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Full Length Research Paper

Soil liquefaction induced settlements with interaction of earthquake hazard analysis

Zeynep Tugce YUKSEL[1], Ferhat OZCEP[2]*, Dilek KEPEKCI[3] and Tazegul OZCEP[4]

[1]Department of Civil Engineering, Institute of Science and Technology, Yildiz Technical University, Istanbul.
[2]Department of Geophysical Engineering, Faculty of Engineering, Istanbul University, Istanbul.
[3]Earthquake Monitoring Center, Kandilli Observatory and Earthquake Research Institute, Bogazici University, Cengelkoy, Istanbul.
[4]Ministry of National Education, Sirinevler Primary School, Sirinevler Istanbul, Turkey.

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The Marmaray Rail Tube Tunnel and Commuter Rail Mass Transit System Project provides an upgrading of the commuter rail system in Istanbul connecting Halkali on the European side with Gebze on the Asian side with an uninterrupted, modern, high-capacity commuter rail system. Railway tracks in both sides of Istanbul Strait will be connected to each other through a railway tunnel connection under the Istanbul Strait. The line goes underground at Yedikule, continues through the Yenikapi and Sirkeci new underground stations, passes under the Istanbul Strait, connects to the Üsküdar new underground station and emerges at Sögütlüçesme. This project is one of the major transportation infrastructure projects in the world at present. The entire upgraded and new railway system will be approximately 76 km long. In this study, by using CPT data and acceleration and magnitude data (obtained seismic hazard analysis of Marmara Region), settlement analysis were carried out for Marmaray Project. As it is known, liquefaction is a soil behavior of saturated sandy soils under the earthquake/dynamic effects. In the first phase of the study, 'cyclic stress ratio approach' was applied to all data to analysis of soil liquefaction. In the second phase of the study, by using Isihara and Yoshimine (1992) approach, possible soil settlements for several design earthquakes (for several acceleration and magnitude values) were estimated.

Key words: Liquefaction, induced settlements, marmaray project.

INTRODUCTION

As it is known by many studies (Borcherdt and Gibbs, 1976; Iglesias, 1988; Gazetas et al., 1990; Seed et al., 1991; Lekkas, 1996; Ozel et al., 2002), soil/site conditions play an effective role as one of cause of induced earthquake damage. The estimation of site conditions requires identification of the soil stratification and mechanical properties of soil layers based on detailed geotechnical and geophysical tests. Soil liquefaction resistance is estimated by *in situ* test or laboratory test. Standard penetration, cone penetration and shear wave tests are the most used for the estimation of liquefaction susceptibility. Methods based on the SPT (standard penetration test) were developed by Seed and Idriss (1971), Seed et al. (2001), Tokimatsu and Yoshimi (1983) and Youd and Idriss (1997). Methods by using the CPT (conic penetration test) include those developed by Seed and Alba (1986). Methods by using the shear waves developed by Stokoe et al. (1988), Andrus and Stokoe (1996, 1997, 1999) and Dobry et al. (1981). State of art of liquefaction analysis is evaluated by Youd et al. (2001). In the last phase of the liquefaction analysis, approach, total soil settlements were estimated by using Isihara and Yoshimine (1992). There are several examples on the analysis of soil liquefaction in Turkey (Cetin et al., 2001; Yilmaz and Yavuzer, 2005; Ozcep and Zarif, 2009; Tosun et al., 2011). The marmaray (rail tube tunnel and commuter rail mass transit system) project is the commuter rail system in Istanbul, connecting on the

*Corresponding author. E-mail: ferozcep@istanbul.edu.tr.

Table 1. Technical properties for Marmaray (rail tube tunnel and commuter rail mass transit system) project.

Total length: 76.3 km
European side: 19.3 km
Asian side: 43.4 km
Immersed tube tunnel: 1.4 km
Bored tunnel: 9.8 km
Cut-and-cover and open cut: 2.4 km
Maximum depth of immersed tube tunnel: 56 m
Number of stations:
Existing stations to be upgraded/rebuilt: 37
New underground stations: 3
Length of platform, minimum: 225 m
Type of platform: Centre platforms
Max. peak capacity per hour per direction:
Existing commuter rail: 10.000 passengers
Upgraded commuter rail: 75.000 passengers
Design speed: 100 km/h
Maximum operational speed: 100 km/h
Expected mean speed: 45 km/hour
Headway (time between trains): 120 - 600 s
Number of new vehicles: Up to 440
Travel time total between Gebze and Halkali:
Existing commuter rail and ferries/taxi to or from boat: (railway - ferry - railway) 185 min
New and upgraded uninterrupted commuter rail: 104 min

European side with the Asian side. Railway tracks in both sides of Istanbul Strait will be connected to each other through a railway tunnel connection under the Istanbul Strait.

Main goal of this work is to estimate the liquefaction induced settlements for Marmaray Project. Technical properties for Marmaray Project are given in Table 1.

EARTHQUAKE HAZARD ANALYSIS OF STUDY AREA

Seismic hazard analysis is the computation of probabilities of occurrence per unit time of certain levels of ground shaking caused by earthquakes. This analysis is often summarized with a seismic hazard curve which shows annual probability of exceedence versus ground motion amplitude. Deterministic and probabilistic seismic hazard analysis was used to evaluate the seismic hazard of region. The Marmara Sea is an intra-continental marine basin between the Aegean and Black seas. It is in a tectonically very active region located on the North Anatolian Fault (NAF) zone (Şengör, 1979; Barka, 1992; Straub et al., 1997; Le Pichon et al., 2001; Şengör et al., 2004). The NAF is a major transform-plate boundary that has produced devastating historical earthquakes along its 1600 km length (Ambraseys and Finkel, 1995; Soysal et al., 1981). Potential earthquake source area for project area was considered the North Anatolian Fault in Marmara Sea (Figure 1b).

Deterministic seismic hazard analysis

Required input for deterministic hazard analysis is a designation of active faults or earthquake sources in the region. For the Marmara Region, it was assumed tree model (A, B and C) for seismic hazard (Erdik, 2003). Model A: approximately 120 km rupture length; Model B: approximately 109 km rupture length; Model C: approximately 174 km rupture length. For these models, magnitudes of design earthquake were estimated (Table 2a and b).

