

# INFLUENCE OF LOW PERMEABILITY CAPPING LAYERS ON LIQUEFACTION INDUCED FAILURE IN STRATIFIED GROUND

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**Abstract.** Large scale permanent ground deformations due to an earthquake is generally termed as flow liquefaction (FL). Flow failures are often observed in gentle slopes where minimum undrained shear strength remain less than static shear stress even after ground motion ceases and the induced deformations continue until a balance is achieved. Such failures are caused by significant strength loss in saturated granular soils due to the seismic induced excess pore water pressure (PWP). As the soil strength is reduced, plastic deformations can occur due to even static shear stresses such as building loads or driving forces on sloping ground. While such deformations are rare in level grounds with saturated sandy deposits, the triggering of such events depends on the ability of soil in retardation of dissipation of excess PWP which controls the effective stress in the layer. The dissipation of PWP is influenced by the permeability of inter-layers in the layered sand deposit. As in natural conditions, sand deposits are highly heterogeneous and stratified, they often tend to contain low permeability capping layers sandwiched between them. This low permeability, tends to inhibit the dissipation of PWP after the onset of ground shaking, which leads to further instability of overlying layer. This study aims to investigate the effect of such capping layers on the PWP dissipation and deformation of saturated sand deposits under dynamic loading conditions. To evaluate the buildup and dissipation of PWP in a simple way, multiple one dimensional (1-D) model tests were conducted by imparting dynamic loading to a soil column with different capping layer conditions. Multiple parametric studies were conducted with varying relative density of sandwiching layer, capping layer thickness and capping layer material.

**Keywords:** Flow liquefaction, capping layer, excess pore water pressure, settlement.

# 1 Introduction

## 1.1 Background

The liquefaction resistance of a site does not merely depend on the soil deposits but also on the heterogeneity of stratification. Typically, in case of gently sloped alluvium deposits, the liquefaction susceptibility depends not only on the shear strength of deposits but also on the permeability characteristics of the soil inter-layers which may have a low permeability coefficient. The permeability of these sandwiched layers is of key importance as they control the dissipation of excess pore water pressure (EPWP) which develops in saturated deposits during shear wave propagation. During liquefaction in a saturated sand deposit, the excess pore water developed dissipates through the overlying deposits to the ground thereby carrying sand and fines along with it creating sand boils [1]. With the expulsion of pore water, the soil particles get packed densely causing settlements.

In past most disastrous damages due to seismically developed EPWP have occurred in inclined ground, earthen dam structures and dikes where a possible failure surface already exists prior to the impact of seismic wave [1]. The failure surface in these types of liquefactions are usually large and cause massive soil displacement, called flow failure. The flow failures are mainly triggered by gravitational forces under static condition which occurs when the strength of soil reduces to strength required for static stability conditions [2]. While, such large scale flow failures are expected to occur in ground with higher gradients, in past earthquakes the flow or movement of soil mass has been observed to be upto several meters even in ground of a very low gradient [3]. Although, liquefaction at sites with no or a very gentle slope is considered to be a simple phenomenon, but the mechanisms involved in initiation of liquefaction like the buildup and dissipation of pore water pressure are quit complex [1].

Such type of flow failure has been observed during the 1964, Niigata earthquake where a considerable ground movement was observed for a ground surface with gentle slope of less than 1% [3]. Further, such failures also seem to occur after the cease of ground motion as observed during past events [4][5]. These case histories imply that for a gentle slope to undergo flow failure, not only gravity but other multiple mechanisms are required to trigger the failure [6]. While the exact mechanism of such failures in level grounds is not known, previous researchers have attributed the cause to presence of impermeable layer sandwiched between permeable sand layers. These interlayers inhibit the dissipation of EPWP due to low permeability thereby stopping the liquefied soil layers to stabilize after the onset of liquefaction [6][7][8][9][10].

