



NEW ZEALAND SOCIETY FOR EARTHQUAKE ENGINEERING

2019 Pacific Conference on Earthquake Engineering

TURNING HAZARD AWARENESS INTO RISK MITIGATION

4 – 6 April | SkyCity, Auckland | New Zealand



Investigating the Role of Clay Layers in Containing Liquefied Soils

C. A. O'Connell, E.K. Lowe, M.E. Stringer

University of Canterbury, Christchurch.

ABSTRACT

During the 2010 - 2011 Canterbury earthquake sequence, extensive liquefaction was observed in many areas of Christchurch city and its surroundings, causing widespread damage to buildings and infrastructure. While existing simplified methods were found to work well in some areas of the city, there were also large areas where these methods did not perform satisfactorily. In some of these cases, researchers have proposed that layers of fine grained material within the soil profile may be responsible for preventing the manifestation of liquefaction. This paper presents preliminary findings on the mechanisms at play when pressure differentials exist across a clay layer. It is found that if the clay layer is unable to distort, then pore fluid is unable to break-through the layer even with relatively high pressures, resulting in dissipation of excess pore pressures by seepage. If the layers are however able to distort, then it is possible for the pore fluid to break through the clay layer, potentially resulting in adverse effects in terms of the severity of liquefaction.

1 INTRODUCTION

During the 2010 and 2011 Canterbury earthquake sequence, the urban area of Christchurch experienced widespread liquefaction throughout the city centre and Eastern suburbs which caused significant damage to foundations, bridges and buried infrastructure. It is estimated that liquefaction contributed to a \$15 billion NZD loss during the Canterbury earthquake sequence (Cubrinovski et al. 2014). In some areas, the extent of surface manifestation were reasonably well captured by state of practice tools including the simplified method of Boulanger and Idriss (2016) when best estimates of ground conditions and strong motions are used. However there were also reasonably large areas where these methods both over-predicted and under-predicted the severity (Maurer et al. 2014) of liquefaction.

A particular issue is the difference between the triggering of liquefaction and the development of surface manifestation of liquefaction (i.e. sand boils). Recently, Cubrinovski et al (2018) analysed the ground performance at 55 research sites with detailed site investigation and site response analyses. These studies highlighted the interplay between the response of different layers in a soil profile, and in particular highlighted the important role that relatively impermeable layers can play in preventing the rapid upward seepage of pore fluid following the end of strong motions and therefore in preventing surface manifestation of liquefaction.

The role of non-liquefiable layers in the surface expression of liquefaction has been recognised for some time. On the basis of post-earthquake observations, Ishihara (1985) developed a criterion to distinguish sites where surface manifestation of liquefaction would be expected based on the thicknesses of an overlying crustal layer, and the layer in which liquefaction has occurred. Seed (1979) noted that layers of fine-grained material located within liquefied soils would completely alter the way that pore pressures dissipate during and after strong shaking. Through a series of laboratory experiments, Kokusho (1999) showed that the presence of impermeable layers can lead to the formation of a water film, which might subsequently trigger large lateral spreading displacements. A number of small-scale 1g shake table experiments have demonstrated the formation of sand boils in a laboratory environment and with soil profiles which included impermeable layers at depth (i.e. Brennan et al. 2008, Miles et al. 2018). Both Brennan et al (2008) and Miles et al. (2018) observed that water films developed below the impermeable layer, but would later break through the layers, allowing for dissipation of excess pore water pressure from the layers below. Brennan et al (2008) reported “silt clouds” (i.e. material from the less permeable layer) forming in the liquefiable layer above. An important question which appears unresolved is the resistance of these impermeable layers to the fluid being driven upwards by hydraulic gradients. Considering the case of a clay layer, three mechanisms by which pore fluid passes through a relatively impermeable layer can be readily identified: bending and consequent cracking of a clay layer (perhaps driven by uneven surface loads); a series of hydraulic fractures, or localised extrusion of material; or gradual seepage. The first two mechanisms would be expected to allow rapid flow from the underlying layers, contributing to increased severity of liquefaction (i.e. Cubrinovski et al. 2017).

This paper describes a preliminary series of laboratory experiments which aim to investigate the ability of weak clay layers to resist differential pressures across the layer, which have the potential to cause bending/cracking, fracturing, seepage, or some other failure mechanism.