Probabilistic seismic hazard analysis of region

In Table 3a, earthquakes were given in our area within 100 km radius. Gutenberg-Richter recurrence relationships were determined as:

$$\text{Log (N)} = 2.81 - 0.69\ M \tag{1}$$

Earthquake occurrence probability were given in Table 2c by using:

Table 2a. Equations for rupture length and magnitude estimations.

Researcher	M (magnitude)	Magnitude type
Abraseys and Zatopek (1968)	M = (0.881 LOG (L)) + 5.62	Ms
Douglas and Ryall (1975)	M= (LOG (L) + 4.673)/0.9	Ms
Patwardan et al. (1980)	M = (LOG (L) 1.1) + 5.13	Ms
Toksöz et al. (1979)	M = (LOG (L)+3.62)/0.78	Ms
Wells and Coppersmith (1994)	M = 5.16 + (1.12 LOG (L))	Mw

Table 2b. Model A: approximately 120 km rupture length; Model B: approximately 109 km rupture length; Model C: approximately 174 km rupture length and magnitude estimations for these models.

Researchers	M (magnitude) ranges for A Model	M (magnitude) ranges for B Model	M (magnitude) ranges for C Model
Abraseys and Zatopek (1969)	7.4	7.4	7.6
Douglas and Ryall (1975)	7.5	7.5	7.7
Patwardan et al. (1980)	7.4	7.4	7.6
Toksöz et al. (1979)	7.3	7.2	7.5
Wells and Coppersmith (1994)	7.5	7.4	7.7

$$Rm = 1- e^{-(N(M) \,.\, D)} \tag{2}$$

Where Rm = risk value (%); D, duration; N (M) for M magnitude of Equation 1 value.

Attenuation relationship was defined by two attenuation models. From a set of attenuation relationships, the design acceleration values of the project area was estimated as 0.51 and 0.55 g for Campbell (1997) model with exceeding probability of 20% in 50 years for 20 km epicentral distance. Finally, a hazard curve for region was estimated. Table 3b shows earthquake occurrence probability for region. Table 3c shows also estimated accelerations for the region by using several attenuation relationships.

METHODOLOGY OF LIQUEFACTION ANALYSIS AND LIQUEFACTION INDUCED SETTLEMENTS

The horizontal earthquake force *F* acting on the soil column (which has a unit width and length) is (Day, 2002):

$$F = ma = (W/g)\, a = (\gamma_1 z/g)\, a_{max} = \sigma_{v0}\, (a_{max}/g) \tag{3}$$

Where:

F = horizontal earthquake force acting on soil column that has a unit width and length, lb or kN.
m = total mass of soil column, lb or kg which is equal to *W*/*g*.
W = total weight of soil column, lb or kN. For the assumed unit width and length of soil column, the total weight of the soil column is $\gamma_t z$.
Z = depth below ground surface of soil column as shown in Figure 1a.
a = acceleration, which in this case is the maximum horizontal acceleration at ground surface caused by the earthquake (*a* = *a*max), ft/s^2 or m/s^2.
X = total vertical stress at bottom of soil column, lb/ft^2 or kPa. The total vertical stress $\gamma_t z$.

As shown in Figure 1a, by summing forces in the horizontal direction, the force *F* acting on the rigid soil element is equal to the maximum shear force at the base on the soil element. Since the soil element is assumed to have a unit base width and length, the maximum shear force *F* is equal to the maximum shear stres τ max or from Equation 1:

$$\tau_{max} = F = \sigma_{v0}\, (a_{max}/g) \tag{4}$$

Dividing both sides of the equation by the vertical effective stress σ_{v0}' gives:

$$(\tau_{max}/\sigma_{v0}) = (\sigma_{v0}/\sigma'_{v0})\, (a_{max}/g) \tag{5}$$

Since the soil column does not act as a rigid body during the earthquake, but rather the soil is deformable; Seed and Idriss (1971) incorporated a depth reduction factor *rd* into the right side of Equation 3 or:

$$(\tau_{max}/_{v0}) = rd\, (\sigma_{v0}/'_{v0})\, (a_{max}/g) \tag{6}$$

For the simplified method, Seed et al. (1975) converted the typical irregular earthquake record to an equivalent series of uniform stress cycles by assuming the following:

$$\tau_{cyc} = 0.65\, \tau_{max} \tag{7}$$

Table 3a. Earthquakes in the study area about 100 km radius.

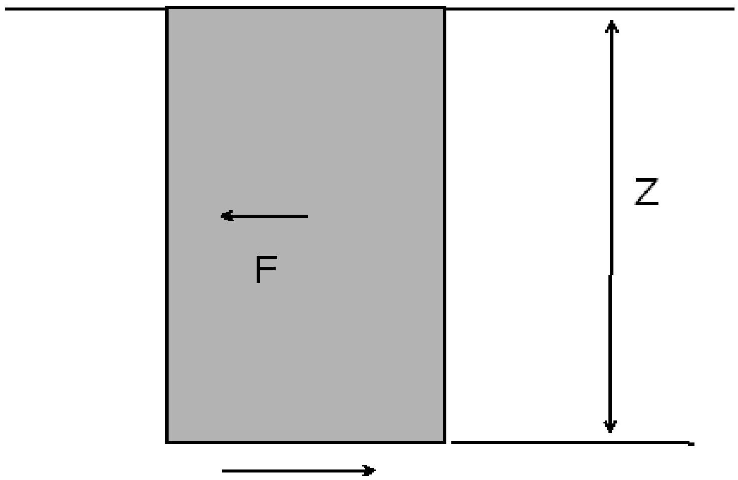
Magnitudes	4.5 ≤ M <5.0	5.0 ≤ M < 5.5	5.5 ≤ M <6.0	6.0 ≤ M <6.5	7.0 ≤ M <7.5
Numbers	21	12	7	1	1

Table 3b. Earthquake occurrence probability for region.

