Journal of Civil Engineering (IEB), 38 (1) (2010) 65-80

Journal of Civil Engineering IEB

Earthquake induced liquefaction vulnerability of reclaimed areas of Dhaka

Mohammad Shariful Islam[1], Md. Tanvir Hossain[2], Syed Fakhrul Ameen[3], Eqramul Hoque[3] and Selim Ahamed[4]

[1, 3,4]*Department of Civil Engineering*
Bangladesh University of Engineering and Technology, Dhaka 1000, Bangladesh
[2]*Department of Civil Engineering,*
Military Institute of Science and Technology, Dhaka, Bangladesh

Received 16 October 2009

Abstract

Historical seismic data and recent seismic activities in Bangladesh and adjoining areas indicate that Bangladesh is at seismic risk. The capital city Dhaka metropolis together with its surrounding is situated in the Seismic Zone 2 (BNBC, 1993), which has a basic seismic coefficient of 0.15g. Most parts of Dhaka city have already been occupied. As a result, the city is expanding on reclaimed sites. Most of these sites are developed by filling lowlands (3~12 m) using dredge materials. However, in this method of filling, segregation of particles occurs. Mean grain size, fines content and uniformity coefficient of the fill materials for developing such areas varies from 0.05~0.25 mm, 6~28%, and 1.83~2.64, respectively. The SPT-N value of the filling depth varies from 1~13. Development procedures and characteristics of the dredged fill material indicate that the reclaimed sites may liquefy if an earthquake of sufficient energy occurs. Liquefaction potential of some selected reclaimed areas of Dhaka city have been estimated based on standard penetration test results. It has been found that some parts of the reclaimed sites are susceptible to liquefaction up to the depth of filling. A proposal for proper reclamation procedure has been presented.

Keywords: Dredge material, earthquake hazards, reclaimed area and liquefaction vulnerability

1. Introduction

Liquefaction problem became important when it started to affect human and social activities by disturbing the function of facilities and also after rapid urbanization by expanding the cities in reclaimed areas. Ground failures generated by liquefaction had been a major cause of damage during past earthquakes e.g., 1964 Niigata, 1971 San

Fernando, and 1989 Loma Prieta earthquakes. Liquefaction affects buildings, bridges, buried pipelines and lifeline facilities etc. in many ways.

The historical seismic data and recent seismic activities in Bangladesh and adjoining areas indicate that Bangladesh is at seismic risk. As Bangladesh is the world's most densely populated area, any future earthquake shall affect more people per unit area than other seismically active regions of the world. Bangladesh including capital city Dhaka is largely an alluvial plain consisting of fine sand and silt deposits with shallow ground water table in most places. Although the older alluvium is less susceptible to liquefaction, the deposits along the river flood plains may liquefy during a severe earthquake. Human-made soil deposits also deserve attention. Loose fills, such as those placed without compaction, are very likely to be susceptible to liquefaction. Over the past 30~40 years Dhaka city has experienced a rapid growth of urban population and it will continue in the future due to several unavoidable reasons. This high population increase (the forecast as per DMDP, 1995 anticipates a 21.1 million of population in 2015 with an average annual growth rate of 3.1%) demands rapid expansion of the city. Unfortunately, most parts of Dhaka city have already been occupied. As a result, new areas are being reclaimed by both government and private agencies. In many cases, the practice for developing such new areas is just to fill lowlands of the depth 3~12 m with dredged material, which is almost silty sand. This invites great liquefaction susceptibility for such areas.

After being recognised the liquefaction phenomenon during the 1964 Great Niigata and Alaska earthquakes, many researches have been conducted concerning liquefaction in different parts of the world. Many researchers have presented the liquefaction determination procedures like Japanese code of bridge design (Tatsuoka et al., 1980) including Chinese criterion (Ishihara, 1993), Seed-Idriss simplified procedure (Seed and Idriss, 1971) which have been updated over the years (e.g., Seed et al., 1985, Youd and Idriss, 2001, Cetin et al., 2004; Idriss and Boulanger, 2004). Recently, researches are being conducted to investigate the liquefaction of clayey soils under cyclic loading (Gratchev et al., 2006). In the later case, the research was conducted on artificial clay-sand mixtures and natural clayey soils collected from the sliding surfaces of earthquake-induced landslides. Yunmin et al. (2005) has established the correlation of shear wave velocity with liquefaction resistance based on laboratory tests.

A few researches have been conducted to estimate liquefaction possibilities at local levels in Bangladesh. Khan (1988) studied the soils liquefaction possibilities in Bangladesh. From the historical background of earthquake hazards on the Bengal Basin and its surroundings, it is interpreted that the western and south-eastern portions of Bangladesh are not so vulnerable to earthquake effect. So, north-eastern Zone was identified as earthquake prone area and liquefaction analysis was conducted for that area based on Seed-Idriss Simplified procedure. Studied areas were Sylhet, Sherpur, Maulvibazar, Sarail, Bhairab Bazar, Narsingdi, Muktagacha. Among them Sherpur–Maulivibazar area were found to be vulnerable to liquefaction. Rashid (2000) developed seismic microzonation map of Dhaka city based on site amplification and liquefaction. Ansary and Rashid (2000) generated liquefaction potential map for Dhaka city. Rahman (2004) updated the seismic microzonation maps for liquefaction as well as site amplification due to earthquake. Saha (2005) developed liquefaction potential map for Rangpur Town. Islam (2005) estimated the seismic losses especially due to the anticipated liquefaction in Sylhet city. However, none of these studies focused on the liquefaction vulnerability of the reclaimed areas of Dhaka city.

