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Influence of index properties on the cyclic failure of fine-grained soils

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ABSTRACT

The purpose of this study is to develop an easy procedure to diagnose the cyclic behavior of fine-grained soils, leading to liquefaction.

There are approaches to this problem with reference to liquid limit (w_L), plasticity index (PI), clay content, (C %), in-situ water content compared to the liquid limit (w_n/w_L), average grain size (D_{50}), and liquidity index (I_L). Previous research has indicated that fine-grained soils defined as "sand-like" are prone to liquefaction. There is consensus that liquefiable silty soils have "sand like" characteristics. In the transition from sandy to clayey, it is generally accepted that it would be appropriate to make a judgment by making use of mechanical testing and that clayey soils do not liquefy.

This paper presents the results of dynamic triaxial shear testing (CTX), where excess pore water pressures and axial strains are evaluated compared to the physical properties to reach the threshold for cyclic failure. Samples of undisturbed soil procured from different locations of the city of Adapazarı, Turkey, were subjected to CTX testing. The results were interpreted in the light of the physical properties to arrive at a judgment of cyclic failure.

Being able to identify a liquefaction prone fine grained soil by merely measuring a few physical properties without resorting to arduous cyclic testing would be an advantage to the practising engineer. This however necessitates the elimination of the '*gray zone*' for which most researchers recommended the use of cyclic testing.

The main finding of this study is that samples reaching double strain amplitude (DSA) of <5% in the CTX test contain no sand and have a clay content of 20% or more. An attempt to establish a relationship between sensitivity to cyclic loading and physical properties is also made.

1. Introduction

Liquefaction is an inherent behavior of loose sand. Cyclic failure of silts and clays of low plasticity have been observed and reported during the 1944 Komei Town and 1964 Alaska earthquakes to be followed by comprehensive reports following the 1975 Haicheng and 1976 Tangshan quakes in China. Extensive research followed attempting to relate liquefiability to consistency and other physical properties.

The well-documented ground failures in the city of Adapazarı, Turkey, during the 1999 Marmara/Kocaeli earthquakes are the subject of this study (Bol, 2003; Bray et al., 2004a). Inspection of sites has shown that liquefaction occurred in the soil profiles dominated by low plasticity silts and silty sands. These profiles are products of flooding and meandering of River Sakarya, which currently flows along the city's eastern boundary (Bol, 2003; Bol et al., 2010). Several investigators have studied the events leading to liquefaction in Adapazarı (Çetin et al., 2002; Bray et al., 2004a; Bray et al., 2004b; Bakır et al., 2005; Erken et al., 2008; Kaya and Erken, 2015). Eventually, it was shown that ML silts can easily reach failure. All findings were used to develop the "*Adapazarı Criteria*", the sequel to the so-called Chinese Criteria, which uses the physical properties to define failure conditions (Bol et al., 2010).

Studies on liquefaction of undisturbed fine-grained soils in the literature were mainly conducted to evaluate the effects of cyclic stress

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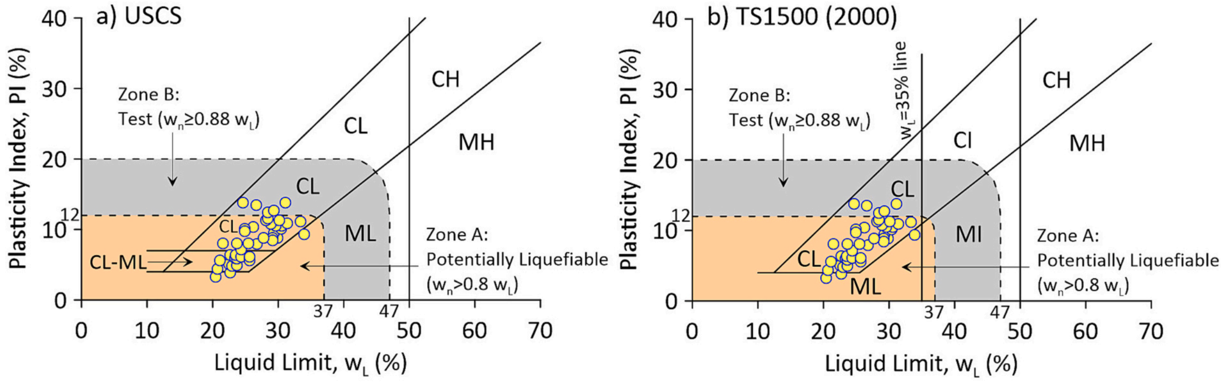

Fig. 1. a) Locations of liquefied soils on the plasticity card from major earthquakes in China (after Wang (1979) and Chang (1987)) (It is presented along with the limits of the Seed et al. (2003)) b) Displaying the same data on Turkish plasticity chart (TS1500, 2000).

Table 1
Limit values of physical properties of a liquefable fine-grained soil.

Reference	w_L(%)	*C (%)*	*PI (%)*	w_n/w_L	D_{50}(mm)	I_L
Wang (1979)	–	≤15–20*	–	>0.90	–	–
Jennings (1980)**	–	<10*	<10	–	>0.02	–
Seed and Idriss (1982)	<35	≤15*	–	≥0.90	–	–
Polito (1999)	≤30	–	≤10	–	–	–
Andrews and Martin (2000)	<32	≤10	–	–	–	–
Polito and Martin (2001)	<25	–	<7	–	–	–
Seed et al. (2003)	<37	–	<12	>0.80	–	–
Bray and Sancio (2006)	–	–	≤12	≥0.85	–	–
Bol et al. (2010)	<33	<10	–	***	>0.02	>0.90
Pathak and Purandare (2016)	–	≤15	≤15	≥0.90	>0.02	–

* clay size <5 μm, others <2 μm; $I_L = (w_n - w_p)/PI$.
** from chart of Institute of Engineering Mechanics (Harbin, China).
*** the same I_L value (0.9) being valid for NP silts with the definition (w_n/w_L).

ratio (CSR) on liquefaction. The most common method encountered in the literature when examining the liquefaction of fine-grained soils in the laboratory is to carry out tests on reconstituted samples. However, there are studies in which experiments were conducted on undisturbed samples (El Hosri et al., 1984; Puri, 1984; Guo and Prakash, 1999) also.

The cyclic behavior of undisturbed samples obtained from failure sites and those of no failure during the 1999 earthquake were studied. First, the basic physical and consolidation properties were measured. A dynamic triaxial test (CTX) was then performed because CTX is the most employed test for the purpose because it can be executed rapidly in contrast to the torsional triaxial, which is more realistic but challenging. The direct simple shear (DSS) also provides realistic results, but the indirect measurement of pore water pressures constitutes a major obstacle.

The records of observations made in the field after the earthquake were subsequently reevaluated with reference to the CTX results and the Adapazarı Criteria were reviewed under the light of laboratory results to diagnose the cyclic behavior of fine-grained soils.

Basic soil testing aimed at cyclic research consisted of determining the grain size distribution, consistency, clay content, and compressibility. Special attention was directed towards determination of overconsolidation ratio, which may be a significant factor in the failure mechanism of fine-grained soil.

2. Physical criteria for liquefaction of silts

Kramer (1996) and Robertson and Wride Fear (1997) stated that liquefaction could be described by two general terms, namely, flow liquefaction and cyclic softening. Cyclic softening is also divided into two as cyclic liquefaction and cyclic mobility. The phenomena referred to as "liquefaction" actually reflects several different ground failure types. Failure may occur in the form of cyclic softening or liquefaction. The volume decrease in the soil due to dynamic loading causes an increase in the pore water pressure, which reduces the average effective stress. The effective stress tends to zero at the end of the process, which is called initial liquefaction (Seed and Lee, 1966). On the other hand, cyclic softening can be defined as a decrease in resistance with the effect of cyclic loading, especially in cohesive soils, regardless of the increase in pore water pressure.

Attempts to diagnose sensitivity to cyclic ground failure by field testing has been partly successful. It is now agreed that the standard penetration test is not suitable, whereas the cone penetration test with pore pressure measurement is deemed to be 'promising'.

Literature on the other hand, is full of methods and rules to determine cyclic sensitivity of fine-grained soil that use physical properties (Wang, 1979; Jennings, 1980; Seed et al., 1983; Finn et al., 1994; Koester, 1994; Guo and Prakash, 1999; Andrews and Martin, 2000; Seed et al., 2003; Bray et al., 2004a; Bol et al., 2010).

Wang (1979) plotted the soils studied on the USCS plasticity chart (Chang, 1987) where the upper limits for liquid limits and plasticity indices are 35% and 15%, respectively (Fig. 1a). The interesting outcome is that numerous soils are located above the A-Line as clay of low plasticity, CL.

Seed et al. (2003) depicted the evaluation of liquefaction proposed by Wang (1979) and Seed and Idriss (1982), where three criteria have to be satisfied for soils above the A-Line. These are i) the soil contains <15% clay size (5 μm), ii) its liquid limit is <35%, iii) the natural water content must be >90% of the liquid limit.

The main problem encountered in the chart proposed by Seed et al. (2003) is specifying the liquefaction Zone A where the plasticity index is 0 to 12. This narrow interval cannot be differentiated easily through laboratory testing. Similar difficulty appears in the description of clay-like/sand-like behavior (Boulanger and Idriss, 2006), where the lower limit is given as PI = 7, above which clay-like performance is predicted. Simply stated, while only cyclic softening is exhibited in clayey silt and clay, liquefaction occurs in sandy mixtures. Boulanger and Idriss (2006) also noted that the cyclic resistance ratio (CRR) of a silt increases with increasing plasticity index.

One notable statement on this difficulty came from Sanin and Wijewickreme (2004). They performed the consistency tests at the University of British Columbia and compared the results with those supplied by an independent laboratory. They found that a difference in

