SCIRT Rebuilding Infrastructure LEARNING LEGACY

Lessons learned from one of New Zealand's most challenging civil engineering projects: rebuilding the earthquake damaged pipes, roads, bridges and retaining walls in the city of Christchurch 2011 - 2016.

SCIRT and EQC Liquefaction Trial Report

Story: Liquefaction Trial Report

Theme: Design

A report detailing the Liquefaction Trial, the observations and discussions of the trial interpretation and findings.

Programme funded by Christchurch City Council New Zealand Government

City Care, Downer, Fulton Hogan, McConnell Dowell, Fletcher

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Liquefaction Trial Report

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Revision History

Revision	Date	Name	Brief Description
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Abbreviations

Abbreviation	Description
BH	Borehole
CCC	Christchurch City Council
CCTV	Close circuit television
CERA	Canterbury Earthquake Recovery Authority
CES	Canterbury earthquake sequence
CPT	Cone Penetration Testing
CSS	CCC Construction Standard Specifications
DEM	Digital elevation model
DN	Diameter nominal
EQC	Earthquake Commission
HDPE	High-density polyethylene
IDS	Christchurch City Council Infrastructure Design Standard
LiDAR	Light detection and ranging
MH	Manhole
NZTA	New Zealand Transport Agency
PE	Polyethylene
PGA	Peak ground acceleration
PPT	Pore pressure transducer
PVC-U	Unplasticised polyvinyl chloride
RL	Reduced elevation, to CCC Drainage Datum (MSL= 9.043mRL)
SCIRT	Stronger Christchurch Infrastructure Rebuild Team
VSVP	Cross hole S and P wave velocity testing

Executive Summary

Christchurch's below ground horizontal infrastructure was subjected to very strong ground motion during the Canterbury Earthquake Sequence (CES) through 2010/2011. Extensive and repeated liquefaction triggering led to heavy damage to Christchurch's infrastructure. Assessment during the wastewater, stormwater and water supply infrastructure rebuild identified that the most significant modes of failure were associated with differential settlement due to post liquefaction volumetric reconsolidation, lateral spread, dynamic structural failure, and in some instances buoyant uplift. Subsequently the Christchurch City Council (CCC) infrastructure design standards were amended to incorporate more conservative detailing aimed at providing greater earthquake resilience. Changes were made to pipe and chamber material selection, design detailing, and backfill material type.

The Stronger Christchurch Infrastructure Rebuild Team (SCIRT) and Earthquake Commission (EQC) Liquefaction Trial provides a controlled field assessment of the performance of below ground infrastructure in simulated liquefied soils. A range of pipes, chambers and backfill materials were assessed at the trial site of 31/31A Ardrossan St within the CERA Residential Red Zone in Avondale. EQC agreed to allow SCIRT to undertake the trial in parallel with the EQC ground improvement trial. The SCIRT trial was developed and implemented to assess design assumptions, and to observe and quantify the improvement in resilience provided by changes to the infrastructure design standards in a controlled and monitored field situation.

Liquefaction was triggered within the soils through a sequenced detonation of explosives within an array of boreholes. The EQC ground improvement trial technical team assisted with blast design and instrumentation, incorporating findings from the earlier phases of their work. Data captured by inspection throughout the trial (construction though to exhumation), survey monitoring and instrumentation is analysed and interpreted in this report. Discussions on detailed trial interpretation and findings are presented.

Interpretation of the trial observations and data supports geotechnical design theory of the anticipated performance and modes of failure. The performance of the buried infrastructure in the trial is in line with the examples of existing infrastructure that generally provided good observed performance in Christchurch during the CES.

The trial provides a legacy of evidence to support the resilient design solutions incorporated into the SCIRT rebuild, which are assessed to be pragmatic and practical, exhibiting an appropriate level of resilience and optimised value. The standard details used by SCIRT are appropriate for the majority of conditions in Christchurch and other locations which exhibit susceptibility to liquefaction. During a significant earthquake damage to Christchurch's buried infrastructure will occur, requiring repair or replacement. However, the resilient design improvements will provide a positive benefit in post disaster functionality and assist with enabling a more controlled and programmed rebuild.

1 Introduction

Christchurch's below ground horizontal infrastructure was subjected to very strong ground motion during the Canterbury Earthquake Sequence (CES) through 2010/2011. Extensive and repeated liquefaction triggering led to heavy damage to Christchurch's wastewater, stormwater and water supply infrastructure, as well as to roads and other facilities.

Assessment during the rebuild identified that the most significant mechanisms leading to earthquake damage of buried infrastructure were; differential liquefaction induced settlement due to post liquefaction volumetric reconsolidation, lateral spread, dynamic structural failure, and in some instances buoyant uplift. Subsequently the Christchurch City Council (CCC) infrastructure design standards were amended to incorporate more conservative detailing aimed at providing greater earthquake resilience. Changes were made to pipe and chamber material selection, design detailing, and backfill material type.

Stronger Christchurch Infrastructure Rebuild Team (SCIRT) identified an opportunity to undertake full scale field trials to assess design assumptions, and to observe and quantify the improvement in resilience provided by changes to the infrastructure design standards (Construction Standard Specifications, CSS). The Earthquake Commission (EQC) was preforming full scale ground improvement field trials within the Canterbury Earthquake Recovery Authority (CERA) Residential Red Zone, using a series of explosive charges to trigger liquefaction. EQC agreed to allow SCIRT to undertake a parallel field trial and provided access to the EQC ground improvement technical team to assist with blast design and instrumentation monitoring. The trial design and implementation was undertaken over a very tight timeframe of 7 weeks, liquefaction was triggered on the 24 October 2013.

This report documents the implementation of the SCIRT and EQC Liquefaction Trial, data gathered, observations and their interpretation. Learnings and comments of the resilience of below ground horizontal infrastructure currently being constructed as part of the Christchurch rebuild are provided along with recommendations for potential modifications to improve resilience.

2 SCIRT and EQC Liquefaction Trial

2.1 Purpose

The purpose of the Liquefaction Trial is to assess the effects of liquefaction on below ground infrastructure in a controlled and closely monitored field situation. Information gained from the trial will be used to validate theory, provide a field assessment of infrastructure performance, and to assess severity of risk and consequence of modes of damage. The trial results have informed SCIRT designers who are reviewing the appropriateness of current CCC standard details and proposed alternatives.

2.2 Limitations

The following limitations of the trial need to be considered during the assessment and interpretation of results:

- The Liquefaction Trial only assesses the vertical effects of liquefaction on buried infrastructure. Deformation and forces imparted onto buried infrastructure associated with the dynamic effects of strong ground motion and lateral spreading are not replicated in the trial.
- Recorded buoyant uplift displacement of chambers would likely be unconservative due to the short duration of shaking and influences of the surface crust.

- The statistical validity of the trial is low as it only preforms one test event, in a discrete location and for a specific set of buried infrastructure.

However, the test provides a good practical assessment which can be compared to theory and observations of infrastructure performance during the CES. Interpretation considering the limitations has informed decision making by SCIRT designers.

3 Trial Site

3.1 Location

The 620 m^2 SCIRT and EQC Liquefaction Trial site at 31 and 31A Ardrossan St is located in Avondale, on flat ground on the inside of a meander 70 m from the Avon River. The site and adjacent land is within the CERA Residential Red Zone. Figure **1** provides the location of the trial site.

Figure 1 **Location for the SCIRT and EQC Liquefaction Trial [Aerial photo flown 24 February 2011]**

3.2 Review of Historic Aerial Photography

Review of aerial photography of the trial site between 1941 and the present has identified that the land use at the site was rural farmland up until the early 1960's, then fill materials were slowly and progressively placed on this low lying floodplain on the inside of the river mender. Residential development accelerated in the early 1970's with ongoing subdivision.

3.3 Canterbury Earthquake Sequence

The area around the trial site exhibited poor performance during the CES with major to severe liquefaction and lateral spread observed.

3.3.1 Strong Ground Motion

The earthquake characteristics and estimated ground accelerations from the significant earthquakes during the CES are provided in Table 3-1.

Table 3-1 Strong Ground Motion experienced during the Canterbury Earthquake Sequence

Earthquake	Magnitude (M_w) (1)	Duration (seconds) (1)	Peak Horizontal Acceleration (2)
4 September 2010	7.1	~50	0.18g
22 February 2011	6.2	~13	0.36g
13 June 2011	6.0	~10	0.24g
23 December 2011	5.9	~8	0.30g

(1) Earthquake data sourced from www.geonet.org.nz

(2) Conditional Peak Ground Accelerations (PGA) at the site developed for conventional liquefaction assessments by Bradley Seismic Ltd. and the University of Canterbury, sourced from, https://canterburygeotechnicaldatabase.projectorbit.com

3.3.2 Canterbury Earthquake Sequence Land Damage Site Observations

Recent seismic activity has been observed to have liquefied the ground and induced settlement and lateral spreading at the trial site. The severity of land damage at the site and surrounding area was high, resulting in CERA designating the land residential red zone.

Key observations at the trial site and surrounding area during the CES were:

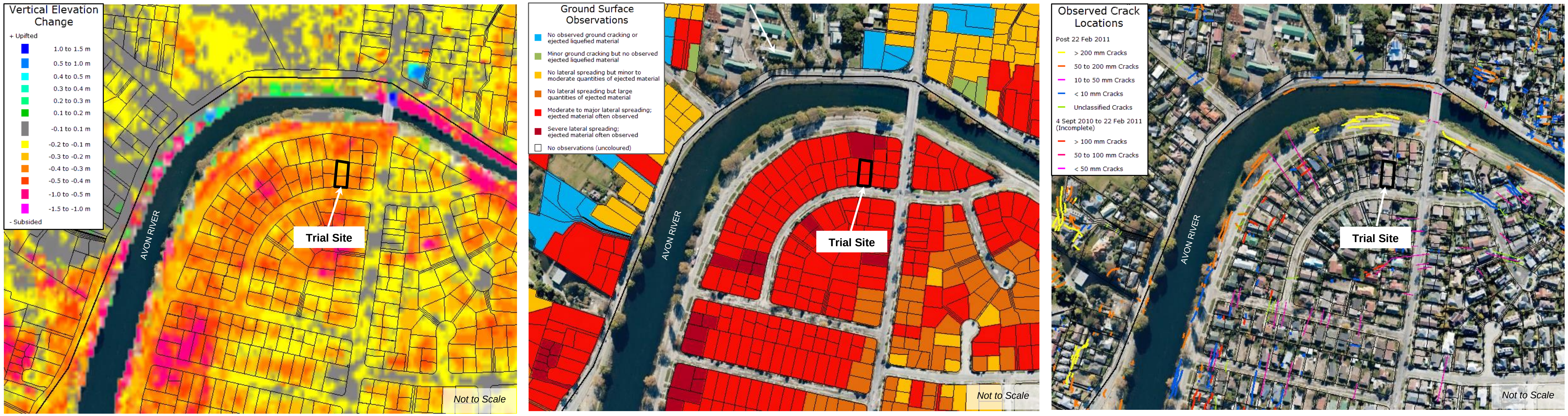
- Evidence of liquefaction with release of minor to major volumes of ejecta fine sands on the trial property, adjacent properties and road corridor, during all of the major earthquakes during the CES (Table 3-1).
- Moderate to severe lateral spread during the 22 February 2011 and 13 June 2011 earthquakes.
- LiDAR Digital Elevation Model (DEM) settlements (corrected for tectonic subsidence) recorded of 0.5 m to 1.0 m, with majority of the liquefaction induced settlement occurring during the 22 February 2011 event (100-400 mm).

Figure 2 provides LiDAR DEM settlement for the 22 February 2011 earthquake, EQC observations of liquefaction and lateral spread on properties, and EQC mapped cracks.

EQC - LiDAR DEM vertical elevation change 22 February 2011 [(1)]

EQC - Surface observations of lateral spreading and liquefaction (22 Feb 2011)

EQC - Recorded crack locations and widths (4 Sept 2010 to post Feb 2011)



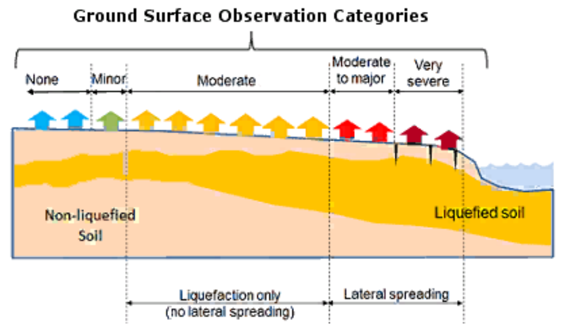


Figure 2 EQC Data inferring extent and surface observation of severity of lateral spread and liquefaction at the trial site and adjacent area

Source https://canterburygeotechnicaldatabase.projectorbit.com, refer Earthquake Commission Disclaimer

(1) LiDAR vertical elevation change corrected for tectonic movement.

3.4 Ground Conditions

3.4.1 Geology

The ‘Geology of the Christchurch Urban Area’ (Brown & Weeber, 1992) geological map indicates that the near surface geology of the trial site is comprised of alluvial sand and silt overbank deposits of the Springston Formation, and the Christchurch formation comprising dominantly sand of fixed and semi-fixed dunes and beaches.

A map of the area from 1856 (the ‘Black Maps’) shows a swamp to the east and flax rushes to the south of the site. Prior to residential development in the 1960’s and 1970’s, the land was filled to form building platforms.

3.4.2 Geotechnical Investigations

Geotechnical investigations were performed by the EQC ground improvement project team on and adjacent to the SCIRT trial site. The investigations include one borehole (BH), six cone penetrometer tests (CPT), and three cross hole seismic tests (VSVP), all up to 10 m depth. Figure 3 provides an investigation layout plan. Investigations were supervised and boreholes logged by geotechnical engineers and engineering geologists from Tonkin & Taylor Ltd. The investigation logs are included in Appendix B, all investigation data is available on the Canterbury Geotechnical Database[1].

In addition, excavation faces were logged during the installation and exhumation of the infrastructure.

Figure 3 Location of Trial Geotechnical Investigations

[1] www.canterburygeotechnicaldatabase.projectorbit.com

Table 3-2 Summary of Geotechnical Investigations

Investigation ID (4)	Easting (mE)(1)	Northing (mN)(1)	RL (m)(2)	Depth (m bgl)(3)	Date Drilled	Adjacent Investigation
BH 34459 (AVD-TCR01-BH07)	2484666.2	5745060.1	10.69	10.5	30/8/2013	CPT 34407
CPT 34404 (AVD-TCR01-CPT076)	2484658.7	5745099.1	10.79	10.1	10/9/2013	-
CPT 34405 (AVD-TCR01-CPT077)	2484685.5	5745062.2	10.63	10.1	10/9/2013	-
CPT 34406 (AVD-TCR01-CPT078)	2484683.7	5745095.3	10.96	10.1	10/9/2013	-
CPT 34407 (AVD-TCR01-CPT079)	2484664.4	5745059.9	10.68	10.1	10/9/2013	BH 34459
CPT 34453 (AVD-TCR01-CPT098)	2484712.7	5745100.1	10.82	10.1	10/9/2013	-
CPT 34454 (AVD-TCR01-CPT099)	2484703.4	5745063.7	10.42	10.1	10/9/2013	VSVP 38180
VSVP 38168 (AVD-TCR01-XH23)	2484695.2	5745080.1	10.64	6	6/10/2013	-
VSVP 38179 (AVD-TCR01-XH25)	2484703.0	5745089.7	10.74	6	6/10/2013	-
VSVP 38180 (AVD-TCR01-XH26)	2484703.0	5745062.6	10.44	6	6/10/2013	CPT 34454

(1) Coordinate System: NZMG
(2) Datum Reference: CCC Drainage Datum
(3) Meters below ground level
(4) Investigation ID's provided correspond to those on the Canterbury Geotechnical Database (https://canterburygeotechnicaldatabase.projectorbit.com)

Soil samples from BH 34459 were taken for laboratory testing to determine grading curves and Atterberg Limits for the soil horizons. The Springston Formation silts and silty sands within the upper 2 m of the soil profile exhibited a fines content of typically 20-60 %, with the underlying Christchurch Formation typically having a fines content of <10 %. A Plasticity Index of 9 was recorded at 1.1 m depth within the Springston Formation silts.

3.4.3 Soil Profile

The subsoil profile encountered was typically 1 m thick non-engineered and highly variable sandy silt and silty sand fill, with inclusions of gravel. The upper 1.0 m to 2.6 m of the soil profile is dominated by alluvial over-bank deposits of the Springston Formation comprising variable silty sands and sandy silts. Layers of silt with clay like behaviour were identified between 2.2 m and 2.6 m depth towards the southern end of the site. Loose to medium dense clean sands (<10 % fines) of the Christchurch Formation dominate the remainder of the near surface soil profile.

Summary plots of shear and primary wave velocity, cone resistance and soil type behaviour index for the investigations are provided in Appendix B.

Excavation cut faces exposed during installation and exhumation of the infrastructure identified that the soil profile within the upper 2-3 m is highly variable in soil composition and is inconsistent with significant changes observed over short distances. Photos of excavation faces are provided in Appendix A. The subsoil profile and parameters adopted for analysis are presented in Table 3-3.

Table 3-3 Typical Soil Profile at Trial Site

Layer No.	Geological Unit	Description	Depth to top of layer (m bgl)	Thickness (m)	CPT cone resistance q_c (MPa)	Soil type behaviour index, I_c	Shear wave velocity (m/s)
L0	Fill	Non-engineered fill materials: silty sand, sand, and silt, with inclusions of gravel and organics. High variability in material type.	0	0.5 – 1.3 (typ. 1)	1 – 10 (typ. 5)	1.0 – 2.6 (typ. 2.0)	-
L1	Springston Formation	Lenses of firm / loose silty sand, sand and silt, with trace gravel and organics. Incorporates historic topsoil layer at top of unit.	0.5 – 1.6	1 – 2	1 – 4 (typ. 2)	1.8 – 2.7 (typ. 2.2)	105 - 115
L2	Christchurch Formation	Loose fine sand with minor silt, with occasional silt lenses, trace organics.	2.3 – 3.0	1 – 3	5 – 7 (typ. 6)	1.4 – 2.6 (typ. 1.6)	110 - 150
L3		Medium dense fine sand with minor silt, with occasional silt lenses, trace organics.	3.6 – 5.2	>7	7 – 13 (typ. 10)	1.4 – 2.6 (typ. 1.6)	150 - 180

3.4.4 Groundwater Profile

Groundwater conditions were assessed from records of site observation, interpretation of pore pressure transducers (PPTs), CPT testing and adjacent EQC groundwater monitoring wells (Canterbury Geotechnical Database, 2014). The adopted near surface groundwater level was at a depth of 1.1 m (9.53 mRL), associated with perched water tables. Bias was placed on site observations, due to seasonal variability of groundwater levels. Measured static pore pressure within the Christchurch Formation infers a lower ground water level of 1.9 m below the ground surface.

3.4.5 Liquefaction Potential

The Liquefaction Trial utilises a series of explosives detonated in a defined sequence to induce cyclic shearing of the soil to induce excess pore pressure triggering liquefaction. The synthetic strong ground motion produced is significantly different to that generated by a natural earthquake, being of high frequency with very high ground accelerations. Standard empirical methods of liquefaction assessment are not appropriate and cannot be used to predict liquefaction triggering for the trial. Triggering of liquefaction during the trial is to be verified through measurement of excess porewater pressure through the ground profile.

However, relative liquefaction potential of the soils at the site has been assessed though varying peak ground accelerations (PGA) during a standard liquefaction assessment. The method and assumptions of the liquefaction assessment are summarised as follows:

- The liquefaction assessment of the CPT data, has been carried out in general accordance with the New Zealand Geotechnical Society (2010) Guidelines following the methods developed by Idriss & Boulanger (2008)
- The liquefaction potential of ground materials has been assessed using the available CPT and shear wave velocity data
- Consideration has been made of potential for liquefaction of soil type from review of the soil type behaviour index, laboratory testing, and soil descriptions
- Cross hole seismic testing with P waves infers that the upper 3 m to 4 m of the ground profile has saturation ratio of less 98.5%, indicating that in this condition this upper layer has low potential for liquefaction (Stokoe et al, 2014)
- A groundwater level of 9.53 mRL has been adopted

The liquefaction assessment indicates liquefaction triggering at the site for peak ground accelerations (PGA's) of ~0.1 g (M_w7.5) with extensive liquefaction within a zone affecting the buried infrastructure developing with PGA's of 0.15 g to 0.20 g (M_w7.5). The Springston Formation silts found in the southern portion of the trial site have been assessed to have low liquefaction potential. Figure 4 summarises the relative liquefaction potential of four of the CPT's.

Figure 4 summarises the relative liquefaction potential of the soil profile assessed for the CPT's showing spatial variability across the trial area.

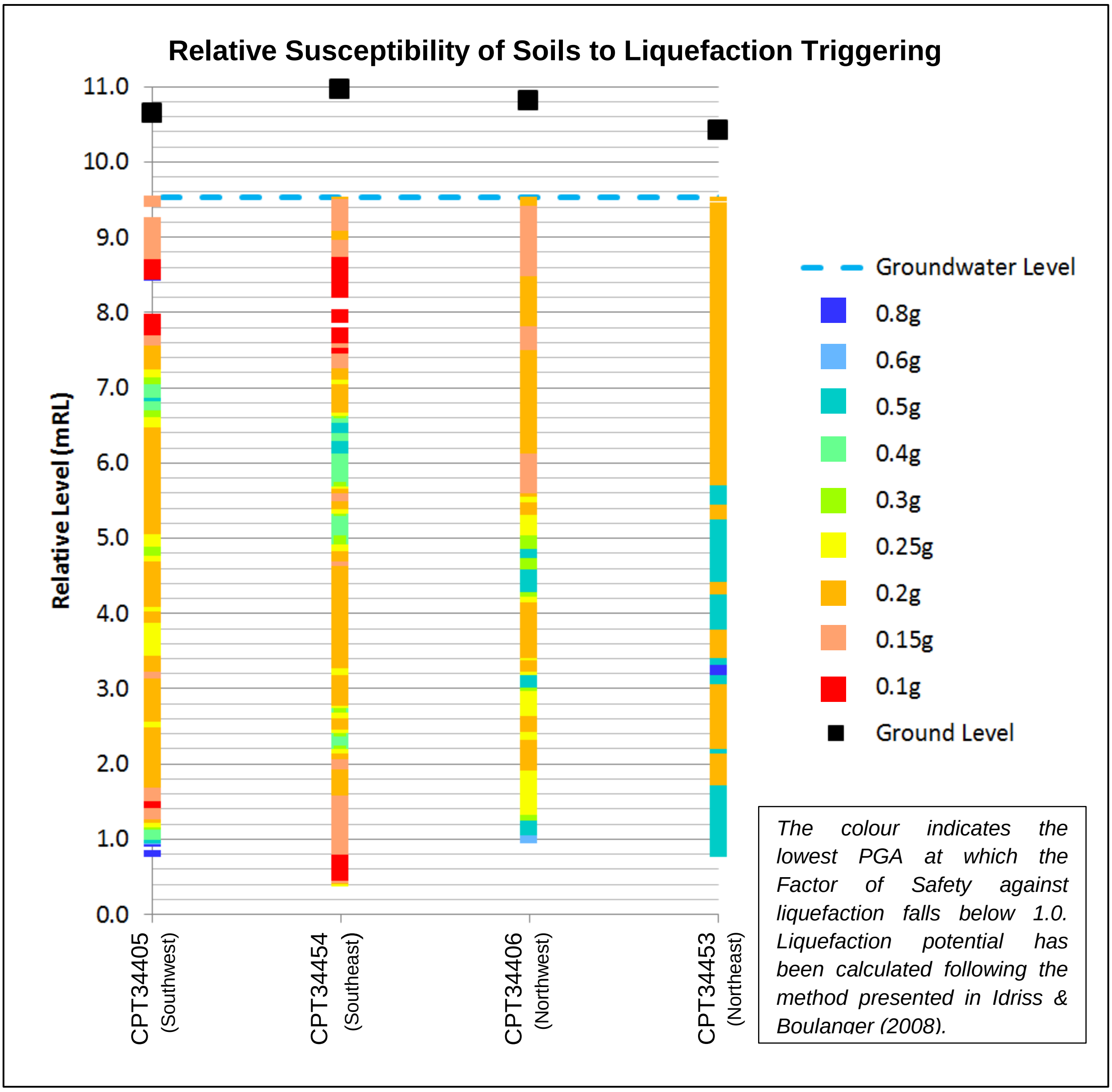


Figure 4 Relative Susceptibility of Soils to Liquefaction

4 Trial Design

4.1 Infrastructure Tested

The design of the trial attempted to maximise the range of infrastructure tested within the site constraints and optimise trial costs, so that maximum benefit could be realised. Careful consideration was given to selecting the specific infrastructure components and materials for the trial. The components comprise pre-earthquake standards, alternative resilient solutions that had been adopted in the rebuild and others which were subject to debate within SCIRT at the time. Table 4-1 provides a summary of the eight assemblies incorporated into the trial with discussion on the purpose and details involved.

Table 4-1 Summary of Trial Infrastructure and Purpose

Test ID	Assembly	Purpose and Details
1	DN150 PVC-U SN16 pipe with easily compacted granular haunching within trench	SCIRT was exploring possible alternative pipe haunching aggregates with low sensitivity to effects of water during placement, to improve construction efficiencies. Though not being considered as a haunching aggregate, Grade 2 sealing chip aggregate (NZTA M6) was selected as an extreme high void ratio nil fines alternative for the trial to allow comparison with a well graded material (Test 2).
2	DN150 PVC-U SN16 pipe with well graded haunching within trench. CCC CSS design standard.	This test provides a baseline assessment of seismic performance of existing CSS and SCIRT pipe and trench design, incorporating NZTA M4 AP20 a well graded aggregate.
3	Pressure Sewer Chamber (PE) – Granular backfill relieving porewater pressure	This is to review the performance of a chamber backfilled with highly permeable material. The trial utilised Grade 2 sealing chip aggregate (NZTA M6) encased within a sewn geotextile bag. The test aimed to quantify the reduction of uplift pressure, comparing observations with design assumptions. Also to assess the risk of clogging the geotextile and ingress of ejecta into the backfill during a liquefaction event.
4	Pressure Sewer Chamber (PE) – Backfill with compacted excavated materials. In accordance with supplier installation details.	This test provides a base line assessment of seismic performance of chambers backfilled with natural excavated materials (fine sand). The test allows an assessment of theory and comparison with the current CCC CSS standard details.
5	1050 mm dia concrete access chamber with connecting PVC-U SN16 pipe, backfilled with AP65 gravel.	This allowed assessment of seismic performance of a CCC CSS standard access chamber, comprising a proprietary precast concrete access chamber with CCC AP65 backfill. The test aimed to quantify the effects of a well graded granular backfill on uplift pressure on the chamber. The test also allowed a review of the interface between connecting pipes.
6	DN600 PE access chamber with connecting PVC-U SN16 pipe, backfilled with AP65	This aspect is a field test of seismic performance of a PE access chamber with CCC AP65 backfill. Key elements being reviewed were the same as for Test No. 5.
7	Pressure Sewer Chamber (PE) – Low strength concrete backfill	This was intended to review performance of a chamber backfilled with low strength concrete, adding weight to resist buoyant uplift.
8	DN150 Restrain PVC-U SN16 pipe installed by directional drilling	This assesses field performance of a directly drilled pipeline in liquefied soil. An alternative pipe material Restrain™ pipe was used for this test (threaded socket and spigot joint).

4.2 Infrastructure Installation

Chambers were installed inside the blast rings, the zone anticipated to liquefy, to allow assessment of buoyant uplift and interface with connecting pipe infrastructure. The pipes were installed from the chambers extending beyond the blast rings, to transition from non-liquefied into liquefied ground to allow assessment of the effects of soil liquefaction potential and associated liquefaction induced settlement on pipe dips and structural performance. The trial layout is provided in Figure 5.

Installation of the infrastructure was performed by the McConnell Dowell SCIRT Delivery Team, constructed in accordance with the CCC Construction Standard Specification (CSS) supported by SCIRT specifications.

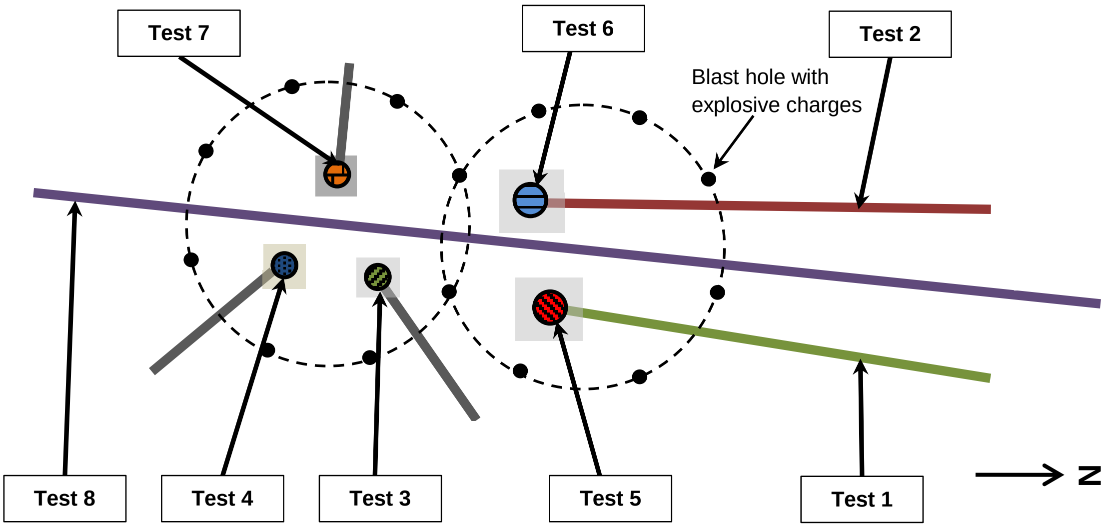


Figure 5 Site Layout for SCIRT and EQC Liquefaction Trial

4.3 Explosive Design and Sequencing

Explosive design and sequencing was developed by the EQC trial technical team, incorporating findings and efficiencies learnt during the EQC ground improvement trials preformed prior. Liquefaction was triggered though detonation of 42 charges in 14 blast holes arranged in two overlapping 10 m diameter circles (refer Figure 5). Three levels of explosive charges were installed in each blast hole, 0.8 kg charge of Orica Pentex PPP plastic explosive at 2.5 m depth and 2.4 kg charges at depths of 6.5 m and 10.5 m. The explosives were detonated in a pre-set sequence, with 300 ms intervals for the low and middle levels and 135 ms for the upper level, alternating across the circles for a total duration of 10 seconds, intended to induce cyclic shear strains in the soil, triggering liquefaction. The explosive charges were detonated on 24 October 2013.

4.4 Trial Instrumentation

Instrumentation and monitoring was incorporated into the trial design to provide baseline measurement of the installed condition of infrastructure, response of the ground to the synthetic cyclic shearing, and the resulting performance of the ground and installed infrastructure. Observation of infrastructure condition change during the detonation was performed by video inspection. Table 4-2 provides a summary of trial instrumentation and monitoring.

Table 4-2 Summary of Trial Instrumentation and Monitoring

Item	Monitoring	Purpose
Instrumentation		
Vibrating wire piezometers / pore pressure transducers (PPT)	Continuous	• PPT installed beneath chambers (No. 5) to measure uplift pressure. • PPT installed within the soil profile to record excess pore pressure and confirm liquefaction triggering. Installed at depths of; 2.8 m, 3.9 m, 4.65 m, 6.95 m, and 9 m.
Accelerometer	Continuous	• An accelerometer was placed on the ground surface during the trial to record the surface ground accelerations during explosive detonation.
Monitoring		
Survey and levelling of discrete points	Discrete	• Spatial location and elevation of all installed infrastructure is recorded to provide baseline condition, and following the detonation of explosive charges to record changes
Vertical settlement profilometers	Discrete	• Settlement profiles provide data on liquefaction induced settlement with depth though the soil profile. Two profilometers were installed within the anticipated zone of liquefaction triggering and one beyond. • The profilometers were calibrated prior to blasting and the settlements measured immediately following the blasting and again a day later.
Profilometer for pipe grade	Discrete	• Recording the vertical profile of pipelines installed, prior to and following the trial to identify total and differential settlements.
Laser profiling for pipe ovality	Discrete	• Pipe ovality was measured along the length of pipes installed, both prior and following liquefaction.
CCTV inspection of pipes	Discrete	• CCTV inspection of the pipelines to provide a visual condition survey prior and following the trial.
Ground surface LiDAR	Discrete	• Pre and post LiDAR DEM survey of the site.
Video recording	Continuous	• Aerial video recording of site from a drone during and following detonation of the explosive charges. • Land based high speed video recording of the trial (No. 4 devices)

4.5 Monitoring

All phases of the trial were closely monitored to maintain a good understanding of the as built condition and condition of infrastructure following liquefaction triggering. Monitoring was performed by SCIRT geotechnical and civil engineers and the McConnell Dowell Delivery Team implementing construction.

5 Recorded Information

A large volume of data was collected though installation, monitoring of the liquefaction triggering and exhuming infrastructure. Recorded information is presented in the appendices, with summary discussion provided in the following subsections.

5.1 Field Observations

5.1.1 Construction Monitoring

Installation of the infrastructure was coordinated by the McConnell Dowell Delivery Team. The physical works were subcontracted to civil contractor Tru-Line Civil Ltd, and were undertaken over the period of 8 October 2013 through to 21 October 2013. The weather conditions during construction were inclement, which delayed construction.

SCIRT geotechnical and civil engineers undertook daily construction monitoring of the installation of the infrastructure, review of ground conditions and installed the PPT beneath the chambers. Photos of the installation of the infrastructure are provided in Appendix A, and construction standard details, material specifications and as built records are provided in subsequent Appendices.

Key observations during construction monitoring are provided below:

- The groundwater was encountered at a depth of approximately 1.1 m depth across the site. The site was dewatered with a series of dewatering spears of 6-7 m length installed at typical spacing's of 1-1.5 m.
- The upper 3 m of the ground profile exposed during construction was highly variable. Fill materials comprising a mixture of gravel, silt and sand overlaid Springston Formation silts and silty sands. The materials encountered were loose to medium dense, and generally exhibited non plastic to low plasticity behaviour. The silt content within the soils and the visually assessed resistance to liquefaction triggering was higher than was desired; however programme constraints and lack of alternative sites for the testing prevented relocation of the trial.
- Observation identified that achieving accuracy in excavation dimensions can be challenging. Designers should consider the reasonably anticipated variance in excavation dimensions in the field, and consider influence on design intent. This was of most concern in the trial for Test 7, with low strength concrete backfill. Where the design dimensions were not achieved due to over excavation and uncertainty of accurate concrete volumes during construction. Selection of robust designs which can accommodate variation in as built dimensions, resilient backfill designs, and use of precast elements are recommended.
- Well graded backfill materials assisted with ease of construction, unlike uniformly graded materials which exhibited tendency to undermine adjacent structures and to unravel to a moderate slope angle (35^o to 40^o).
- Constructing the interface between connecting pipework and the pressure sewer for Test 7 was challenging. To prevent the concrete encapsulating the connecting pipe limiting ability to flex or to be replaced in the future, a void was created with a an oversized PVC-U pipe acting as a former, and use of expanding foam and geotextile wrapping to prevent ingress of concrete and backfill aggregates. Construction to ensure the void was maintained along with adequate clearances was fiddly and time consuming.
- PPT were installed directly beneath manholes by SCIRT engineers. Prior to installation the transducers were de-aired by Geotechnics Ltd and a water filled glove fingertip was taped to the end of the PPT to maintain saturation. The PPT were installed within native soil or backfill materials, surrounded in a small quantity of sand to protect the transducer during construction of the chamber above. Where the backfill was uniformly graded, the PPT sand bedding was encapsulated within geotextile.

- Site constraints resulted in the vertical alignment of the Test 8 pipeline, reaching, but not exceeding, the manufacturer's stated minimum radius of curvature for the Iplex Restrain^{TM} pipe (48 m).

5.1.2 Explosive Liquefaction Triggering

An exclusion zone (>100 m) around the trial site during explosive detonation was maintained to ensure public safety. The explosive liquefaction triggering was video recorded by a drone with a GoPro video camera providing aerial coverage, and the four land based high speed cameras. Following the site being declared to be safe to access, the SCIRT project team inspected the site post liquefaction triggering. Key observations from review of the video coverage and visual inspection following the trial on the 24 October 2013 are summarised below:

- The explosive charges appeared to be effective in inducing excess pore pressure and triggering liquefaction. All explosive charges detonated as planned.
- The volume of liquefaction ejecta material released to the ground surface within the SCIRT trial site was limited. Ejecta released immediately upon completion of the strong ground motion for a period of 10-15 min. The ejecta, typically comprising a fine sand with minor silt, appears to have originated from the Christchurch Formation sands from a depth of greater than 3 m. Ejecta was observed to be released in the following locations;
  - borehole for the explosive charges to the southwest of the southern ring of charges
  - borehole for a PPT within the northern ring of charges
  - adjacent to Test 7 and Test 4 chambers.

  The limited liquefaction ejecta observed at the ground surface is likely due to resistance of rapid porewater migration provided by the cohesive nature of the upper 2-3 m of the ground profile, comprising interbedded layers silt, silty sand and sand. The ejecta was expelled though the pathways of least resistance where the ground had been disturbed by the trial infrastructure construction activity, instrumentation or explosive charges.
- Ejecta was not expelled from the 6-7 m deep holes created by the dewatering wellpoints installed during construction of the trial. The civil subcontractor had backfilled the wellpoint holes with AP20 hardfill upon withdraw.
- No clear visual evidence to suggest that any of the chambers experienced gross buoyant uplift or that the ground had settled significantly relative to the chambers.
- Review of high speed footage indicates the strong vertical accelerations have induced near surface heave and minor vertical movements of the soil and infrastructure during detonation of the explosive charges (less than 20 mm). Cracking of the ground surface was observed.
- The high speed footage shows that the accelerometer which was placed on the ground surface did not move entirely integral with the ground, instead can be observed exhibit some minor bouncing on the ground. This could potentially account for some the very significant ground accelerations recorded.
- The explosive charges and triggering of liquefaction resulted in the groundwater at the site and surrounding property and road to rise by 0.4 to 0.7 m, which dissipated over a period of a few hours.
- No lateral spread occurred due to the localised liquefaction.

5.1.3 Exhuming

Following completion of the SCIRT explosive induced liquefaction trial, the site was exhumed to allow visual assessment of the performance of the buried infrastructure and ground. The site was exhumed in a controlled manner observed and directed by SCIRT geotechnical engineers over the period of 26 November 2013 to 2 December 2013. A summary of key observations during exhuming is provided in the following sections:

Chambers

The five chambers in the trial were carefully and incrementally exhumed by excavator and shovel. Key observations recorded during exhuming of the five chambers are presented below:

- No structural damage was observed on any of the chambers or the standard long socket and Iplex manhole pipe connections.
- Ground movement relative to the chambers was recorded, with gaps between the backfill and pressure sewer chamber ribs (Test 3, 4 & 7). Roughly 20mm of movement at the ground surface reducing linearly to less than 5 mm at the water table, refer Figure 6. The relative displacements observed are not considered to be caused by buoyant uplift of the chamber. The most feasible explanation is the upward temporary heave of the near surface ground in response to the explosive blast energy and settlement of soil near the ground surface.

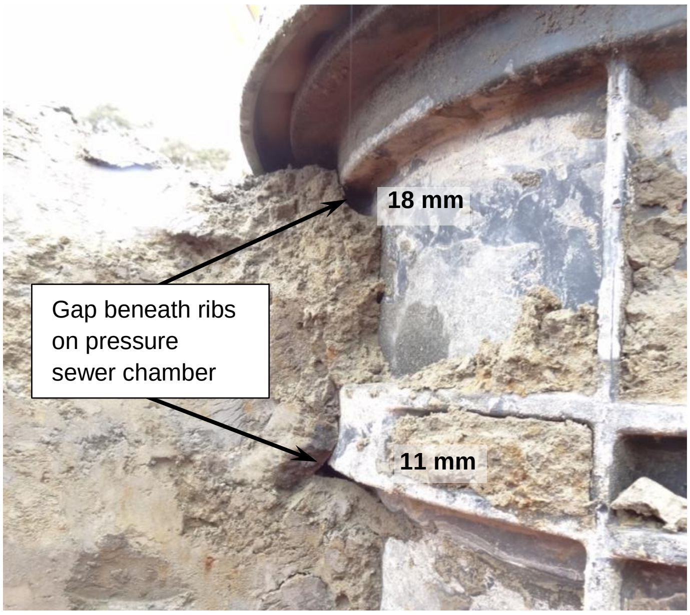


Figure 6 Photo showing Soil Separation from Pressure Sewer Chamber (Test 4)

- Evidence of deposits of liquefaction ejecta sand beneath and within some backfill material confirms that the chambers were exposed to substantial excess pore pressure.
- PPT were observed to be in an undamaged condition when exhumed. Testing by Geotechnics Ltd confirmed that post trial that excavated PPT’s were operating normally.
- The highly permeable backfill in a sewn geotextile bag of Test 3 was observed to be free from ingress of fines. An inclusion of native sand which likely fell into the permeable backfill during construction was observed; material colour and composition did not match the liquefaction ejecta and matched the upper native soils. No evidence of any caking of fines

on the external face of the geotextile was observed. However, a thin (<1mm) coating of ejecta fine sand was observed to adhere to the native soils when the geotextile was peeled away.

- No observable changes could be seen in the natural backfill of Test 4 during exhuming.
- For the standard manhole (Test 5) backfilled with well graded AP65, there was evidence that liquefaction ejecta had permeated though discrete portions of the backfill. The amount of ejecta fines entrained and deposited within the well graded dense backfill was very minor. The ejecta was concentrated at the base, perimeter, and interface with the concrete manhole. No such observations were observed for Test 6.
- The as built dimensions and volume of concrete for Test 7 were recorded, exhuming confirmed variability in construction and deviation from the intended design. Concrete cores of the in situ cast low strength concrete identified an average density of 1550 kg/m^3, this is less than the native soil, and did not satisfy the design intent (>2250kg/m^3). The void former constructed to provide separation between the pipe and concrete backfill was successful in preventing ingress of concrete and native soil materials.

Pipelines

The three pipelines tests (Test 1, Test 2 and Test 8) were exhumed along with the three laterals connecting with the pressure sewer chambers. Key observations recorded during exhumation of the pipes are presented below:

- Liquefaction ejecta was observed to have intercepted pipe haunching and backfill materials in a small number of discrete locations (typically less than 0.2 m^2). Key observations:
    - Most frequently observed in uniformly graded high permeability materials; however evidence was recorded in the well graded granular materials comprising NZTA M4 AP20 haunching and CCC AP65 trench backfill.
    - Ejecta materials were observed to enter from the base of the trench and migrate upward. This was clearly demonstrated where vertical intrusions showed limited lateral flow. Limited lateral flow of ejecta materials is likely associated with; negligible hydraulic gradient for groundwater flow through the backfill, self-filtering action, small volume of ejecta released into the near surface soils, and short duration of strong ground motion.
    - The fine sand deposited by the ejecta filled the void space between the solid particles, increasing the density and overall competence of the granular materials. The intrusions were comprised of fine sand within the centre with silt sized particles deposited around the edge (<50 mm). Beyond this ejecta intrusion and silt surround, the aggregate particles were clean and free of ejecta influence.
    - Deposition of ejecta materials into backfill materials did not adversely affect the suitability of the backfill post-earthquake, in all cases filling of void space between aggregate particles provided a denser higher quality backfill.
    - Near surface ejecta intrusion into uniformly graded materials was observed to terminate near the groundwater surface and/or interface with overlying well graded backfill materials.

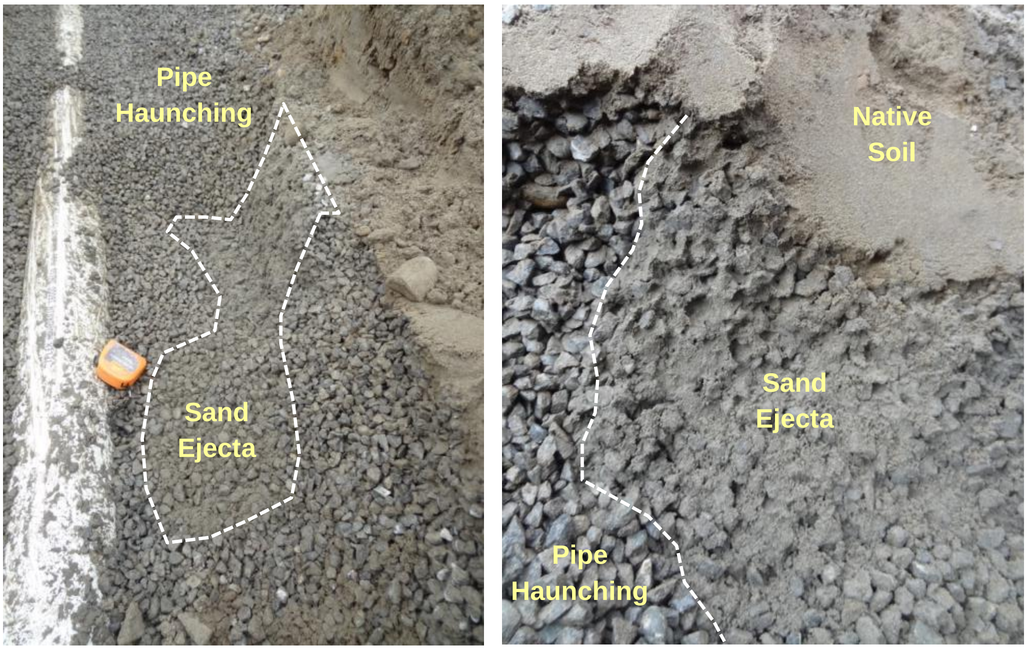


Figure 7 Localised vertical Intrusion of Liquefaction Ejecta Sand into Trench Haunching

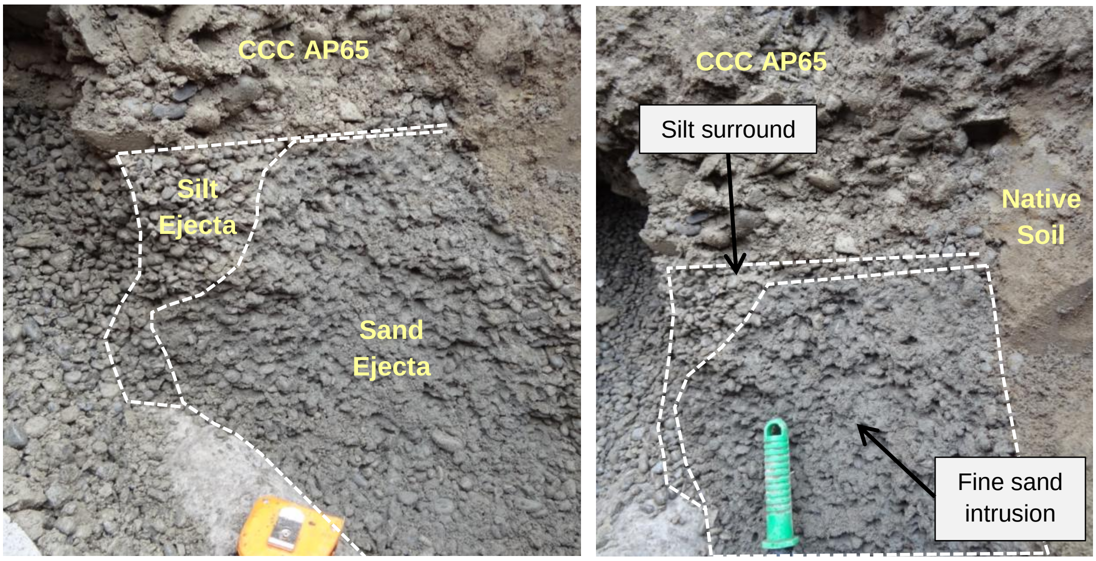


Figure 8 Localised vertical Intrusion of Liquefaction Ejecta Sand into Granular Backfill Materials

- There was no post blast structural damage observed to the pipes, with exception of Test 8.
- The directionally drilled pipeline of Test 8 sustained damage at a pipe joint near the interface with the southern edge of the explosive charges circle (<2 m separation to explosive detonation). The very large ground accelerations and shock waves differ from a natural earthquake; however the trial does highlight potential vulnerability of threaded socket and spigot connections to dynamic loading associated with the thinning of pipe wall thickness.
- The trial was unable to observe or measure any potential deformation of the trench walls or floor associated with liquefaction. Robust conclusions on risk and potential for migration of

fines from adjacent soil into the haunching and backfill cannot be drawn from this trial due to:

- The generally cohesive nature of the adjacent in situ soils, and
- The pore pressure transducer records at 2.0-2.5 m depth suggest that liquefaction may be localised within thin layers due to variability in the soil profile, majority of the liquefaction occurring at depths greater than 3 m.

- The Test 8 directionally drilled pipeline was observed to be surrounded with an annulus of 0-100 mm of very soft silt and clay (drilling muds). The DN150 pipe was not located centrally within the pipe, sitting at the top of the drilled 250mm dia hole due to the curvature of the drill string and static buoyant uplift within the drilling muds during installation. No obvious signs of flotation of the pipe within the native soil were observed.

General

Extensive excavation of the site while exhuming the buried infrastructure provided visual understanding of the variability of the soil profile over short distances within the upper 3 m. The soil layers were typically dominated by cohesive materials with some lenses of liquefiable loose silty sands and clean sands.

Evidence of paleo-liquefaction from multiple past earthquake events was uncovered across the site. Observed varying degrees of weathering in the ejecta materials within the intrusions, and observed intrusion though laterally stretched ground, indicates that the intrusions were not formed by the SCIRT and EQC Liquefaction Trial. The least weathered ejecta materials are likely from the CES. Typical observations are summarised below and in Appendix A.

- Thin (typically 10 mm) continuous cracks thought the ground profile filled with fine sand and silt, being ejecta paths. They are believed to be shaking induced cracks forming paths of weakness though the ground profile. On occasion these were observed to "bend" around more competent soil inclusions, or hit a cohesive or gravel layer and track horizontally for a short distance until a path of weakness through or around is found. No trend in alignment direction was observed.
- Fissures typically >50 mm (25-150 mm) wide and aligned sub parallel to the Avon River. The cracks were filled with fine to medium sand with trace fine gravel present. This pattern and ejecta material is similar to lateral spread cracks observed during the CES.
- Ejecta paths were observed in one case to terminate with a sand volcano which had been subsequently overlain by alluvial deposits.