Islam and Ahamed (2005) conducted preliminary evaluation of liquefaction potential of some selected reclaimed areas of Dhaka city. It was observed that some parts of the reclaimed areas are susceptible to liquefaction. Absence of a definite knowledge of liquefaction possibilities causes difficulties in finalizing design of many important structures. This adds to the construction time and cost. It has been, therefore, felt necessary to undertake a study to estimate the liquefaction potential for some reclaimed areas of Dhaka city.

Many methods are established to evaluate the liquefaction potential of the sandy soil fill. However, in this study Japanese code of bridge design (Tatsuoka et al., 1980), Japanese code of bridge design based on Chinese criterion (Ishihara, 1993) and Seed-Idriss simplified procedure (Seed and Idriss, 1971) are used. The main objectives of this paper are as follows:

(1) To identify the shortcomings of the development procedure of reclaimed areas in Dhaka city and characteristics of fill materials.
(2) To evaluate the liquefaction potential of selected reclaimed areas.
(3) To propose proper reclamation procedure to avoid liquefaction and thus future hazards due to earthquake.

2. Development of reclaimed areas

Due to rapid urbanization, many areas are being reclaimed in and around Dhaka city. Many of the low-lying areas around the Dhaka city are being filled for housing by developers. Huge volume of soil is required to fill the reclaimed lands. Because of severe road traffic congestion around the sites the filling soil has to come through river mainly. The filled soil has to be collected from adjacent river sources of the city.

2.1 Methods of filling

The following methods are presently used for filling and developing low-lying areas.

(1) Soil being carried by country boats from remote sources and manually dumped at the site.
(2) Soil being carried by trucks from remote sources and manually dumped at the filling site.
(3) Soil is collected from riverbeds by cutter-suction dredging into a barge, which is carried to the nearest river site. Soil is then pumped through pipes in a slurry form after mixing with water, and transferred to the point of deposition. This procedure is used in most of the sites in Dhaka.
(4) Soil is dredged from riverbed by cutter-suction dredging and directly pumped to the filling site through discharge pipes.

In most cases, where large volume of fill materials is required, hydraulic filling procedure similar to method No. (3) or (4) is followed. Method No. (1) or (2) are seldom used where small volume of filling material is required. For large projects, method No. (1) or (2) are not followed due to cost effectiveness and heavy traffic congestion. In hydraulic filling, soil suspended in water is pumped through a pipe and the mixture discharged upon the surface being filled. Typically, the ratio of volume of solid particles to volume of sluicing water is about 1~7. As the suspension flows away from the discharge point, the larger soil particles settle out almost immediately. Thus a mound forms about the point of discharge and grow in height. Figure 1 describes the distribution of particles from mouth point of pipe to outwards. Due to gravity the coarser particles fall near the mouth point and the finer particles fall to farther distance. As a result, segregation of particles occurs and that degrades the quality of low land filling by hydraulic method, which is used in most cases in Dhaka city.

2.2 Characteristics of source materials

Since in Dhaka city most of the reclaimed areas are filled by dredged material therefore its quality and characteristics are important. The presence of fines in a hydraulic fill means greater compressibility and greater difficulty in compaction of the fill. Fines also reduce permeability and hence the rate of drainage. For these reasons, contracts governing the placement of hydraulic fill generally contain a

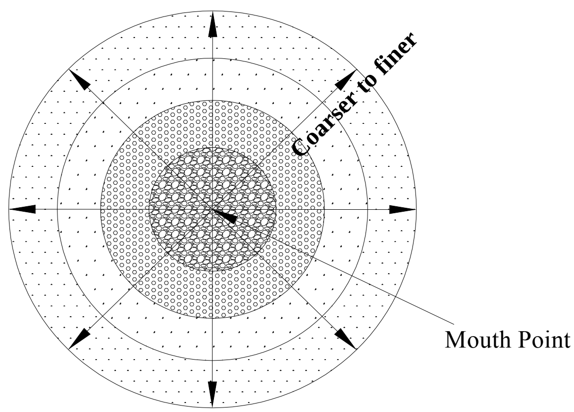

Figure 1. Schematic diagram showing particle distributions after discharge to filling sites.

avoid ponding of water where fines might settle out to form soft pockets or layers. Such specifications are, of course, meaningful only when the borrow material contains just a modest amount of fines. The quality of these soils as dredged material is graded on the basis of fine content (i.e., percent passing No. 200 sieve or 0.075 mm sieve opening) as described in Table 1 (Whitman, 1970).

Characteristics of source soils from river bank and river bed were evaluated by authors and BRTC (2005). A summary of test results are presented in Table 2. Twenty six samples were collected from the river bed of Turag, Buriganga, Dhaleshwari, Sitalakhya, Meghna and Kamrangirchar. Mean grain size (D_{50}) varies between 0.08~0.20, uniformity coefficient (C_u) varies between 1.83~2.64, fines content (F_c) varies between 2.0~45.3 and suitability varies between unsuitability to good (Table 1). Twenty nine samples were collected from the river bank and river char of Balughat, Katpatty, Dhaleshwari Char, Kholamura Ghat and Puranganj Nagar. Mean grain size (D_{50}) varies between 0.012~0.250, uniformity coefficient (C_u) varies between 2.5~21.0, fines content (F_c) varies between 7.3~95.0% and suitability varies between unsuitability to fair (Table 1).

It can be concluded by analysing the data as presented in Table 2 that river bed is better than river bank as a source. So filling material should be collected from river bed. Figure 2 compares the grain size distributions of soil samples collected from river banks and chars with the suitable grain size of soils as a hydraulic fill. It is seen that most of the source materials has very high fines content which is greater than 5% (Table 2 and Fig. 2). Since alternative sources are not available soil containing fines 5% to 15% can be selected for developing the low lands. However, if the lands are developed with high fines content, it may cause problem for ground improvement.

