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Assessment Of Liquefaction Potential From Sirajganj To Kurigram Area, Bangladesh

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Abstract: *This research has evaluated the liquefaction potential from Sirajgonj to Kurigram around 140 kilometers along the Jamuna River based on the standard penetration test (SPT) method by a simplified approach. Liquefaction potential analysis was performed by using LiquefyPro, version 5 (2005) software for the selected peak ground acceleration value (0.28) at particular earthquake magnitudes 5.5 and 7.5. Liquefaction potential zone established by assessing the characteristics of soil properties such as grain size, relative density, the thickness of sandy soils, the factor of safety, ground damage, and settlement. Soils of these regions are susceptible to liquefaction at earthquake magnitude greater than or equal to 5.5. At a lower earthquake magnitude (M = 5.5), liquefaction occurred up to a depth of 10 m inSirajganj, Bogura, Gaibandha, and Kurigram. Whereas, at a higher magnitude (M = 7.5), liquefaction occurred up to 15 to 20m depth along the whole studied area. It was concluded that at greater depths (> 15 to 20 m), soils are dense to very dense and non-liquefiable. A hazard zonation map produced based on the factor of safety and other analyzed parameters of soil. Liquefaction potential zone was identified and the map could effectively be used for planning, mitigation, sustainable development of the researched areas.*

Keywords: *Liquefaction, Earthquake Magnitude, Peak Ground Acceleration (PGA), Factor of Safety (Fs), Standard Penetration Test (SPT), Settlement, Ground Damage.*

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I. Introduction

Bangladesh is a natural hazard-prone country because of its geographical position and tectonic settings. The substantial development of modern infrastructure depends on its sustainability. Sustainable development of cognitive engineering structures should be risk-free or hazard less. Natural hazards are responsible for foundation failure or significant damage to the engineering structure. Liquefaction is one of the leading hazards within these and during earthquake tremors might be one of the crucial reasons for the imminent destruction of engineering structures. The ultimate damage of an engineering structure in liquefied soils can occur because of increasing pore water pressure or loss of its shear strength. Saturated or partially saturated alluvial soils are prone to liquefy. Within a tectonic framework, an active earthquake region with a distinct peak ground acceleration (PGA) is essential for liquefaction. Pore water pressure, soil density, grain size distribution, and the significant thickness of alluvium soils exert an immense effect on considered grounds for liquefaction potential. The regional groundwater table also plays a vital role in the case of liquefaction potential.

From the pristine record, it had been observed that the Jamuna River bank shifted or migrated in both sides by eroding recently deposited loose, unconsolidated alluvial soils. Therefore, many engineering structures, i.e., Bridge, Rail line, Roads, Spurs, Buildings, and protected or unprotected embankment along the riverbank can be considered for potential risk or damage. Riverbank, protected, and unprotected embankment failure is continuously being reported in the national dailies. Many Spurs were constructed on the flowing Jamuna River from 2000 to 2015 and damaged at once after completed construction. There is no significant research found for identifying the causes of damage to these engineering structures. Is there any prominent influence of liquefaction potential on structures? It is indispensable to find out the potential reasons behind the destruction of the Spurs which were constructed within a range of liquefiable soil layers.

In this paper, a comprehensive assessment of liquefaction potential is presented from the shallow to a greater depth in an area of 140 kilometers. Preliminary work was carried out for minor parts at a point within 2-3 km near Simla Spur-2, Sirajganj, based on limited borehole data (Hossain and Haque, 2017). Therefore, a thorough analysis was required for identifying the liquefaction potential zone along the West Bank of Jamuna from Sirajgonj to the Kurigram region. Based on detailed works, one can get information about the liquefaction potential zone, risk and mitigation measures of a specific construction site. It is efficient for choosing a suitable location and foundation type for specific engineering structures.

Practical evaluation of liquefaction carried out by assessing through fieldwork and laboratory tests. The standard penetration test (SPT) was conducted to collect information on the subsurface geology and groundwater table during the field investigation. Soil properties were determined by laboratory testing. The intended results of the empirical calculation demonstrated with maximum peak ground accelerations (PGA) 0.28 at two particular earthquake magnitudes, M = 5.5 and M = 7.5.

II. Location And General Geology Of The Study Areas

The study areas include the north-western part of Bangladesh adjoining the western bank of Jamuna River. It lies between 24°30′N to 25°45′N latitudes and from 89°15′E to 90°00′E longitude. This region comprises four districts Sirajgonj, Bogra, Gaibandha and Kurigram with an aerial extension of 140 kilometers. Location map of Bangladesh and its surrounding area with adjacent to Jamuna River and shown as an enlarged vector image with boreholes position (Fig. 1). It locates between the Barind and Madhupur tract. So, the general geology correlated and consistent with the Barind tract. Lithology and the stratigraphic succession of the Barind tract listed in Table 1 modified after (Huq et al., 1991) and (Khan et al., 1995). A typical river cut soil succession presented in Fig. 2

III. Seismotectonics Of The Investigated Areas

According to the Bangladesh National Building Code (BNBC, 2015), Bangladesh is divided into four seismic zones shown in (Fig. 3). The investigated region pinpointed with Zone-III. The precise map is showing peak ground acceleration (PGA) value 0.28, which specified for the examined section including Chittagong Hill tracts and its surroundings. It is considered that the maximum PGA value is providing a 2% probability of exceedance within 50 years. Bangladesh and it's surrounding area has experienced at least 1000 frequent earthquakes having magnitude M ≥ 4.0 in the past 100 years. Seismotectonics of Bangladesh and its surrounding were re-evaluated using earthquake records (Ansary and Akhter 2018) and represented in (Fig. 4).

Table 1: The study areas generalized stratigraphic successions with its surroundings (modified after Khan, 1991 and Huq et al., 1991).

Age	Group	Formation	Lithology
Holocene (Recent)		Alluvium	Gray to grayish-brown color unconsolidated sand, silt, and clay with organic matter.
----------Unconformity----------			
Pleistocene		Barind Residue	Brownish gray to light brown color plastic clay, clayey sand and sandy clay with few concretions.
----------Unconformity----------			
Pliocene			Yellowish-brown medium to coarse-grained sandstone with the trace of mica, clayey silt, and pebbles
----------Unconformity----------			
Miocene	Tipam	Girujan Clay	Claystone with siltstone and sandstone.
		------Unconformity----------	
		Tipam	Fine to medium and coarse-grained sandstone, sandy silt, silty shale, and siltstone.

