

Liquefaction and Failure Mechanisms of Sandy Sloped Ground during Earthquakes: A Comparison between Laboratory and Field Observations

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ABSTRACT

Prediction of ground failure involving earthquake-induced liquefaction of sloped sandy deposits is a major challenge in geomechanics due to the great number of factors that need to be considered such as initial static shear stress, cyclic shear stress, density state, confining pressure, loading conditions etc. This paper briefly describes the triggers (stress conditions) and the consequences (deformation behaviour) for three distinct failure modes that can be produced by an earthquake on sloped ground consisting of loose saturated sand. Such failure mechanisms were observed in the laboratory by performing undrained monotonic and cyclic torsional simple shear tests on Toyoura sand specimens. Most importantly, a practical method for assessing the failure behaviour of sandy-sloped ground undergoing undrained cyclic shearing based on sand failure characteristics observed in the laboratory is also presented. As an example, this method was used to satisfactorily predict slope failure that occurred in Ebigase area (Niigata City, Japan) during the 1964 Niigata earthquake.

Keywords: earthquake, failure, liquefaction, sand, sloped ground

1. INTRODUCTION

Liquefaction of sloped ground is a major natural phenomenon of geotechnical significance associated with damage during earthquakes. In the last few decades, in most seismic events with a magnitude greater than 6.5, the extensive damage to infrastructures, buildings and lifeline facilities have been associated with the occurrence of lateral spreading and/or flow (i.e. ground failure) of liquefied soils. Detailed information of worldwide case histories of liquefaction-induced slope failures for embankments, dams, levee, natural slopes etc during past earthquakes have been reported by many researchers (e.g. Seed (1987), Hamada *et al.* (1994)).

Prediction of ground failure involving earthquake-induced liquefaction of sandy sloped deposits is vital for researchers and practising engineers to understand comprehensively the triggering conditions and consequences of liquefaction, and to develop effective countermeasures against liquefaction. This paper reports on three distinct failure modes that liquefiable sloped ground can experience during an earthquake, focusing on the triggers

(stress conditions) and consequences (deformation behaviour). Foremost, a method to assess whenever the failure behaviour of sand sloped ground undergoing undrained cyclic shearing is likely to occur is also presented and its applicability to a real case history is described.

2. LIQUEFACTION AND FAILURE MECHANISMS OF FULLY-SATURATED LOOSE SANDY SLOPED GROUND OBSERVED IN THE LABORATORY

To investigate the role which the static shear stress (i.e. slope ground conditions) plays on the liquefaction behaviour and large deformation properties of saturated sand, we performed a series of undrained cyclic torsional simple shear tests on loose fully-saturated Toyoura sand specimens ($Dr = 44-50\%$) under various combinations of static and cyclic shear stresses (for details refer to Chiaro *et al.*, 2012a). From the study of failure mechanisms, three types of failure were identified based on the difference in the effective stress paths and the modes of development of shear strain during both monotonic and cyclic undrained loadings. The study confirmed that to achieve full liquefaction state ($p' = 0$) the reversal of shear stress during cyclic loading is essential. Alternatively, when the shear stress is not reversed, large shear deformation may bring sand to failure. These failure modes are briefly described henceforward. It is worth to mention that:

- τ_{max} ($= \tau_{static} + \tau_{cyclic}$) is the maximum shear stress during cyclic loading;
- τ_{min} ($= \tau_{static} - \tau_{cyclic}$) is the minimum shear stress during cyclic loading; and
- τ_{peak} is the transient undrained shear strength during monotonic loading.

where τ_{static} is the initial static shear stress and τ_{cyclic} is the cyclic shear stress.

