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DEVELOPMENTS IN LIQUEFACTION ANALYSIS FROM OBSERVATIONS DURING EARTHQUAKES

ABSTRACT: Liquefaction has occurred during most earthquakes, Niigata 1964, Tangshan 1976, Loma Prieta 1985, Kobe 1995, Chi-Chi Taiwan 1999, Turkey 1999, Haiti 2010, Mexicali 2010 and others. The performance of structures during the Niigata earthquake (1964) in Japan and the nature of damage suffered by them clearly brought out the fact that soil and soil foundation interaction play a significant role in controlling their performance. The extensive liquefaction of loose saturated sands during the Niigata earthquake provided a field verification of the liquefaction phenomenon. Several other earthquakes in Japan also induced liquefaction. It was mostly believed as a necessary consequence that only cohesionless soils are prone to liquefaction and silts and clay do not liquefy. The observations during the Tangshan earthquake and other earthquakes later on suggested that silts and low plasticity clays may also liquefy. The observations on performance of soils during earthquakes has, thus , contributed to better understanding of the susceptibility of different types of soils to liquefaction and development of methods of analysis of liquefaction as discussed in the paper.

Introduction: Forensic engineering as applied to geotechnical engineering may be defined as investigation of geo-materials and soil-structures that fail or do not perform as intended resulting in damage or injury. Generally the purpose of Forensic engineering investigation is to determine the cause of failure with a view to improve performance in addition to other issues such as liability. From geo-hazard mitigation point of view forensic analysis should provide an answer to 'what happened', 'why it happened' and 'how can it be prevented from happening'. This is very relevant to the case of liquefaction phenomenon. Although liquefaction was known to have occurred in earlier times, the devastating effects of liquefaction during the March 27, 1964 Alaska (M=8.6) and June 16, 1964 Niigata (M= 7.5) earthquakes attracted the attention of geotechnical engineers. These two earthquakes occurred within period of about 90 days. Both these earthquakes caused extensive liquefaction and resulting damage which included slope failures, bridge and building foundation damage and floatation of buried structures. Some well known examples of liquefaction damage during these earthquakes are shown in Figures 1-3. Figure 1 shows the damage suffered by house displacement and tilting caused by liquefaction in the Turnagain Height area of Anchorage during the 1964 Alaska Earthquake. Figure 2a shows Collapse of the super- structure of the Showa Bridge by falling off its piers during Niigata Earthquake . Tilting of apartment buildings at Kawagishi-Cho, Niigata due to ground failure caused by liquefaction is shown in figure 2b.. Since the liquefaction in these earthquakes occurred in poorly graded sands at low to medium density, it was generally believed that only cohesionless soils are prone to liquefaction and fine grained soils do not liquefy. The Tangshan earthquake, July 28,1976 (M=7.5) provided evidence of liquefaction in low plasticity silts and several years later in the mid –eighties liquefaction aspects of silty soils were investigated. Later on , it has been recognized that all soils and including low plasticity clays should be considered liquefiable unless investigations prove otherwise. Liquefaction clayey deposits was observed in some earthquakes in Taiwan and Iran.

It is , thus seen, that developments in liquefaction analysis have been strongly influenced by the evidence on their performance during the significant earthquakes. The paper discusses the developments in investigation of liquefaction analysis of sands and fine grained soils as they developed following several devastating earthquakes. If the liquefaction susceptibility of a soil can be ascertained before hand and it is found to be prone to liquefaction for the design earthquake, then measures can incorporated to mitigate the hazard.

LIQUEFACTION INVESTIGATIONS: The liquefaction became important following the damage caused by the Alaska(1964) and the Niigata (1964) earthquakes. The studies were devoted to sands included the following:

(a) Investigation of sites damaged by earthquakes
(b) Laboratory tests using undarined cyclic triaxial and cyclic simple shear devices.
(c) Vibration or shake table tests
(d) Field tests such Standard Penetration tests (SPT) and Cone Penetration tests (CPT) and Shear Wave Velocity test.
(e) Numerical analysis.