The current research aims in studying large scale flow liquefaction observed on a ground with very gentle slope during the recent 2018, Sulawesi earthquake event where the ground displacement from few meters to about several hundred meters was observed killing thousands of people and devastating many villages during the event. The city of Palu, which lies in the Central Sulawesi Province of Indonesia witnessed massive flow liquefaction during the September 28, 2018 earthquake of magnitude Mw 7.5 epicentered in the Minahasa Peninsula neck.

## 1.2 Site Geology and Seismic History

The city of Palu lies in the Central Sulawesi Province in Sulawesi Island of Indonesia. The city is straddled by a seismically active sinistral Palu – Koro fault which has an annual tectonic slip rate of around 40 mm [11]. The region is overlain by sedimentary and volcanic deposits of Miocene – Quaternary period, which have undergone low grade thermal metamorphism [12]. The Palu valley extends to 7 km in width and flanked by N-S aligned, around 60 m high sharp triangular slopes and short alluvial fans in West, and gently steps faults in East [13]. Hosting an active strike – slip fault, the region has experienced multiple large devastating earthquakes in the past. Historic seismicity of the PKF region shows that the fault activity is concentrated at shallow depth with major earthquakes of magnitude  $M_w \geq 6.5$  during years 1900-2018 have a hypocentral depth of less than 50 m [11][14] as shown in Fig.1 (a) and (b). Where Fig. 1 (a) shows the major seismic activity of PKF for period 1900 – 2018 and Fig. 1 (b) shows the plot of magnitude ( $M_w$ ) to Hypocentral depth.

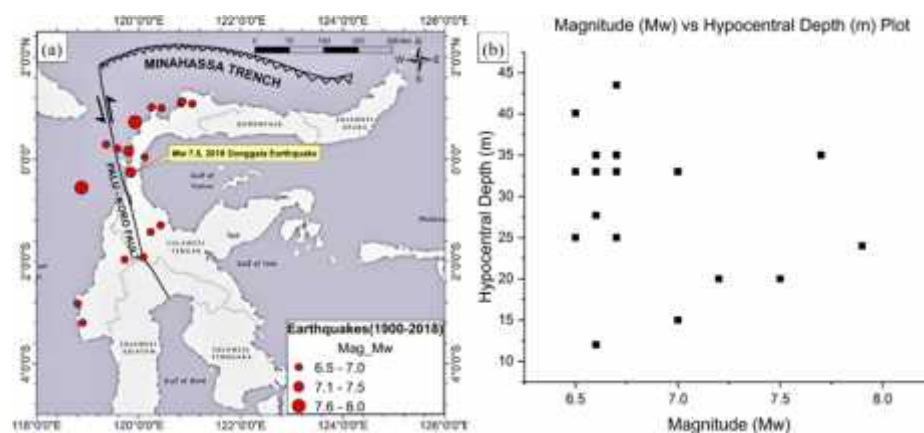


Figure 1: (a) Earthquakes across PKF with magnitude  $M_w \geq 6.5$  during years 1900-2018 and (b) Plot of magnitude ( $M_w$ ) versus Hypocentral depth (m) for the same events (USGS, 2018, Socquet et. al., 2006).

## 2 Case Study

### 2.1 Event History

On September 28, 2018, Central Sulawesi was struck by a powerful earthquake of magnitude  $M_w$  7.8 with strong foreshocks and aftershocks for multiple days. The event occurred as a result of strike – slip faulting at a shallow depth. The seismic waves produced by the earthquake at hypocenter of 20 km [14] caused large scale liquefaction and flow failures across multiple locations in the city of Palu. Even though the distance from epicenter was around 70 km, most damage was observed in Juno Oge, Sibalaya and Balaroa villages in Sigi Regency in Palu. The foreshock,

mainshock and aftershock map of the event is shown in Fig. 2. The event had a major foreshock of magnitude  $M_w$  6.1 just three hours before the mainshock and several aftershocks of magnitude  $M_w \geq 5.5$ , which shows that large amount of elastic strain energy had been accumulated in the fault [14].