Table 1.
Quality of soils as dredged materials

Quality	Fines content (< # 200 sieve)
Very good	<2%
Good	2%-5%
Moderate	5%-10%
Fair	10%-15%
Unsuitable	>15%

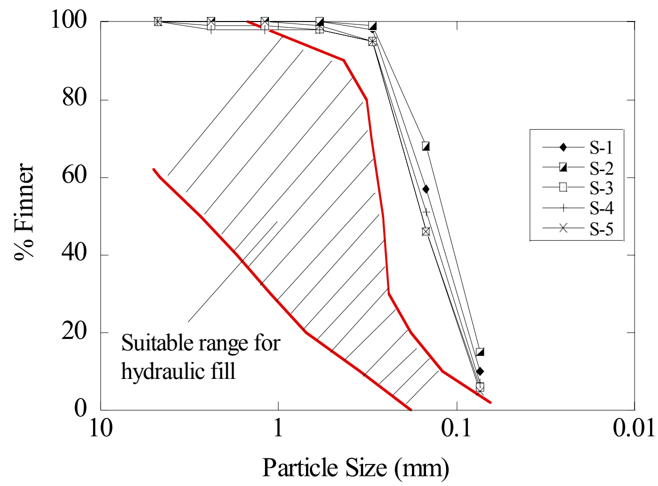

Fig. 2. Comparison of grain size distributions of soil samples collected from river banks and char with the range of particle size for suitable hydraulic fill.

Table 2
Properties of soils collected from river sources.

Source Type	Source Name	D_{50} (mm)	C_u	Fines content (<# 200 sieve)	Suitability as fill soil
River bed	Turag	0.17~0.19	2.22~2.64	2.4~9.4	Moderate~Good
	Buriganga	0.15~0.20	1.83~2.63	2.0~10.0	Moderate~Good
	Dhaleswari	0.08~0.18	2.00~2.50	5.2~33.9	Fair~Unsuitable
	Sitalakhya	0.11~0.14	–	13.3~45.3	Fair~Unsuitable
	Meghna	0.10~0.11	–	15.5~26.1	Unsuitable
River bank	Balughat	0.01~0.13	8.00~21.00	12.5~94.0	Fair~Unsuitable
	Dhaleshawar Char	0.02~0.20	2.50~10.50	9.2~95.0	Moderate~Unsuitable
	Katpatty	0.12~0.25	3.50	7.3~17.2	Unsuitable~moderate
	Kholamura Ghat	0.02~0.18	6.44~7.33	12.8~94.0	Fair~Unsuitable
	Purangonj Nagar	0.03~0.10	5.25~7.80	23.4~92.0	Unsuitable

2.3 Characteristics of developed land

Characteristics of fill materials of different reclaimed sites were tested by authors and BRTC (2005). A summary of test results are presented in Table 3. Samples from shallow depth were collected from the reclaimed sites named Bashundhara, Purbachal, Uttara Model Town (Third Phase), Mirpur DOHS and Banasree. It was found that mean grain size (D_{50}) varies between 0.03~0.25, relative density (D_r) varies between 33~86%, fines content (F_c) varies between 5.5~28.0%. It can be concluded that after being filled in reclaimed sites soil characteristics changes from its original condition (Table 2 and Table 3). It is clear that proper reclamation procedure is required to be followed for better land development.

Table 3
Properties of fill soils collected from various reclaimed sites.

Reclaimed Sites	D_{50} (mm)	D_r (%)	Fines content (<# 200 sieve)
Bashundhara	0.10~0.22	–	12.0~23.0
Purbachal	0.05~0.20	33~86	5.6~23.0
Uttara Model town (third phase)	0.06~0.25	51~82	5.5~15.1
Mirpur DOHS	0.05~0.19	–	19.0~24.0
Banasree	0.03~0.18	–	18.0~28.0

2.4 Proposal for rational land development procedure

It is observed that the quality of the dredged fill materials for developing lowland is not good. In view of above, the following procedure may be tried to obtain a fill of uniform density with adequate bearing capacity for structures:

(1) The height of mound formed at the discharge point should not exceed 5 ft. The water running away should not carry any soil beyond a distance of 30 ft. Care should be exercised to avoid ponding of water where fines might settle out to form soft pockets. If any such soft pocket is found, drainage points need to be installed to improve the soil density by de-watering or other ground improvement techniques.

(2) It is advisable to start filling operation by placing the discharge point at the lowest surface within the filling area. Once an area is filled to a pre-determined elevation, the discharge point may be moved to the lowest point in the next filling area.

(3) If necessary, a trial procedure can be performed to achieve a relative density of 70%. During field application if such density is not achieved, ground improvement technique should be applied.

To develop a rational method more investigations and study in the laboratory and field are necessary based on the above mentioned discussion. Investigations can be made by pumping the materials through vertical pipes. It may improve the distributions of the particles. However, it needs proper investigation.

3. Estimation of liquefaction potential

3.1 Methods for liquefaction potential analysis

The occurrence of liquefaction is affected by various geotechnical factors, which are classified into three categories: soil properties, geological conditions and ground motion characteristics. As a general guide, the characteristics of liquefiable soils are presented in Table 4 (Rao, 2003). Table 3 presents the soil characteristics of reclaimed sites. Comparing the liquefiable soil properties with soils of the reclaimed sites (Table 3 and Table 4) it is seen that most of the filled soil of reclaimed sites fulfilled the characteristics of liquefiable soil if an earthquake of the intensity greater than VI occurs. Therefore, it is necessary to estimate the liquefaction potential of reclaimed sites.