- [1] **Cyclic Liquefaction ($\tau_{max} < \tau_{peak}$ and $\tau_{min} < 0$).** While undergoing a number of cycles comprises between 1 and 15 (i.e. 15 is the number of cycles representative on an earthquake of magnitude 7.5, which in this study is taken as reference to define whether liquefaction occurs or not; hereafter, see also $NN_{(CLQ)}$ behaviour), due to the excess pore water pressure generation, the effective mean principal stress progressively decreases and the stress state moves toward the failure envelope and finally reaches the full liquefaction state ($p' = 0$). Then, in the post liquefaction process, large deformations are developed. Hereafter, this type of failure is referred to as CLQ.
- [2] **Rapid Flow Liquefaction ($\tau_{max} > \tau_{peak}$ and $\tau_{min} < 0$).** Liquefaction takes place in-between the first cycle of loading and a rapid development of residual strain is observed. Herein, this type of failure is referred to as RLQ.
- [3] **Residual Deformation Failure ($\tau_{max} > \tau_{peak}$ and $\tau_{min} > 0$).** During cyclic loading large deformations are achieved rapidly, while in general liquefaction is not reached even after applying more than one hundred cycles. As a result, residual deformation brings the specimens to (shear) failure. Henceforward, this failure mode is referred to as RSD.

3. NO LIQUEFACTION AND NO FAILURE BEHAVIOUR OF FULLY-SATURATED LOOSE SANDY SLOPED GROUND

In addition to the three failure modes previously described, the following two cases in which neither failure nor liquefaction take place even after applying several tens of cycles were reported by the authors in Chiaro and Koseki (2010) by performing a series of numerical simulations using an elasto-plastic constitutive model developed at the Institute of Industrial Science, University of Tokyo (De Silva, 2008; Chiaro, 2010; Chiaro *et al.*, 2011; Chiaro *et al.*, 2012b; and further modifications):

- (i) when $\tau_{max} < \tau_{peak}$ as well as $\tau_{min} > 0$; and (ii) in the case of $\tau_{max} < \tau_{peak}$ and $\tau_{min} < 0$ (i.e. CLQ), but the level of τ_{cyclic} is very low so that liquefaction will occur in more than 15 cycles. These two additional cases hereafter are referred to as NN and $NN_{(CLQ)}$, respectively.

4. PREDICTION OF LIQUEFACTION, FAILURE MODES AND DEFORMATION EXTENT IN SANDY SLOPED GROUND

In this study, a method to identify the stress conditions that trigger the failure modes observed by Chiaro *et al.* 2012a (i.e. CLQ, RLQ and RSD) as well as the NN behaviour is presented. This is made by plotting experimental data (Table 1) in terms of η_{max} ($= \tau_{max}/\tau_{peak}$) vs. η_{min} ($= \tau_{min}/\tau_{peak}$) parameters. Thus, as shown in Figure 1(a), five zones can be distinguished and boundary conditions with clear physical meaning can be established. Each zone corresponds to specific failure behaviour (i.e. RLQ, CLQ, NN, NN_(CLQ) and RSD).

From a practical point of view, it is vital to understand whenever liquefaction occurs and its consequences i.e. development of large deformation. Thus, in the five-zone failure plot, the shear strains, measured after applying 15 cycles, are plotted (Figure 1(b)). It appears clear that when liquefaction occurs, shear strain is likely to exceed 50% (zones 1(a)). Alternatively, a significant amount of combined static and cyclic shear stresses may cause shear failure of slope although liquefaction does not take place (zone 3). Otherwise, only limited shear strain ($\gamma_s < 1\%$) may be developed if liquefaction does not occur (zone 2).

