



# An investigation into dynamic behaviour of reconstituted and undisturbed fine-grained soil during triaxial and simple shear

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## Abstract

This study aims to evaluate the factors controlling the sensitivity of fine-grained soils to seismic stresses and revise the criteria previously proposed by the authors to diagnose liquefaction. To this end, dynamic tests have been performed on artificial mixes as well as natural soils from a wide area of an earthquake devastated city (Adapazari) using two types of dynamic testing. Studies have led to findings suggesting that the gray area between susceptible and non-susceptible soils proposed by several investigators in the past can now be dispensed with. Although physical properties of fine-grained soil supply sufficient information for diagnosis, the dynamic simple shear test is found to be a convenient and rapid way to confirm the judgement. However, it has been seen that dynamic testing alone may not be the last word in the determination of liquefaction, and physical properties should also be addressed. Anomalies observed in test results are also discussed. Conclusions show significant differences from existing proposed criteria in the literature.

**Keywords** Ground failure · Liquefaction · Reconstituted sample · Undisturbed sample · Consistency · Clay content · Average size · Dynamic triaxial test · Direct simple shear test · Regression · Adapazari Criteria

## Abbreviations

CF	Clay fraction
CH	High plasticity clay
CI	Intermediate plasticity clay
CL	Low plasticity clay
CPT	Cone penetration test
CPTu	Cone penetration test with pore water pressure measurement
CSR	Cyclic stress ratio
CTX	Cyclic Triaxial
D <sub>50</sub>	Average particle size

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DSA	Double strain amplitude
DSS	Dynamic simple shear
$f$	Loading frequency
GWL	Groundwater level
$I_L$	Liquidity index
$I_p$	Plasticity index
MH	High plasticity silt
MI	Intermediate plasticity silt
ML	Low plasticity silt
$M_w$	Earthquake magnitude
NP	Non-Plastic
N	Number of loading cycle
NL	Normally loaded (fully consolidated)
$N_{uwmax}$	Cycle number at peak pore pressure
OCR	Overconsolidation ratio
$r_u$	Pore water pressure parameter
$r_{u,res}$	Residual pore water pressure ratio
$r_{u,res,N15}$	Residual pore water pressure ratio at 15 cycle
SM	Silty sand
SPT	Standard penetration test
$S_r$	Degree of saturation
UD	Undisturbed
$u_{w,max}$	Peak excess pore water pressure
$u_{w,res}$	Residual pore water pressure
$w_f$	Final water content
$w_n$	Natural water content
$w_L$	Liquid limit
$w_{L,cone}$	Liquid limit by cone
$w_{L,perc}$	Liquid limit by percussion (Casagrande)
$\gamma$	Shear deformation
$\varepsilon_z$	Axial deformation
$\sigma_c$	Preconsolidation pressure
$\sigma_n$	Normal stress
$\sigma_{n,DSS,ult}$	Ultimate normal stress in DSS
$\tau$	Shear stress

## 1 Introduction and background

The issue of the behaviour of fine-grained soils subjected to seismic loads is controversial. Although some evidence concerning primary liquefaction and the cyclic mobility of silty/clayey soils have been presented in the literature (Ross et al. 1969), some observers remain skeptical to the failure of ground profiles dominated by fine-grained soil during earthquakes. In particular, possibility of triggering primary liquefaction in such soils is often questioned.

The frequently referred to Chinese Criteria on this subject for evaluating silts has been widely criticized because majority of soils used for identifying liquefaction potential were in fact clays of low plasticity (Bray and Sancio 2006). However, it will be seen in the

following sections that the results of this study have confirmed the vulnerability of low plasticity clays to dynamic failure as well as silts.

The study presented in this paper emanates from the observations of the authors during and after the destructive earthquakes in the city of Adapazari, Turkey in 1999. The results of ensuing research programs conducted on the topic are reported here (Önalp et al. 2007, 2010).

It is estimated through surface observations and subsequent laboratory testing that the ground in about 25% of the total area of the city indicated evidence of failure as depicted in Fig. 1 (Bol et al. 2007). The vulnerable soil profiles were predominantly nonplastic silts (ML) and occasional silty sands (SM). The silts were found to contain up to 30% bentonitic clay. The amount of clay depends on the speed and the duration of the spring floods of the river Sakarya which inundated the city almost biannually and remained in the form of lagoons and marshes for months. An early XIXth century traveler described the scene (Texier 1882), citing water buffaloes immersed in large slurry pools on both sides of the road as he was entering the town. Two large dams were constructed upstream in the late sixties which ended flooding. Subsequently, desiccation of the top layers in the profiles throughout hot summers has been causing increasing overconsolidation effects for the past fifty years.

## 2 The scope

This paper gives an account of the results of two complementary research programs aimed at understanding the fundamental features of liquefaction in silty soil. The initial study was performed by determining the measurement of the dynamic properties of Adapazari silt reconstituted by mixing it with different percentages of bentonite and kaolinite. Its aim was to understand the influence of the type of clay mineral on its dynamic behavior (Arel et al. 2018). The data obtained during testing of the ‘artificial’ samples were then used to interpret the failure mechanism in natural soil.

Towards this aim, dynamic properties of the natural soils sampled (UD) strategically throughout the city at 57 locations were determined by laboratory testing while performing

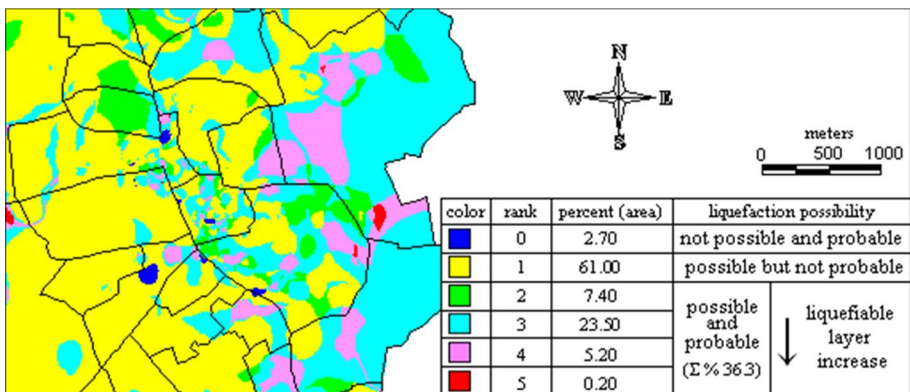


Fig. 1 Ground failure map of Adapazari (black lines delineate administrative districts)

simultaneous in-situ tests (SPT, CPTu) at these locations. This paper intends to present a compendium of two research programs.