Several laboratory tests are in practice to estimate the liquefaction potential of soils such as dynamic triaxial test, cyclic simple shear test and shaking table test. Similarly, several empirical criteria have been developed over the years based on standard penetration test (SPT), cone penetration test (CPT)

Table 4
Characteristics of liquefiable soil

Characteristics	Value
Mean size, D_{50} (mm)	0.02 to 1.00
Fines content (d ≤0.005 mm)	< 10 %
Uniformity coefficient, C_u	< 10
Relative density, D_r	< 75 %
Plastic index, I_p	< 10
Earthquake Intensity	> VI
Depth	< 15 m

and shear wave velocity measurement for the assessment of liquefaction potential (Seed and Idriss, 1971; Seed et al., 1985; Robertson and Campanella, 1985; Youd and Idriss, 2001). However, in this study liquefaction potential of selected reclaimed areas have been determined from SPT and soil characteristics.

Liquefaction Potential Analysis Based on SPT Results

In this study, Seed-Idriss simplified procedure (Seed and Idriss, 1971), Japanese code of bridge design (Tatsuoka et al., 1980) and Japanese code of bridge design procedure based on Chinese criterion by Ishihara (1993) were used to estimate the liquefaction potential. The procedures are described below.

(1) *Seed-Idriss Simplified Procedure (Seed and Idriss, 1971)*

This procedure is known as Seed-Idriss simplified procedure, was originally proposed in 1971 and updated over the years (e.g., Seed et al., 1985; Youd and Idriss, 2001; Cetin et al., 2004). The procedure is described in following steps.

Factor of Safety (F_L)

The liquefaction potential of a sand deposit is evaluated in terms of factor of safety, F_L as per Simplified Procedure is given by Equation 1.

$$F_L = \frac{Cyclic\ Strength}{Cyclic\ Shear\ Stress} = \left[\frac{(CRR_{7.5})_{1_{atm}}}{CSR_{M_W}}\right] * MSF * K_\sigma \tag{1}$$

where, $(CRR_{7.5})_{1_{atm}}$ is the cyclic resistance ratio of the soil in question, CSR_{M_W} is the a ratio of the equivalent uniform cyclic shear stress, *MSF* is magnitude scaling factor, and K_σ is the correction factor.

If $F_L \le 1.0$, liquefaction is said to occur and otherwise liquefaction does not occur.

Cyclic Shear Stress

Here, CSR_{M_W} is expressed as follows (Seed and Idriss, 1971)

$$CSR_{M_W} = \frac{\tau_{av}}{\sigma'_v} = 0.65 \frac{a_{max}\, r_d}{g} . \frac{\sigma_v}{\sigma'_v} \tag{2}$$

where, g is the acceleration due to gravity, r_d is stress reduction coefficient ($r_d = 1.0$ for rock), a_{max} is peak horizontal acceleration at ground surface, σ_v (= γ_t z) is total overburden stress at the depth (z) in question, σ'_v is initial effective overburden stress at the same depth. The value of r_d as used in this method are:

Recommended by Youd and Idriss (2001)

$$r_d = 1.0 - 0.00765z \quad \text{for } z \le 9.15\text{m} \tag{3}$$

$$r_d = 1.174 - 0.0267z \text{ for } 9.15\text{m} \le z \le 23\text{m} \tag{4}$$

Recommended by Robertson and Wride (1998) may be used,

$$r_d = 0.744 - 0.008z \quad \text{for } 23\text{m} < z < 30\text{m} \tag{5}$$

Recommended by Youd and Idriss (2001)

$$r_d = 0.5 \text{ for } z > 30\text{m} \tag{6}$$

Cyclic Strength

For a given $(N_1)_{60_{CS}}$ and earthquake moment magnitude, M_w =7.5 the following equation modified after Youd and Idriss (2001) is used to determine $(CRR_{7.5})_{1_{atm}}$.

$$(CRR_{7.5})_{1_{atm}} = \frac{1}{34-(N_1)_{60_{CS}}} + \frac{(N_1)_{60_{CS}}}{135} + \frac{50}{[10*(N_1)_{60_{CS}}+45]^2} - \frac{1}{200} \tag{7}$$

Here, $(N_1)_{60_{CS}}$ is the corrected normalized equivalent clean sand SPT blow count.

To adjust the $(CRR_{7.5})_{1_{atm}}$ to magnitudes other than M_w =7.5, the calculated $(CRR_{7.5})_{1_{atm}}$ is multiplied by magnitude scaling factor (MSF) corresponding to the design earthquake of moment magnitude M_w. The recommended lower bound MSF values are given by the following equation:

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \tag{8}$$

The upper bound MSF values recommended for $M_w \leq 7.5$ were originally proposed by Andrus and Stokoe (1997).

To adjust the cyclic resistance ratio to initial effective overburden pressures other than 1.0 tsf, the $(CRR_{7.5})_{1_{atm}}$ is multiplied by a correction factor K_σ. Youd and Idriss (2001) based on the works by Hynes and Olsen (1999), recommended the following equation to calculate K_σ

$$K_\sigma = \left(\frac{\sigma'_{vo}}{P_a}\right)^{(f-1)} \tag{9}$$

where, P_a is the atmospheric pressure in the same unit as the initial effective overburden pressure σ'_{vo} and f is an exponent, which is the function of site conditions. The recommended value of f is 0.8 for relative density, $D_r \leq 40$ and 0.7 for relative density, $D_r = 60$.