Magnitude	For D = 10 (years); probability (%)	For D = 50 (years); probability (%)	For D = 75 (years); probability (%)	For D = 100 (years); probability (%)
5	90.3	100.0	100.0	100.0
5.5	65.1	99.5	100.0	100.0
6	37.8	90.7	97.2	99.1
6.5	19.3	65.8	80.0	88.3
7	9.2	38.4	51.7	62.1
7.5	3.7	17.0	24.4	31.2

Table 3c. Estimated accelerations for the region.

M (magnitude)	Δ. Epicentral Distance Uzaklık (km)	H. focal depth (km)	Esteva (1970)	Davenport (1972)	Donovan (1973a)	Esteva and Villaverde (1973)	Donavan(1973b)	Donavan (1973c)	McGuier (1974)	Orphal and Lahoud (1974)	Shah et al. (1973)	Oliviera (1974)	Katayama	Esteva et al. (1978)	Joyner and Boore (1981)	Campbell (1981a)	Campbell (1981b)	Newmark and Roseblueth (1971)	Kanai (1966)	Esteva and Roseblueth (1964)	Fukishima et al. (1988)	Abrahamson and Litehiser (1989)	Campbel (1997)	Average
7.5	20	15	0.20	0.58	0.39	0.54	0.25	0.27	0.36	1.06	0.49	0.20	0.36	0.37	0.53	0.21	0.21	0.35	0.46	0.41	0.32	0.30	0.51	0.40
7.5	25	15	0.17	0.45	0.34	0.48	0.23	0.24	0.33	0.87	0.43	0.17	0.32	0.26	0.42	0.19	0.18	0.28	0.38	0.34	0.29	0.27	0.46	0.34
7.6	20	15	0.22	0.62	0.41	0.59	0.27	0.28	0.38	1.17	0.53	0.22	0.39	0.40	0.56	0.22	0.22	0.38	0.52	0.44	0.33	0.31	0.55	0.43
7.6	25	15	0.19	0.48	0.36	0.52	0.24	0.25	0.35	0.96	0.47	0.19	0.35	0.28	0.45	0.20	0.20	0.30	0.44	0.37	0.30	0.28	0.50	0.36



$$\tau_{max} = F = \sigma_{v0}\left(\frac{a_{max}}{g}\right)$$

Figure 1a. Conditions assumed for the derivation of the CSR earthquake equation (Day, 2002).

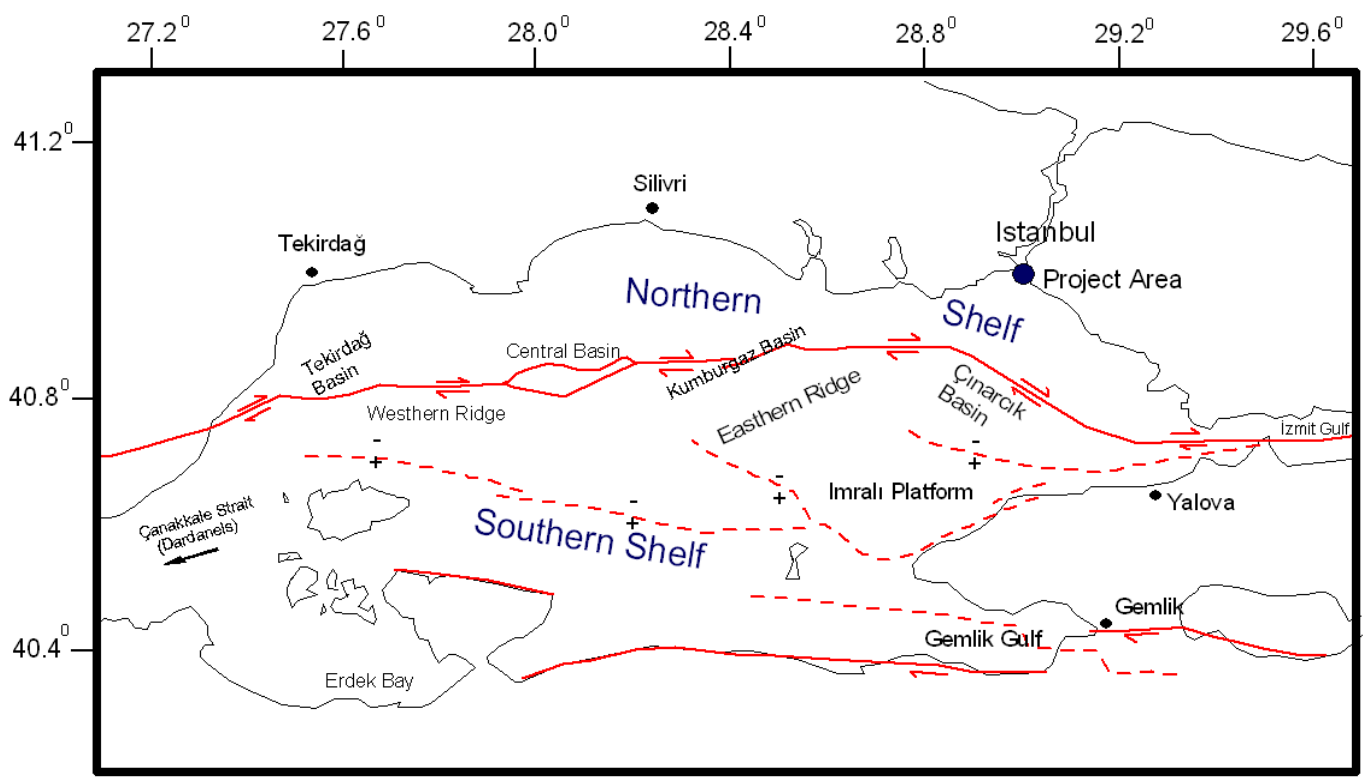


Figure 1b. Fault systems around project area (Le Pichon et al., 2001; Şengör et al., 2004).

Where τ cyc = uniform cyclic shear stress amplitude of the earthquake (lb/ft^2 or kPa).