### 3 Seismic ground failure

Primary liquefaction and cyclic mobility are two main modes of ground failure observed during earthquakes. In the early stress-based approach initial liquefaction, nowadays referred to as cyclic mobility was described to be due to decrease in effective stress down to zero as a result of rapid rise in porewater pressure (Kramer 1996). That is, the residual strength of the soil is higher than that needed to maintain static equilibrium. The alternative approach was to adopt the strain-based model where soil deforms excessively, hence the name ‘flow liquefaction’. In this case the residual strength of loose or soft soil is lower than that needed to maintain static equilibrium (Kramer and Elgamal 2001). Such attributes may have scientific and academic value, but it is often not possible to differentiate between the two events in the field. Attempting to simulate such field conditions by means of short-term laboratory dynamic testing thus deserves scrutiny. However, the possibility of measuring excess pore pressures in laboratory experiments adds value to this process. Because those generated during a laboratory test are quite similar to those recorded in-situ during an earthquake.

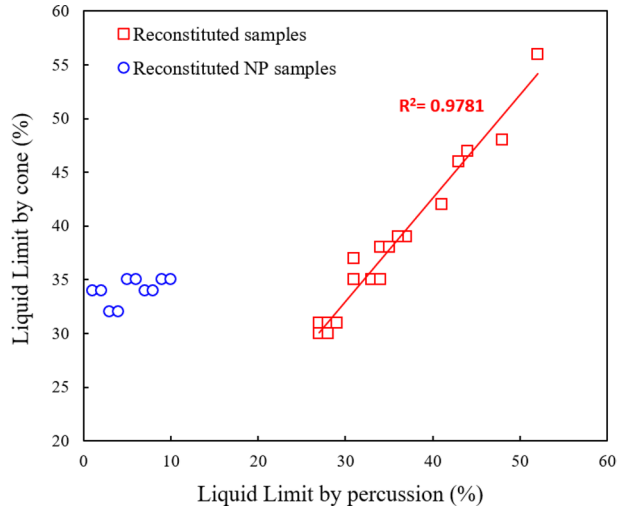
### 4 The method

A drilling program of 15 CPT soundings and 15 boreholes was implemented to sample soils from several districts of Adapazari where ground failure was observed as well as those clayey silt and clay sites where no ground failure had been reported. The results of the simultaneous dynamic triaxial (CTX) and simple shear (DSS) tests on the UD samples were then analyzed with reference to the physical/mechanical properties of the soils to define failure conditions. The findings of the previous mineralogical study conducted on reconstituted samples of Adapazari silt were used as reference to establish upper and lower ‘envelopes’ to gain a better understanding of the behaviour of natural soils during shaking (Donahue et al. 2007). The rise of excess porewater pressures and ensuing deformations were then evaluated to prognose failure.

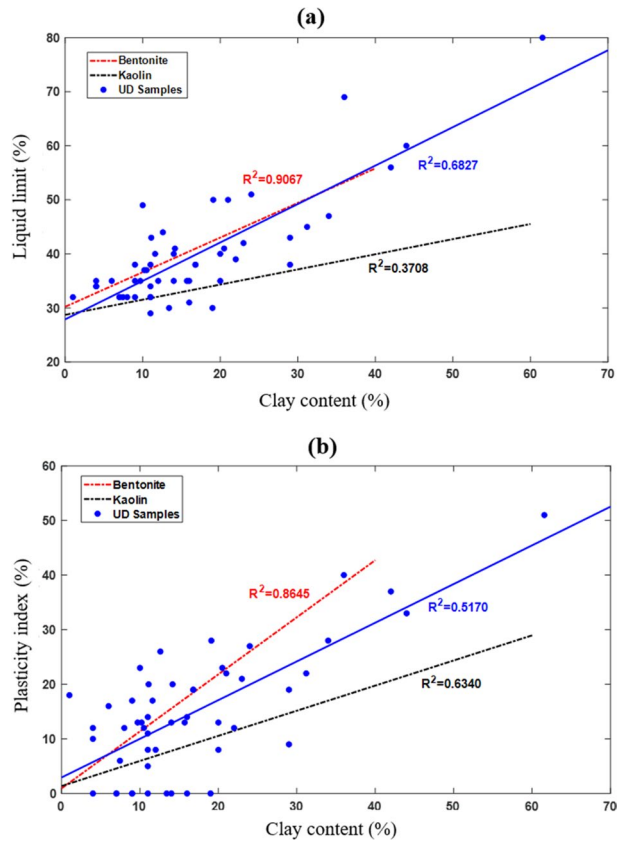
### 5 The physical properties of the silt

Figure 2 shows the relation between liquid limit from fall cone test and percussion test. Figure 3a and b illustrate the consistency of the natural samples (points) compared with those of the reconstituted samples that were cleaned of their clay component and blended with kaolinite and bentonite (lines). The activity of montmorillonite imparts high plasticity to the silt. Almost double the plasticity index ( $I_p$ ) value was measured with bentonite samples compared to those of kaolinite at a clay fraction (CF) of 15%, the typical CF value for natural Adapazari silt. The full blue line representing natural Adapazari soils plots between the two lines developed for the reconstituted samples albeit closer to the bentonite line. This is possibly due to their similar clay mineral contents. The natural samples exhibit significantly lower  $R^2$  values (0.68) due to their different sand contents, percentage and type of

**Fig. 2** The liquid limit values from both tests for reconstituted samples



**Fig. 3** The influence of clay content on the consistency of reconstituted and natural Adapazari soils **a** Liquid limit by fall cone. **b** Plasticity index (points represent natural samples)



clay components, whereas the artificial mixes contain no sand and have precise percentages of single clay mineral. This has caused the scatter in the plotted points. Liquid limit ( $w_{L,conc}$ ) was measured with the fall cone apparatus. Because this test always provides a result whereas it cannot be performed for samples of low plasticity in the Casagrande percussion device ( $w_{L,perc}$ ), resulting in NP (non-plastic) sign. It has also been emphasized in previous studies that fall cone test provides well-correlated parameters with other soil parameters (Monkul et al. 2020). The relationship was determined by testing more than 100 samples as

$$w_{L,conc} = 0.967w_{L,perc} + 4.14 \quad (1)$$

with an  $R^2$  of 0.98. The relationship is obtained from the linear function of data shown in Fig. 2.