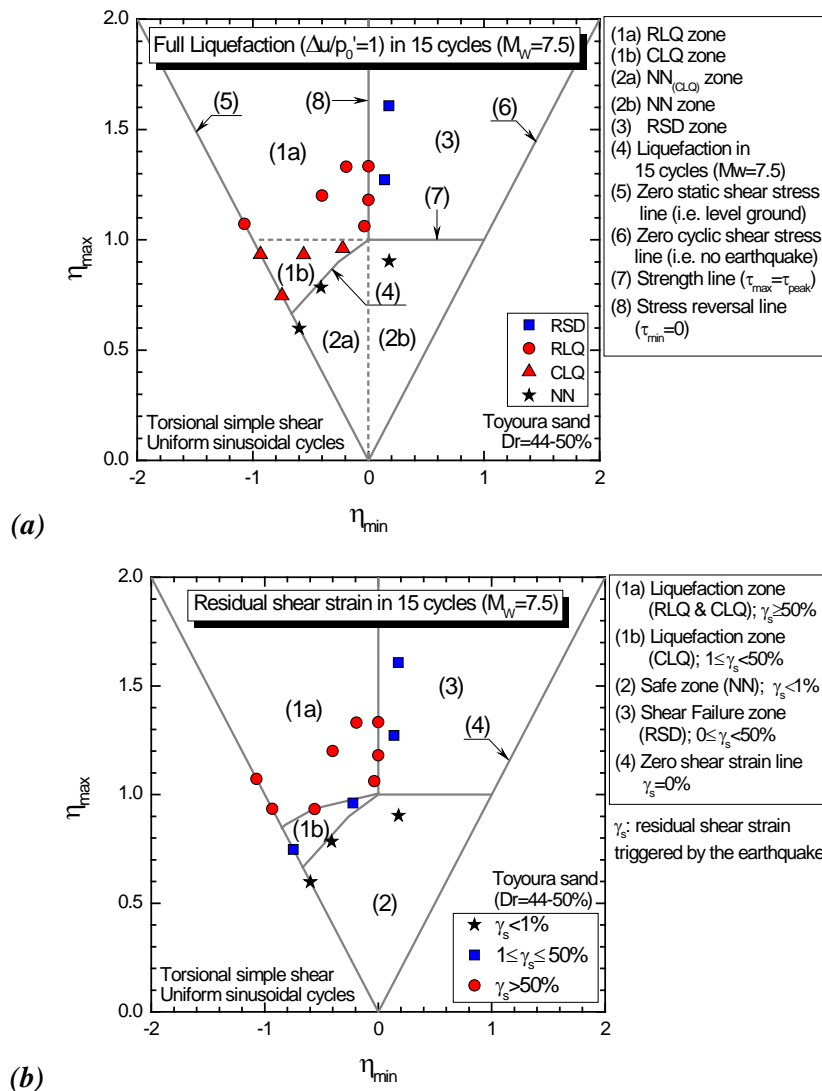


Figure 1: (a) Failure modes and (b) extent of deformation observed for Toyoura sand specimens subjected to undrained torsional simple shear loadings with and without initial static shear (data from Chiaro *et al.* (2012a) Kiyota (2007), De Silva (2008) and Chiaro and Koseki (2010)).

Table 1: Undrained cyclic torsional simple shear tests on loose Toyoura sand ($p_0' = 100$ kPa)

Test	Dr (%)	τ_{static} (kPa)	τ_{cyclic} (kPa)	τ_{max} (kPa)	τ_{min} (kPa)	τ_{peak} (kPa)	η_{max}	η_{min}	N_{LIQ}	γ_s (%) at N=15	Failure Mode	Ref.
Ch 3	46.6	10	16	26	-6	27.1	0.96	-0.22	8	45	CLQ	(a)
Ch 8	48.1	0	20	20	-20	26.8	0.75	-0.75	3	31	CLQ	(a)
Ch 9	48.0	5	20	25	-15	27.1	0.93	-0.46	2	>50	CLQ	(a)
Kiy	46.0	0	25	25	-25	26.8	0.93	-0.93	1.5	>50	RLQ	(b)
Ch 4	44.2	15	16	31	-1	29.2	1.06	-0.03	0.5	>50	RLQ	(a)
Ch 5	46.5	16	16	32	0	27.1	1.18	0	2	>50	RLQ	(a)
Ch 10	45.6	10	20	30	-10	25.0	1.20	-0.40	0.5	>50	RLQ	(a)
Ch 11	44.4	15	20	35	-5	26.3	1.33	-0.19	0.5	>50	RLQ	(a)
Ch 12	46.9	20	20	40	0	33.3	1.33	0	0.5	>50	RLQ	(a)
DeS	49.2	0	30	30	-30	28.0	1.07	-1.07	0.5	>50	RLQ	(c)
Ch 7	45.3	20	16	36	4	28.3	1.27	0.14	N/A	13	RSD	(a)
Ch 13	46.1	25	20	45	5	28.0	1.61	0.18	N/A	14	RSD	(a)
Ch 1	46.4	0	16	16	-16	26.8	0.60	-0.60	35	<1	NN _(CLQ)	(a)
Ch 2	45.5	5	16	21	-11	26.8	0.78	-0.41	19	<1	NN _(CLQ)	(a)
C-K	46.0	15	10	25	5	27.6	0.90	0.18	N/A	<1	NN	(d)