5.2 Instrumentation and Monitoring Records

Raw data from trial instrumentation and monitoring has been analysed to support the review and interpretation of the trial observations. The following sections provide a brief description of the observations provided from analysis and assessment of the recorded data. Collated and manipulated data outputs are presented in their respective appendices.

5.2.1 Ground Motion

Detonation of the 42 explosive charges induced cyclic shearing of the ground with an excess of 20 cycles over a period of approximately 10 seconds. Recorded raw peak ground accelerations at the ground surface were 7 g and 23 g in the horizontal and vertical directions respectively. The high vertical acceleration is associated with the explosive charges focusing energy upward. The explosion ground motion data is provided in Appendix E.

The accelerations were significantly higher than a natural earthquake would produce and the short duration, high wave frequencies and low amplitude ground shaking are also not characteristic of a 'natural' earthquake. As discussed in Section 5.1.2, the accelerometer may have bounced on the ground and this could potentially account for some of the very significant ground accelerations.

5.2.2 Pipeline Performance

CCTV and ovality testing with a laser profiler was preformed pre and post liquefaction triggering to review structural performance of the Iplex Novadrain 1600 DN150 SN16 PVC-U pipe. The baseline testing prior to liquefaction triggering did not identify any construction non-conformities.

Post liquefaction assessment for pipelines of Test 1 and 2, and laterals for Test 3, 4 and 7 did not identify any non-conformities or observable change in pipe condition and ovality. However, damage to Test 8, the directionally drilled pipe, caused flooding and filling of the pipe with sand and hindered testing. All attempts to clean and flush the pipeline failed due to high inflows of sand and water. The pipe was exhumed in the location of the obstruction, uncovering a damaged joint on the Iplex 1600 Restrain™ pipe. Structural damage from shock loading associated with the close proximity of the explosive charges (~2 m separation) is thought to be the primary cause of the damage. Inflows of sand, likely contributed to the observed pipe settlement between chainage distances of 6 m to 13 m (refer Figure 9). The trial only assessed pre and post-trial pipe ovality. It is likely that during the detonation of the explosive charges and associated strong ground motion ovality deviations exceeded measured values. However, no evidence of effects affecting long term performance of the pipes could be observed.

The rate of pipeline differential settlement was relatively smooth, with a maximum rate of differential settlement of 5 mm/m recorded. No structural pipe damage was observed, this being expected as the deformation did not exceed the manufacturer's minimum radius of curvature. The greatest rate of differential settlement recorded was at the transition from the ground profile with extensive liquefaction though to non-liquefied soil, this was inferred from review of the LiDAR DEM.

5.2.3 Liquefaction Induced Settlement

The effects of liquefaction induced settlement on the buried infrastructure have been determined from assessment of the following testing:

- Pre and post topographical survey of specific points on the infrastructure.
- Profiler records for pipelines, recording change in vertical elevation along a pipe length.
- Vertical settlement profilers recording incremental ground settlement in 0.5 m layers to 10 m depth.
- Comparison of pre and post LiDAR DEM surfaces.

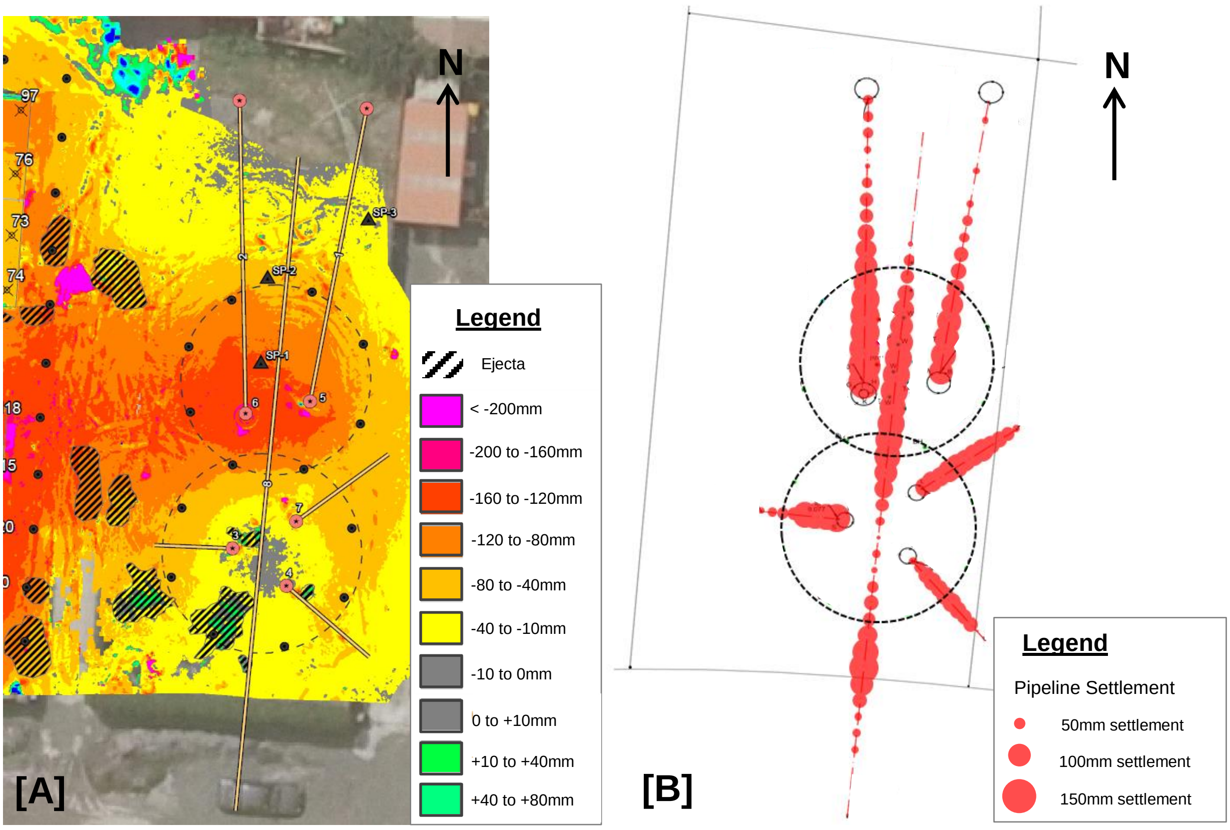
Figure 9 presents a LiDAR DEM map of liquefaction induced settlement of the ground across the site, and a plan showing a schematic representation of relative settlement along the pipe infrastructure. LiDAR DEM indicates different ground response from varying subsurface conditions. With total settlements of 120 – 180 mm within the northern blast ring, and significantly smaller settlements of 0 – 80 mm were recorded in the southern blast ring.

Ground surface observations correlated well with the 0 mm to greater than 150 mm recorded settlements across the site within pipes of Test 1, Test 2 and Test 8. And the negligible settlement observed within the southern blast ring was reflected with profiler readings within Test 8 at located approximately 3 m depth and spot levelling of the pressure sewer chambers (Tests 3, 4 and 7).

Liquefaction triggering at the EQC ground improvement trial located at 29/29A Ardrossan Street (immediately adjacent to the SCIRT trial site) 20 seconds prior to explosive detonation at the SCIRT trial site, induced larger magnitude liquefaction induced settlements over the western half of the SCIRT site. This is discussed further in Section 5.2.4.

Vertical settlement profilers (SP-1, SP-2 and SP-3) located in the positions depicted in Figure 9-[A] suggest that the settlement at the ground surface was 140 – 160 mm, correlating well to the LiDAR DEM. The rate of settlement was relatively uniform below the groundwater table to a depth of approximately 8 m, with a typical volumetric strain estimated to be 1-3 %. The settlement profiler beyond the blast circles (SP-3) shows similar magnitude of settlement to the profilers within the blast circles. However this does not correlate with the <50 mm settlement recorded by the LiDAR DEM and profiler in the Test 1 pipeline, indicating that SP-3 is in error.

The LiDAR digital elevation model of ground settlement suggests that liquefaction was triggered up to 5 m beyond the blast rings.



[A] – LiDAR DEM total settlement induced across trial site
[B] – Schematic assessment of relative settlement within pipe infrastructure recorded by profilometer

Figure 9 Variability in Liquefaction Induced Settlement of the Ground Surface and Pipe Infrastructure

5.2.3.1 Influence of Liquefaction Induced Settlement on Infrastructure

Pipelines

Liquefaction induced settlement of the ground beneath the buried pipelines and laterals connecting to the chambers resulted in corresponding equivalent settlements of the pipes. This is shown in Figure 9[B] where a close correlation can be seen between the relative settlement of LiDAR DEM and the spatial settlement of the pipelines. The magnitude of pipe settlement was typically 100 mm, with a range of 50 mm to 160 mm. No evidence was observed to suggest that haunching or backfill material type influences the magnitude or rate of differential settlement. Differences were within error bounds of estimation/measurement.

The influence of variability of liquefaction induced settlement of the underlying ground on pipeline grade is clearly demonstrated by the pipe dips formed by differential settlement for the directionally drilled pipeline (Test 8). Figure 10 provides a cross section along the alignment of the pipeline showing both vertical deformation, both in real elevations and level difference. The observed negligible settlement of the Test 8 pipeline between chainage distances of 12 m to 19 m corresponds to negligible settlement of the ground surface and infrastructure within the centre of the southern blast circle. There is no evidence of buoyant uplift of the directionally drilled PE pipe founded within liquefiable soil.

The apparent uplift beyond chainage distance 37 m relate to the unsupported length of pipe in the access pit at the end of the pipeline.

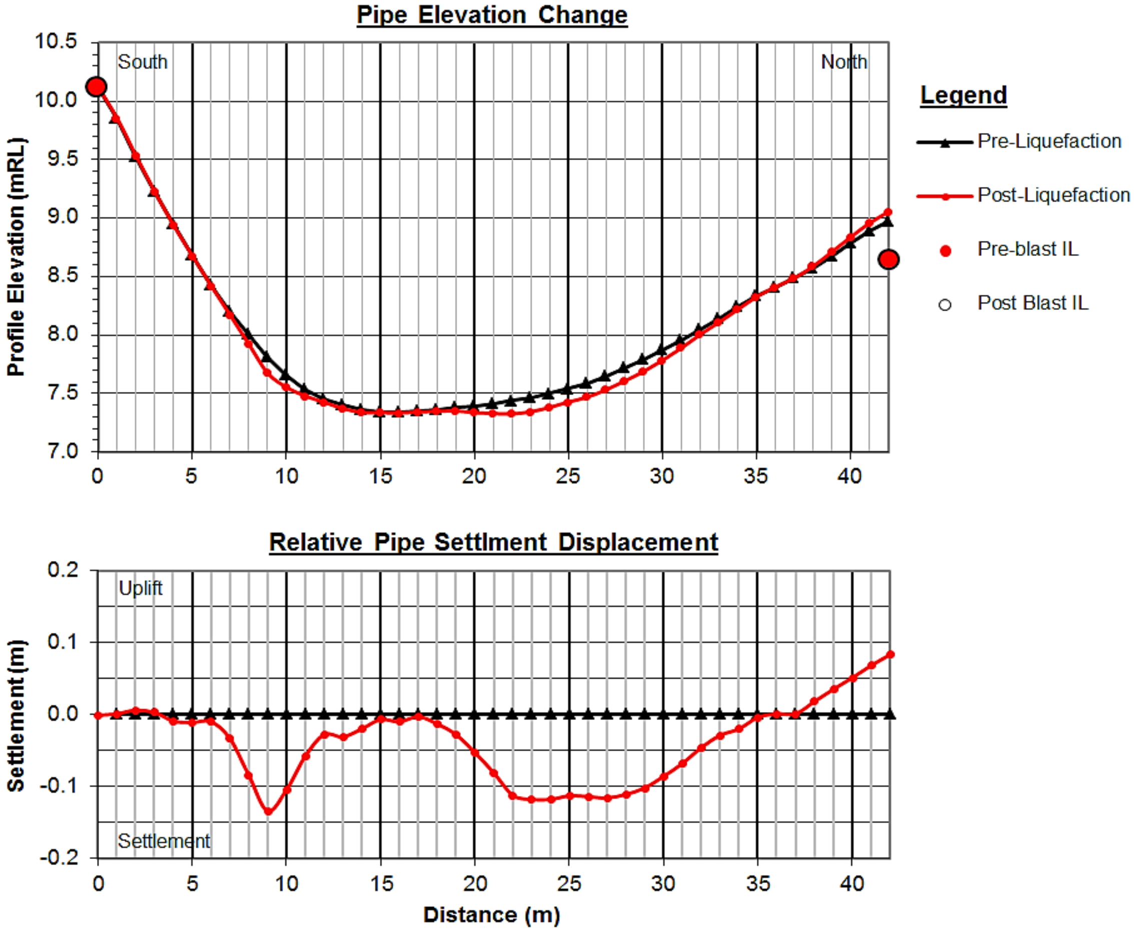


Figure 10 Test 8 – Measured differential settlement along the directionally drilled pipeline

Differential settlement was observed at the interface between chambers and connecting pipe infrastructure. The greater the depth the chamber is founded beneath a connecting pipeline, the greater the differential settlement. The observed magnitude of differential settlement was generally <10 mm, for a difference of 0 mm to 75 mm between foundation level and pipe connection. This observation is in line with the typical volumetric strain of 1-3 % inferred from the vertical settlement profilers.

Chambers

The chambers within the northern blast circle (Tests 5 & 6) have settled a similar magnitude as the underlying ground (refer Table 5-1).

The pressure sewer chambers within the southern blast circle (Tests 3, 4 & 7) recorded negligible change in reduced level. LiDAR survey records indicate that the surrounding ground has also exhibited only very minor settlements of <40 mm, and the directionally drilled pipeline installed 1 m beneath the foundation of the pressure sewer chambers recorded no settlement. During exhumation of the chamber with natural ground backfill (Test 4) settlement of the ground relative to the chamber of <20 mm was observed. This settlement differential was largest at the top of the chamber reducing with depth. It is concluded that the difference between chamber settlement and surrounding ground is likely a result of explosive heave and subsequent settlement of the near surface soil, or potentially compaction settlements during construction. Survey, monitoring and observations all infer that the pressure sewer chambers did not exhibit vertical uplift.

Table 5-1 Summary of measured Chamber Settlement

Chamber Description	Item	Depth to chamber foundation (m)	Average chamber settlement (mm) (1)	Inferred ground settlement at base of chamber (mm) (2)	Chamber displacement relative to estimated ground movement (mm)
PE Pressure Sewer Chamber	**Test 3:** Permeable backfill	1.85	-8	Est. 20 [Range: 0 – 50]	**+12**
	Test 4: Natural sand backfill	1.85	0	Est. 5 [Range: 0 – 10]	**+5**
	Test 7: Concrete backfill	1.85	-6	Est. 20 [Range: 0 – 100]	**+14**
Standard Concrete Manhole	**Test 5:** AP65 backfill	2.74	-93	Est. 100 [Range: 70 – 130]	**+7**
PE Manhole	**Test 6:** AP65 backfill	1.8	-155	Est. 150 [Range: 130 – 160]	**-5**

(1) Based on difference between pre and post blast level survey, on multiple positions on chamber.

(2) Estimated through review of LiDAR DEM, vertical settlement profilers, and profilometer records of pipeline settlement.

5.2.4 Pore Pressure Assessment

Excess porewater pressure recorded by PPTs allowed assessment of buoyant uplift pressure and confirmation of liquefaction triggering. 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. Figure 11 and Figure 12 present plots of excess pore pressure ratio against time within the natural ground and beneath the chambers respectively. Review of the calculated r_u values inferred that the explosives triggered extensive liquefaction down to the deepest installed at 9 m. With the exception of the PPT installed at 2.8 m depth in native soils which achieved a peak r_u of 0.6. PPT's within 2-3 m of the ground surface recorded varying levels of excess pore pressure, indicating liquefaction triggering levels had been reached (or close to). Following the detonation of explosives a liquefied state was maintained for up to 5 minutes as excess porewater migrated upward from the soil strata below. Delayed secondary liquefaction was observed in PPT's installed in native soils 2 to 3 minutes following explosive detonation.

Strong ground motion from the EQC ground improvement trial at 29/29A Ardrossan Street roughly 20 seconds prior to the SCIRT trial explosive generated excess pore pressure within the SCIRT PPT as shown in Figure 13.

Chamber Buoyant Uplift

Buoyant uplift forces exerted on the chambers were inferred from PPTs installed directly beneath the chambers. Figure 12 and Figure 14 present the calculated r_u from measured excess pore pressure for the five chambers, further detail is given in Appendix H. Measured excess pore pressure and uplift is discussed for individual chambers and backfill type below:

- Test 3: Pressure sewer chamber with high permeability backfill

  It was observed that the excess pore pressure in the surrounding liquefied native soil was partially relieved though migration of water into the permeable backfill. The high permeability of the backfill limited the uplift pressure beneath the chamber to a static water head at the ground surface, which slowly dissipated following the test.

- Test 4: Pressure sewer chamber with native soil backfill

  The PPT installed into silty sand beneath the chamber inferred that the native soil beneath the chamber liquefied.

- Test 7: Pressure sewer chamber with concrete backfill

  The PPT inferred that the native soil beneath the chamber liquefied. An important observation was that during liquefaction the pore pressure measured beneath the concrete encased chamber was equivalent to the initial total stress at that location.

- Test 5 & 6: Manholes with CCC AP65 well graded backfill

  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. The CCC AP65 is derived from quarried alluvial deposits in Canterbury and is typically characterised as having an elevated fines content and variable permeability, similar to the adjacent native soils ($1x10^{-7}$ m/s to $1x10^{-4}$ m/s).

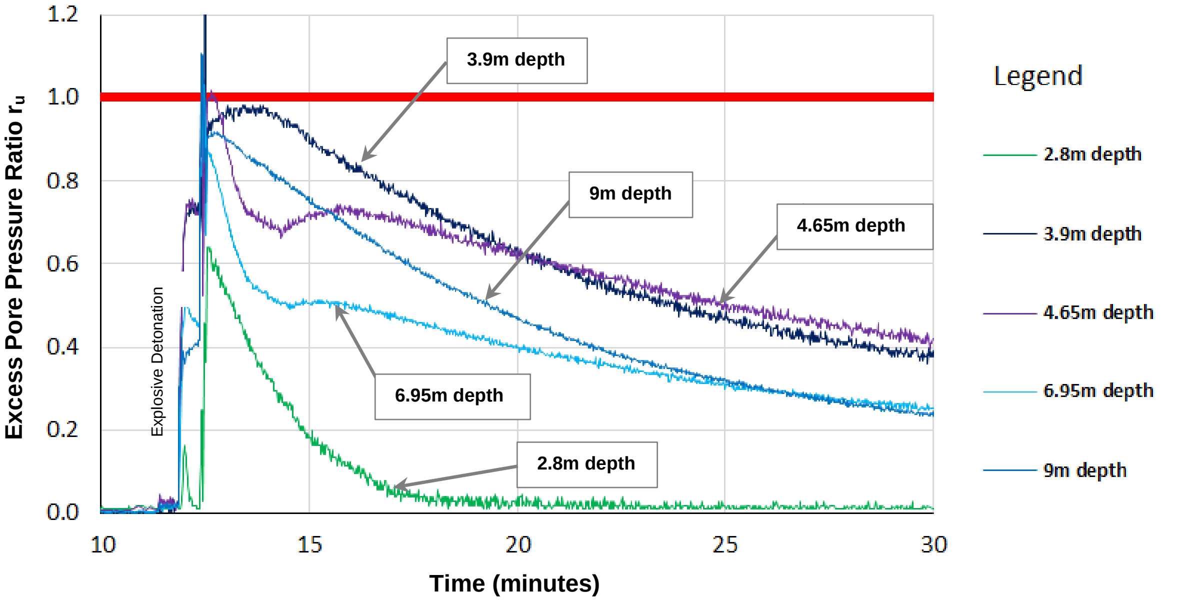


Figure 11 Excess Pore Pressure Ratio for PPT installed within native Soil

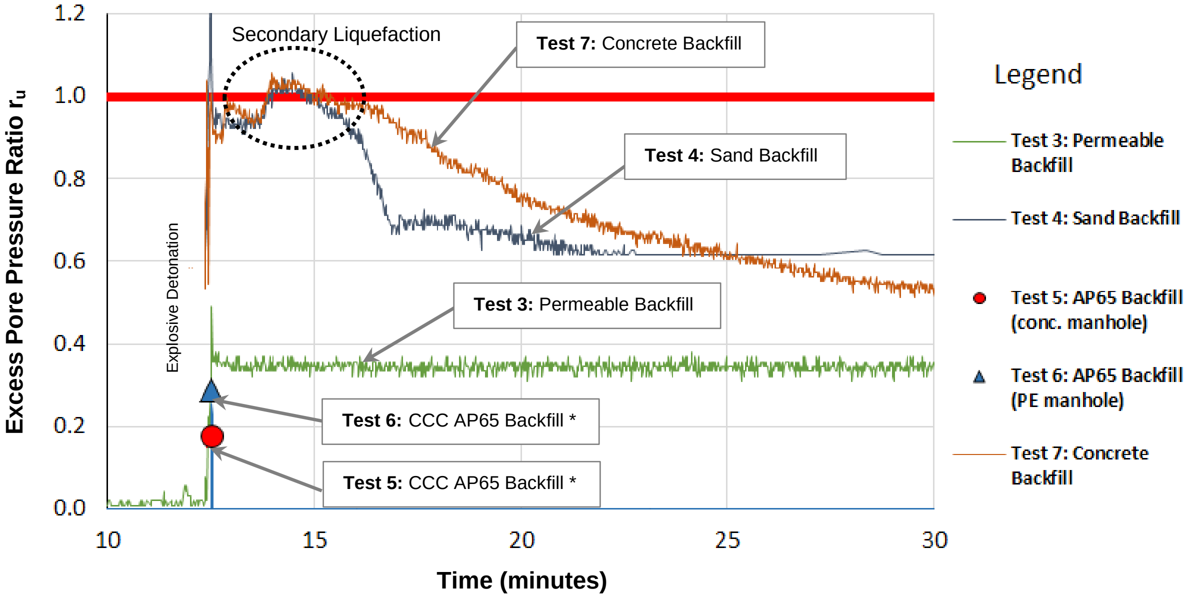


Figure 12 Excess Pore Pressure Ratio for PPT installed beneath Chambers

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

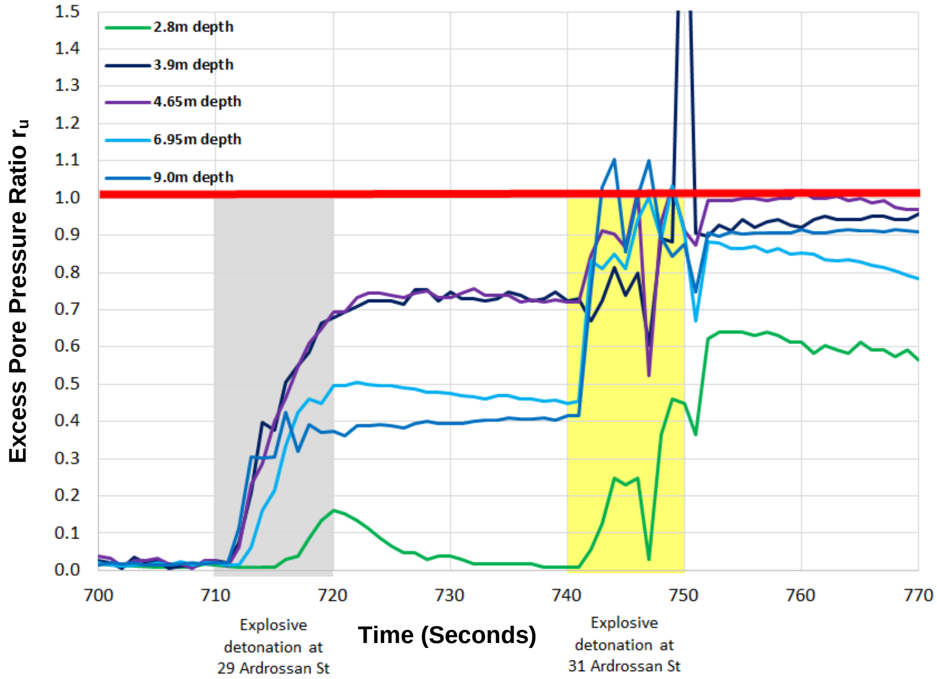


Figure 13 Excess Pore Pressure Ratio for PPT installed within native Soil - Detail

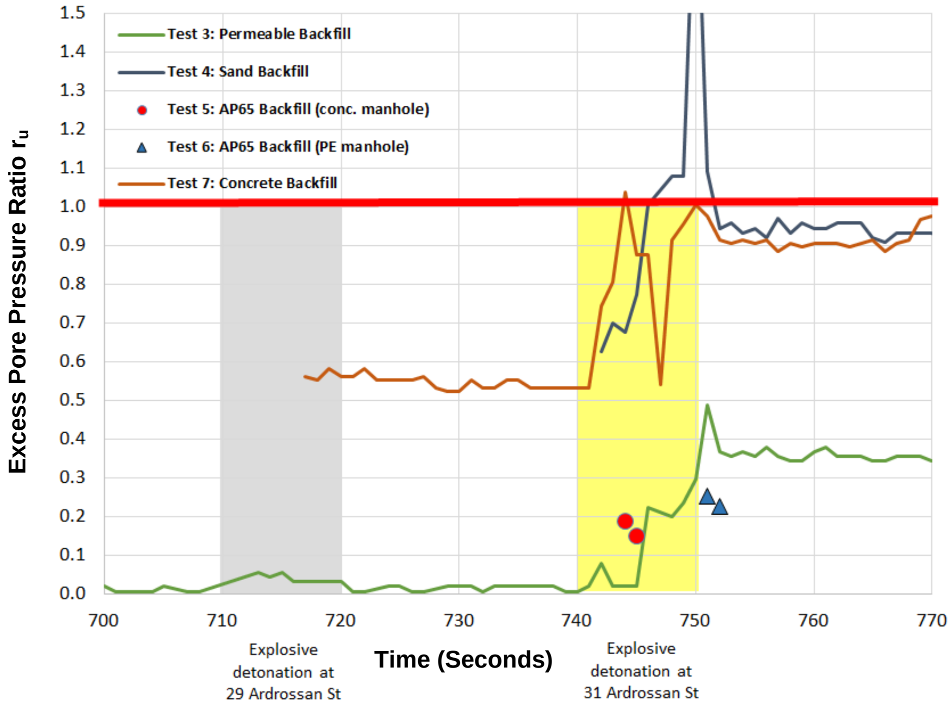


Figure 14 Excess Pore Pressure Ratio for PPT installed beneath Chambers - Detail

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

6 Analysis and Interpretation

6.1 Theory

Interpretation of trial observations and data requires consideration and comparison against current state of art theory of the process leading to liquefaction triggering, buoyant uplift and liquefaction induced settlement. The following sections provide a simplified summary of critical geotechnical theory influencing seismic performance of chambers and pipes in liquefied soil.

6.1.1 Liquefaction Triggering

Cyclic shearing of the soil associated with earthquake strong ground motion can induce excess porewater pressure. The resistance to shearing and generation of excess pore pressure is dependent on the soil density and physical properties. Where the permeability of the soil is insufficient to enable excess pore pressure generated to be completely relieved due to the rapid rate of cyclic shearing, net excess pore pressure for each cycle accumulates. As shaking levels increase beyond a point near the threshold for triggering of liquefaction the excess porewater pressures quickly increase with the shaking levels until liquefaction is triggered. Liquefaction triggers when the total excess pore pressure is equivalent to the initial effective stress (σ_{vo}'). The reduction in effective stress leads to a reduction in soil shear strength, to a point where theoretical liquefaction is triggered, with complete loss of effective stress (σ_v') and an excess pore pressure ratio (r_u) approaching 1. The significant loss of soil shear strength leads to the soil exhibiting physical behaviour similar to a dense viscous liquid. The process of development of excess porewater pressure with strong ground motion is visually explained in Figure 15.

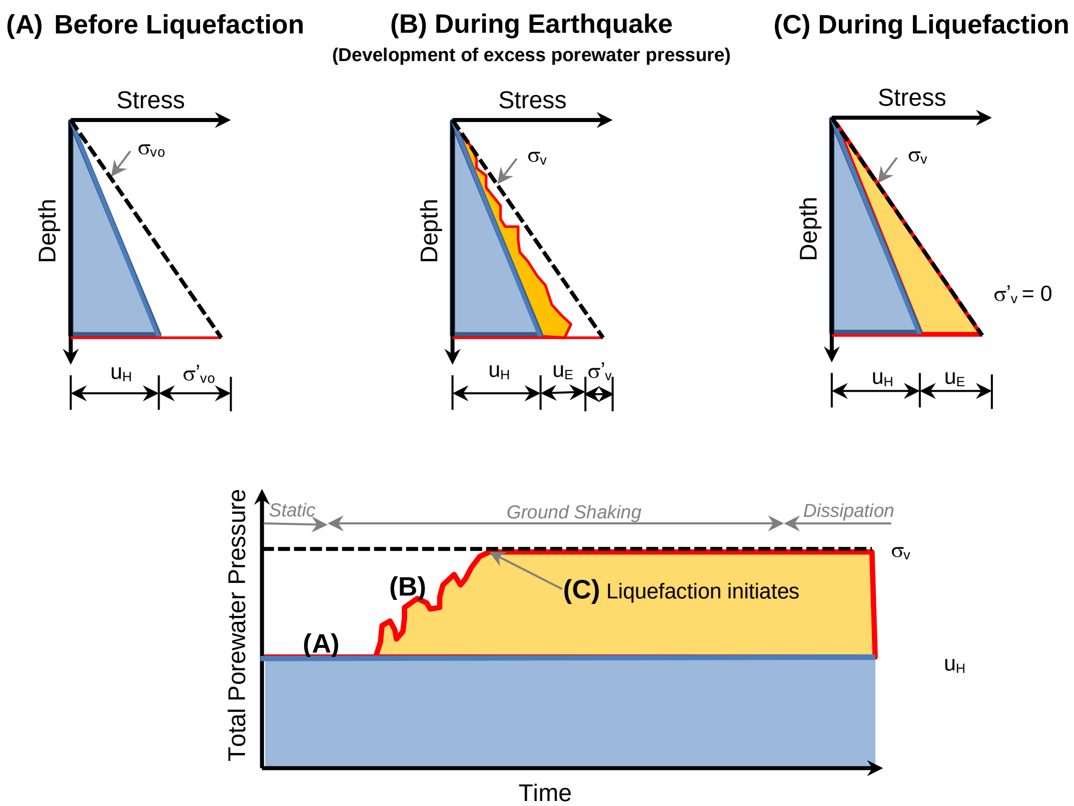


Figure 15 Schematic depicting the Development of excess Porewater Pressure and initiation of Liquefaction during an Earthquake

6.1.2 Buoyant Uplift

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 concluded 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).

6.1.2.1 Static Condition

Hydrostatic porewater pressure exerted perpendicular to submerged faces of a below ground structure induce a net uplift force. Calculation of buoyant uplift force within a hydrostatic fluid is simplified by Archimedes Principle, which states that the uplift force is equivalent to the weight of the fluid that is displaced by the object. Buoyant uplift displacement of the structure results when the Factor of Safety (FoS) to uplift reduces to below unity. This occurs when the static uplift force (F_B) 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 and Madabhushi, 2013).

$$\mathbf{FoS} = \frac{F_B}{F_T + F_{WS} + F_{SP}} \qquad \textbf{Equation 1}$$

Figure 16 provides a schematic visually summarising buoyant uplift mechanisms for pipes and chambers under static conditions.

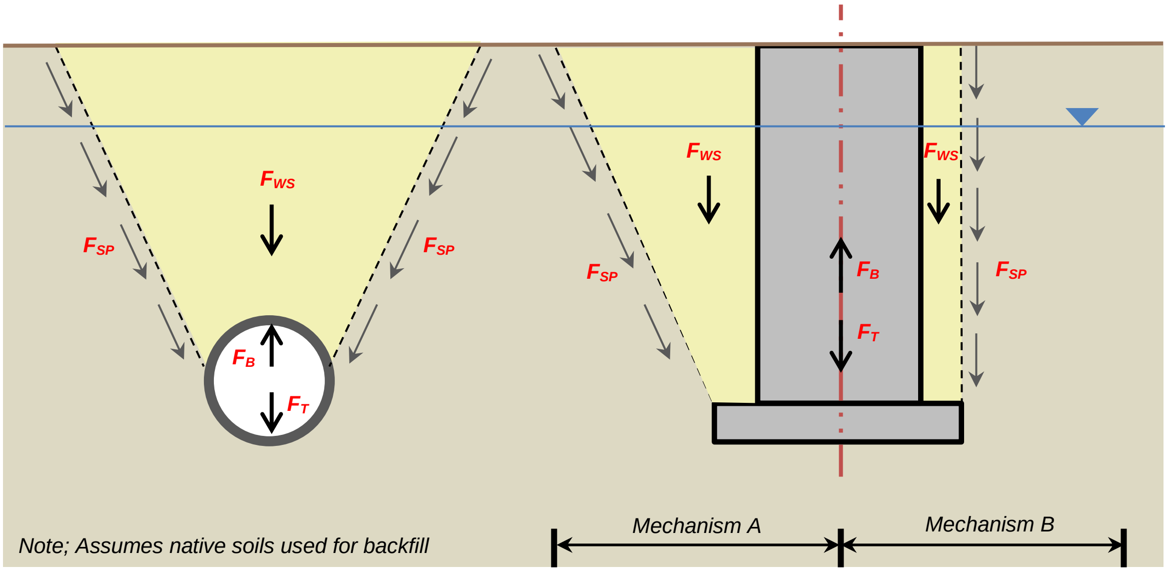


Figure 16 Schematic of typical buoyant Uplift Mechanisms – Static Conditions

6.1.2.2 Liquefied Condition

Seismic - Uplift Pressure

During strong ground motion as the porewater pressure increases the pressure exerted on a submerged structure also progressively increases until the point of liquefaction triggering, where the porewater pressure is equivalent to the initial total stress at a given depth (σ_{vo}). At this point the initial effective stress (σ_{vo}') has reduced to zero.

It is important to consider uplift pressure even when liquefaction is not triggered as FoS against uplift would be reduced. The magnitude of excess porewater pressure is dependent on the intensity and duration of shaking and the permeability of the soil.

Seismic - Buoyant uplift FoS

Development of excess pore pressure 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 porewater 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.
- A reduction in the effective weight of overlying soil.
- Application of seepage forces associated with migration of excess porewater from soil layers below the structure migrating upward towards the ground surface. This phenomenon is often termed as secondary liquefaction and is associated with ejecta release at the ground surface. However testing by Sasaki et. al. (2004) observed during lab testing that seepage forces were relatively minor, being less than 5 % of the total uplift force (F_B + F_{EPP}).

The 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. Permeability of the soil influences the rate of excess pore pressure dissipation. The FoS against buoyant uplift can be calculated through use of the following formula (Chian & Madabhushi, 2013).

$$\mathbf{FoS} = \frac{\mathbf{F_B + F_{EPP}}}{\mathbf{F_T + F_{WS} + F_{SP}}} \qquad \textbf{Equation 2}$$

Refer to Figure 17 for a schematic visually summarising the failure mechanisms and force components under seismic conditions for liquefiable and non-liquefiable backfill. Infrastructure surrounded by liquefiable backfill exhibits reduced resistance soil shearing and the reduced area of the soil block directly above the infrastructure. Use of non-liquefiable backfill provides increased resistance to uplift due to increased shear resilience and wedges of non-liquefiable backfill.

Seismic buoyant uplift displacement

Typically 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. Excess pore pressure observed beneath the structure with the strong ground motion is lower than in the adjacent liquefied native soil. Koseki, et. al. (1997) observed during testing that the horizontal pressure gradient induces flow of liquefied sand towards and beneath the structure during uplift. Where gravel underlies the structure, the lateral pressure from the adjacent liquefied soil squeezes the gravel up to maintain contact with the underside of the structure.

Laboratory testing by Koseki, et. al. (1997) identified that uplift displacement was triggered when FoS reduced to about 0.7 to 0.95 during the period of shaking, and displacement continued until a FoS of close to 1 was achieved. This was supported by Sasaki et. al. (2004) who identified the uplift displacement rate was nearly constant during shaking, with the rate and magnitude of displacement being inversely proportional to the FoS. They observed movement almost stopped upon cessation of shaking, even if elevated pore pressure were maintained.

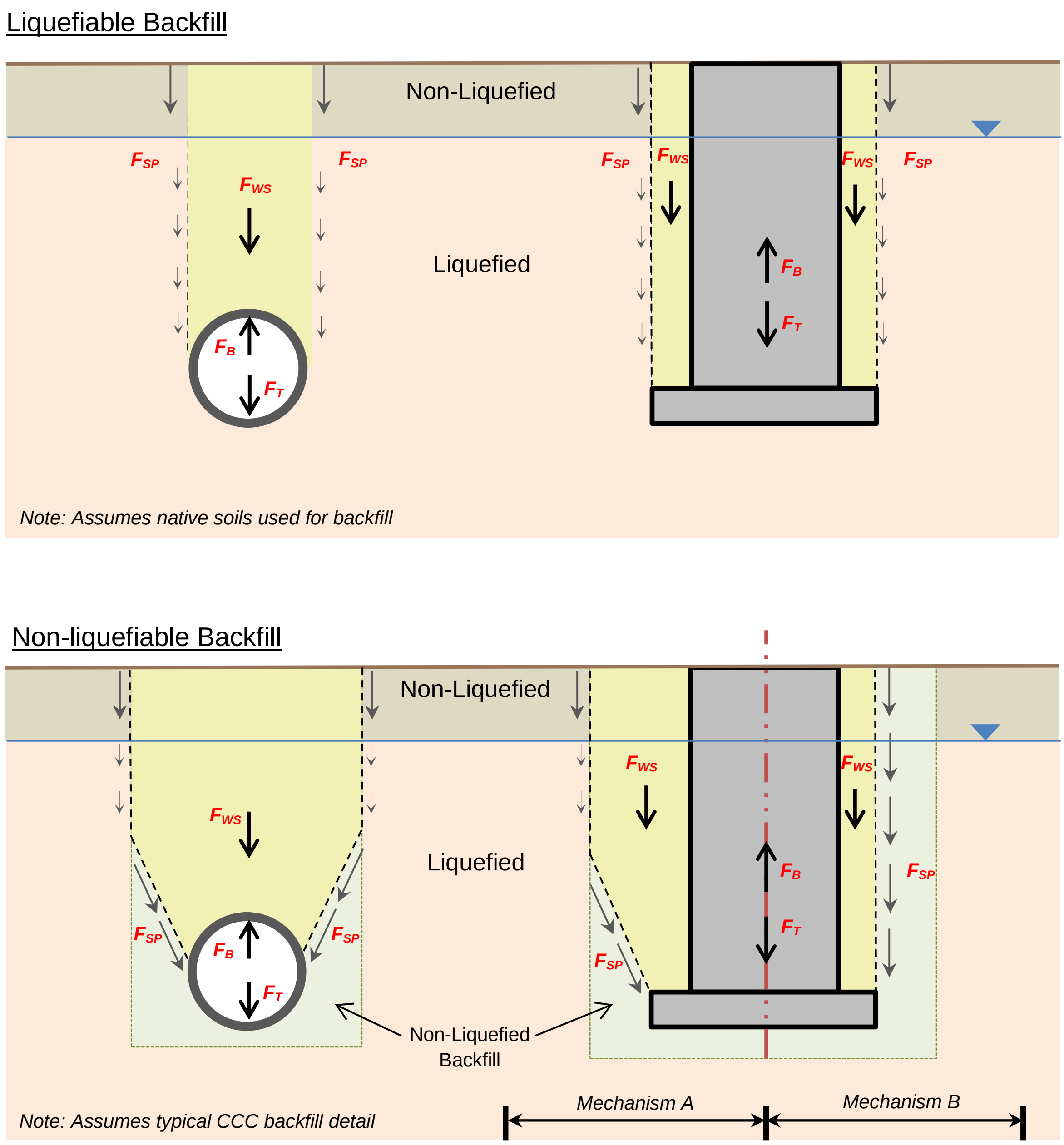


Figure 17 Schematic of typical buoyant Uplift Mechanisms – Seismic, excess Pore Pressure Conditions

6.1.3 Liquefaction Induced Ground Settlement

The magnitude of liquefaction induced settlement is dependent on the severity of shaking, properties of soil (density, void ratio, cohesion, age etc.) and extent of liquefaction. Generally the volumetric strain is within a range of between 0 % - 5 % strain (Idriss & Boulanger, 2008).

6.1.4 Bearing Capacity

Generally the weight of a buried chamber is less than the weight of the native soil displaced, so under static conditions bearing failure is not anticipated unless the distribution of load on the foundation is eccentric.

Excess pore pressure generated by strong ground motion reduces net foundation loads; however the associated reduction of soil shear strength also reduces allowable bearing capacity. When the buoyant uplift force (F_B) exceeds the weight of the structure (F_T) and soil weight (F_{WS}), bearing failure is not anticipated. However, eccentric loading and differential settlement beneath the structure can result in localised elevated foundation pressure exceeding ultimate bearing capacity of the liquefied soil. Bearing failure will likely occur when the weight of the structure and soil exceeds the uplift force, leading to settlement of the structure.

6.2 Assessment of Performance of Buried Infrastructure

Learnings drawn from interpretation of the SCIRT and EQC Liquefaction Trial observations are presented and discussed in the following sections.

6.2.1 Liquefaction and Liquefaction Induced Settlement

The LiDAR DEM of ground settlement at the site inferred moderate to extensive liquefaction extended up to 5 m beyond the blast circles. Typical liquefaction induced settlements recorded were 120-180 mm within the northern blast circle; however the southern blast circle exhibited minor ground settlements of 0-80 mm even though liquefaction was inferred by the PPT and surface expressions of ejecta. The cause of the lesser settlement in the southern blast ring has not been fully explained. However, the presence of the silt lenses and ground improvement associated with the close proximity of the installed infrastructure at this end of the site may be contributing factors. It is possible that the explosive charges on the southern blast ring (compared with the northern ring) did not induce the same level of liquefaction at depth. This cannot be verified however liquefaction ejecta was expelled from the charge boreholes at the south west corner of the site suggesting the soil at depth did liquefy. This highlights the high variability of soils in Christchurch and the corresponding variability in settlement response.

Minor settlement of the ground surrounding chambers of <20 mm relative to the chamber base was observed, Table 5-1. This is in line with the typical observations of the ground settlement relative to chambers during the short shaking duration 22 February and 13 June 2011 earthquakes. Where pipe infrastructure interfaced with chambers, settlement of <20 mm relative to the base of the chamber was observed associated with settlement of 0.5 m to 1.5 m of soil, resulting in minor negative pipe grades at the connection.

Cohesive soils encountered within the upper 3 m of the soil profile and the synthetic ground motion of high intensity and short duration may influence the results of this trial. The observations of this trial are likely of lesser scale compared to those expected during a significant natural earthquake. Consideration of field observations and the inferred performance of buried infrastructure during earthquakes, laboratory testing and engineering judgement are required to interpret observations.

6.2.2 Chamber Performance

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 modes of damage 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 associated magnitude of buoyant uplift displacement for a typical manhole within liquefied backfill is of the order 25 – 250 mm when estimated by the method of Sasaki & Tamura (2004).

Uplift pressure

The excess pore pressure ratio with time measured beneath each of the chambers is presented in Figure 12. The magnitude of excess pore pressure observed in this trial supports theoretical assessment and laboratory testing by researchers, including Tobita et al. (2012), with excess pore pressure being equivalent to initial effective stress.

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. A delayed increase in uplift pressure within the native soils from the migration of excess porewater from soil strata below towards the ground surface was 2 to 3 minutes following explosive detonation (secondary liquefaction) and inferred uplift pressure that would be similar or marginally higher than those generated by initial liquefaction of the adjacent soils. The 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 et al. (2004) which observed that seepage forces were relatively minor, being less than 5 % of the total uplift force (F_B + F_{EPP}). Consideration of elevated uplift associated with seepage, with uplift pressure equivalent to 105 % of total stress is recommended for design of buried infrastructure.

Influence of backfill type on uplift pressure

<u>Liquefiable backfill</u>

The pore pressure measured beneath the Test 4 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 is not affected by the extent of liquefied soils above.

<u>Low strength concrete backfill</u>

The uplift pressure within the liquefiable native sands directly beneath the Test 7 chamber was also equivalent to the initial total stress. The efficiency of this backfill is highly dependent on the as built dimensions and volumes accurately aligning with the design detail, in order to limit potential for bearing failure.

Permeable backfill

The high permeability of the backfill limited the uplift pressure beneath the chamber to a static water head at the ground surface. The pore pressure gradually reduced with time as excess pore pressure in the surrounding native soils reduced and water within the backfill was able to drain back into the adjacent soil.

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 coating.

The resilience of a permeable backfill solution is dependent on providing an adequate drainage path to the ground surface, and a sewn geotextile bag with detailing to prevent ingress of ejecta at pipe penetrations and the ground surface.

Well graded granular backfill

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, indicating the backfill did not liquefy. Excess pore pressure exerted on the chamber from liquefaction was less than 30 % of the excess pore pressure in the surrounding liquefied native soils. The external excess pore pressures within the native soils were attenuated within the well graded granular backfill. Trace infiltration of ejecta materials from soil strata below was observed during exhumation.

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 conservative. A further programme of testing is being developed in conjunction with the University of Canterbury through the Quake Centre to determine the validity of this observation from a single synthetic trial of short duration, and develop relationships for pressure attenuation with time. Caution in directly adopting the recorded observations of magnitude of pressure attenuation from Test 5 and 6 is recommended until the validity is confirmed.

Buoyant uplift potential

A theoretical assessment of the buoyant uplift potential for each chamber was performed considering the 'as built' condition and measured uplift pressure.

Table 7 summarises the theoretical FoS against buoyant uplift calculated in accordance with Equation 2 and the potential buoyant uplift mechanisms presented in Figure 17.

Analysis indicates that Test 4 and possibly Test 7 exhibited the 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 FoS.

For Test 7 the unit weight of the concrete mass backfill was less than assumed during design, providing a reduced weight to resist uplift. However, the uplift pressures recorded beneath the chamber were approximately equivalent to the initial total stress prior to liquefaction. Test 7 supported theory which suggests that the benefits of adding impermeable mass to a structure (below ground) to resist buoyant uplift is limited once a FoS of 1 is achieved, as excess mass increases the total stress and with this the liquefaction uplift pressure.

Table 6-1 Anticipated Chamber Uplift compared with observed Uplift

Test ID	Chamber Type	Backfill Type	Liquefied Buoyant Uplift Factor of Safety		Uplift Anticipated	Uplift Observed
			Pre Trial Theoretical (1)	*Observed Condition* (2)		
3	PE Pressure Sewer	Permeable backfill	3.0	**2.4**	No	No
4	PE Pressure Sewer	Native sands	0.6	**0.9**	Yes	No
5	1050mm dia concrete	CCC AP65	1.5	**4.1**	No	No
6	DN600 PE access	CCC AP65	2.0	**5.1**	No	No
7	PE Pressure Sewer	3MPa Concrete	1.2	**1.0**	Possible	No

(1) Assumes groundwater level at typical 0.5m depth below ground surface and designed dimensions.

(2) Assessed using as-built conditions and dimensions with measured pore pressure from PPTs.

Chamber uplift displacement

No observations of chamber uplift displacement above their 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.

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 pressure was 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 pressure recorded beneath the structure is 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 ejecta sand was observed beneath Test 4 and Test 7.

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 expected to exhibit satisfactory resilience with respect to buoyant uplift.

Review of the trial data indicates that bearing capacity for the underlying liquefied ground was not exceeded. This is in line with theoretical assessment.

6.2.3 Pipeline Performance

Damage to buried pipelines during the CES was largely associated with structural failure of the pipe from dynamic loading, differential settlement, and extension and compression associated with lateral spread of the ground. The SCIRT and EQC trial allows field testing of seismic performance of the refined design details for flexible pipes.

Pipe dips

Differential settlements of 0 mm to greater than 150 mm were recorded across the site within pipes of Test 1, Test 2 and Test 8. This highlights the vulnerability of horizontal infrastructure to variable and differential settlement. Competent pipe haunching and well graded granular backfill within the trench provided negligible mitigation in the magnitude or rate of differential settlement.

Composite action between the pipe and the trench backfilled with well compacted granular materials has limited influence on reducing the rate of total settlements. This is in line with observations during the CES, with the most common pipe defect for modern PVC-U pipe installations being pipe dips. Design of future pipeline infrastructure should consider this residual vulnerability to differential settlements. Selection of system type and appropriate detailing of installed pipe grades and alignments could provide a small reduction in vulnerably. In most instances piling and/or ground improvement are not economically feasible and generally such measures focus or shift the location of the differential settlement, offering limited or little additional resilience.

Structural performance

Pipe materials historically used in Christchurch exhibited poor performance during the CES (earthenware, asbestos cement, cast iron and reinforced concrete). The dominant pipe material type selected and incorporated into the rebuild was PVC-U pipe, which provides superior seismic resilience and improvement in constructability. Pipelines constructed by trenching methods incorporated spigot and socket pipe with rubber seal, while the horizontal directionally drilled pipeline in the trial utilised an alternative Restrain™ threaded socket and spigot pipe.

Well-constructed below ground horizontal infrastructure generally exhibited good structural performance during the Liquefaction Trial. With the exception of the directionally drilled pipe (Test 8) no structural failure of the pipes were observed. The ground motion acceleration experienced during the trial was significantly higher frequency and amplitude than would be experienced during a natural earthquake. However, the magnitude of vertical and horizontal ground deformation in the order of <10 mm is significantly less than a natural earthquake. Dynamic ground displacements in Christchurch during the CES were in order of 100-150 mm for the 22 February 2011 earthquake and 200-300 mm for the 4 September 2010 event. The trial infers that the PVC-U pipeline structural performance will likely be satisfactory during an earthquake though some localised failures associated with large strain compression/tension are possible.

One threaded socket and spigot joint on the directionally drilled restrain pipe failed during the trial explosive detonation (Test 8). The close proximity of the explosive charge (~2 m separation) and the locally reduced effective pipe wall thickness at the threaded joint are likely contributing factors to the joint failure.