(2) *Japanese code of bridge design (Tatsuoka et al., 1980) including Chinese criterion (Ishihara, 1993):*

Factor of safety (F_L)

The liquefaction potential of a sand deposit is evaluated in terms of factor of safety, F_L as per Japanese code of bridge design is given by Equation (10).

$$F_L = \frac{Cyclic\,Strength}{Cyclic\,Shear\,Stress} = \frac{\left(\dfrac{\sigma_{DL}}{2\sigma_0'}\right)_{20}}{\left(\dfrac{\tau_{\max}}{\sigma_v'}\right)} \tag{10}$$

where, $\left(\dfrac{\sigma_{DL}}{2\sigma_0'}\right)_{20}$ is the cyclic strength and $\left(\dfrac{\tau_{\max}}{\sigma_v'}\right)$ is the cyclic shear stress.

In Equation (10), if $F_L \leq 1.0$, liquefaction is said to occur and otherwise liquefaction does not occur.

Cyclic Shear Stress

In Equation (10), cyclic shear stress is determined by the method proposed by Seed and Idriss (1971), which has been already described in preceding Seed-Idriss simplified procedure.

Cyclic Strength

If the material is identified as clean sand with fines content less than 5%, the cyclic strength is determined using the equations as follows:

For 0.04 mm ≤ D_{50} < 0.6 mm

$$\text{Cyclic Strength} = \left(\frac{\sigma_{DL}}{2\sigma_0'}\right)_{20} = 0.0676\sqrt{N_{1(c)}} + 0.225\log_{10}\left(\frac{0.35}{D_{50}}\right) \tag{11}$$

For 0.6 mm ≤ D_{50} < 1.5 mm

$$\text{Cyclic Strength} = \left(\frac{\sigma_{DL}}{2\sigma_0'}\right)_{20} = 0.0676\sqrt{N_{1(c)}} - 0.05 \tag{12}$$

where, D_{50} is the mean particle diameter in mm, $N_{1(c)}$ is corrected SPT value obtained by Equation (18) which is determined through various correction factors as described below.

Correction for effective overburden pressure

$$N_1 = C_N N \text{ and } C_N = \frac{1.7}{(\sigma_v' + 1.7)} \tag{13}$$

Where, σ_v' is the effective overburden pressure in kgf/cm^2

Correction for fines content, F_c

If more than 5% fines are seen to exist in the soil, the measured N_1 value from Equation (13) should be increased using Equation (14) and (15).

For 5% ≤ F_c ≤ 20%

$$\Delta N_1 = 0.5\,(F_c - 5) \tag{14}$$

For F_c > 20%

$$\Delta N_1 = 7.5 \tag{15}$$

$$N_2 = N_1 + \Delta N_1 \tag{16}$$

Correction Factor for Plasticity Index, I_p

If the plasticity index (I_p) of the fines is found to be greater than 10, further correction must be made for the cyclic strength as stated in Equation (17) as given below:

$$R = \frac{I_P - 10}{45} + 1 \tag{17}$$

where, R is resistance cyclic strength of soil at a given depth and I_p is plasticity index.

So, finally Corrected SPT value, $N_{1(c)} = N_2 * R$

(18)

Chinese Criterion

On the basis of the recent large earthquakes in China, the criterion of identifying sandy deposits as being susceptible to or immune to liquefaction was presented in the form of code requirement. Through some numerical manipulation (Ishihara, 1993), the Chinese criterion can be expressed as stated in Equation 19.

$$\text{Cyclic Strength} = \left(\frac{\sigma_{DL}}{2\sigma_0'}\right)_{20} = \left(9.5N_{1(c)} + 0.466N_{1(c)}^2\right)/1000 \tag{19}$$

3.2 Parameters for analysis

Liquefaction analysis based on SPT results need ground motion characteristics, depth of analysis and earthquake intensity. In this analysis, following values of the parameters are considered.

(1) *Ground Motion and Moment Magnitude:* In the current study, the value of a_{max} has been taken as 0.15g as Dhaka city exist in the Zone 2 of seismic zonation map of Bangladesh (BNBC, 1993). Other researchers (Ansary and Rashid, 2000) also used similar values of a_{max} for the similar purpose. Earthquake ground motion is influenced by a number of factors. Most important factors are moment magnitude, epicentral distances, local soil conditions, earthquake sources, etc. In Seed-Idriss simplified procedure moment magnitude (M_w) input parameter is also important correction factor. From Table 5, it is seen that ranges of M_w at nearby faults from Dhaka varies from 7.0~8.5. However, this value can not be considered directly for Dhaka since those faults are at quite distant places from Dhaka. Due to non-availability of suitable correlations between distance and ground motion characteristics for Dhaka, the design moment magnitude is taken 7.0 for this study, which is the lowest value in Table 5.

(2) *Depth of Analysis:* Traditionally a depth of 15 m has been used as the depth of analysis for the routine evaluation of soil liquefaction. This is because past experience has shown that the 15 m depth may be adequate for the evaluation of liquefaction for most sites, in particular, for structures supported on shallow foundations. Furthermore, the widely used “Seed-Idriss simplified procedure” for liquefaction analysis developed by Seed and Idriss (1971) was based on a database of case histories with shallow depth of liquefaction. Some of the

Table 5
Maximum earthquake magnitude in different tectonic blocks (Bolt, 1987)

Tectonic blocks	Magnitude
Bogra fault zone	7.0
Tripura fault zone	7.0
Sub-Dauki fault zone	7.3
Shillong plateau	7.0
Assam fault zone	8.5

numerical quantities in the “simplified analysis” can be reasonably estimated to a depth of about 12 m. However, Seed and Idriss (1971) did not recommend a minimum depth for the evaluation of liquefaction. In the current study, liquefaction potential of the sub-soils up to 18 m from the EGL has been estimated. However, fault lines (i.e., sources of earthquake motion) are far from Dhaka and there is no scope of having epicentre just below the sites examined. So, the depth of liquefaction analysis up to 18 m seems to be sufficient.