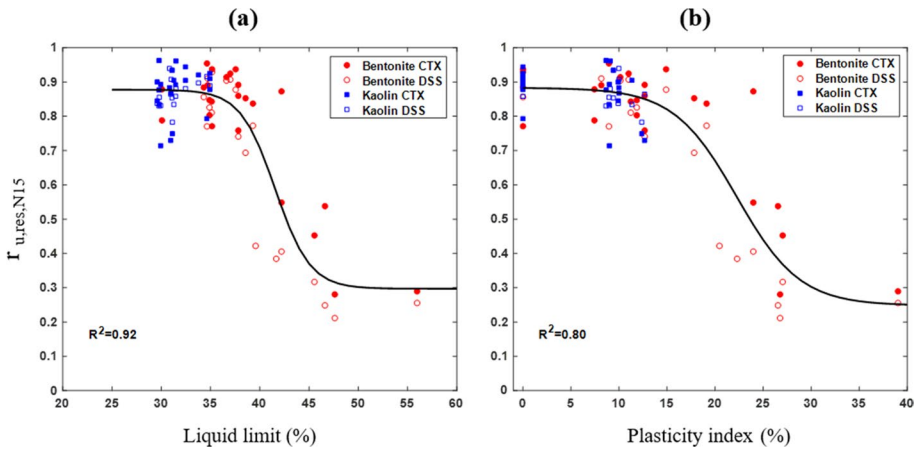
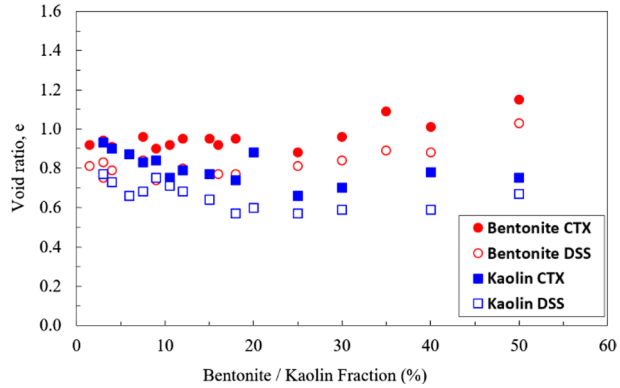
## 6 Testing reconstituted samples

This initial part of the study on reconstituted samples was implemented to support and evaluate the findings of the study to be carried out on UD samples. Adapazari silt contains 5–30 percent clay, which is mainly montmorillonite (Donahue et al. 2007). 200 kg sample was collected from a liquefaction site to prepare its various blends. It was thoroughly mixed with excessive amount of water in a tank and the supernatant drained after 30 min to remove the clay fraction. The process was repeated until most of the clay component was eliminated. The washed silt was then air dried and mixed with increasing percentages of bentonite and kaolinite. The samples mixed with bentonite classified as CI (intermediate plasticity clay) through CH (high plasticity clay). Those samples mixed with kaolinite indicated symbols ML (low plasticity silt) and CL (low plasticity clay). This suggested that the kaolinite samples might eventually represent the mixes that are prone to liquefaction, and bentonite samples constituting the resistant group.

Reconstituted samples were prepared using the slurry deposition method to be representative of in-situ conditions. Wet pluviation is recommended mostly for sands (Bradshaw and Baxter 2007), while the slurry method is recommended for silts which is known to provide homogeneous samples in (Krage et al. 2020). In addition, this is one of the methods that best simulates the sedimentation process of Adapazari soils. Silt–clay mixtures were proportioned by weight and the slurry samples were slightly vacuumed to dispose of the air and excess water. The slurry was then poured into consolidation cylinders. All reconstituted samples were consolidated to a pressure ( $\sigma_c$ ) of 100 kPa (Ishihara 1993), also representing the characteristic depth of liquefaction in Adapazari and then tested in triaxial as well as simple shear conditions at a frequency  $f=0.5$  Hz. and cyclic stress ratio (CSR) 0.35 at this  $\sigma_c$ .

Selection of the dynamic test parameters was done considering several factors. The average CSR value for the Adapazari earthquake ( $M_w=7.5$ ) of 0.35 was computed by the Seed and Idriss (1982) equation for depths of less than 8 m and a ground water depth of around 2 m. Additionally, the studies performed by Guo and Prakash (1999) and Puri (1984) on the dynamic properties of silts were taken into consideration which pointed out that for soils with plasticity index between 10–12, cyclic failure is reached at CSR=0.35 and 15th cycle. Puri (1985) indicated that the CSR and double strain amplitude (DSA) values required for failure of soils with a plasticity index between 11–18 is between 0.30–0.40 and 5%, respectively.

**Fig. 4** The consolidated void ratio range of reconstituted samples



**Fig. 5** Behaviour of reconstituted Samples **a** Liquid limit. **b** Plasticity index

The change in consolidated void ratios of the reconstituted samples is given in Fig. 4. Although the void ratios remained within a narrow band, slight increases were observed as the clay ratio (CF) increased.

Excess ‘peak’ pore pressures ( $u_{w,max}$ ) generated during a test are instantaneous measurements of response and are directly influenced by the deviatoric stresses. To eliminate stress induced effects ‘residual’ pore pressure ( $u_{w,res}$ ) value is used. It is defined as the value of pore water pressure when deviator stress crosses the zero level (Idriss and Boulanger 2006). Its value may differ up to 30% from that of  $u_{w,max}$  depending on the type of soil.

The results were plotted as residual excess pore pressure ratios ( $r_{u,res}$ ) as well as peak values against liquid limit, plasticity index, clay fraction, in-situ water content and average size of the samples. As seen in Fig. 5, an overwhelming number of failed samples plotted above an  $r_{u,res}$  of 0.7 which was defined as the ‘liquefaction line’. The line intersects the curve corresponding to a liquid limit of 40 (~37 for percussion). The plasticity index was also a reasonable indicator of failure at  $I_p < 19$  but with a smaller  $R^2$  of 0.8. It is noteworthy that all kaolin blends failed, regardless of their clay content ( $CF \leq 2 \mu m$ ).

This ‘calibration’ using homogeneous and samples of prescribed composition showed that liquefaction initiates at a residual pore pressure ratio of 0.7 corresponding to a liquid

limit of 40. The kaolinite samples which exhibit low plasticity, reached failure conditions invariably at  $N \leq 20$  loading cycles whereas the bentonite samples showed remarkable resilience and most blends did not fail readily, despite the fact that all samples were tested in normally loaded state.  $N = 15$  cycles was preferred to indicate failure because it is representative of a quake of  $M_w = 7.5$  magnitude experienced in Adapazari.

### 7 Tests on undisturbed (natural) samples

The samples were tested in a cyclic triaxial system (CTX) manufactured by Wykeham Farrance in accordance with ASTM D5311 and in the dynamic simple shear device (DSS) manufactured by Geocomp complying with ASTM D6528 conditions. Same equipment and testing methods were used for all samples (Önalp et al. 2010). Results were listed in a table comprising all physical and dynamic findings. They will be provided to researchers wishing to utilize them in their studies. Table 1 shows 17 characteristic results from the database of 57 UD samples In the Table 1, soil classes are written with bold to draw attention to them, and samples belonging to the same soil class are filled with the same color.