2 EXPERIMENTAL SET-UP

Three materials were used in the course of this study: Commercially available greywacke river gravel ($D_{50} = 3.60$ mm, $D_{60}/D_{10} = 1.1$), and New Brighton sand ($FC = 2\%$, $D_{50} = 0.14$ mm, $D_{10}/D_{60} = 1.9$) were used as overlying layers for the clay layer, while commercially available “Eckafine BDF” kaolin clay (92% finer than $2\mu\text{m}$) was used as the fine grained layer. Permeability of the clay layer is estimated at 1×10^{-8} m/s based on oedometer tests, while shear box testing suggested that the clay had an undrained strength of 5kPa at a vertical effective stress of 20kPa under normally consolidated conditions (i.e. $s_u \approx 0.25\sigma'_v$).

Experiments were conducted within a standard permeameter and consisted of a base sand layer, a clay layer and in some experiments an upper soil layer composed of either sand or gravel. Given the number of different configurations, tests will be referred to using a 3-part system. The first part indicates the state of the clay layer which is either unconsolidated slurry (U) or consolidated to a vertical effective stress of 20 kPa (C). The clay layers are prepared to a nominal thickness (t_{clay}) of 1, 5 or 10 cm immediately prior to testing. The second part indicates that the upper soil layer is either non-existent (N), sand (S) or gravel (G). The final part refers to the boundary condition which is either free (N), edge restrained (E) or fully fixed (F). An explanation of the boundary conditions is provided at the end of the description of the models. As an example, a test code of “U5-S-F” means that the test was performed with a 5 cm thick unconsolidated clay layer, which is overlain by a sand layer that is subject to the fully fixed top boundary condition. A schematic of this test configuration is shown below in Figure 1.

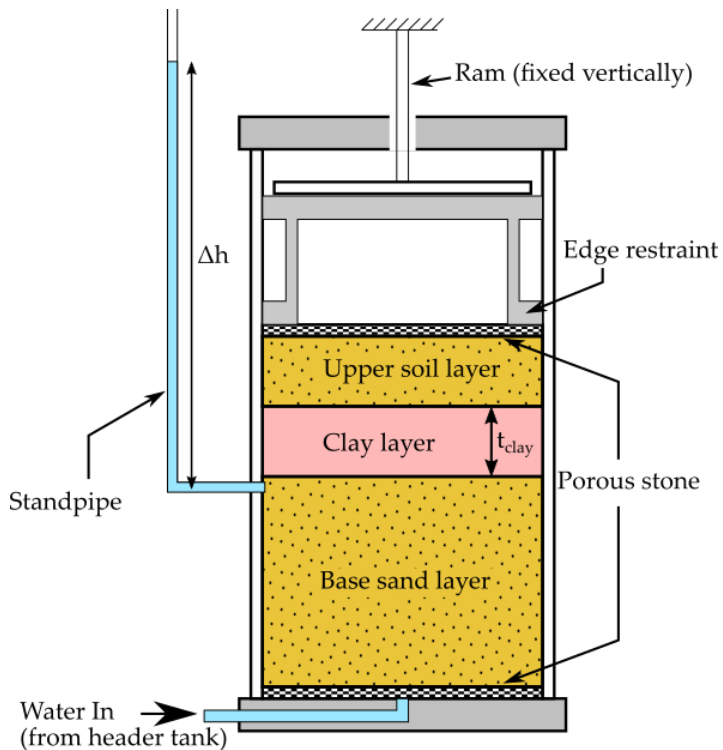


Figure 1: Experimental set-up (with upper soil layer, and “fixed” boundary condition).

In each experiment, a bottom layer of sand was created in lifts of 2cm by wet sedimentation (sprinkling dry sand through a water column), followed by shaking to densify the sand and create a level surface. The sand layer was constructed to a height of approximately 140 mm such that the top of the sand layer was coincident with the top of the middle standpipe. After completion of the sand layer, the water level inside the permeameter was lowered to the top of the sand layer.

The clay was prepared as a slurry at a water content of 160 %. The slurry was then carefully placed inside the permeameter using a piping bag to the specified thickness. In tests where the clay was consolidated, a porous stone was placed on top of the clay slurry and loads gradually applied (doubling the load each time and allowing sufficient times for the clay to reach 90 % of consolidation) until 20kPa was being applied to the clay. After consolidation, the porous stone was removed from the top of the clay layer.

