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The SCIRT and EQC Liquefaction Trial – Learning's from the Field Testing of Below Ground Infrastructure Within Liquefied Soils

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ABSTRACT

Christchurch's infrastructure was severely damaged by repeated liquefaction during the 2010/2011 Canterbury Earthquake Sequence. The SCIRT and EQC Liquefaction Trial was implemented to provide controlled field assessment of the performance of below ground infrastructure within liquefied soils and validate revised standards and proposed concepts for changes to those standards. Performance improvements were investigated by comparing; infrastructure material selection, design detailing, and backfill type. Explosives were used to trigger liquefaction of the soil surrounding the buried infrastructure. The trial only considered the vertical effects of liquefaction; liquefaction induced settlement, bearing capacity and buoyant uplift. The performance of the infrastructure was assessed by visual observation, instrumentation, exhuming the infrastructure, and supported by laboratory testing. This paper presents and discusses SCIRT's learnings associated with buoyant uplift of buried chambers through interpretation of the trial observations. Recommendations are made on how to improve the relative resilience of buried chambers in liquefiable soils from these learnings.

Introduction

The city of Christchurch, New Zealand, and surrounding areas were subjected to very strong ground motion during the Canterbury Earthquake Sequence (CES) through 2010 and 2011. Extensive and repeated liquefaction led to significant damage to Christchurch's infrastructure. The Stronger Christchurch Infrastructure Rebuild Team (SCIRT) was established in response to the extensive damage sustained during the 22 February 2011 Christchurch Earthquake (M_w6.2). The SCIRT alliance was tasked with the assessment and repair of earthquake damaged horizontal (wastewater, stormwater, water supply and roading) infrastructure, creating a legacy of resilient infrastructure, whilst also providing value for the client organisations.

Observations from the earthquakes informed amendments to the Christchurch City Council (CCC) infrastructure design standards (CCC, 2013) and construction standard specifications (CCC, 2014) incorporating theoretical improvements for earthquake resilience. Improvement was incorporated through pipe and chamber material selection, design detailing and backfill material type. SCIRT identified an opportunity to undertake full scale field trials to assess the effects of liquefaction on below ground infrastructure in a controlled and closely monitored field situation. This was performed in parallel with the Earthquake Commission's (EQC's) series of full scale ground improvement field trials. Information gained from the SCIRT and EQC liquefaction trial was used to validate theory, assess infrastructure performance, and to understand severity of risk and consequence of failure mechanisms. The results would inform SCIRT designers in reviewing

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the resilience of CCC standard details and proposed alternatives.

A wide range of observations and knowledge was drawn from the trial. Gibson & Rowland (2015) present background and detail of the trial design, providing commentary on observed seismic performance, and summarises high level learnings gained from the trial. This paper focuses on the learnings associated with backfill material type to mitigate potential chamber uplift, correlating trial observations with field observations during the CES and theory/laboratory testing by others.

Site location and ground conditions

The trial was performed on CERA residential red zone land at 31 Ardrossan St in the suburb of Avondale. The area exhibited poor performance during the CES with significant liquefaction and ejecta, lateral spread and liquefaction induced settlement due to post liquefaction volumetric reconsolidation observed. Horizontal infrastructure and residential dwellings in the area were extensively damaged.

The subsoil conditions typically comprised up to 1 m of non-engineered and highly variable sandy silt and silty sand fill, with some pockets of gravel. The fill was underlain to a depth of 1.5 m to 2.6 m below ground by alluvial over-bank deposits of the Springston Formation comprising variable silty sands and sandy silts, with some layers of silt with clay like behaviour identified between 2.2 m and 2.6 m depth. The remainder of the near surface soil profile is dominated by loose to medium dense clean sands (<10% fines) of the Christchurch Formation. The near surface groundwater level was at 1.1 m depth, associated with perched water tables.