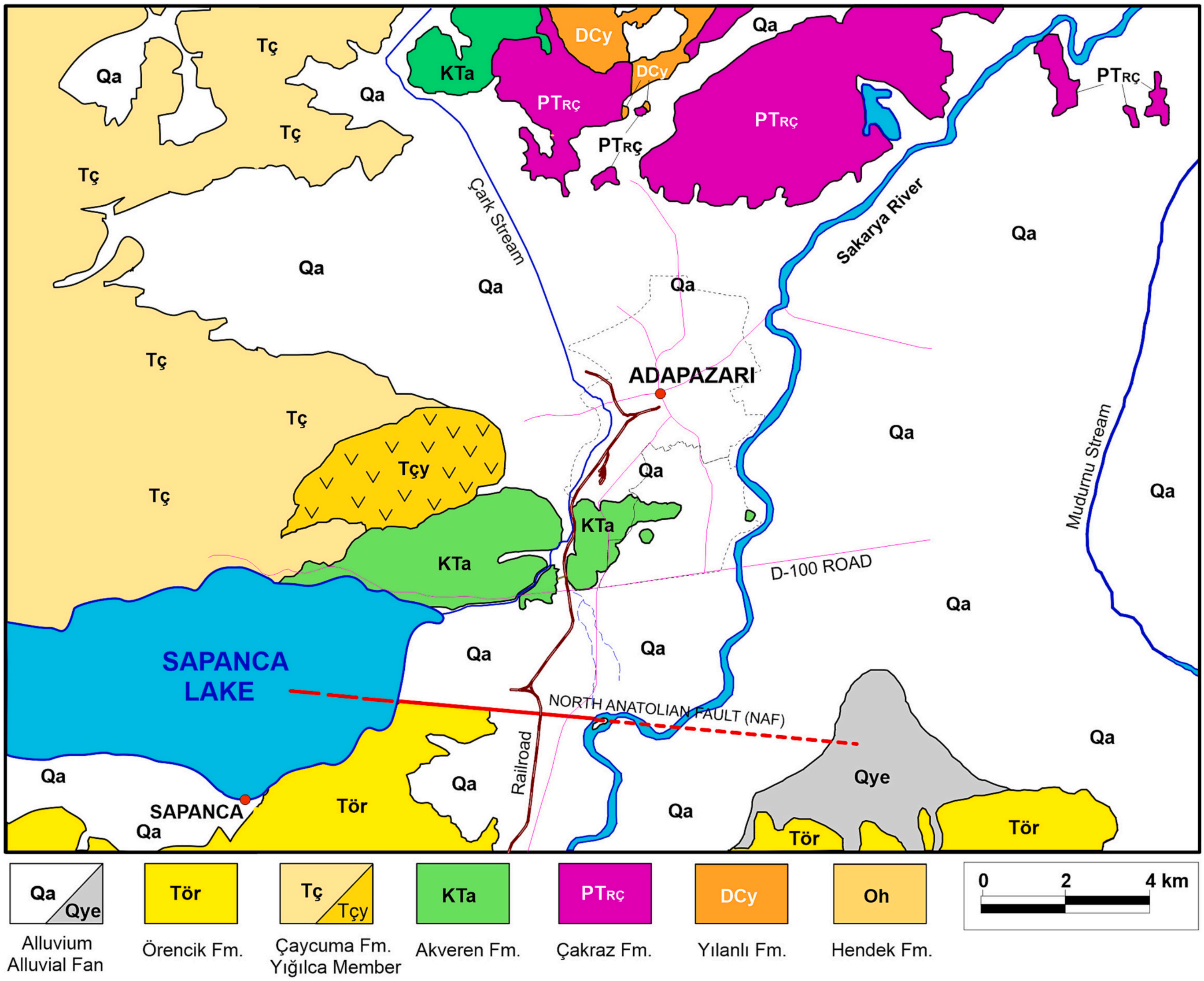

Fig. 2. Adapazarı central geological map (Revised from Sarıaslan et al., 1998).

the range of ±% 3 was expected due to the methods used in the measurement.

A different study (Andrews and Martin, 2000) expressed the opinion that the probability of cyclic failure of natural silts and fills depends on the key properties, clay content (<2 μm), and liquid limit only, where the boundaries are $w_L < 32$ and %C = 10%.

In contrast, Bray and Sancio (2006), evaluating the silts of Adapazarı, claimed that determination of cyclic failure can be made by the use of natural water content and plasticity index: All fine-grained soils with $w_n/w_L > 0.85$ and PI<18% are prone to liquefaction. The state where PI<12 is a sign of certain liquefiability, notwithstanding the fact that it is not possible to determine the liquidity index for non-plastic silt.

Bol et al. (2010) developed the "Adapazarı Criteria" in the light of data available and the field performance of soils during the 1999 earthquake of $M_w = 7$. The criteria were based mainly on site evidence whereby ground failure was visually registered. They found that the following four must be satisfied for a fine-grained soil to reach liquefaction; i) liquid limit $w_L < 35$ (percussion), ii) clay content (2 μm) %C < 10%, iii) average size $D_{50} > 0.02$ mm and iv) liquidity index $I_L > 0.9$.

The liquidity index ($w_n - w_P/PI$) may be overlooked if the plastic limit cannot be measured, and replaced by the ratio of natural water content to liquid limit (w_n/w_L). The gray zone was defined at $25 < w_L < 33$ and $10 <$ %C < 15.

Table 1 shows the criteria proposed by several investigators that have found generally used w_L, %C, PI, and w_n as indicators of sensitivity to cyclic loading. Liquidity index and grain size have also been considered. There is, however, no consensus on the limits. Resorting to laboratory cyclic testing in cases of uncertainty is recommended by all. There is agreement that liquefaction is likely to occur in sand-like profiles, and dynamic testing becomes significant as the transition zone from sand-like to clay-like response is approached. Clay-like soils are likely to resist to cyclic failure under normal circumstances.

As can be seen from Table 1, the physical properties of the soil are taken into account to identify liquefiable fine-grained soils. However, there is no consensus on which physical property will be used and the limit values of the physical property considered. This study investigates the effects of each of the physical parameters discussed in the literature.

3. Study area

The undisturbed samples used in this study were taken from within the boundaries of the Adapazarı city center. The Akveren Formation (KTa), represented by the Upper Cretaceous flysch, has a small part in the southwest of the study area (Fig. 2). The flysch sandstone is composed of claystone, marl, and local limestone. The region where the city of Adapazarı is established has a mostly flat topography, and the city

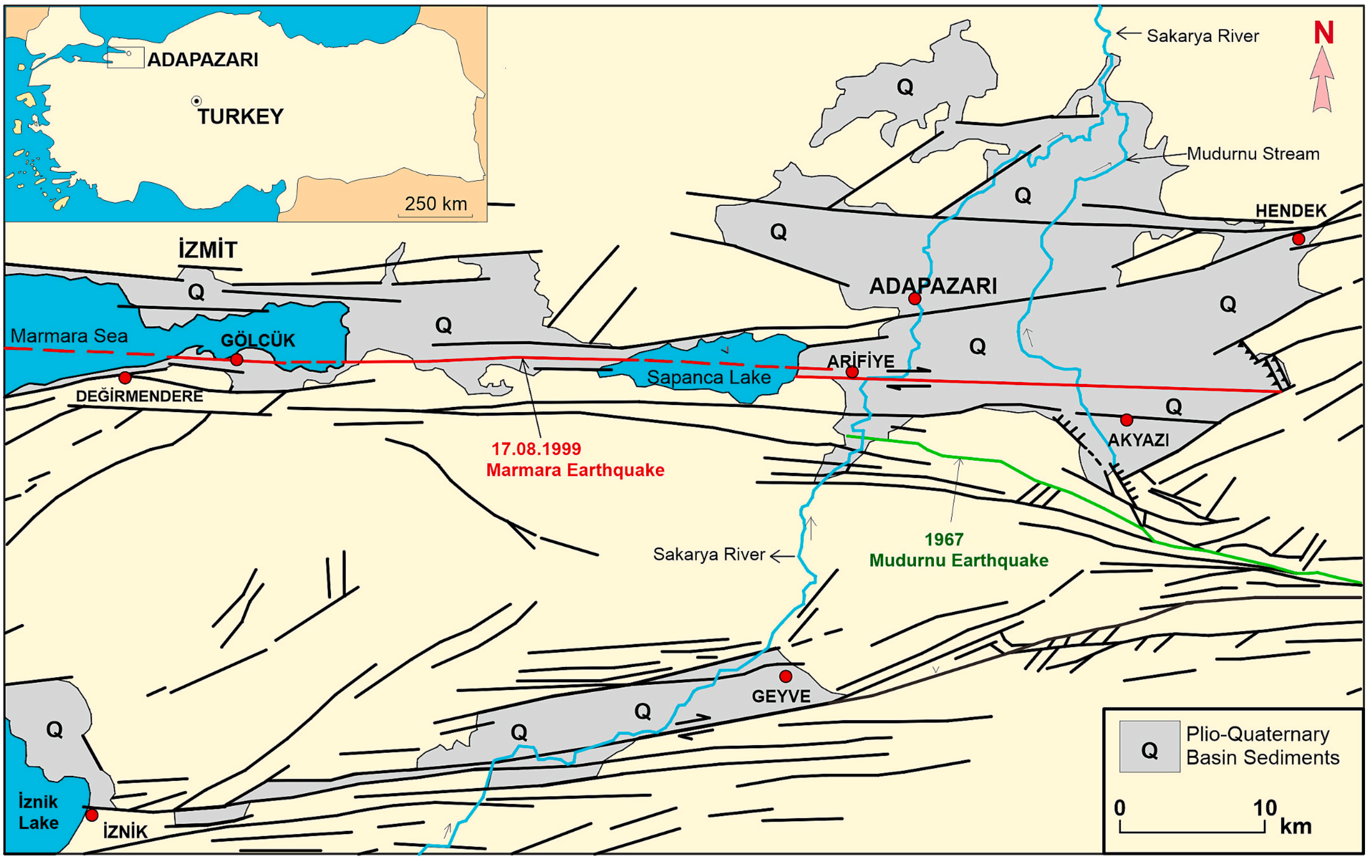

Fig. 3. Neotectonic map of Adapazarı and its surroundings (from Koçyiğit et al., 1999).

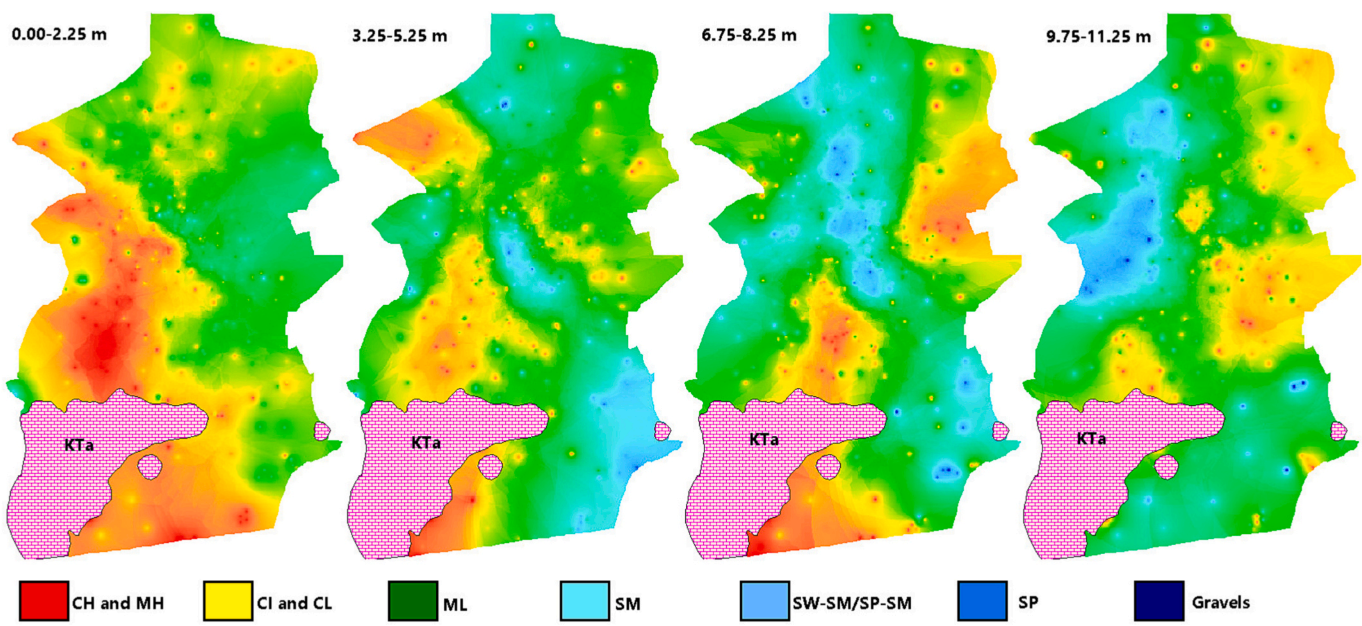

Fig. 4. Soil maps of study area for different depths (Bol, 2003; Bol, 2012).

is located on a thick alluvial fill of fluvial origin. Alluvium (Qa) was formed by the materials brought by the Sakarya river. In the 1999 earthquake, while damage to the structures on these rocky units (even though they have the same structural features as the buildings on the plain) remained at a minimum level, there are up to 50% heavily damaged/destroyed buildings in the settlements on the alluvium (Bol, 2012).

