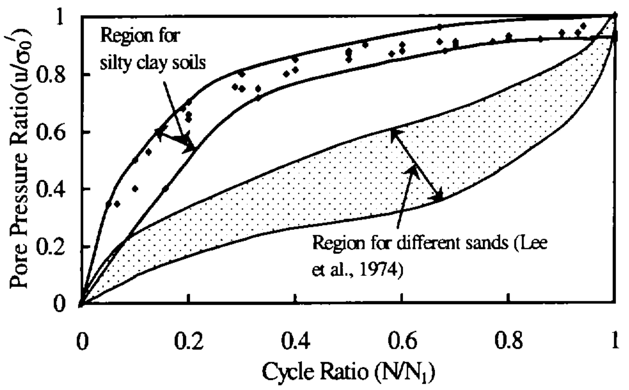

Fig. 4 Pore pressure ratio vs. cycle ratio

SITE CHARACTERIZATION

Cyclic strength of the silty clay was estimated based on the results of the triaxial testing program with the correction for boundary conditions (simple shear vs. triaxial) and multi-direction shaking in field (Seed 1979).

$$CSR_{field} = 0.9\, C_r\, CSR_{triaxial} \qquad (1)$$

Boulanger et al. (1998) compared the cyclic simple shear and cyclic triaxial test data for several fine-grained soils and found C_r closer to 0.7 for normally consolidated conditions. The slow cyclic triaxial tests were performed to investigate the pore water pressure reliably. Zergoun and Vaid (1994) concluded that slow cyclic triaxial tests are better for clayey soils. However, the cyclic strengths, obtained from slow cyclic tests, were 25 – 40 % lower than the fast cyclic strength. If correction for slow rate of testing is taken as 1.4. Then,

$$CSR_{field} = 0.9 \times 0.7 \times 1.4 \times CSR_{triaxial} \qquad (2)$$

$$CSR_{field} \approx 0.9\, CSR_{triaxial} \qquad (3)$$

An additional correction factor would be needed to account for the effect of sloping ground, but this effect can be neglected for mild slopes.

Shear stresses developed at any point in a soil deposit during earthquake appear to be due to upward propagation of shear waves. Seed et al. (1971) have given the simplified procedure for evaluating the average cyclic shear stress, at any point at depth h, induced by earthquake as:

$$\tau_{av} \approx 0.65 \times (\gamma h/g) \times a_{max} \times r_d \qquad (4)$$

where, γ = the unit weight of soil, r_d = stress reduction factor with a value less than 1, a_{max} = maximum ground surface acceleration and g = acceleration due to gravity.

Ratio of cyclic stress causing liquefaction or cyclic mobility and cyclic stress induced by expected earthquake gives the factor of safety against liquefaction or cyclic mobility.

Due to lack of sufficient data for the Indo-Gangatic plain, no correlations are available to predict maximum horizontal acceleration (a_{max}) in this region. Singh et al. (1996) have given a correlation to calculate a_{max} from different earthquake magnitudes as a function of epicenter distance, for the North Indian Region (Himalayan Region). The correlation has been given based on various earthquakes recorded in the northern part of India since 1986 with magnitudes of 5.7 to 7.2 (Richter's Scale). The peak horizontal acceleration (a_{max}) has been given as:

$$\log_{10} a_{max} = 1.14 + 0.31\, M - 0.615 \log_{10} R \qquad (5)$$

where, a_{max} = peak horizontal acceleration in cm/sec^2, M = earthquake magnitude and R = epicenter distance in km

The undisturbed samples were obtained from I.I.T. Kanpur campus, which falls in seismic zone – III, as given in IS: 1893 - 1984. The site may be subjected to earthquakes of magnitude 6 to 6.5. The assumption was made that the site is made up of homogeneous silty clay deposit, and the samples tested are representative of the whole deposit. The water table was assumed to be at ground level, which represents the worst condition.

The analysis of this homogeneous deposit was done for earthquake magnitudes of 6, 6-1/2, 6-3/4, 7-3/4 on Richter's scale. The horizontal acceleration decreases with the increase in distance of epicenter. For each earthquake magnitude, site was analyzed with varying epicenter distance (varying a_{max}).