In essence, the erratic earthquake motion was converted to an equivalent series of uniform cycles of shear stress referred to as τ cyc. By substituting Equations 7 into 6, the earthquake-induced cyclic stress ratio is obtained.

$$CSR = (\tau_{cyc}/\sigma_{v0}) = 0.65\ rd\ (\sigma_{v0}/\sigma'_{v0})\ (a_{max}/g) \qquad (8)$$

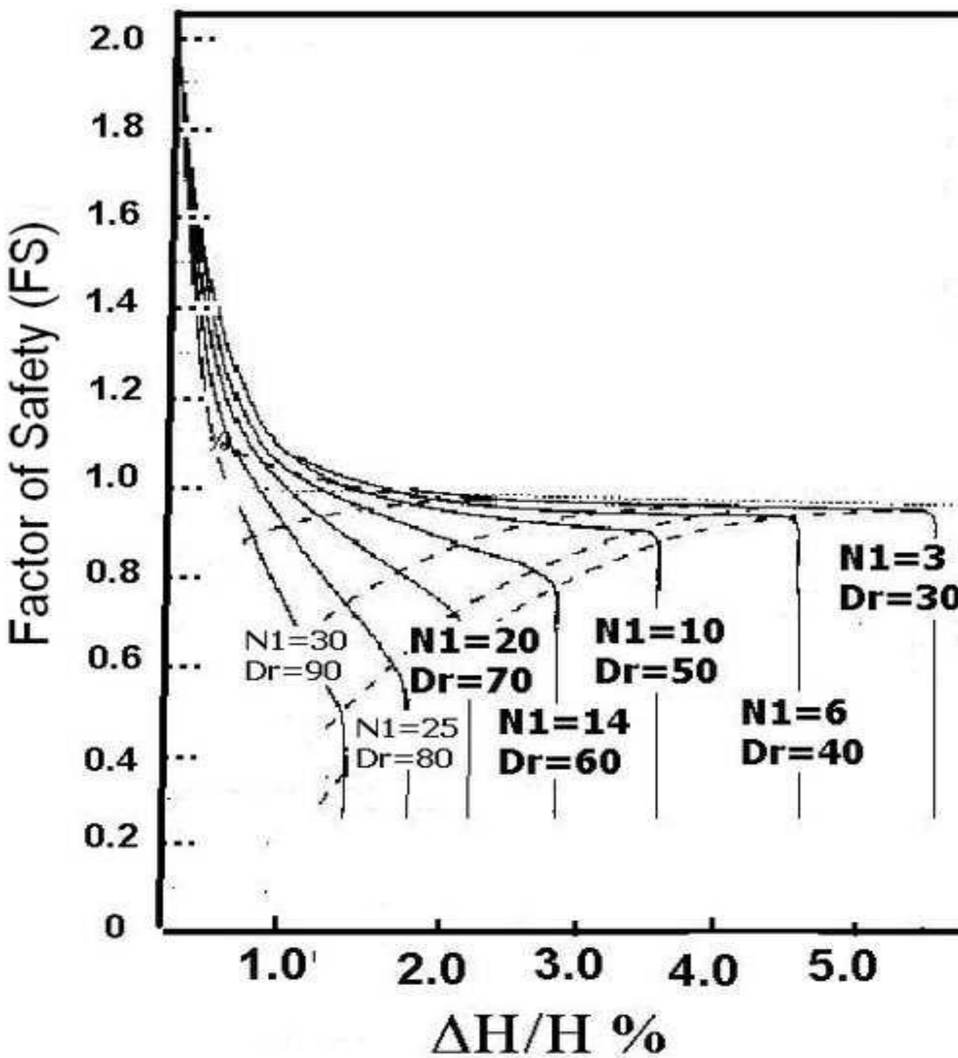


Figure 2. Relation of safety factor (FS) of liquefaction and volumetric strain (ΔH/H) with standard penatration test (N1) and relative density (Dr) (Ishihara and Yoshimine, 1992).

Where CSR = cyclic stress ratio (dimensionless), also commonly referred to as seismic stress ratio.

The most widely used simplified SPT-N method is that proposed by Seed et al. (1983). This method calculates the earthquake-induced cyclic stress ratio in a soil layer via the simplified equation as follows:

CSR (cyclic stress ratio) = 0.65 (Amax/g) ($\sigma o/\sigma o'$) rd (z)/MSF (M) (9)

where $\sigma o'$ and σo are the effective and total vertical overburden pressures at some specified depth; Amax is the peak horizontal ground acceleration; rd (z) is the stress reduction factor at depth z, MSF (M) is a magnitude scaling factor that considers the duration effect of different earthquake magnitudes.

In Equation 1, $\sigma o'$ and σo are directly computed from boring log and laboratory test data and can therefore be regarded as deterministic values with no variance; the rd (z) and MSF (M) vary with the depth and the earthquake magnitude. The safety factor to liquefaction can be calculated by the simple equation as follows:

FS = CSR/CRR (10)

Criteria for evaluation liquefaction resistance based on SPT, CPT or Shear wave data are largely embodied in the CRR versus N1,60 plot (Youd et al., 2001). This procedure is based on the relationship of SPT *N*- values, corrected for both effective overburden stress and energy, equipment and procedural factors affecting SPT testing (to N1,60-values) versus intensity of cyclic loading, expressed as magnitude-weighted equivalent uniform cyclic stress ratio (CSReq). The correlation between corrected N1,60-values and the intensity of cycling required to trigger liquefaction is also a function of fines content (Seed et al., 2001). In the last phase of the liquefaction analysis, by using Isihara (1992) approach, total soil settlements were estimated by:

$S = \Sigma H \varepsilon$ (11)

Where H = thickness of layer; ε = volumetric strain of layer. The volumetric strain, ΔH/H (that is ε) are estimated by using Ishihara and Yoshimine (1992) as shown in Figure 2. Relation between

Table 4. Relation between damage extent and approximate settlement (Ishihara, 1996).

Extent of damage	Settlements (cm)	Phenomena on the ground surface
Light to no damage	0-10	Minor cracks
Moderate damage	10-30	Small cracks, oozing of sand
Extensive damage	30-70	Large cracks, spoting of sands, large offsets and lateral movements.