The region is tectonically active due to the 1600 km-long strike-slip North Anatolian Fault (NAF) extending from east to west of Turkey (Şengör, 1979). In Fig. 3, the distribution of the faults and Plio-Quaternary basin sediments around the research area is shown, and the faults broken in the last earthquakes are depicted. In this way, it is

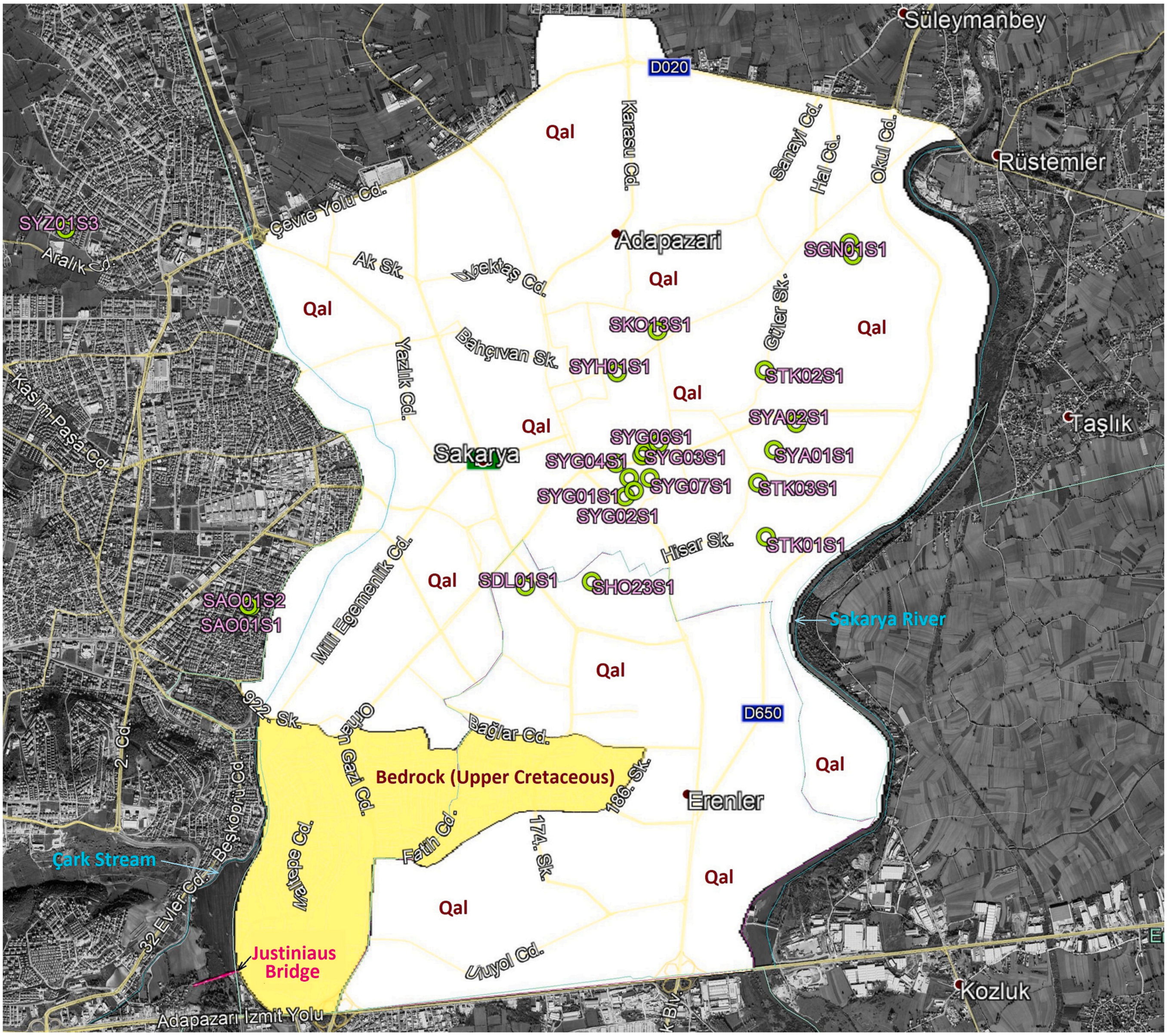

Fig. 5. Investigation sites.

seen how critical the study area is in terms of seismicity. The city of Adapazarı was heavily damaged by the severe earthquakes that occurred on various dates, as it is located on a thick alluvial deposit. Due to geology and local soil conditions, there is a high potential for liquefaction and soil amplification during earthquakes (Bray et al., 2004a; Bol et al., 2010; Bol, 2012; Bol, 2013).

No bedrock was found in the 200-m research boring carried out in the city. Komazawa et al. (2002) stated that the bedrock is at a depth of about 1000 m based on geophysical measurements. This thick alluvium also amplifies the effects of earthquakes on the bedrock. The properties of alluvium was studied in detail (Bol, 2003, 2012). Bol (2003) described the origins of all soils representing the city by dividing them into facies. 626 boreholes and 289 probings (CPT) data in the city center were analysed by geographic information systems and ground maps for different depths were produced. Ground maps representing different depths (0.00–2.25 m, 3.25–5.25 m, 6.75–8.25 m, and 9.75–11.25 m) of the city are illustrated in Fig. 4. As can be understood from here, the study area appears to have been influenced by streams flowing from the east and west borders. The soil profiles show abrupt changes both laterally and vertically. Thus, several different facies such as abandoned river channels, crevasse splays, backswamp deposits that a typical meandering river can leave behind are encountered in different zones and depths.

Silts are formed as a result of several physical processes, often by breaking down of the weak connections in the lattices of quartz crystals of sand grains. Adapazarı silt, which was used in this study, is a fluvial sediment whose properties have been investigated in detail previously (Önalp et al., 2007). These silts were deposited in limited volumes by the Sakarya River during the Holocene by flooding and meandering effect at various zones of the Adapazarı Plain (Akova). In the study area, clays cover wider areas and thicker layers. They were transported farther in the plain and to more areas than sand and silt due to their fine size during the floods of the Sakarya river.

The deposition stopped after the construction of flood prevention barriers on both sides of the river and three large dams upstream in the 1960s. In Adapazarı city, where this research program was carried out, the surface area is approximately 27 km^2. The ground profile has been influenced by a shift of the bed of the current Sakarya or another large river from west to east at about 5 km during the past 7000 years (Bol, 2003). This opinion is supported by the presence of a 6-span bridge built

Table 2
Physical properties of undisturbed soil samples tested in CTX.

	Site	GWT (m)	Depth (m)	e_0	GS	w_L (%)	w_n (%)	PI (%)	Clay (%)	D_{50} (mm)	Soil Class (TS1500)	Soil Class (USCS)	Consolidation Time (min)	Back Pressure (kPa)
1	HO23S1TX2	1.55	2.90–3.50	0.91	2.65	42	28.89	14	22	0.010	MI	ML	1200	250
2	DL01S1TX	1.98	1.50–2.00	1.03	2.64	60	38.41	33	44	0.003	CH	CH	3800	140
3	DL01S1TX	1.98	3.00–3.50	0.74	2.65	29	31.66	NP	16	0.020	ML	ML	25	250
4	YZ01S3TX	1.90	5.50–6.00	0.75	2.69	37	30.28	16	14	0.015	CI	CL	250	300
5	YG01S1TX	1.10	3.30–3.70	1.01	2.66	46	35.85	27	34	0.005	CI	CL	1500	140
6	YG02S1TX	2.00	2.45–2.95	0.61	2.64	32	24.32	10	9.7	0.026	CL	CL	250	240
7	YG02S1TX	2.00	3.70–4.10	1.03	2.70	43	28.09	23	29	0.007	CI	CL	700	100
8	YG03S1TX	1.87	2.50–3.00	0.92	2.65	36	32.40	9	11	0.035	MI	ML	60	190
9	YG03S1TX	1.87	3.30–3.80	0.84	2.64	36	35.10	12	11	0.025	CI	CL	–	–
10	YG05S1TX	1.65	1.50–2.00	0.86	2.65	30	28.01	NP	7	0.052	ML	ML	75	250
11	YG05S1TX	1.65	2.00–2.50	0.81	2.66	33	29.86	NP	9	0.030	ML	ML	75	250
12	YG053S1TX	1.65	2.50–3.00	0.69	2.65	30	33.30	NP	9	0.050	ML	ML	60	250
13	YG06S1TX	–	1.20–1.45	0.77	2.69	37	31.23	14	11.6	0.019	CI	CL	600	350
14	YG07S1TX	1.25	1.50–2.00	0.81	2.67	32	28.66	9	11	0.032	CL	CL	600	190
15	YG07S1TX	1.25	3.00–3.50	1.02	2.69	28	31.98	NP	19	0.013	ML	ML	55	250
16	YG07S1TX	1.25	4.50–5.00	0.91	2.67	31	31.59	7	11	0.043	ML	ML	90	350
17	TK01S1TX	2.60	3.85–4.35	0.77	2.61	34	33.32	15	6	0.057	CL	CL	170	150
18	TK01S1TX	2.60	6.00–6.50	0.82	2.68	44	31.64	22	19	0.008	CI	CL	500	140
19	TK02S1TX	2.00	4.00–4.50	0.80	2.65	34	31.40	7	20	0.016	ML	ML	100	190
20	TK03S1Tx	2.10	4.50–5.00	1.10	2.57	65	35.11	36	36	0.005	CH	CH	2200	190
21	GN01S1TX	1.90	1.50–1.95	0.78	2.58	37	28.84	16	16	0.022	CI	CL	550	200
22	GN01S1TX	1.90	3.00–3.50	0.85	2.58	40	33.92	18	16	0.020	CI	CL	950	140
23	GN01S1TX	1.90	5.00–5.50	1.01	2.66	47	37.07	24	31	0.006	CI	CL	1300	190
24	GN01S2TX	2.60	3.50–4.00	0.91	2.70	30	33.65	6	10	0.090	ML	ML	130	200
25	GN01S2TX	2.60	5.90–6.40	1.23	2.57	63	47.04	44	42	0.004	CH	CH	5000	200
26	YH01S1TX	1.39	1.95–2.45	0.98	2.65	36	29.59	15	9	0.020	CI	CL	500	140
27	YH01S1TX	1.39	2.45–3.00	0.80	2.71	30	30.17	6	4	0.100	ML	ML	400	200
28	YH01S1TX	1.39	5.00–5.50	0.83	2.62	39	30.22	21	13	0.035	CI	CL	260	140
29	YA01S1TX	2.22	2.00–2.50	0.82	2.65	36	31.43	9	22	0.012	MI	ML	900	190
30	YA02S1TX	1.88	2.50–3.00	1.17	2.69	47	33.81	23	24	0.007	CI	CL	1200	190
31	YA02S1TX	1.88	3.50–4.00	0.87	2.68	34	31.53	7	14	0.022	ML	ML	500	140
32	AO1S1TX2	1.50	3.00–3.45	0.88	2.69	35	25.68	16	17	0.033	CL	CL	80	290
33	AO01S1TX	1.50	5.00–5.35	0.71	2.64	35	33.00	10	11	0.025	ML	ML	50	350
34	AO01S1TX	1.50	5.80–6.30	0.74	2.68	32	27.41	5	12	0.030	ML	ML	250	200
35	AO01S2TX	1.80	4.00–4.50	0.82	2.64	31	27.44	11	8	0.032	CL	CL	300	250
36	TI01S1TX	2.35	2.00–2.50	0.83	2.65	30	29.10	NP	7	0.044	ML	ML	50	250
37	TI01S1TX	2.35	2.50–3.00	0.93	2.71	27	28.96	NP	11	0.052	ML	ML	50	300
38	TI01S1TX	2.35	3.00–3.50	0.80	2.72	33	32.48	NP	14	0.023	ML	ML	60	350
39	YG06S1TX-1	1.20	1.45–1.70	0.77	2.69	37	31.58	14	11.6	0.019	CI	CL	–	–
40	YA01S1TX	2.22	2.70–3.25	0.85	2.69	34	38.59	8	5	0.050	ML	ML	100	400