Trial design

Trial design attempted to maximise the range of infrastructure tested within the site area and budget constraints, so that maximum value could be realised. The trial comprised eight different tests; Gibson & Rowland (2015) explain the different tests in detail. This paper focuses on the buried chambers which are identified as Tests 3 to 7. Liquefaction was triggered by detonating a sequence of 42 charges in 14 blast holes arranged in two overlapping 10 m diameter circles around the tests. The explosive design and sequencing was developed by the EQC trial technical team. An indicative site layout plan is shown in Figure 1, and details of Tests 3 to 7 are summarised in Table 1.

The five chamber designs tested covered three general methods to mitigate uplift; [1] improve the ground surrounding the chamber to limit excess pore pressures (Test 5 and 6), [2] provide a means of drainage to allow rapid dissipation of excess pore water to reduce/limit the uplift pressures, (Test 3), [3] resist uplift by adding mass to the buried chamber (Test 7).

Pore pressure transducers (PPT) were installed at different levels though the ground profile and directly beneath the chambers. These measured excess pore pressures, confirm liquefaction triggering, and determine chamber uplift pressures. Vertical movement of the chambers and adjacent ground was quantified though comparison of baseline monitoring and post liquefaction elevation changes recorded by: survey and LiDAR digital elevation model (DEM).

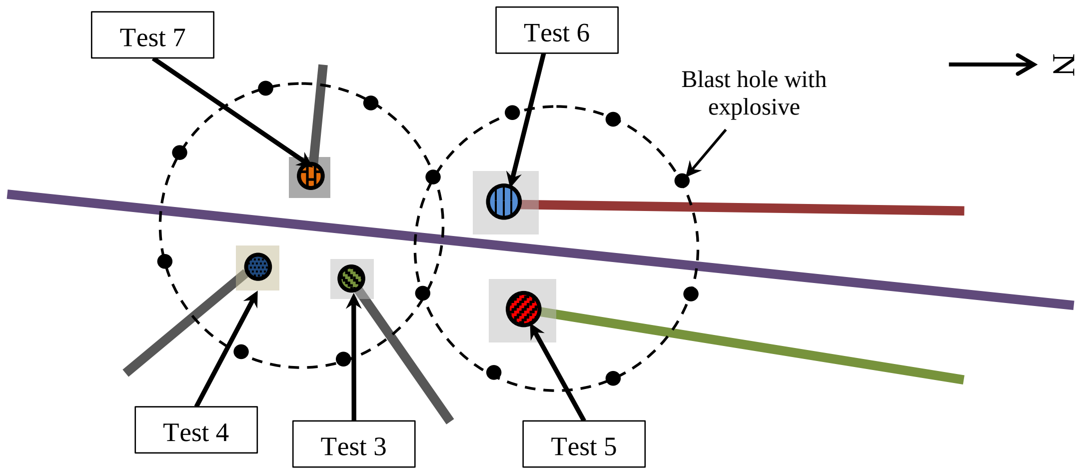


Figure 1. Site layout for SCIRT and EQC liquefaction trial

Table 1. Description of chamber tests discussed in this paper

Test	Chamber	Backfill	Test Purpose
3	PE Pressure Sewer	NZTA M6 Grade 2 chip encased within a sewn geotextile bag	To determine the impact of highly permeable backfill materials on uplift and buoyancy
4	PE Pressure Sewer	Excavated silty sand materials	Provides a baseline for comparison of other tests against natural materials
5	1050mm dia concrete	CCC AP65 gravel	To determine the effects of well graded backfill on uplift and buoyancy of a concrete chamber
6	DN600 PE access	CCC AP65 gravel	As for Test 5 but with a PE chamber
7	PE Pressure Sewer	Low strength concrete	To determine the effects of low strength concrete backfill on uplift and buoyancy

Buoyant uplift theory

Interpretation of trial observations and data requires consideration and comparison against current state of the art theory on the processes influencing buoyant uplift. Buoyant uplift displacement of a structure results when the Factor of Safety (FoS) to uplift reduces to below unity. This occurs when the static uplift force (F_B) and elevated buoyant uplift force associated with excess pore water pressure (F_{EPP}) exceeds the resistance provided by the weight of the structure (F_T), soil weight (F_{WS}) and shear strength of the overlying soil (F_{SP}), (Chian & Madabhushi, 2013). The FoS against buoyant uplift can be calculated using Equation 1.