(3) *Sources of Earthquake near Dhaka City*: Bangladesh covers one of the largest deltas and one of the thickest sedimentary basins in the world. According to Bolt (1987) considering geology and tectonic of Bangladesh and neighbourhood five tectonic blocks can be identified which have been producing damaging

earthquakes. These are Bogra fault zone, Tripura fault zone, Sub-Dauki fault zone, Shillong plateau and Assam fault zone.

4. Results and discussions

Liquefaction potential analyses have been conducted for four reclaimed sites of Dhaka namely Bashundhara, Mirpur DOHS, Banasree and Purbachal based on SPT results. In case of Bashundhara, liquefaction potential analysis was conducted at 28 borehole locations. Among them, liquefiable soil was found at 10 borehole locations. In case of Mirpur DOHS, liquefaction potential analysis was conducted at 13 borehole locations. Among them, liquefiable soil was found at 8 borehole locations. In case of Banasree, liquefaction potential analysis was conducted at 11 borehole locations. Among them, liquefiable soil was found at 5 borehole locations. For Purbachal area, liquefaction potential analysis was conducted at 3 borehole locations. Among them, liquefiable soil was found at all of these 3 borehole locations.

Table 6
Liquefaction potentiality of sub-soil at different reclaimed areas of Dhaka.

Location of the test		Properties of liquefied soil			Liquefiable depth (m)		
Reclaimed Area	BH No.	Type of soil	D_{50} (mm)	F_c (%)	M_1	M_2	M_3
Basundhara	BH-1	Fine sand	0.150	10	-	EGL~1.5	-
	BH-3	Sandy silt	0.013	50	-	1.5~4.5	1.5~4.5
	BH-21	Fine sand	0.150	09	-	EGL~12.0	EGL~12.0
	BH-22	Fine sand	0.149	10	-	EGL~13.5	EGL~ 13.5
	BH-23	Fine sand	0.149	10	-	EGL~12.0	EGL~12.0
	BH-24	Fine sand	0.152	8	9.0~12.0	EGL~12.0	EGL~12.0
	BH-25	Loose sand	0.149	18	1.5~4.5	EGL~4.5	EGL~4.5
	BH-26	Loose sand	0.149	18	1.5~4.5	EGL~4.5	1.5~4.5
	BH-27	Loose sand	0.135	19	-	EGL~4.5	EGL~3.0
	BH-28	Loose sand	0.144	20	-	EGL~7.5	EGL~7.5
Mirpur DOHS	BH-5	Fine sand	0.170	27	EGL~1.5	EGL~4.5	EGL~4.5
	BH-6	Silty sand	0.160	27	EGL ~1.5	EGL~3.0	EGL ~3.0
	BH-7	Fine sand	0.160	27	EGL~ 1.5	EGL~1.5	EGL~1.5
	BH-8	Silty sand	0.170	27	EGL~ 1.5	EGL~1.5	EGL~1.5
	BH-9	Fine sand	0.160	27	EGL~1.5	EGL~3.0	EGL~3.0
	BH-10	Silty fine sand	0.160	27	1.5~4.5	EGL~4.5	1.5~4.5
	BH-11	Silty fine sand	0.160	27	3.0~4.5	EGL~4.5	EGL~4.5
	BH-13	Fine sand	0.160	27	EGL~3.0	EGL~4.5	EGL~4.5
Banasree	BH-1	-	-	-	-	-	-
	BH-2	Silty sand	0.120	45	-	EGL~4.5	EGL~4.5
	BH-3	Silty sand	0.180	42	-	EGL~ 6.0	EGL~6.0
	BH-4	-	-	-	-	-	-
	BH-5	Silty sand	0.100	45	-	EGL~6.0	EGL~6.0
	BH-6	-	-	-	-	-	-
	BH-7	Sandy silt	0.100	53	-	1.5~4.5	1.5~4.5
	BH-9	Silty fine sand	0.150	10	EGL~3.0	EGL~3.0	EGL~3.0
Purbachal	BH-1	Silty sand	0.200	13	-	-	1.5~4.5
	BH-2	Silty sand	0.320	6	1.5~3.0	EGL~ 3.0	1.5~3.0
	BH-3	Silty sand	0.220	18	EGL~4.5	EGL~4.5	EGL~4.5

Note: M_1: Japanese code of bridge design; M_2: Japanese code of bridge design based on Chinese criterion; M_3: Seed-Idriss simplified procedure

As described in the previous sections, it is seen that liquefaction potential analysis based on Japanese code of bridge design (Tatsuoka et. al., 1980) depends on mean particle diameter (D_{50}) range (Equation 11 and 12), but the analysis based on Japanese code of bridge design based on Chinese criterion (Ishihara, 1993) and simplified procedure by Seed and Idriss (1971) do not depend on mean particle diameter (D_{50}) range (Eqn. 19). Therefore, both Japanese code of bridge design including Chinese criterion and simplified procedure have been used for preliminary investigation of liquefaction potential of selected reclaimed sites. Fines content (F_c) and mean diameter (D_{50}) of the soils up to liquefiable depth from existing ground level (EGL) have been presented in Table 6. Soil types of liquefiable depth have also been presented in the same table. Typical liquefaction potentiality analysis has been presented in Figure 3.

