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Full Length Research paper

Assessment of liquefaction susceptibility of Adapazari City after 17th August, 1999 Marmara earthquake

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The August 17, 1999 Marmara (Turkey) earthquake of magnitude (M_w = 7.4) struck the Marmara and Sakarya provinces in north-western part of Turkey. The earthquake caused substantial casualties and severe damages to structures. Adapazari in Sakarya province clearly suffered the worst damage due to geotechnical effects since the city is located over young riverbed sediments with soft and liquefiable silts and sands. Structures located on the surface of the liquefiable soils severely settled or tilted owing to the loss of bearing capacity of the soil. In this study, an assessment of liquefaction potential in Adapazari city during 17 August, 1999 Marmara earthquake is investigated based on Standard Penetration Test (SPT) measurements available. The evaluation of liquefaction potential has been analyzed by four well-known methods namely the Simplified Procedure, Tokimatsu-Yoshimi, Seed-De Alba method and the Japan Road Association. A computer program developed originally by the writers which achieves the computations for liquefaction susceptibility. The results of assessment based on the Simplified Procedure were generally consistent with the actual performance of the ground investigation.

Key words: Liquefaction potential, Marmara earthquake, liquefaction, standard penetration test.

INTRODUCTION

Earthquakes have great influences on human life due to their destructive damage on social and economic structures. After the earthquake, important parts of damages occur due to the dynamic behavior of a ground. Liquefaction may cause extensive damages on structures founded in or on the ground. Structures located on the surface of the liquefiable soil may severely settle or tilt due to the loss of bearing capacity of the soil. Lifeline structures buried in the liquefiable soil may be uplifted to the surface.

The Marmara earthquake of August 17th, 1999 was the most recent destructive one in Turkey. At 03:02 am local time (01:02 am GMT) on Tuesday, a very strong earthquake of moment magnitude of M_W = 7.4, with its epicenter at 40.702N, 29.987E and depth of 17 km, occurred. It was associated with faulting over a length of approximately 100 - 120 km. As a result, urban facilities in north-eastern part of the Marmara region experienced serious damage. Among several cities affected, Adapazari clearly suffered the worst damage due to the geotechnical effects and site response. Since, the city is located over young riverbed sediments with soft and liquefiable silts and sands. Thousands of structures located on the surface of the liquefiable soil severely settled, tilted or overturned due to the loss of bearing capacity of the soil or liquefaction. Lifeline structures buried in the liquefiable soil were damaged extensively in Adapazari. Sand boils also were observed at several locations. After the earthquake collapsed buildings, ignited fires in houses and in a petrol refinery which is the biggest one in Turkey, wrecked motorways, railways and bridges, and landslides were observed. According to the Turkish government data, in Adapazari, around 29752 buildings were either severely damaged or collapsed (30% of the building stock).

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In Adapazari, 3891 deaths and 5180 injuries were reported officially (Governor of Sakarya, 2000).

Most of the reinforced concrete buildings in Adapazari of 4 - 6 stories have been constructed close together. Foundations of those building are raft foundations. Their depth usually varies between 0.5 - 1.2 m. During the Marmara earthquake, many buildings settled and tilted especially in Tigcilar, Orta and Yenicami Districts (Figure 1).

Beyen and Erdik (2004) used two-dimensional modeling to determine the effects of local site conditions in the Adapazari plain crossing the severely damaged central part of Sakarya during the 17th August earthquake. Also, authors made attempts to provide greater insight into the local site response phenomena through the use of a two-dimensional nonlinear analysis. It was found that the simulated site responses were in global agreement with the recorded data. Eventually, site responses were larger in the alluvial basin for all aftershocks except the magnitude 5.8 event.

The liquefaction resistance of soils was evaluated with the empirical methods (Chu et al., 2004). Also, the performance functions obtained through artificial neural network modeling was used to compare with the results. Prediction on the cases of nonliquefaction with the SPT-based performance function was better than other empirical methods. It was also found that impacts of fines content (FC) on liquefaction resistance were reflective. For soils with fines content larger than 35%, the empirical boundary was suggested.

Several published equations specifying the cut-off distance for use in probabilistic seismic hazard assessment of liquefaction initiation were reviewed and new criteria based on minimum level of input wave energy were proposed by Trifunac and Todorovska, 2004. It was concluded that graphically formulated definitions of the maximum distance versus earthquake magnitude were too rough for seismic hazard analyses at sites with N (corrected standard penetration test value in blows per foot) less than about 10. Also, they proposed that maximum distance defined in terms of the minimum input wave energy known to have liquefied a site should be used instead.

Seed et al. (2001) indicate that lessons learned from and data provided by serious of earthquakes over the past two decades supply to do rapid evolution in the treatment of both liquefaction and site response. Even though the rate of progress has been impressive, more remains need to be done. A number of major recent and ongoing developments in each of these two important areas of seismic practice, and offers insights regarding work/research in progress, as well as suggestions regarding further advances needed were highlighted.

For several parameters, probabilistic and deterministic analyses were used to determine the safety factors (Ozcep and Zarif, 2009). The magnitudes and acceleration values of the earthquakes in hazard analysis were respectively chosen as 6.5, 7.0 and 7.5 (magnitudes), and as 0.25, 0.30, 0.35, 0.40, 0.45 and 0.50 g (accelerations). The field data (both SPT (N) and S wave data), obtained from the Yalova region, were used for cyclic stress ana-lysis of liquefaction. First, the study of the cyclic stress ratio approach was applied for all data to analysis of soil liquefaction. Then, factor of safety values of liquefaction were estimated with this approach.

Figure 1. Tilted building about 60° in Tigcilar District (Fırat et al., 2002).

Observed ground deformations and displacements after the earthquake, the results of field investigations by means of borings and in situ index tests including SPT, static cone penetration tests (CPT) and piezocone (CPTU) tests, analyses of observed ground settlements and lateral deformations by a suite of methods, and comparisons of observed and calculated ground movements were mainly discussed by Cetin et al., 2002.