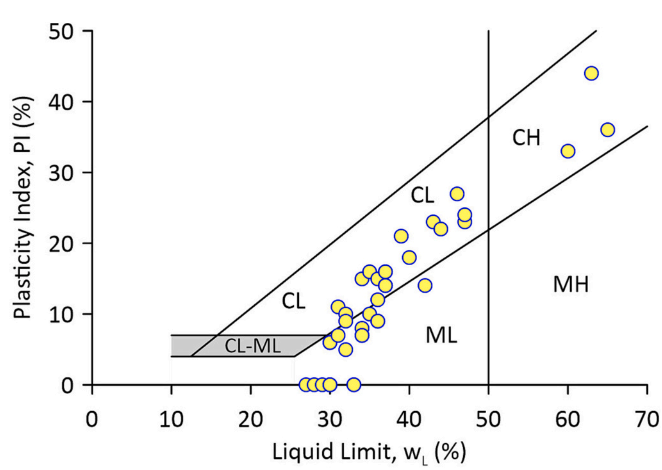

Fig. 6. Fine-grained soils used in the study on the USCS plasticity chart.

in Roman times, currently sitting on a little stream (Çark) that only discharges the excess water of the nearby Sapanca Lake.

The undisturbed samples tested in this study were taken from the sites in Fig. 4 where silts are frequently encountered, and liquefaction was widespread in the 1999 earthquake. The undisturbed samples were obtained with Shelby tubes made of seamless stainless steel (chrome). The use of the chrome tubes is advantageous because the friction between the soil and the tube is reduced. This tube area ratio defined by Hvorslev (1949) (100×[Outer Diameter^2-Inner Diameter^2]/Inner Diameter^2)) is %7.61. In addition, the samples could be extruded in the laboratory without difficulty. Bray et al. (2004a) reported that the type of clay in Adapazarı soils was determined as smectite or random layered illite/smectite, chlorite, illite, and kaolinite in the analyses performed. Smectite, the source of montmorillonite, is the dominant mineral of fine clay, and illite is abundant in larger clay sizes. Sizes <0.2 μm can be said to contain 85% smectite, 10% illite, and 5% chlorite and kaolinite in all cases.

40 UD samples collected from the study site were tested (Fig. 5) (Önalp et al., 2010). The consistency, grain size properties, void ratios, and specific gravities of the tested soils are summarized in Table 2. Soil classification was performed according to USCS, (ASTM D2487-17e1, 2017) and TS1500 (2000) standards. According to these classification systems, 19 of the samples, about half of the total, classified as silt and the remaining 21 were clay (Fig. 6). Silts tested are either of low plasticity or non-plastic. Only three clays showed high plasticity, and the others were of low plasticity according to the USCS. The significant difference between USCS and TS1500 is that USCS qualifies fine-grained soils as of low plasticity if the liquid limit is <50. In contrast, when the liquid limit is between 35 and 50, the soil is characterized as of medium plasticity by TS1500. Interestingly, most of the areas shown as the absolute liquefaction zone in most of the criteria mentioned above fall within the zone of low plasticity (w_L < 35) in TS1500 and not intermediate.

Fig. 7 shows the statistical distribution of the physical properties of

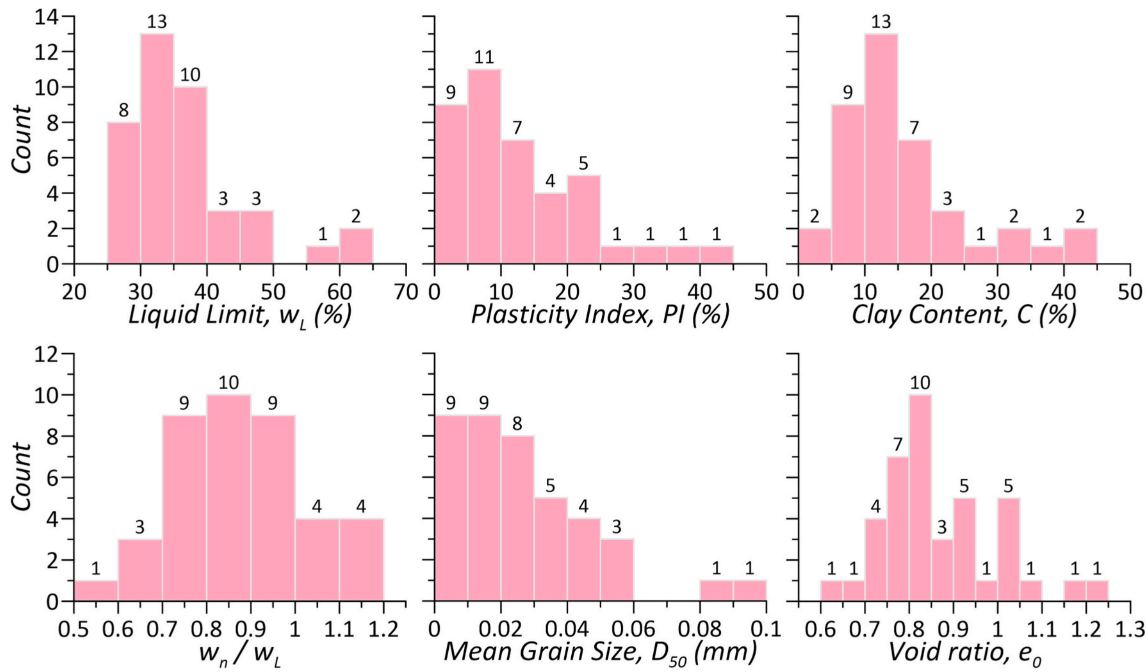

Fig. 7. Statistical distribution of the physical properties of the soils used in this study.

the samples listed in Table 2. It can be inferred from here that the majority of the samples are clustered within the liquefiable zone defined in the literature.

4. The cyclic triaxial test (CTX)

The triaxial shear test (TX) developed in the 1930s presented the researcher with the possibility of analyzing the stresses on any plane and subsequently measuring the pore water pressures to arrive at effective stresses. The CTX is a development of the seventies, followed by the torsional version. Some researchers suggest that the dynamic simple shear test (DDSS) should be preferred because of its ability to reflect the earthquake effect in a better way and its ease of use (Jefferies and Been, 2006; Kramer, 1996).

In this study, load controlled cyclic triaxial (CTX) test system of Wykeham Farrance (Controls) was used. The experiments were carried out according to the ASTM standard (ASTM D5311, 2011). The CTX differs from the static test system mainly in its load application. Here, too, the sample is first saturated, then consolidated with increased cell pressure against a reasonable back pressure and the shearing phase is performed without drainage by applying the cyclic deviator stress. The readings are recorded electronically since the loading is repetitive.

The soils of the study area are normally loaded or lightly overconsolidated, as they are composed of very young alluvial deposits. Cell pressure is a significant variable in dynamic tests. The reason for selecting $\sigma_3 = 100$ kPa is that liquefied soils in Adapazarı during the 1999 earthquakes were mostly observed in the upper 10 m.

In loose uniform sands where liquefaction is reached rapidly, the mechanism is defined as the stage at which the pore water pressure rises to reduce the effective stress to zero to achieve "initial" liquefaction. Strain values increase with the application of the deviator stress. Since drainage is not allowed, pore water pressures and axial strains (ε_z) increase rapidly as the number of cycles N increase. In this case, the cyclic stress in the probable shear plane is expressed as

$$CSR = \frac{\sigma_{dev}}{2\sigma_3'} = \frac{(\sigma_1 - \sigma_3)}{2\sigma_{3c}'} \tag{1}$$

The readings taken during the CTX are the volume change in the sample during the consolidation phase (ΔV), pore pressures (u_w), deviator stresses (σ_d), and axial strains (ε_z) during the shearing phase.

Using the test results of El Hosri et al. (1984) on undisturbed silt-clay mixtures, Guo and Prakash (1999) focused on the CSR values required for liquefaction of samples at different cycles. According to this study, resistance to liquefaction decreases with the increase of plasticity in the ranges where the plasticity index is low (PI<5), but the resistance to liquefaction increases with the increased plasticity at medium and high plasticity values (PI>5). However, the researchers stated that the development of excess pore water pressure and deformation properties in silt-clay mixtures were different from sand.

Andersen (2009) and Boulanger and Idriss (2006) associated CSR with undrained shearing resistance. An evaluation of CSR was made considering the studies of Guo and Prakash (1999) and Puri (1984) in this study. The statistical evaluation of the plasticity indices of the soils used in this study can be seen in Fig. 7, where the average plasticity indices of the investigated soils are in the range 10–15. According to Guo and Prakash (1999), a CSR value of 0.35 is required for soils with a plasticity index in the range of 10–12 to reach failure at the 15th cycle. In addition, in his study on fine-grained soils Puri (1984) stated that while the plasticity index is in the range 11–18, if the CSR value applied is between 0.30 and 0.40, the selected failure value of 5% DSA (double strain amplitude) is reached. The cyclic stress was kept constant in all tests of this study. Cell pressure, CSR, and frequency values were 100 kPa, 0.35, and 0.50 Hz respectively.

Pore pressure-based or deformation-based approaches stand out among the methods used in the literature for determining liquefaction or cyclic mobility. Strength-based evaluations are also performed.

Seed and Lee (1966) defined the initial liquefaction as when excess pore water pressure reaches the initial effective cell pressure ($u_{excess} = \sigma'$ or $r_u = 100\%$). The peak excess pore water pressure level required to initiate liquefaction is related to the amplitude and duration of the cyclic loading.