$$FoS = (F_B + F_{EPP}) / (F_T + F_{WS} + F_{SP}) \quad (1)$$

Koseki, Matsuo & Koga (1997) investigated the uplift mechanism caused by liquefaction of the surrounding soil for a variety of underground structures through scaled laboratory testing on a shake table. This study confirmed that utilising a FoS of equilibrium of vertical force acting on the structure was a reasonable method of assessing uplift potential. This was supported by further laboratory centrifuge testing performed by Sasaki & Tamura (2004), Kang, Tobita, Iai & Ge (2013), Chian, Tokimatsu & Madabhushi (2014).

The pore water pressure exerted on a submerged structure quickly increases just below the threshold for triggering of liquefaction. Liquefied soil does not exhibit a hydrostatic pressure distribution initiating at the groundwater table, but is equivalent to the total stress pressure distribution for the soil. Development of excess pore pressures leads to a reduction in the FoS against buoyant uplift due to the following reasons (Koseki et al., 1997):

- Elevated buoyant uplift force associated with excess pore water pressure (F_{EPP}) induced within the surrounding soil by strong ground motion
- Reduction of resistance associated with soil shearing due to reduction of effective stress
- Reduction in the effective weight of overlying soil
- Application of seepage forces associated with migration of excess pore water from soil layers of a greater depth than the structure migrating upward towards the ground surface

A schematic of the failure mechanisms and force components is provided in Figure 2.

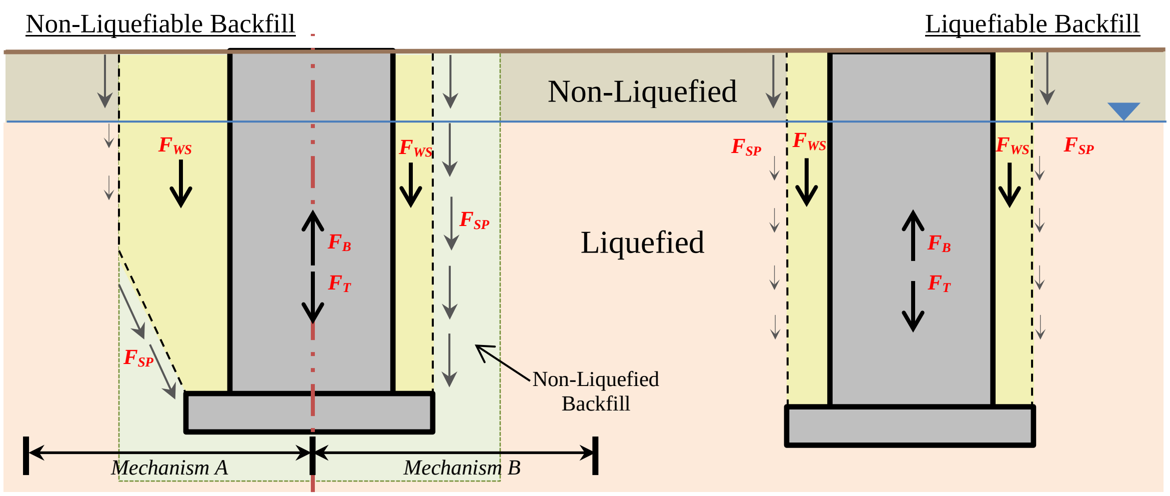
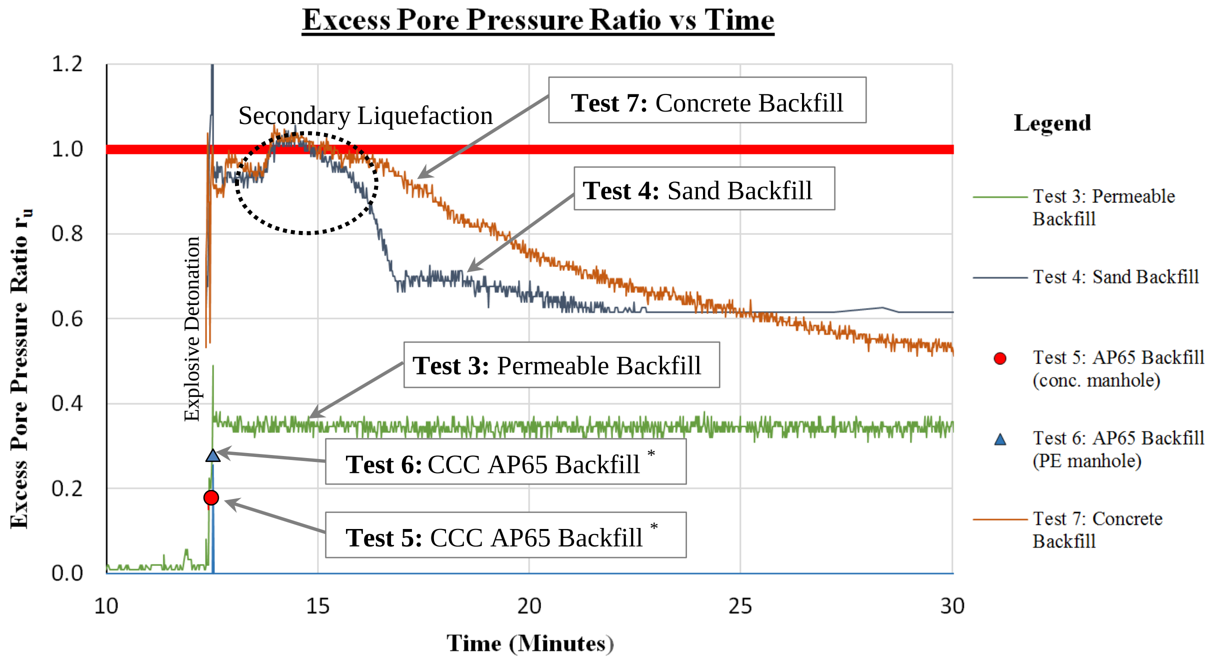