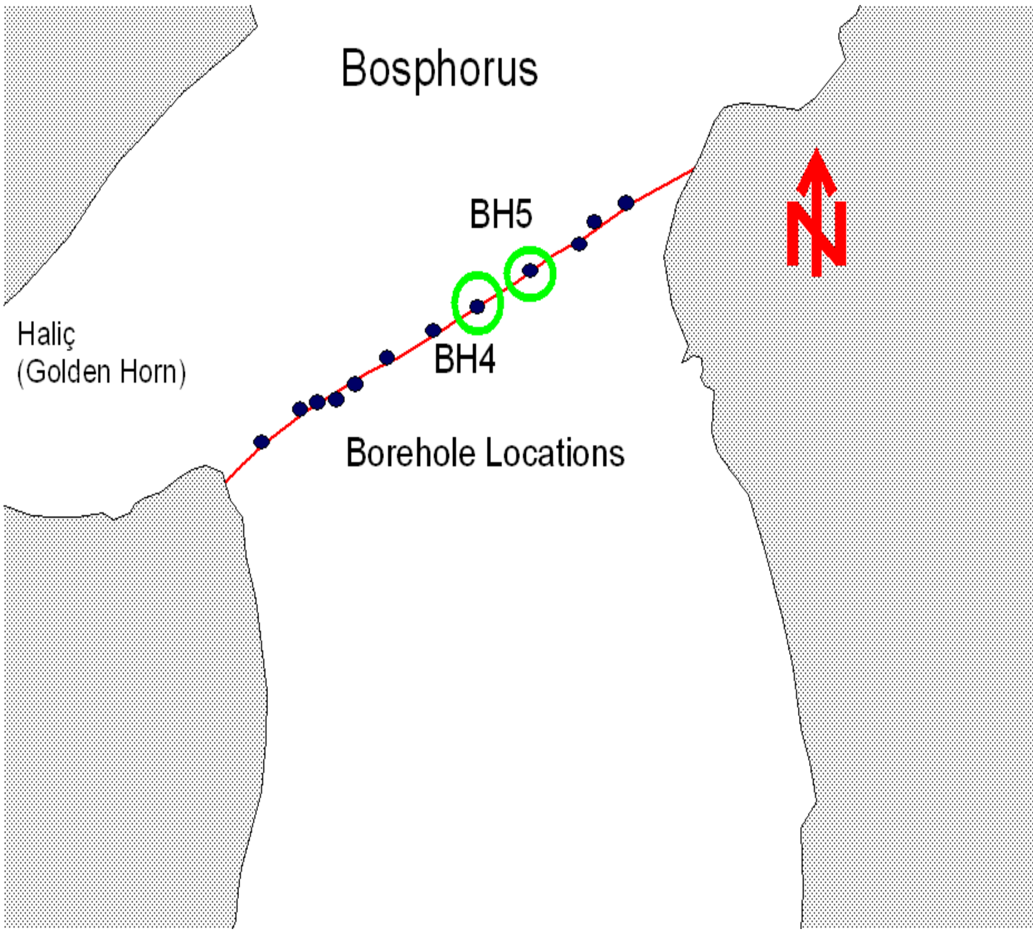


Figure 3. Location of boreholes (BH4 and BH5) in study area.

damage extent and approximate settlement is shown in Table 4 (Ishihara, 1996).

LIQUEFACTION ANALYSIS AND INDUCED SETTLEMENTS FOR REGION

In this study, a practical reliability-based method is developed for assessing the soil liquefaction potential of Marmaray Project (Turkey). The approach, based on conventional theory, enables the earthquake-induced cyclic stress ratio (CSR) and soil cyclic resistance ratio (CRR) on the basis of the simplified SPT-N method proposed by Seed et al. (1983). In this study, borehole and geotechnical data was obtained by DLH regional directorate. Location of boreholes (BH4 and BH5) are shown in Figure 3. Liquefaction analysis is carried out to depth of 25 m. Figure 3 shows detailed location plan of boreholes for Marmaray Project. Figure 4a shows straigraphic column of BH4 and BH5 boreholes. In BH4 borehole, there is shell debris sandy soils between 0.0 and 7.0 m, fine to medium sands between 7.0 and 15.5 m, and sandy units between 15.5 and 25.0 m. In BH5 borehole, there are shell debris sandy soils between 0.0 and 13.40 m, fine to coarse sands between 13.4 and 21.0 m, and fine to medium sands between 21.0 and 25.0 m. In the literature, there have been attempts to correlate CPT data for sands with results of ‘standard penetration tests’. A typical correlation (Robertson et al., 1983) is ratio of (CPT q_c) (SPT N) is a function of D50 particle size of the soil (Figure 4d). Obtained CPT data form BH4 and BH5 boreholes (Figures 4b and c) are transformed to SPT (N) values by Seed ve diğ (1983) and Robertson et al. (1983) by using:

$$N60 = qn/4.5 \quad (12)$$

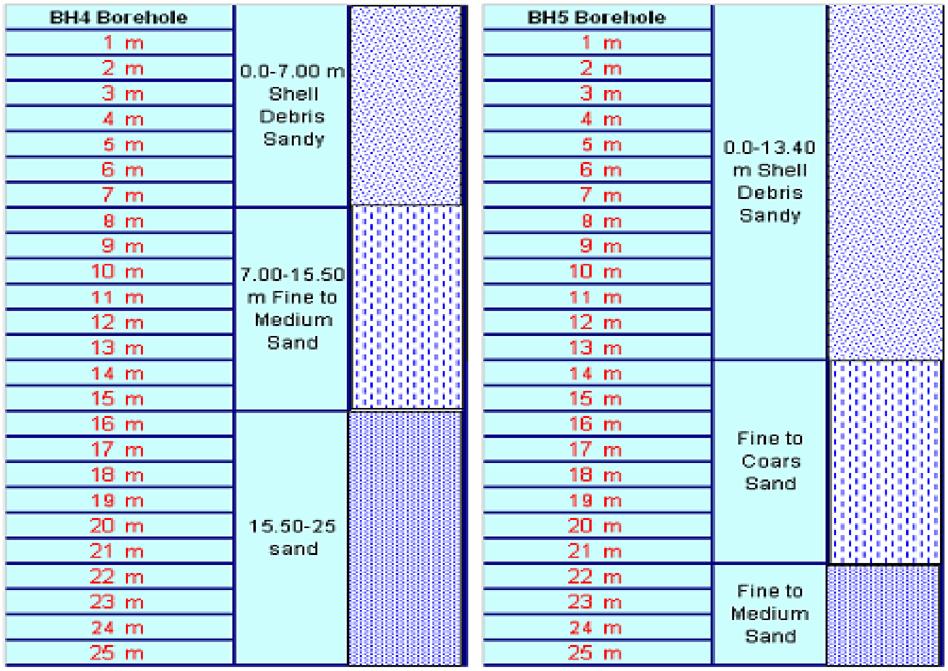


Figure 4a. Straigraphic column of BH4 and BH5 boreholes for marmaray project.