The cyclic stress approach assumes that the generation of excess pore pressure is mainly related to cyclic shear stresses; thus, seismic loading is expressed in terms of cyclic shear stresses. The loading can be estimated using a soil response analysis or a simplified approach. Laboratory data for which liquefaction resistance can be predicted is usually obtained from testing where cyclic shear stresses have uniform amplitudes of sinusoidal shape. Therefore, comparing the earthquake loading with the

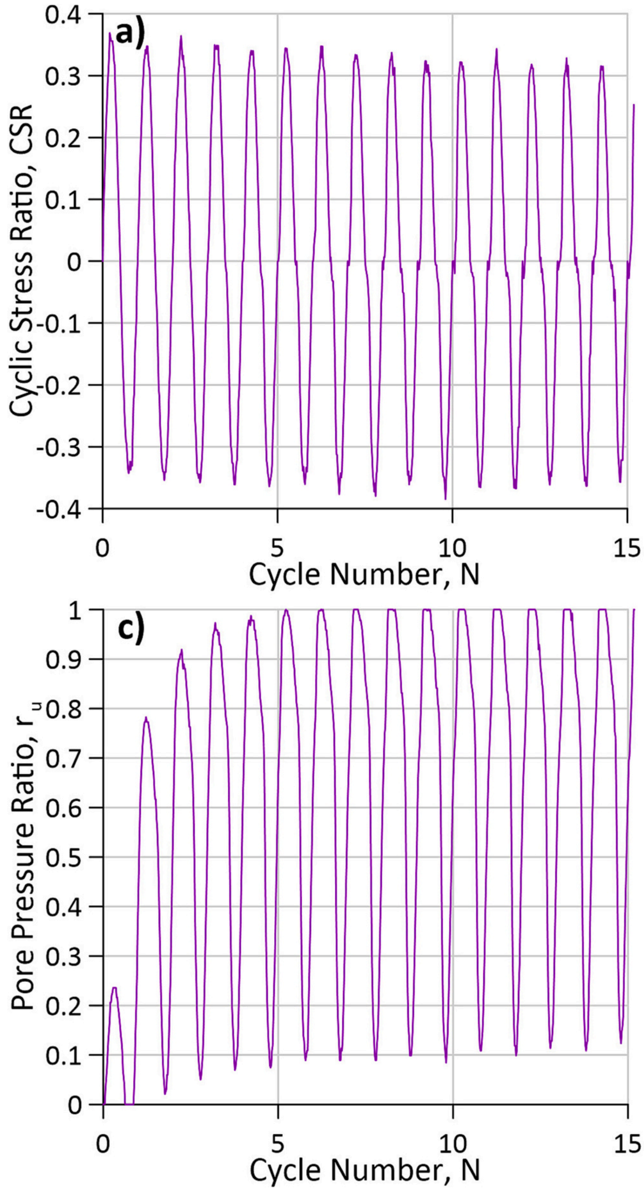

Fig. 8. Cyclic triaxial test results of a low-plasticity silt (ML) (No.27).

resistance determined in the laboratory requires converting the irregular shear stress time history into an equivalent set of uniform stress cycles. Seed and Lee (1966), Seed and Idriss (1971), and Seed et al. (1975) based on a large number of earthquake records, determined that an equivalent number of uniform cycles (N) can be correlated with the earthquake magnitude (M_w). Thus, soil samples can be tested in the laboratory under dynamic loading conditions similar to those produced by an earthquake, despite the fact that earthquake loading pattern is highly uneven. A $M_w = 7.5$ earthquake can be represented with 15 cycles in the laboratory setting (Seed and Idriss, 1982; Ishihara, 1996; Idriss, 1999). Zergoun and Vaid (1994) stated that the frequency of seismic waves is around 1 Hz according to Singh et al. (2017), soft soils can be characterized in the frequency range of 0.32–0.8 Hz. So, a constant frequency of 0.5 Hz was selected and kept constant in all samples during this study.

Ishihara (1993, 1996) showed that the pore water pressure ratio reaching 100% or the DSA reaching 5% in sands of a wide range of relative densities indicates liquefaction. However, he stated that liquefaction could be initiated in silty sand and sandy silt when the pore water pressure ratio reaches 90%. Both criteria were evaluated in this study. Pore water pressure values and DSA values of each sample in the 15th cycle were taken as the criterion for failure. Samples were considered critical, where the pore water pressure ratio was achieved at or above 90% or the DSA value reached 5% in the 15th cycle. Ishihara (1993) examined several laboratory cyclic triaxial tests and proposed using 5% axial double strain amplitude (DSA = ± 2.5% axial strain) in the CTX test as a criterion to define liquefaction and cyclic softening for both clean sands and fine sands.

5. Results

This section investigates the relationship between pore water pressure ratio, 5% DSA developed in the 15th cycle, and the physical properties. In this context, the liquid limit, plasticity index, clay ratio, w_n/w_L ratio, and average grain size associated with the liquefaction of fine-grained soils have been taken into account.

Even if samples exhibit high pore water pressure at the point of failure may not mean that this reflects the pore water pressure inside the clay structure. That is, with the increase in axial strain the sample starts to deform and cracks, fissures may form especially in natural high

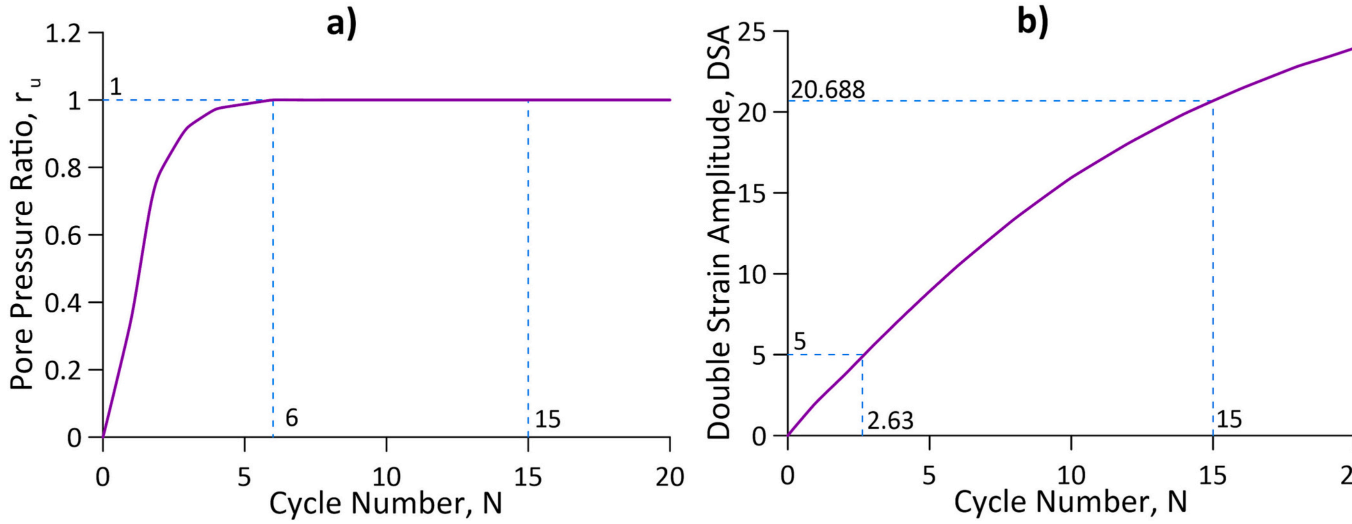

Fig. 9. The relationship between the pore pressure and DSA values of the ML soil (No.27) with the number of cycles.

Table 3
Summary of cyclic triaxial test results.

No	Site	Depth (m)	N for %5 DSA	% DSA for N15	r_u for N15	No	Site	Depth (m)	N for %5 DSA	% DSA for N15	r_u for N15
1	HO23S1TX2	2.90–3.50	17.45	4.57	0.89	21	GN01S1TX	1.50–1.95	3.50	15.02	0.97
2	SL01S1TX	1.50–2.00	30.43	3.45	0.27	22	GN01S1TX	3.00–3.50	6.23	7.98	0.97
3	DL01S1TX	3.00–3.50	3.00	14.83	0.98	23	GN01S1TX	5.00–5.50	18.82	4.07	0.74
4	YZ01S3TX	5.50–6.00	5.07	9.67	0.86	24	GN01S2TX	3.50–4.00	2.00	14.94	1.00
5	YG01S1TX	3.30–3.70	23.92	3.39	0.59	25	GN01S2TX	5.90–6.40		0.93	0.50
6	YG02S1TX	2.45–2.95	9.04	6.99	0.92	26	YH01S1TX	1.95–2.45	4.68	14.21	1.00
7	YG02S1TX	3.70–4.10	9.02	7.30	0.86	27	YH01S1TX	2.45–3.00	2.63	20.69	1.00
8	YG03S1TX	2.50–3.00	3.47	11.34	0.88	28	YH01S1TX	5.00–5.50	3.38	10.78	0.94
9	YG03S1TX	3.30–3.80	3.14	12.21	0.93	29	YA01S1TX	2.00–2.50	4.48	12.35	0.96
10	YG05S1TX	1.50–2.00	2.09	16.44	1.00	30	YA02S1TX	2.50–3.00	10.63	6.77	0.79
11	YG05S1TX	2.00–2.50	2.50	17.78	1.00	31	YA02S1TX	3.50–4.00	2.88	11.24	1.00
12	YG053S1TX	2.50–3.00	1.80	29.78	1.00	32	AO1S1TX2	3.00–3.45	2.39	17.61	0.98
13	YG06S1TX	1.20–1.45	13.45	5.41	0.77	33	AO01S1TX	5.00–5.35	1.13	20.40	0.97
14	YG07S1TX	1.50–2.00	2.00	18.55	1.00	34	AO01S1TX	5.80–6.30	2.36	13.39	0.91
15	YG07S1TX	3.00–3.50	2.48	25.18	0.96	35	AO01S2TX	4.00–4.50	4.62	15.44	1.00
16	YG07S1TX	4.50–5.00	1.33	27.03	1.00	36	TI01S1TX	2.00–2.50	2.37	16.44	1.00
17	TK01S1TX	3.85–4.35	1.87	13.58	1.00	37	TI01S1TX	2.50–3.00	2.45	19.44	1.00
18	TK01S1TX	6.00–6.50	5.41	8.29	0.92	38	TI01S1TX	3.00–3.50	2.14	20.72	0.99
19	TK02S1TX	4.00–4.50	1.76	23.75	0.92	39	YG06S1TX-1	1.45–1.70	6.31	7.97	0.81
20	TK03S1Tx	4.50–5.00	17.92	4.30	0.53	40	YA01S1TX	2.70–3.25	4.19	19.18	1.00

plasticity clayey samples. The pore water pressure may act along these fissured surfaces but not inside the clay structure itself because of low permeability (Andersen et al., 1980). Because of this it should be noted that pore pressure readings recorded for clayey soils may not have reached equilibrium particularly in the initial loading cycles. Although the clay contents, liquid limits and plasticity indices of the soils used in this study are generally low; for the reason mentioned above, it may be more appropriate to evaluate high plasticity clayey soils according to the deformation criteria.

Fig. 8 shows the CTX results for a low-plasticity silt (ML) assigned No. 27 in Table 2. Fig. 9a shows the relationship between the pore pressure ratio (r_u) in this sample and the number of cycles. For this graph, maximum pore water pressures in each cycle in Fig. 8c have been considered; thus it was possible to trace a continuous curve. In Fig. 9b, the peak-to-peak deformations in each cycle (Fig. 8d) are shown cumulatively and plotted against the number of cycles to achieve clearer visibility of peak-to-peak values.