Figure 2. Schematic of typical failure mechanisms for liquefied and non-liquefied backfill

Trial observations

Excess pore pressures recorded by the PPTs allowed measurement of buoyant uplift pressures on the buried chambers and provided confirmation that liquefaction was triggered. Excess pore pressure was normalised to an excess pore pressure ratio (r_u) to simplify the assessment. Liquefaction is effectively triggered when r_u approaches 1 (uplift pressure equal to initial total stress). Explosive detonation at the adjacent EQC ground improvement trial at 29 Ardrossan St 10 seconds prior to the SCIRT denotations provided an initial increase in pore pressures aiding the triggering of extensive liquefaction. Review of PPTs at various depths in the surrounding native soils confirmed that the explosives triggered extensive liquefaction down to the deepest installed at 9 m. PPTs within 2-3 m of the ground surface recorded varying levels of excess pore pressures, indicating liquefaction triggering levels had been reached (or close to). Limited liquefaction ejecta was observed at the ground surface, with the colour of the entrained sands suggesting their source to be from below 3 m depth. The LiDAR DEM of ground settlement at the site inferred moderate to extensive liquefaction extended up to 5 m beyond the blast circles. Different ground response from varying subsurface condition let to typical settlements of 100-150 mm within the northern blast circle and minor settlements of 0-80 mm in the southern blast circle; even though liquefaction was confirmed by the PPT and surface expressions of ejecta.

Excess pore pressure ratio with time measured beneath each of the chambers is presented Figure 3. Following the detonation of explosives a liquefied state was maintained in the native soils immediately beneath the foundation of chambers (Test 4 and Test 7) for up to 5 minutes as excess pore water migrated upward from the soil strata below. Delayed secondary liquefaction was observed in PPT's installed in native soils (Test 4 and Test 7) two to three minutes following explosive detonation.



* Discrete Points shown as <10 data points measured exceeded the sensitivity threshold for the PPT.

Figure 3. Excess Pore Pressure Ratio for PPT installed beneath chambers

No observations of chamber uplift displacement above original elevation were recorded during the Liquefaction Trial. The chamber displacement matched the inferred post liquefaction settlement of the underlying ground, within an error tolerance of less than ±15 mm.