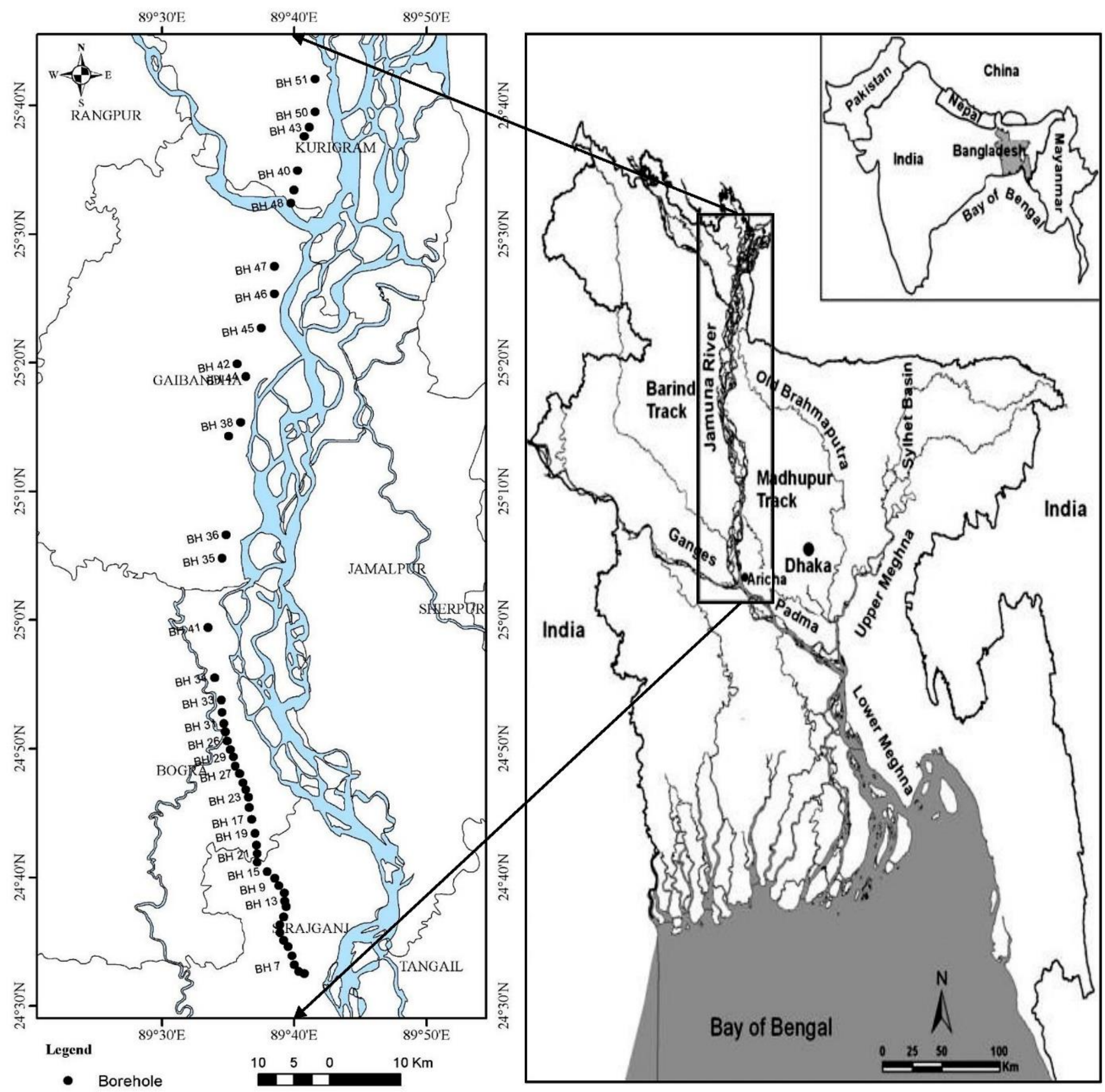

Fig. 1. Location map of the study area with the position of the boreholes.

Fig. 2. A typical stratigraphic succession of observed sediments in the study area.

IV. Materials And Method

Site investigations were executed under British Standard (BS 5930, 1999). Standard penetration tests (SPT) were performed according to British Standard (BS 1377-9, 1990) at 1.5 m intervals by using a donut-type hammer with the rope. The light cable percussion drilling method was used for collecting selected samples and wash boring techniques were used for borehole advancement. Fifty-one boreholes at particular locations were drilled (Fig. 1) to obtain the essential SPT data. During fieldwork, disturbed and undisturbed soil samples were collected and geologic information was recorded in a borehole log sheets through the whole drilling depths. The maximum and minimum drilling depth was 45 m and 11 m. The groundwater level is shallow around a 2.0–5.0 m was recorded from the boreholes (excepts the boreholes higher than 5.0 m). Local variation of groundwater level is close to the seasonal rainfall and flood level. Most of the land inundated during the flood and rainy season. Therefore, the groundwater level was considered 0.5 m below the ground surface for each selected location in this analysis.

There is a fundamental approach to predict the liquefaction potential of soil by using in situ tests and empirical methods. Theoretical development of liquefaction evaluation started when Seed and Idriss (1971) published a prevalent process termed as "simplified procedure" based on empirical work. It is recognized as a global standard which has modified and improved through (Seed, 1979; Seed and Idriss, 1982; Seed et al., 1985; Idriss, 1982; Youd and Idriss, 1997; Youd et al., 2001; Cetin et al., 2004 and Idriss and Boulanger, 2006).

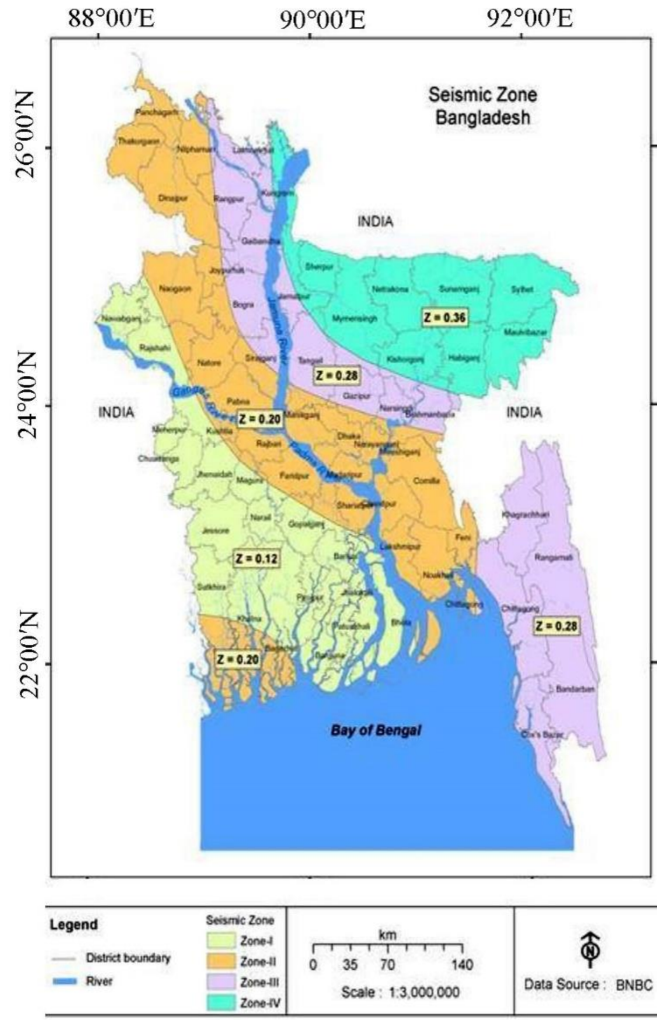

Fig. 3. Seismic Zoning map of Bangladesh (BNBC, 2015).