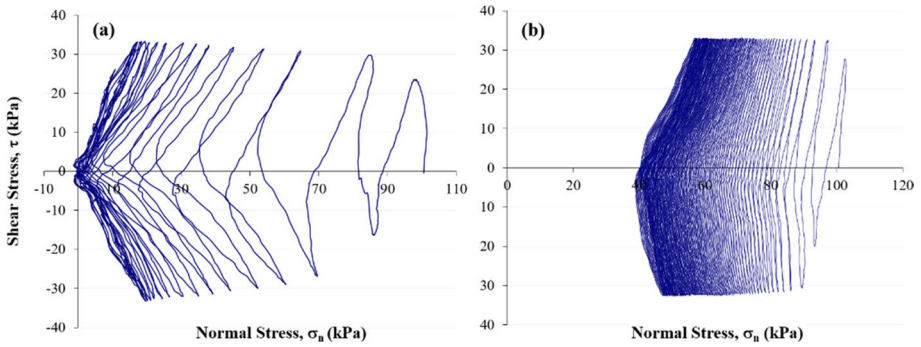
It can be seen from Columns 1 and 2 that almost all the UD samples were collected from below the groundwater table. Column 3 lists the number of cycles required to reach peak porewater pressure during CTX and DSS tests. Maximum and residual porewater pressures recorded are listed in Column 4 and Column 5.

An interesting feature of the DSS test is the progress of the vertical stress measured as the number of cycles increases. The  $\sigma_{n,DSS}$  decreases steadily with the number of cycles because the porewater pressure is rising. It was found that for a sample which liquefied

**Table 1** Characteristic properties of some Adapazari soils (UD samples)

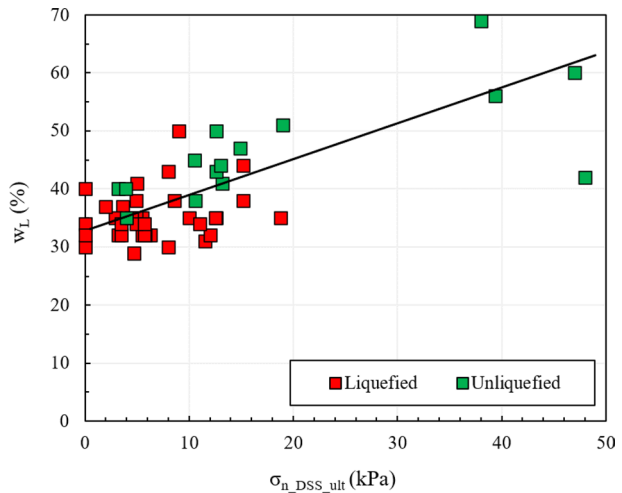
0	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15
Sample Code	Depth (m)	GWL (-m)	N <sub>uw,max</sub> (cycles)	u <sub>w,max</sub> (kPa)	u <sub>w,res</sub> (kPa)	σ <sub>n,DSS,ult</sub> (kPa)	w <sub>L,conc</sub> (%)	I <sub>p</sub> (%)	I <sub>L</sub>	S <sub>r</sub> (%)	CF (%)	CLASS	σ <sub>c</sub> (kPa)	OCR	LIQ?
GN01S2	3.50	2.60	36 CTX 33 DSS	98 91	87 88	2	37	13	2.3	100	10	<b>SM</b>	NL	1	+
YH01S1	2.45	1.39	4 CTX 32 DSS	100 90	82 86	4.9	34	10	0.5	92	4	<b>SM</b>	NL	1	+
YG01S1	5.15	1.10	67 CTX 63 DSS	74 87	51 83	0.0	30	NP	1.1	100	13	<b>ML</b>	420	6.8	+
YG02S1	4.70	2.00	87 CTX 9 DSS	93 101	73 100	0.0	34	NP	1.0	100	4	<b>ML</b>	175	2.9	+
YA02S1	3.50	1.88	81 CTX 20 DSS	99 89	86 84	0.0	40	13	1.3	100	14	<b>ML</b>	320	6.5	+
YG05S1	1.50	1.65	3 CTX 15 DSS	100 92	93 90	3.2	32	NP	1.0	96	7	<b>ML</b>	170	6.3	+
AO01S1	5.00	1.50	3 CTX 45 DSS	100 83	91 82	3.6	37	12	0.3	97	11	<b>ML</b>	310	5.3	+
YG03S1	2.50	1.87	50 CTX 22 DSS	99 98	77 95	5.5	32	5	1.1	98	11	<b>MI</b>	-	-	+
YA01S1	2.00	2.22	50 CTX 46 DSS	100 52	85 49	15	39	12	1.2	100	22	<b>MI</b>	NL	1.0	+
YG02S1	8.30	2.00	14 CTX 23 DSS	100 90	92 85	5.8	32	18	0.8	76	1	<b>CL</b>	200	1.6	+
TK01S1	3.85	2.6	10 CTX 17 DSS	104 96	81 94	4.9	35	16	1.1	100	6	<b>CL</b>	390	6.7	+
HO23S1	3.70	1.55	36 CTX 6 DSS	80 98	67 100	5.0	41	23	0.6	84	21	<b>CI</b>	-	-	+
YG01S1	3.30	1.10	74 CTX 100DSS	59 39	42 38	14.9	47	28	0.7	100	34	<b>CI</b>	198	3.9	-
YH01S1	1.95	1.39	14 CTX 28 DSS	100 86	87 85	8.6	38	17	0.9	92	9	<b>CI</b>	-	-	+
TK03S1	4.50	2.10	90 CTX 100DSS	53 33	35 31	38	69	40	0.3	100	36	<b>CH</b>	140	2.4	-
GN01S2	5.90	2.60	172 CTX 94 DSS	50 34	24 32	39.4	56	37	0.7	100	42	<b>CH</b>	-	-	-
TK03S1	5.50	2.10	5 CTX 23 DSS	100 89	87 86	9.0	50	22	0.3	93	21	<b>CH</b>	210	3.1	+

GWL: Groundwater level; N<sub>uw,max</sub>: cycle number at u<sub>w,max</sub>; u<sub>w,max</sub>: max. pore pressure; u<sub>w,res</sub>: residual pore pressure; σ<sub>n,DSS,ult</sub>: ultimate normal stress in DSS; w<sub>L,conc</sub>: liquid limit by fall cone; I<sub>p</sub>: plasticity index; I<sub>L</sub>: liquidity index; S<sub>r</sub>: degree of saturation; CF: clay fraction; σ<sub>c</sub>: preconsolidation pressure; OCR: overconsolidation ratio; LIQ?: judgement on liquefaction; NL: normally loaded



**Fig. 6** The progress of the measured normal stress during DSS test **a** ML silt. **b** CH clay

**Fig. 7** Ultimate normal stresses from DSS tests



readily, the normal stress rapidly dropped to zero, whereas a sample which did not fail sustained the normal stress at around 40 kPa (Fig. 6) regardless of the number of cycles. This is an indication of dropping effective stresses and was employed as an extra tool to check on failure. Column 6 in Table 1 lists these ultimate values.  $\sigma_{n,DSS,ult}$  may be viewed as the inverse of the pore pressure, but more sensitive to changes in the effective stress. Using the critical liquid limit value of 40, it is tentatively proposed from here the relation in Fig. 7 that the samples showing ultimate normal stress of  $\sigma_{n,DSS,ult} < 15$  kPa are very likely to have liquefaction potential.