A stiff SN16 pipe was used during the Liquefaction Trial and exhibited good resistance to ovaility deformation both under construction, static and post dynamic conditions during the trial. Pipe ovality during the strong ground motion was not recorded, and was likely greater than recorded, however no visible evidence of pipe damage or structural fatigue was observed.

Pipe dips induced along the pipes were not sufficiently abrupt to exceed the maximum bend radius specified by the pipe manufacturer, or to induce structural damage.

All pipe connections to the manholes and pressure sewer chambers exhibited adequate performance and no pipe damage to chambers or connector. However, differential movements were relatively small.

Haunching and backfill performance

The CCC specified haunching material for flexible pipes comprised NZTA M4 AP20 which is a well graded crushed gravel hardfill. Placement of well graded gravel in wet or saturated conditions, frequently encountered in Christchurch, can reduce pipe laying productivities. At the time of the trial SCIRT was considering alternative haunching materials with lower fines content. Trial testing incorporates a range of extreme haunching materials including: NZTA M6 Grade 2 chip, 8-10 mm pea gravel, and 10/14 chip. No appreciable difference in seismic performance between the haunching materials was observed during the trial. Ejecta dykes from beneath the trenches intersected the base of the haunching and migrated vertically, locally filling the void space between aggregate particles with fine sand. This does not adversely affect future performance of the pipe haunching. Observations for the CCC AP65 backfill were in line with the haunching.

Migration of adjacent potentially liquefied soils laterally into the trench haunching and backfill was not observed. This observation does not provide conclusive evidence that this is not anticipated because the silty nature and cohesive soil layers encountered at the trial site limited liquefaction and mobility of the soil, and the observation that majority of the liquefaction occurred below 3 m. The evidence of the ejecta materials from below the trench migrating into the trench indicates that sands with low fines content could potentially migrate into a trench haunching or backfill having a high void content.

The annulus of the horizontally directionally drilled pipe (Test 8) filled with drilling mud did not exhibit any adverse or significant deformation. The cohesive native soil surrounding the pipeline in the areas exhumed likely did not liquefy.

7 Earthquake Rebuild Infrastructure Resilience

Observations from the trial have been compared against theory, design assumptions, and observations of damage and performance of buried infrastructure during the CES. Though the trial is not directly representative of all ground conditions found in Christchurch and natural earthquake characteristics, it provides an effective means to assess the effects of liquefaction and settlement on buried infrastructure in a controlled manner.

Greater extents of liquefaction, a longer duration of shaking, lateral spreading and persistent elevated pore pressure due to aftershocks is expected in a natural earthquake sequence. During a significant natural earthquake sequence a portion of Christchurch's buried infrastructure will be exposed to more severe conditions than experienced in the SCIRT and EQC Trial.

The SCIRT and EQC trial observations indicate that the CCC and SCIRT design details adopted in the Christchurch rebuild are a pragmatic, practical solution exhibiting an appropriate level of resilience and optimised value.

The standard details are appropriate for the majority of conditions in Christchurch, however exceptions will occur. During a significant earthquake, damage to Christchurch's buried infrastructure will occur, requiring repair or replacement. This is due to the infrastructure designs not mitigating or eliminating liquefaction potential or the consequential lateral spreading, differential settlement, buoyant uplift, and dynamic soil structure interaction. However, the anticipated extent, severity and influence on post disaster functionality will be somewhat improved from what was observed during the earthquakes of 2010/2011. Structural damage to pipes will be reduced due to use of materials exhibiting improved seismic durability and flexibility. The potential and frequency of chamber buoyant uplift will be significantly reduced. However, the influence of lateral spread and differential settlement on pipelines will remain resulting in damage associated with pipe extension/ dislocation and pipe dips, respectively. The impact of differential settlement to gravity pipes can be reduced through steepening minimum design grades, incorporating additional pump stations, or adopting alternative technologies such as pressure sewer or vacuum sewer.

The resilient design improvements will provide a positive benefit in post disaster functionality and assist with enabling a more controlled and programmed rebuild.

8 Recommendations and Conclusions

The SCIRT and EQC Liquefaction Trial provided a controlled field assessment of the performance of below ground chambers and pipes in liquefied soils. Interpretation of the trials observations and data supports geotechnical design theory, anticipated relative performance and modes of failure. The performance of the buried infrastructure in the trial is in line with the examples of existing infrastructure that generally provided good observed performance in Christchurch during the CES. The trial findings support the resilient design solutions incorporated into the SCIRT rebuild of horizontal infrastructure to address examples where performance during the CES was not good, e.g. brittle pipes including those made of earthenware and asbestos cement.

Key recommendations for design are listed below:

- Variability in soil type and liquefaction potential over short distances can lead to significant differential settlement. Therefore designers and asset owners should consider residual differential settlement risk and its influence on asset functionality during project scoping and design. PVC-U pipelines exhibited pipe dips as the dominant mode of damage during the CES. This shows that a well-constructed trench, backfilled with well compacted granular materials, will support the pipe but has little influence on differential settlement.
- Use of native soils for backfill is not recommended unless stabilised or adequate compaction can be provided and demonstrated to mitigate liquefaction triggering.
- Designers should focus on simple designs and detailing for buried infrastructure and their backfill to provide improved resilience through lower sensitivity to construction tolerance and/or quality, and future maintenance. The harder it is to construct the more likely it will be constructed incorrectly, potentially affecting the performance during an earthquake.
- An extended chamber base is an effective method for limiting potential for buoyant uplift that utilises the effective weight of backfill materials to resist the buoyant forces. This design allows flexibility in backfill type, provides access for future maintenance, and is a cost effective solution of high value.
- Relieving excess pore pressure by drainage with highly permeable backfill within a sewn geotextile bag is effective at limiting uplift pressure. Resilience of this method is dependent on preventing migration of fines into the backfill.
- The low excess pore pressure observed within the well graded granular backfill (CCC AP65) beneath the chambers during the trial, suggests that the level of resilience provided by the backfill is likely to be higher than often assumed in design. Further laboratory testing is required before a reliable conclusion could be drawn. The well graded granular backfill is a pragmatic solution.
- Adding mass to a structure is a feasible solution to mitigate buoyant uplift however the effectiveness reduces if adding mass results in an increased impermeable volume below ground.

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A

Appendix A Photographic Records

General Construction

Photo 1: General site photo

Photo 2: General site photo

General Construction (continued)

Photo 3: General site photo

Photo 4: General site photo

Construction Test 1: Pipe with easily compacted granular haunching

Photo 5: Test 1 – General photos

Photo 6: Test 1 – Easily compactible haunching

Construction Test 2: Pipe with well graded granular haunching

Photo 7: Test 2 – Trench wall soil profile

Photo 8: Test 2 – NZTA M/4 AP20 haunching and CCC AP65 backfill

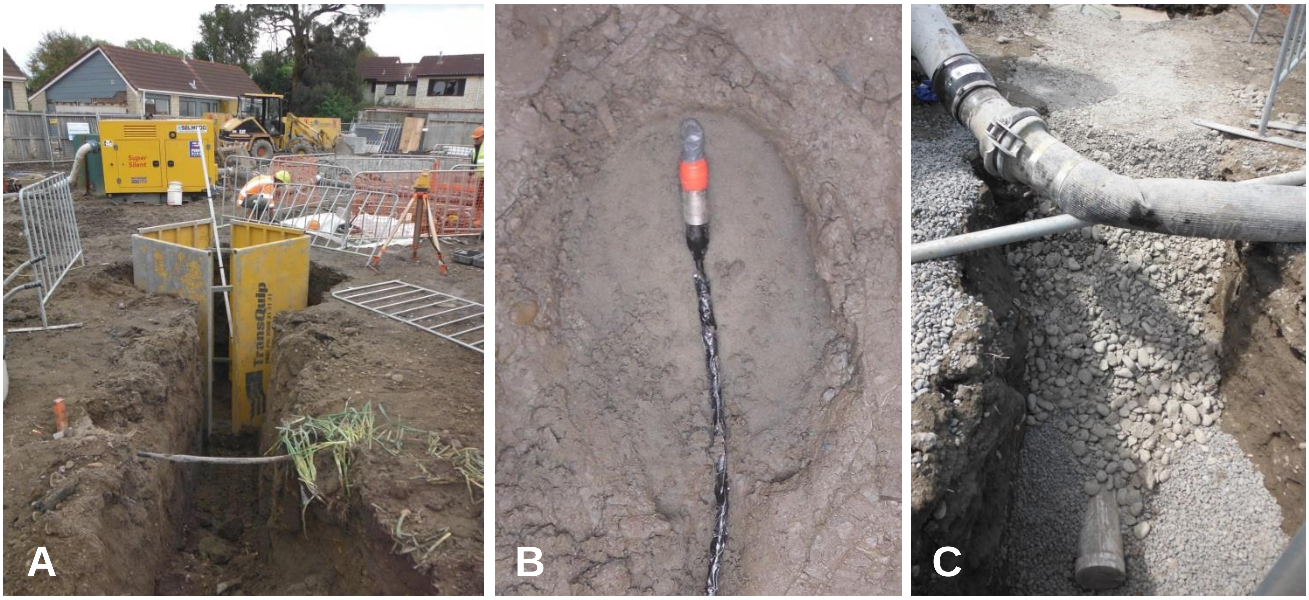
Construction Test 3: Pressure Sewer Chamber, reliving porewater pressure

Photo 9: Test 3 – Installing PPT within uniformly graded backfill beneath pressure chamber

Photo 10: Test 3 – Installing chamber

Photo 11: Test 3 – Backfilled chamber prior to folding geotextile over the backfill and placing topsoil

Construction Test 4: Pressure Sewer Chamber, native sand backfill



[A] General site.

[B] Installation of PPT.

[C] CCC AP65 Backfill and 14/10 chip haunching for connecting lateral.

Photo 12: Test 4 – General photos

[D] Preparation of foundation soils.

[E] Installation of pressure sever chamber prior to backfill.

Photo 13: Test 4 – Installation of chamber

Construction Test 5: Standard concrete manhole chamber with AP65 backfill

Photo 14: Test 5 – Excavation and ground conditions

[A] PPT installed within CCC AP65.

[B] AP65 foundation prior to installing chamber.

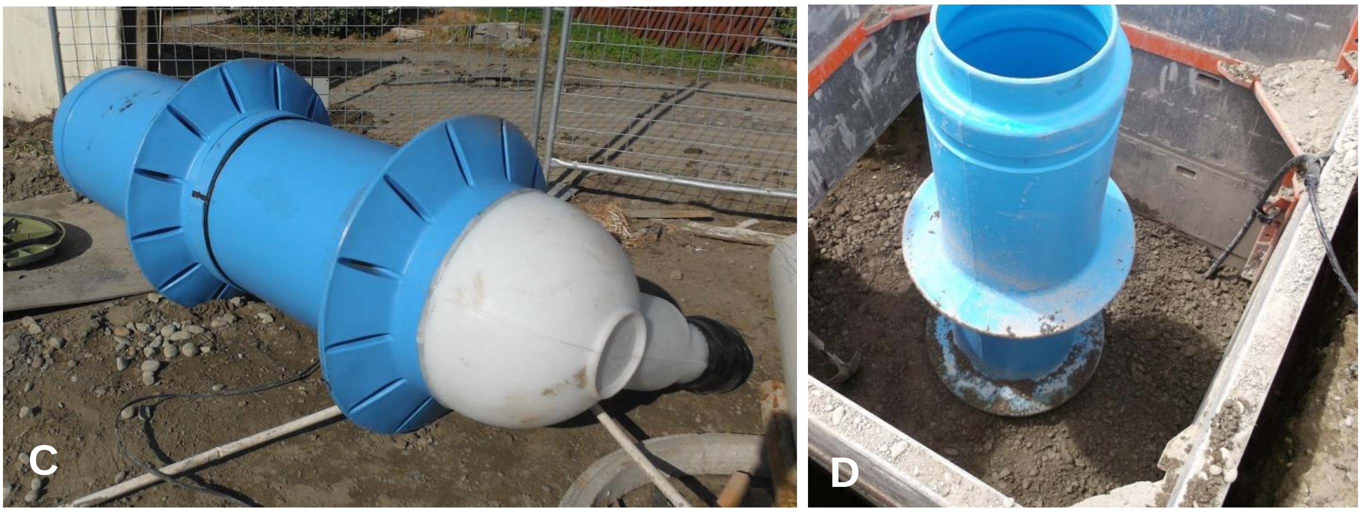
Photo 15: Test 5 – Preparing foundation for chamber

Construction Test 6: PE manhole chamber with AP65 backfill

[A] Installing PPT within CCC AP65.

[B] AP65 foundation prior to installing chamber.

Photo 16: Test 6 – Preparation beneath chamber



[A] Smartstream 600 Mini Manhole

[B] Installation of chamber.

Photo 17: Test 6 – Installation of chamber

Construction Test 7: Pressure Sewer Chamber, low strength concrete backfill

Photo 18: Test 7 – Pressure sewer chamber prior to installation

Photo 19: Test 7 – Low strength concrete backfill

Construction Test 8: Directionally Drilled Pipeline

Photo 20: Test 8 – Installation of directionally drilled pipeline within a hole reamed to 250 mm diameter

Photo 21: Test 8 – Iplex Restrain™ pipe joints, and pipe connector for drawing pipe though reamed hole

Instrumentation

Photo 22: Instrumentation – PPT installed within native soil to assess liquefaction triggering

Photo 23: Instrumentation – Vertical settlement profilers

Explosive charge installation

Photo 24: Explosive charge installation – General site 23 October 2013

Photo 25: Explosive charge installation – Blast holes, stemming, explosive charges and detonation control

Liquefaction Triggering – 24 October 2013

Photo 26: Liquefaction triggering – Immediately liquefaction triggering

Photo 27: Liquefaction triggering – Immediately liquefaction triggering

Liquefaction Triggering – 24 October 2013 (continued)

Photo 28: Liquefaction triggering – Immediately liquefaction triggering

Photo 29: Liquefaction triggering – Immediately liquefaction triggering

Liquefaction Triggering – 24 October 2013 (continued)

[A] Test 4.
[B] Test 3.
[B] Test 5.

Photo 30: Liquefaction Triggering – Test 3, 4 & 5

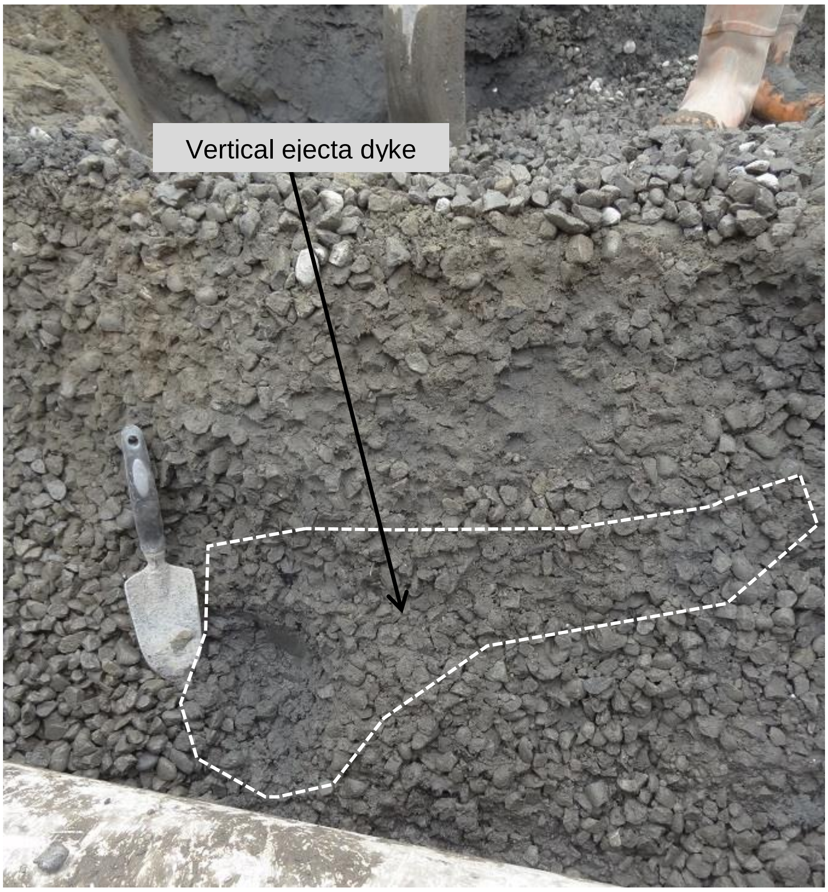
[D] Ground cracking from blast heave.
[E] Liquefaction ejecta from charge borehole.

Photo 31: Liquefaction Triggering – Visible land damage

Exhuming Test 1: Pipe with easily compacted granular haunching

Photo 33: Exhuming Test 1 – Exposing CCC AP65 Backfill and NZTA M6 Grade 2 roading chip haunching

Exhuming Test 1: Pipe with easily compacted granular haunching



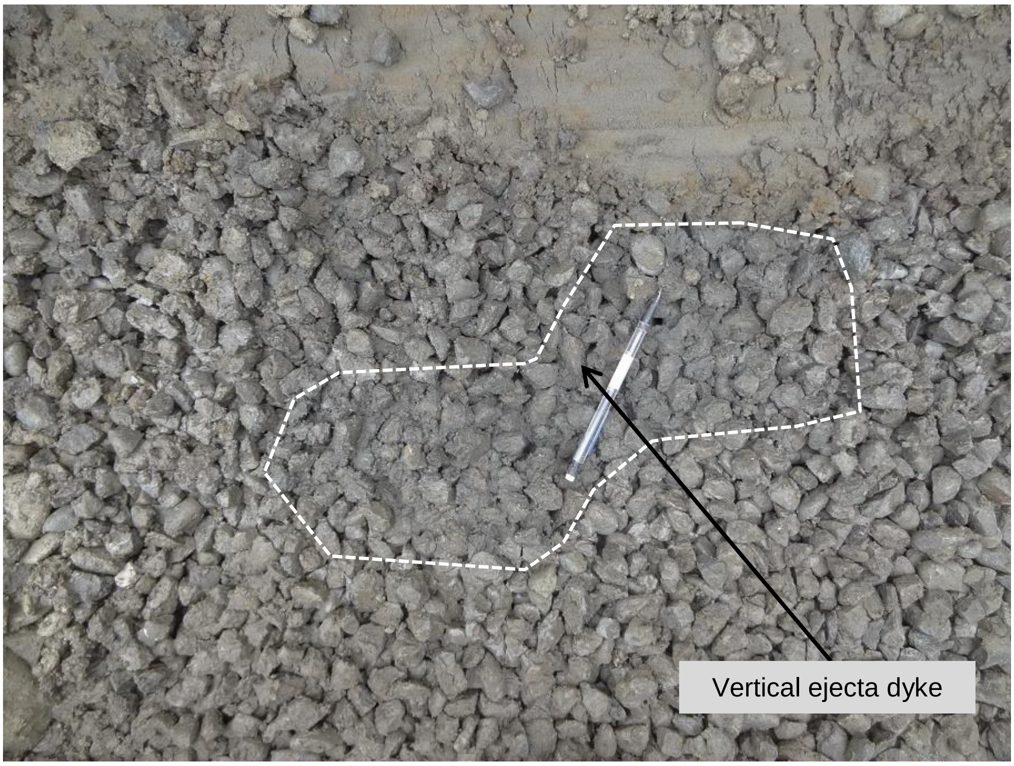


Photo 34: Exhuming Test 1 – Exposing CCC AP65 Backfill and NZTA M6 Grade 2 roading chip haunching

Exhuming Test 2: Pipe with well graded granular haunching

Photo 35: Exhuming Test 2 – Excavating and exposing DN15 SN16 PVC-U pipe within NZTA M4 AP20

Exhuming Test 2: Pipe with well graded granular haunching

[A] Slowly exposing the side interface between AP20 haunching and native soil.

[B] No migration of soil from side wall into haunching.

[C] Native soil from immediately beneath the trench did not migrate into haunching.

Photo 36: Exhuming Test 2 – Exposing interface between NZTA M/4 AP20 haunching and native soil

Exhuming Test 2: Pipe with well graded granular haunching

[D] Clean interface between base of trench and native ground, no deformation.

[E] Localised vertical ejecta dykes intersected haunching, fine sand from depth.

[F] Ingress of ejecta material into well graded NZTA M4 AP20 haunching.

Photo 37: Exhuming Test 2 – Migration of ejecta material into haunching

Exhuming Test 3: Pressure Sewer Chamber, reliving porewater pressure

[A] Commencing exhuming Test 3.

[B] Opening sewn geotextile bag to expose permeable Grade 2 chip backfill material.

[C] Inclusion of sand within backfill – considered to be inclusion of material during construction not ingress of ejecta sand.

[D] ~7mm of settlement of fill materials above permeable backfill relative to chamber.

Photo 38: Exhuming Test 3 – Exhuming Test 3 chamber with permeable backfill

Exhuming Test 3: Pressure Sewer Chamber, reliving porewater pressure

[E] Pealing geotextile away from native soil. Fine layer of sand observed at the soil/ geotextile interface. No evidence to suggest that geotextile has been clogged with fines.

[F] Observed ejecta dyke (fine sand) outside geotextile bag at interface with native soil.

[G] Piecewise deconstruction of Test 3.

[H] Clean uniformly graded backfill, exposed lateral pipe. No observed ingress of ejecta materials.

Photo 39: Exhuming Test 3 – Exhuming Test 3 chamber with permeable backfill

Exhuming Test 3: Pressure Sewer Chamber, reliving porewater pressure

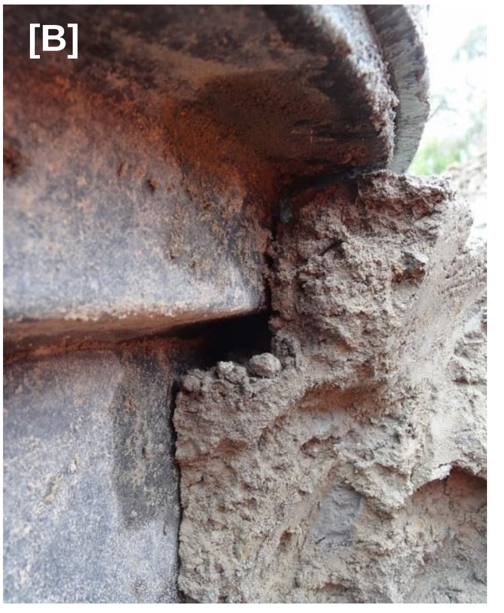
[I] Excavation extent when exhuming infrastructure.

[J] Unwrapping PPT.

[K] Exposing PPT within permeable backfill beneath presser sewer chamber.

Photo 40: Exhuming Test 3 – Exhuming Test 3 chamber with permeable backfill

Exhuming Test 4: Pressure Sewer Chamber, native sand backfill



[A] Commencing exhuming.

[B] Settlement of sand backfill relative to chamber.

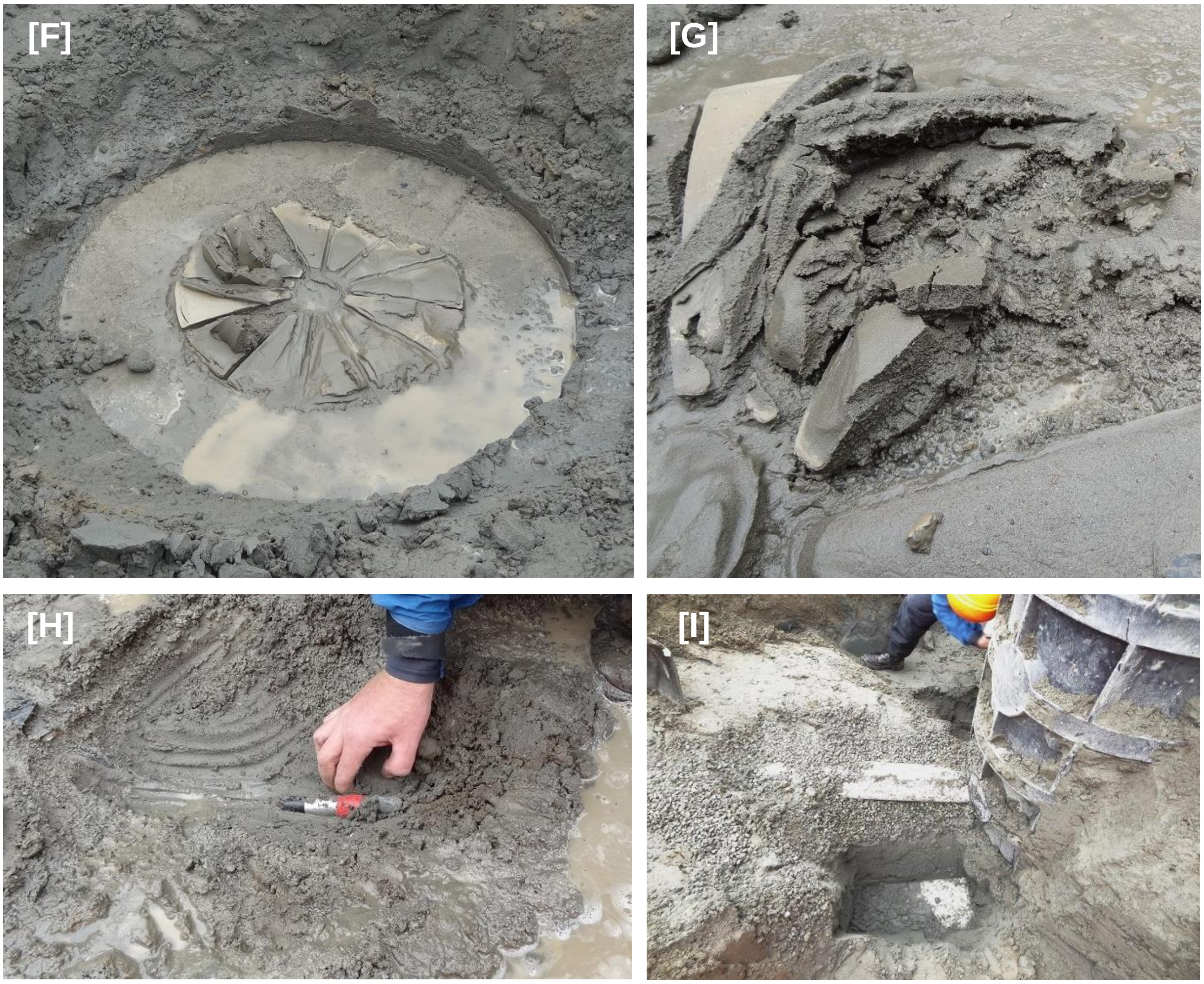
[C] Incremental exposure of chamber.

[D] Hand excavating adjacent to chamber.

[E] Chamber prior to lifting out.

Photo 41: Exhuming Test 4 – General exhuming photos.

Exhuming Test 4: Pressure Sewer Chamber, native sand backfill



[F] Underside of chamber upon removal of Pressure Sewer chamber.

[G] Accumulation of fine sand (ejecta) beneath the base of the chamber – 10-15mm.

[H] Exposing PPT form beneath chamber.

[I] Connecting lateral to chamber.

Photo 42: Exhuming Test 4 – Exposing ejecta and PPT beneath chamber

Exhuming Test 4: Pressure Sewer Chamber, native sand backfill

[J] Exposed 14/10 chip haunching with vertical intersection of ejecta (fine to medium sand).

[K] No evidence of migration of adjacent native soil into uniformly graded haunching.

[L] Close up of ingress of ejecta sand though haunching.

Photo 43: Exhuming Test 4 – Exhuming 14/10 chip haunching around Test 4 lateral.

Exhuming Test 5: Standard concrete manhole chamber with AP65 backfill

Photo 44: Exhuming Test 5 – General photos of excavation and ground conditions

Exhuming Test 5: Standard concrete manhole chamber with AP65 backfill

[A] General photos of exhumed standard concrete manhole.

[B] Long socket manhole connector

[C] Settlement of backfill material relative to manhole lid.

[D] Minor ingress of liquefaction ejecta sand into AP65 backfill.

Photo 45: Exhuming Test 5 – General photos and close up of AP65 backfill

Exhuming Test 5: Standard concrete manhole chamber with AP65 backfill

[E] Backfill beneath chamber upon removal.

[F] Exhumed PPT in good condition.

Photo 46: Exhuming Test 5 – Exposing soil beneath chamber and PPT

Exhuming Test 6: PE manhole chamber with AP65 backfill

Photo 47: Exhuming Test 6 – General photos of exhuming backfill around Smartstream 600 Mini Manhole

Exhuming Test 6: PE manhole chamber with AP65 backfill

[A] Settlement of backfill material relative to manhole ribs.

[B], [C]

Plywood beneath chamber to prevent damage to PPT

[D] Exposed undamaged PPT.

[E] General photo of site following exhuming chamber.

Photo 48: Exhuming Test 6 – AP65 backfill chamber interface, and PPT beneath chamber

Exhuming Test 7: Pressure Sewer Chamber, low strength concrete backfill

[A], [B] & [C] Exposing the top surface of 3MPa concrete backfill and surrounding soil conditions.

[D] Excavation beneath low strength concrete.

Photo 49: Exhuming Test 7 – Excavation and ground conditions

Exhuming Test 7: Pressure Sewer Chamber, low strength concrete backfill

[E] Cohesive nature of soils.

[F] Concrete cores taken for lab measurement of density.

[G] PVC-U sleeve though concrete backfill.

[H] Sealing lateral to prevent ingress of concrete backfill at chamber end.

[I] Deconstructing low strength concrete backfill.

Photo 50: Exhuming Test 7 – exposing low strength concrete backfill

Exhuming Test 7: Pressure Sewer Chamber, low strength concrete backfill

[J] Soil beneath chamber upon removal.

[K] Lifting plywood which was placed over PPT to protect.

[L] Exposing PPT within native soil.

Photo 51: Exhuming Test 7 – Exposing soil beneath chamber and PPT

Exhuming Test 7: Pressure Sewer Chamber, low strength concrete backfill

[M], [N], [O], & [P]

Ingress of vertical liquefaction ejecta pipe into pea gravel hauncing, depositing sand and a ring of silt.

[Q] No ingress of native soil into haunching material.

Photo 52: Exhuming Test 7 – Exposing lateral; with pea gravel haunching

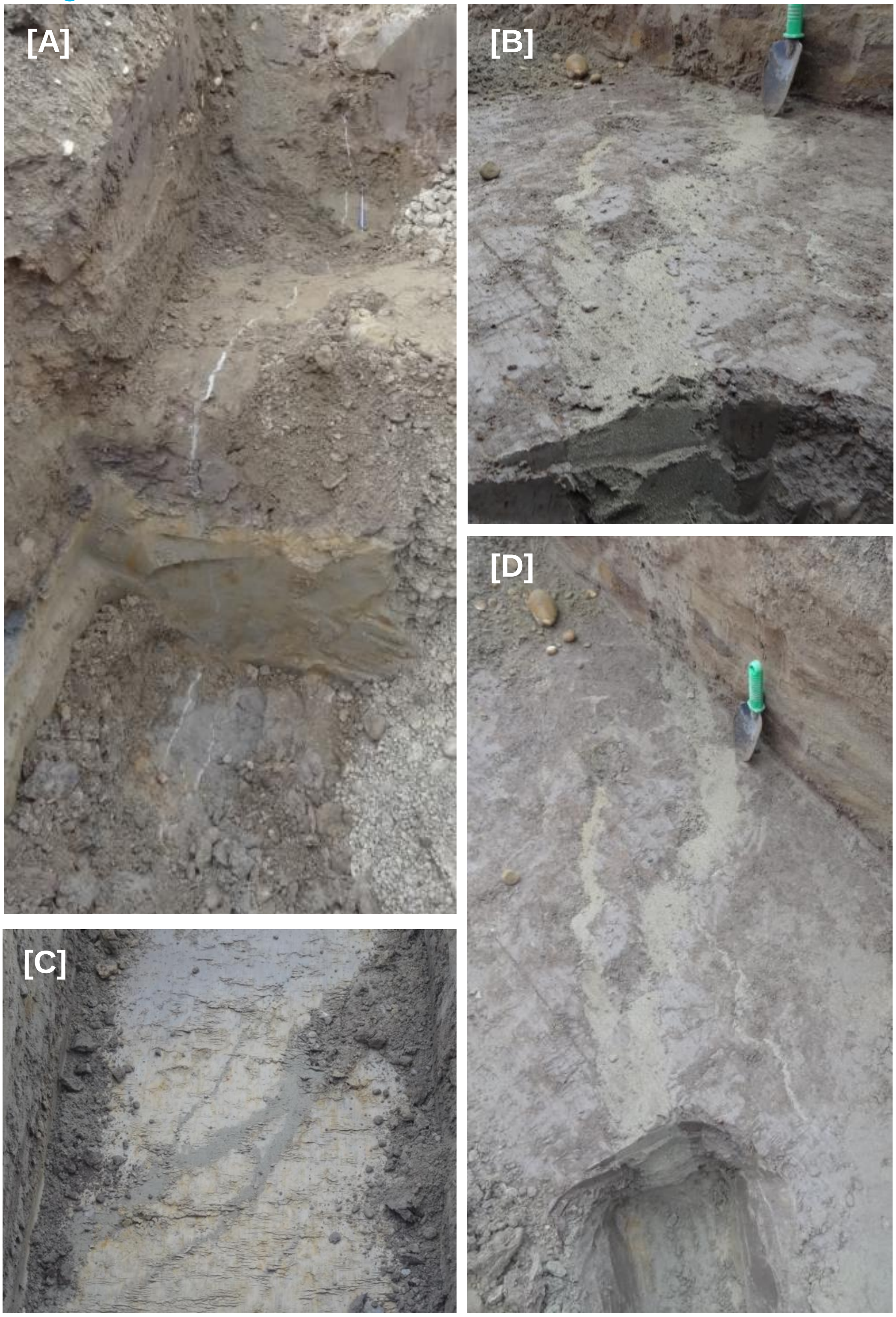
Exhuming Test 8: Directionally drilled pipeline

Photo 53: Exhuming Test 8 – Exposed section of undamaged directionally drilled pipe

Exhuming Test 8: Directionally drilled pipeline

Photo 54: Exhuming Test 8 – Exposing damaged section of pipe, possible additional damage induced by excavator

Exhuming: Observations of paleoliquefaction



[A] Thin (<10mm) vertical seams filled with light blue grey sandy Silt.
[B], [C], [D] Yellow grey fine to medium Sand within defects formed by lateral spread.

Photo 55: Exhuming – Paleoliquefaction observations

B

Appendix B Geotechnical Investigation Data

Geotechnical Investigation Location Plan

Figure B1 – Location of Trial Geotechnical Investigations

Geotechnical investigation data can be sourced in electronic format form the Canterbury Geotechnical Database, https://canterburygeotechnicaldatabase.projectorbit.com.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: AVD-TCR01-BH07
Hole Location: 27 Ardossan Street

PROJECT: Ground Improvement Trials | LOCATION: Site Four | JOB No: 52020.022

CO-ORDINATES: 5745060.09 mN 2484666.18 mE
R.L.: 1.65 m
DATUM: NZMG, MSL (CCC 20/01/12 Datum -9.043m)

DRILL TYPE: Fraste Multidrill - XL
DRILL METHOD: Sonic
DRILL FLUID: Drill Pro/Water

HOLE STARTED: 30/8/13
HOLE FINISHED: 30/8/13
DRILLED BY: Prodrill - Kane
LOGGED BY: JXXM CHECKED: BMcD

GEOLOGICAL

Depth (m)	Geological unit	Core recovery (%)	Method	Tests
0.0–0.95	Fill	100	Sonic	*FC@0.6m = 24.2%
0.95–1.25	Previous Topsoil Surface	100	Sonic	
1.25–1.65	Yaldhurst Member of the Springston Formation	100	Sonic	*FC@1.1m *= 64.9% Atterburg limits @1.1m
1.65–5.0	Christchurch Formation	100 (1.5–3.0), 100 (3.0–4.5)	Sonic	*FC@2m = 50.8%; *FC@3m = 6.2%; *FC@4m = 6.7%

ENGINEERING DESCRIPTION

Classification symbol	Moisture condition	Soil description
ML	W	TOPSOIL - SILT with some organics and trace sand and gravel, grey, low plasticity, slow dilatancy. Organics are fibrous. Sand is fine. Gravel is fine, subrounded.
		Sandy SILT with some sandy pockets/lenses and trace gravel and organics, brownish grey, very low plasticity, quick dilatancy. Gravel is fine to medium, subrounded. Organics are fibrous.
SM		Fine SAND with some silt and gravel, brownish grey. Gravel is fine to coarse, subrounded.
ML		SILT with minor gravel, organics and thin sand lenses, grey, low plasticity, slow dilatancy. Gravel is fine to medium, subrounded. Organics are amorphous.
SM		Silty fine SAND with trace gravel and organics, grey. Gravel is fine to medium, subrounded. Organics are amorphous. - 2.0m: grades to trace wood flecks.
MH	M	SILT with minor clay and trace ceramic pipe and gravel, grey, medium to high plasticity, non-dilatant. Gravel is medium to coarse, subrounded. - 1.35m: 20mm Organic SILT lens, dark grey. - 1.37m: grades to sandy with clay, ceramic pipe and gravel absent, non-plastic, quick dilatancy.
ML	Sat	SILT with minor sand and trace gravel, grey, low plasticity, quick dilatancy. Sand is fine. Gravel is medium to coarse, subrounded.
SM	W	Silty fine SAND, grey. - 1.85m: 20mm SILT lens. - 1.9m: grades to some silt.
ML		SILT with trace sand and organics, grey, low plasticity, slow dilatancy. Sand is fine. Organics are amorphous and fibrous.
SM		Silty fine SAND with some thin silt lenses and trace wood flecks, grey. - 2.4m: grades to some silt. Thin silt lenses and wood fragments absent.
ML		SILT with minor thin sand lenses and trace organics, grey, low to moderate plasticity, very slow dilatancy. Sand is fine. Organics are fibrous.
SM		Fine SAND with some silt, grey. - 2.95m: 10mm SILT lens.
SP		- 3.0m: grades to minor silt. Sand is fine to medium, predominantly fine. - 3.5m: grades to trace gravel. Gravel is fine to medium, subrounded. - 3.8m: grades to minor gravel. - 4.0m: grades to trace gravel. - 4.1m: grades to gravel absent. Sand is fine. - 4.5m: Sand becomes fine to medium, predominantly fine. - 4.8m: grades to trace gravel. Gravel is fine, subrounded.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: AVD-TCR01-BH07
Hole Location: 27 Ardossan Street

PROJECT: Ground Improvement Trials | LOCATION: Site Four | JOB No: 52020.022

CO-ORDINATES: 5745060.09 mN, 2484666.18 mE
R.L.: 1.65 m
DATUM: NZMG, MSL (CCC 20/01/12 Datum -9.043m)
DRILL TYPE: Fraste Multidrill - XL
DRILL METHOD: Sonic
DRILL FLUID: Drill Pro/Water
HOLE STARTED: 30/8/13
HOLE FINISHED: 30/8/13
DRILLED BY: Prodrill - Kane
LOGGED BY: JXXM
CHECKED: BMcD

GEOLOGICAL | ENGINEERING DESCRIPTION

Geological unit	Core recovery (%)	Method	Tests	R.L. (m)	Depth (m)	Classification symbol	Moisture condition	Soil description
Christchurch Formation	100	Sonic	*FC@5m = 10.9%	-3.5		SP	W	Fine to medium, predominantly fine SAND with minor silt and trace gravel, grey. Gravel is fine, subrounded.
						SM		- 5.2m: grades to some silt and minor gravel. Gravel is fine to medium.
				-4.0	5.5			- 5.55m: grades to gravel absent. Sand is fine.
	100	Sonic	*FC@6m = 4.1%	-4.5	6.0			- 6.0m: grades to trace carbonaceous wood flecks.
				-5.0	6.5			
						SP		- 6.85m: grades to minor silt. Sand is fine to medium, predominantly fine.
			*FC@7m = 4.5%	-5.5	7.0	SW		- 7.05m: grades to trace gravel. Sand is fine to coarse, predominantly fine to medium. Gravel is fine, subrounded.
						SP		- 7.2m: sand grades to fine to medium.
	93	Sonic		-6.0	7.5	SM		- 7.8m: grades to some silt with gravel absent.
			*FC@8m = 6.7%	-6.5	8.0			
				-7.0	8.5	SP		- 8.45m: grades to minor silt and trace gravel. Gravel is fine. - 8.6m: grades to gravel absent.
								No Recovery: 8.9 - 9.0m.
	53	Sonic	*FC@9.1m = 12.1%	-7.5	9.0	SP		
				-8.0	9.5	SM		- 9.45m: grades to some silt. Sand is fine.
			*FC@9.8m = 3.6%		10			No Recovery: 9.8 - 10.0m

Column headings (not filled): FLUID LOSS, WATER, CASING, SAMPLES, GRAPHIC LOG, WEATHERING, STRENGTH/DENSITY CLASSIFICATION, SHEAR STRENGTH (kPa) 10 25 50 100 200, COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250, DEFECT SPACING (mm) 50 250 500 1000 2000

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.
SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour.
ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.

TONKIN & TAYLOR LTD

BOREHOLE LOG

BH No: AVD-TCR01-BH07
Hole Location: 27 Ardossan Street

PROJECT: Ground Improvement Trials | LOCATION: Site Four | JOB No: 52020.022

CO-ORDINATES: 5745060.09 mN, 2484666.18 mE
R.L.: 1.65 m
DATUM: NZMG, MSL (CCC 20/01/12 Datum -9.043m)

DRILL TYPE: Fraste Multidrill - XL
DRILL METHOD: Sonic
DRILL FLUID: Drill Pro/Water

HOLE STARTED: 30/8/13
HOLE FINISHED: 30/8/13
DRILLED BY: Prodrill - Kane
LOGGED BY: JXXM | CHECKED: BMcD

GEOLOGICAL UNIT, GENERIC NAME, ORIGIN, MINERAL COMPOSITION.	FLUID LOSS	WATER	CORE RECOVERY (%)	METHOD	CASING	TESTS	SAMPLES	R.L. (m)	DEPTH (m)	GRAPHIC LOG	CLASSIFICATION SYMBOL	MOISTURE CONDITION / WEATHERING	STRENGTH/DENSITY CLASSIFICATION	SHEAR STRENGTH (kPa) 10 25 50 100 200	COMPRESSIVE STRENGTH (MPa) 1 5 20 50 100 250	DEFECT SPACING (mm) 50 250 1000 2000	SOIL DESCRIPTION: Soil type, minor components, plasticity or particle size, colour. ROCK DESCRIPTION: Substance: Rock type, particle size, colour, minor components. Defects: Type, inclination, thickness, roughness, filling.
Christchurch Formation								-8.5	10.5								No Recovery: 10.0 - 10.5m.
								-9.0	11.0								**End of Borehole at 10.5m** **Target Depth Reached**
								-9.5	11.5								
								-10.0	12.0								
								-10.5	12.5								
								-11.0	13.0								
								-11.5	13.5								
								-12.0	14.0								
								-12.5	14.5								
								-13.0	15								

T+T DATATEMPLATE-SPT.GDT saff

GEOTECHNICS
15C Amber Crescent, Judea, Tauranga
New Zealand
p. +64 7 571 0280
f. +64 7 571 0282
w. www.geotechnics.co.nz

LAB REF: 1014/13
JOB NO: 650885.000

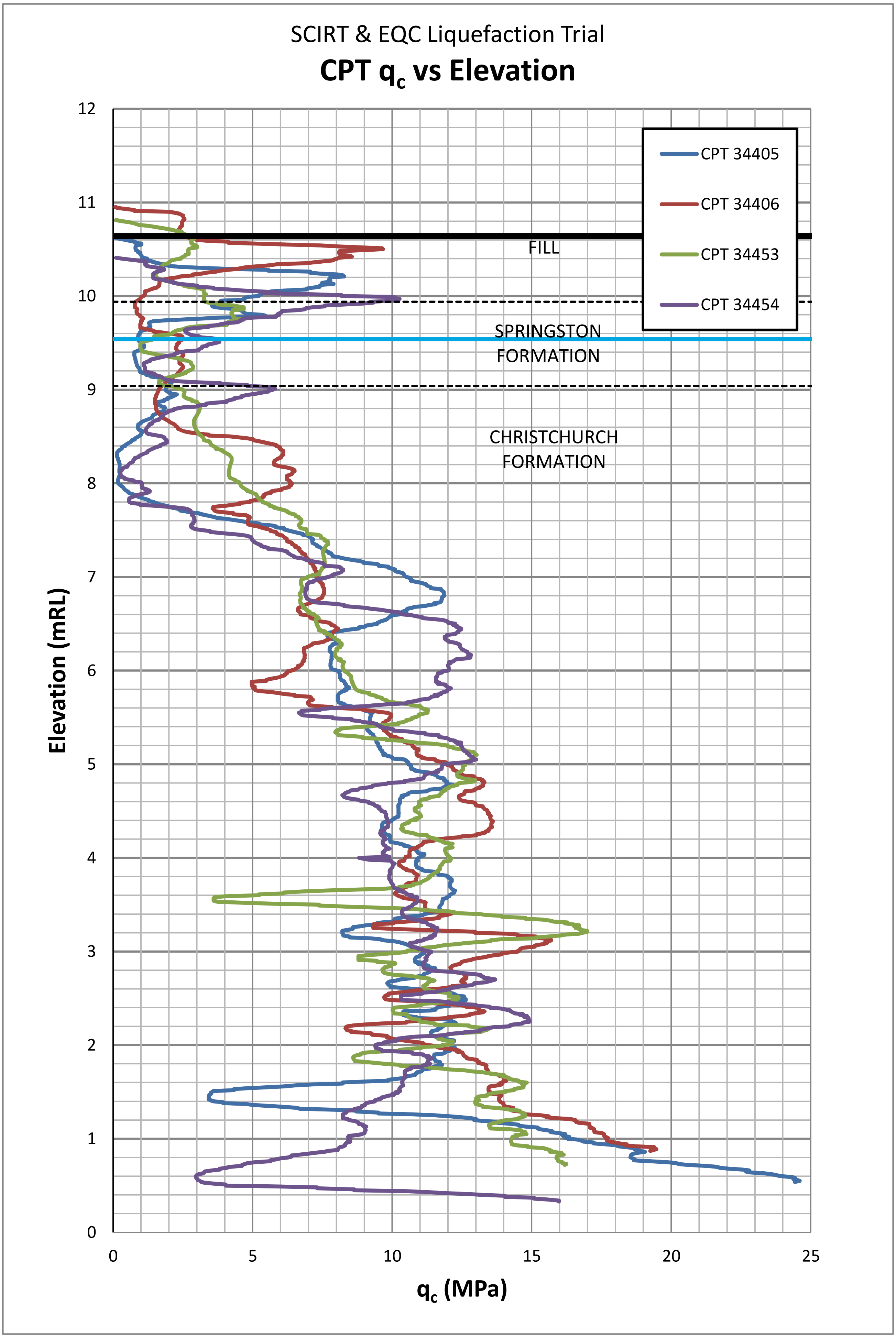
TREX LABORATORY TESTING, WORKSTREAM 1B

TEST RESULTS

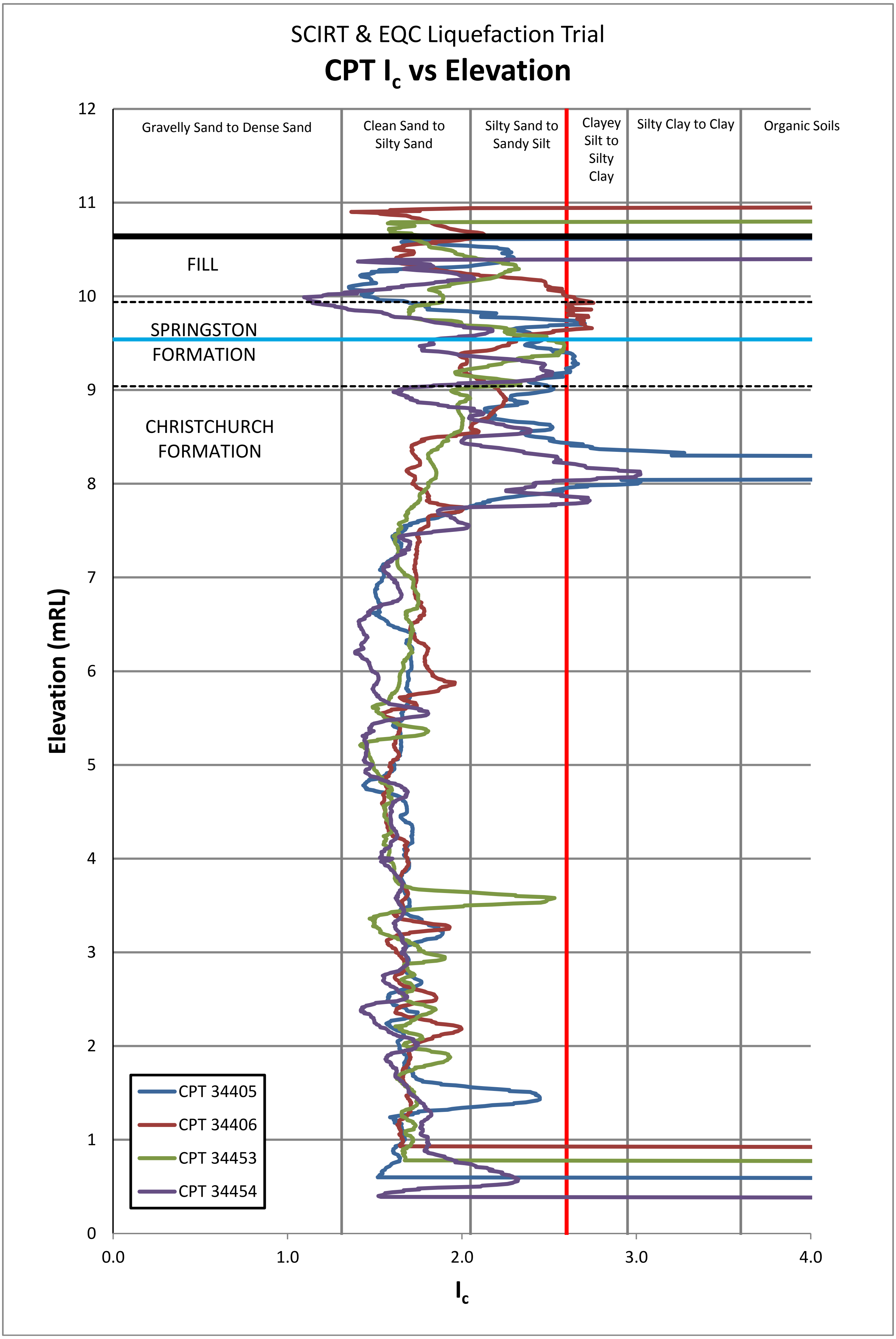
Avondale	Christchurch	AVD-TCR01-BH007

SAMPLE IDENTIFICATION	TEST RESULTS				
	LL	PL	PI	FINES CONTENT PASSING 75µm (%)	WATER CONTENT (%)
0.6m				24.15	26.5
1.1m	31	22	9	64.92	22.7
2.0m				50.76	29.4
3.0m				6.24	22.7
4.0m				6.73	21.9
5.0m				10.88	21.1
6.0m				4.07	24.3
7.0m				4.46	22.4
8.0m				6.67	21.4
9.1m				12.08	25.7
9.8m				3.55	23.9
Test method for Liquid Limit, Plastic Limit and Plasticity Index: *ASTM D4318-10*					