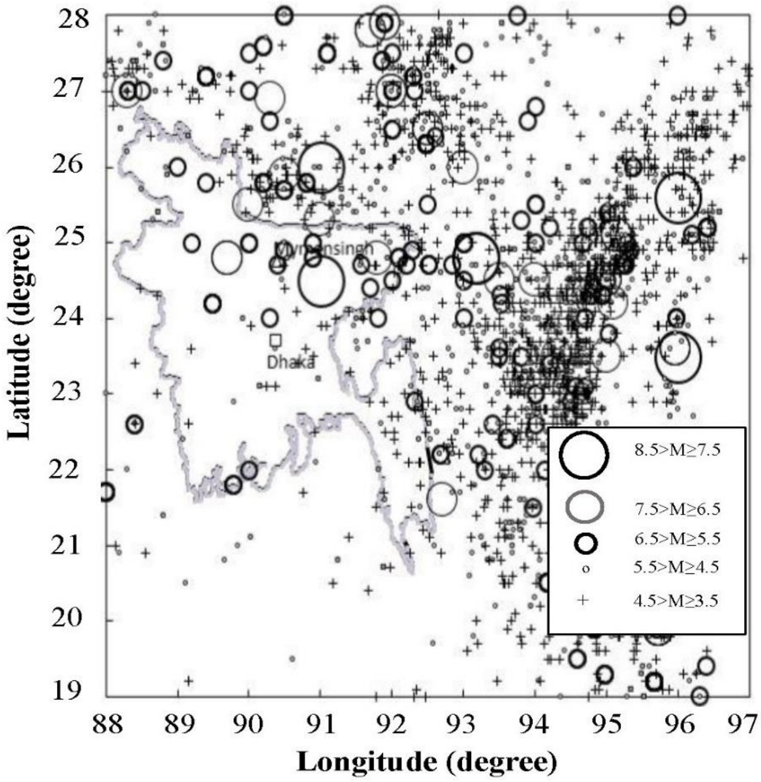

Fig. 4. Seismic activity in Bangladesh and its surrounding area (Ansary and Akhter, 2018).

The standard procedures are used in this research paper to assess the liquefaction resistance of a soil deposit. Predicting the liquefied thickness of favorable soil based on the standard penetration test (SPT). Liquefaction potential has evaluated based on a simplified approach thorough (Seed and Idriss, 1971; Seed and Idriss, 1982; Youd and Idriss, 1997; Martin and Lew, 1999; Youd et al., 2001). Settlement amounts of the soil have been calculated by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992) methods. The preferred "Boundary Curves" method is used for the manifestation of ground damage (Ishihara, 1985) because of liquefaction. Liqueifypro, Version 5 (2011) software using SPT data analyzed liquefaction potential of considered soils of the study areas. The well-established procedure for using field SPT data for liquefaction potential estimation is:

Estimate the cyclic stress ratio (CSR), which relates to the peak acceleration (a_{max}) at the ground surface during the design earthquake developed by Seed and Idriss (1971).

$$\mathrm{CSR} = \frac{\tau_{av}}{\sigma_0^{'}} = 0.65\left(\frac{a_{max}}{g}\right)\left(\frac{\sigma_0}{\sigma_0^{'}}\right)r_d$$

Where,

τ_{av} = average cyclic shear stress induced by design ground motion.

$\sigma_0^{'}$ = initial static effective overburden stress on the sand layer under consideration.

σ_0 = initial total overburden stress on the sand layer under consideration.

a_{max} = peak ground acceleration in g's, g is the acceleration of gravity.

r_d = a stress reduction factor is varying from a value of 1.0 at the ground surface to a value of 0.5 at a depth of about 30 m.

The cyclic resistance ratio (CRR) is considered functional as the resisting force of soils liquefaction and computed by the following equation, as noted (Youd et al., 2001).

$$\mathrm{CRR} = \mathrm{CRR}_{7.5}.\mathrm{MSF}$$

$\mathrm{CRR}_{7.5}$ is the cyclic resistance ratio for the considerable earthquakes magnitude 7.5 and can be calculated by:

$$\mathrm{CRR}_{M=7.5} = \frac{1}{34-(N_1)_{60}} + \frac{(N_1)_{60}}{135} + \frac{50}{[10.(N_1)_{60}+45]^2} - \frac{1}{200}$$

MSF is the magnitude scaling factor. It used for a magnitude more lesser or greater than 7.5 calculated by (Seed and Idriss, 1982):

$$\mathrm{MSF} = \frac{10^{2.24}}{\mathrm{M}_{\mathrm{w}}^{2.56}}$$

It noted (Youd et al., 2001) that, corrections of field SPT values [$N_{1(60)}$] calculated according to the following formula by considering this correction factor as listed by (Robertson and Wride, 1998):

$$\mathrm{N}_{1(60)} = \mathrm{N_m C_N C_e C_b C_r C_s}$$

Where,
N_m = measured SPT values during the field investigation.
C_N = depth factor.
C_e = hammer energy ratio (ER).
C_b = borehole diameter.
C_r = rod length.
C_s = correction factor for samplers with or without liners.

The following equation presents the factor of safety (Fs) against liquefaction:

$$\mathrm{Fs} = \frac{\mathrm{CRR}}{\mathrm{CSR}} = \frac{\mathrm{CRR}_{7.5}}{\mathrm{CSR}} \times \mathrm{MSF}$$

Assessment of potential liquefaction hazards was carried out by the simplified procedure. For specific cases of earthquake magnitude 5.5 and 7.5 where PGA value is 0.28 and the groundwater level is 0.5 m below the ground surface. In this research, liquefaction potential is defined as a determined value of the factor of safety (Fs) and represented as: liquefiable (Fs < 1.0), marginally liquefiable (1 < Fs ≤ 1.2) and non-liquefiable (Fs > 1.2).

V. Experimental Results And Discussion

The analyzed results are discussed and presented for identifying liquefaction potential zone in the following ways: Distinctive characteristics of the considered soils and probability of liquefaction is discussed based on the graphs (Figs. 5 to 8). For the same PGA values, variation in the factor of safety and potential liquefaction depth for changing earthquake magnitude is illustrated in figures (Figs. 9 to 10). The settlement that occurred during liquefaction is graphically presented with comparing calculated methods and earthquake magnitudes are shown in Figures (Figs. 11 to 13). Ground damage because of liquefaction is estimated by the boundary curve method as illustrated in Fig. 14. A potential liquefaction hazard zonation map (Fig. 15) is prepared and discussed for the effective use of sustainable development planning of the investigated regions.

The distinctive characteristics of soils are illustrated in Fig. 5. The green colors at the top mark the soils characterized by silty clay and medium-stiff to stiff clayey silt with fine sand up to a depth of 3.0 to 6.0 m. Middle layers are loose to medium dense silty fine sand, which is marked by the deep yellow color and extended up to 20 m. Sometimes it continues greater than this depth. This distinct layer is the most potential zone for liquefaction. Gray dense to very dense medium to fine sand with scattered gravel and presence of mica extended from 20 to 45 m at the bottom is characterized by the last two units of the soil profile (light yellow and ash color). These last two units of soils are considered non-liquefiable. It is observed from the grain size analysis results: soils dominated by sand, which contains 8% to 98% sand, 2% to 81% silt and 2% to 22% clay. The direct results define the soils as silty fine sand to clayey silt. According to British soil classification (BS 5930, 1999), the clay soils are characterized as intermediate to low plasticity (CI to CL) and silty clay inorganic to organic nature. Undrained triaxial tests indicate that clay soils are normally to slightly over consolidated in nature.

From the grain size distribution curves, it is found that the sand samples are uniformly graded fall within the range of 0.05 to 4 mm (Fig. 6). Most of the samples lie within 0.05 to 1.0 mm range, which shows that these soils are potential to liquefy. The direct results are much more consistent with (Tsuchida, 1970) and (Iwasaki, 1986). Shear strength test results suggest that the granular soils had an angle of internal friction (φ) value ranges from 34° to 42° and a cohesion (c) value zero. Geomechanical aspects of Jamuna River soils are discussed at Mahabub and Hossain (2015).