STUDIES ON LIQUEFACTION OF SANDS: The laboratory studies helped identify the factors governing liquefaction of soils. Seed and Lee (1966) reported the first comprehensive data on liquefaction of sand using the cyclic triaxial test. Peacock and Seed (1968) used oscillatory shear device to study liquefaction in sand and a comparison was made of the shear stresses causing liquefaction in sand in the cyclic triaxial and the cyclic simple shear tests. It was observed that cyclic stresses causing liquefaction in loose saturated sands under cyclic simple shear conditions were only about 35 % of the cyclic stresses required to cause liquefaction under cyclic triaxial conditions. Since field conditions are more realistically duplicated in cyclic simple shear test but the cyclic triaxial tests are relatively easier to perform, therefore correction factors were proposed to correlate the cyclic triaxial data with the cyclic simple shear data. This resulted in the well known ‘simplified procedures’ for liquefaction analysis of sand deposits. The sample size used in the cyclic triaxial and cyclic simple shear device being small, it was pointed out by Finn (1972) that testing large samples using shake table may better represent the liquefaction of field deposits. The results of the shake table studies were in general qualitative agreement with data obtained from cyclic triaxial and cyclic simple shear tests. Limited studies on liquefaction of undisturbed samples of sand were also attempted and it was observed that natural undisturbed samples were somewhat more resistant to liquefaction compared to laboratory made samples at the same relative density due to aging effect and strength increase in sand due to development of bond between sand particles . Because of difficulty in procuring undisturbed sand samples and the associated cost of performing such tests, they cannot be routinely used for liquefaction analysis. The same argument applies to shake table tests. This lead to the search for a field test which could be used for ascertaining liquefaction susceptibility at a site. The standard penetration test which is routinely used for sub-soil exploration showed promise for estimating the liquefaction also. Standard penetration data was collected for sites which had experienced major earthquakes and where liquefaction had or had not occurred (Seed et al, 1985). The SPT value $(N_1)_{60}$ has been adopted by the profession as an index for liquefaction of saturated sand deposits. The plot in Fig. 3 (Seed et al.; 1985) has been commonly used for this purpose. The plot (Fig. 3) with fines content of less than 5% is typical for the case of sands. The relationship between the cyclic stress ratios and $(N_1)_{60}$ in Fig. 4 is for an earthquake of magnitude 7.5. For an earthquake of magnitude different from 7.5, the cyclic stress ratio obtained from Fig. 1, should be modified by multiplying with the magnitude scaling factor (MSF) proposed by Seed et.al; 1975).

Liquefaction potential is seen to decrease with an increase in the fine content in sand (Fig.3). Seed (1987) suggested the use of effective SPT value to account for the effect of fines in sand. The effective SPT value modifies the observed penetration resistance to equivalent clean sand penetration resistance and may be obtained as follows:

$$(N_1)_{60eff} = (N_1)_{60} + \Delta(N_1)_{60} \qquad (1)$$

$(N_1)_{60eff}$ = Effective standard penetration resistance or equivalent clean sand penetration resistance

$\Delta(N_1)_{60}$ = Correction for silt content

and, $(N_1)_{60}$ = observed SPT value for the silty sand.

The values of $\Delta(N_1)_{60}$ given below (Seed, 1987):

Fines content (%)	$\Delta(N_1)_{60}$
10	1
25	2
50	4
75	5

The tip resistance from cone penetration test (CPT) can also be used as a criteria for liquefaction (Fig. 4) (Mitchell and Tseng (1990)). CPT has the advantage over SPT in its ability to detect thin seams of loose soil.

Shear velocity has also been recognized as a useful indicator for liquefaction. Stokeo et., al. (1988) have used the cyclic strain approach and equivalent linear ground response analysis to investigate the relationship between peak ground acceleration for stiff soil site and shear wave velocity and correlated the data with conditions under which liquefaction may or may not develop. Tokimatsu et. al. (1991) used laboratory tests to develop plots correlating the cyclic stress ratios required to produce cyclic strain amplitude of 2.5 % in given number of cycles as a function of shear wave velocity. A typical co-relation is shown in Fig.5.

Another approach known as the 'cyclic strain approach' has also been proposed to determine the susceptibility of liquefaction by estimating the shear strain induced in the soil due to seismic loading and comparing it with the threshold strain required to develop liquefaction. The typical value the threshold strain is about 0.01 % (Dobry et al.; 1982) .

SUMMARY OF PROGRESS ON LIQUEFACTION ANALYSIS OF SANDS

It may be noted that remarkable progress has been made in the procedures for estimating liquefaction potential of sands based on laboratory investigations or on simple in-situ test data such as standard penetration values (N_1 or $(N_1)_{60}$) or on cone penetration data, and the experience during the past earthquakes, (Mitchell and Tseng, 1990; Robertson, 1990; Robertson and Campnella , 1985; Prakash,1981; Seed and Idriss, 1981; Seed and DeAlba, 1986; Seed , Idriss and Arango , 1983; Seed and Lee, 1966; Seed and Harder, 1990; Seed The cyclic stress approach (Seed and Idriss ,1981) and the cyclic strain approach (Dobry et al, 1982) are commonly used for evaluation of liquefaction potential of sands.