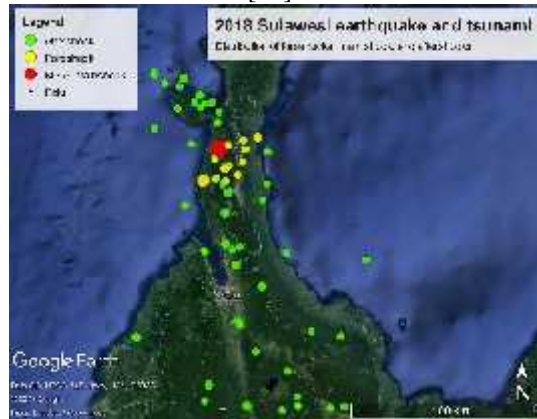


Figure 2: Distribution of foreshock, mainshock and aftershock for 2018 Sulawesi earthquake.

## 2.2 Site Investigation

A team of researchers from Kyushu University visited the flow liquefaction sites in Juno Oge, Sibalaya and Balaroa in Palu aftermath of earthquake. The survey elements included site reconnaissance, disturbed soil sampling, portable dynamic cone penetration tests (PDCPT) and aerial survey with drone. Fig. 3 here depicts the condition of the flow failure site at Juno Oge before (Fig. 3(a)) and after the seismic event (Fig. 3(b)). The site mainly composed of agricultural fields with a scant population and an a very gentle sloping ground of less than 2% (Fig. 3(c)). It was also found that an irrigation channel which was passing from the eastern side of failure zone breached due to liquefaction during the shockwave propagation as shown in Fig. 3(b). Due to the presence of irrigation channel, the ground water level was very high ( $< 5\text{m}$  from ground surface) which is also considered to be a key factor contributing to flow failure. The breach of water channel discharged a large volume of water into the already failed zone causing a massive mudflow as depicted in Fig. 3(b).



Figure 3: Lateral flow area in Juno Oge village due to Sulawesi Earthquake, (a) before flow failure, (b) after flow failure with mud flow and water channel and (c) slope profile of the flow area (modified from Google Earth).

The maximum site amplification occurred in the Palu valley, which has deposits with low shear wave velocity profile [15], as can be seen in Fig. 4 (a). It can be seen that the valley region along with the coast line has very low shear wave velocity while the mountainous region has high  $V_{s30}$  values. Fig. 4 (b) depicts the satellite image of liquefaction site at Balaroa before the event while Fig. 4 (c) shows the condition of the site after liquefaction taken by a drone camera. It can be ascertained from Fig. 4 (b) and (c) that a large scale liquefaction occurred devastating and displacing all the housing structures in the area to hundreds of meters.

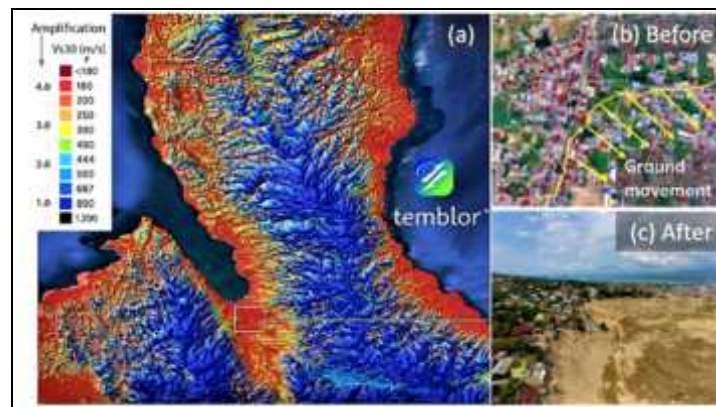


Figure 4:  $V_{s30}$  map of Palu region along with a site amplification forecast for an Mw 7.5 earthquake (Patton et al., 2018), (b) Google Earth image for site in Balaroa, Palu prior to 2018, Sulawesi Earthquake and (c) Drone Image of Balaroa site after the event showing mass displacement of structures from original location.