Columns 7, 8, 9, 10, 11 and 12 describe the physical properties and class of the samples. The measured degrees of saturation in Column 10 have remained conspicuously below 100% at the end of a test for several samples. This finding is significant because all samples were initially saturated by a back pressure of 500 kPa in CTX or fully submerged in DSS to ensure  $S_r \cong 1$ . Such results may be an implication of cavitation developing during a test in silty soil.

The preconsolidation pressure values (Casagrande) from the oedometer tests ( $\sigma_c$ ) in Column 13 confirm that several samples have undergone heavy preconsolidation (Column 14). Dadashiserej et al. (2023) noted that the cyclic characteristics of intact silt

samples are more sensitive to OCR than reconstituted ones. Jana and Stuedlein (2021) emphasized that intact natural silts are highly affected by OCR and intact and reconstituted OC silts show different cyclic resistances. Izadi et al. (2008) indicated that the behavior of low plasticity silts is quite dependent on the level of overconsolidation. At an OCR of 2.0, they stated that the silts became dilative, negative pore pressures developed, and the specimens gained resistance against liquefaction.

UD samples in the current study indicated strikingly high and variable OCR values. Moreover, no relationship between the generated pore pressure and OCR was indicated contradicting the results reported in the literature. Figure 8 illustrates the findings. High values of OCR measured on several samples down to 10 m depth can be attributed to the mode of overconsolidation of these young sediments, which have undergone extensive desiccation. The peculiar behaviour of silts challenges the researcher once again.

It is generally assumed that clays of high plasticity do not tend to liquefy under normal conditions. An unexpected result came from one CH sample (TK03S1) that failed in both tests showing that dynamic testing alone may not provide the correct answer all the time and additional check on physical properties would be prudent.

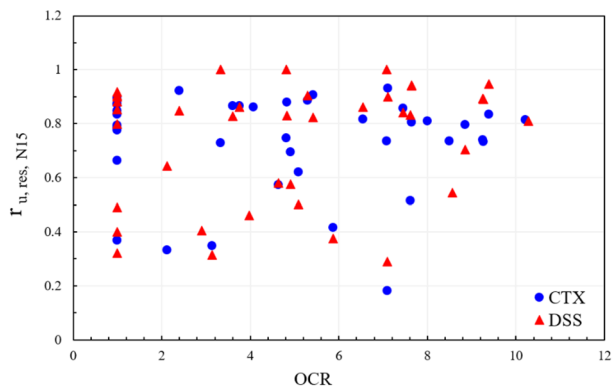
Figure 9 illustrates the response of all the natural samples during dynamic testing. The logistic regression was performed by using the sigmoidal curve of the form;

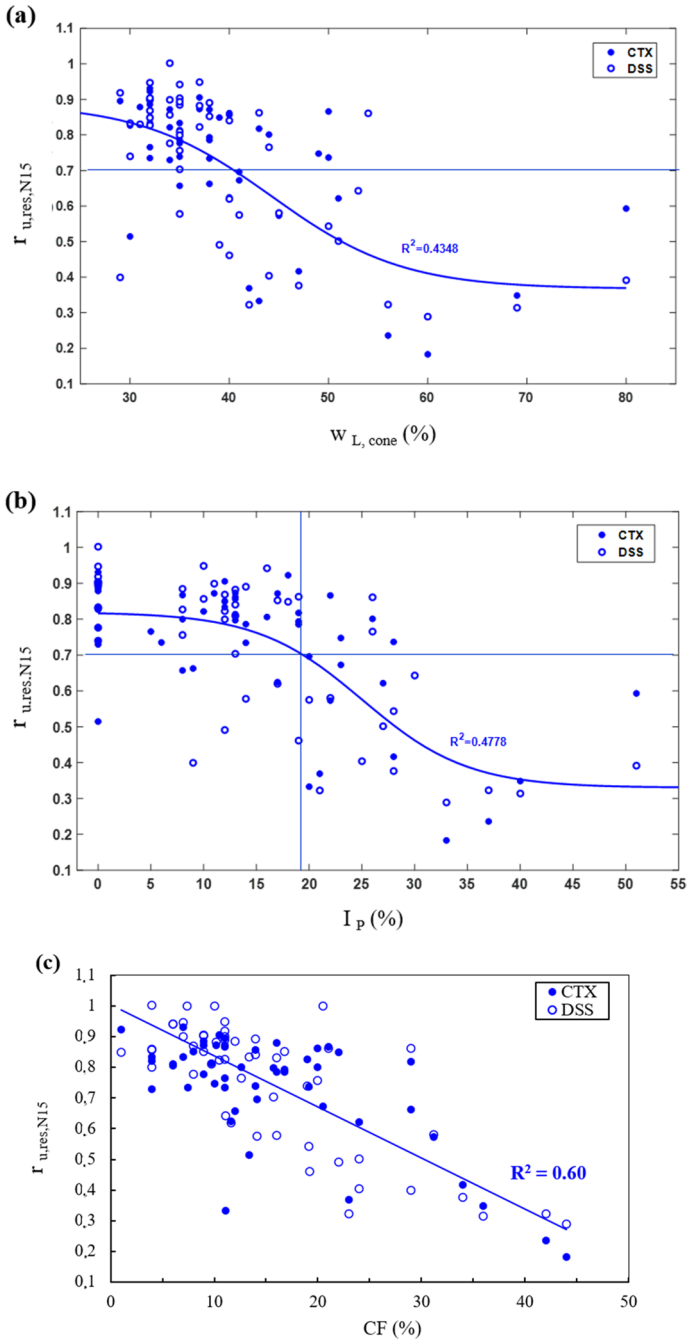
$$y = y_0 \frac{a}{1 + e^{-\left(\frac{x-x_0}{b}\right)}} \tag{2}$$

which was found to reasonably represent the relationship between consistency and residual porewater pressure. In this sigmoidal equation form,  $y$  corresponds to  $r_{u,res}$ , while  $x$  is the consistency limit with which the relationship is intended to be established.  $x_0$ ,  $y_0$ ,  $a$  and  $b$  are the curve fitting parameters. If  $r_{u,res} = 0.7$  is taken as the onset of failure, a liquid limit of  $w_{L,cone} = 40\%$  and a plasticity index of  $I_p = 19\%$  corresponding to a clay content  $CF = 18\%$  emerged to be the limits of failure. However, unlike reconstituted mixtures relatively low  $R^2$  values of around 0.5 were observed. This is to be expected, because the natural samples tested varied from silty sands to fat clays, as well as having different liquidity indices ( $I_L$ ) and overconsolidation ratios (OCR).