STUDIES ON LIQUEFACTION OF FINE GRAINED SOILS

Fine soils such as silts, clayey silts and sands with fines and silty soils were generally considered non-liquefiable. This concept, however, changed after observations following the Haicheng (1975) and Tangshan (1976) earthquakes. The soils that liquefied during Tangshan earthquake had clay fraction less than 20%, liquid limit between 21-35%, plasticity index between 4% and 14% and water content more than 90% of their liquid limit. Kishida (1969) reported liquefaction of soils with upto 70% fines and 10% clay fraction during Mino-Owar, Tohankai and Fukui earthquakes. Tohno and Yasuda (1981) reported that soils with fines up to 90% and clay content of 18 % exhibited liquefaction during Tokachi –Oki earthquake of 1968. Soils with up to 48 % fines and 18 % clay content were found to have liquefied during the Hokkaido Nansai –Oki earthquake of 1993. Gold mine tailings liquefied during the Oshima- Kinkai earthquake in Japan

(Ishihara, 1984). These tailings had silt sized particles and liquid limit of 31%, plasticity index of 10 % and water content of 37 %.

Seed et al (1983) found that some soils with fines may be susceptible to liquefaction. Such soils (based on Chinese criteria) appear to have the following characteristics:

Percent finer than 0.005 mm (5 microns) ‹15%

Liquid limit ‹ 35 %

Water content › 90

The authors conducted studies on liquefaction of low plasticity silts using naturally occurring and laboratory prepared soils and conducting dynamic triaxial tests. Most significant results of these studies were that the nature of fines has rather significant influence in determining the susceptibility of silts to liquefaction. The effect of nature fines can be best defined in terms of plasticity index of the fine grained soil. It was observed that the cyclic stress ratio causing liquefaction in a given number of cycles decreases with the increase in plasticity index. It was observed during the testing phase that cyclic loading of plastic silts results in pore pressure build up which becomes equal to the initial effective confining pressure resulting in development of the initial liquefaction for $PI < 10$.

OTHER RESEARCH ON EFFECT OF FINES ON LIQEFACTION

There are several research findings worth mentioning on the effect of fines on liquefaction potential of soils. Specifically:

1. Seed et al. (1985) have recommended that for sands containing less than 5% fines, the effect of fines may be neglected. For sands containing more than 5% fines, the liquefaction potential decreases as shown in Fig. 3. Neglecting the effect of fines should therefore be expected to lead to conservative estimates of liquefaction potential. However this suggestion is not based on experimental or field data.
2. Zhou (1981) made an interesting observation based on CPT tests on silty sands at one site and clean sands at another site that an increase in the fines content in sand decreases the CPT resistance but increases the cyclic resistance of the soil. No explanation is given for this peculiar behavior.
3. Ishihara and Koseki (1989) had suggested that low plasticity fines (PI‹ 4) do not influence the liquefaction potential. However, they did not consider the effect of the void ratio in their analysis.

4. Finn (1991) made an observation about the effect of fines in sand in developing equivalent clean sand behavior. If the void ratio of silty sand and clean sand is the same the liquefaction resistance decreases. If the comparison is made at the same $(N_1)_{60}$, the effect of fines is to increase the liquefaction resistance. If comparison is made using the “the same void ratio in

and skeleton" as the criteria, then there is no effect on the cyclic strength provided the fines can be accommodated within the sand voids.

5. Ishihara (1993) mentioned that in soils in which the fines content is sufficient to separate the coarser particles, the nature of the fines controls the behavior. Low plasticity or non-plastic silts and silty sands may be highly susceptible to liquefaction. This will be the case when PI is less than about 10. For soils with moderately plastic fines (fines content more than about 15 % and $8 \leq PI \leq 15$), the liquefaction behavior may be uncertain and may need further investigation. It is obvious that it is still not possible to evaluate the likelihood of liquefaction of silts or silty clays with the same confidence as for clean sands without additional investigations.

6. Seed et al., (2001) observed that there is significant controversy and confusion regarding the liquefaction potential of silty soils (and silty /clayey soils), and also coarser, gravelly soils and rockfills.

7. Finn et al., (1994), Perlea et al., (1999) and Andrews and Martin (2000) have provided general criteria about liquefaction susceptibility of soils with fines. The findings of Andrews and Martin (2000) are summarized in Table1. For use of Table 1 clay refers to fraction finer than 0.002 mm and liquid limit should be determined using Casagrande type equipment.