It is seen that the excess pore water pressure rose in sample No. 27 became equal to the cell pressure in the 6th cycle and the sample reached 5% DSA in the 2.63th cycle, thus the sample was considered to have liquefied in both conditions.

The summary of the dynamic triaxial test results is presented in Table 3. This table gives pore water pressure ratios and double strain amplitude values developed in the 15th cycle for each sample.

As can be seen from Table 3, the pore water pressure ratio and DSA values in the 15th cycle were also examined. This pore water pressure ratio in the 15th cycle was 1.00, and the deformation value in double strain amplitude was 20.688%.

Fig. 10 shows the dynamic test results of low plasticity (No:7-CL) and high plasticity (No:2-CH) clay samples. Results are shown for the first 15 cycles. Accordingly, it can be observed that r_u did not reach 1.0 in both clay samples. However, pore water pressure development in high plasticity clay is minimal compared to low plasticity clay.

Fig. 11 shows the effect of liquid limit on pore pressure ratio and DSA at the 15th cycle. In the evaluation of both criteria, if the liquid limit is <35, r_u is higher than 0.90 in the 15th cycle, resulting in a DSA higher than 5%. In cases where the liquid limit is >45, the pore water pressure does not reach the total stress, and the DSA value is lower than 5% in all cases. In cases where the liquid limit is between 35 and 45, pore water pressures and DSA can both reach or remain below critical limits. This zone is called the transition zone/Gy zone/test zone.

Fig. 12 shows the effect of plasticity index on pore pressure ratio and DSA at the 15th cycle. In the evaluation of both criteria, if the plasticity index is <12, r_u is higher than 0.90, and DSA $\geq$ 5% at the 15th cycle. In

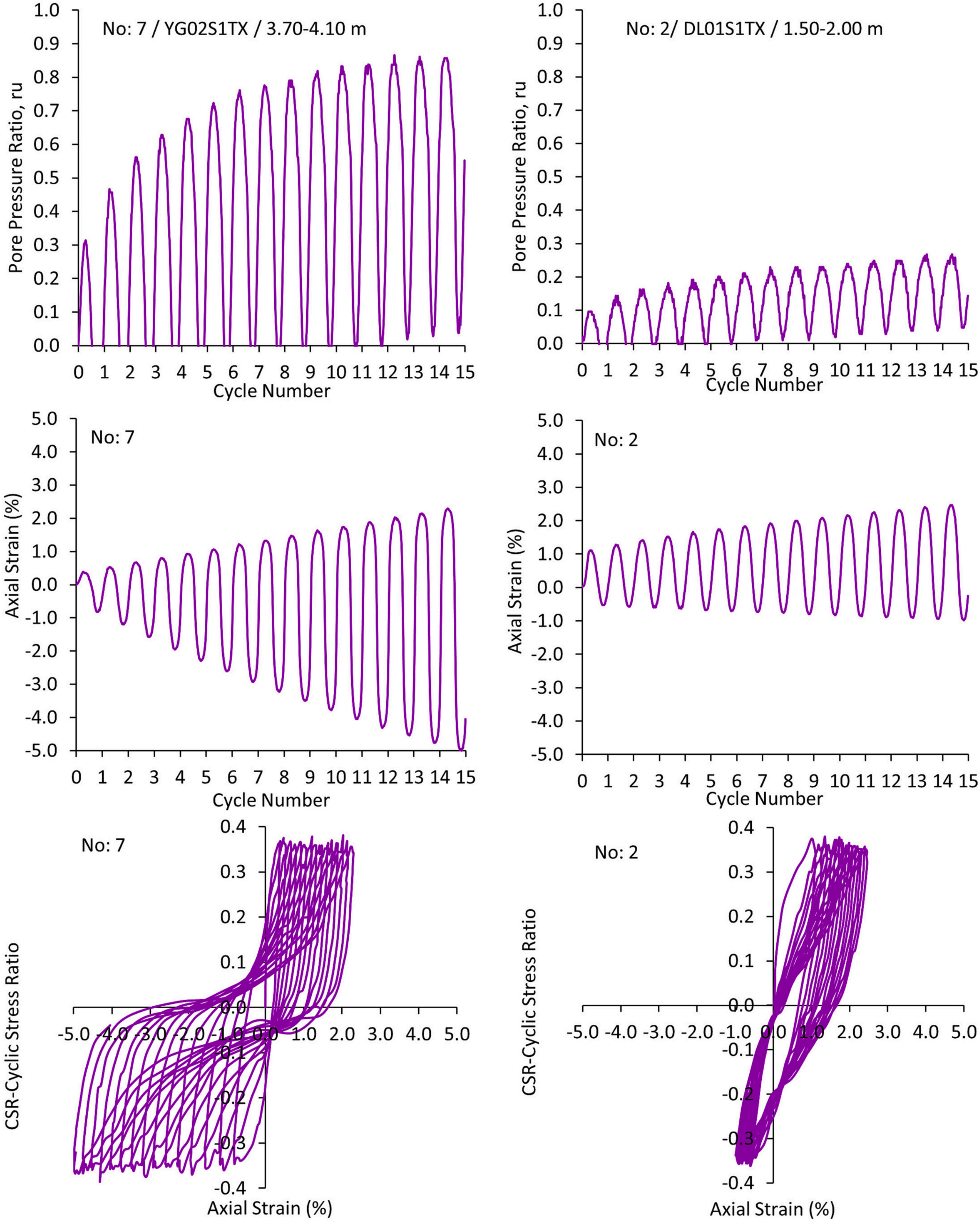

Fig. 10. Cyclic triaxial test results of CL (No: 7) and CH (No:2) clay samples.

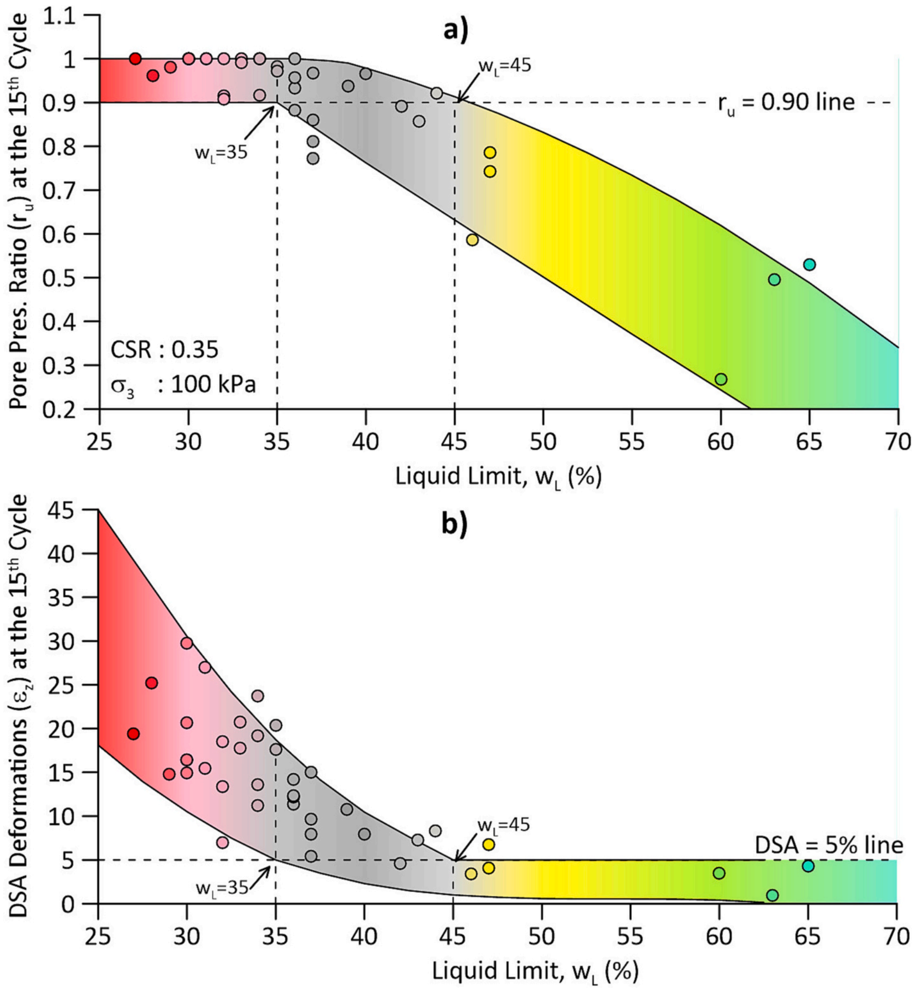

Fig. 11. The effect of liquid limit on pore pressure ratio and DSA at 15th cycle.

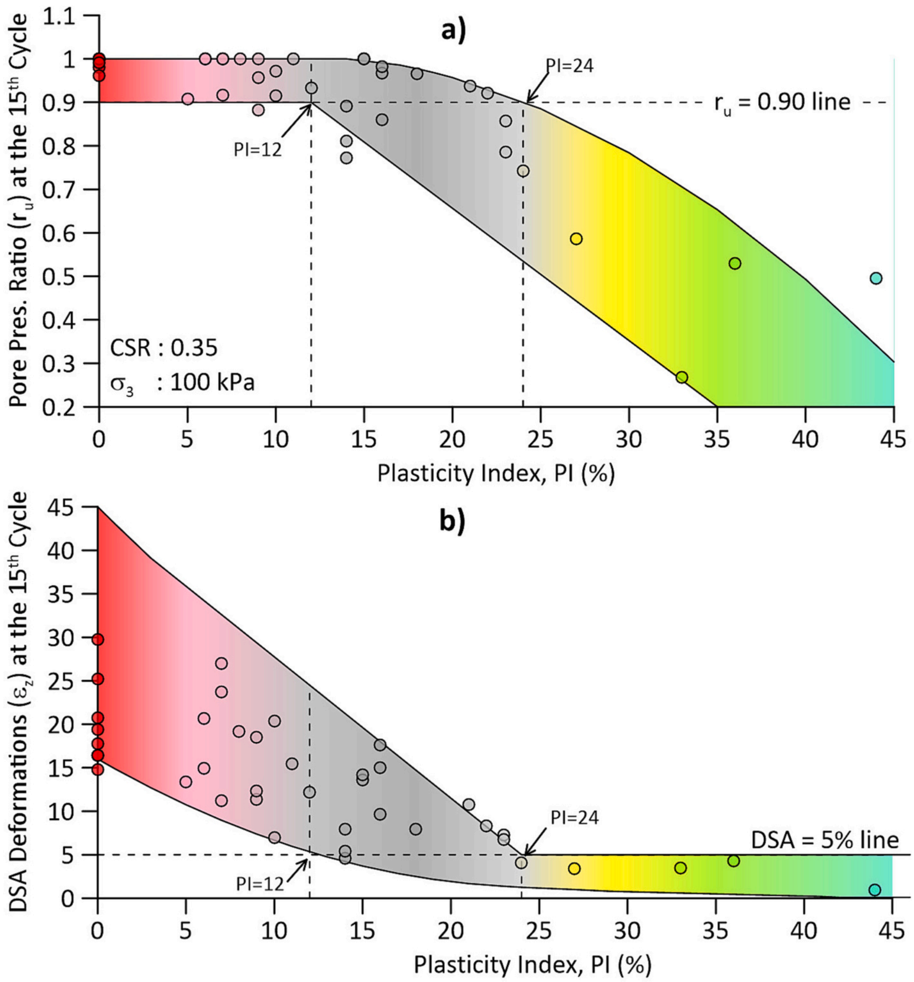

Fig. 12. The effect of plasticity index on pore pressure ratio and DSA at 15th cycle.