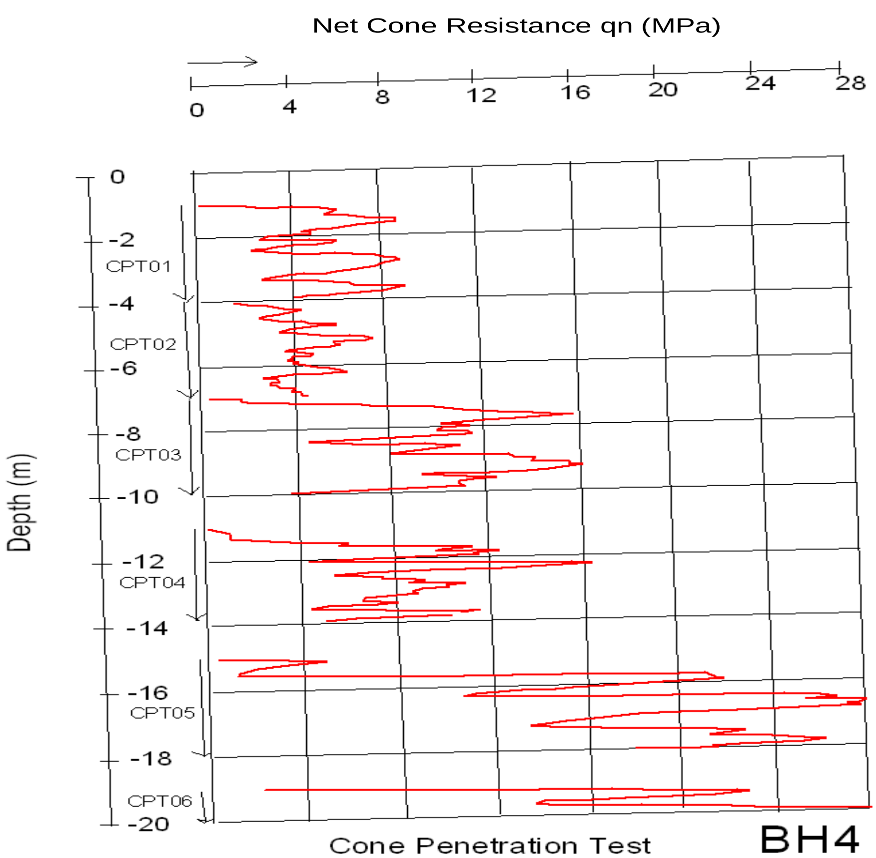


Figure 4b. Net cone resistance (qn) obtained by cone penetratio test in BH4 borehole for marmaray project (data provided from DLH, Demiryollari, Hava Meydanlari Insaati Genel Mudurluğu).

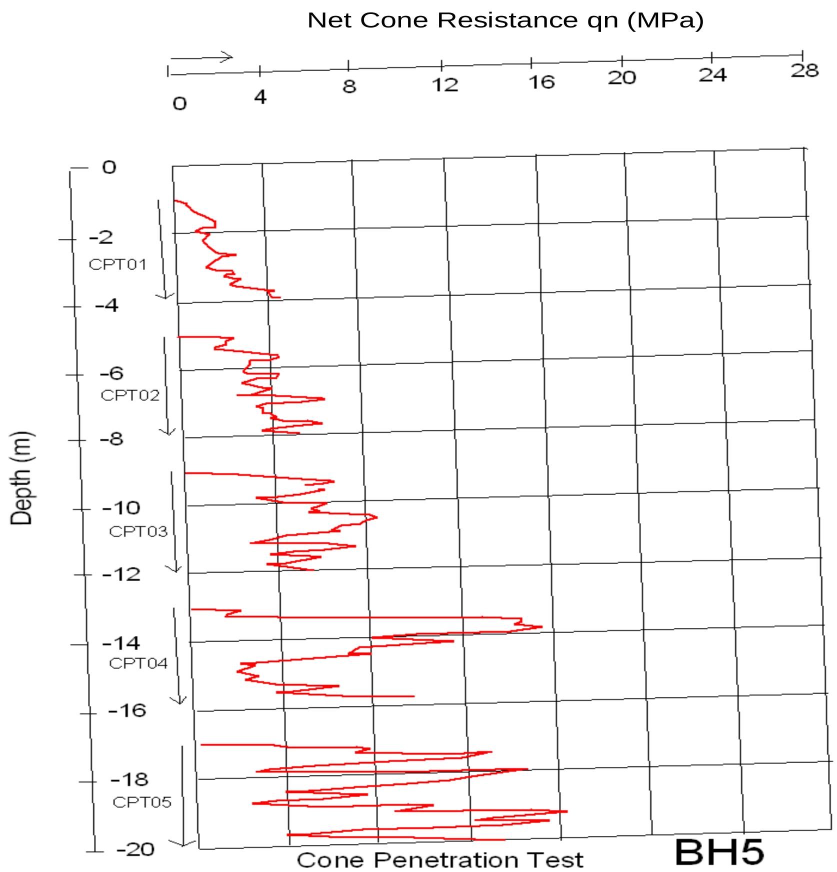


Figure 4c. Net cone resistance (qn) obtained by cone penetration test in BH5 borehole for Marmaray Project (data provided from DLH, Demiryollari, Hava Meydanlari Insaati Genel Mudurlugu).