SPT was used to determine stiffness and consistency of the Quaternary aged alluvium soils (Ulamıs and Kilic, 2008). Along the selected seismic profiles, P and S wave velocities of soil were measured. The index and physical properties of the samples were also determined in the laboratory. Two different methods based on SPT and Vs were used to investigate liquefaction potential and safety factor of the sandy levels in Quaternary aged alluvium. At different levels of the boreholes, liquefaction seems to be a significant risk in case of an earthquake with a_{max} = 0.48 g and M_w = 7.5 and may bring out environmental problems in the future.

Cyclic triaxial tests were used to evaluate the liquefaction resistance of a saturated fine to medium sand mixed with varying amounts of non-plastic fines (Xenaki and Athanasopoulos, 2003). The test results indicated that, for fines content increasing from 0 to 44%, the liquefaction resistance of mixtures with constant global void ratio decreased compared to that of the clean sand, whereas this trend is reversed for values of fines content greater than 44%. Nevertheless, for increasing values of fines content, when intergranular or interfine void ratios were kept constant, the liquefaction resistance of the mixtures varied monotonically (that is continuously increased or decreased respectively).

Liam (2002) presents the evaluation of seismic liquefaction potential in saturated sands and silts under earthquake shaking for level ground conditions which is the most common situation in engineering practice. Some standard liquefaction assessment charts, for level ground, earthquake moment magnitude M_W = 7.5 and effective confining stress of 100 kPa, based on data from various penetration tests and on in situ shear wave velocity are also presented, discussed and critically reviewed.

In this study, an assessment of liquefaction potential in Adapazari city after 17, August 1999 Marmara earthquake is made using borehole logs where SPT data are available. Over the past four decades, significant efforts have been done in both understanding and practice with regard to engineering evaluation of seismic soil liquefaction and site response. The evaluation of liquefaction potential has been made by four well-known state of art approaches, namely, Seed-Idriss simplified empirical procedure (Seed and Idriss, 1971; Seed et al., 1985; Youd et al., 2001), Tokimatsu-Yoshimi approach (Tokimatsu and Yoshimi, 1983), Seed-De Alba methods (Seed and De Alba, 1986) and Japan Road Association (Manual for Zonation on Seismic Geotechnical Hazards (Revised Version), 1999). Software developed by the writers was used to compute liquefaction susceptibility. The aim of this study is to compare the distribution of liquefaction occurrences generated by 17 August 1999 Marmara earthquake and numerically calculated liquefaction potentials. In addition, this paper presents the results of the four well-known approaches, comparatively.

GEOLOGY AND LOCAL SITE CONDITIONS OF ADAPAZARI REGION

Sakarya (Adapazari) province is placed on the low land formed by two rivers, the Sakarya River, which is one of the biggest in Turkey, and the Cark River that flows on the east and west sides of the city. Downtown Adapazari is almost flat with an altitude of 31 m. The flat area was filled by very deep alluvial deposits transported by two rivers. The Sakarya basin is mainly made of Quaternary alluvial deposits consisting mostly gravelly and silty sands. Extrusive rocks and Eocene Flysch are usually encountered in the western part of Adapazari Valley. Eocene Flysch is mostly found uncomfortably on Upper Cretaceous aged limestone (Ambraseys and Zatopek, 1969).

At downtown Adapazari, there is numerous numbers of logs available from boreholes drilled by various government agencies, the local public body and private consultants. Depth of these boreholes ranges from 15 -30 m. Also, some deep borehole logs (up to 150 m, Bakir et al., 2005) performed to obtain general soil and formations characteristics of Adapazari city. Due to its geology and local ground conditions, the valley has a high liquefaction potential during earthquakes.

Many soil profiles characterized as loose silts and silty sand layers, although at some locations a 4 - 5 m thick layer of dense coarse sand or fine gravel lies between surficial silt or silty sand layer and deeper clay layers (Celebi et al., 2009). Groundwater level in Adapazari region changes significantly and may come to within 0.5 m of the ground surface during the spring seasons.

SEISMO-TECTONICS OF ADAPAZARI REGION

Turkey is located on active earthquake belt, known as the Alpine belt. The 1939 Erzincan, 1966 Varto, 1967 Adapazari-Mudurnu, 1970 Gediz, 1971 Burdur and Bingol, 1992 Erzincan, 1995 Dinar, 1998 Adana-Ceyhan, 1999 Marmara, 1999 Bolu-Duzce and 2002 Afyon-Sultandagi earthquakes show how active the area is. The NAF (North Anatolian Fault), which is a well-known strike-slip fault, runs from east to west. City of Adapazari is located on the west side of the NAF and has experienced of big earthquakes in several times in the history (Figure 2).

In Adapazari region, there have been many earthquakes recorded since 1881. 5 of these earthquakes had magnitudes greater then 6 (M_s > 6) and 4 greater than 7 (M_s > 7). The August 17, 1999, Marmara earthquake (M_w = 7.4) is the biggest and most destructive one in the history of the region. It produced right-lateral onshore surface slips along an east-west trending zone of right-stepping fault strands over a distance of about 120 km. The slip was typically 2.5 - 4.5 m, reaching a maximum of approximately 5 m at a location about 30 km to the east of the epicenter. The surface expression of rupture consisted of tension cracks and fissures with limited positive relief along a 10 - 20 m zone (Erdik, 2001).

LIQUEFACTION ASSESSMENT AT DOWNTOWN ADAPAZARI

To assess the potential for liquefaction of a site, it is necessary to determine the geotechnical characteristics through laboratory and field tests. Generally speaking, liquefiable soils are loose ranging from silts to gravel. It is very difficult to obtain a representative samples for laboratory tests and create the same in-situ stress conditions in the laboratory environment. Therefore, *in-situ* testing of soil deposit to predict liquefaction potential is used engineering practice. The types of *in-situ* tests currently being used to predict of the liquefaction susceptibility are SPT, CPT, Flat Plate Dilatometer Test (DMT) and other variants of these methods. In that sense, these methods are attractive, since they are uncomplicated and they always direct contact with the soil (Glasser and Chung, 1995).