Further in Fig. 5(a) shows the location where the PDCPT and SPT tests were performed at the flow failure site in Juno Oge. Fig. 5(b) shows the condition of soil profile at crown of failure, also depicting the difference in elevation of ground after liquefaction. Fig. 5(c) represents the plot of N values versus depth obtained from

PDCPT test conducted outside the crown of failure zone. It can be seen from the plot that the subsoil condition is weak with a maximum N-value reaching upto 15 and dropping back to low numbers and presence of water table at a shallow depth of 2.15 m from the ground surface. Further Fig. 5(d) shows the subsoil condition at location of Borelog 1. It can be seen from the illustration that the silt layers are sandwiched in between sand layers at shallow depth with a higher fines content and some plasticity characteristics. These soft sandwiched silt layers are considered to be a probable contributor to the large scale flow failure by inhibiting the dissipation EPWP from below sand and gravel layers.

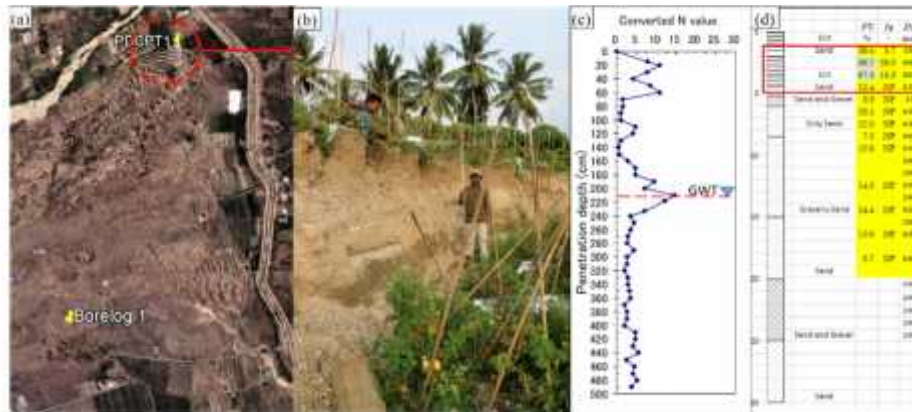


Figure 5:(a) Location of PDCPT and SPT at Juno Oge, (b) Location of crown of the flow failure at Juno Oge site with high vertical settlement, (c) plot of N-value versus depth obtained by PDCPT1 test outside the failure zone and (d) Borelog1 profile of soil in failure zone.

### 2.3 1 – D Model Tests

To evaluate the effect of a low permeability sandwiched soil layer between sand or gravel layers and understand the mechanism of lateral flow at Juno Oge, 1D model tests were conducted in laboratory by trying to mimic the soil stratification conditions in which a silt or clay seam is sandwiched between sand layers. In the current study, 1D model tests were conducted using loose saturated Toyura sand with a seam Pearl Clay sandwiched between the sand layers. Pearl Clay which has 50% clay and 50% silt content, was used as a capping layer to simulate the presence of a silt seam or clay layer in field and evaluate its effect on the dissipation of excess pore water pressure. The grain size distribution of Toyura sand and Pearl clay are given below in Figure 6. The Toyura sand has specific gravity,  $G_s$  is 2.65 and maximum ( $e_{max}$ ) and minimum ( $e_{min}$ ) void ratios of 0.977 and 0.615 respectively. For Pearl clay, the  $G_s$  is 2.71 and the liquid limit and plasticity index are 49% and 27% respectively [16]. The schematic experimental setup of 1D test is given in Figure 7.

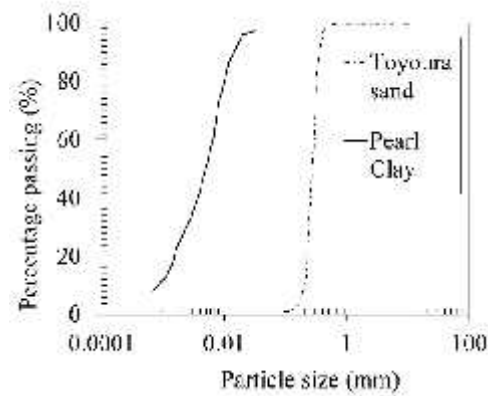


Figure 6: Gradation curve for Toyura sand and Pearl clay.