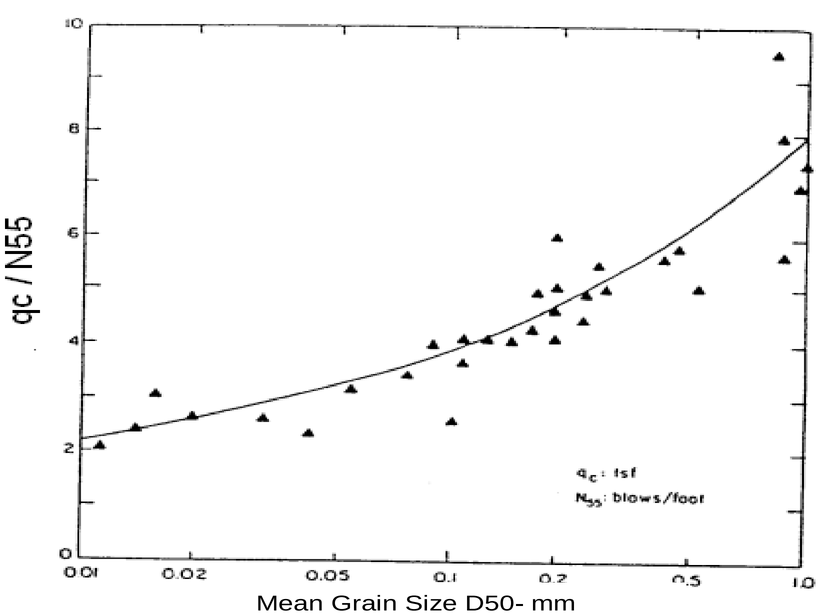


Figure 4d. Ratio of (CPT q_c) (SPT N) as a function of D50 particle size of the soil (Robertson et al., 1983).

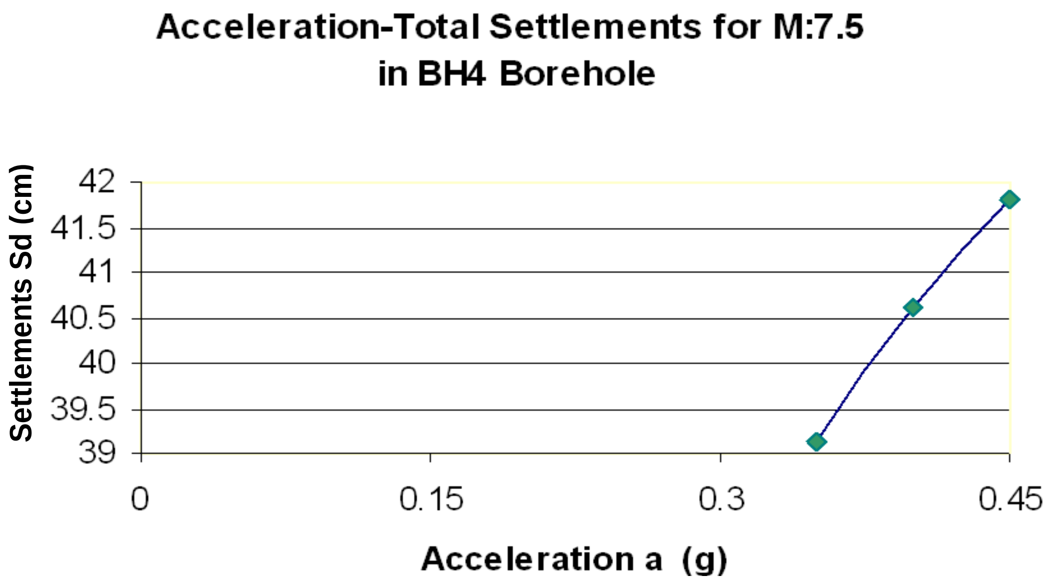


Figure 5a. Variation acceleration with total settlement for selected design earthquake (0.35, 0.40 and 0.45 g for accelerations and 7.5 magnitude) in BH4 borehole.

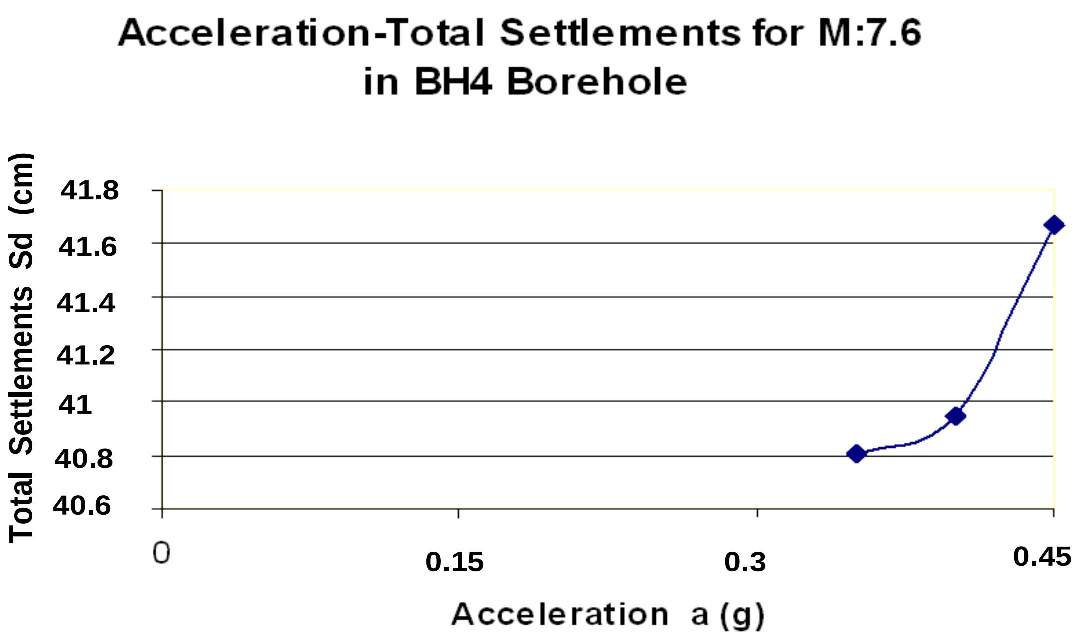


Figure 5b. Variation acceleration with total settlement for selected design earthquake (0.35, 0.40 and 0.45 g for accelerations, and 7.6 magnitude) in BH4 borehole.