In tests with an upper soil layer, the sand or gravel was carefully placed on top of the clay layer in small amounts (to reduce mixing with the clay layer) to a total thickness of 3 – 4 cm.

Three different boundary conditions were applied to the top surface of the model: free (i.e. surface not restrained in any way), edge restraint and fully fixed. To provide edge restraint a circular ring was placed on the top surface of the model. The ring was prevented from upwards displacement using the permeameter’s adjustable ram. The fully fixed condition was imposed by placing a porous disc on top of the soil surface, with the same edge ring on top.

During the experiment, water was fed from a header tank into the base of the permeameter. A standpipe connected to the middle port was used to measure the pressure acting on the base of the clay layer. It was assumed that in all configurations, the head loss above the clay layer was minimal up to the point of failure and therefore the pressure difference acting across the clay layer is obtained directly from the head measurement. The pressure acting on the base of the clay layer was gradually increased (from zero) until failure of the clay layer was observed, or the head reached the top of the standpipes. “Failure” of the clay layer was determined visually. In cases where there was no overlying soil layer, the failure point was determined when either the clay was deforming (and continued to deform) under a steady pressure. In cases

where there was an overlying soil layer, it was not possible to directly observe deformations, so failure corresponded to a sudden increase in flow out of the top surface.

3 RESULTS

A summary of the pressures at the base of the clay layer, (as well as the thickness of the clay layer in the test) at the point of failure for the different test configurations is shown in Table 1. It should be noted that for most configurations, three different clay layers were tested, such that in the first five rows, there are three results per row.

Table 1: Test Results

Configuration	t_{clay} (cm)	p_{fail} (kPa)	t_{clay} (cm)	p_{fail} (kPa)	t_{clay} (cm)	p_{fail} (kPa)
U-N-N	1	NM	5	NM	10	NM
U-N-E	1	0.4	5	1.1	10	1.9
C-N-E	2.5	2.0	5	4.9	9	8.8
U-S-E	1	1.3	5	4.0	10	7.2
U-S-F	1	NF	5	NF	10	NF
C-S-F	1	NF	-	-	-	-
U-G-E	1	0.8	-	-	-	-

In the first set of tests, U-N-N, the test procedures were still being refined and the pressures at the point of failure were not clear, hence “NM” refers to no measurement. In this series, failure occurred at very low pressures, and it was observed that the clay layer would initially displace upwards, with a gap forming below the clay, followed by a rotation of the clay layer. Once the clay had rotated, water flowed easily along the sides of the permeameter.

Tests without an upper soil layer, but with edge restraint showed failure pressures increasing with the clay layer thickness, and with higher failure pressures when the clay had been consolidated, as shown in Figure 2. In these tests, it was observed in all cases that a “plug” of clay would be extruded from the inside of the restraining ring, as shown in Figure 3 a. As might be expected, the consolidated clay layer showed a significantly larger failure pressure in these tests than the equivalent unconsolidated clay slurry. Importantly, for all three thicknesses of clay layer tested, and both strengths, the overall mechanism of failure did not change. This test series was extended by performing three tests on unconsolidated clay overlain by a sand layer. The failure pressures in Figure 2 lie between that of the U-N-E and C-N-E tests – the sand layer has increased the failure pressure for the same strength of clay. The failure mechanism in these tests is much less obvious, but as shown in Figure 3 b, it appears that the clay has again failed in a plug-type manner. The additional pressures required to cause the failure is relatively large in comparison to the failure pressures at equivalent clay layer thicknesses in the U-N-E tests. It is assumed that this is the result of arching (against the edge restraint) in the sand layer.