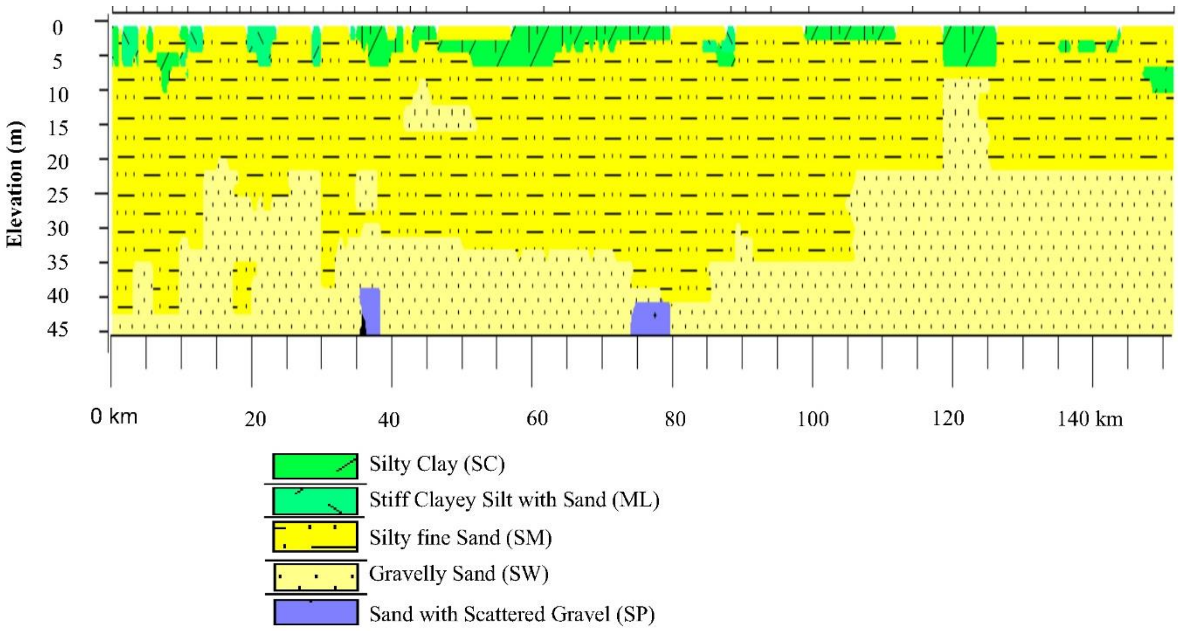

Fig. 5. Generalized vertical distribution of soils of the study area based on boreholes lithology.

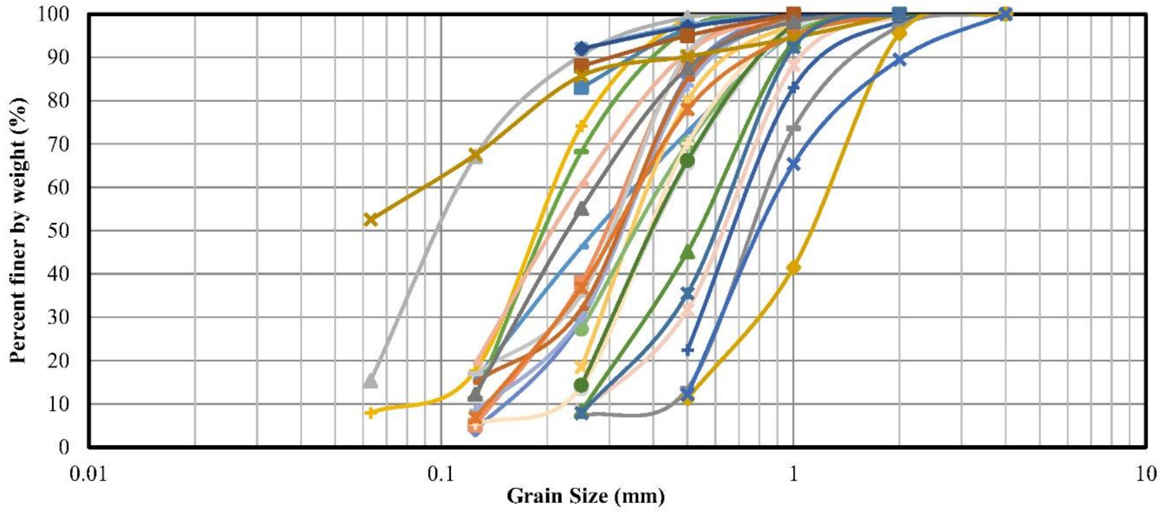

Fig. 6. Particle size distribution curves of soil samples of the investigated area.

Soil layers recognized with an SPT value less than 22 and or fewer than 30 as liquefiable revealed by (Seed et al., 1985) and (Marcuson et al., 1990). The determined SPT values are consistent with (Seed et al., 1985) and (Marcuson et al., 1990) for liquefiable soils. All liquefiable zones correspond to a distinct layer of sand, silty sand, silty clay, and a complex mixture of sand and gravel. Nevertheless, liquefaction does not occur in sandy soils or sand-gravel mixtures at a greater depth (>15-20 m) because of their higher relative densities. It is observed that dense to very dense sandy soil below 15-20 m depth exhibits higher SPT values (>30) considered safe and non-liquefiable as shown in Fig. 7.

Liquefaction analyzes for PGA value 0.28 were evaluated by the CSR versus corrected SPT, [$N_{1(60)}$] plot based on (Youd et al., 2001) and shown in Fig. 8. Silty fine sand to clayey silt or sand, sand, and sand with scattered gravel identified as liquefiable. Those samples falling to the left of the line (FC=35%) are classified as liquefiable soils and right of the line (FC≤5%) are non-liquefiable. Fine contents 15% (< 0.005 mm) is preferable for liquefaction (Seed et al., 1983). Fine materials in between 5% to 35% of soil are considered marginally liquefiable.

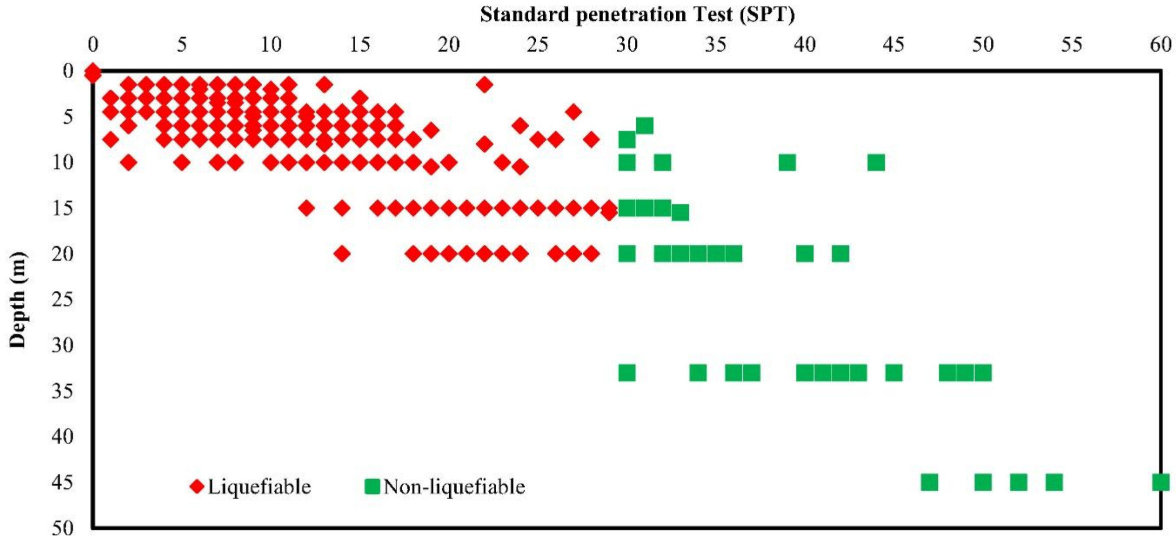

Fig. 7. The relative density of soils based on SPT values with comparing liquefiable depths for particular boreholes.