SPT is an *in-situ* dynamic penetration test designed to provide information on the geotechnical engineering properties of soil. The key reason of the test is to supply an indication of the relative density of granular deposits, for example sands and gravels from which it is virtually impossible to obtain undisturbed samples. The soil strength

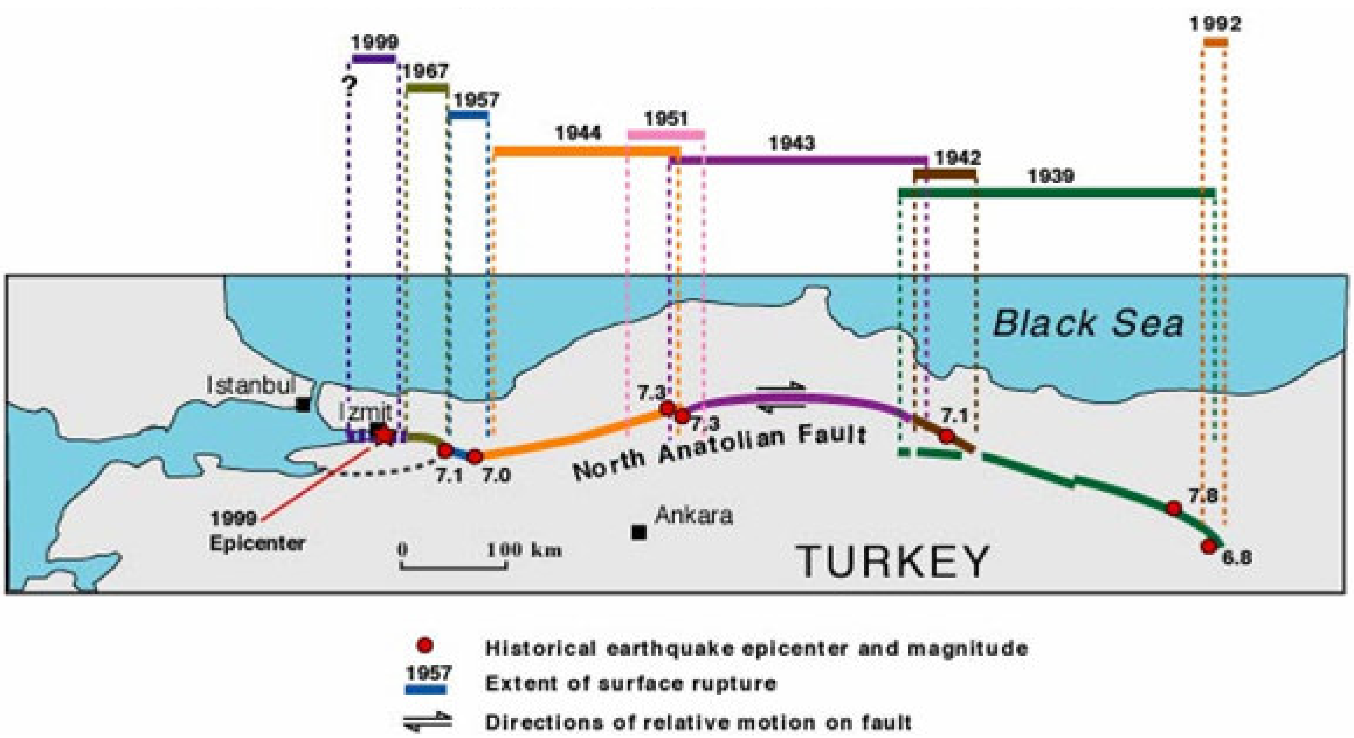

Figure 2. Recent earthquakes on the NAF (Location of August 17, 1999 Turkish Earthquake) (USGS, 2009).

parameters which can be inferred are approximate, but may give a useful guide in ground conditions where it may not be possible to obtain borehole samples of adequate quality like gravels, sands, silts, clay containing sand or gravel and weak rock. In conditions where the quality of the undisturbed sample is suspect, e.g. very silty or very sandy clays, or hard clays, it is often advantageous to alternate the sampling with standard penetration tests to check the strength (Soils and Foundations Handbook, 2004). The study area is investigated imaginary three lines to observe the liquefaction and detail borehole logs are also given in Figure 3.

SPT follows three main steps in evaluation the liquefaction assessment of an area: (1) calculation of the Cyclic Stress Ratio (CSR) induced in the soil by an earthquake, (2) assessment of the capacity of the soil to resist liquefaction using in-situ test data from SPT, expressed as Cyclic Resistance Ratio (CRR) and (3) evaluation of liquefaction potential by calculating a factor of safety (FS) against liquefaction, where; $FS = \frac{CRR}{CSR}$.

Liquefaction analysis

Simplified method: The simplified procedure, which is mainly developed by Seed and Idriss, 1971; Seed et al., 1985, for liquefaction assessment and it is widely used to analyze in the subsurface profile that is susceptible for liquefaction. This method has been updated by the other researchers (Youd et al., 2001).

For a particular SPT-N value, the simplified procedure mentioned above compares CSR generated by the earthquake to CRR to resist liquefaction in the ground. If CSR exceeds CRR, the ground is considered to be liquefied. The maximum cyclic shear stress developed on a horizontal plane during cyclic loading is estimated by the approximate equation:

$$\tau_{max} = \frac{a_{max}}{g} \gamma z r_d \tag{1}$$

Where; τ_{max} is maximum shear stress developed on horizontal plane, a_{max} is maximum horizontal ground acceleration, g is acceleration of gravity, γ is unit weight of soil, z is depth from the surface and r_d is stress reduction factor ($r_d = 1 - 0.015z$).