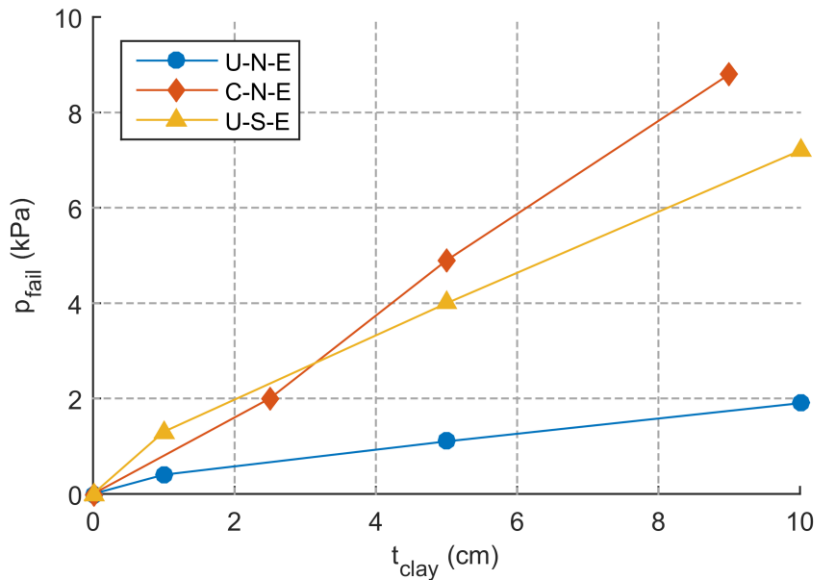


Figure 2: Failure pressures for tests with edge supported boundary conditions.



(a) No upper soil layer



(b) Sand upper layer

Figure 3: Plug-type failure in tests with edge supported boundary conditions

In the test series U-S-F and C-S-F (only one test), a porous stone had been placed on the sand surface, and prevented from displacing vertically. In all of these tests, regardless of the layer thickness, the water in the standpipe reached the top of the standpipe (approx. 2m of head), without an obvious “failure,” even when the clay was unconsolidated and only 1cm thick. Instead, the water gradually seeped through (and around the edges) of the clay layer.

The final test in configuration U-G-E was performed with a 1cm clay layer only. In this test, it was observed that the clay moved upwards into the gravel layer. Eventually, failure occurred when a leak path formed through the gravels.

4 DISCUSSION

The preliminary tests with a free top surface condition, have indicated that even with very weak clay, the clay layer tended to lift upwards rather than develop a hydraulic fracture. This mechanism would also be expected in cases with much stronger clay. Similarly for the layers with edge support (with and without an

overlying sand layer), the mechanism being observed is linked to a shear failure rather than a hydraulic fracturing of the layer. These two types of test configuration might be linked to the behaviour of a clay layer at depth where the soil both above and below the layer have liquefied since a fully liquefied soil layer will tend to offer little resistance to deformation in the clay layer. It must be recognised that the edge restraints being applied in these tests do not represent a likely field condition and were imposed in these tests to prevent the rotation of the clay layer that was observed in the first series of tests. In the field, if the layer geometry is not uniform, or there are locations where the pressures being applied to the top of the layer are not uniform, or the layer has variable thickness, then it is likely that the clay layer would instead deform in bending, with cracks potentially forming as a result, which would allow the break-through of pore fluid from the underlying liquefied layer. It should also be stated that in these tests, the aspect ratio of the clay layer to the diameter of the permeameter was small (1:5 in the case of the thinnest clay layer). In real scenarios, the aspect ratio would be much larger, which again would tend to favour failures by bending stresses rather than the plug type seen in these tests.

The tests carried out with the fixed boundary condition revealed that if the clay layer is unable to distort (either bending or extruding through the overlying layer) then it is not expected that the pore water could break through the layer, even with relatively large pressure differences relative to the strength and thickness of the clay layer (i.e. hydraulic fracture did not eventuate). This implies that in the case where a clay layer is underlain by a liquefied sand layer, but confined above by another layer which has not liquefied (and therefore able to resist distortion from below), then the pore pressure will dissipate by seepage only (i.e. a slow process), or laterally to a different location which may be more vulnerable. It should however be noted that in this case, it would also be expected that a water film would tend to form beneath the clay layer. If stresses in the overlying profile are non-uniform, then loss of support from below the clay layer due to liquefaction and formation of a water film may then result in the clay layer distorting, with the potential for cracks to form. In this case, the distortion may allow for the fluid to break through the overlying clay layer. However, this issue has not been investigated here.

In all of the tests which were carried out with either the fixed or edge restraint boundary conditions, the pressures beneath the clay layer were allowed to rise so that they were much larger than the vertical stresses developed due to the weight of the clay layer itself. It should be noted that in the case of a clay layer sandwiched between two liquefied soil layers that the pressure difference across the layer (which might have caused failure) is limited to the stresses from the weight of the clay for vertical equilibrium. This means that the pressures which have been applied in these tests are therefore larger than those which would be possible in the field for layers of similar thickness.