CHECKED:	
DATE:	16/9/13

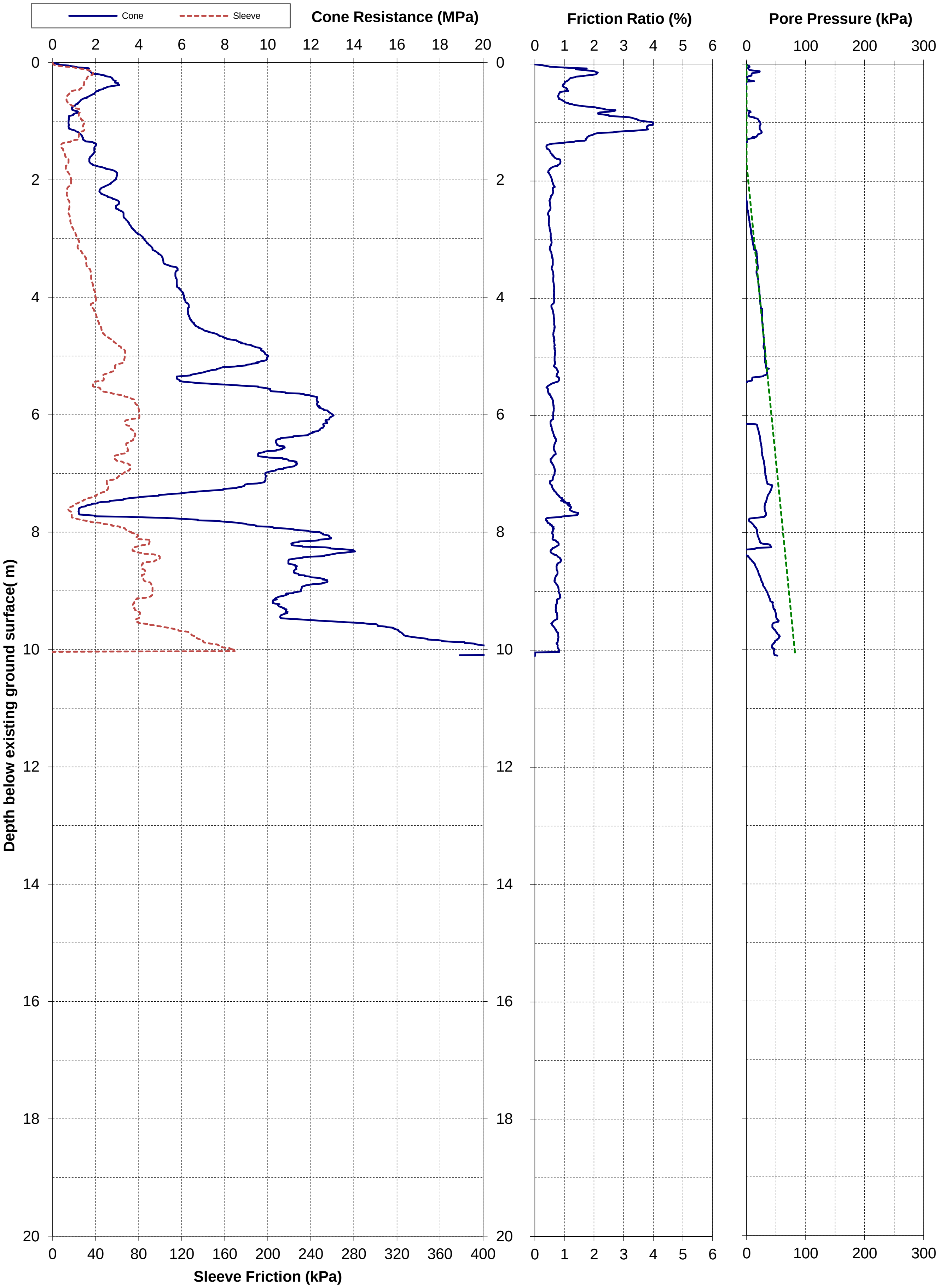


SCIRT & EQC Liquefaction Trial
CPT q_c vs Elevation
Elevation (mRL)
q_c (MPa)
0
1
2
3
4
5
6
7
8
9
10
11
12
0
5
10
15
20
25
CPT 34405
CPT 34406
CPT 34453
CPT 34454
FILL
SPRINGSTON
FORMATION
CHRISTCHURCH
FORMATION

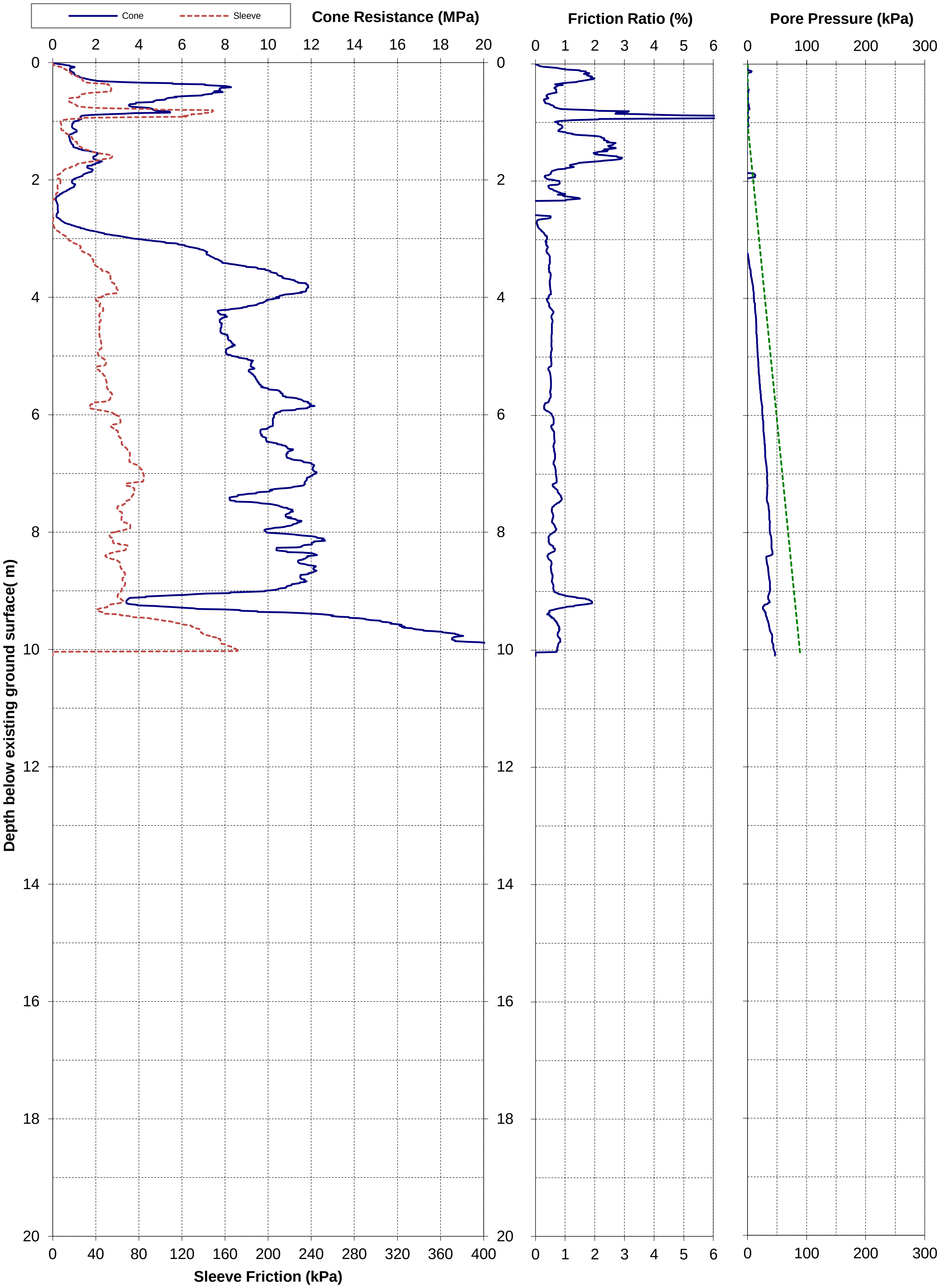


SCIRT & EQC Liquefaction Trial
CPT I_c vs Elevation
Gravelly Sand to Dense Sand
Clean Sand to Silty Sand
Silty Sand to Sandy Silt
Clayey Silt to Silty Clay
Silty Clay to Clay
Organic Soils
FILL
SPRINGSTON FORMATION
CHRISTCHURCH FORMATION
Elevation (mRL)
I_c
0
1
2
3
4
5
6
7
8
9
10
11
12
0.0
1.0
2.0
3.0
4.0
CPT 34405
CPT 34406
CPT 34453
CPT 34454

Project:	Christchurch Geotechnical Investigations			Page: 1 of 1	AVD-TCR01-CPT076
Test Date:	10-Sep-2013	Suburb:	Avondale	Operator:	Perry Geotech
Pre-Drill:	0m	Assumed GWL:	1.7mBGL	Located By:	Survey GPS
Position:	2484658.74mE 5745099.05mN		1.75mRL	Coord. System:	NZMG
Address:	1142 Avonside Dr			Datum Reference:	MSL (CCC 20/01/12 Datum -9.043)



Project:	Christchurch Geotechnical Investigations				**Page:** 1 of 1		**AVD-TCR01-CPT077**
Test Date:	10-Sep-2013	**Suburb:**	Avondale	**Operator:**	Perry Geotech		
Pre-Drill:	0m	**Assumed GWL:**	1mBGL	**Located By:**	Survey GPS		
Position:	2484685.48mE	5745062.23mN	1.59mRL	**Coord. System:**	NZMG		
Address:	29 Ardrossan St			**Datum Reference:**	MSL (CCC 20/01/12 Datum -9.043)		



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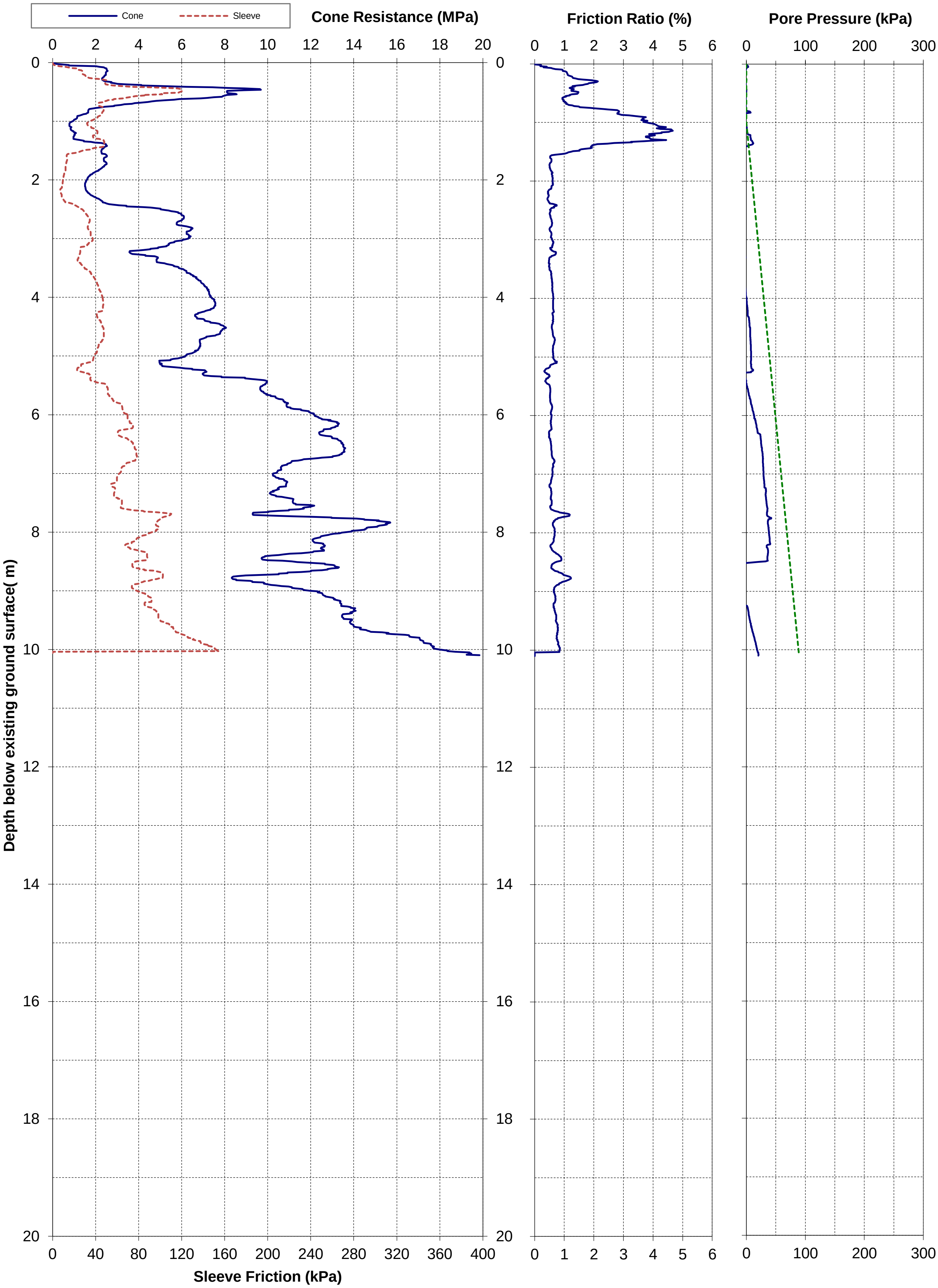
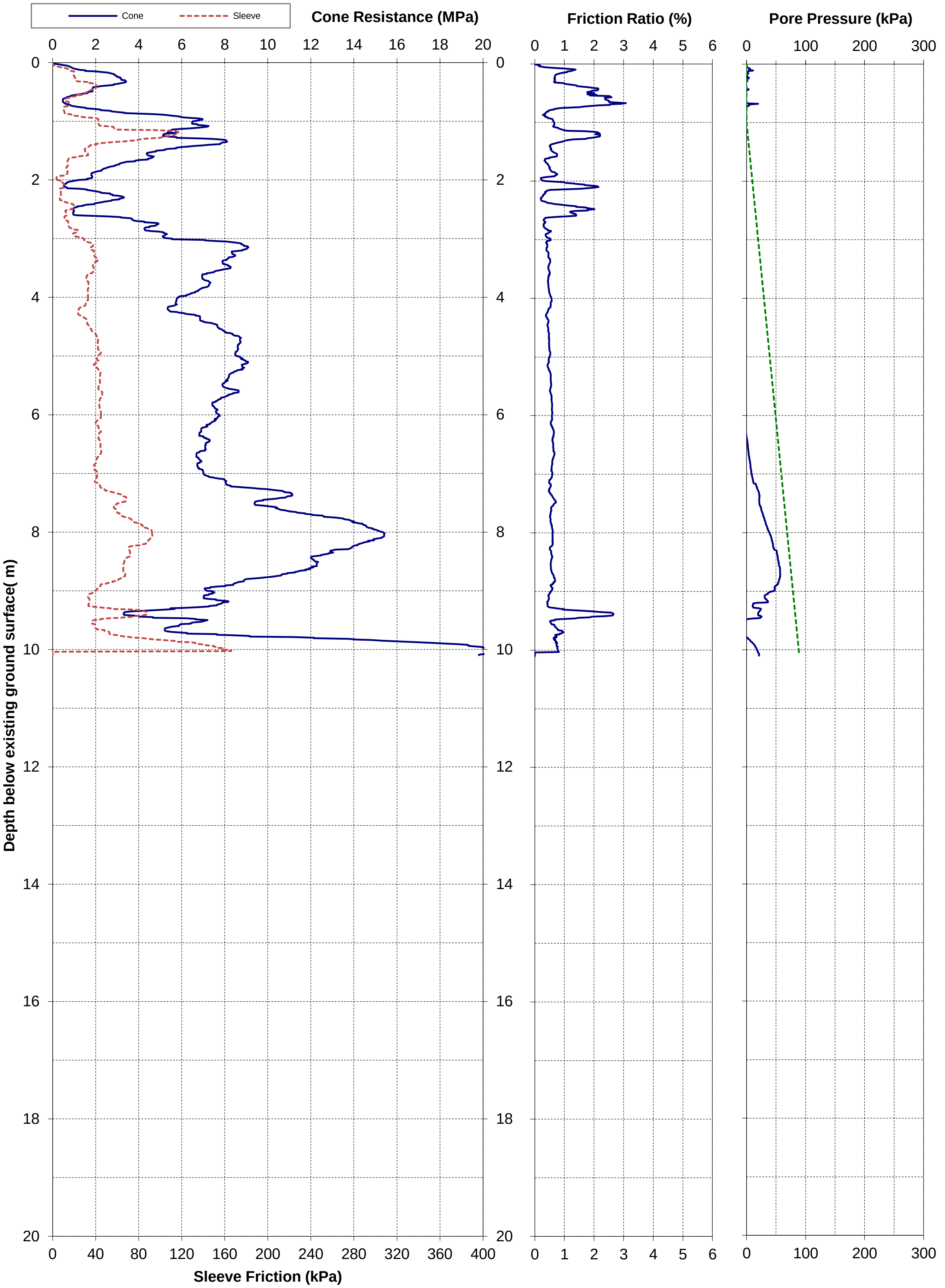
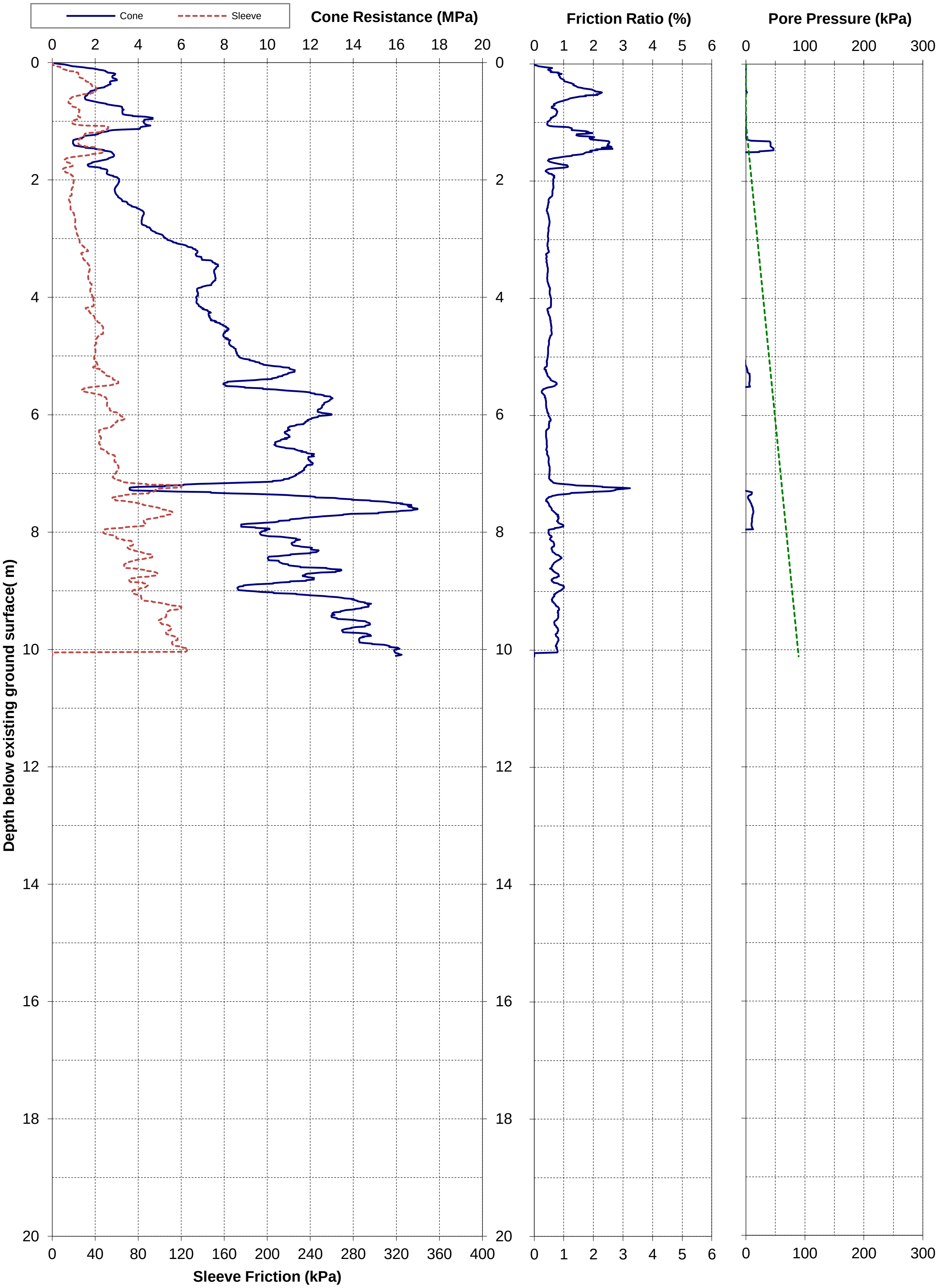
Project:	Christchurch Geotechnical Investigations			**Page:** 1 of 1	**AVD-TCR01-CPT078**
Test Date:	10-Sep-2013	**Suburb:**	Avondale	**Operator:**	Perry Geotech
Pre-Drill:	0m	**Assumed GWL:**	1mBGL	**Located By:**	Survey GPS
Position:	2484683.73mE	5745095.27mN	1.92mRL	**Coord. System:**	NZMG
Address:	29 Ardrossan St			**Datum Reference:**	MSL (CCC 20/01/12 Datum -9.043)



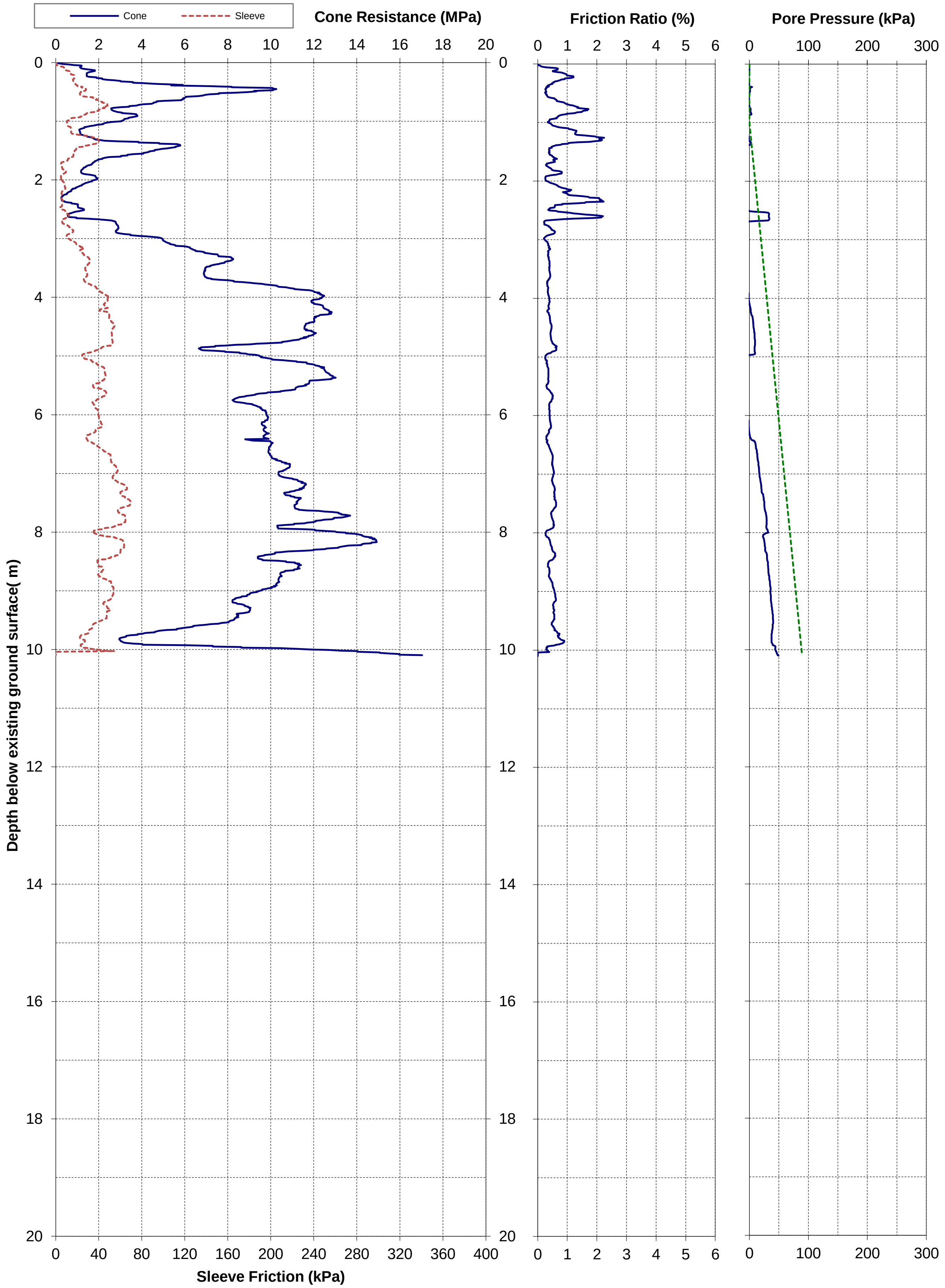
Project:	Christchurch Geotechnical Investigations			**Page:** 1 of 1	**AVD-TCR01-CPT079**
Test Date:	10-Sep-2013	**Suburb:**	Avondale	**Operator:**	Perry Geotech
Pre-Drill:	0m	**Assumed GWL:**	1mBGL	**Located By:**	Survey GPS
Position:	2484664.44mE	5745059.87mN	1.64mRL	**Coord. System:**	NZMG
Address:	27 Ardrossan St			**Datum Reference:**	MSL (CCC 20/01/12 Datum -9.043)

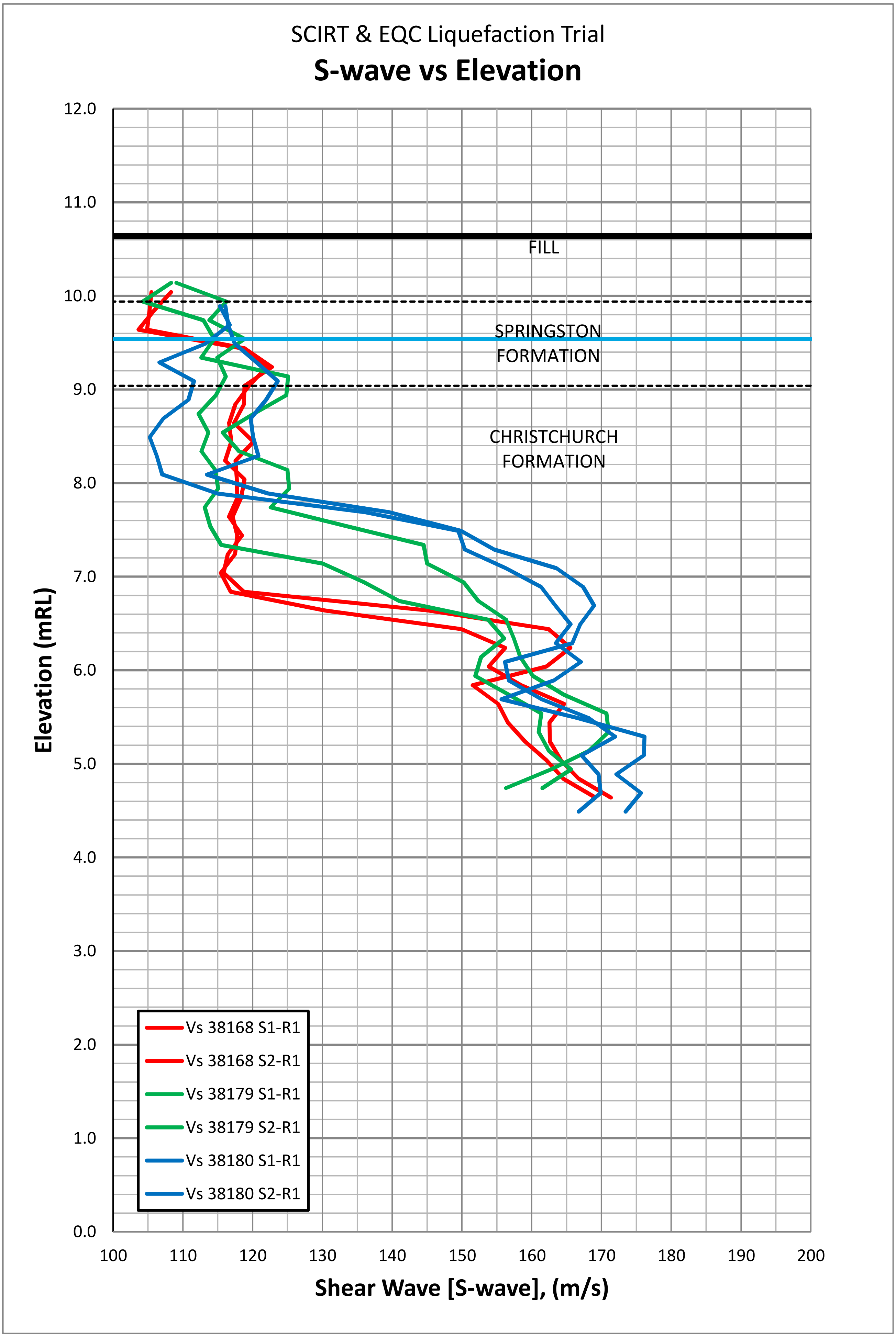


Project:	Christchurch Geotechnical Investigations					Page: 1 of 1	AVD-TCR01-CPT098
Test Date:	10-Oct-2013	Suburb:	Avondale	Operator:	Perry Geotech		
Pre-Drill:	0m	Assumed GWL:	1mBGL	Located By:	Survey GPS		
Position:	2484712.69mE	5745100.12mN	1.78mRL	Coord. System:	NZMG		
Address:	1150 Avonside Dr			Datum Reference:	MSL (CCC 20/01/12 Datum -9.043)		

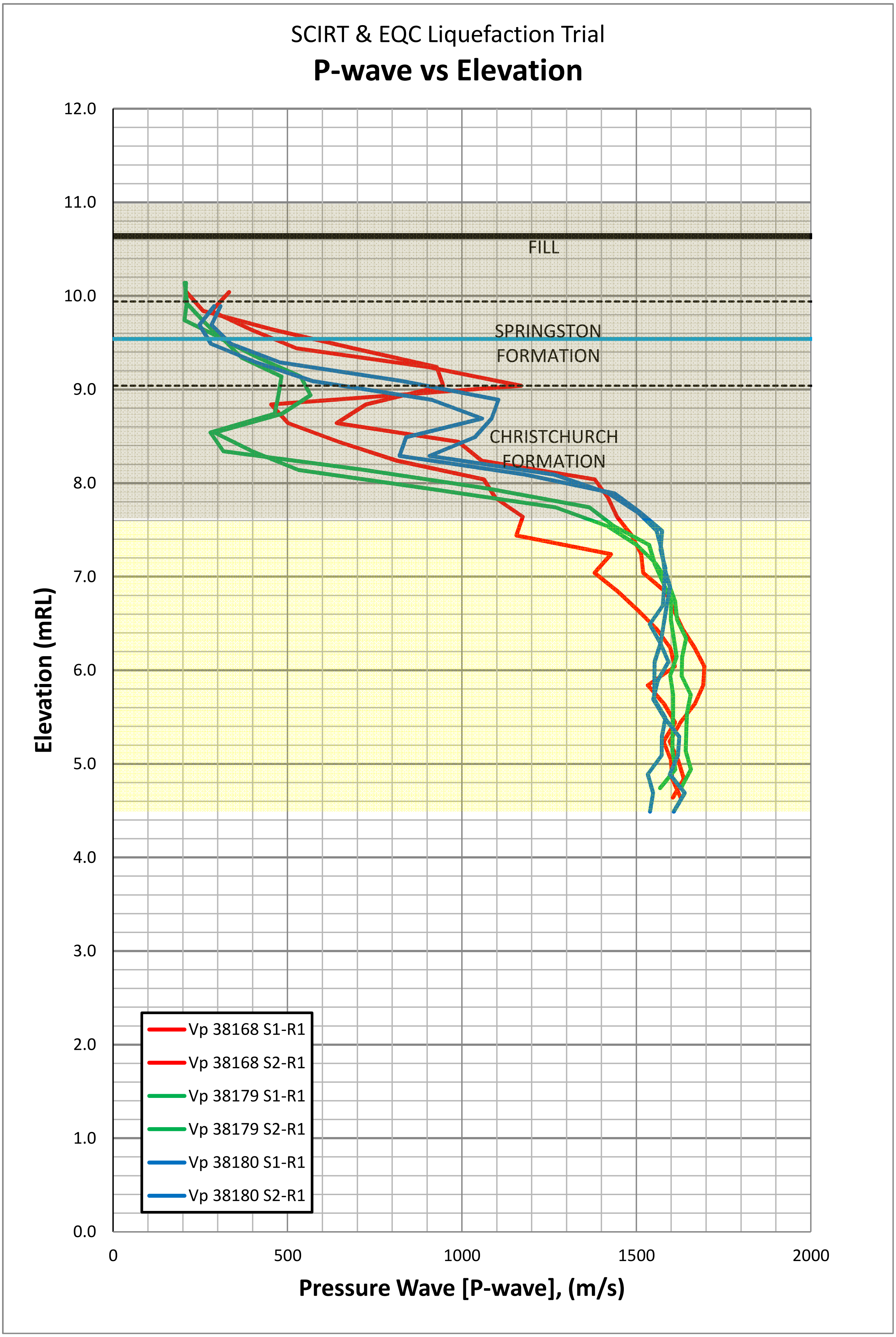


Project:	Christchurch Geotechnical Investigations			**Page:** 1 of 1		**AVD-TCR01-CPT099**
Test Date:	10-Oct-2013	**Suburb:**	Avondale	**Operator:**	Perry Geotech	
Pre-Drill:	0m	**Assumed GWL:**	1mBGL	**Located By:**	Survey GPS	
Position:	2484703.39mE	5745063.71mN	1.38mRL	**Coord. System:**	NZMG	
Address:	31 Ardrossan St			**Datum Reference:**	MSL (CCC 20/01/12 Datum -9.043)	



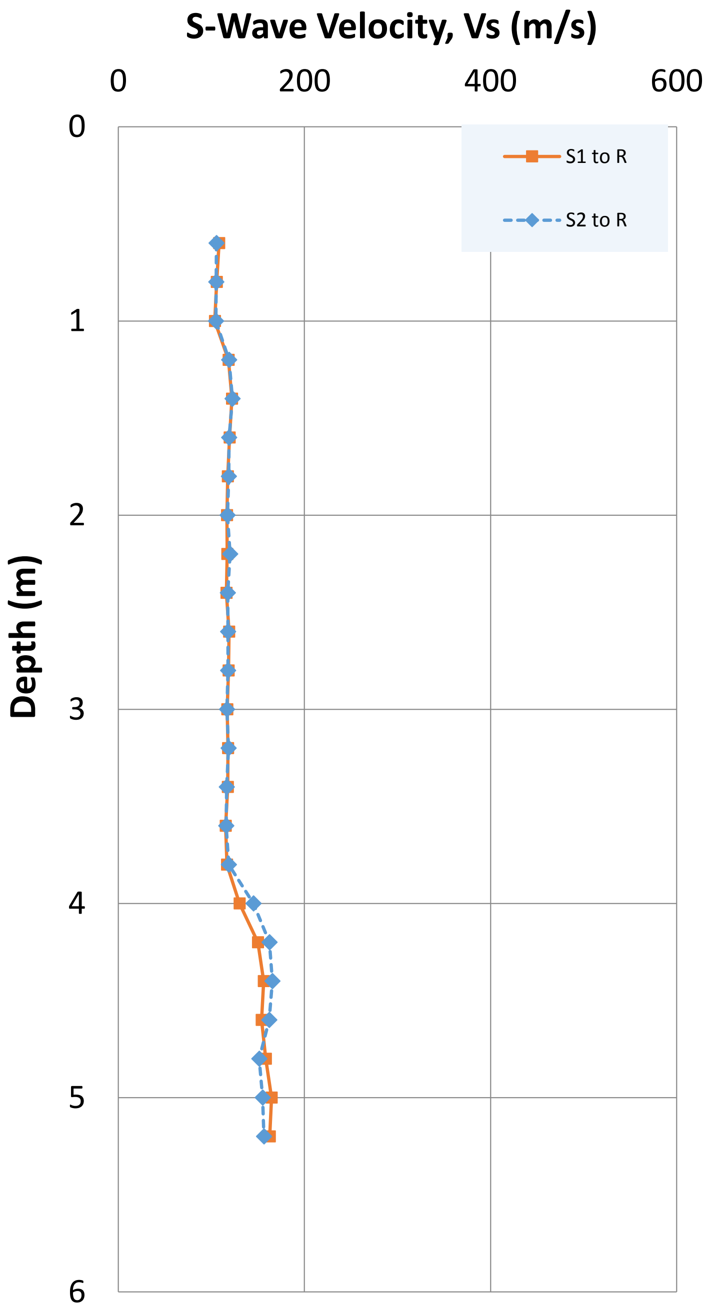


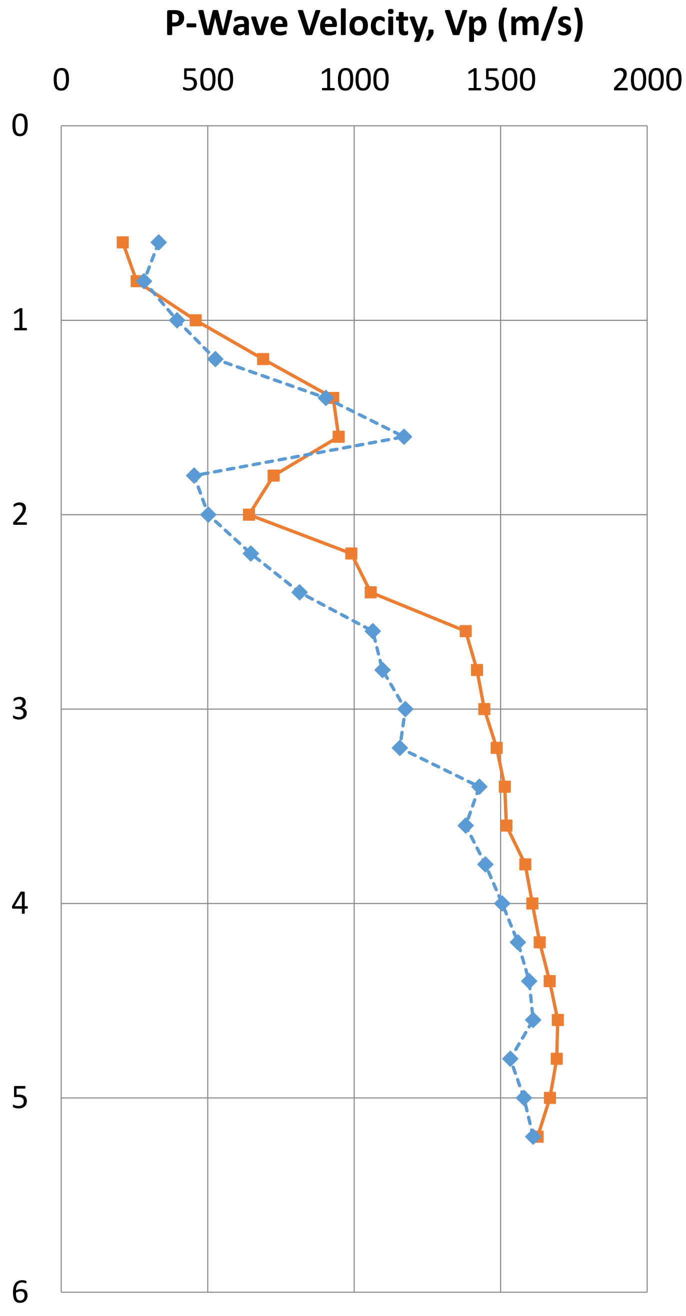
SCIRT & EQC Liquefaction Trial
S-wave vs Elevation
Elevation (mRL)
Shear Wave [S-wave], (m/s)
FILL
SPRINGSTON FORMATION
CHRISTCHURCH FORMATION
Vs 38168 S1-R1
Vs 38168 S2-R1
Vs 38179 S1-R1
Vs 38179 S2-R1
Vs 38180 S1-R1
Vs 38180 S2-R1
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
100
110
120
130
140
150
160
170
180
190
200



SCIRT & EQC Liquefaction Trial
P-wave vs Elevation
12.0
11.0
10.0
9.0
8.0
7.0
6.0
5.0
4.0
3.0
2.0
1.0
0.0
Elevation (mRL)
FILL
SPRINGSTON FORMATION
CHRISTCHURCH FORMATION
Vp 38168 S1-R1
Vp 38168 S2-R1
Vp 38179 S1-R1
Vp 38179 S2-R1
Vp 38180 S1-R1
Vp 38180 S2-R1
0
500
1000
1500
2000
Pressure Wave [P-wave], (m/s)

AVD-TCR01-XH23





Project:

T-Rex Ground Improvement Trials

Test Date: 6 October 2013

Suburb: Avondale

Located By: Survey GPS

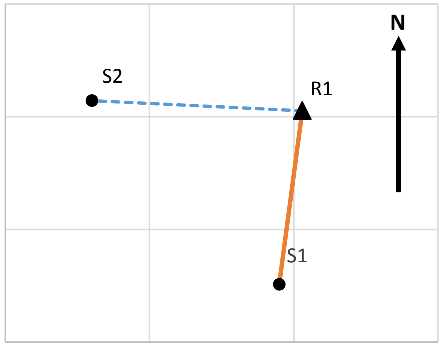
Coord System: NZTM

Vertical Datum: Lyttleton Vertical Datum 1937

Probe Positions:

Probe	Easting (mE)	Northing (mN)	Elevation (mRL)
R1	1574693.1	5183467.1	1.6
S1	1574692.9	5183465.5	1.6
S2	1574691.6	5183467.1	1.5

Indicative Location Plan



Comments:

29 Ardrossan Street: S1 to R; Unimproved ground. S2 to R; Unimproved ground.

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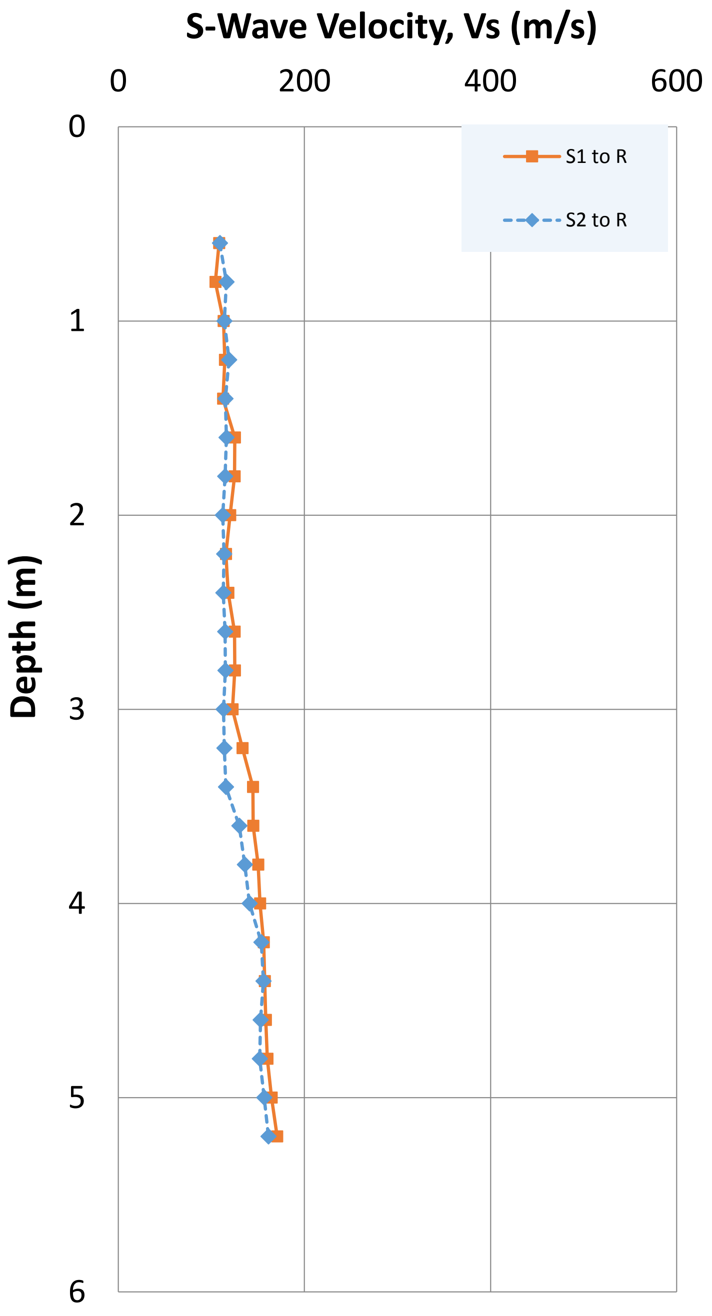
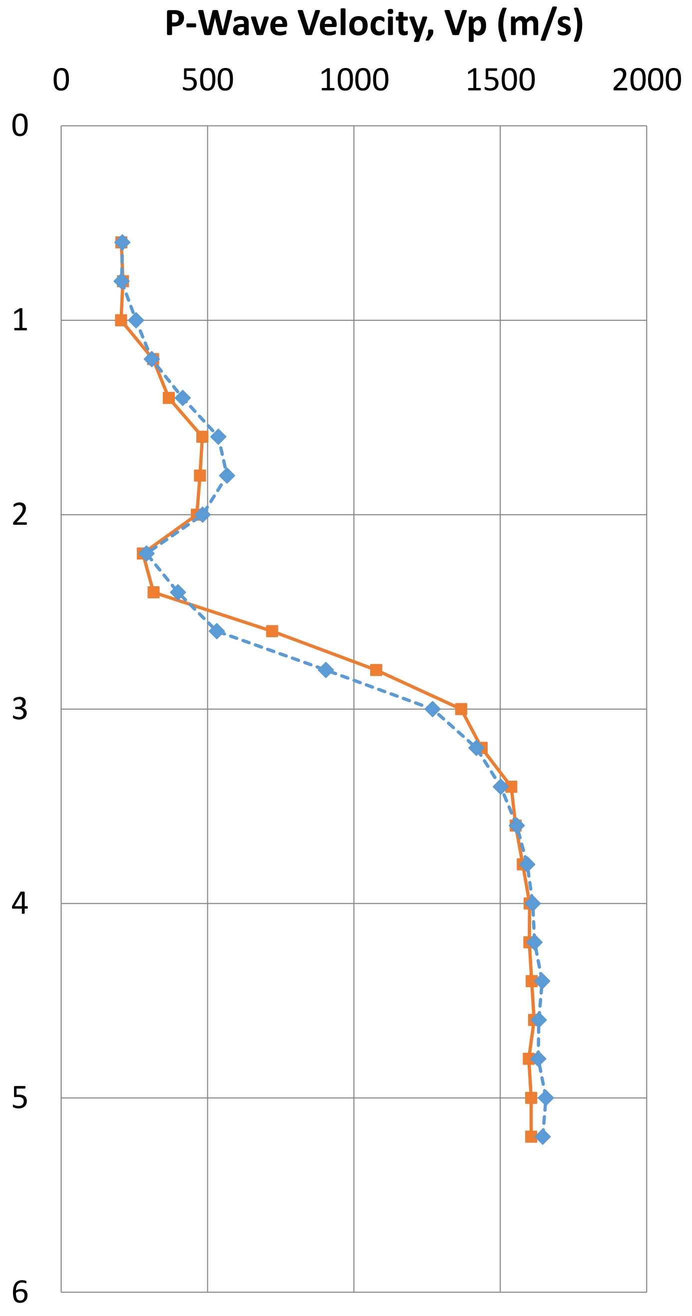




AVD-TCR01-XH25

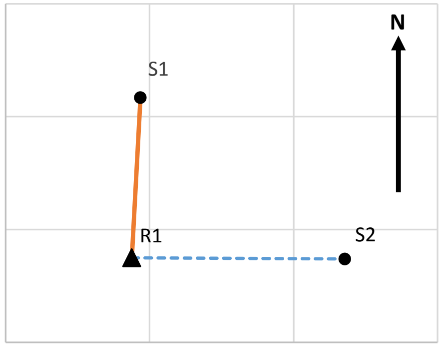
Project:

T-Rex Ground Improvement Trials

Test Date:	6 October 2013
Suburb:	Avondale
Located By:	Survey GPS
Coord System:	NZTM
Vertical Datum:	Lyttleton Vertical Datum 1937
Probe Positions:	

Probe	**Easting (mE)**	**Northing (mN)**	**Elevation (mRL)**
R1	1574700.9	5183476.7	1.7
S1	1574700.9	5183478.2	1.7
S2	1574702.4	5183476.7	1.7

Indicative Location Plan



Comments:

SCIRT Northern: S1 to R; Unimproved ground. S2 to R; Unimproved ground.