Assessment and interpretation

Influence of backfill type on uplift pressures

The magnitude of excess pore pressure observed in this trial supports theoretical assessment and laboratory testing by researchers including Tobita, Kang & Iai (2012). An important observation in Test 4 and Test 7 founded on native soil was that the pore pressure measured beneath the chamber when liquefied was equivalent to the initial total stress. When the base of a chamber is founded within a liquefied layer the magnitude of the excess pore pressure within the native soil directly beneath a chamber are not affected by the extent of liquefied soils above.

The delayed increase in uplift pressures within the native soils from migration of excess pore pressure to the ground surface (secondary liquefaction) inferred uplift pressure that would be similar to marginally higher than would be generated by initial liquefaction of the adjacent soils.

The recorded excess pore pressure recorded was up to 2% greater than the initial effective stress. The SCIRT field trial observations support the laboratory testing by Sasaki & Tamura (2004) which observed that seepage forces were minor, being <5% of the total uplift force (F_B + F_{EPP}).

The chamber encapsulated in permeable backfill (Test 3) was successful at limiting uplift pressure. High permeability of the backfill limited the uplift pressure beneath the chamber to a static water head at the ground surface. The resilience of drainage to improve seismic performance of a buried chamber is dependent on maintaining a high permeability condition and ability to drain to the ground surface. Exhuming the Test 3 chamber found that the sewn geotextile bag was successful in preventing ingress of ejecta sands into the backfill, and the geotextile was free of any silt/ clay coating.

Test 5 and Test 6 chambers were backfilled with well graded granular backfill (CCC AP65). The CCC AP65 is derived from quarried alluvial deposits in Canterbury and is typically characterised as having a moderate to high fines content and variable permeability, similar to the adjacent native soils ($1x10^{-7}$ m/s to $1x10^{-4}$ m/s). The maximum r_u recorded for the PPT installed within the CCC AP65 was 0.18 and 0.29 for Tests 5 and 6 respectively. Excess pore pressures exerted on the chamber from liquefaction were <30% of the excess pore pressures in the surrounding liquefied native soils. Trace infiltration of ejecta materials from soil strata below was observed during exhumation. The well graded gravel did not liquefy and external excess pore pressures within the native soils were attenuated within the well graded granular backfill. The assumption often made during design, that the uplift pressure within a granular backfill is equivalent to the excess pore pressure within the adjacent native soils, may be overly conservative. The authors are currently implementing further testing to determine the validity of this observation by comparing the pressure attenuation with time and duration of shaking. The authors recommend caution in directly adopting the recorded observations from Test 5 and 6 until validation is established.

Buoyant uplift potential

Theoretical assessment of potential chamber buoyant uplift was preformed considering the 'as-built' condition and measured uplift pressure. Table 2 summarises the theoretical FoS against buoyant uplift calculated in accordance with the method provided in Equation 1, and the potential failure mechanisms presented in the Figure 2 schematic. Analysis indicates that Test 4 and possibly Test 7 exhibited potential for uplift. The extended base incorporated into chambers in Test 3, 4, 5, and 6 was effective in utilising the soil weight and shear strength of the overlying soil to resist uplift. Limiting the net vertical excess pore pressure applied to the chambers, though use of permeable backfill and well graded granular backfill (CCC AP65), was effective at increasing the liquefied buoyant uplift FoS. Test 7 supported theory which suggests that the benefits of adding impermeable mass to a structure (below ground) to resisted buoyant uplift is limited once a FoS of 1 is achieved.