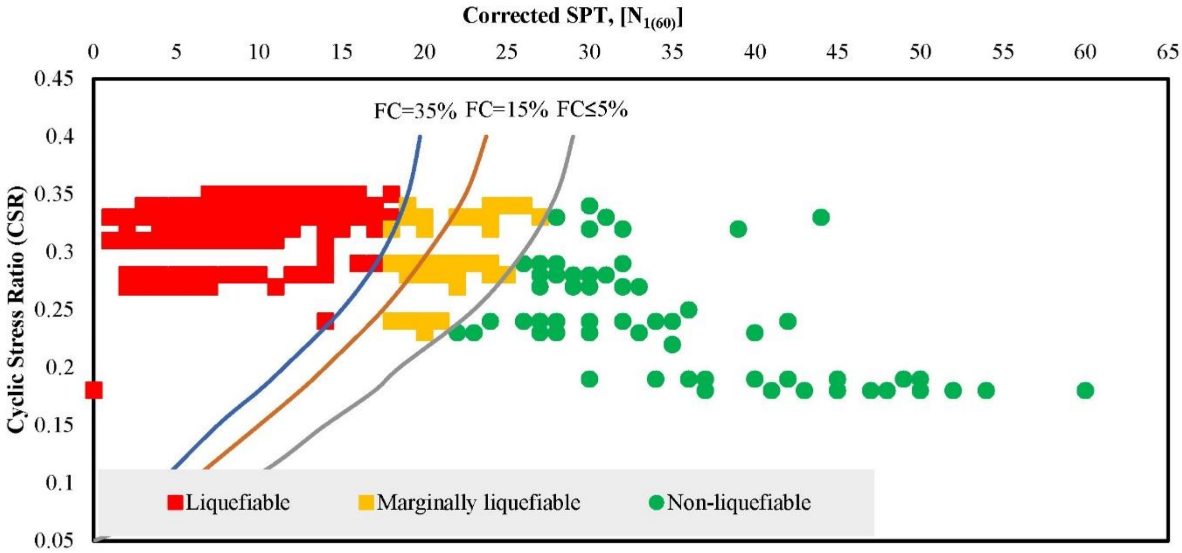

Fig. 8. Liquefaction susceptibility against CSR versus corrected SPT, $[N_{1(60)}]$ for significant fine content.

Liquefaction potential is defined based on the obtained factor of safety values and is presented against specific depth (Fig. 9 and Fig. 10). The graphic distribution of the factor of safety (Fs) values: most of the liquefiable and marginally liquefiable soils concentrated on the range of 0.0-10 m depth for low earthquake magnitude (M = 5.5) shown in Fig. 9. Similarly, soils at high earthquake magnitude (M=7.5) are liquefiable and marginally liquefiable at an extensive range of 15-20 m depth (except few boreholes) shown in Fig. 10. Liquefiable or marginally liquefiable soils concentrated in upper to middle parts and non-liquefiable distributed in lower parts of the study area.

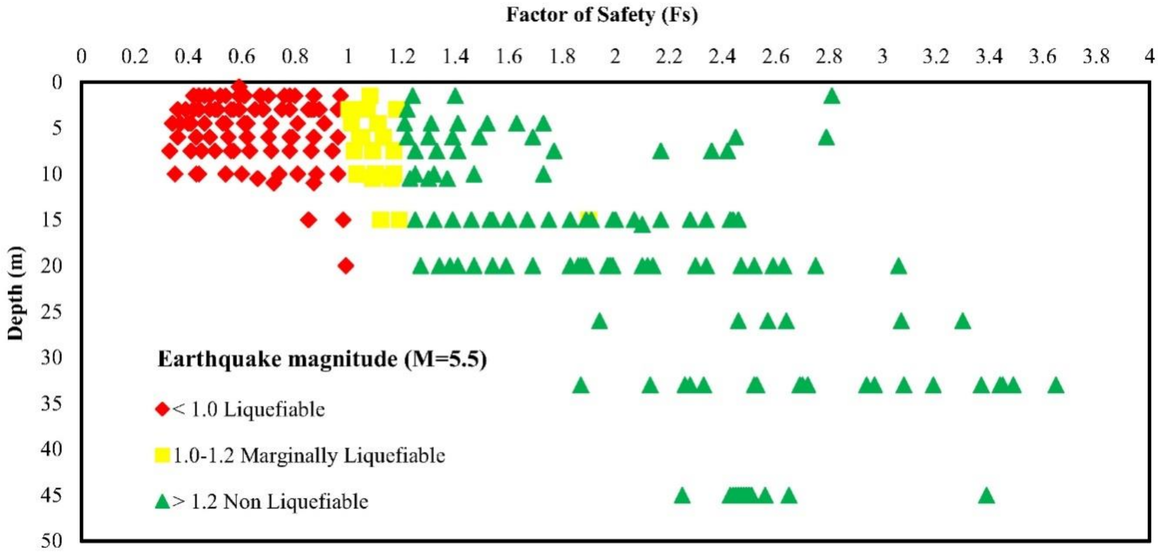

Fig. 9. Distribution of the factor of safety against liquefaction depth for low earthquake magnitude (M=5.5) at 0.28g.

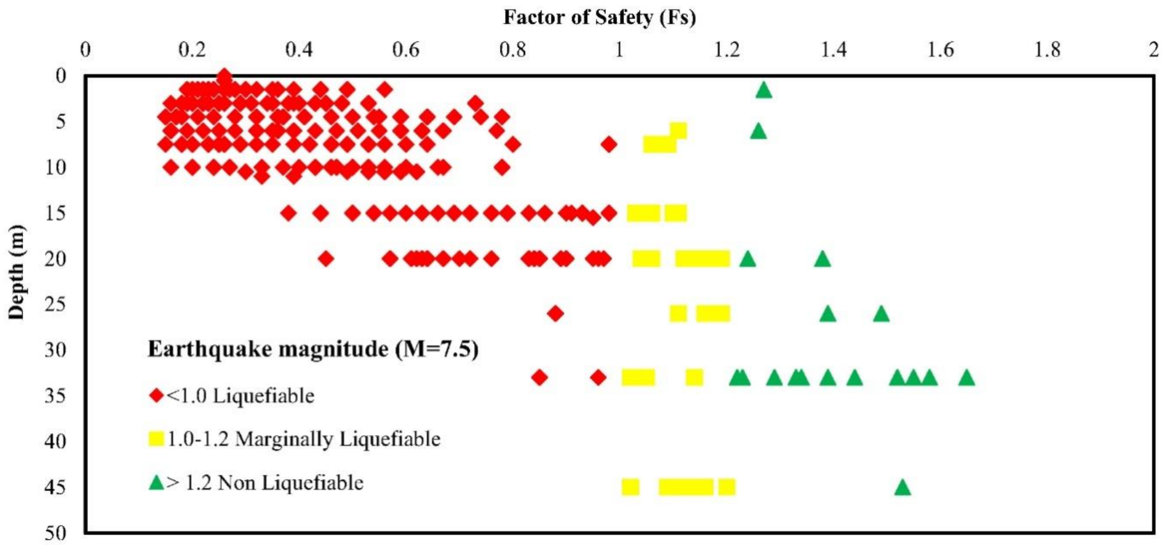

Fig. 10. Distribution of the factor of safety against liquefaction depth for high earthquake magnitude (M=7.5) at 0.28g.