The soil in the field is considered to undergo an average stress τ_{avg} which is 0.65 of τ_{max}. Then, the average shear stress is normalized by the vertical effective stress to obtain CSR induced by the earthquake:

$$CSR = \frac{\tau_{avg}}{\sigma_V'} = 0.65 \frac{a_{max}}{g} \left(\frac{\sigma_v}{\sigma_v'} \right) r_d \tag{2}$$

Where; σ_v is the total vertical stress and σ_v' is vertical effective stress, τ_{avg} is the average cyclic shear stress generated by the earthquake, σ_v and σ_v' are total and effective overburden

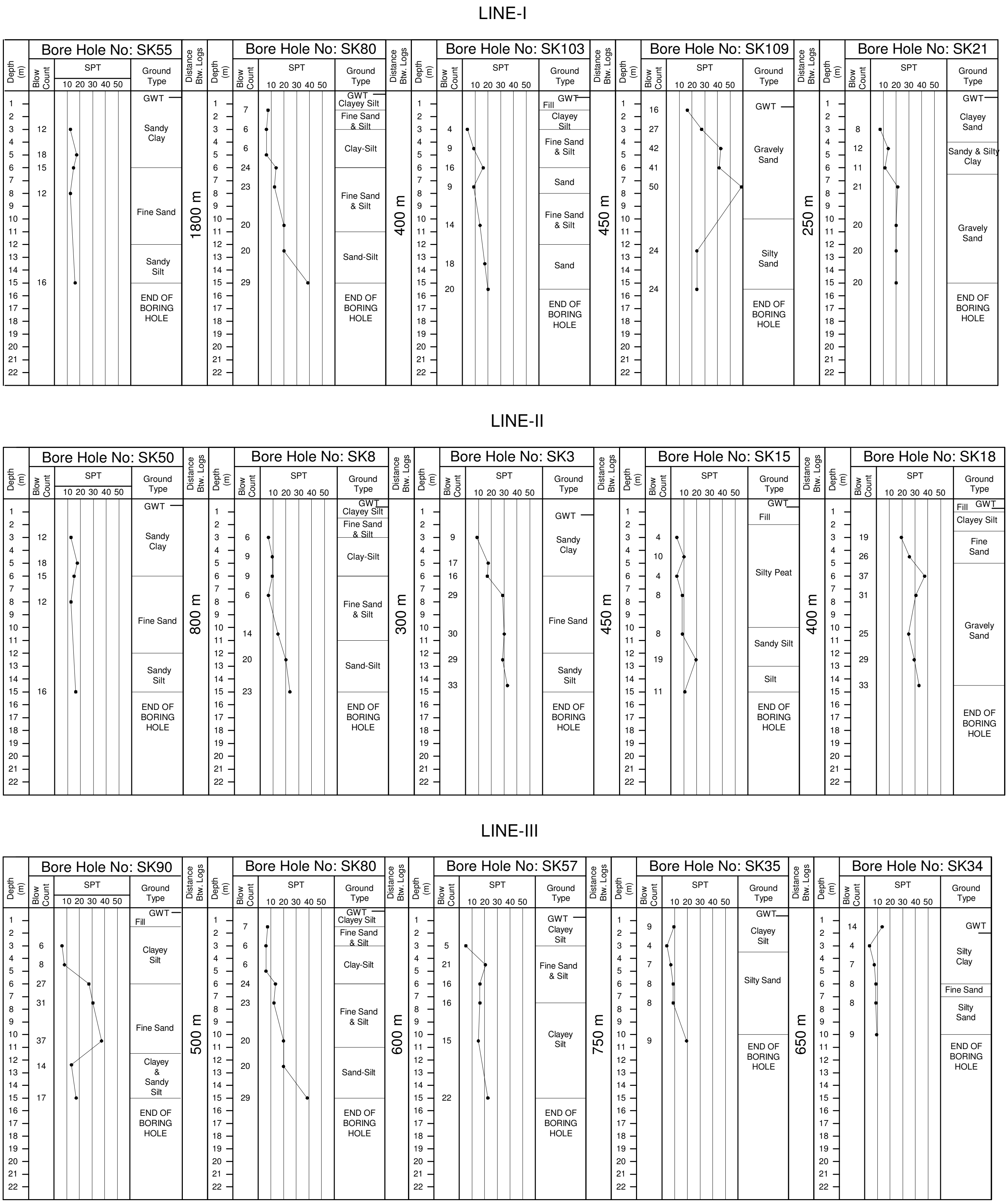

Figure 3. Typical ground cross section of downtown Adapazari.

stresses, respectively, 0.65 is weighting factor calculate the number of uniform stress cycles required to produce the same pore water pressure increase as an irregular earthquake ground motion, and r_d is a stress reduction coefficient which is defined in Table 1.

The evaluation of *in-situ* liquefaction susceptibility based on SPT test requires the determination of the cyclic strength of the soil deposits. In Seed and Idriss, 1971 and Seed et al., 1985 simplified empirical procedure, CRR is determined from CSR versus corrected blow count $(N_1)_{60}$ curves developed by Seed et al., 1985, (Figure 4). Youd et al. (2001) approximated the simplified base curve plotted on Figure 4 using the following equation:

$$CRR_{7.5} = \frac{a + cx + ex^2 + gx^3}{1 + bx + dx^2 + fx^3 + hx^4} \tag{3}$$

Where; $CRR_{7.5}$ is the cyclic resistance ratio for magnitude 7.5 earthquakes; $x = (N_1)_{60}$; $a = 0.048$; $b = -0.1248$; $c = -0.004721$; $d = 0.009578$; $e = 0.0006136$; $f = -0.0003285$; $g = -1.673E-05$ and $h = 3.714E-06$.

Equation (3) is valid for $(N_1)_{60}$ less than 30 and may be used in spreadsheets and other analytical techniques to approximate the simplified base curve for engineering calculations. For the values of $(N_1)_{60}$ greater than 30, CRR is set to 1.20.

Tokimatsu and Yoshimi method: In Tokimatsu and Yoshimi (1983) method, CSR induced by earthquake may be defined as follow:

$$CSR = \frac{\tau_{avg}}{\sigma_V'} = 0.1\,(M_s - 1)\frac{a_{max}}{g}\left(\frac{\sigma_v}{\sigma_v'}\right) r_d \tag{4}$$

Where; M_s is magnitude of earthquake and a_{max} is the maximum horizontal ground acceleration which is used in this research 0.407 g obtained by 17 August, 1999 Marmara Earthquake from Adapazari SKR acceleration record station. The rest of parameters in Equation (4) are same as Equation (1).