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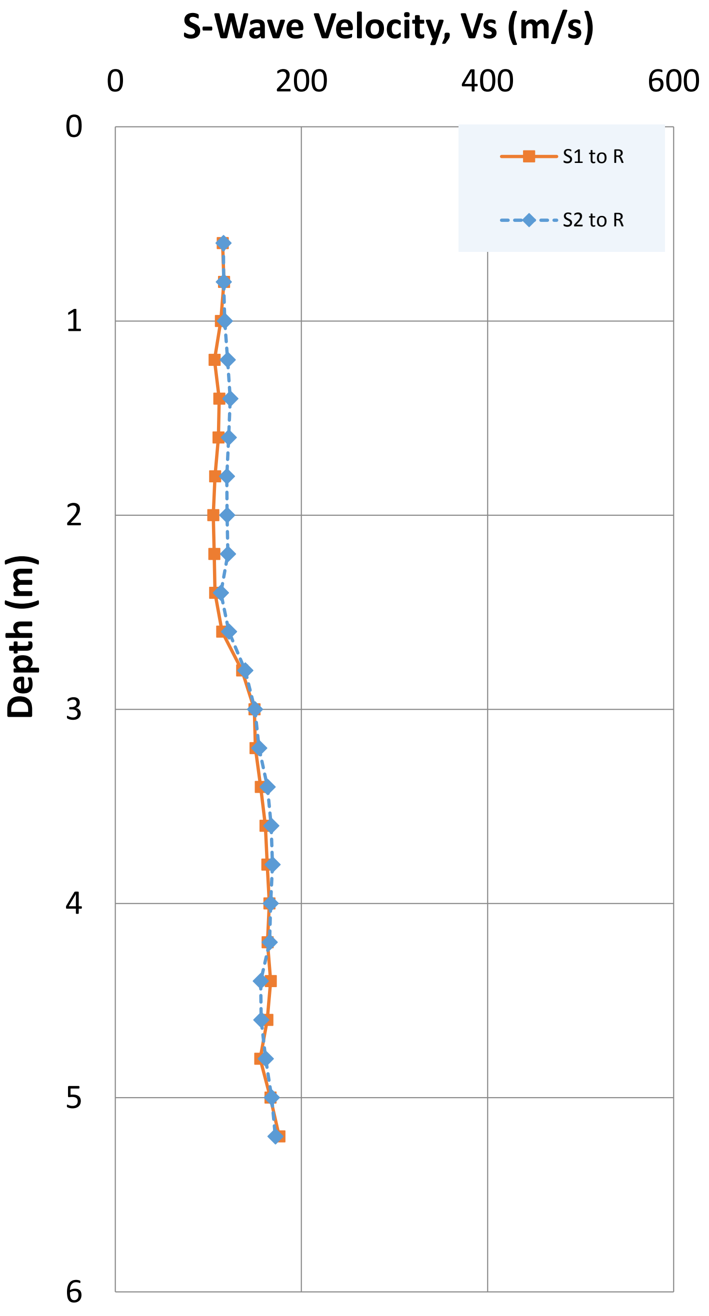
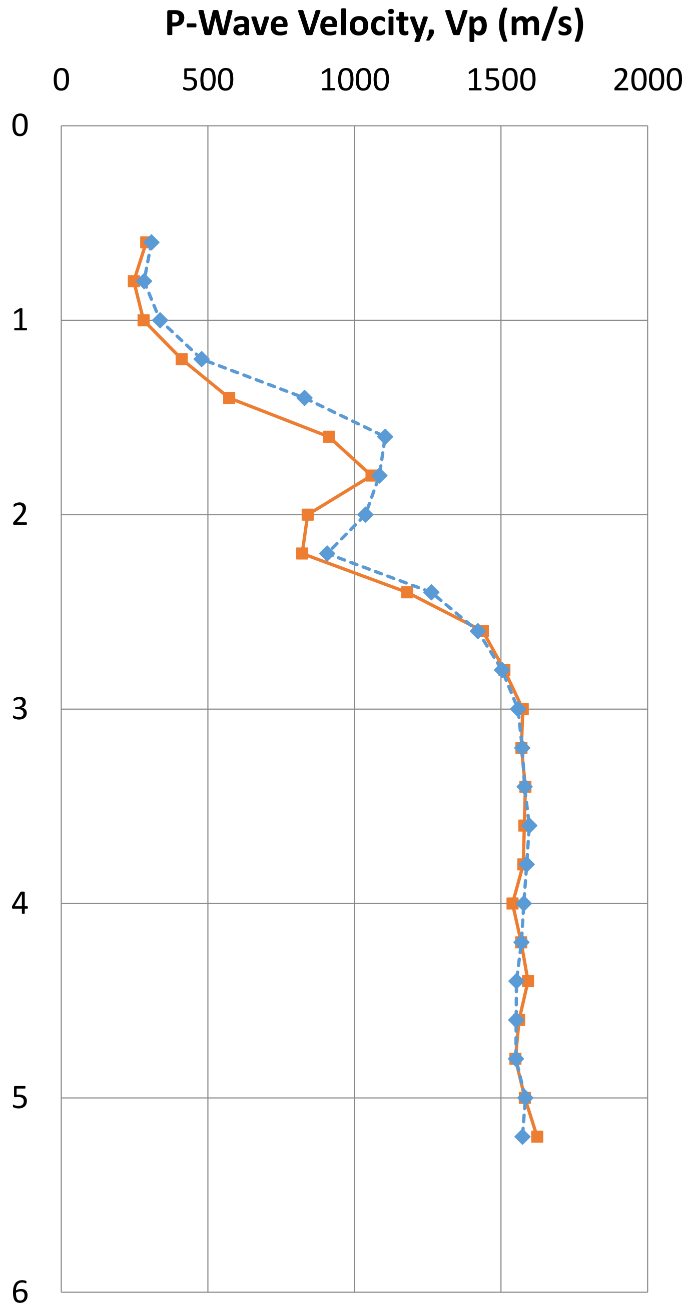




AVD-TCR01-XH26

Project:

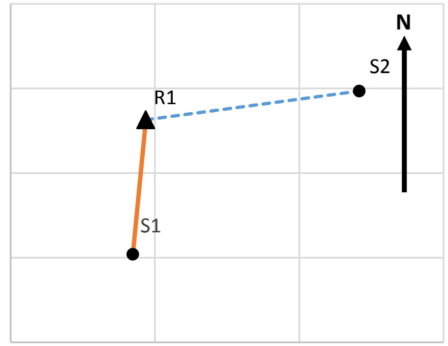
T-Rex Ground Improvement Trials

Test Date:	6 October 2013
Suburb:	Avondale
Located By:	Survey GPS
Coord System:	NZTM
Vertical Datum:	Lyttleton Vertical Datum 1937

Probe Positions:

Probe	Easting (mE)	Northing (mN)	Elevation (mRL)
R1	1574700.9	5183449.6	1.4
S1	1574700.8	5183448.0	1.5
S2	1574702.4	5183450.0	1.4

Indicative Location Plan



Comments:

SCIRT Southern: S1 to R; Unimproved ground. S2 to R; Unimproved ground.

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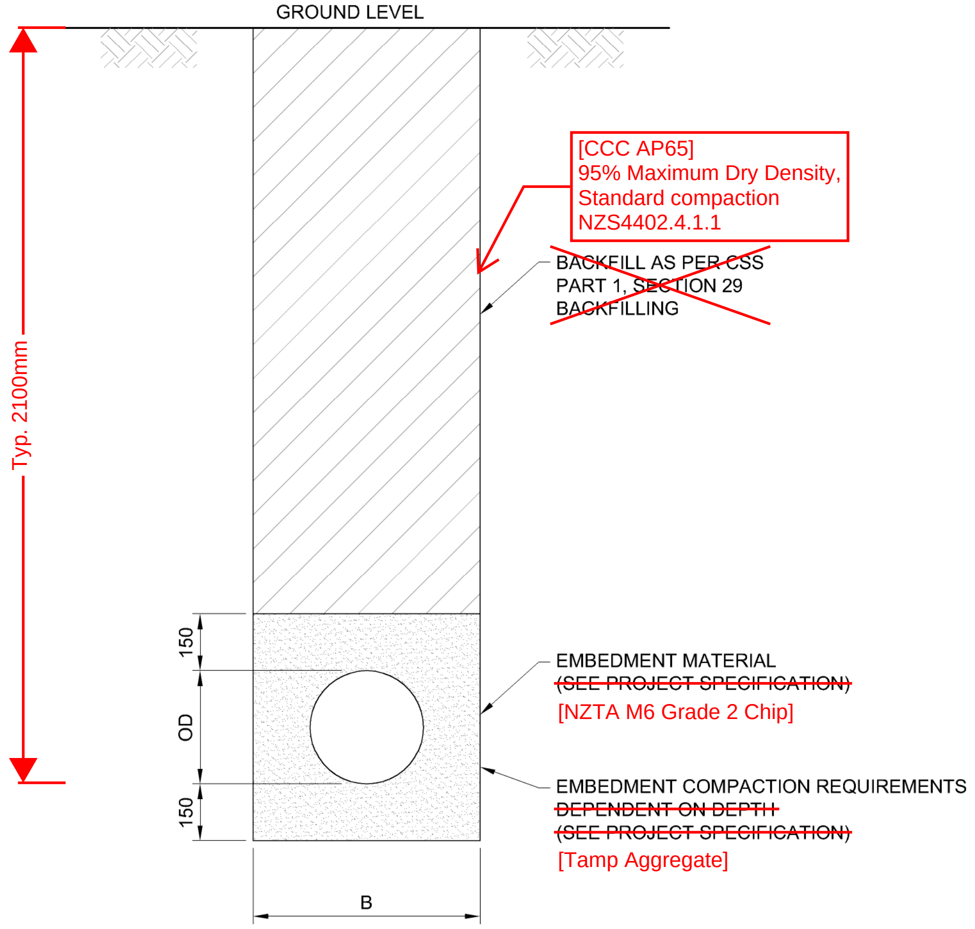
C

Appendix C As Built Details

As-Built

TEST 1 - NZTA M6 Grade 2 Chip Pipe Haunching

10001-DE-WW-DG-6315.dwg



PIPE OUTSIDE DIAMETER OD (mm)	MINIUMUM TRENCH WIDTH B (mm)
≤ 300	OD + 350
> 300, ≤ 450	OD + 400
> 450, ≤ 900	OD + 600

FOR PVC-U PIPES

NOMINAL PIPE DIAMETER DN	MINIMUM TRENCH WIDTH B
100	460
150	500
175	550
200	575
225	600
250	630
300	700
375	800
475	1100

NOTES:

1. SEE PROJECT SPECIFICATION FOR EMBEDMENT MATERIAL DETAILS.

2. ~~WHERE THERE IS AN IDENTIFIED RISK OF SOIL PARTICLE MIGRATION, EITHER WRAP THE WHOLE EMBEDMENT IN GEOTEXTILE OR CONSTRUCT TRENCH STOPS. SEE PROJECT SPECIFICATION FOR DETAILS.~~

3. FOR NON-PRESSURE PVC UP TO 3.5m DEEP, PIPE CLASS TO BE SN16 FOR DIAMETERS DN100 - DN475. OTHER CASES REQUIRE SPECIFIC DESIGN.

SCIRT Rebuilding Infrastructure

PIPE EMBEDMENT FOR FLEXIBLE PIPES

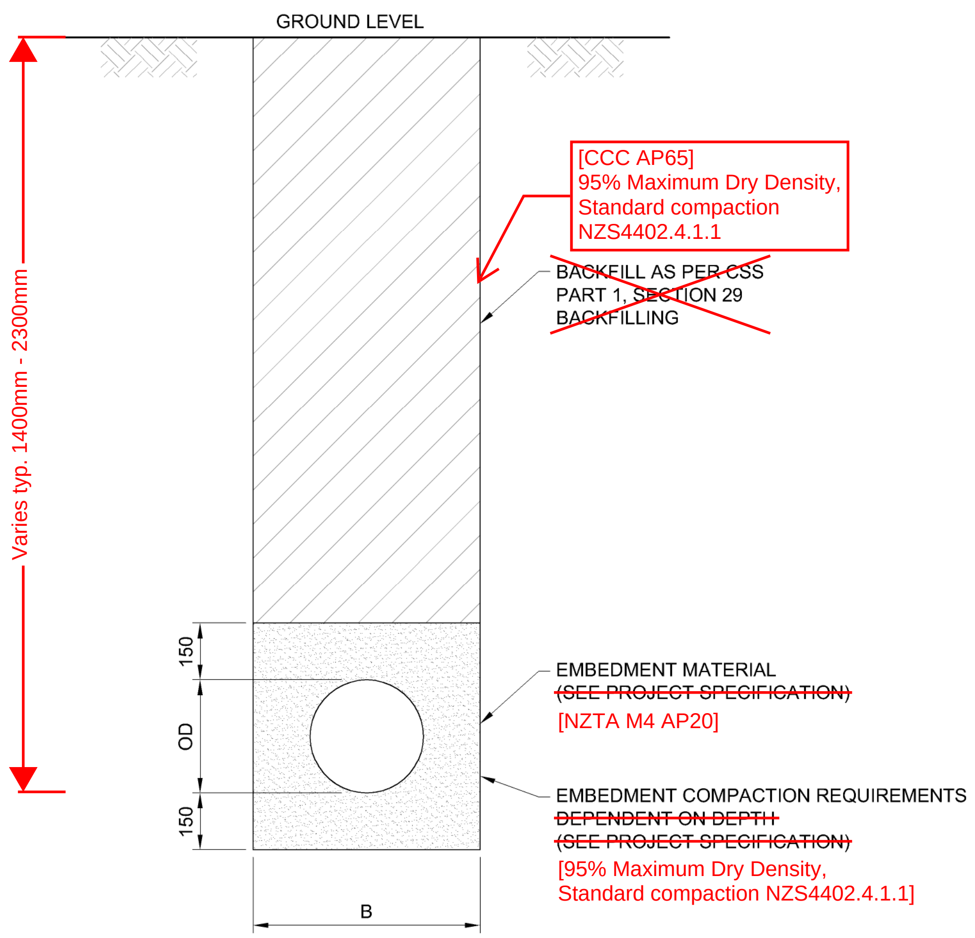
ISSUE DATE	FEB 2014

SC6315

SHEET 1 OF 1

As-Built

TEST 2 - NZTA M4 AP20 Pipe Haunching



PIPE OUTSIDE DIAMETER OD (mm)	MINIUMUM TRENCH WIDTH B (mm)
≤ 300	OD + 350
> 300, ≤ 450	OD + 400
> 450, ≤ 900	OD + 600

FOR PVC-U PIPES

NOMINAL PIPE DIAMETER DN	MINIMUM TRENCH WIDTH B
100	460
150	500
175	550
200	575
225	600
250	630
300	700
375	800
475	1100

NOTES:

1. SEE PROJECT SPECIFICATION FOR EMBEDMENT MATERIAL DETAILS.
2. ~~WHERE THERE IS AN IDENTIFIED RISK OF SOIL PARTICLE MIGRATION, EITHER WRAP THE WHOLE EMBEDMENT IN GEOTEXTILE OR CONSTRUCT TRENCH STOPS. SEE PROJECT SPECIFICATION FOR DETAILS.~~
3. FOR NON-PRESSURE PVC UP TO 3.5m DEEP, PIPE CLASS TO BE SN16 FOR DIAMETERS DN100 - DN475. OTHER CASES REQUIRE SPECIFIC DESIGN.

SCIRT
Rebuilding Infrastructure

PIPE EMBEDMENT FOR FLEXIBLE PIPES

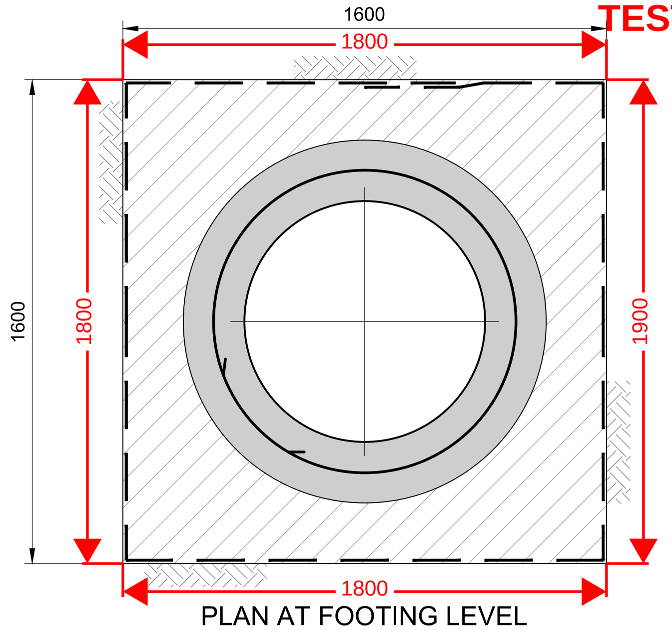
ISSUE DATE	FEB 2014

SC6315

SHEET 1 OF 1

As-Built

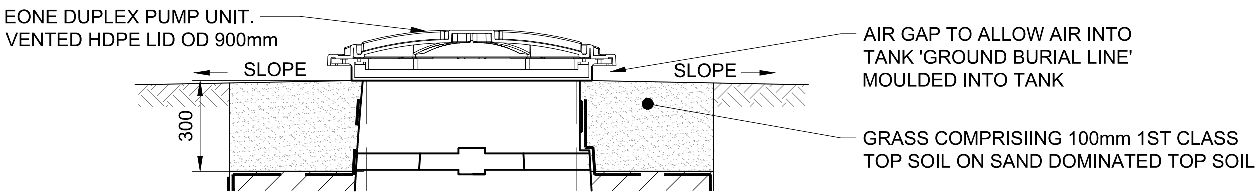
TEST 3 - High Permeable Backfill



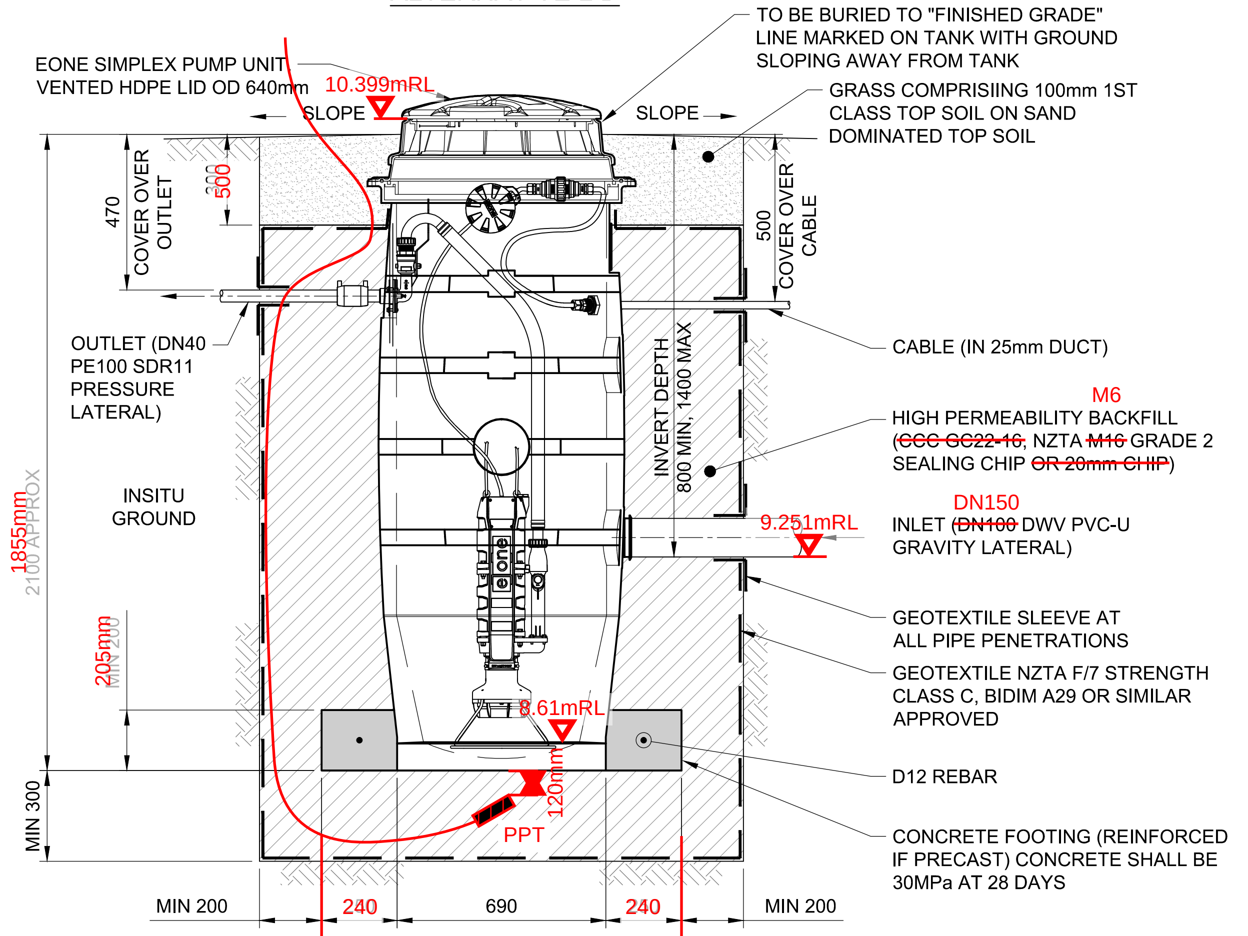
PLAN AT FOOTING LEVEL

NOTES

1. MAKE ALL CONNECTIONS THROUGH THE TANK WALL IN ACCORDANCE WITH EONE INSTALLATION INSTRUCTIONS AND USING EONE SUPPLIED CONNECTION FITTINGS.
2. THE ALTERNATIVE LID CHANGES EXCAVATION AND COVER DEPTHS. CONTACT EONE FOR DETAILS.
3. FOR GROUND WATER DEPTHS >2000mm A CONCRETE FOOTING WITH AP20 BACKFILL MAY BE USED INSTEAD OF TYPE 1 OR TYPE 2 BACKFILL.



ALTERNATIVE LID



SECTION

RL to CCC Drainage Datum

SCIRT Rebuilding Infrastructure

PRESSURE SEWER SIMPLEX TANK TYPE 1 BACKFILL (NON-TRAFFICABLE)

ISSUE DATE	APR 2014

SC6325

SHEET 1 OF 2

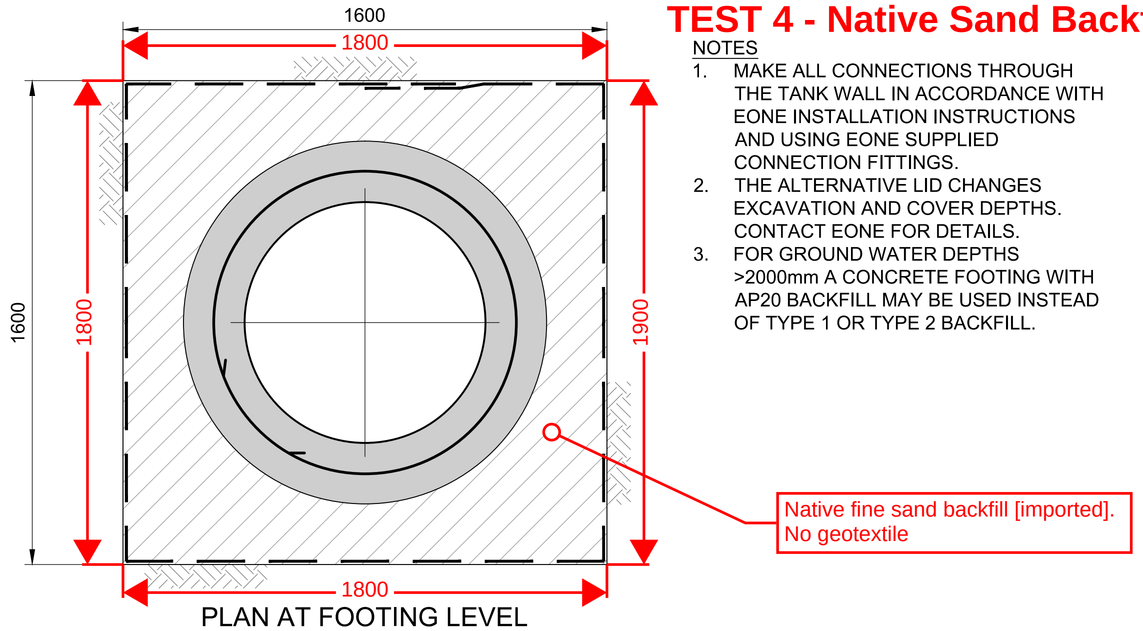
As-Built

TEST 4 - Native Sand Backfill

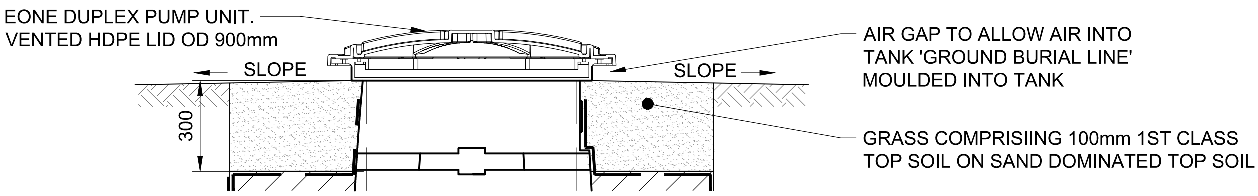
NOTES

1. MAKE ALL CONNECTIONS THROUGH THE TANK WALL IN ACCORDANCE WITH EONE INSTALLATION INSTRUCTIONS AND USING EONE SUPPLIED CONNECTION FITTINGS.
2. THE ALTERNATIVE LID CHANGES EXCAVATION AND COVER DEPTHS. CONTACT EONE FOR DETAILS.
3. FOR GROUND WATER DEPTHS >2000mm A CONCRETE FOOTING WITH AP20 BACKFILL MAY BE USED INSTEAD OF TYPE 1 OR TYPE 2 BACKFILL.

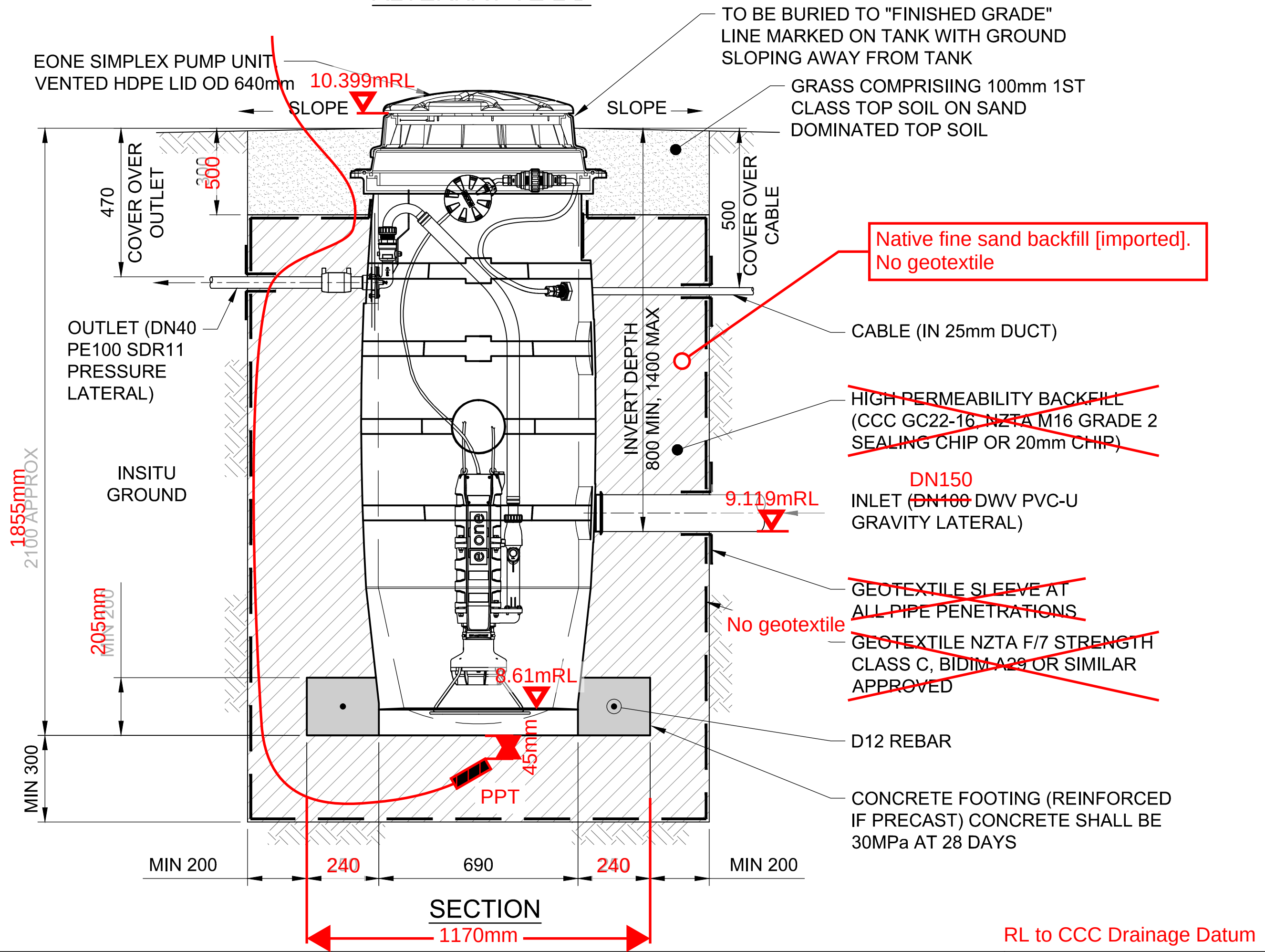
10001-DE-WW-DG-6325-1.dwg



PLAN AT FOOTING LEVEL



ALTERNATIVE LID



SECTION

RL to CCC Drainage Datum

SCIRT Rebuilding Infrastructure

PRESSURE SEWER SIMPLEX TANK TYPE 1 BACKFILL (NON-TRAFFICABLE)

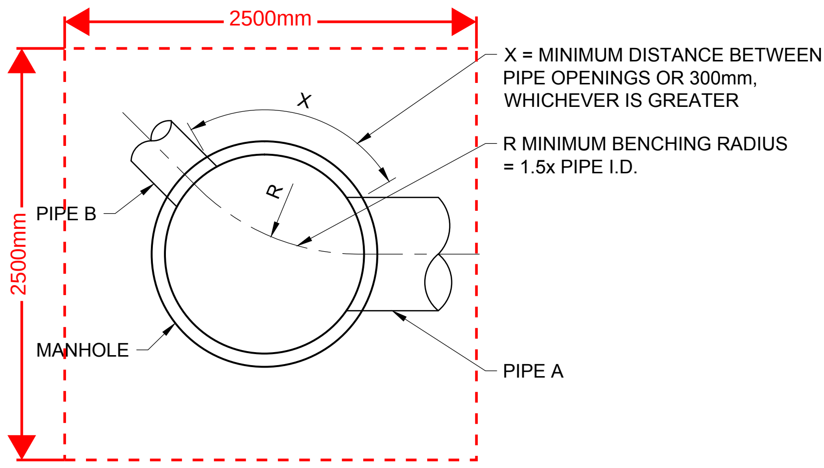
ISSUE DATE	APR 2014
SC6325	
SHEET 1 OF 2	

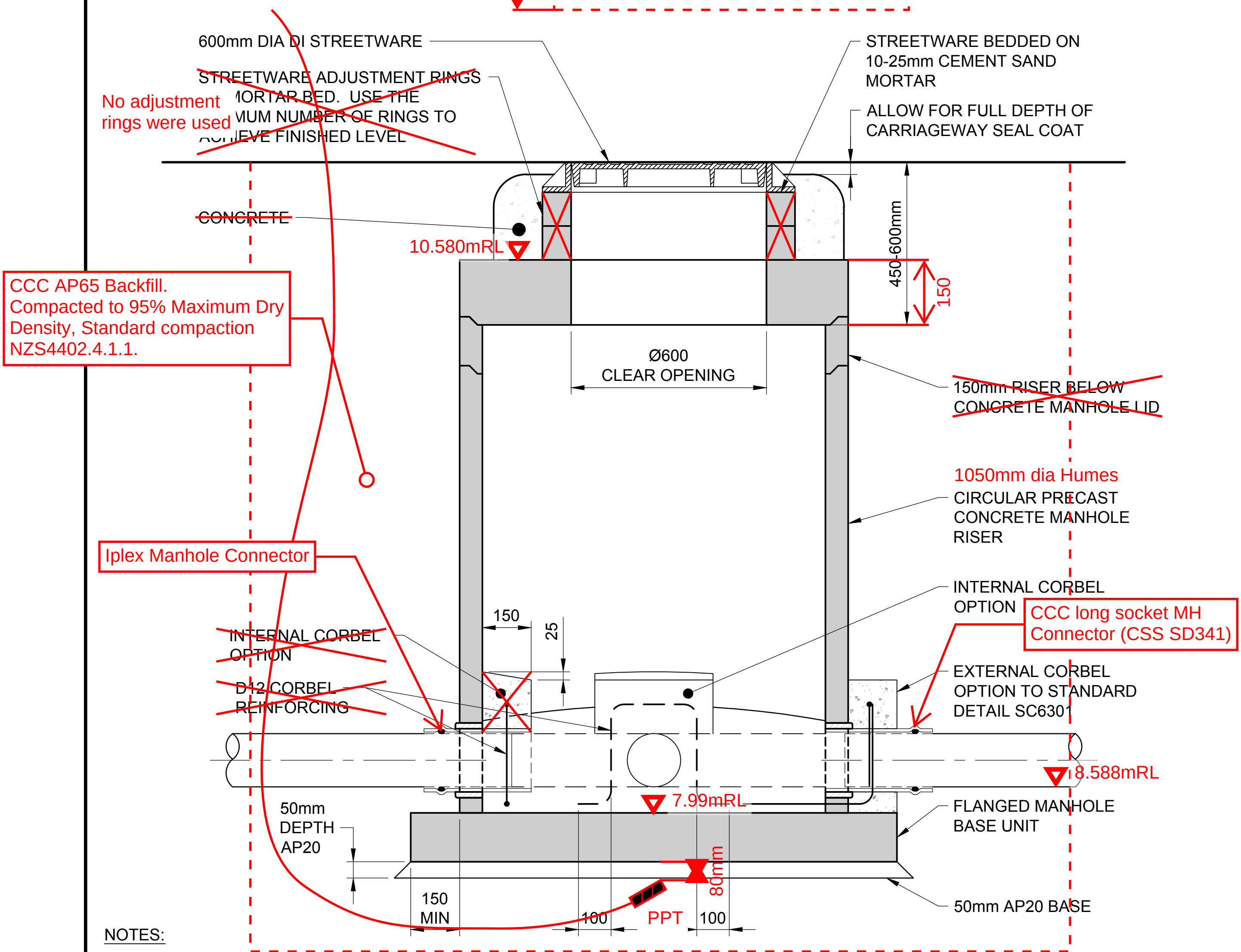
As-Built **TEST 5 - Concrete Chamber**

10001-DE-WW-DG-6309.dwg

NOTES FOR MANHOLE SELECTION:

1. MANHOLE SELECTION
   MINIMUM BENCHING RADIUS = 1.5x PIPE I.D.
2. MINIMUM DISTANCE BETWEEN PIPES
   - EQUAL SIZED PIPES
     - X MIN DISTANCE = 0.75x PIPE OD (FOR EQUAL SIZE PIPES) OR 300mm, WHICHEVER IS GREATER.
   - UNEQUAL SIZED PIPES
     - X MIN DISTANCE = 0.65x LARGE OD OR 300mm, WHICHEVER IS GREATER.





NOTES:

1. ALL DIMENSIONS IN MILLIMETRES.
2. INTERNAL OR EXTERNAL CORBELS MAY BE USED SUBJECT TO THE MANHOLE SELECTION CRITERIA.
3. ALL CORBEL CONCRETE SHALL BE 30MPa.
4. REINFORCING SHALL BE GRADE 300.
5. REINFORCING STEEL TO HAVE 60mm COVER MIN.
6. MINIMUM REINFORCING STEEL BEND RADIUS 5x DIAMETER.
7. DO NOT SCALE DRAWING.
8. PROPRIETRY MANHOLE COMPONENTS SHALL COMPLY WITH THE CCC APPROVED MATERIALS LIST.

RL to CCC Drainage Datum

SCIRT Rebuilding Infrastructure

TYPICAL PRECAST CIRCULAR MANHOLE ASSEMBLY FOR PIPES ≤ DN450

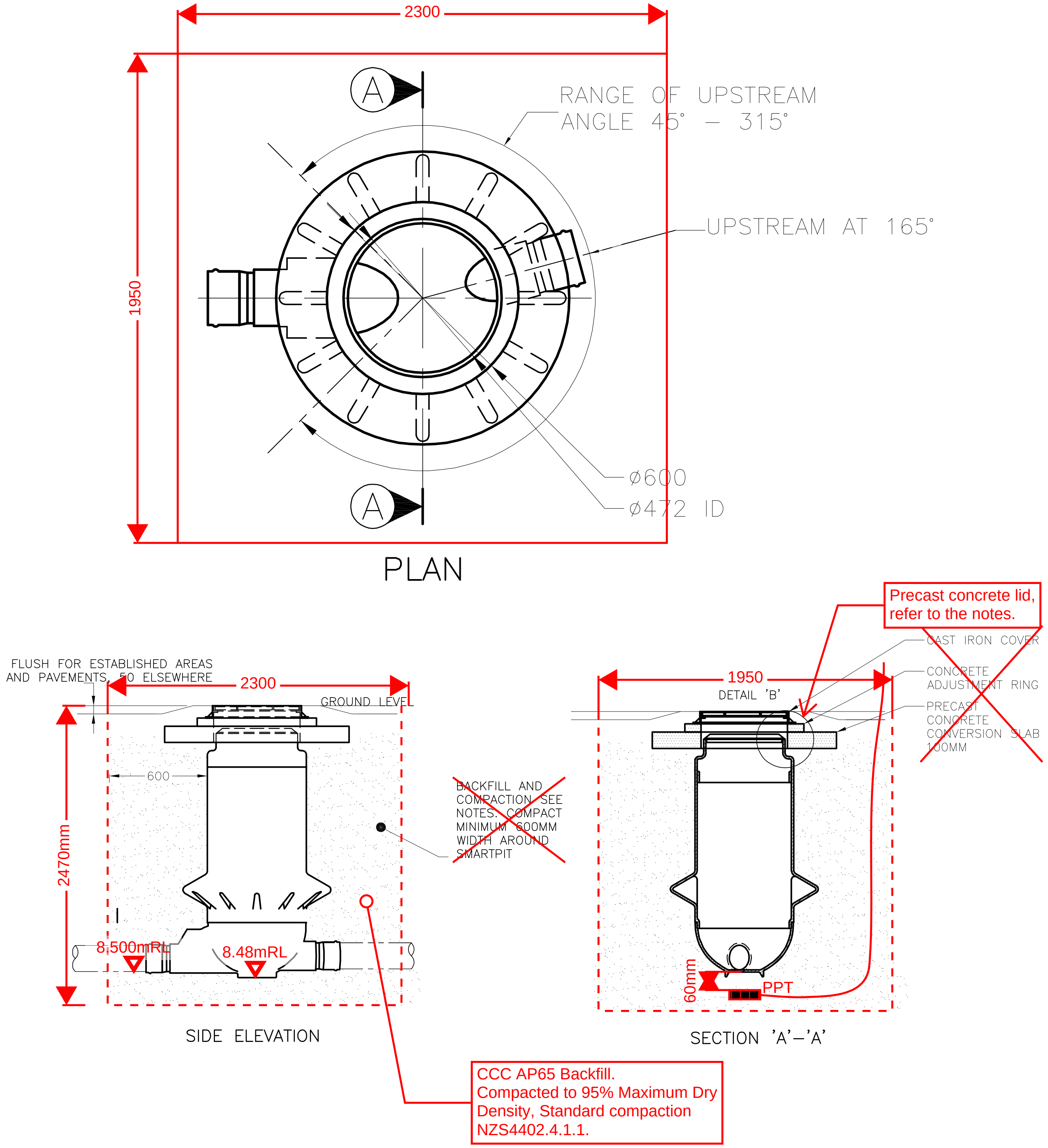
ISSUE DATE	NOV 2013

SC6309

SHEET 1 OF 1

As-Built

TEST 6 - Smartstream DN600 Manhole



NOTES:
- Lid: 200mm thick 1.35m OD concrete lid with 600mm opening. [HN-HO-72 designed lid used for 1.05m manhole]
- The top of the riser part at the chamber should be installed to penetrate halfway into the 600mm opening (100mm).

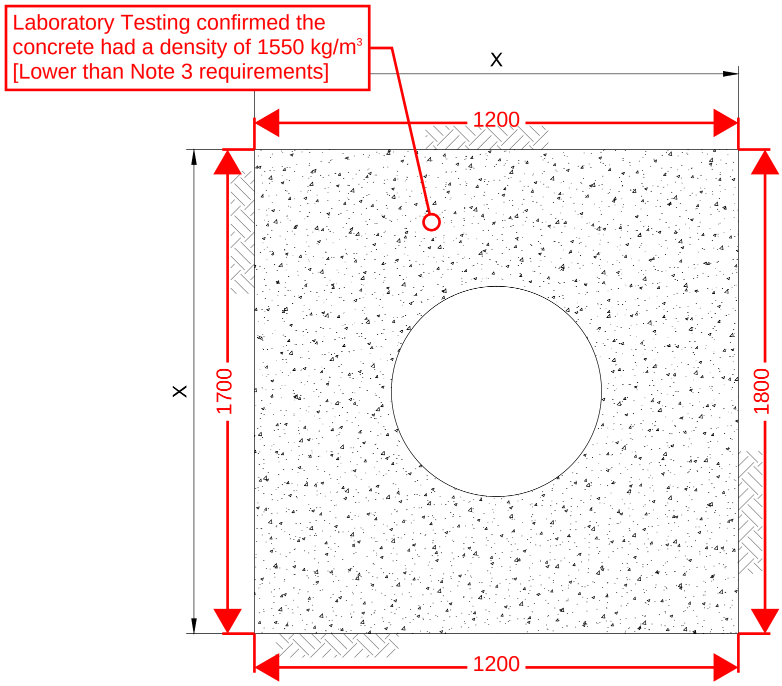
RL to CCC Drainage Datum

As-Built

TEST 7 - Concrete Backfill

TYPE 2 CONCRETE BACKFILL DIMENSIONS	
GROUNDWATER DEPTH - Y	CONCRETE BACKFILL - X
0	1700
500	1250
>1000	1000

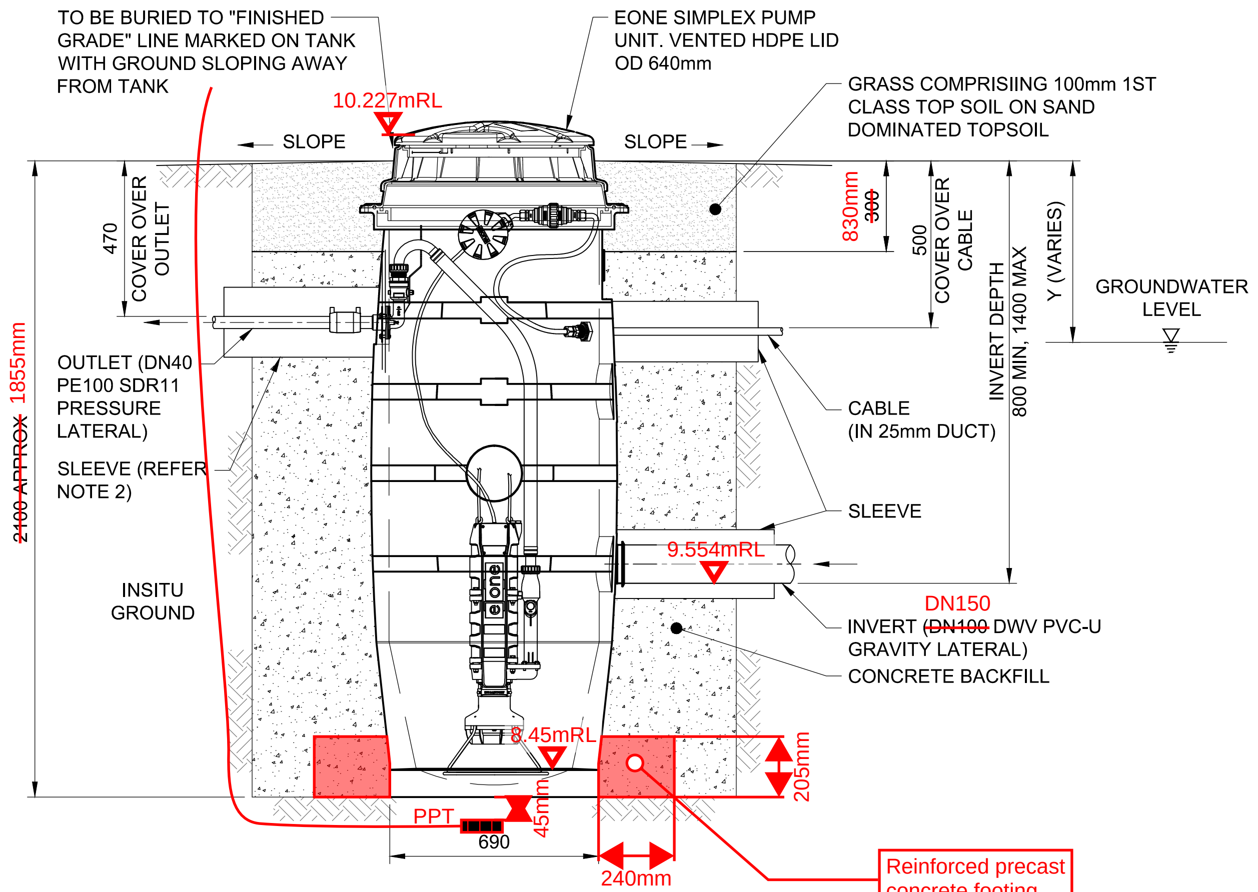
10001-DE-WW-DG-6325-2.dwg



PLAN AT FOOTING LEVEL

NOTES

1. MAKE ALL CONNECTIONS THROUGH THE TANK WALL IN ACCORDANCE WITH EONE INSTALLATION INSTRUCTIONS AND USING EONE SUPPLIED CONNECTION FITTINGS.
2. FOR TYPE 2 BACKFILL THE INLET PIPE, OUTLET PIPE, AND CABLE MUST BE SLEEVED WITH A 200mm TUBE. INSTALL THE CONNECTIONS (INLET PIPE, OUTLET PIPE, AND CABLE) PRIOR TO POURING CONCRETE ABOVE THE INVERT OF THE SLEEVES.
3. CONCRETE SHALL BE LOW STRENGTH CONCRETE (1MPa TO 3MPa AT 28 DAYS) ~~WITH MINIMUM DENSITY 2250kg/m³~~
4. GROUNDWATER DEPTH TO BE DETERMINED BY THE DESIGNER. REFER TO THE PROJECT SPECIFICATION.
5. FOR GROUNDWATER DEPTHS >2000mm A CONCRETE FOOTING (REFER SHEET1) WITH AP20 BACKFILL MAY BE USED INSTEAD OF TYPE 1 OR TYPE 2 BACKFILL.



SECTION

RL to CCC Drainage Datum

SCIRT Rebuilding Infrastructure	PRESSURE SEWER SIMPLEX TANK TYPE 2 BACKFILL (NON-TRAFFICABLE)	ISSUE DATE	APR 2014
		SC6325	
		SHEET 2 OF 2	

RESTRAIN™

PVC PIPE FOR TRENCHLESS INSTALLATION

simply secure

Iplex Pipelines (NZ) Ltd has developed Iplex **RESTRAIN**™ specifically for gravity pipeline projects installed using trenchless technology. With an easy to assemble threaded socket and spigot joint together with a rubber seal ring, **RESTRAIN**™ brings the advantages of PVC pipe systems to the trenchless industry.

RESTRAIN™ is suitable for use in a range of applications...

- Gravity sewer
- Gravity stormwater
- Electrical/telecommunications ducting
- Ducting

and installation methodologies*...

- Horizontal directional drilling
- Auger boring
- Guided boring
- Pipe bursting/cracking

RESTRAIN™

RESTRAIN™

PVC PIPE FOR TRENCHLESS INSTALLATION

The **RESTRAIN™** system offers significant on site benefits.

Feature	Benefit
Threaded socket and spigot joint.	Easy to assemble, no special tools or training required. No solvent cement or fusion welding required. Joints can be undone and pipe re-used if necessary. No long pipe strings enabling small footprint workspace.
Pipe lengths customized to suit installation / project requirements.	Enables use with a range of installation technologies or site requirements. Pipe available in lengths from 1m to 6m.
Compatible with industry standard PVC sewer and stormwater fittings	Simple connection to manholes and service laterals.
Two directional installation capability	Pipe can be "pushed" or "pulled" into place depending on installation methodology
High performance rubber seal ring	No risk of leaks or root infiltration.
Complies with relevant industry Standards - AS/NZS 1260 , AS 2053	Product meets proven performance levels.
Manufactured from uPVC pipe	Stiffer than PE pipe of equivalent dimensions making it more suitable for installation on flat grades.

Product Range

Sewer/Stormwater

Pipe Size	Pipe OD/mm	Pipe Specification	Maximum Tensile Load During Installation	Maximum Compressive Load During Installation
DN100	110	AS/NZS 1260 SN16	1,800 kg	1,800 kg
DN150	160	AS/NZS 1260 SN16	3,500 kg	3,000 kg
DN225	250	AS/NZS 1260 SN16	9,500 kg	9,500 kg
DN300	315	AS/NZS 1260 SN16	12,000 kg	12,000 kg

Pipe is manufactured in accordance with AS/NZS 1260: 1999. *PVC pipes and fittings for drain, waste and vent applications.*
Elastomeric seal ring complies with Section 3.4 of AS./NZS 1260

- Cl. 3.4.2 - Hydrostatic pressure test in accordance with AS/NZS 1462.10
- Cl. 3.4.3 – Liquid infiltration test in accordance with AS/NZS 1462.8
- Cl. 3.4.4 – Contact width and interface pressure test in accordance with AS/NZS 1462.13

* Disclaimer: RESTRAIN™ is not suitable for use in pipe ramming or impact moling. However, as terminology and definitions of trenchless methods do vary, please contact Iplex Pipelines to confirm that RESTRAIN™ is compatible with your installation technology.