From the comparative analysis of settlement data of the saturated soil for both methods (Tokimatsu and Seed, 1987) and (Ishihara and Yoshimine, 1992): it is established that the maximum settlement that occurred up to a depth of 10-20 m and below this depth a little settlement is found in some boreholes. In Tokimatsu and Seed (1987) method, the settlement is the same for earthquake magnitude 5.5 and 7.5 (Fig. 11). In Ishihara and Yoshimine (1992) method, the settlement is variable for earthquake magnitudes 5.5 and 7.5 (Figs. 12 & 13). In Tokimatsu and Seed (1987) method, the maximum and minimum settlements were observed 85 cm and 25 cm for low and high earthquake magnitudes respectively. In Ishihara and Yoshimine (1992) method: the minimum and maximum settlement were found 18 cm and 74 cm for low magnitude (M=5.5). Similarly, the minimum and maximum settlement was 15 cm and 115 cm for high magnitude (M=7.5) as shown in (Figs. 12 & 13). The settlement amount of soils is increased with increasing earthquake magnitude at the same peak ground acceleration (PGA). The Settlement amount is decreased with increasing depth (Figs. 11, 12 & 13). The thickness of the potential liquefiable soil layer is consistent with settlement and ground damages. A resemblance is observed between the two methods in terms of possible liquefaction depths based on the settlement amount.

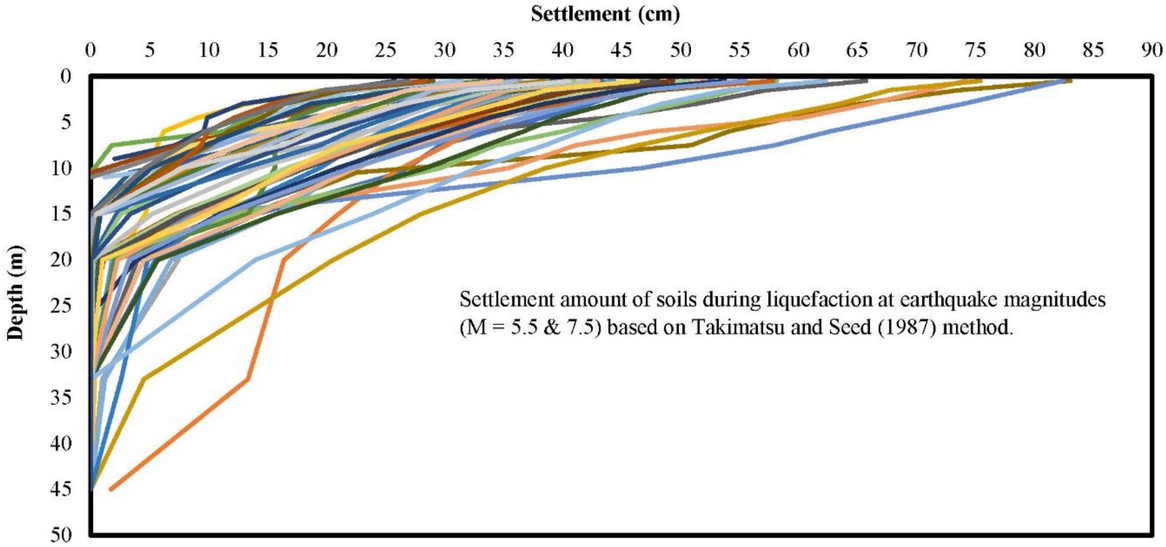

Fig. 11. Estimated settlement amount with the variation of depth at low and high earthquake magnitudes (M = 5.5 & 7.5).

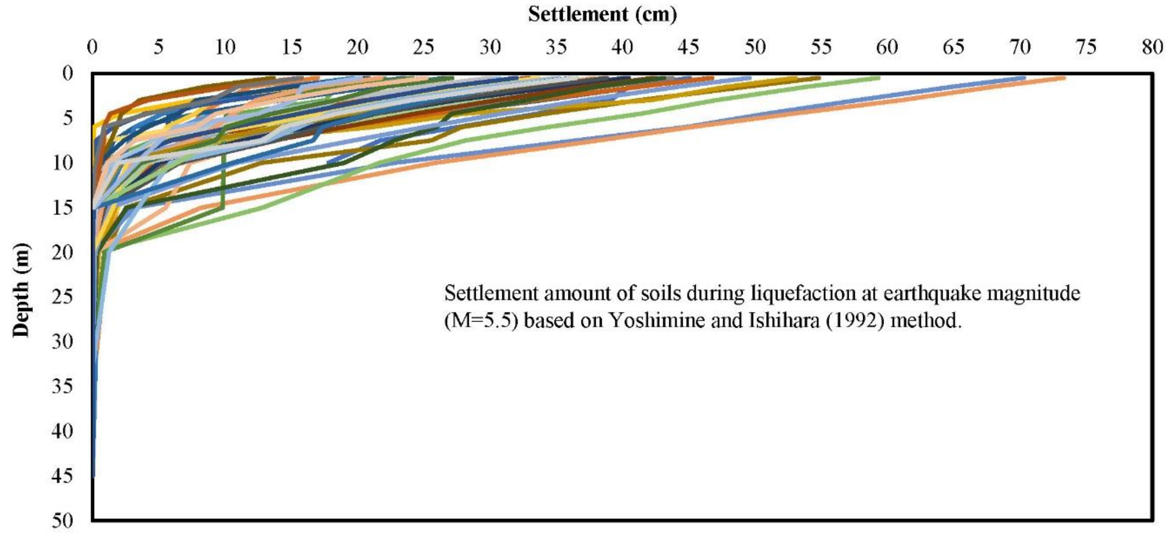

Fig. 12. Estimated settlement amount with the variation of depth at low earthquake magnitude (M=5.5).

According to Ishihara (1985) method, ground damage because of liquefaction is shown in Fig. 14. The evolution of ground damage is estimated based on SPT calculation by using "Boundary Curves." Ishihara calculates the thickness of the soils for potentially liquefiable (H_2) and non-liquefiable layers (H_1) at a given site. The relation between liquefiable and non-liquefiable soil layers is presented in Fig. 14 with a typical chart (for the maximum ground acceleration of 0.25g). The thickness of potentially liquefied soil layers is extended up to 22.5 m depth (except the boreholes which drilled less than 20 m and in three boreholes, it is extended up to 28.5 m). Most of the cases liquefied soil layers thickness is around 15 to 20 m. From the analysis, it established that the ground damage due to liquefaction would be significant and consistent with settlements.

Advancement of information technology and computer application program is beneficial for data representation. Nowadays, it used for illustration, explanations, or interpretation of data or information. In this research, an attempt has taken to identify the sub-surface liquefaction potential zone of the investigated area by implementing GIS software. For these reasons, the factors of safety (Fs) values are calculated at different depth intervals for earthquake magnitudes (M=5.5) and (M=7.5). It is presented vertically from the ground surface to 45 m depth based on the factor of safety values by using three different colors on the map. The factor of the safety values plotted gradually at 0.5, 1.5, 3.0, 4.5, 6.0, 7.5, 10.5, 15.0, 19.5, 26.0 or 33.0, and 45.0 m depth. Therefore, Liquefaction potential hazards map (Fig. 15) is illustrated the liquefiable zone in the cases of both earthquake magnitude, M=5.5 (left) and M=7.5 (right):

- Liquefiable and marked by the red color when the factor of safety (Fs) value is less than 1.0.
- Marginally liquefiable when the (Fs) value ranged between 1.0 - 1.2 and marked as a yellow color.
- Non-Liquefiable when the (Fs) value greater than 1.2 and marked as a green color.