Table 2: Anticipated chamber uplift compared with observed uplift

Test	Chamber Type	Backfill Type	FoS Liquefied Buoyant Uplift	Uplift Anticipated	Uplift Observed
3	PE Pressure Sewer	Permeable backfill	2.4	No	No
4	PE Pressure Sewer	Native sands	0.9	Yes	No
5	1050mm dia concrete	CCC AP65	4.1	No	No
6	DN600 PE access	CCC AP65	5.1	No	No
7	PE Pressure Sewer	3MPa Concrete	1.0	Possible	No

Chamber uplift displacement

No observations of chamber uplift were recorded during the trial. Laboratory testing by Koseki et al. (1997) observed that uplift displacement triggered when FoS reduced to between 0.7 to 0.95 during the period of shaking, and continued until a FoS of close to 1 was achieved. This was supported by Sasaki & Tamura (2004) who identified the uplift displacement rate was nearly constant during shaking, with movement observed to almost stop upon cessation of shaking, even if elevated pore pressures were maintained. Negligible uplift for Test 4 and Test 7 observed during the SCIRT trial is considered to be the consequence of the short shaking duration where FoS for uplift was marginally below unity for less than five seconds. Theoretical uplift displacement was estimated for Test 4 and Test 7 by the method proposed by Sasaki & Tamarua (2004), indicating the potential displacement to be less than 90 mm (15 – 90 mm).

Often a buried structure has an initial weight that is less than the weight of the native soil displaced; this leads to a lower total stress directly beneath the structure than adjacent native soil at an equivalent elevation. Therefore excess pore pressures generated beneath the structure are lower than in the adjacent liquefied native soil. Koseki et al. (1997) observed during testing that the horizontal pressure gradient induces a flow of liquefied sand towards and beneath the structure during uplift, a minor flow of 5-10 mm of sand was observed beneath Test 4 and Test 7.

Typical earthquake damage to underground structures, such as during the 1995 Hyogoken-Nanbu earthquake, was the result of earthquake induced ground displacement such as lateral stretch, settlement and shaking inertial force; surprisingly damage associated with buoyant uplift was insignificant (Sasaki & Tamura, 2004). The same mechanisms were observed by the authors to be the main cause of damage to below ground infrastructure during the CES, similar observations were also made by Cubrinovski et al. (2014). Many manholes and pump stations were observed to protrude above the ground surface. Review of Christchurch manhole performance by Menefy & Scally (2013) identified that only 3.5% to 5.5% of their dataset exhibited relative displacements in excess of 150 mm. Where differential movement is minor, it is best explained by post liquefaction volumetric reconsolidation of the soil above foundation level. Cases of buoyant uplift were observed during the CES, however this was largely associated with large lightweight deep structures, often with eccentric loading or inconsistency in foundation arrangement across the structure. The duration of the 22 February 2011 earthquake was 10-14 seconds and the estimated magnitude of buoyant uplift displacement for a typical manhole within liquefied backfill is of the order 25 - 250 mm (Sasaki & Tamura, 2004).

Sasaki & Tamura (2004) postulated that limited observation of buoyant uplift failures of underground structures may be due to current evaluation methods during design being conservative. The trial indicates that the SCIRT and CCC standard details adopted for the Christchurch Rebuild for minor below ground structures, comprising standard concrete manholes, and PE chambers, are anticipated to exhibit satisfactory resilience with respect to buoyant uplift.

Conclusions and Recommendations

The SCIRT and EQC Liquefaction Trial provided a controlled field assessment of the performance of below ground chambers in liquefied soils. Interpretation of the trial observations and data supports geotechnical design theory, anticipated relative performance and failure mechanisms. The performance of the buried infrastructure in the trial is in line with the generally good performance observed during the CES. The trial learning's support the resilient design solutions incorporated into the SCIRT rebuild of horizontal infrastructure.

The authors recommend focusing on simple and reliable designs for buried chambers as this provides improved resilience through lower sensitivity to construction tolerance and/or quality, and reduces future maintenance. The trial showed that an extended chamber base that utilises the effective weight of backfill materials was efficient at resisting uplift. Gravel backfill was found to be the most effective at reducing buoyant uplift pressure, the use of native sandy soils for backfill is not recommended unless stabilised to prevent liquefaction. Highly permeable backfill within a sewn geotextile bag was effective at limiting uplift pressures; however resilience is dependent on preventing migration of fines into the backfill. Well graded granular backfill exhibited low excess pore pressure during the trial; this suggests that current assumptions made during design of uplift pressure equivalent to total stress of native soils may be conservative. The well graded granular backfill is a pragmatic solution, though further laboratory testing is required before a reliable conclusion can be drawn.

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