An interesting coincidence appeared when the curves for the artificial mixes and natural samples were superposed. Figure 10 illustrates this feature. It can be observed that the intersection point indicates  $r_{u,res} = 0.7$  and  $w_{L,cone} = 40\%$ , establishing a reference for

**Fig. 8** The Effect of Overconsolidation Ratio (OCR) on excess pore pressures





**Fig. 9** The effect of consistency and clay content on excess pore pressures generated **a** liquid limit. **b** plasticity index. **c** clay content

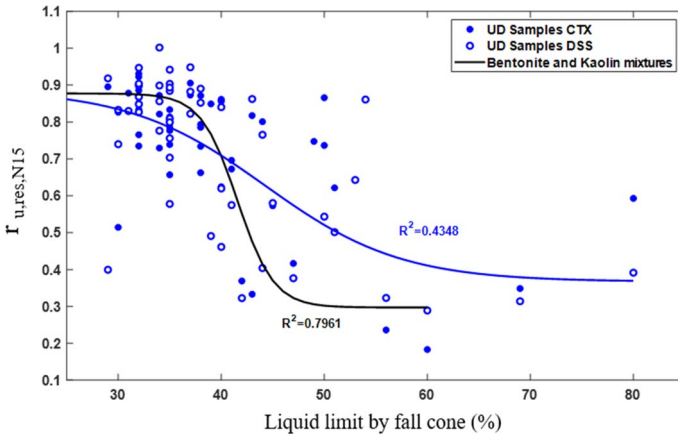


Fig. 10 Comparison of results for natural and artificial samples

the evaluation of natural samples. Similar intersection was found to apply to the plasticity index.

### 7.1 Influence of clay content

It was found that most of the liquefied samples plotted below a clay content of 20%, when the components of all UD samples are shown on the Ferret’s triangle (Fig. 11). This

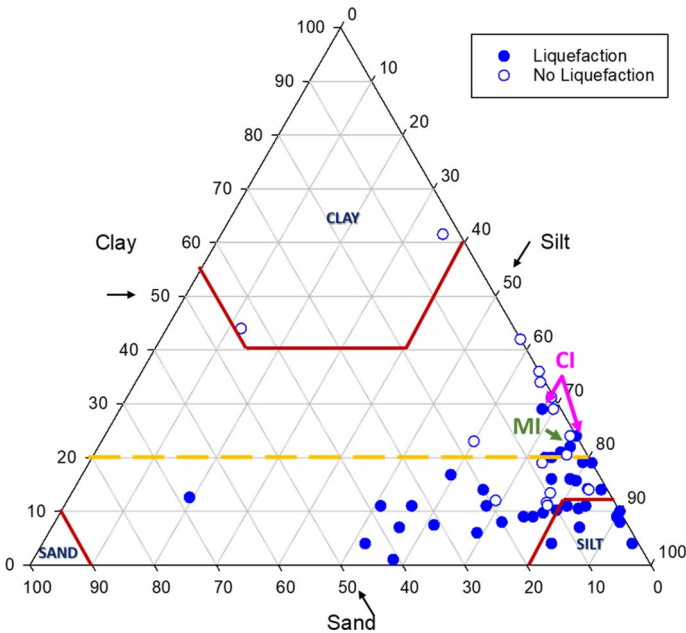


Fig. 11 Position of the samples on the classification chart

supports the finding that soils with clay fraction of less than this are most likely vulnerable to shaking (Fig. 9c). Although 20% clay content is considered the limit based on this figure, the fact that two CI samples exceeded this limit cannot be overlooked. However, if Table 1 is examined together with this figure, it is observed that both the CTX or DSS results of the samples indicated that they are liquefiable. If an interpretation needs to be made by taking this single test result into consideration, it may be speculated that if the silt content was greater than 70%, the clay ratio required for liquefaction increased up to 29% notwithstanding possible experimental error.

### 7.2 Influence of natural water content

The liquidity index  $I_L$  of a fine-grained soil is known to be a reliable indicator of its mechanical behaviour. However, dynamic testing showed that it has poor relationship with the pore pressures generated. Alternatively, if liquid limit is a significant indicator of dynamic performance, ratio of the in-situ water content ( $w_n$ ) to the liquid limit was expected to be an indicator of liquefaction, as proposed by several investigators (Wang 1979; Bray and Sancio 2006; Bol et al. 2010). Again, the relationship appears to be non-definitive with numerous samples liquefying which have  $w_n/w_L$  ratios lower than 0.9 (Fig. 12). Same scatter appeared for water contents after the test ( $w_f$ ).

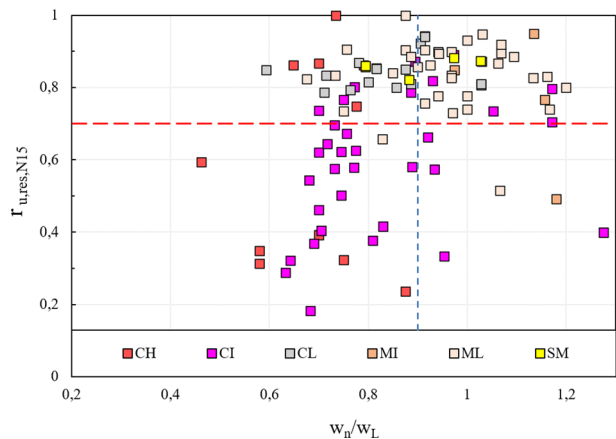
### 7.3 Influence of average size

The average size  $D_{50}$  of a soil reflects mechanical behaviour of a soil as well as its several index properties. It also represents the characteristic pore size of the soil. However, tests on UD samples did not show a reliable correlation with the pore pressures during shaking as can be seen from Fig. 13 with unacceptably low  $R^2$  value.