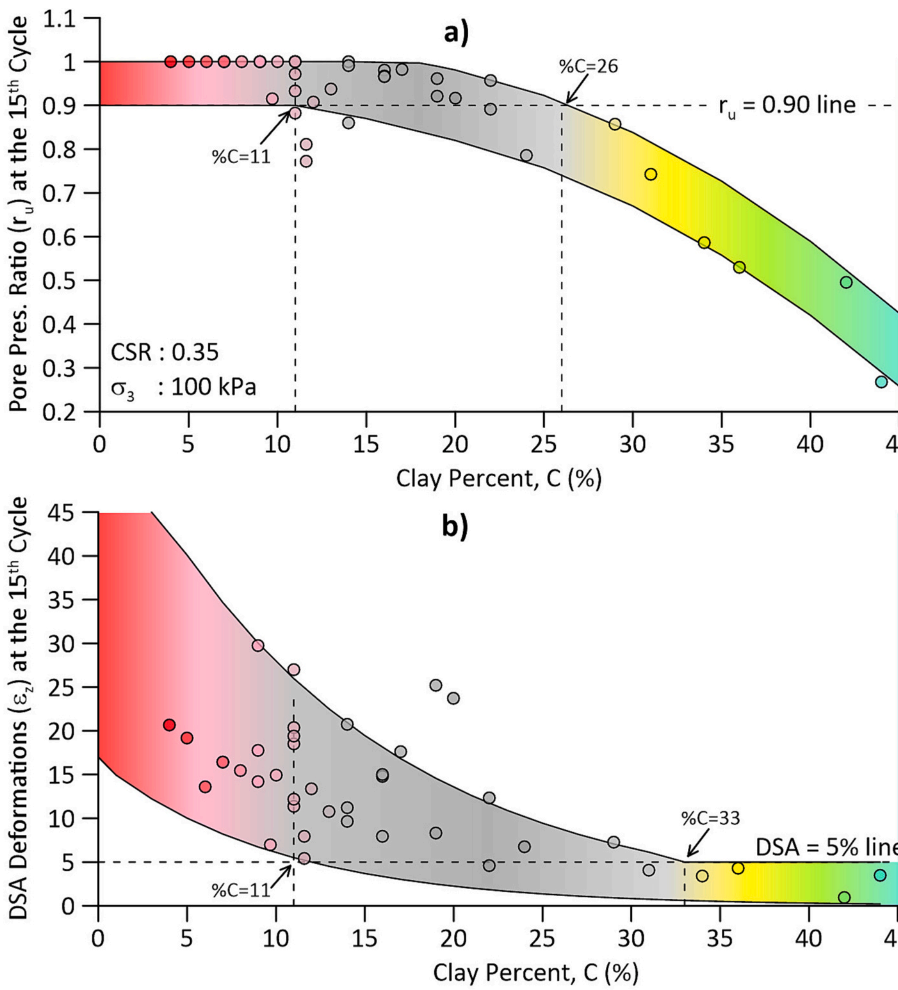

Fig. 13. The effect of clay content on pore pressure ratio and DSA at 15th cycle.

cases where the plasticity index is >24, pore water pressure does not rise sufficiently and DSA is <5%. In cases where the plasticity index is between 12 and 24, pore water pressures and DSA may reach critical limits or fall behind these limits, and remain in the gray zone.

Fig. 13 shows the effect of clay content (<0.002 mm) on pore pressure ratio and DSA at the 15th cycle. In the evaluation made according to both criteria, if the clay ratio is <11, r_u exceeds 90%, and the DSA is higher than 5% at the 15th cycle. According to the pore water pressure criterion, in cases where the clay content is >26, pore water pressure does not reach total stress under any circumstance. On the other hand, if the clay ratio is >33, DSA is smaller than 5%. Considering both criteria, pore water pressures and DSA where the clay ratio is between 11 and 35 in a wide range can both reach critical limits and fall behind and remain in the gray zone. Although pore water pressure ratios do not exceed 0.90 in the range of 36–33, deformations can reach extreme values.

Fig. 14 shows the effect of w_n/w_L ratio on pore pressure ratio and DSA at 15th cycle. In the evaluation made according to both criteria, in every case where the w_n/w_L ratio is >0.9, in the 15th cycle, r_u is higher than 0.90, and double strain amplitude is higher than 5%. In cases where w_n/w_L ratio is <0.7, pore water pressure does not reach total stress in any case, and DSA values are generally <5%. In cases where w_n/w_L ratio is between 0.7 and 0.9, pore water pressures and DSA both reach critical limits and fall behind and remain in the test zone.

Fig. 15 shows the effect of mean grain size (D_{50}) on pore pressure ratio and DSA at the 15th cycle. In all cases, r_u is higher than 0.90 and double strain amplitude higher than 5% at the 15th cycle if the average grain size is >0.02 mm. r_u does not reach unity under any circumstance in cases where the average grain size is smaller than 0.007 mm. On the other hand, DSA is <5% only when the average grain size is <0.005 mm. Considering both criteria, where the average grain size is between 0.005 and 0.02 mm, pore water pressure ratios and DSA can reach both critical limits and fall behind and remain in the gray zone. Although pore water pressure ratios in the range of 0.005–0.007 mm of average grain size do not exceed 0.90, deformations can reach extreme values.

Table 4 shows the ranges of the physical properties indicating liquefaction and non-liquefaction cases. In addition to above results; in dynamic tests especially those conducted on high plasticity soils, pore water pressures may not increase to a critical value even the sample has failed. In these situations, samples may be referred to as samples that have failed due to cyclic softening or simply cyclic failure. The last column of the Table 4 shows the samples in which pore water pressures and double strain amplitude values reached a critical level. The table also shows the Adapazarı Criteria (Bol et al., 2010) values with an asterisk. Evaluating the results of this study, it is seen that some of the threshold physical property values (w_L, PI, %C) are slightly higher compared to the Adapazarı Criteria. On the other hand, w_n/w_L ratio and D_{50} values are slightly lower. It has also been observed that clay ratio (% C) and average grain size (D_{50}) differ in limit values determined for non-liquefiable soils according to r_u and DSA criteria. The reason for the diagnoses by using clay ratio and average grain size being similar is that there is an exponential relationship between clay ratio and grain size (Fig. 16).

In Fig. 17, the DSA at $N = 15$ cycles and the peak excess pore water pressures are represented in the triangular diagram showing the

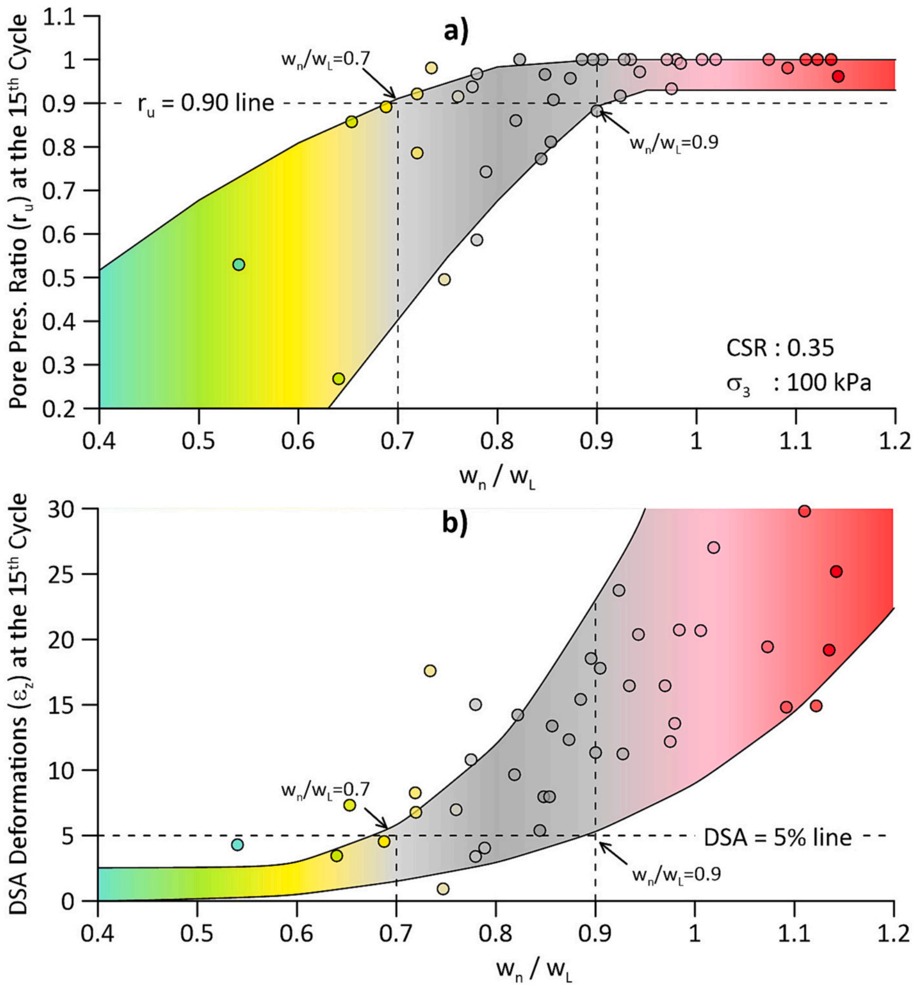

Fig. 14. The effect of w_n/w_L on pore pressure ratio and DSA at 15th cycle.

granulometry of the soils used in this study. Different deformation ranges in Fig. 17b are shown by coloring. Here, a striking finding is that soils showing <5% DSA strain contain almost no sand and >20% clay. This finding coincides with the consensus in the literature that the liquefiable silty soils are of "sandy" character and that it would be appropriate to make a judgment by making use of mechanical experiments for the transition from sandy to clayey, confirming clay-like soils will not liquefy (Idriss and Boulanger, 2006).

Fig. 18 depicts the influence of clay content and the liquid limit on excess pore pressure generated, the primary indicators of cyclic failure. It is possible to evaluate the liquefaction potential defined in the discussion above by following the colored zones. For example, the point for soil sample No.14, which reached failure ($r_u > 0.9$) with $w_L = 32$ and clay content of $C = 11\%$ plots in the red zone. The non-liquefied sample No. 01 ($w_L = 42\%$, Clay Content $= 22\%$) is plotted in the green zone in Fig. 17. It is clear from this figure that the pore water pressure development in fine-grained soils is strongly dependent on the liquid limit and clay content.