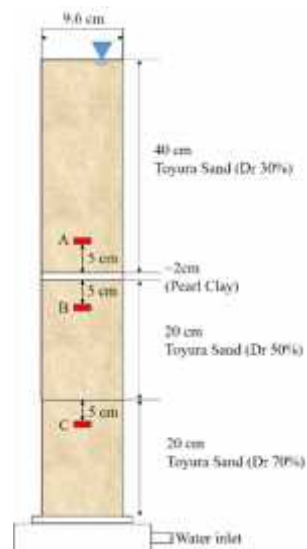


Figure 7: Schematic diagram of 1 – D experimental setup.

As shown in Fig. 7, the  $D_r$  is the relative density of the sand layer. A, B and C are the pore water transducers. A condition of liquefaction was generated by providing a shock through a hammer blow to the 1D setup at the base. The resulting excess pore water pressures were measured and analyzed for conditions with clay seam and without clay seam.

### 3 Results and discussions

The results for 1D model test after a single impact of hammer are plotted as the ratio of excess pore water pressure to maximum excess pore water pressure ( $u/u_{max}$ ) versus



time (secs). The  $u/u_{max}$  value has been plotted against the duration to show the effect of clay seam on development and dissipation of excess pore water pressure in saturated sand under impact loading. Fig. 8 shows the plot for condition with no clay seam present (Case 1). Here in Fig. 8 it can be seen that the EPWP developed for pore water transducer C is higher than A and B. This may be due to its location at the bottom from where the dissipation of EPWP takes time, while at A the EPWP is dissipated quickly since it is near to surface. Further, at B, the EPWP is developed but gets dissipated quicker than at C due to shorter seepage path. Further in Fig. 9, the plot for case for clay seam sandwiched between sand layers is shown (Case 2). Here, the EPWP developed at pore water transducer B is highest due to presence of clay seam above which inhibits the dissipation of EPWP. It can also be observed that the EPEP at A is higher than at C which may be attributed due to a higher relative density at the bottom during sample preparation.

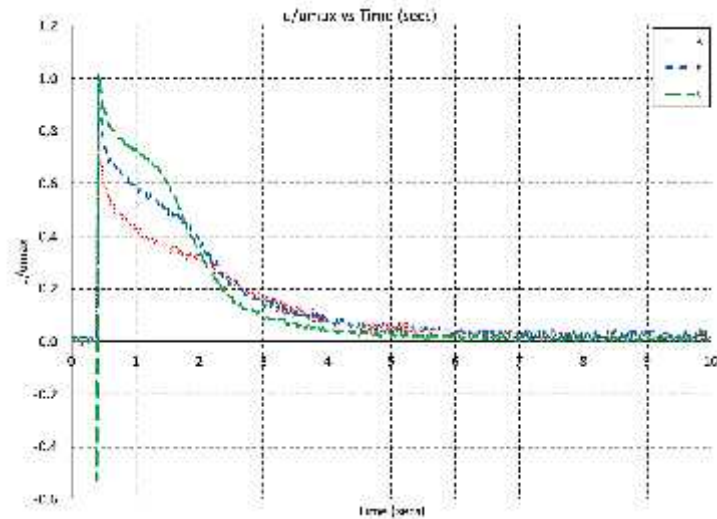


Figure 8: Plot of  $u/u_{max}$  versus time (secs) for Case 1 with pore water transducers at three locations across the height.