Cable Duct

Pipe Size	Pipe OD/mm	Pipe Specification	Maximum Tensile Load During Installation	Maximum Compressive Load During Installation
DN100	110	AS 2053 Light Duty	1,800 kg	1,800 kg
DN150	160	AS 2053 Light Duty	3,500 kg	3,000 kg
DN225	250	AS 2053 Light Duty	9,500 kg	9,500 kg

Pipe is manufactured in accordance with AS 2053; 1984. *Non-metallic conduits and fittings.*

iPLEX Pipelines

RESTRAIN™

simply secure

Iplex Pipelines NZ Limited.

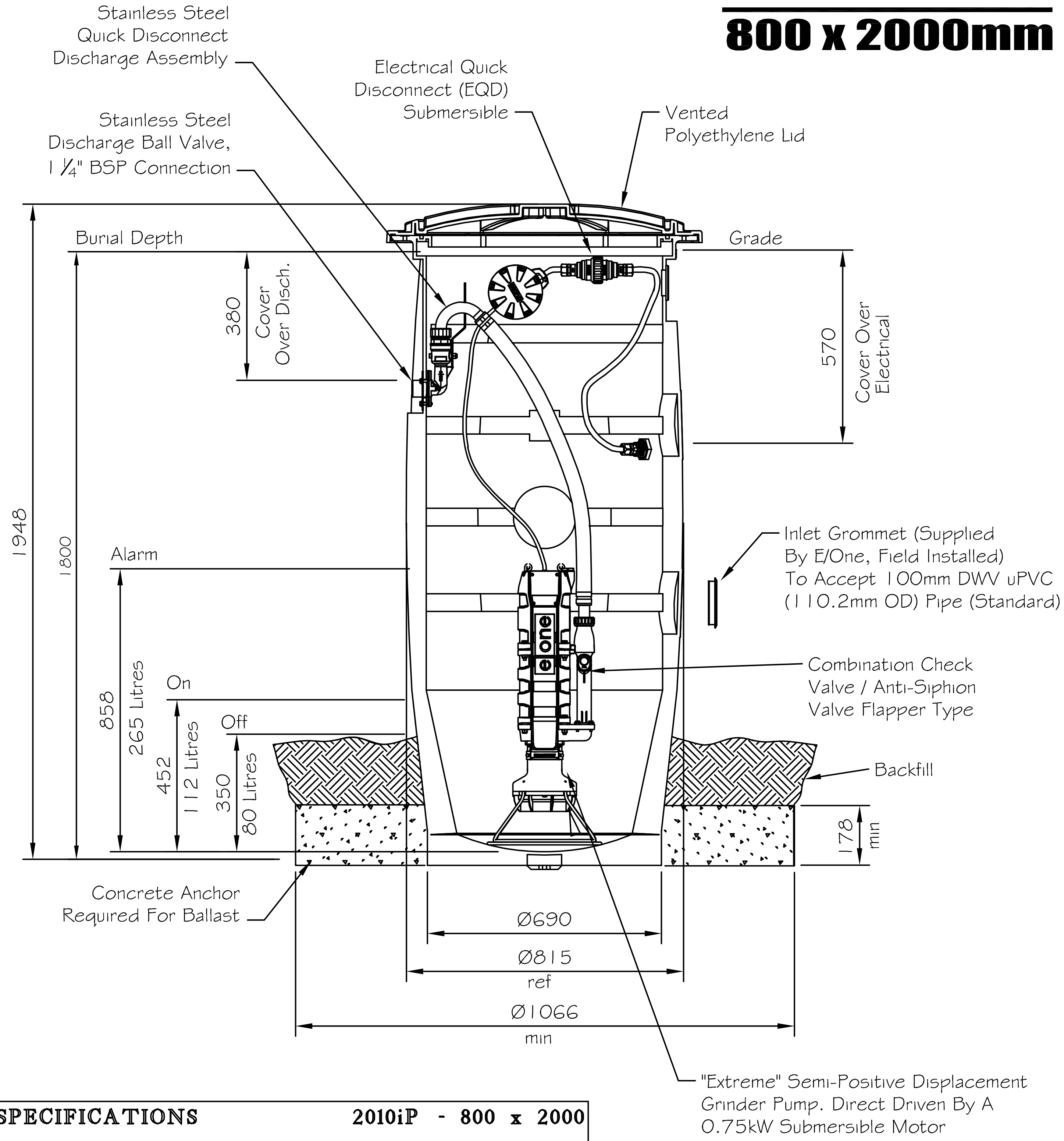
Call Centre - Phone: 0800 800 262
Fax: 0800 800 804
Web: www.iplex.co.nz

Offices At:

Auckland:
PO Box 13772, Onehunga, 2 Rockridge Avenue, Penrose.
Palmerston North:
Private Bag 11019, 67 Malden Street.
Christchurch:
PO Box 16225, 22 Braeburn Drive, Sockburn.

11.9.09

MODEL 2010iP
800 x 2000mm



SPECIFICATIONS 2010iP - 800 x 2000

BASIN	Diameter (mm)	Height (mm)	Weight (kg)	Total Capacity (l)	Capacity Above Alarm (l)	Ballast (m3)	Inlet Pipe Size (mm)
	800	2000	74	650	379	SEE DWG	100

PUMP	Pump (s)	Dimensions (mm)	Weight (kg)	Nominal Pump Rate @ 0m TDH (lps)	Nominal Pump Rate @ 20m TDH (lps)	Nominal Pump Rate @ 42m TDH (lps)	Discharge Size (mm)
	Simplex (1)	330 X 800	47	.75	.63	.47	32

ELECTRICAL	Voltage (VAC)	Phase	Cycle (Hz)	Supply Cable (m)	Alarm Panel
	240	Single	50	As Specified	Simplex (1)

ecoflow

Ecoflow Ltd.
16b Piermark Drive, North Harbour (NZ)
P.O Box 300-249, Albany, Auckland
Ph (09)447-1793 Fax (09) 447-3901

Environment One
Simplex 800x2000iP

eone SEWER SYSTEMS

Concrete Manhole Systems
strength and durability

Access for Wastewater or Stormwater Systems

Humes manholes are extensively used throughout New Zealand. Made from strong, dense concrete Humes manholes are capable of withstanding infiltration and attack from corrosive environments. Components such as lids can be designed to HN-HO-72 loadings when required. Design expertise from experienced engineers is available for assistance on standard and non-standard products.

Features

- Strength and durability
- Manufactured to New Zealand standards
- Reinforced
- Modular precast system

Benefits

- High resistance to infiltration and leaking
- Able to meet all design requirements
- Reduced construction time with fewer traffic hold-ups, when compared to cast insitu

Applications

- Stormwater Manholes
- Sewer Manholes
- Pipeline junctions
- Pipeline direction changes

HUMES

Concrete Manhole Systems
strength and durability

A Humes manhole is comprised of many components. These pages are guide to Humes standard sizes and weights.

Riser sections: Mass Data (kg) — Concrete density 2500

Nominal Diameter	Nominal Riser Length (mm)							
mm	300	600	900	1200	1500	1800	2100	2400**
300	23	45	68	91				185
450	44	89	133	177				360
600	68	136	204	272	340	409		553
750	98	195	293	391	489	586		795
900	131	261	392	523	654	784	915	1063
1050	170	341	511	681	851	1022	1192	1385
1200	213	425	638	851	1064	1276	1489	1730
1350	258	516	775	1033	1291	1549	1807	2100
1500	286	572	858	1143	1429	1715	2001	2325
1800	403	805	1208	1611	2013	2416	2819	3275
2050*	513	1027	1540	2053	2567	3080	3593	4175
2300	732	1463	2195	2926	3658	4389	5121	5950
2550	1057	2115	3172	4230	5287	6344	7402	8600
3060								10675

Notes:
- Standard sizes vary from region to region. Check local sales centre for availability
- Non-standard sizes can be manufactured upon request

*Even though this size is commonly referred to as 1950, the actual diameter is closer to 2050
**Actual length is 2440 and weights are given accordingly
***Designed in conjuction with leading local government engineers, Humes new titan manhole has been developed to provide a positive seal. Mastic or silicon sealant is used to obtain a flexible joint and epoxy for where rigid joints are required. Lids and risers locate easily and are less difficult to set up for jointing when compared to the conventional manhole joint. (currently available in 1050mm dia units)

Flanged Base Slab:
Mass Data (kg) — Concrete density 2400

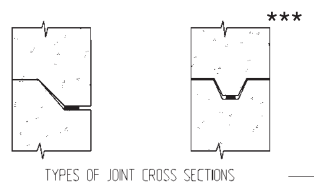
External Diameter	Internal Diameter	Nominal Base Thickness (mm)			
mm	mm	100	150	200	250
1330	**900**	333	500		
1495	**1050**	421	632		
1660	**1200**	519	779		
1825	**1350**	628	942		
1975	**1500**	735	1103		
2310	**1800**	1006	1509	2012	
2540	**2050**		1824	2432	
2870	**2300**		2329	3105	
3180	**2550**		2859	3812	4765
3710	**3060**		3892	5189	6486

Frames And Covers

Generally these are of cast iron or ductile iron construction conforming to the requirements of the appropriate local authority.

Riser Sections

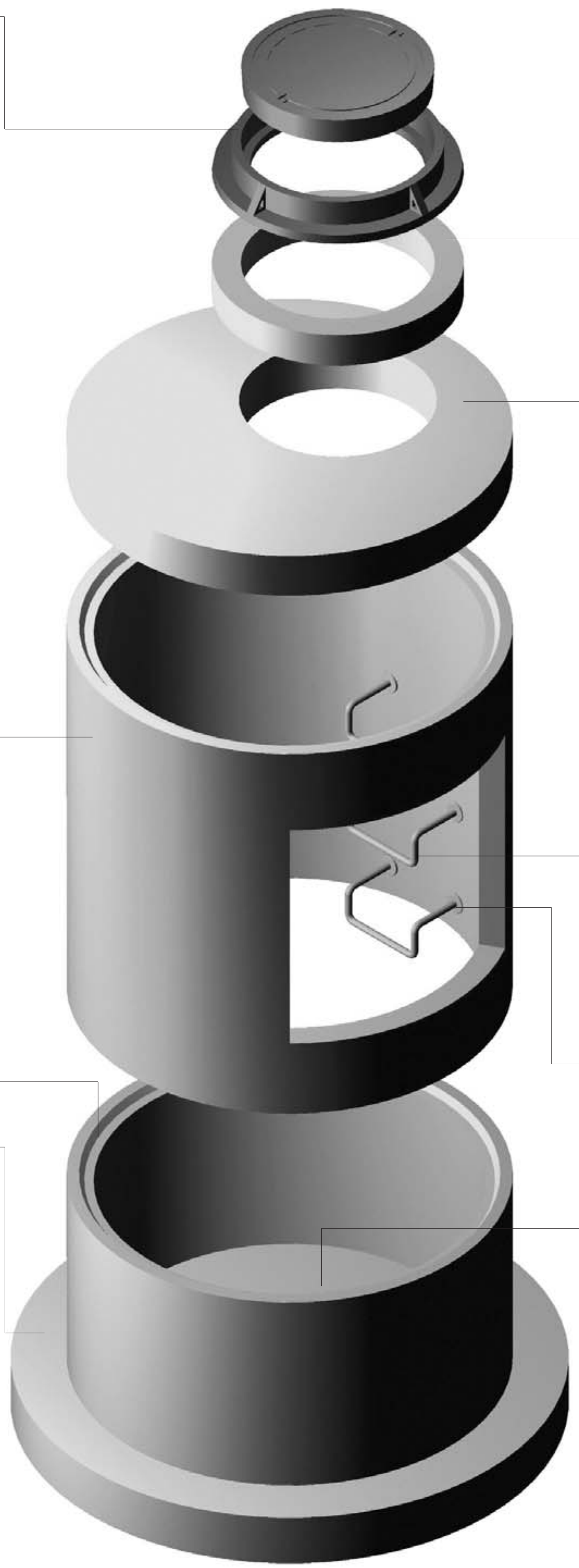
All manhole lengths have flush joint male and female type ends. Provision is made in 1050 diameter risers for the following two options:
- Humes joint clamps
- New tongue and groove flush joint



Flanged Base Slab

Standard manhole flanged bases comprise circular slabs with minimum thicknesses shown in the table below.
The flange projects 150mm from the outer diameter of the manhole riser to provide resistance to floatation.
These reinforced base slabs are factory cast into a manhole riser of any specified standard length.
Provision is made for lifting and placing using swiftlift anchors.

Notes:
- Add appropriate riser mass to obtain overall mass of complete base unit



Adjustment Rings

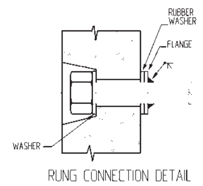
Humes reinforced concrete adjustment rings have an internal diameter of 520mm (outside diameter 675mm) and are available in thicknesses from 30 to 300mm.
The rings are placed over the opening in the manhole lid to make the final height adjustment when placing the frame and cover.

Lids

Reinforced concrete flat lids have diameters which conform to the outer diameters of manhole risers.
The standard opening of 530mm diameter is eccentrically located approximately 150mm radially from the inside of the manhole wall.
Swiflifts are provided.
The minimum thicknesses of standard lids are given in the table below.

Ladder Rungs

Standard rungs comprise mild steel hot dipped galvanised units of 20mm diameter, 250mm width and 150mm depth; of plain or stepped (safety) type.
Rungs are supplied complete with nuts and steel and rubber washers. See inset diagram for assembly detail.
Provision is made in riser sections for rungs to fitted at 300mm intervals.
Stainless steel and plastic coated rungs are available, as are various types of ladders.



Unflanged Base

The standard unit comprises a base cast into the bottom section of a manhole riser.
Thicknesses vary according to the manhole diameter, or client's specification.
All bases have reinforcement which is keyed into the riser wall.

Adjustment Rings

Nominal Weight (kg)	11	18	25	35	45	81	112	134
Thickness (mm)	30	50	75	100	150	200	250	300

Lids: Mass Data (kg) — Concrete Density 2400

External Diameter	Internal Diameter	Nominal Lid Thickness (mm)					Opening
mm	mm	75	100	150	200	250	
365	**300**	19	25				None
540	**450**	41	55				None
700	**600**	69	92				None
865	**750**	106	141	220			None
1030	**900**		200	300			None
1195	**1050**		216	324	432	541	530mm
1360	**1200**		296	444	591	739	530mm
1525	**1350**		385	578	771	964	530mm
1675	**1500**		476	714	952	1190	530mm
2010	**1800**			1063	1417	1771	530mm
2240	**2050**			1339	1786	2232	530mm
2570	**2300**			1788	2384	2980	530mm
2880	**2550**			2266	3021	3776	530mm
3410	**3060**			3208	4278	5347	530mm

Note:
Add appropriate riser mass to obtain overall mass of complete base unit.

Unflanged Base: Mass Data (kg) — Concrete Density 2400

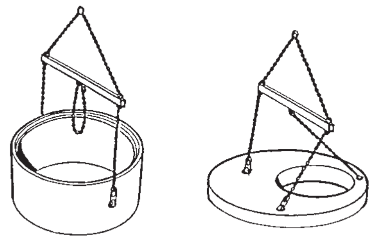
Internal Diameter	Nominal Base Thickness (mm)				
mm	75	100	150	200	250
300	13	17			
450	29	38			
600	51	68			
750	80	106			
900		153	229		
1050		208	312		
1200		271	407		
1350		344	515		
1500		424	636		
1800		611	916	1221	
2050			1188	1584	
2300			1496	1994	
2550			1839	2451	3064
3060			2647	3530	4412

Freephone 0800 502 112 **www.humes.co.nz**

Installation

The Swiftlift Manhole Lifting System

The Swiftlift Manhole Lifting System utilises specially designed spreader beams that are available for sale or hire from your local Humes outlet or branches of Alan H. Reid Engineering Ltd. The spreader beam has three chains which enables it to be used for single, double or three point lifts. When lifting a manhole lid with three lifting points the middle chain should be attached to the lifting point closest to the access hole. This minimises the load being applied to the centre of the beam.



Sealing Strip

BM100

Commonly referred to as BM100, a preformed grey sealant based on high molecular weight cross linked butyl rubber. Used to join manhole risers, this product has a moderate amount of surface tack and deforms readily under moderate loading. Has a moderate amount of surface tack and deforms readily under moderate loading.To ensure a water tight seal, do not stretch the strip to fit the joint diameter. Instead, join two strips together ensuring an overlap of product of at least 50mm at each join in the strip. Has shelf life of 6 months when sealed and stored in cool dry conditions.

Titanseal

Titanseal is a self-adhering butyl rubber compound extruded into ready to use tape form for non-structural permanent, weathertight sealing of concrete surfaces. Titanseal in easy to handle rolls of tape with virtually unlimited storage life is easily applied, especially in confined spaces as there is no mixing and no agitation of the product. Titanseal can even be made to tack to damp surfaces, but it is prepared to be used on a dry primed surface. Titanseal adheres immediately, does not shrink and is unaffected by prolonged climate exposure.

For instructions on use, please refer to manufacturers instructions.

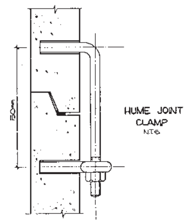
Humebond Epoxy Mortar

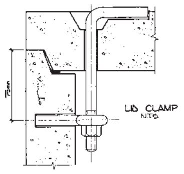
Used to join manhole risers and repair, although this can differ from area to area, eg cement mortar used as an alternative.

Humebond Epoxy Mortar is a convenient to use two part silica sand filled adhesive and jointing system developed especially for construction and concrete work.
Humebond will cure under most conditions in thick film to an extremely hard durable surface with negligible exotherm and shrinkage.
Suitable as a patching and forming compound for all concrete products. Used to seal concrete in drainage and construction applications.

Humes Joint Clamps

In some regions this jointing system is required to hold risers, lids and bases together to maintain water tightness where manholes could be subject to lateral forces.
The system uses galvanised mild steel clamps fitted across the joints after the placement of a sealing compound. Clamps are fitted after the basic construction of the manhole.





Non-standard manholes

Components of non-standard type and dimensions can be produced for special applications, such as pumping stations wells and shafts. These requirements should be discussed with the nearest Humes sales office.

Manufacturing standards

Humes Concrete Manholes are manufactured to pipe standards NZS 3107:1978 and precast standards NZS 3109:1997 with surface finishes to NZS 3114:1987.

SMARTSTREAM TECHNOLOGY

POLYETHYLENE
600 MINI MANHOLE

- One Piece Rotomolded Polyethylene Construction
- Does Not Require Concrete Encasement

D4.9 SMARTSTREAM MINI MANHOLE **DECEMBER 2012**

0800 WE PIPE (93 7473)
www.***hynds***.co.nz

HYNDS

Rotomolded LMDPE mini manhole for sewer or stormwater.

Application

Sewer or stormwater

Features

- One piece rotomolded polyethylene construction
- Very robust 13mm wall thickness
- Welded one piece unit eliminates root intrusion and groundwater ingress
- Does not suffer from gas attack (Internal or external)
- Manufactured to suit all types and sizes of pipe
- Lightweight one piece construction
- Does not require concrete encasement
- For installation from 600mm invert to 1.8m invert
- All units are pressure tested prior to dispatch

Precast Cover Slab and Class D cover

Description
600 mini manhole for 150 PVC RRJ 2 way connection
600 mini manhole for 160 o/d PE 2 way connection
600 mini manhole for 180 o/d PE 2 way connection
600 mini manhole for 250 o/d PE 2 way connection

Installation

- Bedding material is as per local council requirements
- Units supplied to suit any junction requirement or configuration
- Connect downstream and ensure that top of chamber is level
- Pack / tamp to support product
- Connect upstream
- Backfill and compact
- Install concrete cover slab and access cover

Design Specifications

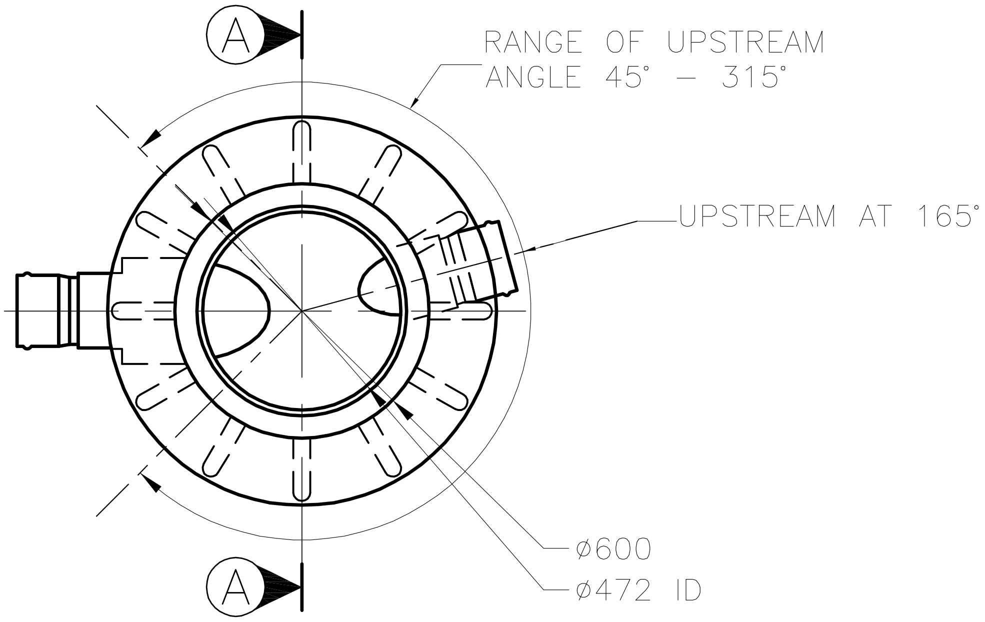
- 600mm spherical base has excellent hydraulic properties
- All independent tests show zero deterioration after 50 Years
- Polyethylene does not suffer from gas attack

Testing

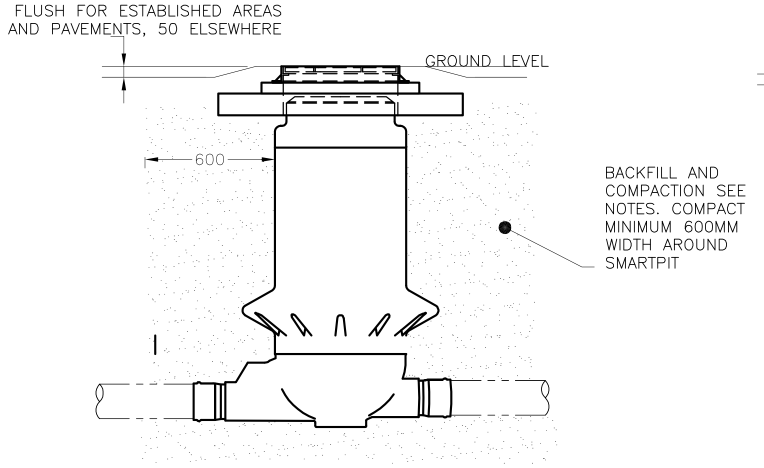
Designed, manufactured and independently tested to meet AS/NZS, and ASTM.

Durability

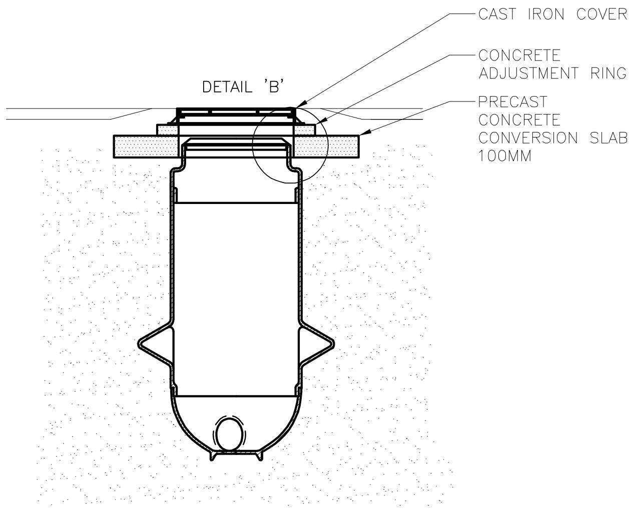
All FEA testing shows zero deterioration after 50 years. PE does not suffer from gas attack.



PLAN



SIDE ELEVATION



SECTION 'A'–'A'

D

Appendix D Construction QA Records

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Laboratory Test Report

Particle Size Distribution

TestLab CANTERBURY
CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS
57 Putiki Drive
P O Box 4256
Wanganui
Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	SCIRT trench trials	Lab Ref	13/286
Material/Origin	Fine grey subgrade sand ex trench	Report No.	132861
Condition	wet	Date of report	25/10/2013
Sampled by	TestLab	Date sampled	9/10/2013
Sampling method	ex trench	Date received	16/10/2013
Client	MacDow Canterbury, attn: Mr Clark	Date tested	17/10/2013
Comments			

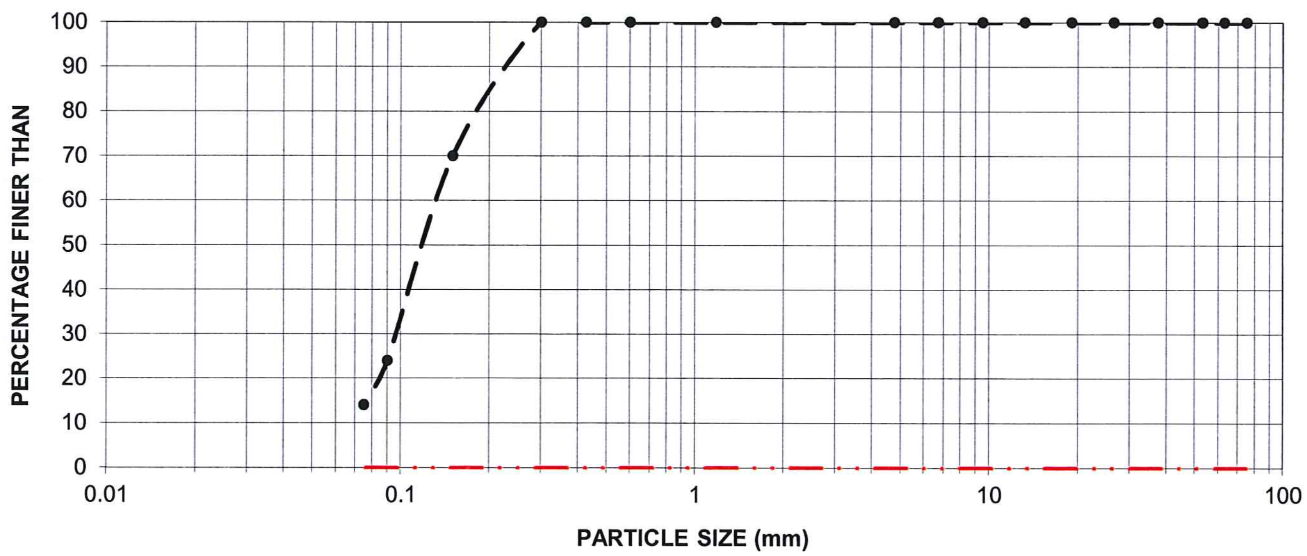
Test Methods used
Particle Size Distribution - NZS 4407:1991 Test 3.8.1 Method by Wet Sieving

Particle Size Distribution
Coarse sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0		**100**	
63.0		**100**	
53.0		**100**	
37.5		**100**	
26.5		**100**	
19.0		**100**	
13.2		**100**	
9.50		**100**	
6.70		**100**	
4.75		**100**	

Particle Size Distribution
Fine Sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
1.18		**100**	
0.60		**100**	
0.425		**100**	
0.300		**100**	
0.150		**70**	
0.090		**24**	
0.075		**14**	



All tests reported herein have been performed in accordance with the Laboratory's scope of accreditation.

This Laboratory Test Report is confidential and may be legally privileged. This report may only be reproduced in full. Alteration of the IANZ Accreditation status will invalidate this Test Report. Please contact the Laboratory to verify the status

Approved Signatory
M R Cleveland
Date 29.10.13

new zealand accredited laboratory

Laboratory Test Report

Particle Size Distribution

TestLab CANTERBURY
CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS

57 Putiki Drive
P O Box 4256
Wanganui

Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	SCIRT Trench Trials
Material/Origin	Silty gravel ex MH test 6 @ 0.6m
Condition	Damp
Sampled by	Client
Sampling method	ex trench/unknown
Client	MacDow Canterbury, attn: Mr Clark
Comments	

Lab Ref	13/289
Report No.	132891
Date of report	29/10/2013
Date sampled	21/10/2013
Date received	22/10/2013
Date tested	25/10/2013

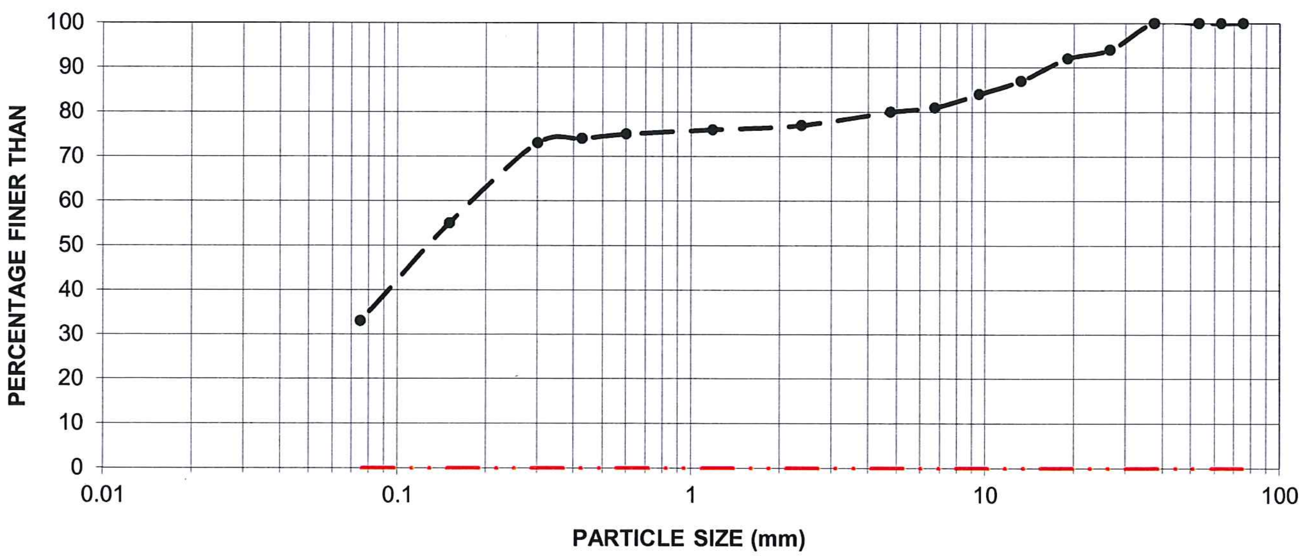
Test Methods used
Particle Size Distribution - NZS 4407:1991 Test 3.8.2 Subsidiary Method by Dry Sieving - test on oven-dried specimen

Particle Size Distribution
Coarse sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0		**100**	
63.0		**100**	
53.0		**100**	
37.5		**100**	
26.5		**94**	
19.0		**92**	
13.2		**87**	
9.50		**84**	
6.70		**81**	
4.75		**80**	

Particle Size Distribution
Fine Sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
2.36		**77**	
1.18		**76**	
0.600		**75**	
0.425		**74**	
0.300		**73**	
0.150		**55**	
0.075		**33**	



All tests reported herein have been performed in accordance with the Laboratory's scope of accreditation.

Approved Signatory
M R Cleveland
Date 29.10.13

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laboratory

Laboratory Test Report

Particle Size Distribution

TestLab CANTERBURY
CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS

57 Putiki Drive
P O Box 4256
Wanganui

Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	AP65 Stock Check
Material/Origin	Ex Winstones West Coast Rd Crusher
Condition	Damp
Sampled by	TestLab
Sampling method	NZS4407:Cl:2.4.2.6.2
Client	MacDow Canterbury, attn: Mr Clark
Comments	A typical AP65 grading envelope is shown as a guideline.

Lab Ref	13/281
Report No.	132811
Date of report	21/10/2013
Date sampled	11/10/2013
Date received	16/10/2013
Date tested	18/10/2013

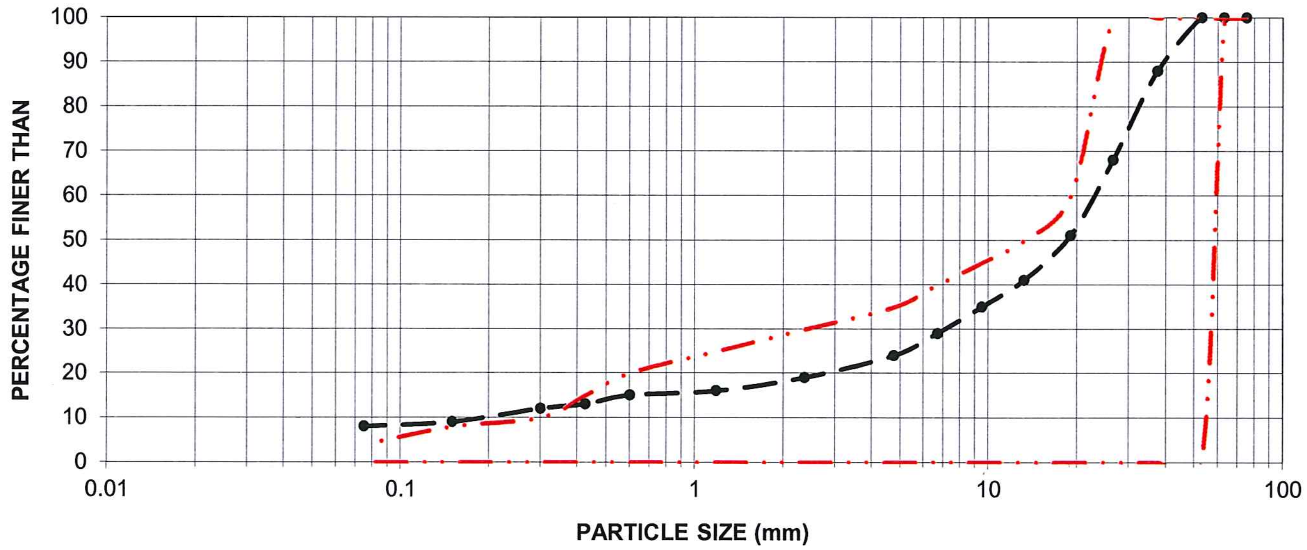
Test Methods used
Particle Size Distribution - NZS 4407:1991 Test 3.8.2 Subsidiary Method by Dry Sieving - test on oven-dried specimen

Particle Size Distribution
Coarse sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0	100	**100**	100
63.0	100	**100**	100
53.0	0	**100**	100
37.5	0	**88**	100
26.5	0	**68**	100
19.0	0	**51**	60
13.2	0	**41**	50
9.50	0	**35**	45
6.70	0	**29**	40
4.75	0	**24**	35

Particle Size Distribution
Fine Sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
2.36	0	**19**	30
1.18	0	**16**	25
0.600	0	**15**	20
0.425	0	**13**	15
0.300	0	**12**	10
0.150	0	**9**	8
0.075	0	**8**	4



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Approved Signatory
M R Cleveland
Date 29.10.13

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Laboratory Test Report

Particle Size Distribution

TestLab CANTERBURY — CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS

57 Putiki Drive
P O Box 4256
Wanganui

Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	G2 Chip Stock Check
Material/Origin	21/15 Seal Chip ex Winstones West Coast Rd. Crusher
Condition	Damp
Sampled by	TestLab
Sampling method	NZS4407:Cl:2.4.6.2.1
Client	MacDow Canterbury, attn: Mr Clark
Comments	

Lab Ref	13/284
Report No.	132841
Date of report	25/10/2013
Date sampled	11/10/2013
Date received	16/10/2013
Date tested	23/10/2013

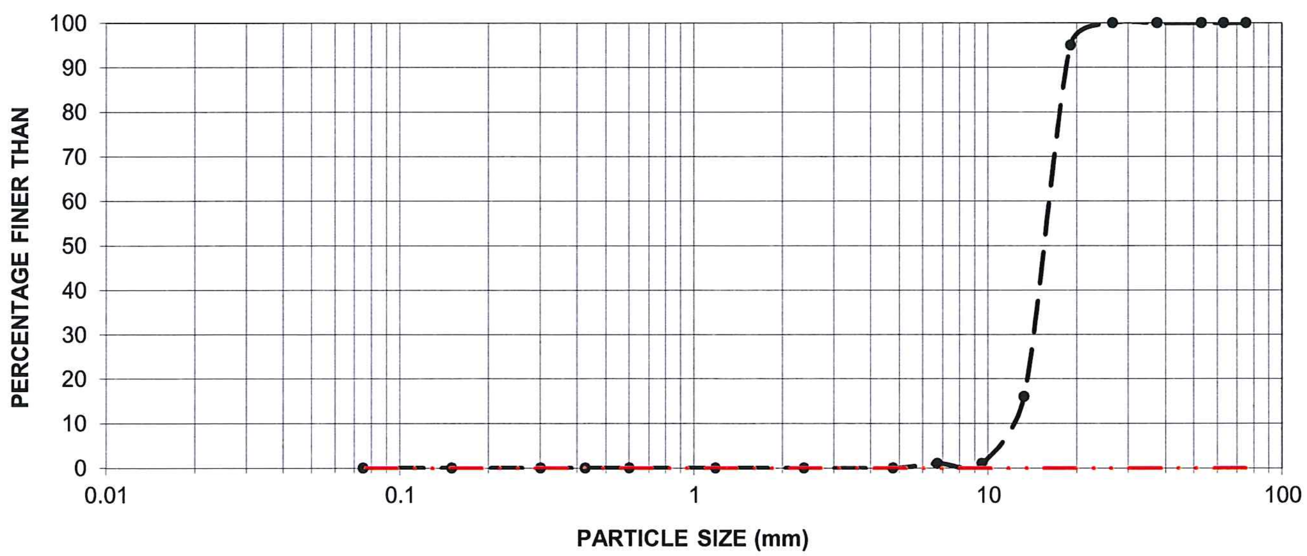
Test Methods used
Particle Size Distribution - NZS 4407:1991 Test 3.8.2 Subsidiary Method by Dry Sieving - test on oven-dried specimen

Particle Size Distribution
Coarse sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0		**100**	
63.0		**100**	
53.0		**100**	
37.5		**100**	
26.5		**100**	
19.0		**95**	
13.2		**16**	
9.50		**1**	
6.70		**1**	
4.75		**0**	

Particle Size Distribution
Fine Sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
2.36		**0**	
1.18			
0.600			
0.425			
0.300			
0.150			
0.075			



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Date 29.10.13

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Laboratory Test Report

Particle Size Distribution

new zealand accredited laboratory

TestLab CANTERBURY CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS
57 Putiki Drive
P O Box 4256
Wanganui
Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	SCIRT trench trials
Material/Origin	14/10 chip ex test 4 lateral haunching
Condition	Dry
Sampled by	Client
Sampling method	ex trench/unknown
Client	MacDow Canterbury, attn: Mr Clark
Comments	

Lab Ref	13/293
Report No.	132931
Date of report	29/10/2013
Date sampled	21/10/2013
Date received	22/10/2013
Date tested	25/10/2013

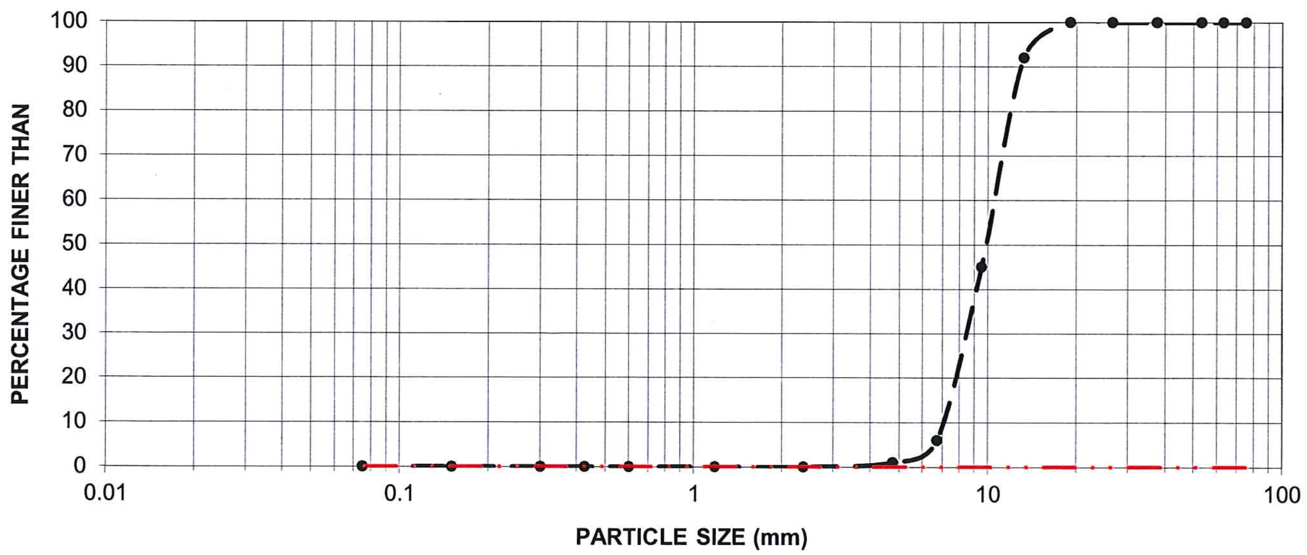
Test Methods used
Particle Size Distribution - NZS 4407:1991 Test 3.8.2 Subsidiary Method by Dry Sieving - test on oven-dried specimen

Particle Size Distribution
Coarse sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0		**100**	
63.0		**100**	
53.0		**100**	
37.5		**100**	
26.5		**100**	
19.0		**100**	
13.2		**92**	
9.50		**45**	
6.70		**6**	
4.75		**1**	

Particle Size Distribution
Fine Sizes

Size Size Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
2.36		**0**	
1.18			
0.600			
0.425			
0.300			
0.150			
0.075			



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Approved Signatory	
M R Cleveland	[signature]
Date	29.10.13

new zealand accredited laboratory

Laboratory Test Report

Particle Size Distribution

TestLab CANTERBURY — CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS

57 Putiki Drive
P O Box 4256
Wanganui

Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	SCIRT trench trials
Material/Origin	8/10 pea metal/rounds
Condition	Dry
Sampled by	Client
Sampling method	ex trench/unknown
Client	MacDow Canterbury, attn: Mr Clark
Comments	

Lab Ref	13/294
Report No.	132941
Date of report	29/10/2013
Date sampled	21/10/2013
Date received	22/10/2013
Date tested	25/10/2013

Test Methods used

Particle Size Distribution - NZS 4407:1991 Test 3.8.2 Subsidiary Method by Dry Sieving - test on oven-dried specimen

Particle Size Distribution

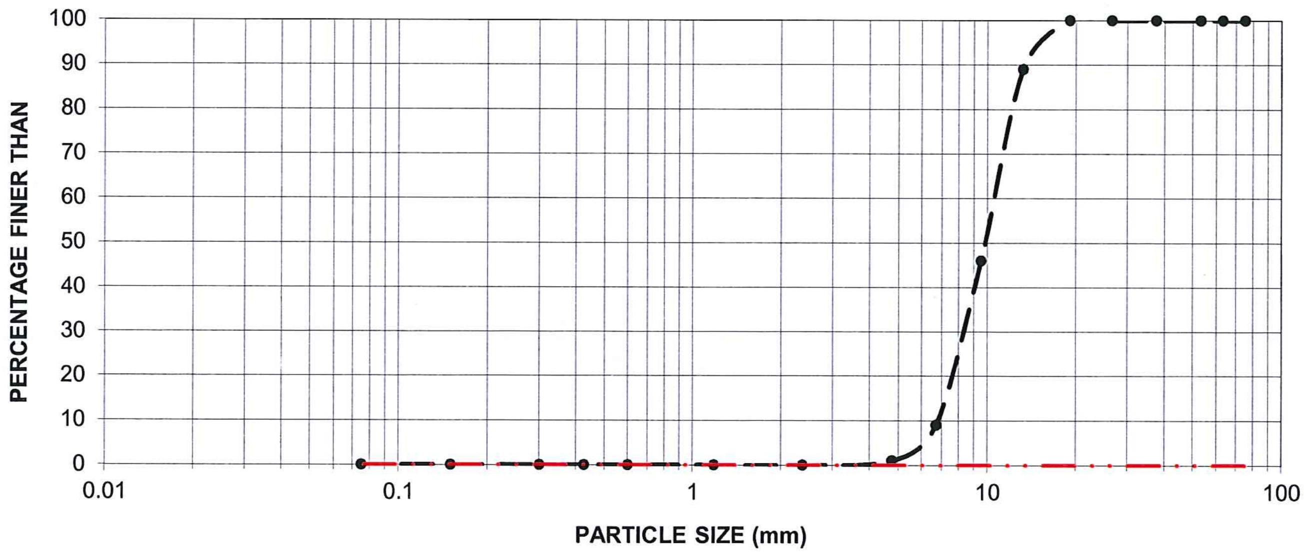
Coarse sizes

Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
75.0		**100**	
63.0		**100**	
53.0		**100**	
37.5		**100**	
26.5		**100**	
19.0		**100**	
13.2		**89**	
9.50		**46**	
6.70		**9**	
4.75		**1**	

Particle Size Distribution

Fine Sizes

Size (mm)	*Coarse Limit (%)*	*Cumulative Percent Passing (%)*	*Fine Limit (%)*
2.36		**0**	
1.18			
0.600			
0.425			
0.300			
0.150			
0.075			



Approved Signatory

M R Cleveland

Date 29.10.13

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Laboratory Test Report

AP20 Basecourse

TestLab CANTERBURY — CIVIL ENGINEERING MATERIALS TESTING SPECIALISTS

57 Putiki Drive
P O Box 4256
Wanganui

Phone 348 4199
Fax 06 348 7399
office@testlab.co.nz

Job	AP20 Stock Check	Lab Ref	13/283
Material/Origin	M4 AP20 ex Winstones West Coast Rd. Crusher	Report No.	132831
Condition	Damp	Date of report	25/10/2013
Sampled by	TestLab	Date sampled	11/10/2013
Sampling method	NZS4407:Cl:2.4.6.2.2	Date received	16/10/2013
Client	MacDow Canterbury, attn: Mr Clark	Date tested	18/10/2013
Comments			

Test Methods used
Sand Equivalent - NZS 4407:1991 Test 3.6 - hand shaking
Particle Size Distribution - NZS 4407:1991 Test 3.8.1 Preferred Method by Wet Sieving - % passing 75um was obtained by difference
Broken Faces content of Aggregate - NZS 4407:1991 Test 3.14

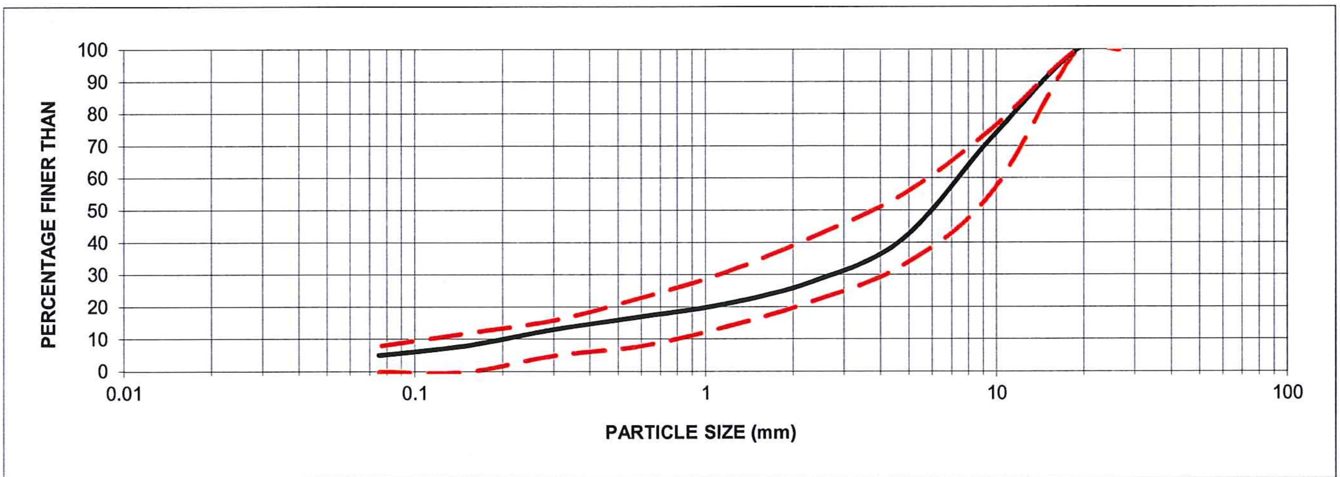
Particle Size Distribution

Size (mm)	*Coarse Limit (%)*	*Grading (%)*	*Fine Limit (%)*
26.5	100	**100**	100
19.0	100	**100**	100
9.5	55	**72**	75
4.75	33	**41**	55
2.36	22	**28**	42
1.18	14	**21**	31
0.600	8	**17**	23
0.300	5	**13**	16
0.150	0	**8**	12
0.075	0	**5**	8

Broken Faces Content

Material Fraction	(%)	Specified
37.5 to 19.0 (mm)	**Not tested**	≥70
19.0 to 9.5 (mm)	**Not tested**	≥70
9.5 to 4.75 (mm)	**Not tested**	≥70

	Result	*Specified*
Sand Equivalent	**Not tested**	≥40



Fraction Limits (mm)	*Low Limit (%)*	*Within Fraction*	*High Limit (%)*
Fraction 19.0-4.75 mm	28	**59**	48
Fraction 9.5-2.36 mm	14	**44**	34
Fraction 4.75-1.18 mm	7	**20**	27
Fraction 2.36-0.600 mm	6	**11**	22
Fraction 1.18-0.300 mm	5	**8**	19
Fraction 0.600-0.150 mm	2	**9**	14

All tests reported herein have been performed in accordance with the Laboratory's scope of accreditation.