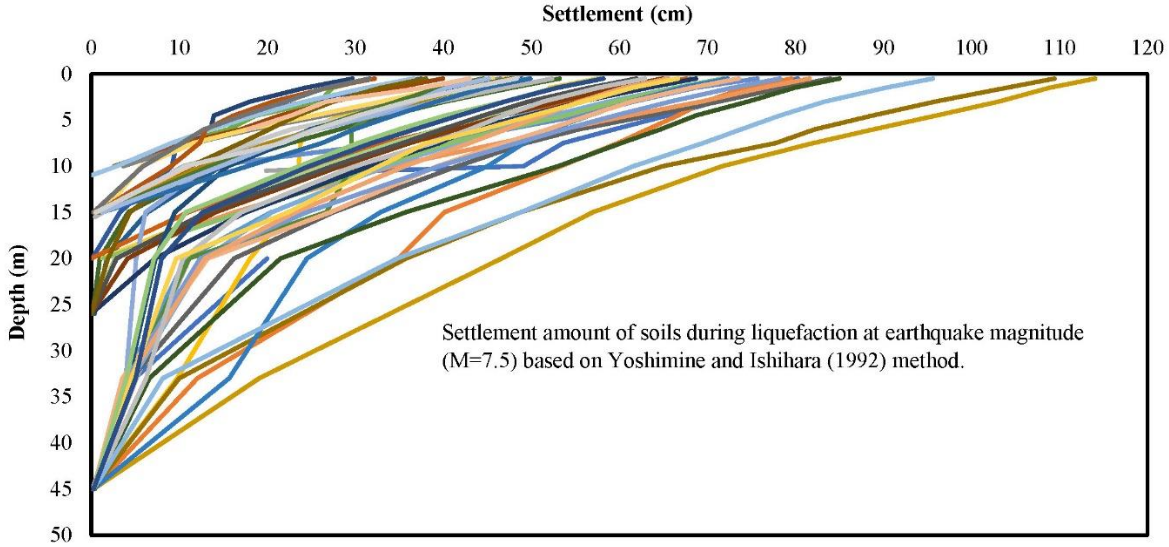

Fig. 13. Estimated settlement amount with the variation of depth at high earthquake magnitude (M=7.5).

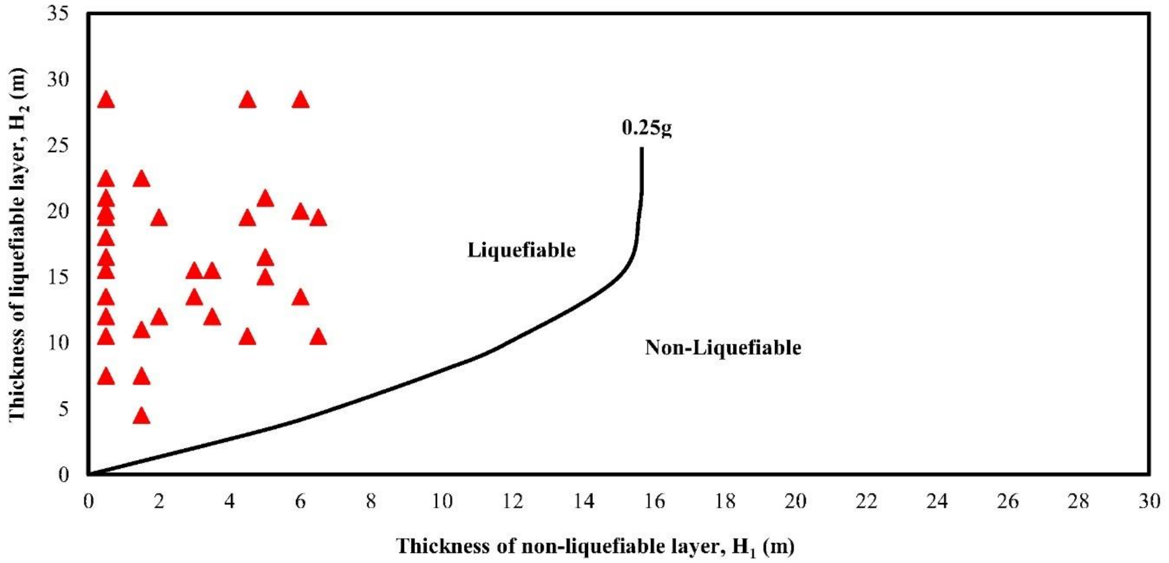

Fig. 14. Evolution of ground damage due to liquefaction at the investigated area by "Boundary Curves," according to Ishihara (1985).

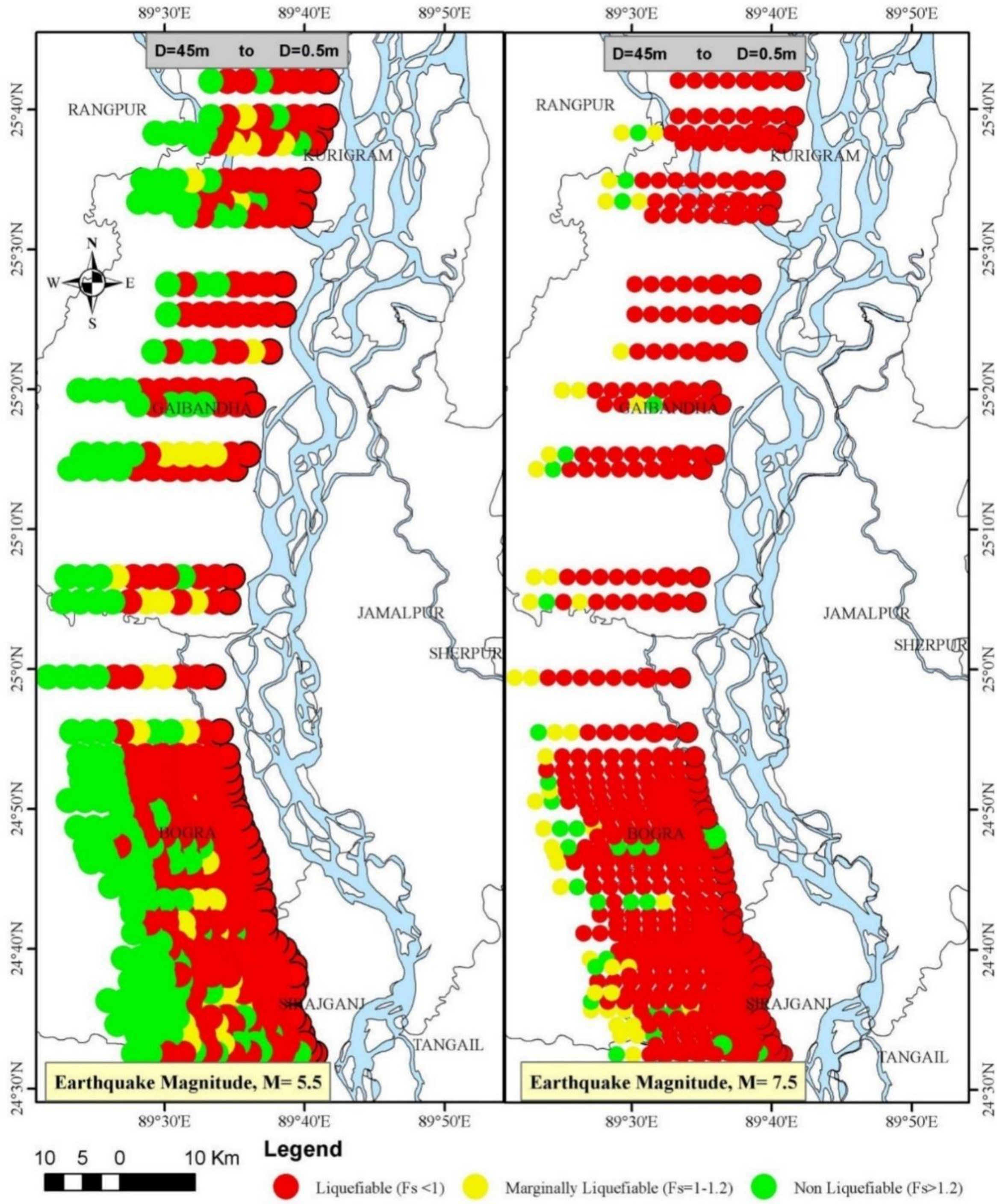

Fig. 15. Liquefaction potential hazards map of the study area at earthquake magnitude, M = 5.5 (left) and M = 7.5 (right).