References: (a) Chiaro *et al.*, 2012a; (b) Kiyota, 2007; (c) De Silva, 2008; and (d) Chiaro and Koseki, 2010

5. BACK CALCULATION OF STRESS STATE AND SOIL STRENGTH IN SLOPED GROUND: EBIGASE AREA, 1964 NIIGATA EARTHQUAKE

On June 1964, a 7.5 moment magnitude (M_w) earthquake hit Niigata City (Japan) and its neighbouring area. Hamada *et al.* (1994) reported that the peak ground acceleration (a_{max}) in those areas was approximately 0.16 g. Due to the severity of seismic shaking and particular soil conditions, a large area suffered liquefaction, which caused severe damage to buildings, infrastructures and lifeline facilities as well as casualties. Large permanent horizontal displacement, subsidence and rising zones, a number of ground fissures and various sand boiling, observed in the Ebigase area (Niigata City), were the evidence that the gentle slope of a natural levee was extensively damaged by liquefaction. A post-seismic field survey revealed that the soil consisted mostly of sand from a sand dune (TS), alluvial sandy soils (As-1 and As-2) and alluvial clayey soils (Ac), as shown in Figure 3(a). The alluvial sandy soil layer was very loose to loose since its SPT N values were typically below 10. The estimated liquefied soils had a thickness (H) of 4-7 m, while the maximum horizontal displacement (D) measured at the ground surface was over 8 m. Thereby, extremely large shear strains ($\gamma = H/D$) of 50-90% were triggered by liquefaction.

Torsional simple shear conditions are a closer representation of the actual stress conditions in the ground during an earthquake. In this study, the earthquake-induced cyclic stress ratio as well as the gravity-induced static stress ratio at a depth z below the ground (Figure 2) was formulated in terms of torsional shear conditions in order to establish a framework to directly compare field (i.e. Ebigase area) and laboratory liquefaction behaviours of sand. To do so, the Seed and Idriss (1971) simplified procedure for evaluating the cyclic stress ratio was adopted and adjusted to the torsional shear condition. By converting the typical irregular earthquake record to an equivalent series of uniform stress cycles (Seed and Idriss, 1975), considering the flexibility of the soil column throughout a stress reduction coefficient (Iwasaki *et al.*, 1978) and introducing a magnitude scaling factor (MSF; Idriss and Boulanger, 2004), the following expression was derived (refer to Chiaro (2010) for details):

$$(\text{CSR})_{7.5} = \frac{\tau_{cyclic}}{p_0'} = \frac{0.65 (a_{max} / a_g) r_d}{\text{MSF} [(1 + 2 K_0) / 3] [1 - 0.5 (z_w / z)]} \quad (1)$$

$$\text{MSF} = [6.9 \exp(-M_w/4) - 0.058] \leq 1.8 \quad \text{and} \quad r_d = (1 - 0.015 z) \quad (2), (3)$$

where a_{\max} (in g) is the peak ground (horizontal) acceleration; a_g is the gravity acceleration (=1 g); M_w is the moment magnitude of the earthquake; K_0 is the coefficient of earth pressure at rest; and z (in metres) is the depth below the ground surface. It should be noted that the stress reduction coefficient (r_d) is a dimensionless factor. MSF is a factor for adjusting the earthquake-induced CSR to a reference $M_w = 7.5$, provided that such an earthquake induces 15 equivalent stress cycles of uniform amplitude.