Approved Signatory
M R Cleveland
Date 29.10.13

DENSITY TEST REPORT

Project :	**Material Investigation**
Location :	**SCT-SCT#1405**
Client :	**City Care Laboratory**
Contractor :	**City Care Laboratory**
Sampled by :	**Not Advised**
Date sampled :	**Not Advised**
Sampling method :	**Not Advised**
Sample description :	**SCIRT & EQC Liquifaction Trial**
Sample condition :	**Air Dry as Received**

Project No :	**6-JCITY.14/006LC**
Lab Ref No :	**12297**
Client Ref No :	**PO914917487**

Test Results

	Sample 1	Sample 2
Density (t/m^3)	**1.58**	**1.52**

Test Method		**Notes**
Density	NZS 3112:1986, Part 1, Test 1.4	Sampling is outside the laboratory's scope of accreditation

Date tested : 18 February 2014
Date reported : 18 February 2014

Sampling is not covered by IANZ Accreditation. Results apply only to sample tested.
This report may only be reproduced in full

IANZ Approved Signatory

Designation : *Senior Civil Engineering Technician*
Date : 18 February 2014

IANZ ACCREDITED LABORATORY

Tests indicated as not accredited are outside the scope of the laboratory's accreditation

Opus International Consultants Ltd
Christchurch Laboratory
Quality Management Systems Certified to ISO 9001

52C Hayton Rd, Wigram
PO Box 1482, Christchurch Mail Centre,
Christchurch 8140, New Zealand

Telephone +64 3 343 0739
Facsimile +64 3 343 0737
Website www.opus.co.nz

Canterbury Laboratory

325 Pound Rd, Yaldhurst
PO Box 16-064, Christchurch 8441
Telephone: +64 3 349 9142
Facsimile: +64 3 349 9143
www.fultonhogan.com
0800 LABORATORY

Report No: ND:CAN13W4181
Issue No: 1

Nuclear Density Report

Client:
Tru-Line Civil Limited
PO Box 522

Greymouth 7840

Project: QA Testing - Tru-Line Drainage Ltd

The test (s) reported herein (unless indicated) have been performed in accordance with the laboratory's scope of accreditation. Results only apply to samples as received. This report must be reproduced in full.

Approved Signatory: Murray Cleveland
(Lab Technician)
IANZ Accreditation No:200
Date of Issue: 23/10/13

Testing Details

Site Tested: Trenchline, Androssan Street, Avondale
Tested By: Daniel Daly
Date Tested: 17/10/13
Time Tested: 08:30
Material: Compacted On-site Material
Specification: ND - Testing 4431:1989 Cohesive
Field Methods: NZS 4407:1991 Test 4.2.2

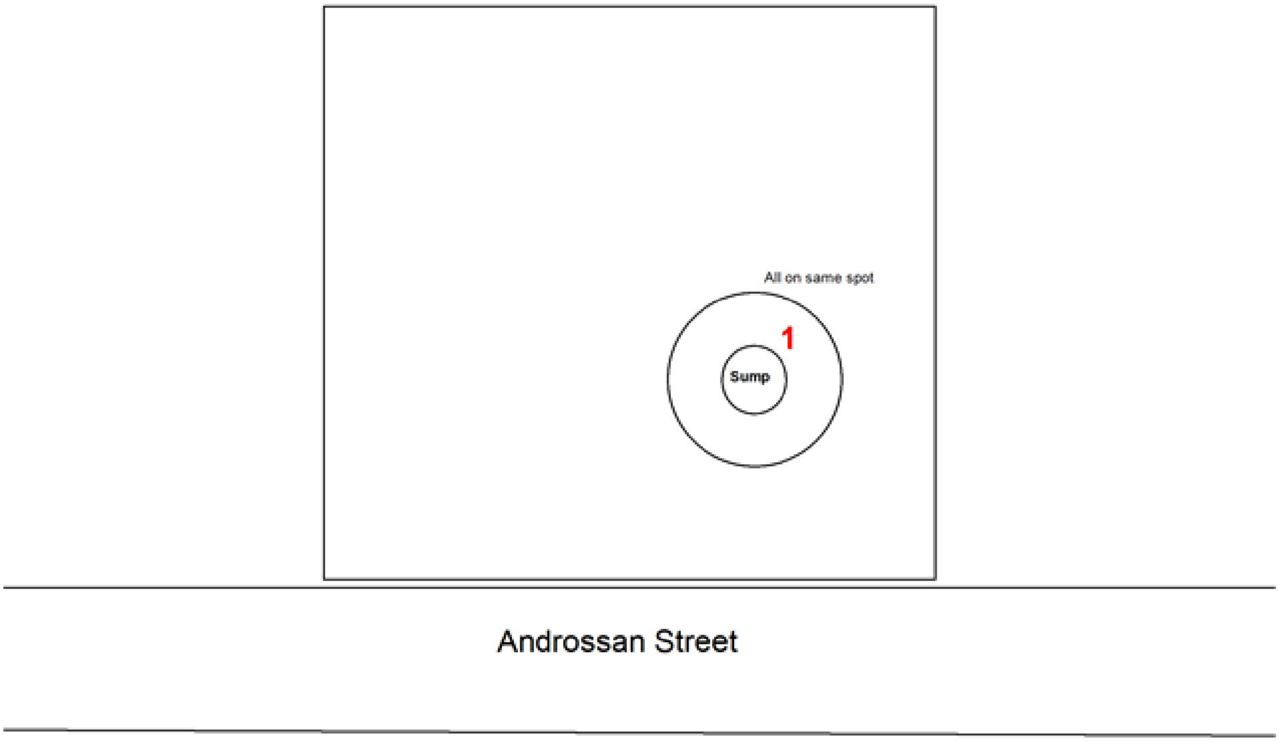
Test Results

Site No	Layer	Moisture (%)	Wet Density (t/m³)	Dry Density (t/m³)
1	Layer 1	12.5	1.70	1.51
2	Layer 2	12.5	1.82	1.62
3	Layer 3	15.5	1.81	1.57
4	Layer 4	15.0	1.75	1.52
5	Layer 5	18.0	1.94	1.64

Comments

CAN13W4181

Test Site: Trenchline, Androssan Street, Avondale



Site plan is not to scale & test sites are approximate only.

Canterbury Laboratory

325 Pound Rd, Yaldhurst
PO Box 16-064, Christchurch 8441
Telephone: +64 3 349 9142
Facsimile: +64 3 349 9143
www.fultonhogan.com
0800 LABORATORY

Report No: ND:CAN13W4221
Issue No: 1

Nuclear Density Report

Client:
Tru-Line Civil Limited
PO Box 522

Greymouth 7840

Project: QA Testing - Tru-Line Drainage Ltd

The test (s) reported herein (unless indicated) have been performed in accordance with the laboratory's scope of accreditation. Results only apply to samples as received. This report must be reproduced in full.

IANZ ACCREDITED LABORATORY

Approved Signatory: Maciej Gaworecki
(Senior Civil Technician)
IANZ Accreditation No:200
Date of Issue: 22/10/13

Testing Details

Site Tested: Trenchline, Ardrossan Street, Avondale
Tested By: Daniel Daly
Date Tested: 17/10/13
Time Tested: 08:00
Material: Compacted 65mm Aggregate
Specification: ND - TNZ B/2 Sub-Base
Field Methods: NZS 4407:1991 Test 4.2.2
Lab Methods: NZS 4402:1986 Test 4.1.3 - 1986

Compaction Target Details

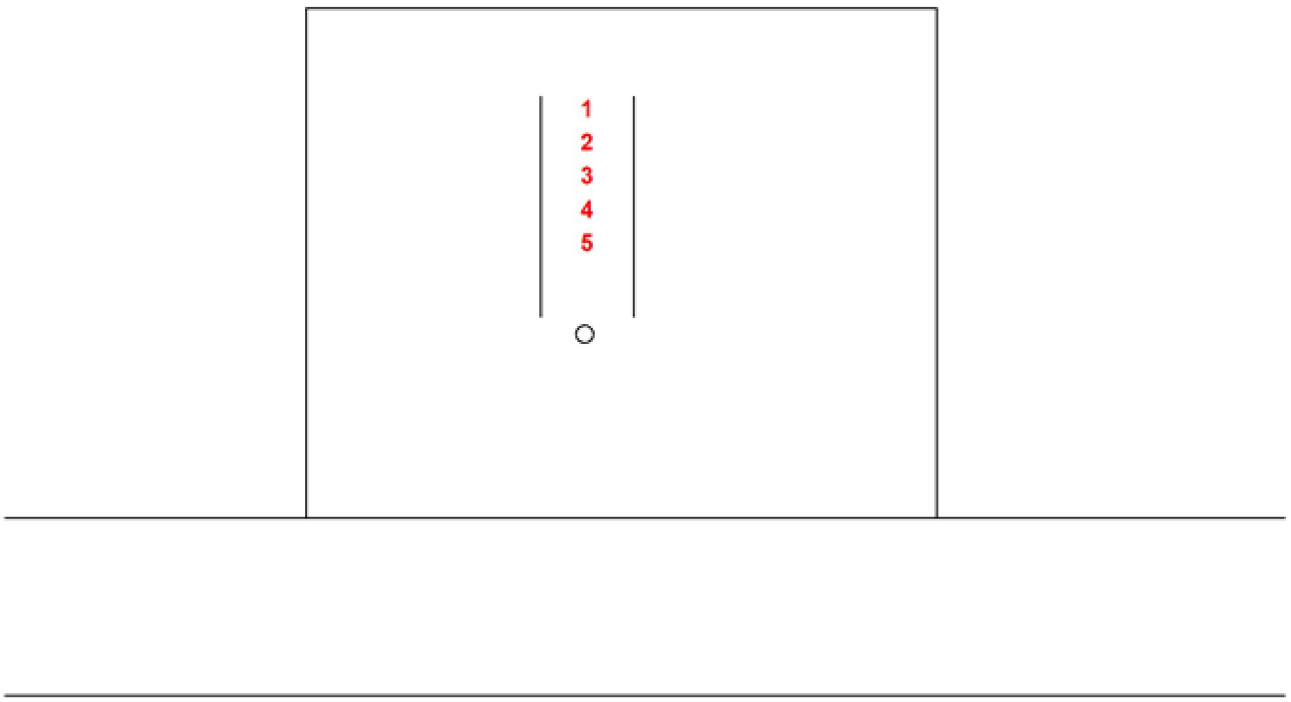
Material Sample ID: CAN13S-00633
MDD Method: NZS 4402:1986 Test 4.1.3 - 1986
Max. Dry Density: 2.34 t/m³ @ 3.8 %
Min. Dry Density (t/m³): 2.22
Solid Density Type: Assumed

Test Results

Site No	Layer	Moisture (%)	Wet Density (t/m³)	Dry Density (t/m³)	Relative Compaction (%)
1	Final Layer	4.5	2.39	2.28	97.4
2	Layer 1	3.0	2.30	2.23	95.3
3	Layer 2	4.0	2.37	2.28	97.4
4	Layer 3	5.5	2.40	2.27	97.0
5	Layer 4	4.5	2.34	2.24	95.7

Comments

CAN13W4221
Test Site: Trenchline, Androssan Street, Avondale



Site plan is not to scale & test sites are approximate only.

Canterbury Laboratory

325 Pound Rd, Yaldhurst
PO Box 16-064, Christchurch 8441
Telephone: +64 3 349 9142
Facsimile: +64 3 349 9143
www.fultonhogan.com
0800 LABORATORY

Fulton Hogan

Report No: ND:CAN13W4255
Issue No: 1

Nuclear Density Report

Client:
Tru-Line Civil Limited
PO Box 522

Greymouth 7840

Project: QA Testing - Tru-Line Drainage Ltd

The test (s) reported herein (unless indicated) have been performed in accordance with the laboratory's scope of accreditation. Results only apply to samples as received. This report must be reproduced in full.

IANZ ACCREDITED LABORATORY

Approved Signatory: Maciej Gaworecki
(Senior Civil Technician)
IANZ Accreditation No:200
Date of Issue: 22/10/13

Testing Details

Site Tested: Trenchline, Androssan Street, Avondale
Tested By: Daniel Daly
Date Tested: 18/10/13
Time Tested: 08:00
Material: Compacted 20mm Aggregate
Specification: ND - TNZ B/2 Basecourse
Field Methods: NZS 4407:1991 Test 4.2.2
Lab Methods: NZS 4402:1986 Test 4.1.3 - 1986

Compaction Target Details

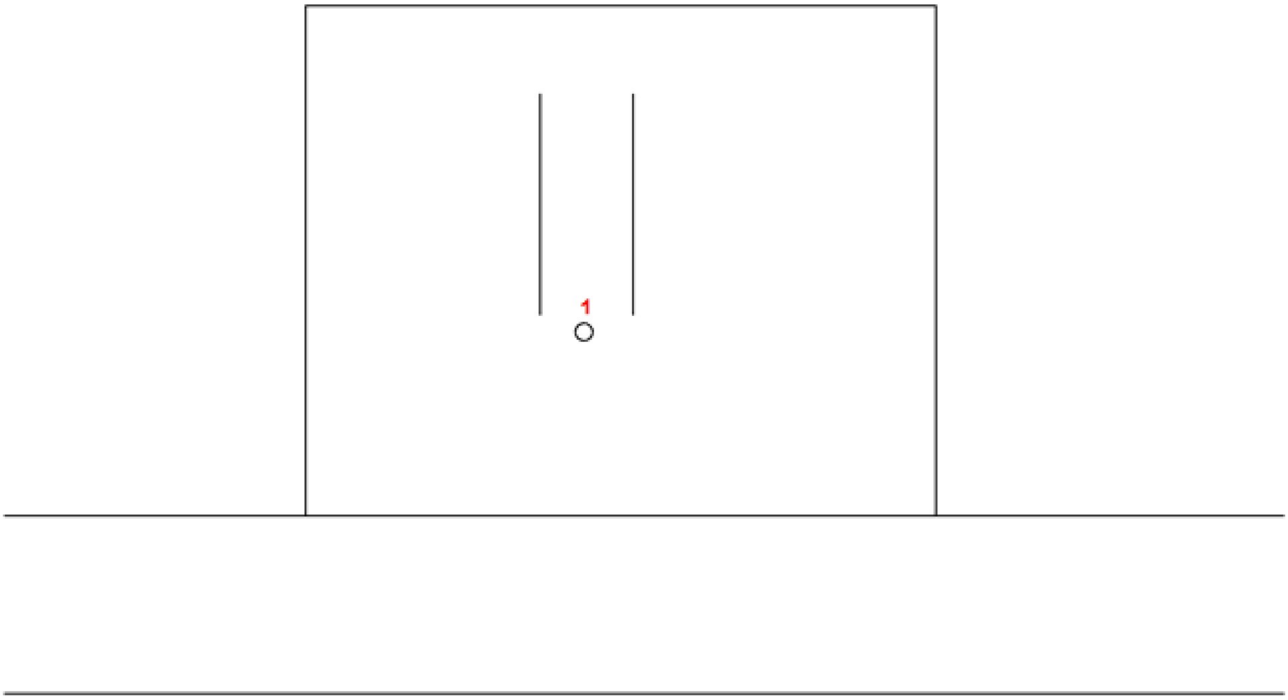
Material Sample ID: CAN11S-05164
MDD Method: NZS 4402:1986 Test 4.1.3 - 1986
Max. Dry Density: 2.32 t/m³ @ 4.6 %
Min. Dry Density (t/m³): 2.09
Solid Density Type: Measured

Test Results

Site No	Layer	Moisture (%)	Wet Density (t/m³)	Dry Density (t/m³)	Relative Compaction (%)
1	Haunching	6.5	2.25	2.11	90.9

Comments

CAN13W4255
Test Site: Trenchline, Androssan Street, Avondale

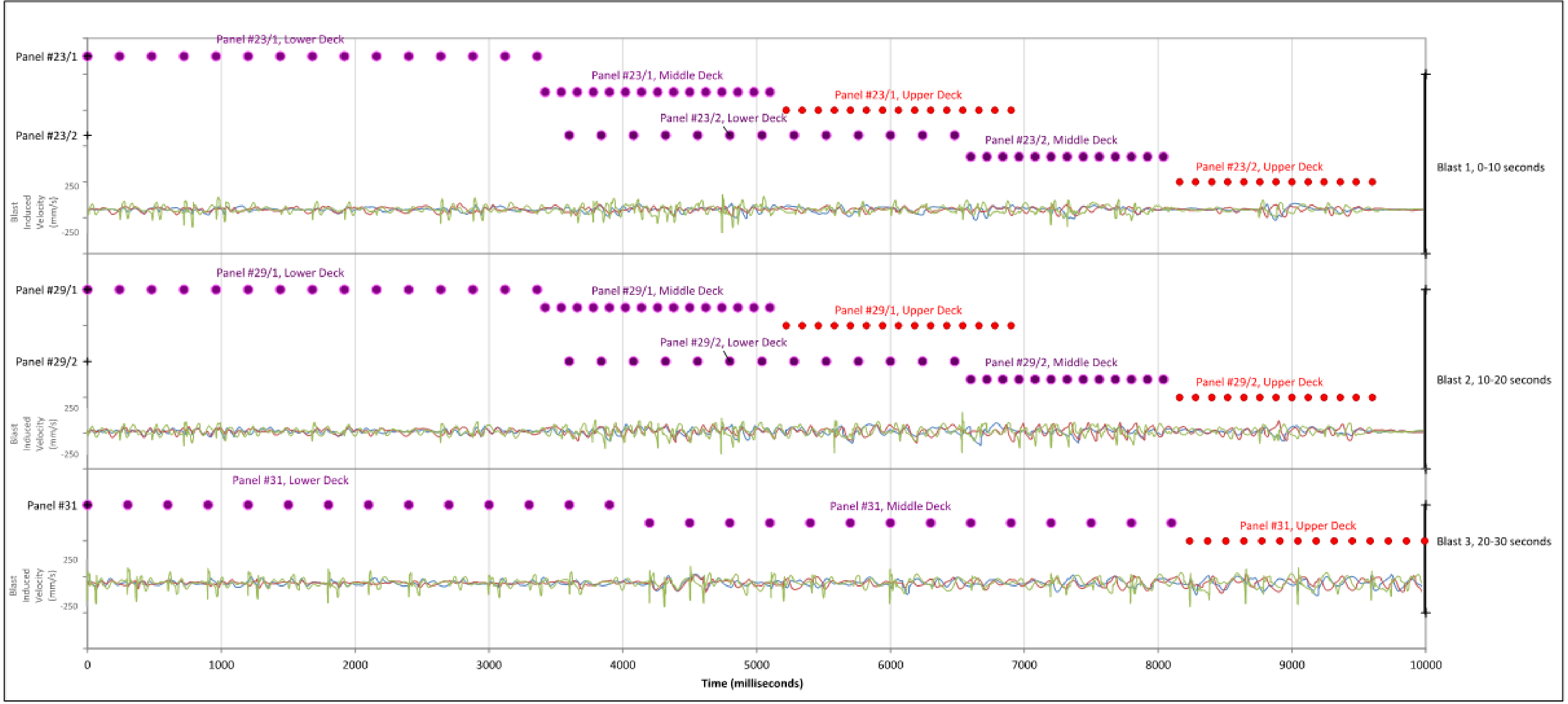


Site plan is not to scale & test sites are approximate only.

E

Appendix E Explosion Ground Motion Data

Production Blast 4: Blast at 3:44pm, 24/10/2013



Panel #23/1	Lower Deck	Middle Deck	Upper deck
Blast Casing ID (Panel # - Casing #)	Time (ms)	Time (ms)	Time (ms)
H23-03	0	3420	5220
H23-07	240	3540	5340
H23-27	480	3660	5460
H23-09	720	3780	5580
H23-25	960	3900	5700
H23-05	1200	4020	5820
H23-01	1440	4140	5940
H23-08	1680	4260	6060
H23-26	1920	4380	6180
H23-04	2160	4500	6300
H23-02	2400	4620	6420
H23-11	2640	4740	6540
H23-24	2880	4860	6660
H23-06	3120	4980	6780
H23-28	3360	5100	6900

Panel #23/2	Lower Deck	Middle Deck	Upper deck
Blast Casing ID (Panel # - Casing #)	Time (ms)	Time (ms)	Time (ms)
H23-15	3600	6600	8160
H23-19	3840	6720	8280
H23-13	4080	6840	8400
H23-21	4320	6960	8520
H23-17	4560	7080	8640
H23-10	4800	7200	8760
H23-23	5040	7320	8880
H23-14	5280	7440	9000
H23-20	5520	7560	9120
H23-12	5760	7680	9240
H23-22	6000	7800	9360
H23-16	6240	7920	9480
H23-18	6480	8040	9600

Panel #29/1	Lower Deck	Middle Deck	Upper deck
Blast Casing ID (Panel # - Casing #)	Time (ms)	Time (ms)	Time (ms)
H29-03	0	3420	5220
H29-07	240	3540	5340
H29-27	480	3660	5460
H29-09	720	3780	5580
H29-25	960	3900	5700
H29-05	1200	4020	5820
H29-01	1440	4140	5940
H29-08	1680	4260	6060
H29-26	1920	4380	6180
H29-04	2160	4500	6300
H29-02	2400	4620	6420
H29-10	2640	4740	6540
H29-24	2880	4860	6660
H29-06	3120	4980	6780
H29-28	3360	5100	6900

Panel #29/2	Lower Deck	Middle Deck	Upper deck
Blast Casing ID (Panel # - Casing #)	Time (ms)	Time (ms)	Time (ms)
H29-15	3600	6600	8160
H29-19	3840	6720	8280
H29-13	4080	6840	8400
H29-21	4320	6960	8520
H29-17	4560	7080	8640
H29-11	4800	7200	8760
H29-23	5040	7320	8880
H29-14	5280	7440	9000
H29-20	5520	7560	9120
H29-12	5760	7680	9240
H29-22	6000	7800	9360
H29-16	6240	7920	9480
H29-18	6480	8040	9600

Panel #31	Lower Deck	Middle Deck	Upper deck
Blast Casing ID (Panel # - Casing #)	Time (ms)	Time (ms)	Time (ms)
S-04	0	4200	8235
S-13	300	4500	8370
S-07	600	4800	8505
S-05	900	5100	8640
S-10	1200	5400	8775
S-02	1500	5700	8910
S-14	1800	6000	9045
S-06	2100	6300	9180
S-11	2400	6600	9315
S-01	2700	6900	9450
S-09	3000	7200	9585
S-03	3300	7500	9720
S-12	3600	7800	9855
S-08	3900	8100	9990

Figure I2.4.3

Legend
Measured Ground Surface Velocities
- Radial Velocity (mm/s)
- Transverse Velocity (mm/s)
- Vertical Velocity (mm/s)

Explosive Charge Mass
- 2.4kg
- 1.1kg

EQC EARTHQUAKE COMMISSION

Ground Improvement Blasting Trials
Site 4 (Avondale) Production Blast 4
Blast Timing Chart and Measured Ground Surface Velocities

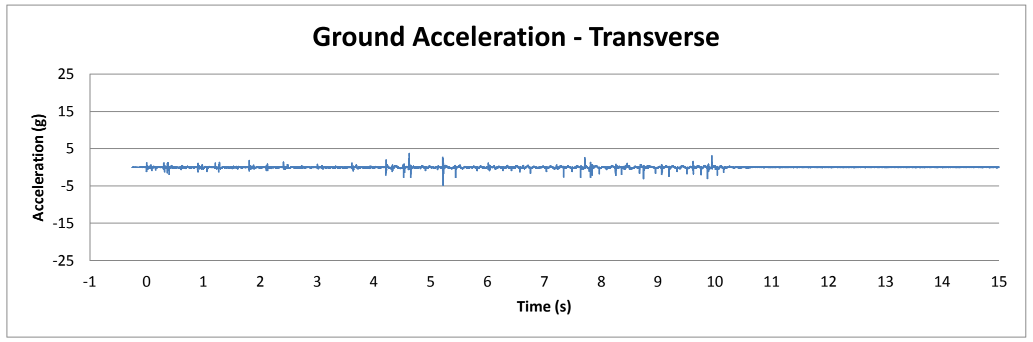
Location: Site 4
Job No.: 52020.02
Version No.: 1.0

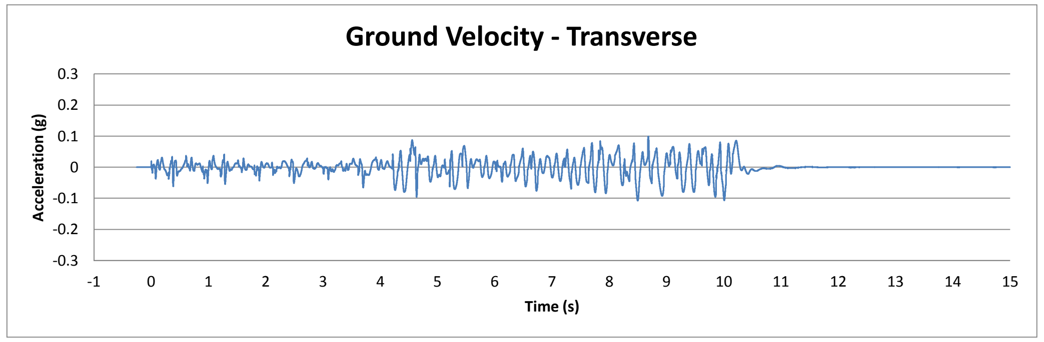
Date: 17/04/2014
Prepared by: KCH/JMZ
Checked by: PWQ

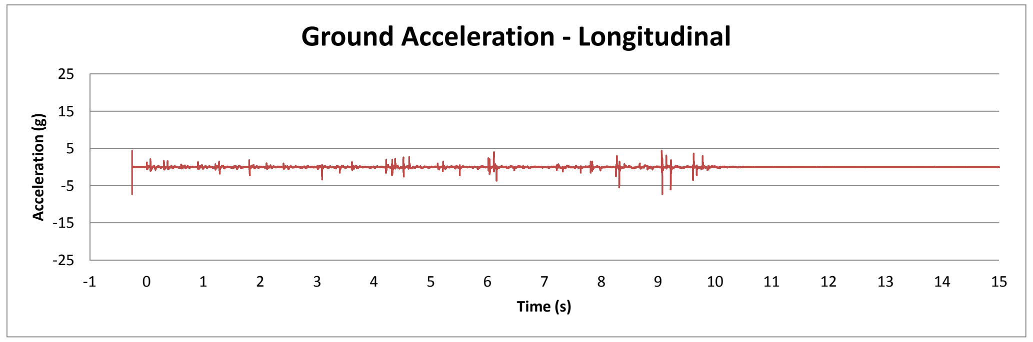
SCIRT and EQC Liquefaction Trial

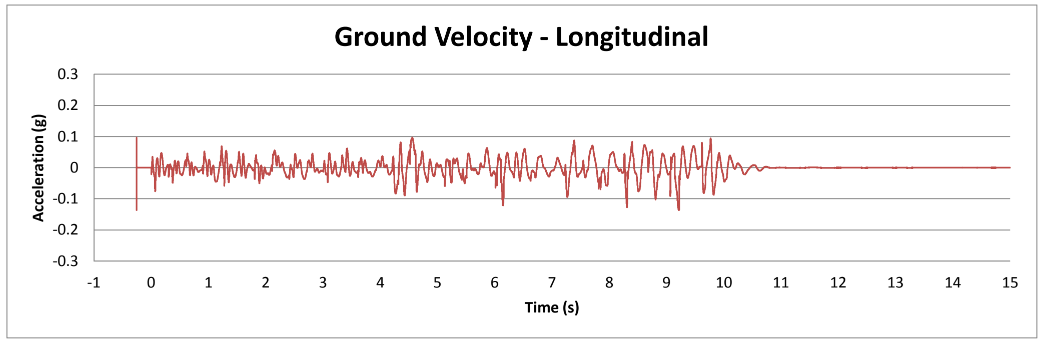
Analysis of Accelerometer

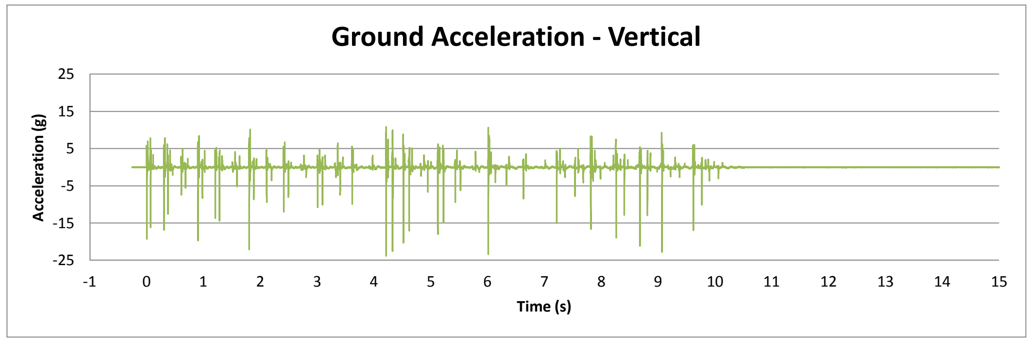
Event Type	Full Waveform
Serial Number	BE14104
Version	V 10.52-8.17 Minim ate Plus
File Name	P104F1C8.MB0
Event Time	15:34:59
Event Date	October 24, 2013
Trigger	Vert
Geo Trigger Level	0.910 mm/s
Pre-trigger Length	-0.250 sec
Record Time	15.0 sec
Record Stop Mode	Fixed
Sample Rate	4096 sps
Step Interval:	0.000244141 sec
Battery Level	6.2 Volts
Calibration	March 21, 2013 by Instantel
Units	mm/s and pa.(L)
Geo Range	254 mm/s
Tran PPV	108 mm/s
Vert PPV	*** mm/s
Long PPV	137 mm/s
Tran ZC Freq	5.2 Hz
Vert ZC Freq	7.2 Hz
Long ZC Freq	4.9 Hz
Tran Time of Peak	8.500 sec
Vert Time of Peak	4.327 sec
Long Time of Peak	9.218 sec
Tran Peak Acceleration	4.83 g
Vert Peak Acceleration	23.8 g
Long Peak Acceleration	7.32 g
Tran Peak Displacement	3.68 mm
Vert Peak Displacement	5.01 mm
Long Peak Displacement	4.47 mm
Peak Vector Sum	*** mm/s
Peak Vector Sum Time	8.264 sec
Microphone	Linear Weighting
MicL PSPL	256.3 pa.(L)
MicL Time of Peak	9.071 sec
MicL ZC Freq	15.6 Hz
Tran Test Freq	7.5 Hz
Tran Test Ratio	3.9
Tran Test Results	Passed
Vert Test Freq	7.5 Hz
Vert Test Ratio	3.7
Vert Test Results	Passed
Long Test Freq	7.2 Hz
Long Test Ratio	4.2
Long Test Results	Passed
MicL Test Freq	20.5 Hz
MicL Test Amplitude	495 mv
MicL Test Results	Passed
***	Out of Range
Monitor Log(s)	
Oct 24 /13 15:34:58Oct 24 /13 15:35:14Event recorded.	
PC SW Version	V8.12 - 8.12

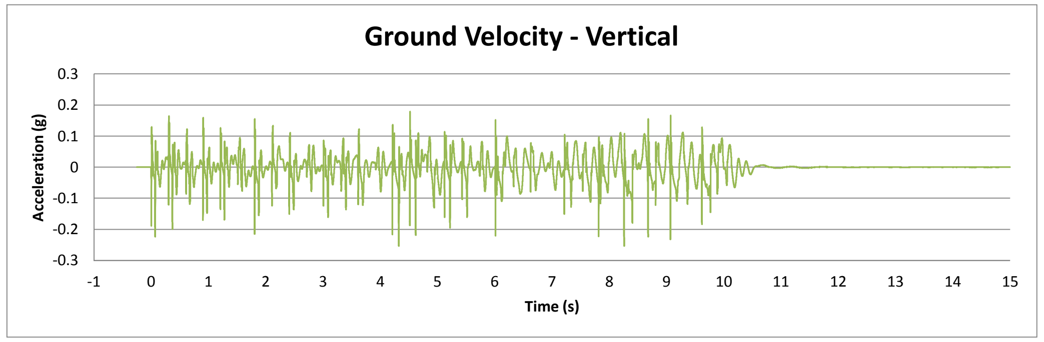


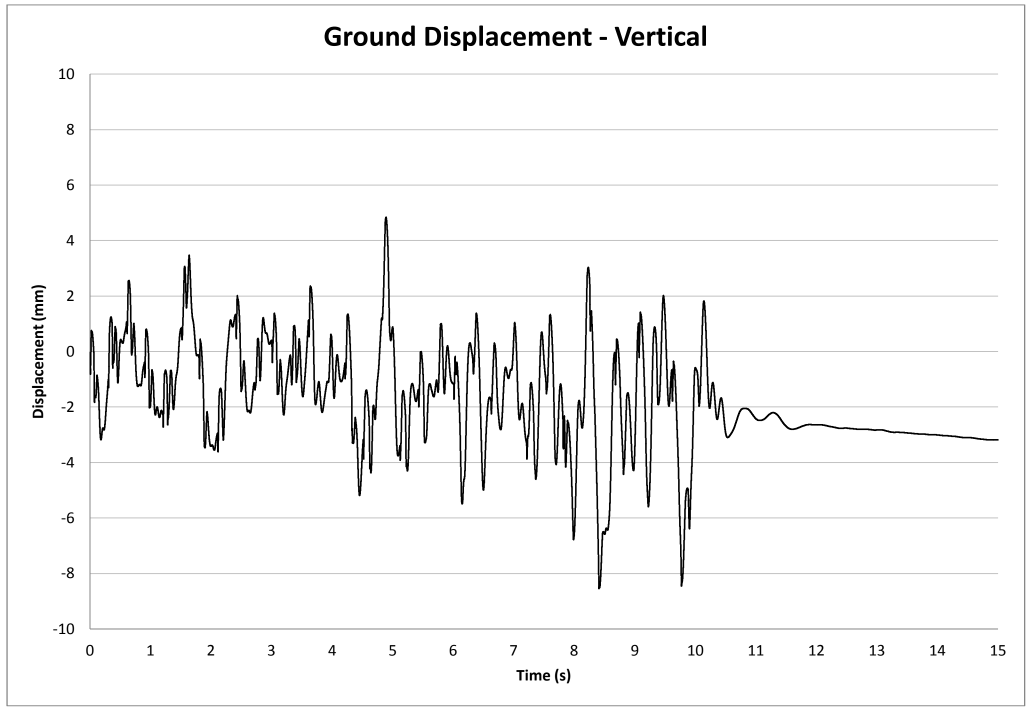


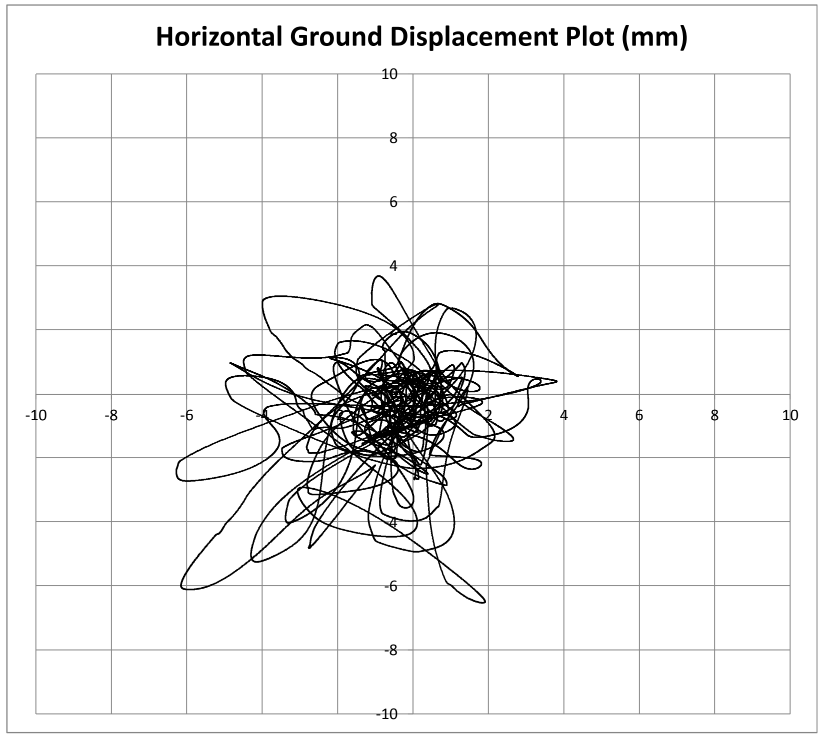












F

Appendix F Pore Pressure Transducer Results

Calibration Correction

Pore pressure transducers (PPT) were installed at the site at depths of 2.8 m, 3.9 m, 4.65 m, 6.65 m and 9 m depth to measure the porewater response to the blasting. Additional PPTs were installed immediately beneath each chamber to record pore pressure at these locations.

Special PPT's were utilised to accommodate the very high blast pressure from the explosives. Calibration following the trial has identified that the transducers have a lower bound pressure which can be recorded (approximately 28 kPa, equating to about 2.4 m of static water head). For any pressure below the lower bound the PPT will recorded the lower bound value. A shift of 4.5 kPa was also identified during calibration as shown in Figure C1 below.

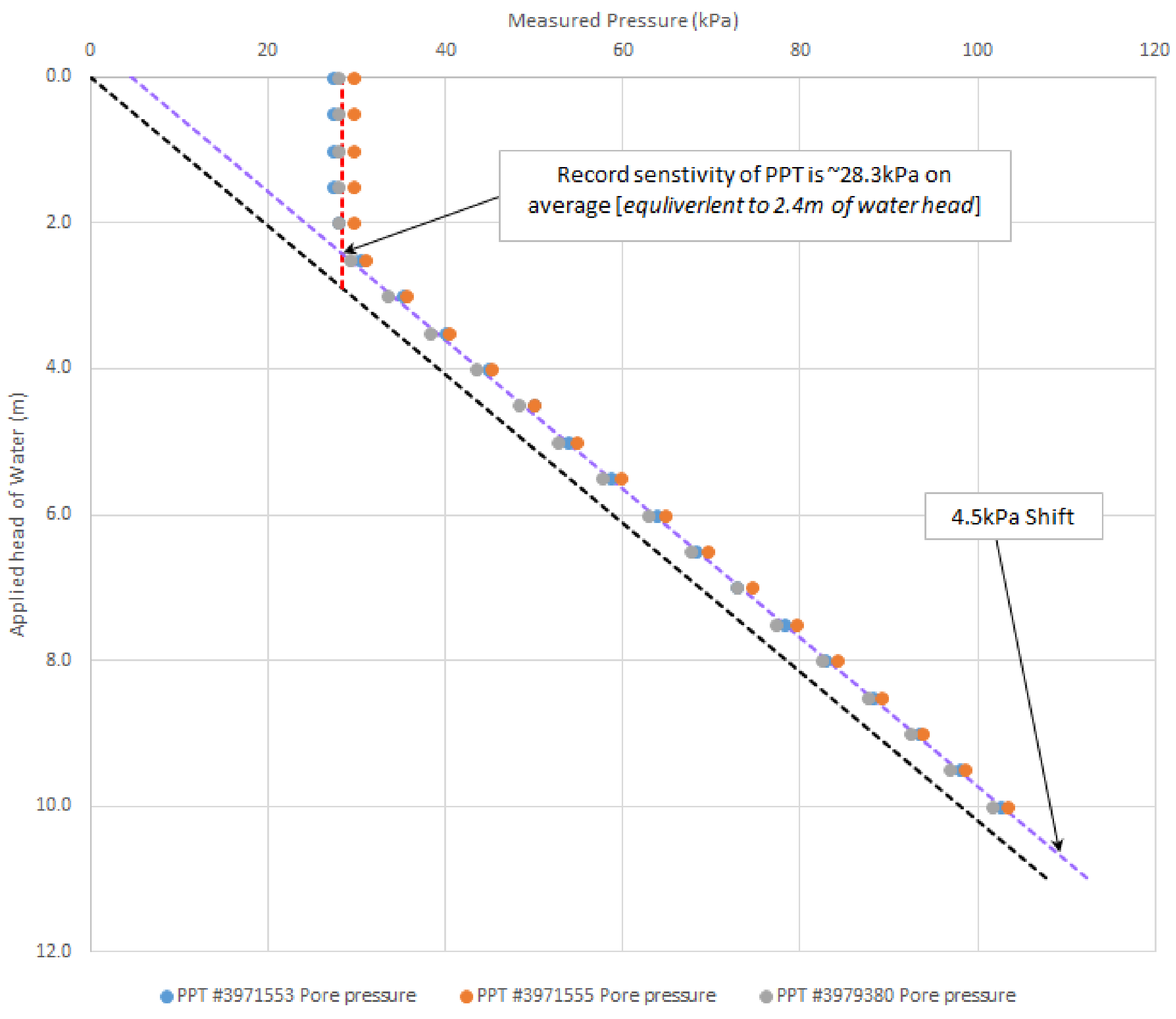


Figure C1: Example of PPT calibration

The transducers located beneath the chambers are approximately 1.0 m to 1.8 m below the ground water level. These PPTs are initially below the lower threshold and did not record increases in pore pressure until the pressure exceeded the threshold. An adjustment to the recorded PPT data has been made to account for the threshold and shift identified during calibration. For this adjustment an initial static groundwater level was adopted based on site observations. This is used to estimate the initial porewater pressure for each PPT. The adjusted change in porewater pressure is calculated by adding the difference between the PPT threshold and adopted initial porewater pressure to the PPT recorded value.

$$\Delta u = u_{PPT} + (u_{threshold} - u_{initial})$$

Where:

$\Delta u = change\ in\ pore\ water\ pressure$

$u_{PPT} = recorded\ PPT\ value$

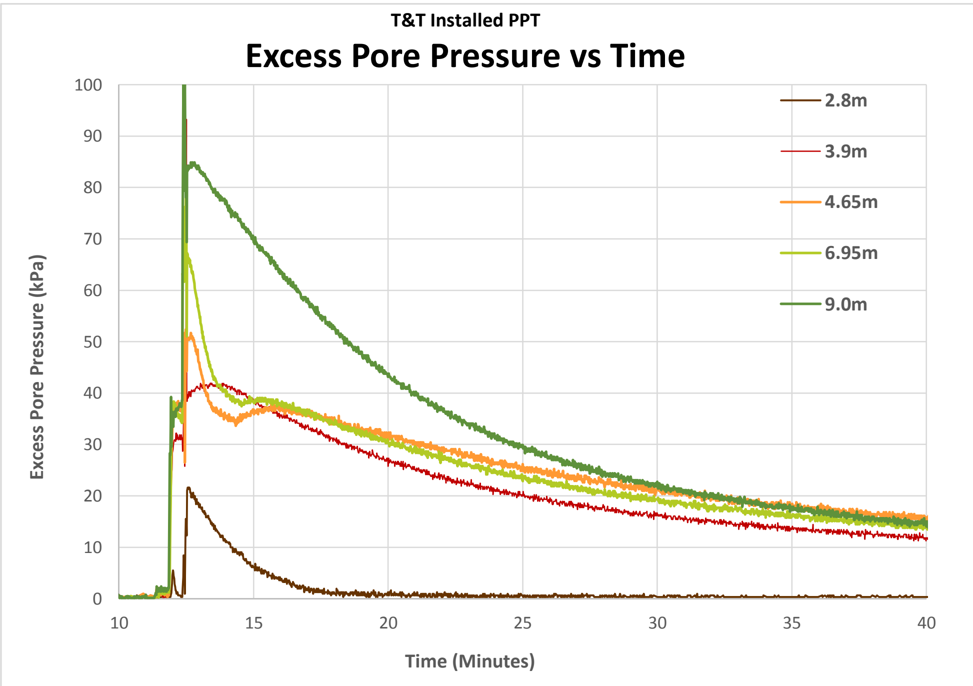
$u_{threshold} = PPT\ threshold$

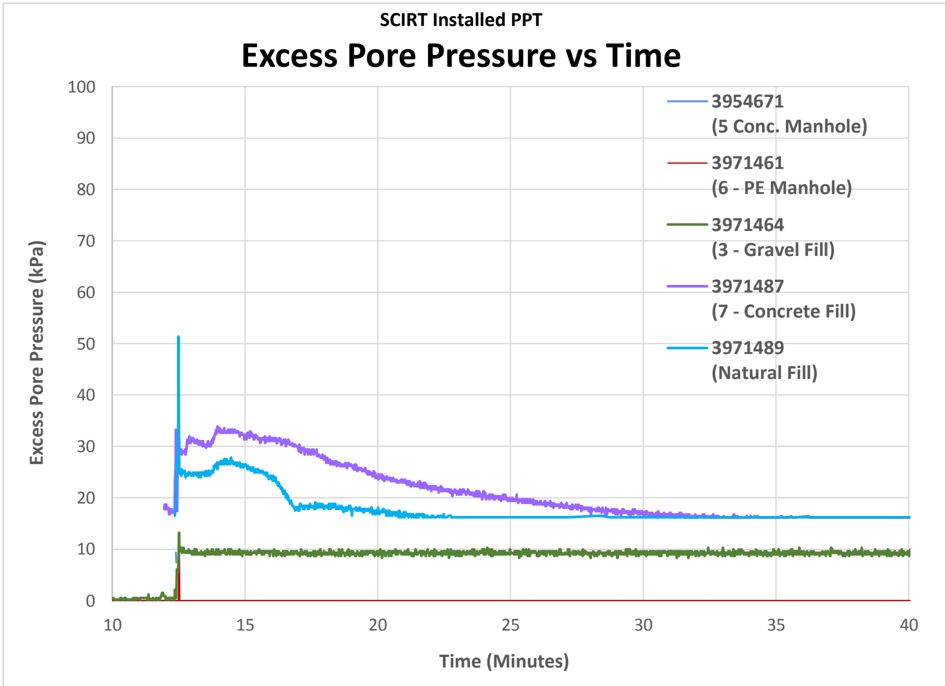
$u_{initial} = adopted\ initial\ pore\ water\ pressure$

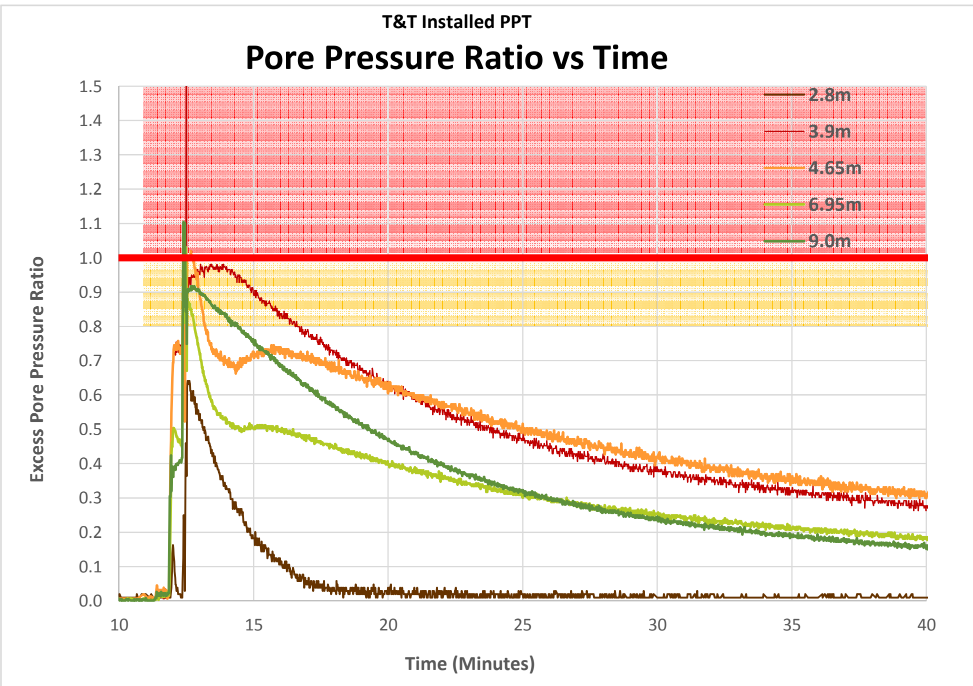
SCIRT & EQC Liquefaction Trial

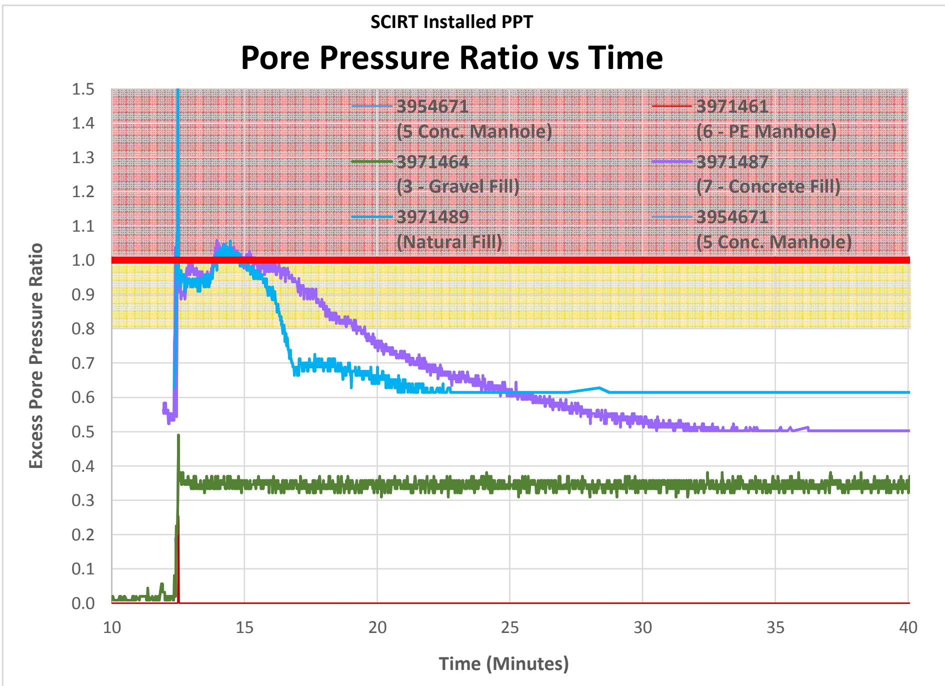
Interpretation of Pore Pressure Transducers