It is found that most parts of the Bashundhara site contain silty clay in the filling depth. But some parts contain fine sand and silty sand in the filling depth. Some parts of Mirpur DOHS site contain silty sand, some parts of the site contain silty clay and some parts contain clay only in the filling depth. The Banasree site contains mostly sandy silt and silty sand in the filling depth. The Purbachal site contains mostly silty sand in the filling depth. It is also found that fines content of the filling soil (F_c) varies between 8~99%, 77~100%, 31~100% and 6~100% for Bashundhara, Mirpur DOHS, Banasree and Purbachal site, respectively. Mean particle diameter (D_{50}) of the filling soil varies between 0.01~0.15 mm, 0.16~0.17 mm, 0.007~0.180 mm, 0.20~0.22 mm for Bashundhara, Mirpur DOHS, Banasree and Purbachal site, respectively.

From Table 6, it is seen that liquefaction depth from existing ground level (EGL) varies between 1.5~13.5 m, 1.5~4.5 m, 1.5~6.0 m and 1.5~4.5 m for Bashundhara, Mirpur DOHS, Banasree and Purbachal site, respectively. However, some parts of Bashundhara, Mirpur DOHS and Banasree sites contain only clay and silty clay in the filling depth. Hence, sub-soil of those parts is not liquefiable.

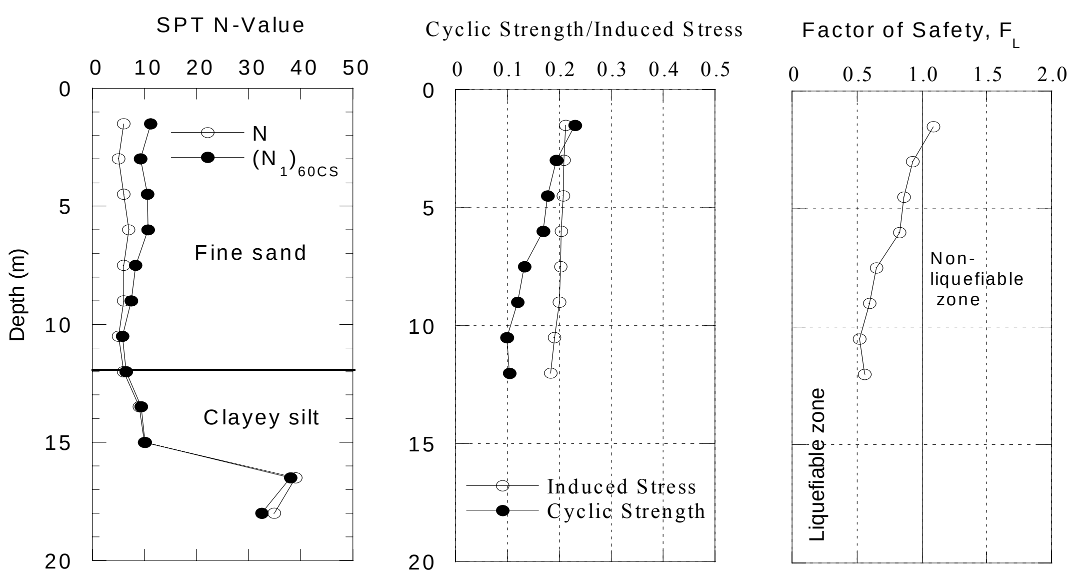

Fig. 3. Typical liquefaction potentiality analysis at BH No. 23 of Bashundhara using Seed and Idriss (1971) method (M_w= 7.0, a_{max}= 0.15g)

5. Conclusions and recommendations

Many reclaimed areas are being developed in and around Dhaka city due to continuously growing urbanization. Dhaka city is at seismic risk. This study investigates the shortcomings of the development procedure and liquefaction potential of reclaimed areas of the city. The major findings of this study are:

(1) Most of the reclaimed areas are being developed by filling dredge materials of river bed which is silty sand. However, the quality of this material does not meet with the requirement of suitable dredge fill material. Since this filling material's quality is not suitable and these silty sand layer is not well compacted, these reclaimed area is susceptible to liquefaction if an earthquake of sufficient energy occurs.

(2) A proposal for rational land development has been proposed in this study. However, it needs proper investigation.

(3) Liquefaction potential of four reclaimed sites of Dhaka has been estimated using Japanese code of bridge design including Chinese criterion and simplified procedure by Seed and Idriss (1971). It is found that some parts of the sites are liquefiable to some depth. It is found that liquefaction depth from existing ground level (EGL) varies between 1.5~13.5 m, 1.5~4.5 m, 1.5~6.0 m and 1.5~4.5 m for Bashundhara, Mirpur DOHS, Banasree and Purbachal site, respectively. Since in the most places, the liquefaction depth is shallow it may not cause problems for deep foundations. However, it may damage roads, lifelines, etc.

(4) The liquefaction potential of the reclaimed sites has been estimated based on three different methods. But three methods gave different results as it is seen in Table 6. But in many sites Japanese code of bridge design based on Chinese criterion and Seed-Idriss simplified procedure gave similar results. However, Japanese code of bridge design gave different results in almost all cases. Japanese code of bridge design considers the effect of mean grain size. But other two procedures do not consider this property. This may be the governing reason of the difference in results. Since Japanese code uses both the mean grain size and fines content, it may be more reliable than other two methods.

(5) Since the results obtained are different by different methods, it is necessary to confirm the estimated results. The liquefaction potential of the studied area can be estimated by other field test like shallow seismic survey where shear wave velocity of various soil layers is determined. Then these results may be compared with the results obtained in this study to propose a suitable method for estimating liquefaction potential of Dhaka soil.