Finally, the tests and associated discussion have so far only considered the period following the end of strong ground motion, and assumed that the clay layers are intact at the start of this point. The results have however suggested the importance of clay layer distortion, leading to cracking or shear failures, which then allow the pore fluid to break through the layer. In this context, it should not be forgotten that the strong ground motions are inducing significant dynamic stresses, which may well cause co-seismic deformation of the clay layers, such that the initiation of pore fluid breaking through a clay layer may well begin during the strong motion itself. This aspect has not been considered in the tests described and further research is required to address this aspect.

5 CONCLUSION

Clay layers play an important role in containing liquefied soils and preventing the manifestation of sand boils at the ground surface. Results from a series of tests which attempted to investigate the pressure differences required for pore fluid to break-through a clay layer have been presented. The tests showed that in the cases where the clay layer is unable to distort (fixed boundary condition), it was not possible to cause a break-

through of the clay layer by hydraulic fracturing or bending/cracking/shear, and pore fluid can only seep through the clay layer. However, in the cases where the clay layer was restrained at the edges (and therefore able to deform), a plug type failure was observed, highlighting the role of distortion/bending in enabling pore fluid to break through a clay layer.

It is important to note that the tests described in this paper were carried with both very weak clay and thick clay layers relative to the diameter of the testing apparatus. With stronger clay layers, different materials, and larger model cross-sections, it is possible that the mechanisms observed will change. It is also the case that this test series considers only the post-seismic period, assuming that the clay layers have remained intact. Further research is required to consider these effects of material and strong motion shaking.

ACKNOWLEDGEMENTS

The authors wish to acknowledge the help and support of Siale Fiatatonu during this project.

REFERENCES

- Brennan, A.J. (2008) Observations on sand boils from simple model tests. In Zeng, X, Manzari, M. & Hiltunen, D. (eds). *Geotechnical Earthquake Engineering and Soil Dynamics IV. GSP181, Sacramento, 18-22 March 2008*, ASCE.
- Cubrinovski, M., Taylor, M., Robinson, K., Winkley, A., Hughes, M., Haskell, J., & Bradley, B. (2014), Key factors in the liquefaction-induced damage to buildings and infrastructure in Christchurch: Preliminary findings, in Wotherspoon, L. & Ma, Q. (eds), *NZSEE Conference, Auckland, 21-23 March 2014*, NZSEE, Paper Number O78, 2-8.
- Cubrinovski, M., Rhodes, A., Ntritsis, N. & Van Ballegooy, S. (2018), System response of liquefiable deposits. *Soil Dynamics and Earthquake Engineering*, In-Press. <https://doi.org/10.1016/j.soildyn.2018.05.013>
- Boulanger, R. W. & Idriss, I. M. (2015). CPT-based liquefaction triggering procedure. *Journal of Geotechnical and Geoenvironmental Engineering*. 142(2): 04015065
- Ishihara, K. (1985) Stability of natural deposits during earthquakes. *Proc. 11th Int. Conf. Soil Mech. Found. Engineering, San Francisco 12-16 August 1985*, Rotterdam: A.A. Balkema, 321-376.
- Kokusho, T. (1999). Water film in liquefied sand and its effect on lateral spread. *Journal of Geotechnical and Geoenvironmental Engineering*. 125(10) 817-826
- Maurer, B.W., Green, R.A., Cubrinovski, M. & Bradley, B.A. (2014) Evaluation of the liquefaction potential index for assessing liquefaction hazard in Christchurch, New Zealand. *Journal of Geotechnical and Geoenvironmental Engineering*, 140(7):04014032
- Miles, S, Still, J, & Stringer, M. (2018) Investigating the effects of layering on the formation of sand boils in 1g shaking table tests. In McNamara, A., Divall, S., Goodey, R., Taylor, N., Stallebrass, S. & Panchal, J. (eds) *Physical Modelling in Geotechnics 2018, London, 17- 20 July 2018*. London: Taylor & Francis. 999-1004
- Seed, H.B. (1979) Soil Liquefaction and Cyclic Mobility Evaluation for Level Ground During Earthquakes. *ASCE Journal of the Geotechnical Engineering Division*. 105(GT2): 201-255.