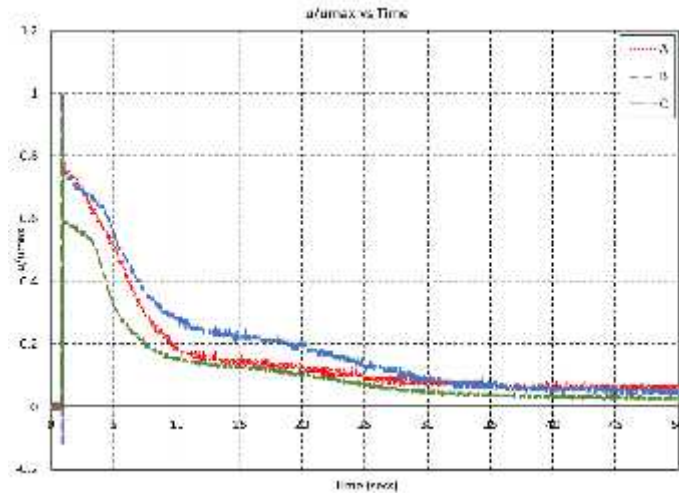


Figure 9: Plot of  $u/u_{max}$  versus time (secs) for Case 2 with pore water transducers at three locations across the height.

Further in Fig. 10, the plot for  $u/u_{max}$  versus time for transducer B is compared for both the cases (1&2). From the figure it can be observed that ratio  $u/u_{max}$  for B (2) takes longer duration to dissipate after reaching peak and still has a higher value while that for B (1) is dissipated. This implies the capping effect of clay seam which prevents the dissipation of excess pore water pressure in the sand layer below. In Fig. 11 the plots for  $u/u_{max}$  versus time for pore pressure transducers A and C are shown, where A is located above the clay seam and C is located at a distance of 15 cm from the bottom of the sand layer with  $D_r$  70%. Here for both the transducers the duration for dissipation of excess pore water pressure for sand with clay seam is higher than the condition without clay seam. This shows that the presence of a low permeability layer can not only hinder the dissipation of pore water pressure in adjacent areas but also in regions at a higher distance from the layer vertically.

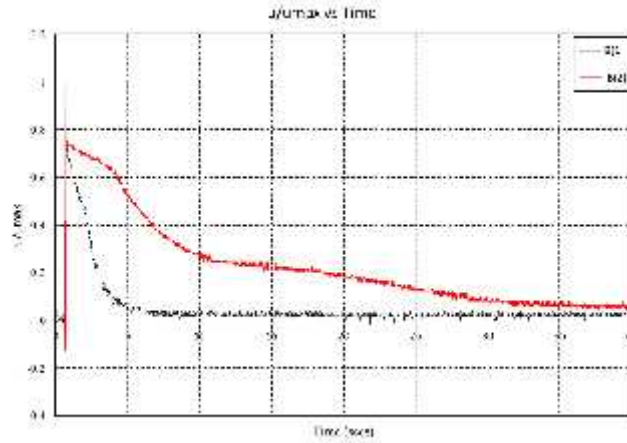


Figure 10: Dissipation of excess pore water pressure versus time (secs) at pore pressure transducer B for Case 1 and 2 .

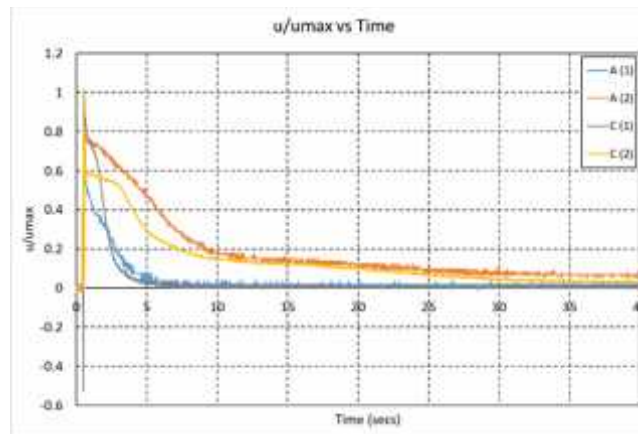


Figure 11: Dissipation of excess pore water pressure versus time (secs) at pore pressure transducers A and C for Case 1 and 2 .