To compute CRR, the following approximate equation is given:

$$CRR = \frac{\tau_1}{\sigma_V'} = A\,C_r\left[\frac{16\sqrt{N_a}}{100} + \left(\frac{16\sqrt{N_a}}{C_s}\right)^N\right] \tag{5}$$

Where; τ_1 is shear stress on horizontal plane of representative samples of in-situ soil, σ_v' is vertical effective stress, $A = 0.45, C_r = 057, N = 14$ and $C_s = 0.85$ are coefficients. N_a is defined as follows:

$$N_a = N_1 + \Delta N_f \tag{6}$$

The SPT-N values, normalized for σ_v' = 1 kgf/cm² (98 kPa), N_1 may be approximately given by:

Table 1. Stress reduction coefficient definition based on z* values.

r_d= 1.0-0.00765z	for z ≤ 9.15 m	(2a)
r_d= 1.174-0.02676z	for 9.15 m < z ≤ 23 m	(2b)
r_d= 0.744-0.008z	for 23 m < z ≤ 30 m	(2c)
r_d= 0.05z	for z > 30 m	(2d)

*where z is depth below ground surface in meters.

$$N_1 = C_N N = \frac{1.7}{(0.7 + \sigma_v')} \tag{7}$$

in which C_N is a function of the effective vertical stress, σ_v', in kgf/cm^2, at the time when and at the depth where the penetration test was conducted. ΔN_f is an corrective factor for $SPT - N_a$ value for different fine grain size ratio. ΔN_f can be obtained from Table 2.

Seed and DeAlba method: Equation (2) is used to compute CSR generated by the earthquake in Seed and DeAlba (1986) method. CRR required to resist liquefaction in the soil is calculated from SPT-N value, vertical effective stress (σ_v'), the amount of energy delivered by the drilling hammer and the finest content of soil (< 0.075 mm). The following approximate equation estimated from the existing relationship between τ_1/σ_v' and $(N_1)_{60}$ is purposed to compute the CRR:

$$CRR = \frac{\tau_1}{\sigma_v'} = 0.0013 M_s (N_1)_{60}^{1/2} \tag{8}$$

Where; $(N_1)_{60}$ is the final corrected blow count.

The standard penetration blow count, N, is normalized to an effective stress of 1 kgf/cm^2 (98 kPa) may be approximately given by:

$$N_1 = C_N N = \frac{3.0}{(2 + \sigma_v')} N \tag{9}$$

The value N_1 is then corrected for the measured hammer energy ratio, $ER_r (ER_r \approx 65)$, delivered to the drill stem (a hammer energy ratio of 60 percent is the standard and used in the USA).

$$(N_1)_{60} = \frac{ER_r}{60} N_1 \tag{10}$$

Japon road association method: In Japan Road Association, revised in 1999 by ISSMGE (Manual for Zonation on Seismic Geotechnical Hazards (Revised Version, 1999), CSR during an earthquake is calculated as follows:

$$CSR = r_d k_{hc} \frac{\sigma_{vo}}{\sigma'_{vo}} \tag{11}$$

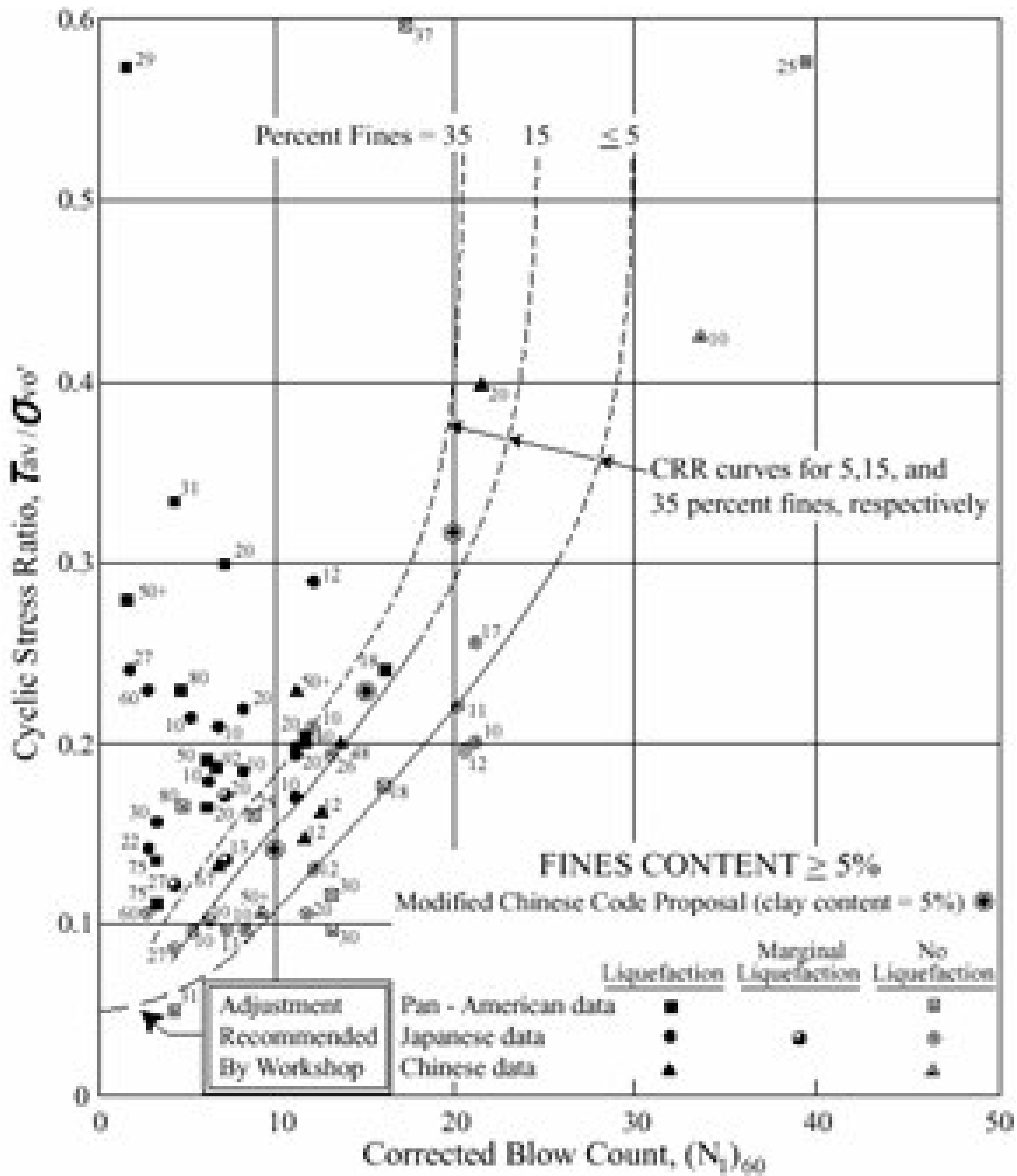

Figure 4. Simplified base curve recommended for calculation of CRR from SPT data along with empirical liquefaction data (Youd et al., 2001).