VI. Conclusion

Based on the assessment, it can be established that the study areas are unconsidered for liquefaction potential hazards below earthquake magnitude 5.5. The soil of the study areas is only susceptible to liquefaction at earthquake magnitude greater than or equal to 5.5. For lower earthquake magnitude (M=5.5), liquefaction hazards occurred up to a depth of 0.0-10 m in the areas of Sirajgonj, Bogra, and Gaibandha. At higher earthquake magnitude (M=7.5), the whole investigated area is susceptible to liquefaction extended depth of 15 to 20 m. Extensive settlement occurred up to 10 to 20 m, and a little settlement was found in the vulnerable areas below this depth. Settlement amounts decreased with increasing depth and were consistent with ground damage because of the liquefiable soil layer. Unconsolidated, very loose to medium dense, uniformly graded soils with significant thickness and fine content, shallow groundwater table, ground manifestation, and settlement of soils are responsible for liquefaction. It is concluded that at greater depths (>15-20 m), soils are dense to very dense and considered non-liquefiable because of higher SPT values (> 30). Destruction of the engineering structure founded within this depth (< 20 m) is because of liquefaction potential. Liquefaction potential risk might reduce by using ground improvement techniques or methods. Speeding up drainage capacity can be done by prefabricated vertical drain (PVD) method and soil densification can be done by compaction. Soft ground stabilization can be done by grouting or deep mixing method (DMM). These research findings can be used as a general guideline to ground failure susceptibility, land use planning and earthquake-induced river erosional hazards mitigation. It is also significant for sustainable development and mitigation planning of the study area.

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References

[1]. Ansary MA, Akhter T. Seismic Loss Estimation for Ward 14 of Mymensingh, Bangladesh. J Geogr Nat Disasters. 2018;08(03).
[2]. British Standards I. BS 1377-9: 1990 Methods of test for Soils for civil engineering purposes — Part 9: In-situ tests. British Standard. 1990.
[3]. BSI. BS 5930:1999 Code of practice for site investigations. British Standard. 1999.
[4]. Cetin KO, Seed RB, Der Kiureghian A, Tokimatsu K, Harder LF, Kayen RE, et al. Standard penetration test-based probabilistic and deterministic assessment of seismic soil liquefaction potential. J Geotech Geoenvironmental Eng. 2004;130(12):1314–40.
[5]. Hossain ATMS, Haque SA. Liquefaction hazards on the western bank of Jamuna river, Bangladesh. J Sci Eng Res. 2017;4(7):334–41.
[6]. Huq MA, Whitney JW, Das SK AR. Geological map and report of the western part of Rajshahi District, Bangladesh. 1991.
[7[. Idriss IM. Liquefaction of soils during earthquakes. In: NATO Advanced Study Institutes Series, Series C: Mathematical and [8]. Physical Sciences. Washington, D.C: National Research Council, National Academy of press; 1982. p. 507–30.
[8]. Idriss IM, Boulanger RW. Semi-empirical procedures for evaluating liquefaction potential during earthquakes. Soil Dyn Earthq Eng. 2006;26(2–4):115–30.
[9]. Ishihara K. Stability of natural deposits during earthquakes. In: Proc 11th international conference on soil mechanics and foundation engineering, San Francisco, August 1985 Vol 1, (Balkema). 1985. p. 321–76.
[10]. Ishihara K, Yoshimine M. Evaluation of settlements in sand deposits following liquefaction during earthquakes. Soils Found. 1992;32(1):178–88.
[11]. Iwasaki T. Soil liquefaction studies in Japan: state-of-the-art. Soil Dyn Earthq Eng. 1986;5(1):2–68.
[12]. Khan FH, Brammer H, Reimann K-U. Geology of Bangladesh. Geogr J. 1995;161(1):89.
[13]. Mahabub MS, Hossain ATMS. Geomechanical aspects of some soils on the western bank of Jamuna River, Bangladesh. Bangladesh Geosci J. 2015;21:15–34.
[14]. Marcuson WF, Hynes ME, Franklin AG. Evaluation and Use of Residual Strength in Seismic Safety Analysis of Embankments. Earthq Spectra. 1990;6(3):529–72.
[15]. Martin G, Lew M. Recommended procedures for implementation of DMG Special Publication 117, Guidelines for analyzing and mitigating liquefaction hazards in California. In: University of Southern California [Internet]. 1999. p. 63. Available from: http://scholar.google.com/scholar?hl=en&btnG=Search&q=intitle:Recommended+Procedures+for+Implementation+of+DMG+Special+Publication+117+Guidelines+for+Analyzing+and+Mitigating+Liquefaction+Hazards+in+California#0
[16]. Robertson PK, Wride CE. Evaluating cyclic liquefaction potential using the cone penetration test. Can Geotech J. 1998;35(3):442–59.
[17] Seed HB. Soil liquefaction and cyclic mobility evaluation for level ground during earthquakes. ASCE J Geotech Eng Div. 1979;105(2):201–55.
[18]. Seed HB, Idriss IM. Simplified procedure for evaluating soil liquefaction potential. ASCE J Soil Mech Found Div. 1971;97(SM9):1249–73.
[29]. Seed HB, Idriss IM. Ground motion and soil liquefaction during earthquakes. Monograph Series. 1982.
[20]. Seed HB, Idriss IM, Arango I. Evaluation of liquefaction potential using field performance data. J Geotech Eng. 1983;109(3):458–82.
[21]. Seed HB, Tokimatsu K, Harder LF, Chung RM. Influence of SPT procedures in soil liquefaction resistance evaluations. J Geotech Eng. 1985;111(12):1425–45.
[22]. Tokimatsu K, Seed HB. Evaluation of settlements in sands due to earthquake shaking. J Geotech Eng. 1987;113(8):861–78.
[23]. Tsuchida H. Prediction and countermeasure against liquefaction in sand deposits. In: The Seminar of the Port and Harbour Research Institute. 1970. p. 1–33.
[24]. Youd TL, Idriss IM. Proceeding of the NCEER workshop on evaluation of liquefaction resistance of soils. Tech Rep NCEER-97-0022. 1997;
[25]. Youd TL, Idriss IM, Andrus RD, Arango I, Castro G, Christian JT, et al. Liquefaction resistance of soils: Summary report from the 1996 NCEER and 1998 NCEER/NSF workshops on evaluation of liquefaction resistance of soils. J Geotech Geoenvironmental Eng. 2001;127(10):817–33.
[26]. LiquefyPro, Version 5. Liquefaction and Settlement Analysis Software Manual. USA: CivilTech Software; 2011.
[27]. Bangladesh National Building Code. In Housing and Building Research Institute, Bangladesh Standards and Testing Institutions; 2015.