To estimate K_0 , the well-known Jaky's relationship (Jaky, 1944) was employed as shown by Eq. (4). In addition, the empirical correlation between internal friction angle (ϕ') and relative density (D_r) proposed by Schmertmann (1978) for (fine) clean sand was used:

$$K_0 = 1 - \sin \phi' \quad \text{and} \quad \phi' = 28 + 0.14 D_r \quad (4), (5)$$

The range of soil density (D_r) was evaluated from field data by referring to the relationships between SPT N-value and D_r reported in Lambe and Whitman (1969).

Assuming infinite slope state and torsional shear conditions, the static shear stress ratio induced by gravity on a soil element of sloped ground, at a depth z underneath the ground surface and a depth z_w beneath the water table, was calculated as follows (Chiaro, 2010):

$$\text{SSR} = \frac{\tau_{\text{static}}}{p_0'} = \frac{\tan \beta}{[(1 + 2 K_0)/3][1 - 0.5 (z_w/z)]} \quad (6)$$

where β (in radian) is the gradient of slope.

Finally, based on laboratory test results on Toyoura sand (clean sand), the undrained shear strength of Niigata sandy soils was estimated as follows:

$$\text{UPSR} = \frac{\tau_{\text{peak}}}{p_0'} = 0.1015 + 0.0046 D_r + 0.180 \frac{\tau_{\text{static}}}{p_0'} \quad (7)$$

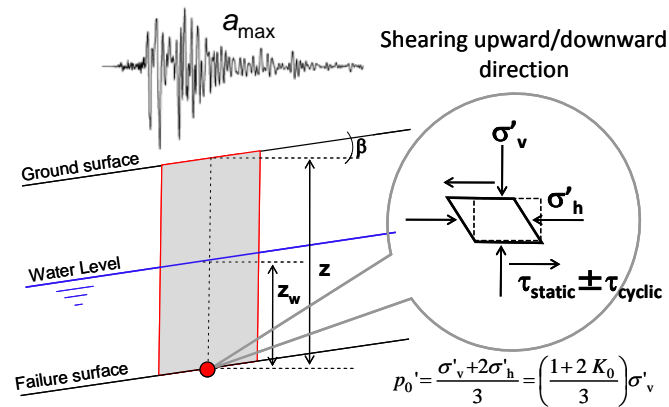


Figure 2: Stress conditions acting on a soil element beneath sloped ground during an earthquake

The $(\text{CSR})_{7.5}$, SSR, UPSR and D_r were evaluated for various soil elements, located at different depths beneath sloped ground level, by referring to the soil profile reported in Figure 3(a). The obtained values are listed in Table 2 for reference. In Figure 3(b), the predictions of failure mode of gentle slope at Ebigase are reported for the cases with and without earthquake in the plot $\eta_{\max, \text{field}}$ vs. $\eta_{\min, \text{field}}$. One can see that due to the severe seismic loading conditions (i.e. $a_{\max} = 0.16$ g and $M_w = 7.5$), soil is likely to experience extremely large deformation ($\gamma_s > 50\%$) triggered by liquefaction, except for the case of dense soil (N-2). These predictions are well in accordance with the results presented by Hamada *et al.* (1994). In particular, it seems clear that the predicted rapid flow liquefaction (RLQ)

conditions may give a reasonable explanation for the large deformation behaviour of liquefied sloped ground observed at Ebigase during the 1964 Niigata earthquake.

Table 2: Stress conditions evaluated for Ebigase area during the 1964 Niigata Earthquake