### 7.4 Evaluation of parameters

All foregoing discussion showed once more that behaviour of silts is notoriously unpredictable. Common knowledge dictates that fat clays do not fail/liquefy under normal

**Fig. 12** The relationship between in-situ water content and porewater pressure



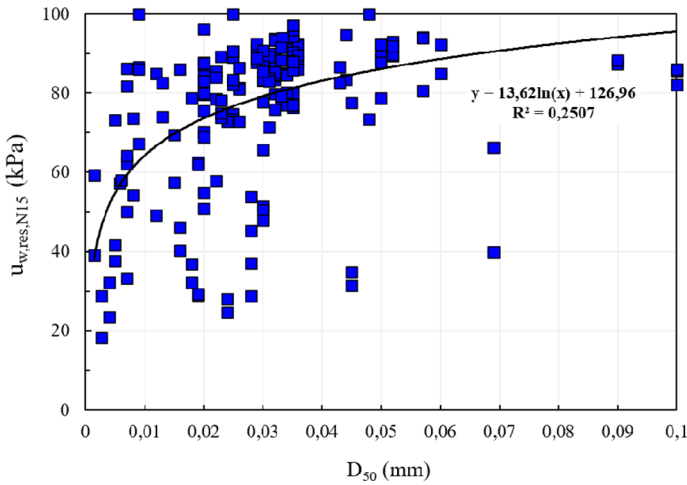


Fig. 13 The influence of average size on pore pressure (all samples)

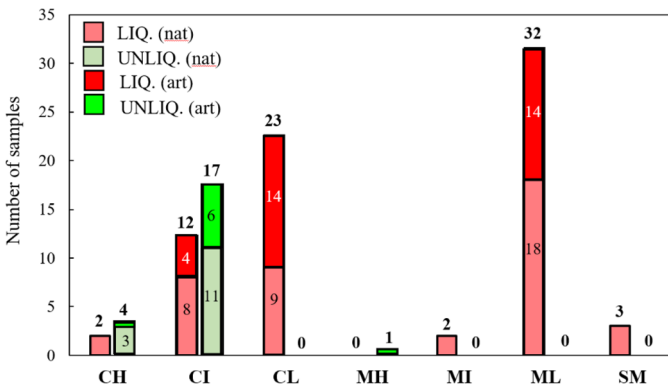
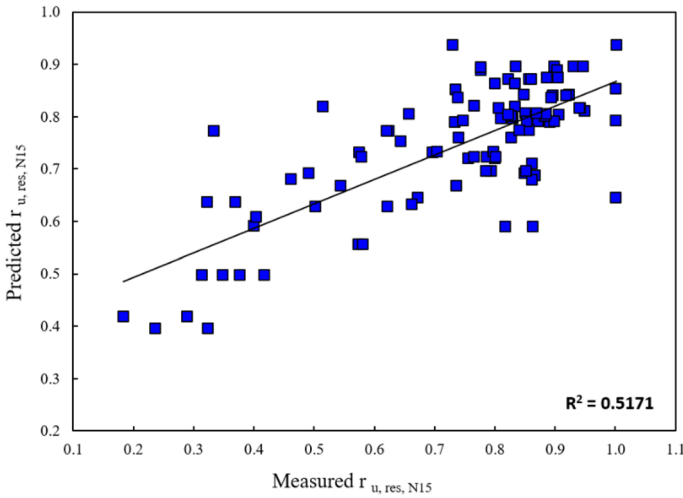


Fig. 14 Liquefied/unliquefied number of samples of soils (nat: natural, art: artificial)

conditions. Nevertheless, a look at the bar charts in Fig. 14 shows that several samples of high and intermediate plasticity reached the state of failure under intense shaking. Furthermore, all samples of low plasticity clays liquefied. This finding suggests that referring to dynamic laboratory tests alone may be misleading as physical properties can be. It is speculated that another variable, the pore/microstructure may be involved in the process. This requires further research.

The literature and the results from this study show that dynamic behaviour of a fine-grained soil cannot be identified by a single property. Factors contributing to its vulnerability to seismic acceleration have been defined by five values. These are the liquid limit, plasticity index, clay content, average grain size and water content in-situ. A multiple linear regression analysis performed on these variables indicated that  $D_{50}$  and  $w_n$  have relatively low correlation coefficients. Accordingly, they were not included in further analyses. The maximum residual pore pressure is expressed as a function of three



**Fig. 15** Comparison of Measured and Predicted (Eq. 3) Pore Pressures

variables: liquid limit ( $w_{L, cone}$ ), plasticity index ( $I_p$ ) and clay content (CF) resulting in Eq. (3) with an  $R^2$  of 0.517 as illustrated in Fig. 15.

$$r_{u, res@N15} = -1.06 CF - 0.65 I_p + 0.46 w_L + 0.82 \tag{3}$$

### 8 Evaluation of strain

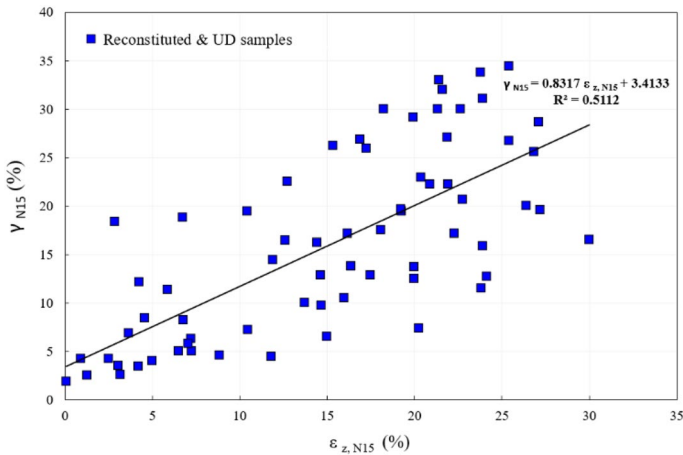
Current definition of liquefaction involves the rise of porewater pressures during shaking essentially with no reference to deformations. Indeed, failure of sands may be reflecting both components. Fine-grained soils subjected to dynamic stresses on the other hand, may not always indicate immediate excessive pore pressures but may undergo unacceptable deformations leading to ground failure.

The relationship for the deformations measured in CTX ( $\epsilon_z$ ) and in the DSS ( $\gamma$ ) tests would be defined with the equation

$$\gamma \cong (1 + \nu)\epsilon_z \tag{5}$$

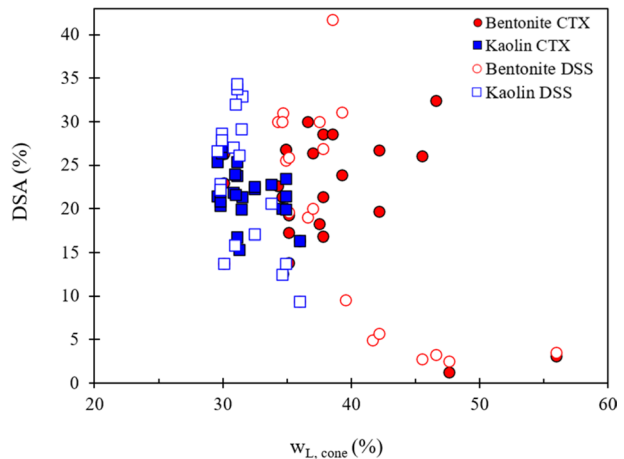
if the soil was nearly elastic as in the case of dense sand (Vucetic and Dobry 1991). Assuming fine-grained soil as an elastic material with a Poisson ratio  $\nu$  of 0.3, one would expect a 30% increase in the  $\gamma_{DSS}$  readings compared to the  $\epsilon_{z, CTX}$  readings. This was not found to be the case as illustrated by Fig. 16 where shear strain  $\gamma$  in DSS and axial strain  $\epsilon_z$  in CTX is compared, because there is near proportionality. It was thus decided to assume equality of  $\epsilon_z$  and  $\gamma$  in the following stages. In Fig. 16, the deformation values in this relation are the values in the 15th cycle, which correspond to double strain.