Where N60 is corrected SPT (N) value for hammer energy ratio (60%).

For liquefaction analysis, magnitudes and accelerations of design earthquake were selected as 0.35, 0.40 and 0.45 g for accelerations, and 7.5 and 7.6 for magnitudes. Liquefaction induced settlements (Isihara and Yoshimine, 1992) depending on selected design earthquake (0.35, 0.40 and 0.45 g for accelerations, and 7.5 and 7.6 for magnitudes) are shown in Figures 5a, b, c and d.

RESULTS AND DISCUSSION

Firstly, the study focused on the analysis of soil liquefaction by cyclic stress ratio approach. In the second phase of the study, possible soil settlements for several design earthquakes (for several acceleration and magnitude values) were estimated. by using Isihara and

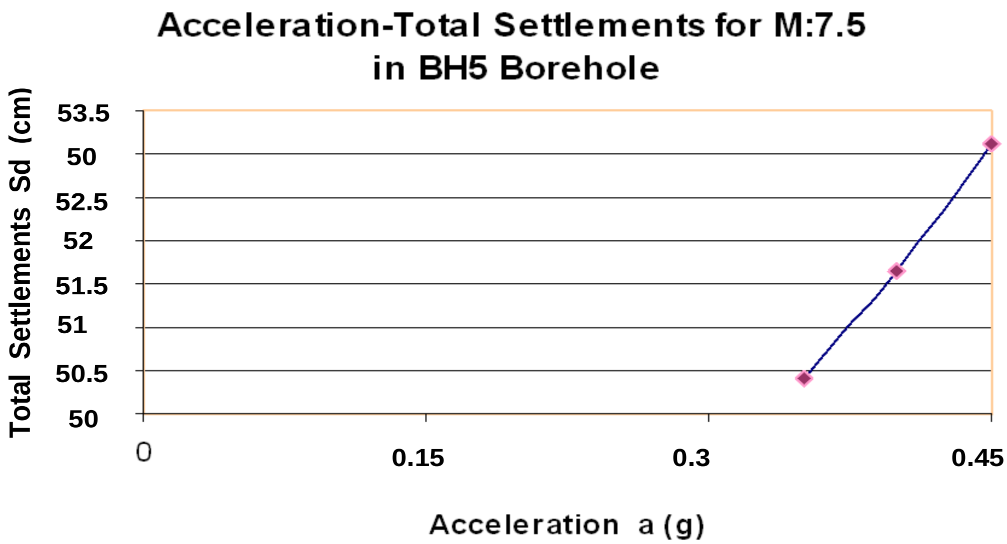


Figure 5c. Variation acceleration with total settlement for selected design earthquake (0.35, 0.40 and 0.45 g for accelerations and 7.5 magnitude) in BH5 borehole.

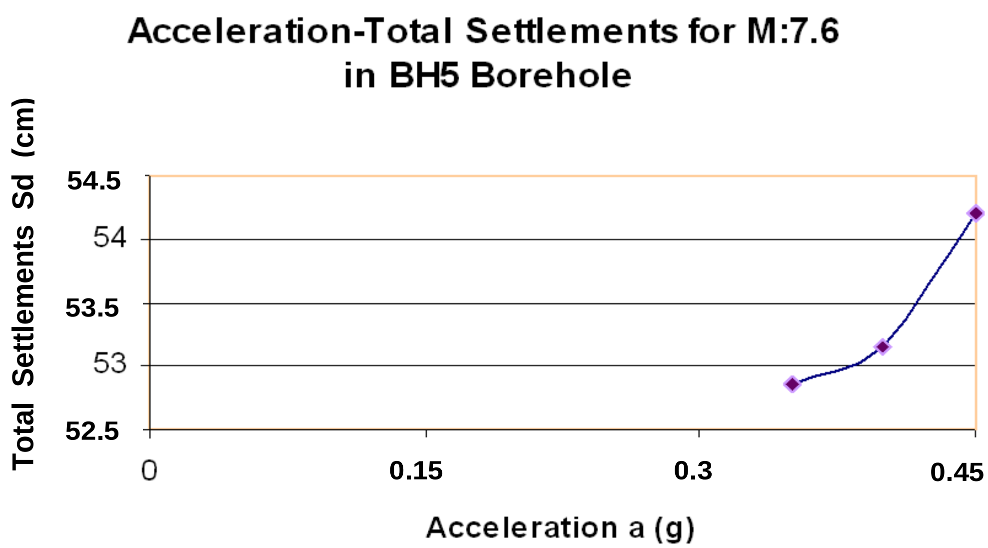


Figure 5d. Variation acceleration with total settlement for selected design earthquake (0.35, 0.40 and 0.45 g for accelerations, and 7.6 magnitude) in BH5 borehole.

Yoshimine (1992) approach. In this study (shown in Figure 5a, b, c and d), liquefaction induced settlements depending on selected design earthquake (0.35, 0.40 and 0.45 g for accelerations, and 7.5 and 7.6 for magnitudes) are estimated between 40 and 55 cm. When we looked, the liquefaction analysis and possible settlements with increased earthquake accelerations, the relation between damage extent and approximate settlement is “extensive damage” according to Ishihara (1996) damage classification. An immersed tunnel (IMT) is made up of a number of elements produced in a dry-dock or a shipyard. Immersed tunnel will be installed in 16 m from sea bottom. For this reason, possible liquefaction induced effects is efficient in deeper parts than this depth (that is 16 m). For these deeper parts, soil improvement process is required. With interaction of earthquake hazard analysis, a cyclic stress ratio-based soil liquefaction analysis is presented in the study.

The results of analysis show that shallow part of study areas could be suffered by the lateral displacements with the possible earthquakes at the ground surface. Another important result obtained during this study is that, the magnitude and acceleration of possible earthquake for triggering the soil liquefaction is one of the most important parameters.

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