6. Conclusion

Excess pore water pressure ratio and DSA value, which are indicators of cyclic failure in silts, were evaluated regarding their physical properties. Dynamic triaxial tests (CTX) were carried out on undisturbed fine-grained samples taken from Adapazarı city center. The findings reached in CTX testing were compared with the Adapazarı Criteria, which was developed based on field observations made following the 1999 earthquake, and the threshold values of the physical properties were determined to reach judgment whether fine-grained soils will arrive at cyclic failure. The following results were obtained in this study:

1. According to the test findings, in cases where the liquid limit is <35, r_u rose above 0.90, and DSA is higher than 5% at the 15th cycle. In cases where the liquid limit is >45, the excess pore water pressure does not reach the total stress, and the DSA value is <5%. In cases where the liquid limit is between 35 and 45, pore water pressures and DSA can both reach or remain below the critical limits. This zone is named the transition zone/Gy zone/test zone.
2. By similar reasoning, it is found that the plasticity index must be <12 for r_u to exceed 0.90 and DSA above 5% at the 15th cycle in all cases. This is interpreted as liquefaction risk being close to zero, where the plasticity index is >24. The interval between PL = 12–24 is defined as the gray zone, the area to be decided with the help of additional dynamic testing.
3. When the liquefaction was evaluated by physical properties of the silts, the threshold values were found as 11% and 33% for the clay ratio (<0.002 mm), and 0.7 and 0.9 for the w_n/w_L ratio. In addition, it has been found that the average grain diameter (D_{50}) is also an important parameter in determining liquefiability. In cases where D_{50} is smaller than 0.007 mm, liquefaction risk does not appear, and it becomes certain when it is >0.02 mm. Likewise, the gray zone is between these two values.
4. It can be stated that evaluation for liquefiability of silt can be carried out by the use of all five physical properties liquid limit, plasticity

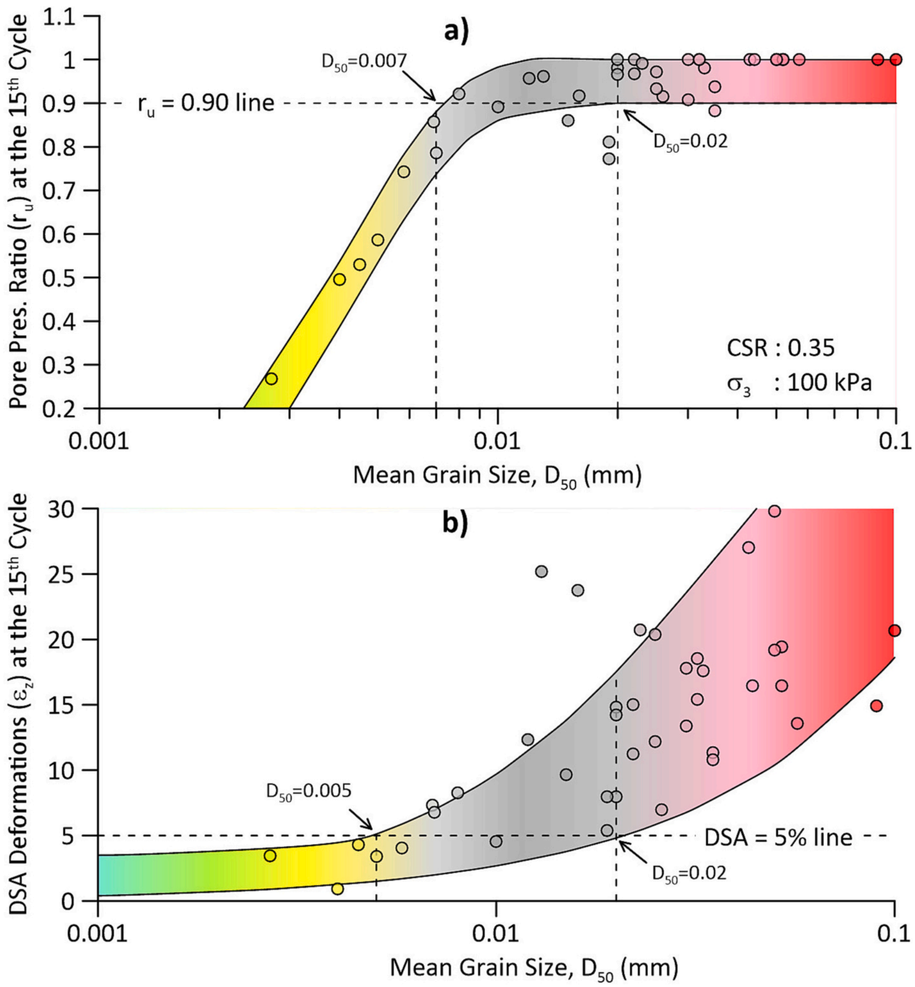

Fig. 15. The effect of mean grain size (D_{50}) on pore pressure ratio and DSA at 15th cycle.

Table 4
Limit values of physical properties of soils for evaluation of liquefiability.

	Non Liquefied		Gray (Gy) Zone	Liquefied	
	r_u	DSA		r_u	DSA
Liquid Limit, w_L (%)	$w_L > 45$ $w_L > 33$*	$w_L > 45$	$45 > w_L > 35$ $33 > w_L > 25$*	$w_L < 35$ $w_L < 25$*	$w_L < 35$
Plasticity Index, PI (%)	PI >24	PI >24	$24 > w_L > 12$	PI <12	PI <12
Clay Content, C (%)	C > 26 C > 15*	C > 33	33 > C > 11 15 > C > 10*	C < 11 C < 10*	C < 11
w_n/w_L ratio	$w_n/w_L < 0.7$ $w_n/w_L < 0.9$*	$w_n/w_L < 0.7$	$0.9 > w_n/w_L > 0.7$	$w_n/w_L > 0.9$ $w_n/w_L > 0.9$*	$w_n/w_L > 0.9$
Mean Grain Size, D_{50} (mm)	$D_{50} < 0.007$ $D_{50} < 0.02$*	$D_{50} < 0.005$	$0.02 > D_{50} > 0.005$ $0.06 > D_{50} > 0.02$*	$D_{50} > 0.02$ $D_{50} > 0.06$*	$D_{50} > 0.02$

* Adapazarı Criteria (Bol et al., 2010) values.

index, average grain size, clay content, and water content ratio. Limit values for the physical properties are summarized in Table 4. According to the table, although pore water pressures could not fully develop in all samples falling into the gray zone, it was observed in almost all criteria except w_n/w_L that the DSA values were above the critical level (DSA > 5%) in that zone. Thus, it has been concluded that these soils may be subject to cyclic failure even if they do not liquefy.

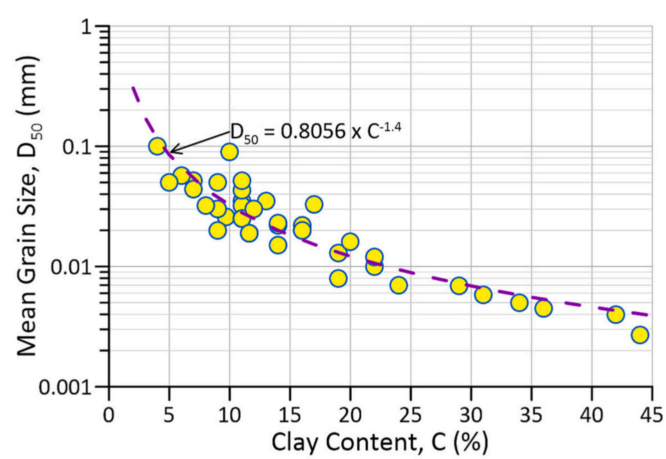

Fig. 16. Relation between average grain size and clay content.

5. As a result of dynamic triaxial testing, it has been an important finding that the soils showing <5% DSA contain almost no sand and >20% clay, which supports ~~is in~~ the findings in the literature.

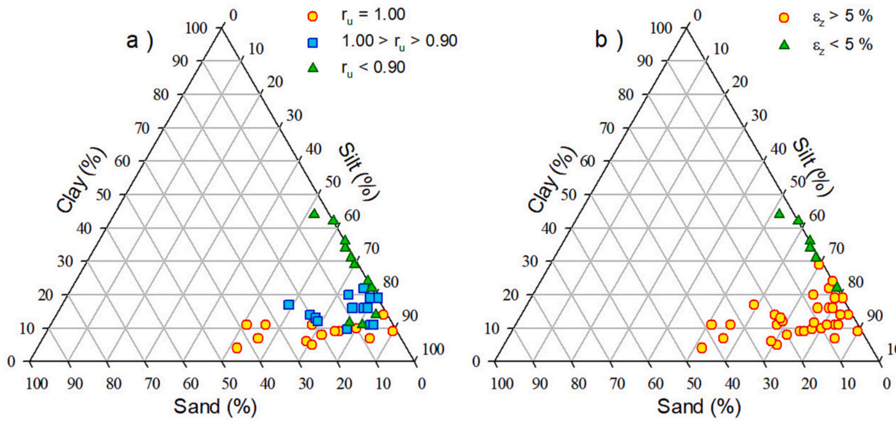

Fig. 17. a) Pore pressure ratio (r_u) and DSA (ε_z) at 15th cycle.

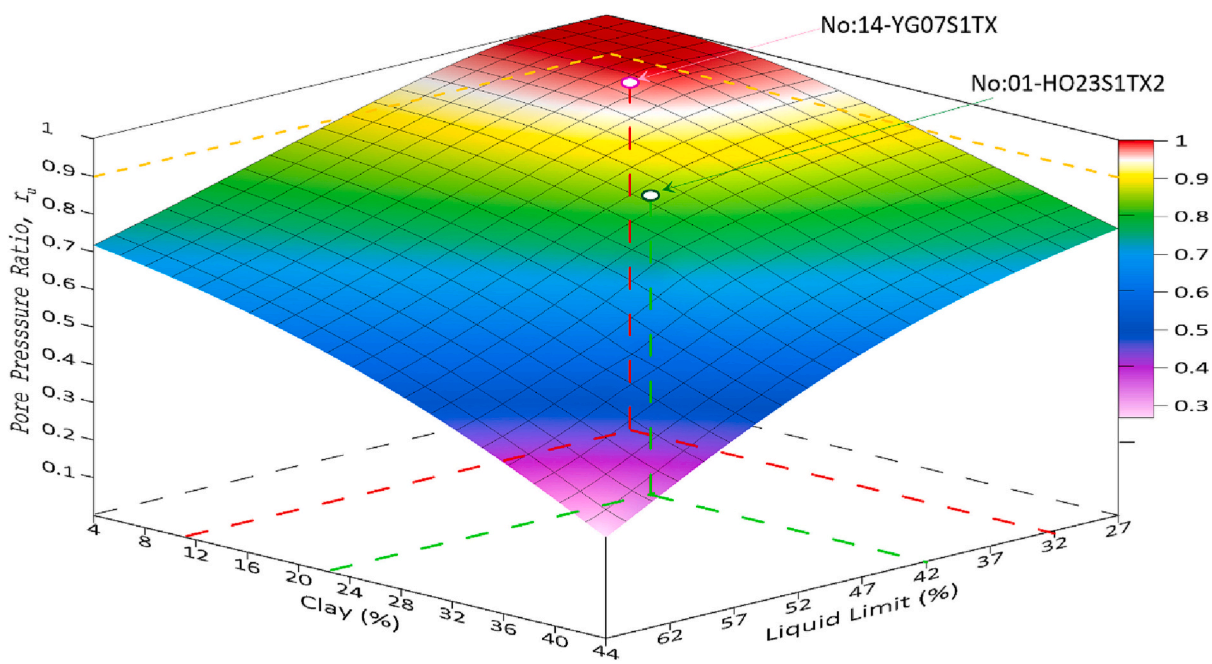

Fig. 18. The influence of clay content and liquid limit on excess pore pressure generated.

Declaration of Competing Interest:

None.

Data availability

Data table is shared in the manuscript.

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