Figure 17 illustrates the relationship between the consistency and strains at  $N_{15}$  for artificial mixes where  $w_{L, cone} = 40\%$  has shown that it is a reasonable limit where deformations



**Fig. 16** Relationship for axial (CTX) and lateral (DSS) strains (this study)

**Fig. 17** Deformations of the artificial (reconstituted) samples



are at the acceptable Double Strain Amplitude (DSA) of 5%. Some bentonite CTX samples showed higher deformations suggesting the effect of high plasticity and void ratio.

The trend was confirmed when test results for natural and artificial samples were plotted together. Figure 18 shows that for liquid limit values of less than 40%, the deformations are very high. The DSA drops to below 10% for liquid limit range of 40–55%. DSA values are less than 5% for higher values of liquid limit. This confirms the view that a clay can show significant deformations during cyclic loading, even though the excess pore water pressure does not rise to the level of total stress. The natural samples demonstrate higher resistance to shaking in general, possibly due to OCR values higher than unity. The unexpected behaviour of clayey samples deforming less as consistency increased, requires further investigation.

When the two outstanding variables, liquid limit and clay fraction were plotted against strain the surface represented a reasonable relationship, as illustrated in Fig. 19.

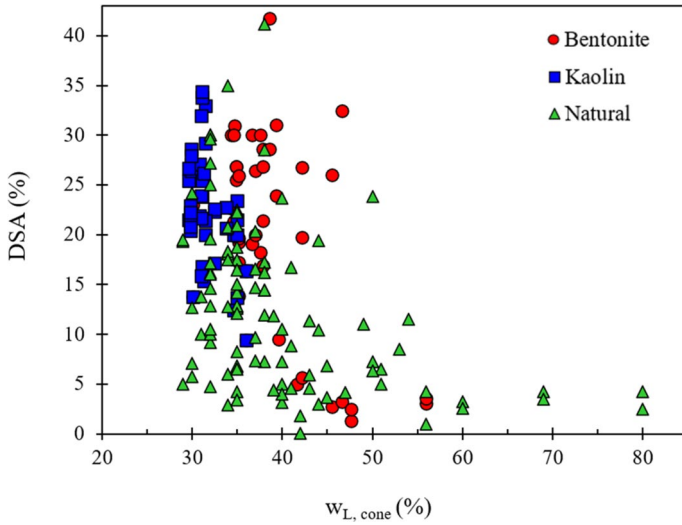


Fig. 18 Deformability as a function of liquid limit (all samples)

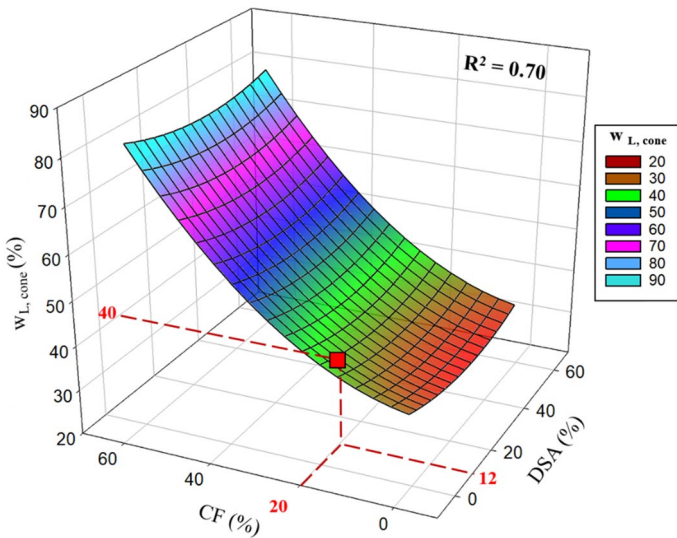


Fig. 19 Failure surface with reference to deformation

The failure surface created in Fig. 19 is a surface where the data are concentrated up to the 40% liquid limit value. The zone of the surface which contains liquid limit values above this, is the region that required extrapolation due to insufficient data. The failure surface requires improvement for values above the limit of  $w_{L, \text{cone}} > 40\%$  and should be used with caution beyond this range.

## 9 Discussion

A comprehensive study on probable behaviour of ground under dynamic conditions was carried out both on reconstituted and undisturbed and samples of Adapazari fine-grained soils. The results for the reconstituted samples support those of UD samples, although majority of the UD samples were found to be overconsolidated with OCR values rising to unprecedented levels. There is a clear transition from vulnerable to resistant when the relationship is represented by a sigmoid function. Several properties have been proposed by previous researchers (liquefaction liquid limit, plasticity index, clay content, average size and in-situ water content) as contributing to liquefaction. However, the results of this study show that the influence of average grain size and natural water content on failure are not significant.

It may be worthwhile to quote that there are several cases recorded in Adapazari where buildings at sites of high plasticity clay that exhibited excessive settlement on clay including toppling of the superstructures, although no sign of liquefaction was observed.

## 10 Conclusions

It has been confirmed by this study that dynamic behaviour of fine-grained soil cannot be identified by a single property. Factors contributing to its vulnerability to seismic acceleration can be prescribed by three values, these being the liquid limit, plasticity index and clay content. It is recommended that the liquid limit be measured in the fall cone test. Surprisingly, in-situ water content and average grain size have not been found to contribute to the process.

The direct simple shear test (DSS) is effective as the triaxial test to study dynamic behaviour, but simpler and quicker to perform. The ultimate normal stress measured in the DSS test is an additional advantage to diagnose sensitivity to shaking.

The excess residual porewater pressure can now be predicted within reasonable range of confidence. Those soils reaching a residual pore pressure ratio of 0.75 using Eqs. (2) and (3) can be deemed to be vulnerable to liquefaction during earthquakes of magnitude 7.5 or bigger.

Based on past visual experience and laboratory studies, it can be stated that a soil under the groundwater table can be declared as “non-liquefiable” if all the limits below are satisfied:

Liquid limit (percussion)  $w_L > 35$  (cone)  $w_L > 40$ .

Plasticity Index (percussion)  $I_p > 14$  (cone)  $I_p > 20$ .

Clay Fraction (hydrometer)  $CF > 18$ .

Excessive deformations recorded during laboratory testing suggest that it may not be reflecting the actual response of the ground in-situ.

The criteria proposed in this paper is being checked for less adverse shaking conditions by the authors.

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## Declarations

**Conflict of interests** The authors have no relevant financial or non-financial interests to disclose.

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