A C – code have been developed to characterize a site against liquefaction. The code has two different options for input data 1) cyclic stress ratio and 2) N –value from standard penetration. Two SPT based methods (Tokimatsu et al., 1983 and Ishihara, 1993) are used to develop the code. The interfacing of the code is done using Microsoft Visual Basic 5.0. The factor of safety against liquefaction was found out using cyclic triaxial test data (after applying corrections as described above). The program also provides the plot of factor of safety vs. depth for a depth of 30m. The site was characterized as highly liquefiable if the factor of safety was found less than one; it was characterized as low if the factor of safety was found between 1 and 1.5. If the factor of safety was found more than 1.5, the site was characterized as non-liquefiable. Results for earthquake magnitude 7-1/2 are shown in Fig. 5.

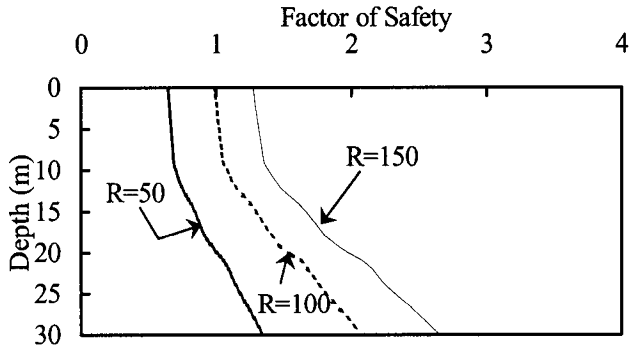

Fig. 5. Factor of safety against liquefaction for earthquake magnitude 7.5 and varying epicenter distances R Km.

STANDARD PENETRATION TEST

Using standard penetration test data from the same site, relevance of some field correlations, given by different researchers, has been checked. Tokimatsu and Yoshimi (1983) method and Ishihara (1993) method were used to analyze the site. The simplicity of these methods and application of the methods to the fine-grained soils are the main criteria for selection of the field methods.

Standard penetration test was performed at the site at the depth of 2m (at which samples for cyclic triaxial tests were obtained). The observed N – value was found as six. The N – value obtained from SPT using rope and pulley mechanism was used. For Tokimatsu and Yoshimi (1983) method, the observed N-value was corrected for energy delivered in SPT as:

$$N_j = 0.833\ N \qquad (6)$$

where, N_j is the N – value used for Tokimatsu and Yoshimi (1983) method and N is the observed N – value.

Ishihara (1993) method suggests the use of the chart, to correlate N_1 and cyclic stress ratio, given by Seed et al. (1983). The chart is based on the N – value obtained from SPT using rope and pulley mechanism. Therefore, no correction to observed N – value is applied in this method. The value is than corrected for overburden as per prescribed equations given in the two methods mentioned above. From this corrected N – value (N_1) the cyclic stress ratios are evaluated for different methods.

Tokimatsu and Yoshimi (1983) method gives flexibility to vary fines content but it does not incorporate the effect of the nature of fines. The method also takes into account the effect of strain. The CSR obtained from this method was found to be low as compared to the CSR obtained from cyclic triaxial tests. Ishihara (1993) method takes into account the effect of fines content and also the effect of the nature of fines. The CSR obtained from this method was compared with the CSR obtained from cyclic triaxial test data. The results from both the methods are very much the same.

The computer program developed can characterize a site against liquefaction, up to 30m depth, using main input data as observed N –value (from SPT). Tokimatsu et al. (1983) and Ishihara (1993) methods were used to develop the code. The computer program can analyze a site against liquefaction and results (depth vs. factor of safety) can be obtained in the tabular form or in the form of a curve. The results obtained from both methods can be compared.

SUMMARY AND CONCLUSIONS

Based on the results of the liquefaction analysis using cyclic triaxial test and SPT, the following general conclusions can be drawn. The soil studied in this investigation can liquefy at higher cyclic stress ratios, more than 0.3. Higher pore pressures can develop for lower cyclic stress ratios also, which will lead to large deformations. In cyclic triaxial tests, the pore pressure increases up to 100% of initial effective overburden pressure for higher CSR and 90 – 95% for lower CSR. Whereas, in the triaxial compression test pore pressure rises only up to 50% of the initial effective overburden pressure. The rate of increase of pore pressure for silty clay soil is more rapid at the beginning of the cyclic load application as compared to the sands. The site has low potential for liquefaction, if the epicenter distance is less than 40km. For other epicenter distances, site is characterized as non-liquefiable. If this type of soil occurs in region with higher seismic activity zone (earthquake magnitude more than 7) the soil can be characterized as highly liquefiable up to certain depths. Ishihara (1993) method is appears to be the most suitable method for calculating liquefaction potential of fine-grained soils as it incorporates both the effect of fines content and nature of fines (cohesiveness).

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