The pictorial evidence of the effect of clay seam in sand layer during liquefaction can be seen in Fig. 12. Here the development of water layer below the clay seam at different time frames is shown. Fig. 12(a) is recorded at  $T = t$  seconds when the hammer impact was made. Fig. 12 (b) depicts the maximum thickness of the water film at  $t + 5$  secs while Fig. 12 (c) shows the decreased water film thickness at  $t + 35$  secs. This shows how the water film dissipates slowly thereby influencing the dissipation of excess pore water pressure from the below sand layers by forming an impermeable capping layer. The water film thus created causes settlements in the clay seam after the overall dissipation of EPWP which can be seen in Fig. 13. Here in Fig. 13(a) the base of clay seam can be seen at 39.8 cm which settles to 39.0 cm after the hammer blow due to liquefaction as seen in Fig. 13(b).

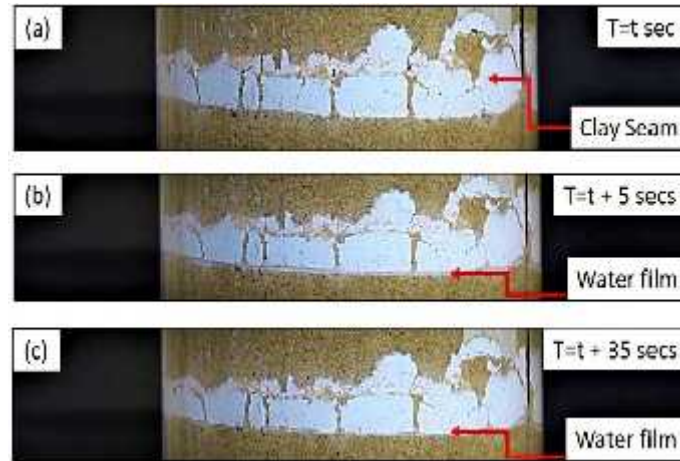


Figure 12: Formation of water layer under the clay seam at different time durations: (a) Time ( $T$ ) =  $t$  secs, (b)  $T = t + 5$  secs and (c)  $T = t + 35$  secs.

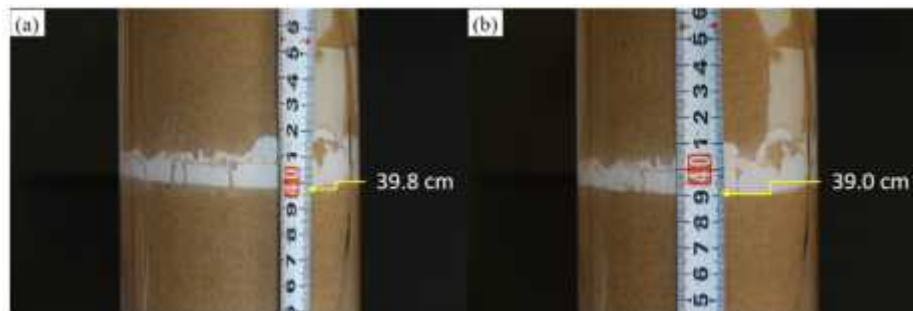


Figure 13: Height of clay seam (a) before liquefaction and (b) after liquefaction.

#### 4 Conclusion

The team of authors conducted site investigation at Juno Oge, Sibalaya and Balaroa villages in Palu, Indonesia after the September 2018, Sulawesi earthquake which caused large loss of life as well as massive damage to infrastructure. Multiple field tests including PDCPT and aerial drone photography were conducted. From the field tests it was observed that sandy soil consists of silt layers with some clay content sandwiched in between sand and gravel layers. 1D model tests were conducted to ascertain the cause of flow failure by simulating the site conditions as per the observations of field investigations. From the results of 1D model tests, it was observed that the presence of a low permeability silt or clay seam sandwiched between saturated sand layers can inhibit the dissipation of excess pore water pressure due to impact of dynamic loading. This delay in turn causes the reduction in residual strength of soil to the static shear stress. Which can cause further instability in the top layer by causing excessive settlements and flow failure. Although, liquefaction is a complex phenomenon involving multiple

parameters, the authors conclude that the presence of such low permeability silt or clay layers can act as one of the major contributing factor in flow failure.

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