Table 2. Correlation factors for determining $SPT - N_a$ value.

Fines content FC (%)	ΔN_f
0 - 5	0
5 - 10	interpolate
10 -	$0.1 \times FC + 4$

Where; $k_{hc} = k_{hc0} \, x \, c_z$ in which k_{hc} is the horizontal seismic coefficient at the ground surface, c_z is the seismic zone factor taken as 0.8 and k_{hc0} is the standard horizontal seismic coefficient taken as 0.4. Two types of ground motion, which is the plate boundary type large scale earthquake ground motion with a number of large amplitude acts cyclically for longer time (seismic motion Type I) and the inland direct strike type earthquake ground motion of relatively short duration with a less number of cycles (seismic motion Type II), was considered, and type of ground was classified into three classes based on the ground characteristics (ISSMGE Manuel, 1999).

Calculation of factor of safety

To adjust the cyclic resistance ratio determined for magnitude 7.5

earthquakes to magnitudes smaller or larger than 7.5, introduced correction factor called "magnitude scaling factors", MSF. It is defined by the following equation given by Youd et al., (2001):

$$MSF = \frac{10^{2.24}}{M_w^{2.56}} \quad (12)$$

The appropriate cyclic strength is obtained by:

$$CRR = (CRR_{7.5})\,MSF \quad (13)$$

The final step is the calculation of the factor of safety. All methods follow the same procedure after obtaining CSR and CRR as below,

$$FS_{Liquefaction} = \frac{CRR}{CSR} \quad (14)$$

Assessment of liquefaction using different methods

Adapazari is recently constructed city. General form of construction is a typical 4 - 5 story R/C frame structures. After the earthquake, regulations concerning construction have been changed and 2 - 3 story buildings are allowed.

The study area is located over deep alluvial sediments. A deep boring recently performed in Yenigun District by the General Directorate of State Hydraulic Works (DSI) did not reach bedrock at a depth of 200 m. The shallow soils (10 m) are recent deposits laid down by the Sakarya and Cark rivers, which often flooded the area until flood control dams were built recently. Sands accumulated the length of bends of the roundabout rivers, and the rivers flooded periodically leaving behind predominantly non-plastic silts, silty sands, and clays throughout the city (Sancio et al., 2002).

This site category is characterized by the occurrence of brown to reddish brown, loose non-plastic silt and sandy silt in the upper 4 m of the soil layer. The depth of this stratum across the area explored ranges from 0.5 - 2.5 m. Liquid limit (LL) indices for the silt range from 25 to 35% and its natural water content is generally greater than 0.9 LL. The fines content (FC) of the soil samples recovered in this stratum ranges from 52 to 97%, and is generally greater than 75%. The percentage of particles smaller than 5 mm ranges from 10 - 35%, and is normally between 20 and 30%. The corrected penetration resistance of this stratum, $(N_1)_{60}$, ranges from 3 to 15 (blows/30 cm), and is generally between 7 and 10. Organic matter within this material at a depth of 4 m was dated to be approximately 1000 years old, indicating that the upper brown silty materials are recent flood plain deposits that have a high susceptibility to liquefaction (Sancio et al., 2002).

Assessment of liquefaction for boreholes SPT data determined using authors' developed computer program. Flow chart of the program is given in Figure 5. As can be seen from Figure 6, which obtained by observation, downtown Adapazari was liquefied during the earthquake. The city investigated imaginary three lines to observe the liquefaction (Figure 6). The lines started and finished appropriate long distance from the city center. Liquefaction susceptibility data were plotted on Figure 7. The program evaluates the liquefaction susceptibility by using above mentioned four well-known liquefaction methods. By comparing Figure 6 and 7, similar liquefaction area was obtained. Comparisons are made with site investigations and theoretical approaches. It has been found that the results of assessment based on the Simplified Procedure were generally consistent with the actual performance of the ground investigation with a few exceptions. On the other hand, liquefaction estimation by Seed-De Alba and Tokimatsu-Yoshimi Methods has been concluded to be too conservative. Japan Road Association was not widely applicable to Adapazari due to its formulation. Especially, fines content and main grain size constrained applicability of this method to downtown of Adapazari.

RESULTS AND DISCUSSION

Extensive soil liquefaction was observed during the 1999 Marmara earthquake, causing settlements and tilting of many buildings, destroying drinking water supply lines and sewage systems in Adapazari. Uniform and non-uniform liquefaction settlement of 0-1.5 m was observed. Local variations in the characteristics of alluvial sediments in Adapazari come into sight to have played an essential function in the occurrence and non-occurrence of liquefaction. The degree of ground failure observed along three lines that pass through five downtown districts appears to be mainly controlled by soil condition, with ground failure occurring in zones that are susceptible to liquefaction analyzed using above mentioned procedures.

Most seriously damaged area was the central of Adapazari, where hundreds of five to six story buildings tilted and sunk substantially into the ground softened by liquefaction. Liquefaction induced ground failure caused sinking, settlement and tilting of structures especially, Tigcilar, Orta Mahalle, Yenicami, Kurtulus, Yenigun, Karaosman, Cumhuriyet, Yenidogan, Pabuccular, Akincilar districts, where they are located central of Adapazari. According to computer coded program by using boreholes SPT data, Liquefaction occurred usually 4 – 7 m under the ground. Site characterization played important role during the earthquake and of course will play important role again during the future earthquake.