Case	z (m)	z _w (m)	N-SPT	Dr (%)	(CSR) _{7.5}	SSR	UPSR	η _{max, fld}	η _{min, fld}	Field behaviour	
										Hamada <i>et al.</i> (1994)	This study
L-1	2.5	0.4	10	30	0.176	0.061	0.251	0.95	-0.46	Liq.	Liq.
L-2	3.5	1.4	10	30	0.199	0.071	0.252	1.07	-0.51	Liq.	Liq.
L-3	4.5	2.4	10	30	0.214	0.077	0.253	1.15	-0.54	Liq.	Liq.
L-4	5.5	3.4	10	30	0.223	0.082	0.254	1.20	-0.56	Liq.	Liq.
L-5	6.5	4.4	10	30	0.230	0.085	0.255	1.24	-0.57	Liq.	Liq.
L-6	7.5	5.3	15	50	0.242	0.091	0.348	0.96	-0.43	Liq.	Liq.
N-1	8.5	6.4	22	60	0.248	0.096	0.395	0.87	-0.39	No Liq.	Liq.
N-2	9.5	7.4	30	70	0.254	0.099	0.441	0.80	-0.35	No Liq.	No Liq.

Note: η_{max, field}=(SSR+CSR)/UPSR and η_{min, field}=(SSR-CSR)/UPSR

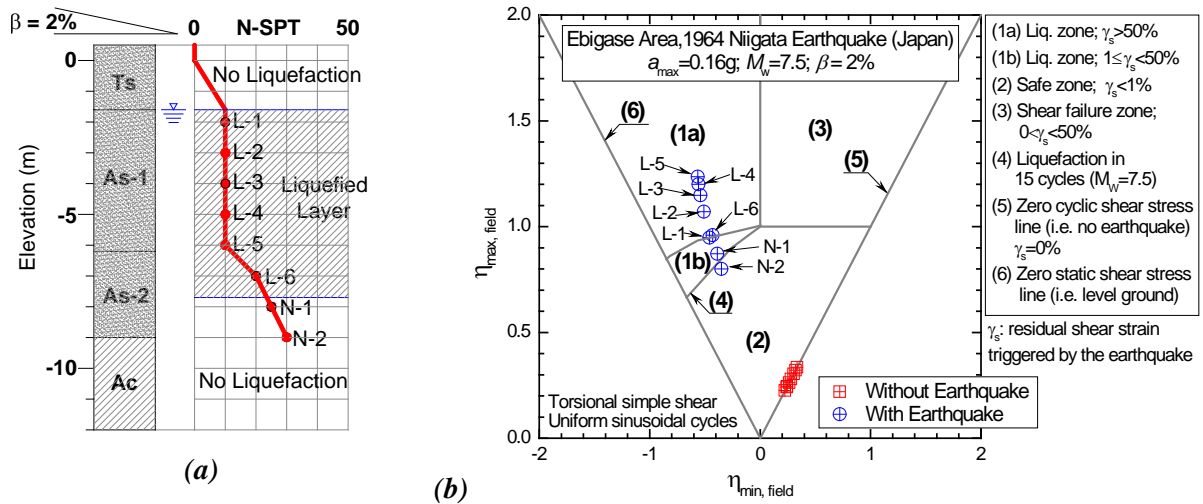


Figure 3: (a) Soil column examined (Ts: dune sand; As-1 and As-2 alluvial sandy soils; and Ac: alluvial clayey soil); and (b) Predictions of failure mode for Ebigase area during the 1964 Niigata Earthquake

6. CONCLUSIONS

Prediction of ground failure involving earthquake-induced liquefaction of sloped sandy deposits is essential for understanding comprehensively the triggers and consequences of liquefaction. In this paper, an attempt is made to identify key factors that govern failure of sandy sloped ground during earthquakes and a method to assess whenever liquefaction or shear failure occurs within a saturated sandy sloped deposit is presented. It is shown that the proposed method, defined based on laboratory investigations (i.e. undrained cyclic torsional simple shear tests), is capable of predicting with good accuracy the failure behaviour observed in the field i.e. slope failure in Ebigase (Japan) during the 1964 Niigata earthquake. Despite the number of approximations that can be made in this kind of study (with regards to determination of soil densities, cyclic and static stress ratios, and undrained strength in the field), the proposed method provides a useful framework for assessing liquefaction and shear failure of sloped ground in many practical proposes. Whenever greater accuracy is justified, the method can be readily supplemented by test data on particular soils or by ground response analysis to provide evaluations that are more definitive.

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