8.Bray et al. (2004) and Boulanger and Idriss (2005) and Idriss and Boulanger (2008) have investigated the liquefaction of soils with fines and shown that fine grained soils with more than 50 % passing US sieve # 200 can be reasonably grouped either into soils that exhibit sand-like stress-strain behavior or soils that exhibit clay like stress-strain behavior during monotonic and cyclic undrained shear loading. They observed that clay like behavior should be expected for silts (ML and MH) that have $PI \geq 7$ and for clays (CL and CH). Sand like behavior should be expected if their PI is < 7. For sand like materials, field test data such as N-values or CPT data may be used for determination of liquefaction potential. For clay like materials, laboratory testing may be necessary for ascertaining their behavior during cyclic loading. They also suggested that both sand-like and clay-like soils can develop excess pore water pressures and significant strains during undrained cyclic loading.

9. Bray et al. (2004) and Plito (2001) have suggested that the plasticity index rather than percent of clay size particles as a criterion for assessing the susceptibility of fine grained soils to liquefaction. Bray et al. (2004) found that soils that were observed to have liquefied in Aadapazari during the Koceli (1999) earthquake did not typically meet the Chinese criteria for liquefaction susceptible fine grained soils. During their investigation they found that soils with PI < 12 underwent liquefaction, soils between 12 and 18 were moderately prone to liquefaction and soils with PI > 18 were not prone to liquefaction at the effective confining pressures used in the experiments.

Table1. Liquefaction susceptibility of silty and clayey sands (Andrew and Martins, 2000)

	Liquid limit < 32	Liquid limit ≥ 32
Clay content < 10 %	Susceptible	Further studies required (Considering plastic non-clay sized grains such as Mica)
Clay content > 10 %	Further studies required (Considering non-plastic clay sized grains such as mine and quarry tailings	Not susceptible

10.Wang, Yuan and LI (2007) investigated the liquefaction susceptibility of saturated loess (silty soil) and fine sand obtained from an airport site near Lanzhou, China. This loess had PI varying from 7.2 to 9. Their studies indicated that this loess was more susceptible to liquefaction than fine sand.

11.Towhata (2008) has mentioned that it was previously thought that soils with fines are more resistant to liquefaction. However, he has also mentioned that the fines employed in those studies meant silts and clays that were cohesive in nature and fine materials without cohesion may still be vulnerable. It is the opinion of the authors based on the data presented here that the soils with low plasticity (PI < about 7) may liquefy or develop large deformations under cyclic loading.

13. Ghalandarzadeh, Ghahremani and Konagai (2007) Investigated liquefaction behavior of clayey sand from a site where large sand boiling, softening and large deformations had been observed in Iran due to an earthquake of magnitude 6.4. The soil had a liquid limit of 38 %, PI=18 %, and fine fraction (finer than 75 microns) of 44%. They performed cyclic triaxial tests . The analysis of data indicated that the clayey sand deposit likely developed high residual excess pore water pressures and significant shear strains during the earthquake and experienced liquefaction.

14. Thevanaygam (2010) has observed that at the same void ratio, the cyclic resistance of sand decreases with an increase in silt content upto a certain threshold value of fines content (fc_{th}); thereafter the cyclic resistance increases with further increase in silt content. Silt content affects the inter-grain contact density of soil compared to that of sand at the same void ratio. When this is taken into account , sand and silty sand show similar liquefaction resistance at same equivalent

void ratio $(e_c)_{eq}$. Figure 6 shows that at the same equivalent void ratio , the number of cycles inducing 5 % strain is almost the same for clean sand (OS-00), sand with 15 % silt (OS-15) and sand with 25 5 silt (OS-25)

SUMMARY OF PROGRESS ONLIQUEFACTION OF FINE GRAINED SOILS

The liquefaction of fine grained soils has certainly received the attention of researchers and progress is being made in this direction. In the early eighties , the soils with fines were considered less prone to liquefaction . The role of the nature of fines was not given the required importance. The authors work highlighted the role of plasticity of fines on their susceptibility to liquefaction.

Others have recently pointed out that based on fines content and the characteristics of fines, the soils may be expected to show sand like or clay like behavior. Presently (2010) there is no well defined index to ascertain the liquefaction potential of fine grained soils like SPT or CPT in case of sands.

CONCLUSIONS

Progress in liquefaction research and procedures of analysis have developed following the observations on liquefaction related damage and efforts to mitigate such damage . These observations have provided a field verification of this hazardous phenomenon. Most of the efforts was devoted to liquefaction of sands since major earthquakes in 1964 occurred in alluvial deposits. Liquefaction of fine plastic soils in later earthquakes has resulted in development criteria which identify liquefaction susceptibility of fine grained soils, though, some aspects still need clarification.