The results indicate that the methods of soil liquefaction analysis should be carefully used for the assessment of site liquefaction. Any developed methods inhibit its own phenomena because they are developed in different parts of the world. As well-known, different soil sites have completely different characteristics. Simplified procedure was generally consistent with actual site investigation. Seed-De Alba and Tokimatsu-Yoshimi Methods have been concluded to be too conservative. Japan Road Association was not widely applicable to Adapazari due to its formulation. It can be concluded that these methods are practical but it is difficult and questionable to say that they are reliable for the assessment of liquefaction susceptibility of a ground due to uncertainties involved in estimation. Liquefaction and bearing capacity of soil in the region should

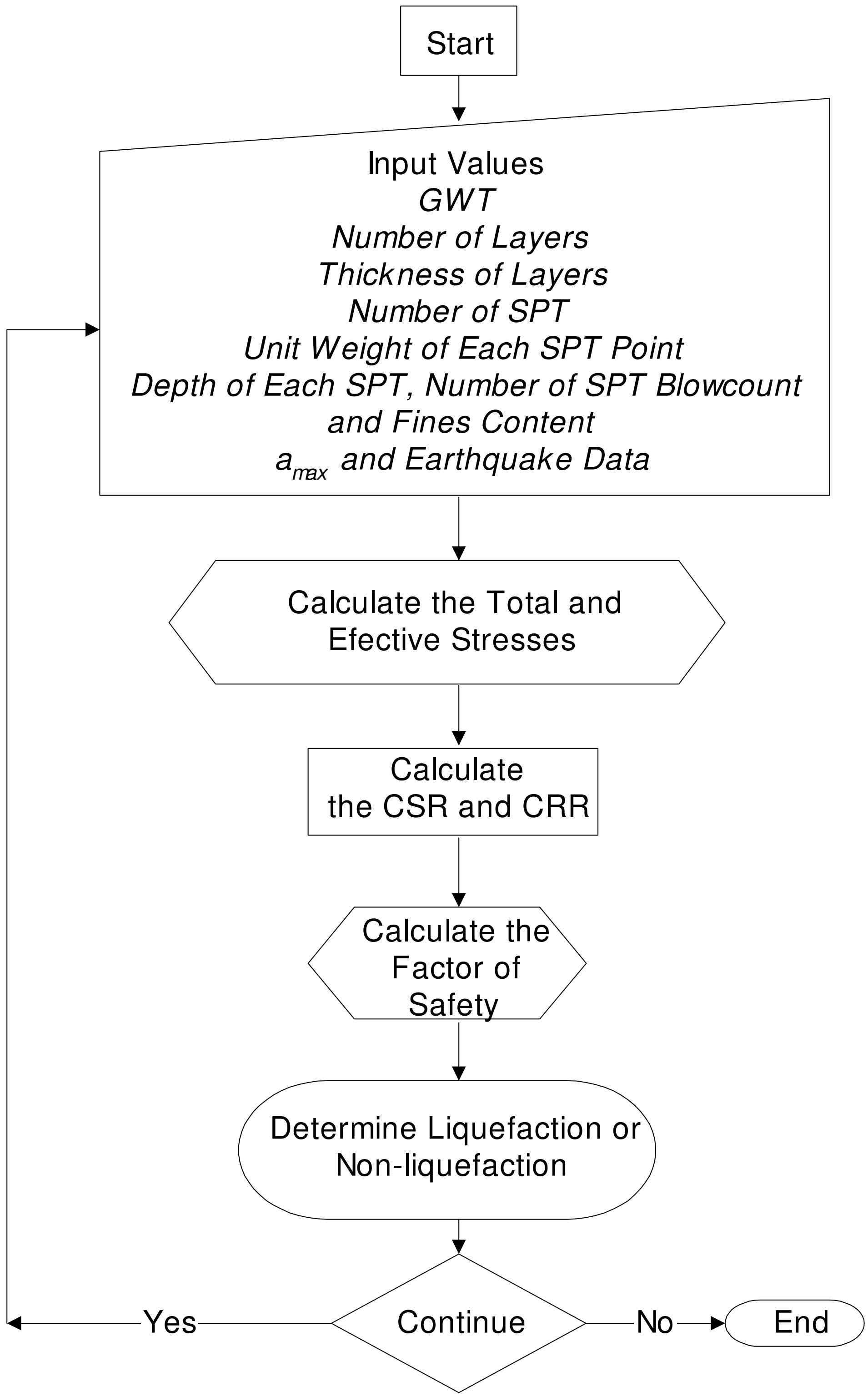

Figure 5. Flow chart of the liquefaction computer program.

be very carefully analyzed and considered for the necessary calculations. Corrective measurements and stabilization of ground are taken into account before any construction activities started.