Further studies/researches are going on at BUET to determine the liquefaction potential of such areas based on other field test(s) data such as shallow seismic survey data and also for the rational development procedure of reclaimed sites to avoid future possible hazard due to earthquake.

References

Andrus, R. D. and Stokoe, K. H., (1997). Liquefaction resistance based on shear wave velocity, Proc., NCEER Workshop on Evaluation of Liquefaction Resistance of Soil, National Centre for Earthquake Engineering Research, State University of New York at Buffalo, pp. 89~128.

Ansary, M. A. and Rashid, M. A. (2000). Generation of liquefaction potential map for Dhaka, Bangladesh, Proceedings of 8th ASCE Speciality Conference on Probabilistic Mechanics and Structural and Reliability, Paper No. PMC 200~061.

BNBC (1993), "Bangladesh National Building Code." House and Building Research Institute, Bangladesh Standards and Testing Institutions.

Bolt, B. A. (1987). Site specific study of seismic intensity and ground motion parameters for proposed Jamuna River Bridge, Bangladesh. Jamuna Multipurpose Bridge Authority, Govt. of the People's Republic of Bangladesh, pp. 1~36.

Bureau of Research, Testing and Consultation, BRTC (2005). Development proposal for Uttara Residential Model Town (3rd Phase) Project, Final Report, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Cetin, K. O., Seed, R. B., Kiureghian, A. D., Tokimatsu, K., Harder, L. F., Kayen, R. E., and Moss, R. E. S. (2004). Standard penetration test based probabilistic and deterministic assessment of seismic soil liquefaction potential, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 130, No. 12, pp. 1314~1340.

Dhaka Metropolitan Development Plan, DMDP (1995-2015), Dhaka Structure Plan (1995~2015), December 1995, Dhaka Metropolitan Development Plan Package prepared by DMDP Project, Rajuk (MOHPW) under authority of the Government of the People's Republic of Bangladesh using UNDP/UNCHS-BGD/88/052 Technical Assistance, Vol. 1.

Gratchev, I. B., Sassa, K., Osipov, V. I. and Sokolov, V. N. (2006). The liquefaction of clayey soils under cyclic loading. Engineering geology, Vol. 86, pp. 70~84.

Idriss, I. M. and Boulanger, R.W. (2004). Semi-empirical procedures for evaluating liquefaction potential during earthquakes, Soil dynamics and Earthquake Engineering, Vol. 26, pp. 115~130.

Ishihara, K. (1993). Liquefaction and flow failure during earthquakes, The 33rd Raknine Lecture of the British Geotechnical Society, Reprinted from Geotechnique, Vol. 43, No. 3, pp. 349~415, the Institution of Civil Engineers.

Islam, M. R. (2005). Seismic Loss Estimation for Sylhet City. M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Islam, M. S. and Ahamed, S. (2005). Liquefaction potential of selected reclaimed areas of Dhaka city, Proceedings of the Third Annual Paper Meet and International Conference on Civil Engineering, The Institute of Engineers, Bangladesh.

Khan, S. H. (1988). Soils Liquefaction Possibility in Bangladesh. M. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Rahman, M. G. (2004). Seismic Damage Scenario for Dhaka City. M.Sc. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Rao, K. S. (2003). Evaluation of liquefaction potential for seismic microzonation of Delhi region. Proceedings of the Twelfth Asian Regional Conference on Soil Mechanics and Geotechnical Engineering, Vol. 1, Singapore, 2003. (CF Leung, K K Phoon, Y K Chow, CITeh & K G Young editors).

Rashid, M. A. (2000). Seismic Microzonation of Dhaka City Based on Site Amplification and Liquefaction. M. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Robertson, P. K. and Campanella, R. G. (1985). Liquefaction potential of sands using the CPT, J. GED, ASCE, Vol. 111, No. 3, pp. 384~403.

Robertson, P. K. and Wride, C. E. (1998). Evaluating cyclic liquefaction potential using cone penetration test, Canadian Geotechnical Journal, Vol. 35, No. 3, pp. 442~459.

Saha, A. (2005). Development of Liquefaction Potential Map for Rangpur Town. M. Engg. Thesis, Department of Civil Engineering, Bangladesh University of Engineering and Technology, Dhaka, Bangladesh.

Seed, H. B., and Idriss, I. M. (1971). Simplified procedure for evaluating soil liquefaction Potential. Journal of the Soil Mechanics and Foundations Division, ASCE, Vol. 97, No. SM9, Proceedings Paper 8371, pp. 1249~1273.

Seed, H. B., Tokimatsu, K., Harder, L. F., and Chung, R. M., (1985). Influence of SPT procedures in soil liquefaction resistance evaluations, Journal of Geotechnical Engineering, American Society of Civil Engineers, Vol. III, No. 12, pp. 1425~1445.

Tatsuoka, F., Iwasaki, T., Tokida, K., Yasuda, S., Hirose, M., Imai, T. and Kon-no, M. (1980).

Standard penetration test and soil liquefaction potential evaluation, Soils and Foundations, Vol. 20, No. 4, pp. 95-111.

Whitman, R. V. (1970). Hydraulic fills to support structural loads, Journal of Soil Mechanics and Foundations Division, ASCE, Vol. 96, No. SM1, pp. 23~47.

Youd, T. L. and Idriss, I. M. (2001). Liquefaction resistance of soils, Summary from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluations of Liquefaction Resistance of Soils, ASCE, Vol. 127 (4), pp. 297~313.

Yunmin, C., Han, K. and Ren-peng, C. (2005). Correlation of shear wave velocity with liquefaction resistance based on laboratory tests. Soil Dynamics and Earthquake Engineering, Vol. 25, pp. 461~469.