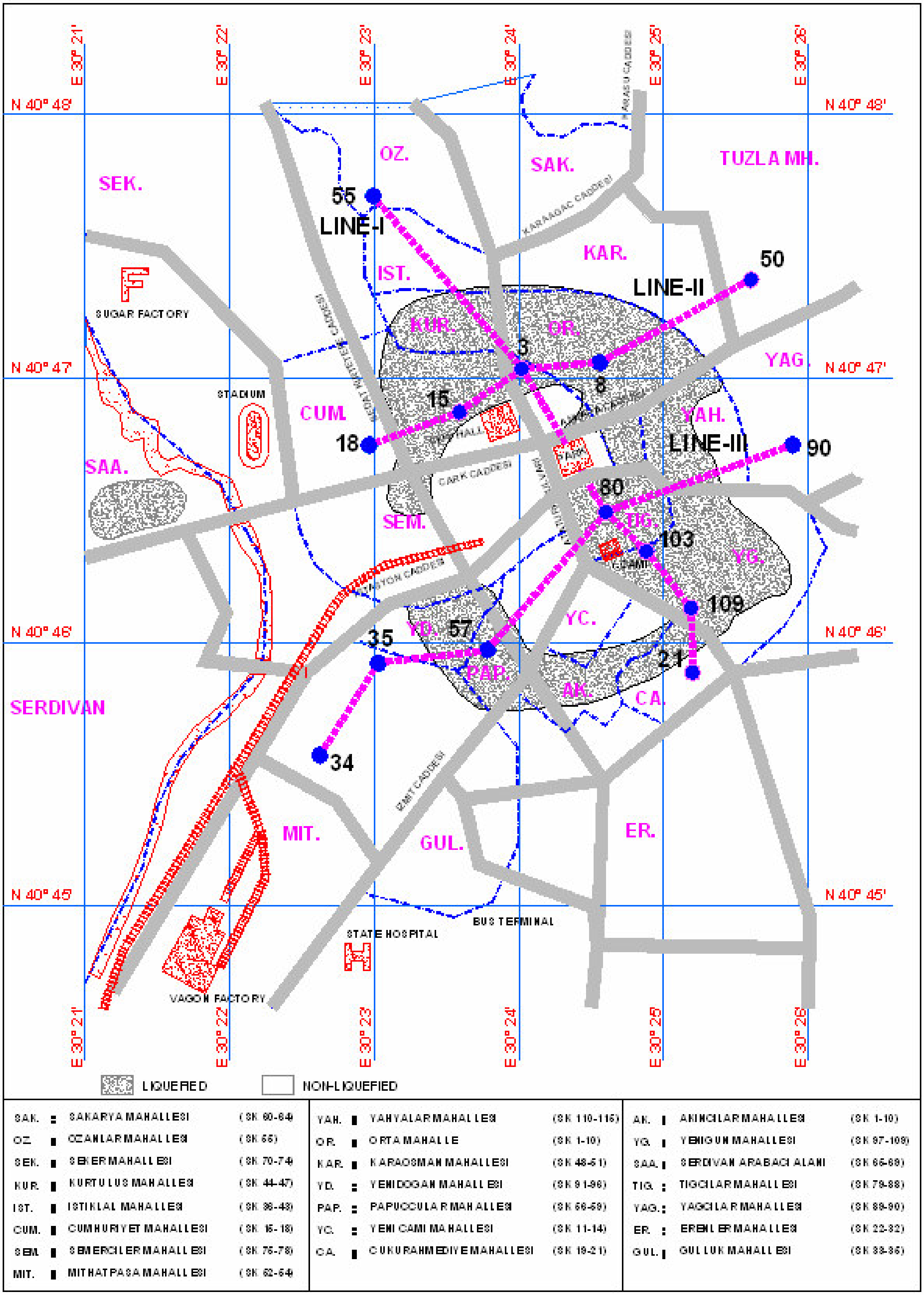

Figure 6. Observation of liquefaction and SPT locations.

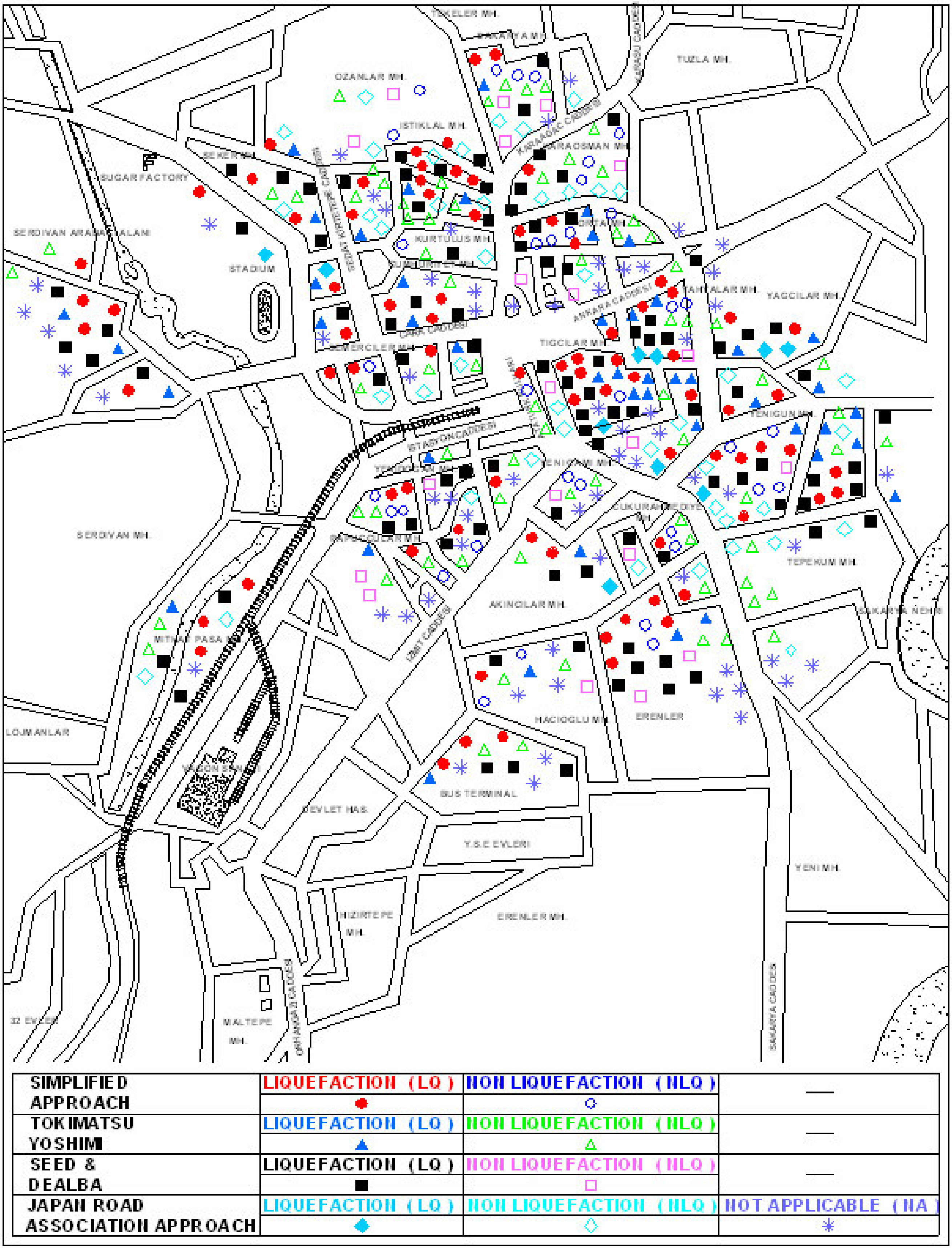

Figure 7. Schematic presentation of different liquefaction methods of analyses (a_{max}=0.407 g and M_w=7.4).

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