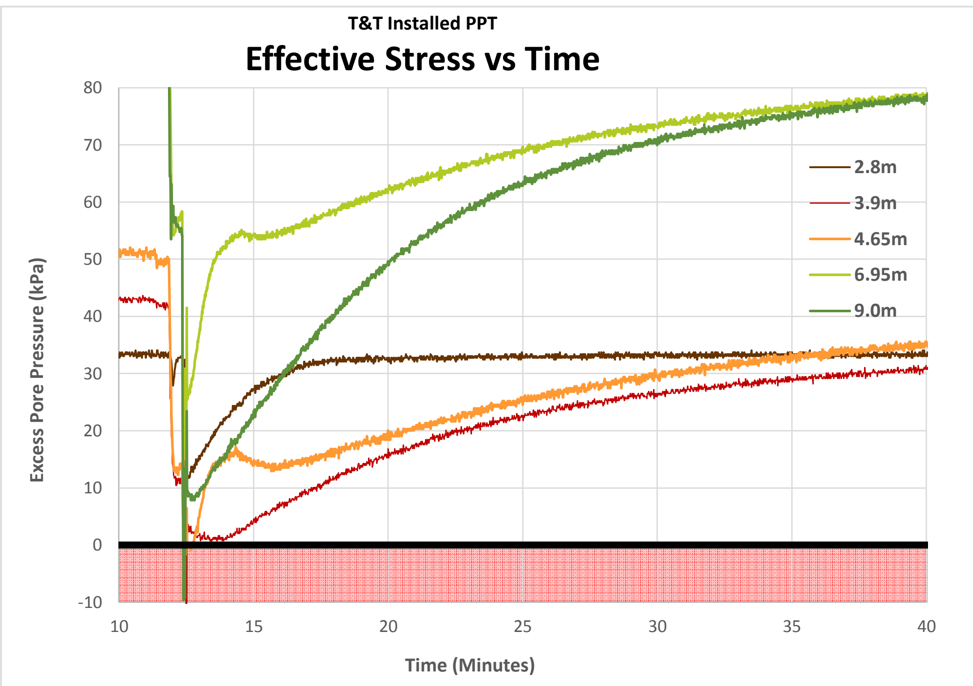
mfg
9/01/2014

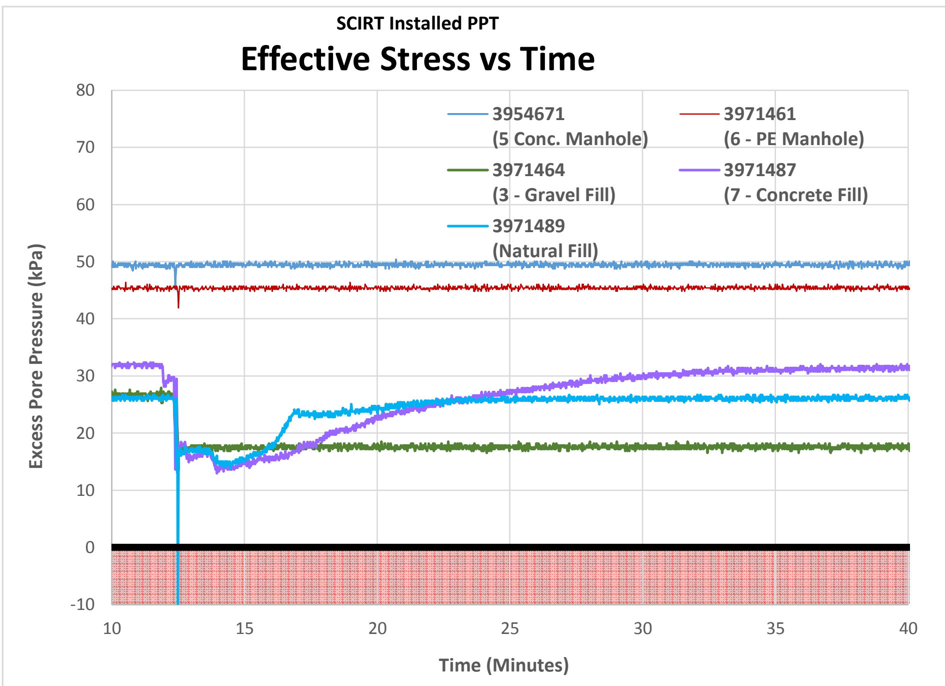










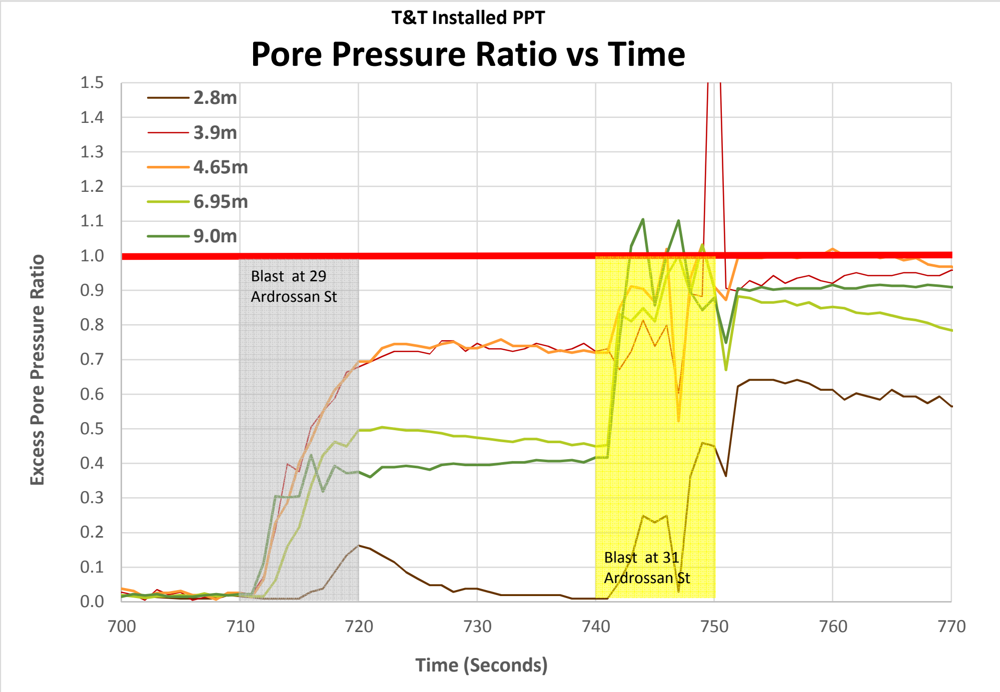


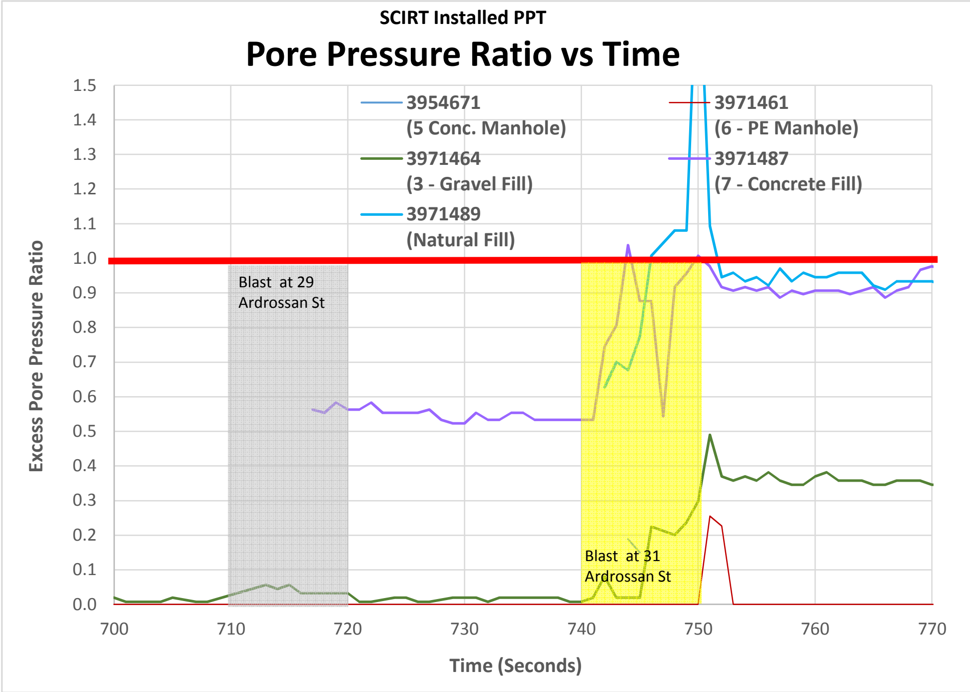
SCIRT & EQC Liquefaction Trial

Interpretation of Pore Pressure Transducers

mfg

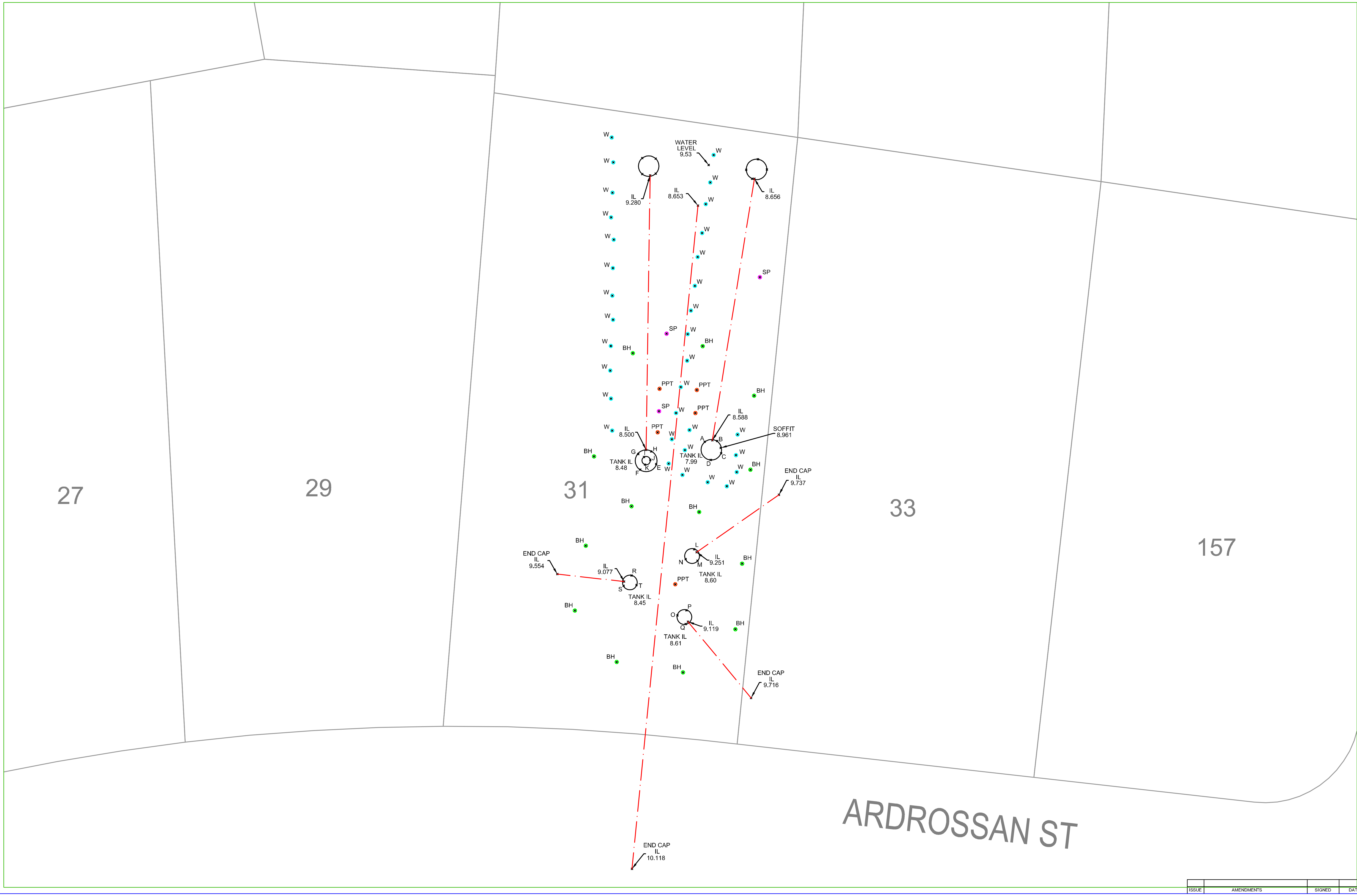
9/01/2014





G

Appendix G Survey and Settlement Data



ISSUE	AMENDMENTS	SIGNED	DATE

Christchurch City Council

DATUM	C.D.D.		NAME	SIGNED	DATE	APPROVED	
BENCH MK.		DESIGNED			**.**.2011	FOR TENDER	
RL		DES. REVIEW			**.**.2011	DATE	SIGNED
SURVEY		DRAWN				-	-
SURVEY LB		DRW. CHECK			**.**.2011	FOR CONSTRUCTION	
		FILE LOCATION	C:\Project Data\As-built Information\			DATE	SIGNED
DRAIN. REF.		Ardrossan Site Test.dwg				-	-
SAP WBS	722/**	PRINTED ON 24-Oct-13 BY OGradyP					

McCONNELL DOWELL

CONSULTANT PROJECT REF. 42178407

CONSULTANT FILE REF. ****_***_***

PROJECT TITLE

INFRASTRUCTURE REBUILD EARTHQUAKE SIMULATION TEST ARDROSSAN ST, AVONDALE

DRAWING TITLE

AS-BUILT SURVEY PRE - TEST

CONTRACT NUMBER -

ORIGINAL SHEET SIZE

SCALES 1:200 @ A3

CAD DRAWING FILE REF.

CPG PROJECT FILE NUMBER MD - AB - 0403 - 02

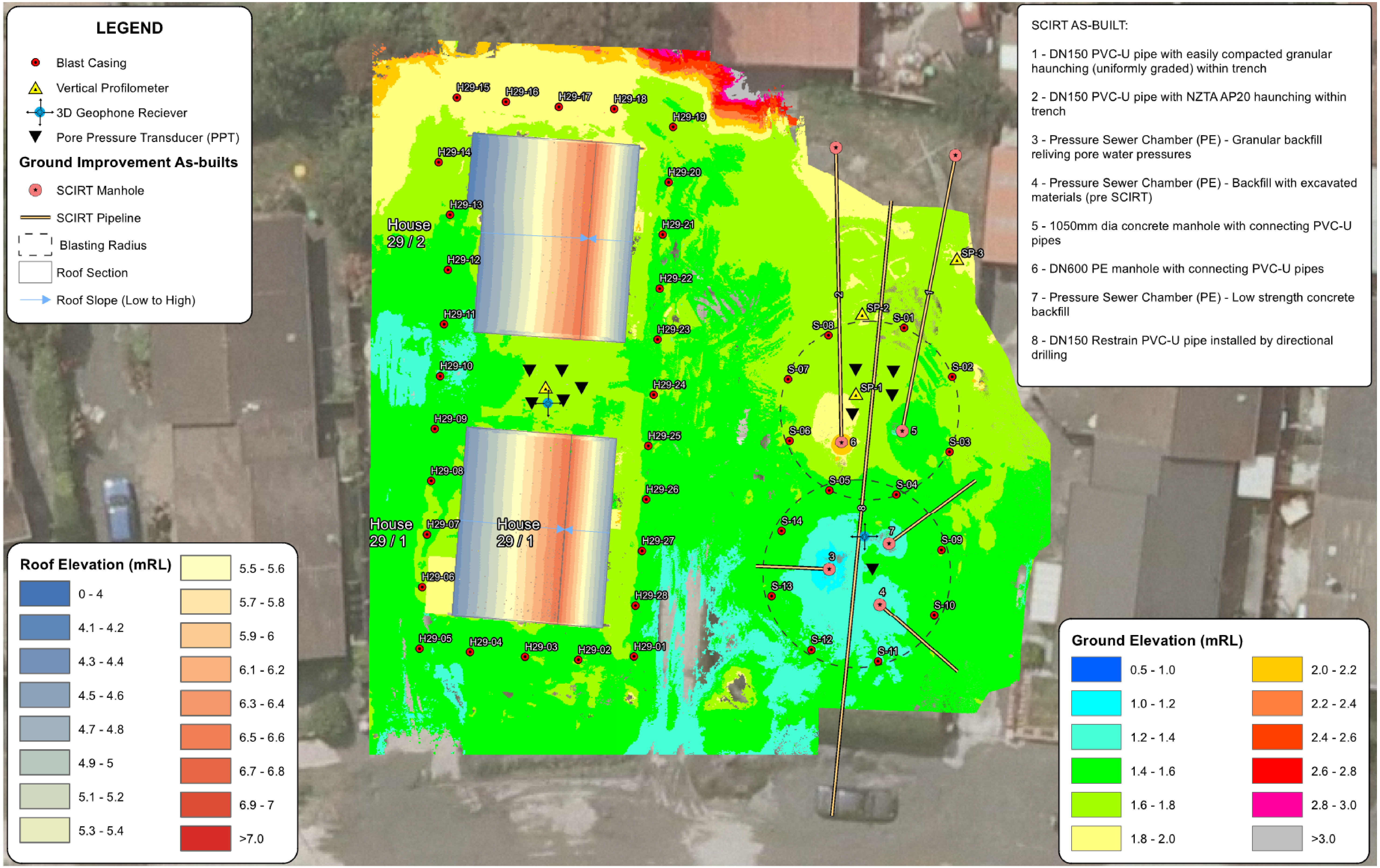
SHEET

SCIRT and EQC Liquefaction Trial

31 ARDROSSAN MONITORING POINTS

McConnell Dowell

Pre - Liquefaction				**Post Liquefaction**			**Difference**		
Pt No.	*mE*	*mN*	*RL*	*mE*	*mN*	*RL*	*mE*	*mN*	*RL*
PT A	396775.569	809912.409	10.578	396775.553	809912.387	10.480	-0.016	-0.022	-0.098
PT B	396776.285	809912.455	10.582	396776.270	809912.439	10.488	-0.015	-0.016	-0.094
PT C	396776.515	809911.740	10.581	396776.505	809911.725	10.492	-0.010	-0.015	-0.089
PT D	396775.918	809911.361	10.579	396775.911	809911.342	10.486	-0.007	-0.019	-0.093
PT E	396772.761	809911.129	11.107	396772.770	809911.104	10.954	0.009	-0.025	-0.153
PT F	396771.851	809910.806	11.124	396771.859	809910.779	10.966	0.008	-0.027	-0.158
PT G	396771.728	809911.704	11.141	396771.735	809911.676	10.984	0.007	-0.028	-0.157
PT H	396772.475	809911.850	11.122	396772.484	809911.824	10.969	0.009	-0.026	-0.153
PT I	396772.058	809911.530	10.950	396772.065	809911.503	10.798	0.007	-0.027	-0.152
PT J	396772.425	809911.367	10.941	396772.430	809911.340	10.789	0.005	-0.027	-0.152
PT K	396772.124	809911.092	10.939	396772.132	809911.067	10.785	0.008	-0.025	-0.154
PT L	396774.955	809906.210	10.409	396774.962	809906.202	10.397	0.007	-0.008	-0.012
PT M	396775.151	809905.546	10.392	396775.156	809905.535	10.384	0.005	-0.011	-0.008
PT N	396774.467	809905.640	10.397	396774.472	809905.632	10.393	0.005	-0.008	-0.004
PT O	396773.993	809902.387	10.403	396773.995	809902.386	10.405	0.002	-0.001	0.002
PT P	396774.510	809902.689	10.408	396774.511	809902.686	10.407	0.001	-0.003	-0.001
PT Q	396774.463	809901.891	10.403	396774.465	809901.889	10.401	0.002	-0.002	-0.002
PT R	396771.343	809904.689	10.238	396771.345	809904.689	10.231	0.002	0.000	-0.007
PT S	396770.889	809904.097	10.222	396770.891	809904.095	10.214	0.002	-0.002	-0.008
PT T	396771.630	809904.141	10.221	396771.631	809904.138	10.218	0.001	-0.003	-0.003



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A3 SCALE 1:200

0 2 4 6 8 10 (m)

Location Plan

NEW BRIGHTON ROAD · AVONSIDE DRIVE · AVONDALE ROAD · Burwood · Avondale

EQC EARTHQUAKE COMMISSION Kōmihana Rūwhenua

DRAWN	JJXC	Apr.14
CHECKED	PWQ	Apr.14
APPROVED	SVB	Apr.14

ARCFILE 52020-0200-TRX106

SCALE (AT A3 SIZE) 1:200

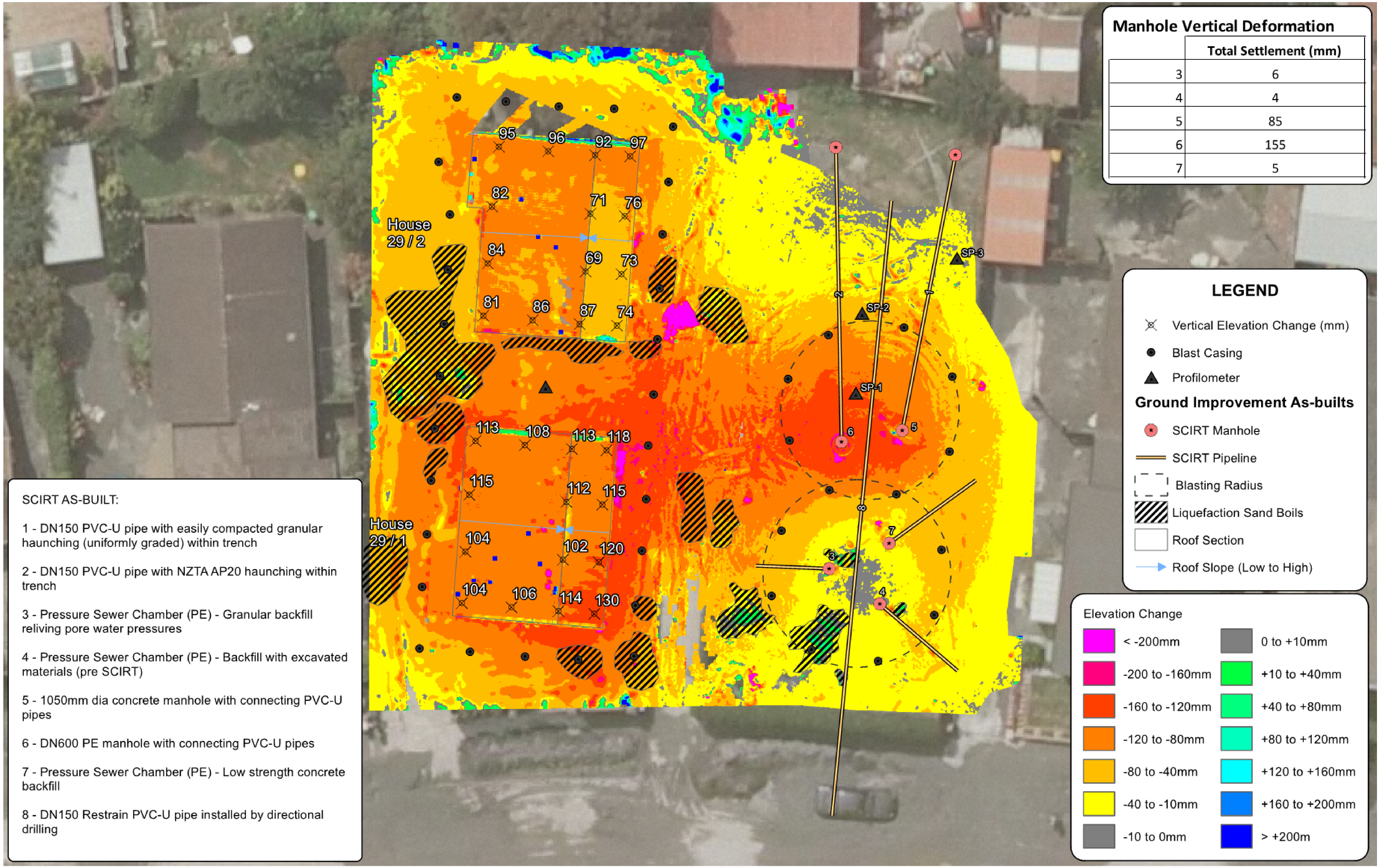
Prepared by Tonkin & Taylor Ltd. Ref. 52020-023

Ground Improvement Trials
Site 4 (Avondale)
Laser Terrestrial Scan Elevation
Production Blast 4-2 & 4-3

Figure I2.4.2

Rev. 0

Date: 15/04/2014 Time: 6:23:32 p.m.



Manhole Vertical Deformation

	Total Settlement (mm)
3	6
4	4
5	85
6	155
7	5

LEGEND

- Vertical Elevation Change (mm)
- Blast Casing
- Profilometer

Ground Improvement As-builts

- SCIRT Manhole
- SCIRT Pipeline
- Blasting Radius
- Liquefaction Sand Boils
- Roof Section
- Roof Slope (Low to High)

Elevation Change

< -200mm	0 to +10mm
-200 to -160mm	+10 to +40mm
-160 to -120mm	+40 to +80mm
-120 to -80mm	+80 to +120mm
-80 to -40mm	+120 to +160mm
-40 to -10mm	+160 to +200mm
-10 to 0mm	> +200m

SCIRT AS-BUILT:

1 - DN150 PVC-U pipe with easily compacted granular haunching (uniformly graded) within trench

2 - DN150 PVC-U pipe with NZTA AP20 haunching within trench

3 - Pressure Sewer Chamber (PE) - Granular backfill reliving pore water pressures

4 - Pressure Sewer Chamber (PE) - Backfill with excavated materials (pre SCIRT)

5 - 1050mm dia concrete manhole with connecting PVC-U pipes

6 - DN600 PE manhole with connecting PVC-U pipes

7 - Pressure Sewer Chamber (PE) - Low strength concrete backfill

8 - DN150 Restrain PVC-U pipe installed by directional drilling

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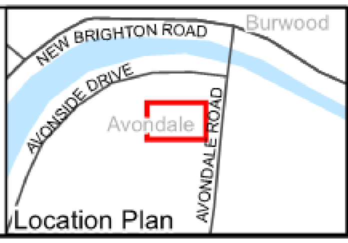
A3 SCALE 1:200

0 2 4 6 8 10 (m)



EARTHQUAKE COMMISSION
Kōmihana Rūwhenua

DRAWN	JJXC	Sep.14
CHECKED	PWQ	Sep.14
APPROVED	SVB	Sep.14

ARCFILE 52020-0200-TRX107

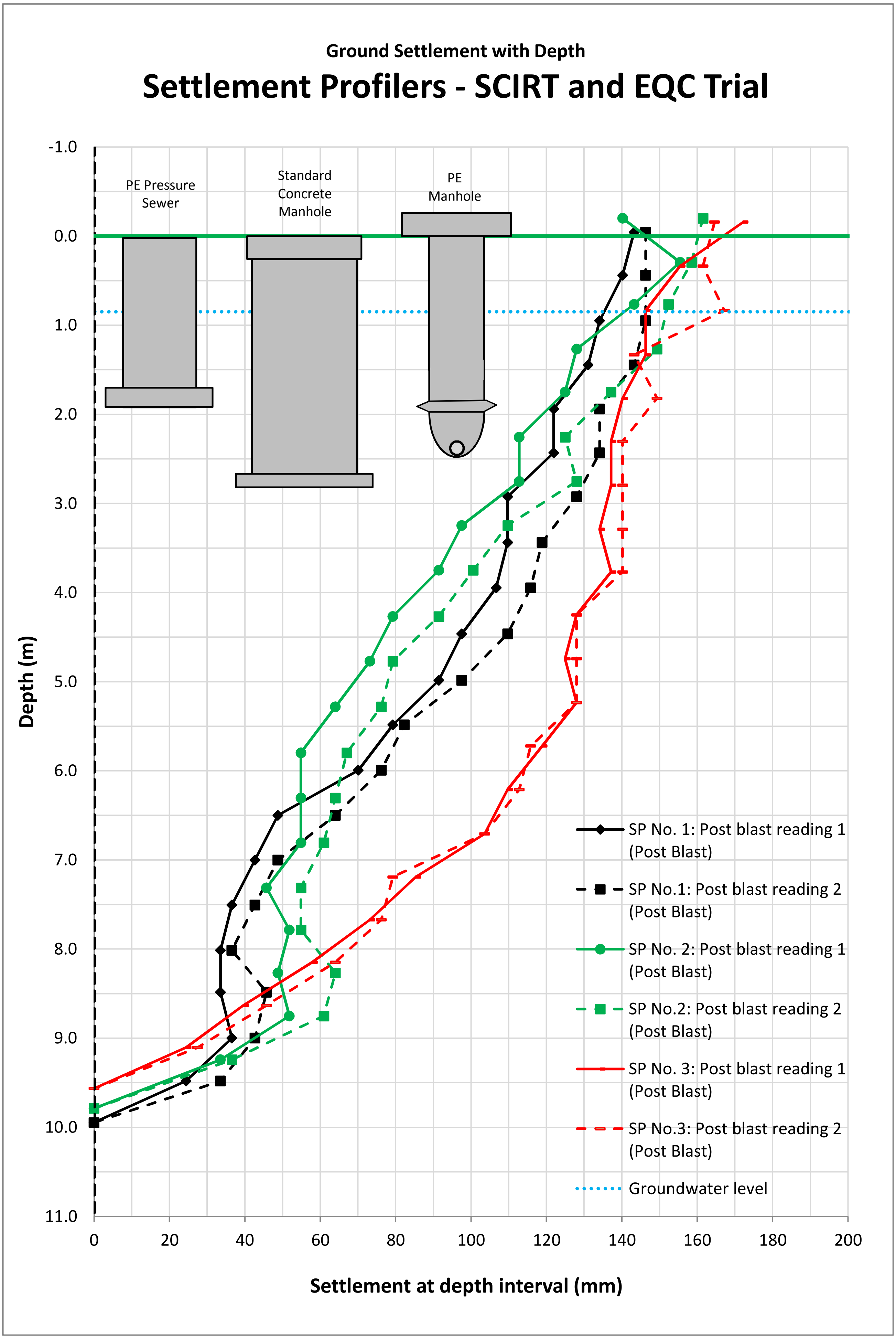
SCALE (AT A3 SIZE) 1:200

Prepared by Tonkin & Taylor Ltd.
Ref: 52020-023

Ground Improvement Trials
Site 4 (Avondale) 29 Ardrossan Street
Laser Terrestrial Scan Elevation Change
Production Blast 4-2 & 4-3 (Roof and Land)

Rev. 0

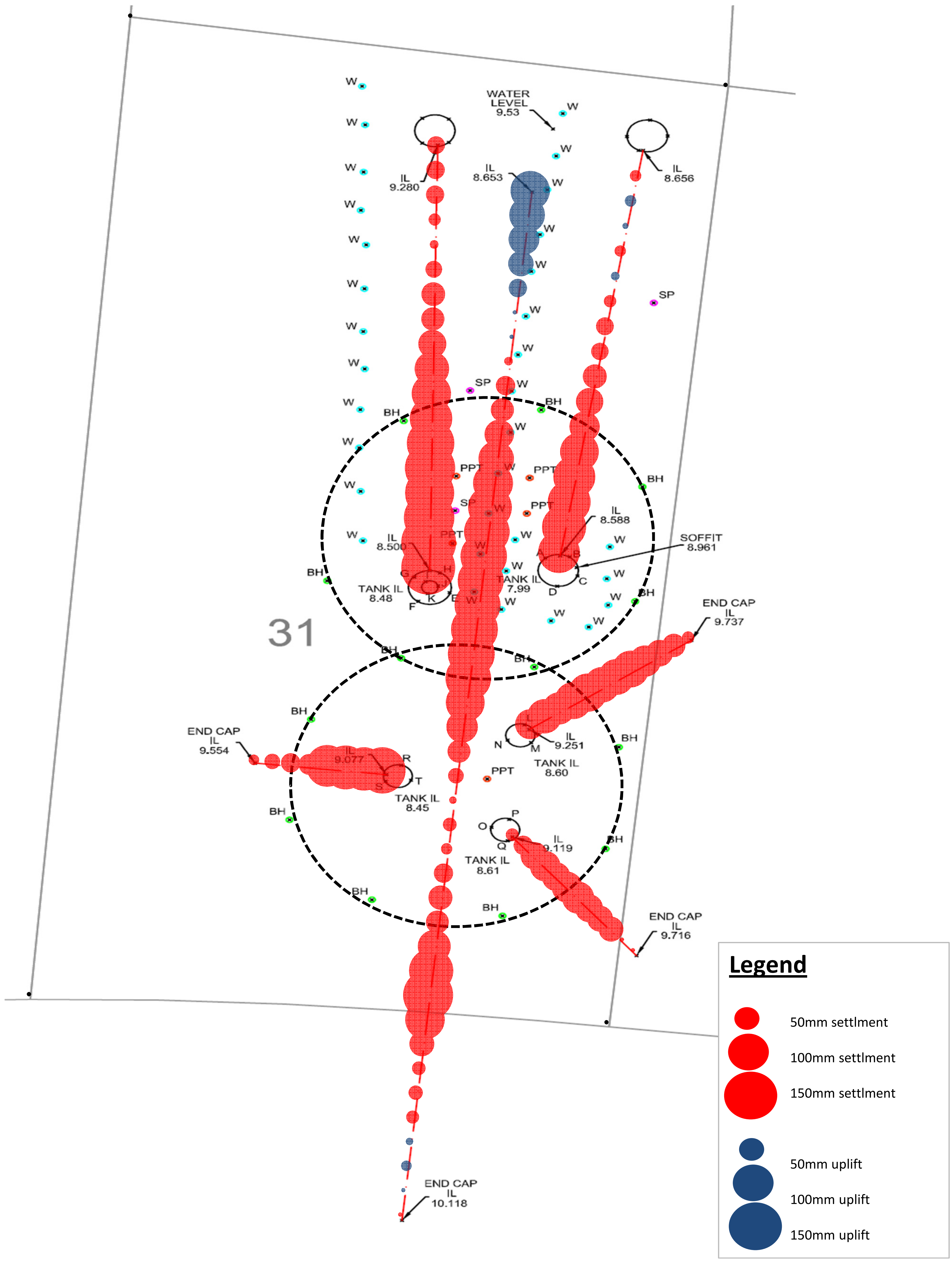
Date: 4/09/2014 Time: 12:26:47 p.m.


Ground Settlement with Depth
Settlement Profilers - SCIRT and EQC Trial
PE Pressure Sewer
Standard Concrete Manhole
PE Manhole
Depth (m)
-1.0
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
0
20
40
60
80
100
120
140
160
180
200
Settlement at depth interval (mm)
SP No. 1: Post blast reading 1 (Post Blast)
SP No.1: Post blast reading 2 (Post Blast)
SP No. 2: Post blast reading 1 (Post Blast)
SP No.2: Post blast reading 2 (Post Blast)
SP No. 3: Post blast reading 1 (Post Blast)
SP No.3: Post blast reading 2 (Post Blast)
Groundwater level

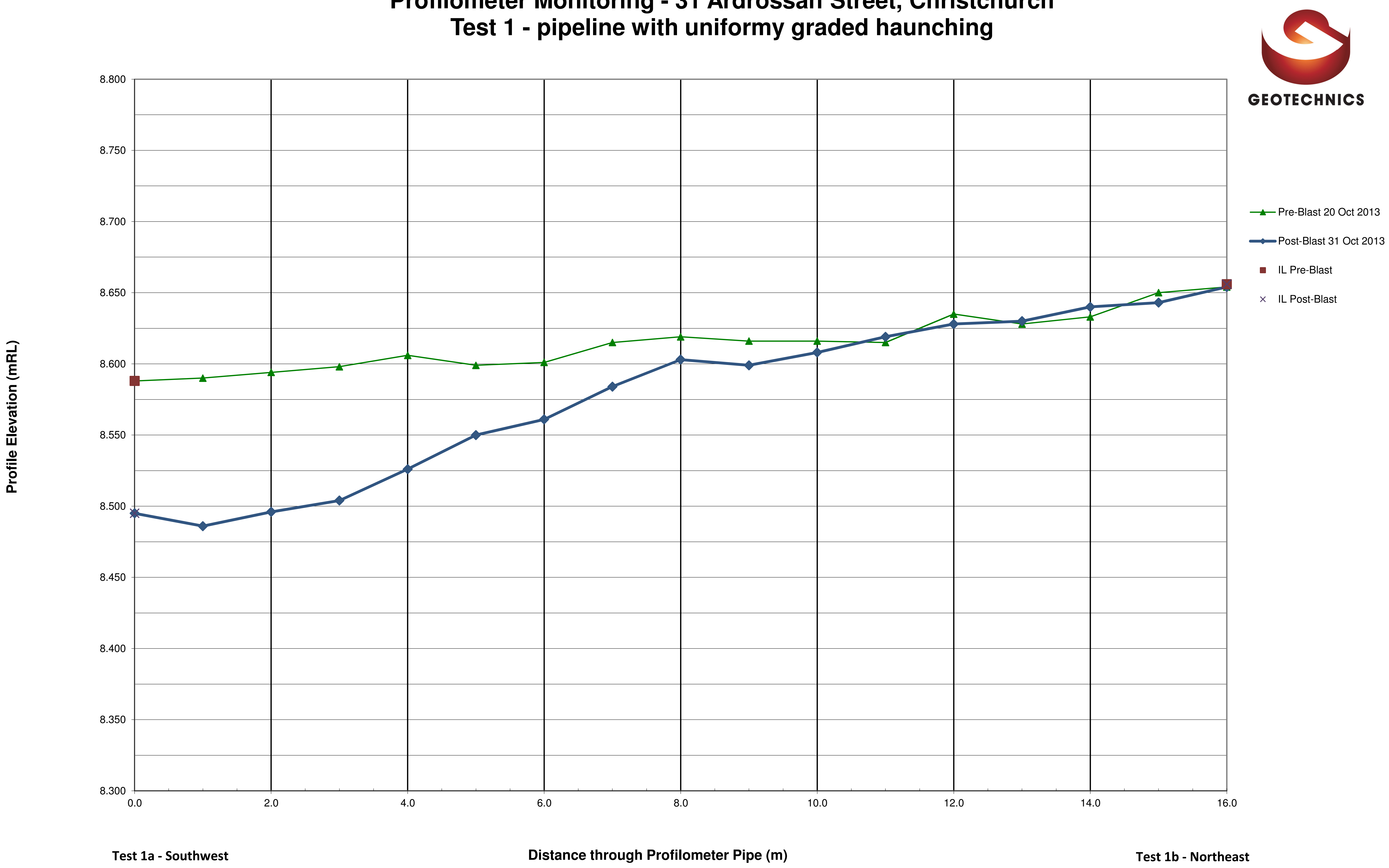
SCIRT Liquefaction Trial

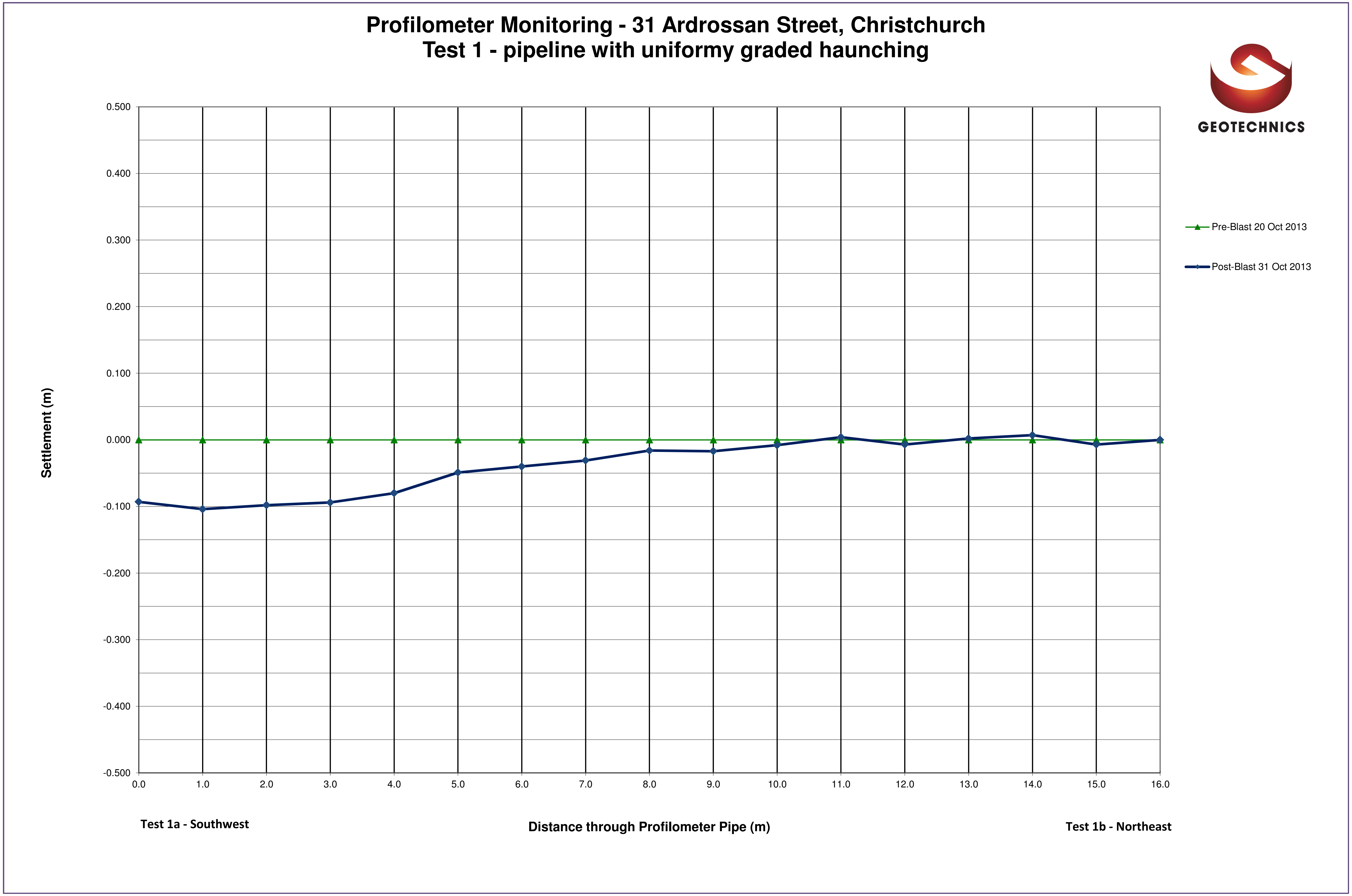
Spacial seismic settlment of pipes within SCIRT test area

mfg 10/01/2014



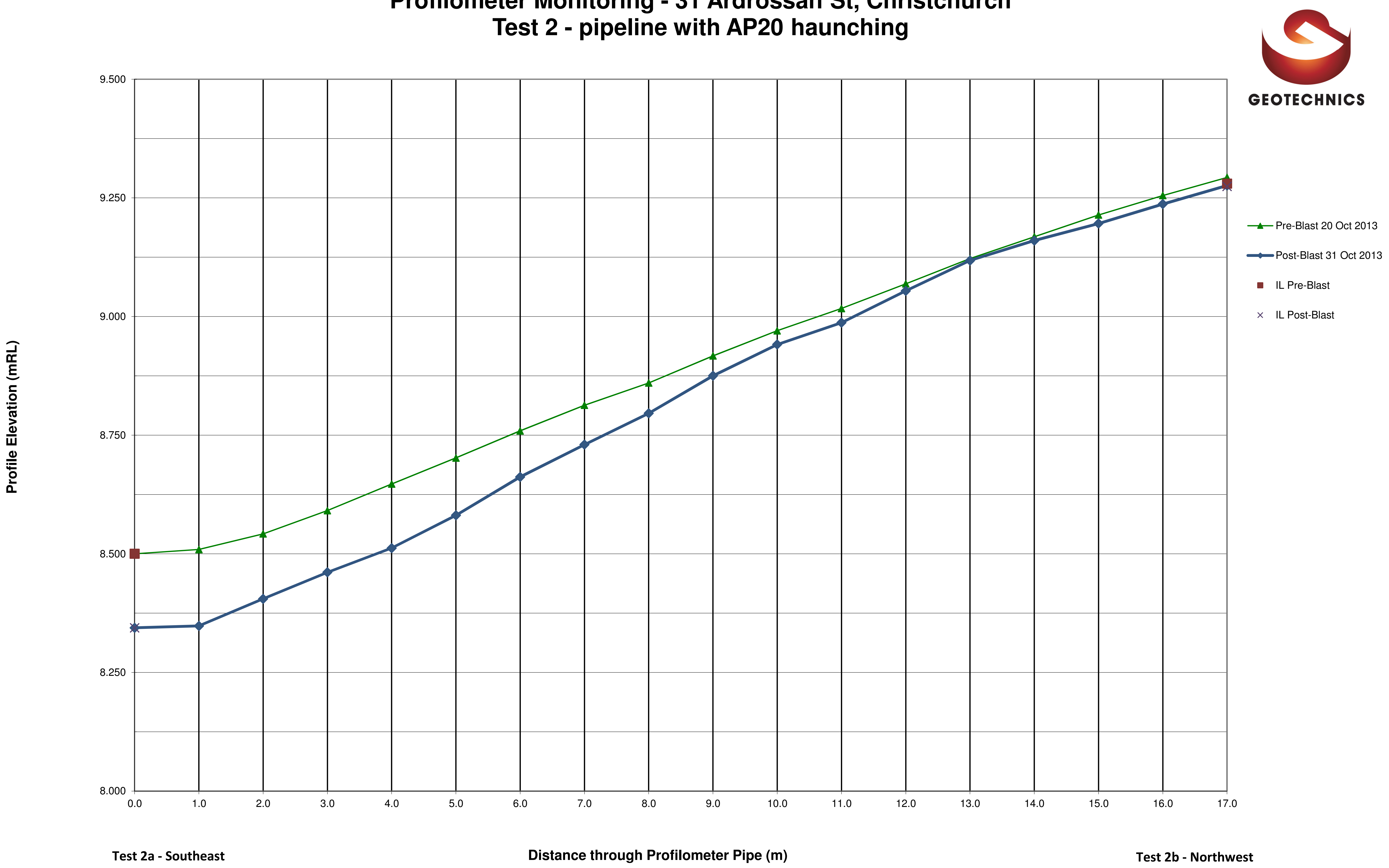
Profilometer Monitoring - 31 Ardrossan Street, Christchurch
Test 1 - pipeline with uniformy graded haunching




Profilometer Monitoring - 31 Ardrossan Street, Christchurch
Test 1 - pipeline with uniformy graded haunching
GEOTECHNICS
Settlement (m)
0.500
0.400
0.300
0.200
0.100
0.000
-0.100
-0.200
-0.300
-0.400
-0.500
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
Pre-Blast 20 Oct 2013
Post-Blast 31 Oct 2013
Test 1a - Southwest
Distance through Profilometer Pipe (m)
Test 1b - Northeast

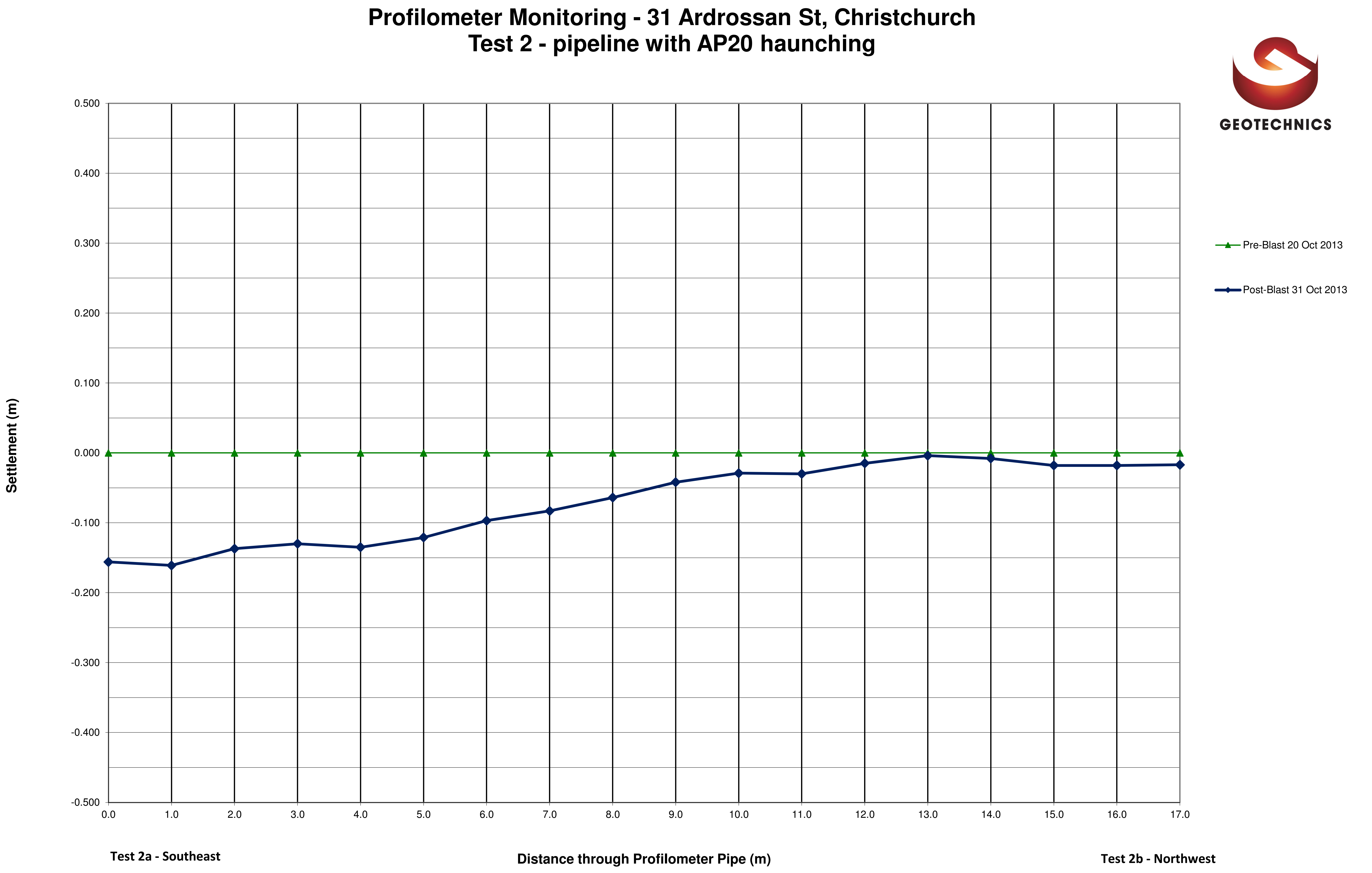
Geotechnics Ltd

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 2 - pipeline with AP20 haunching

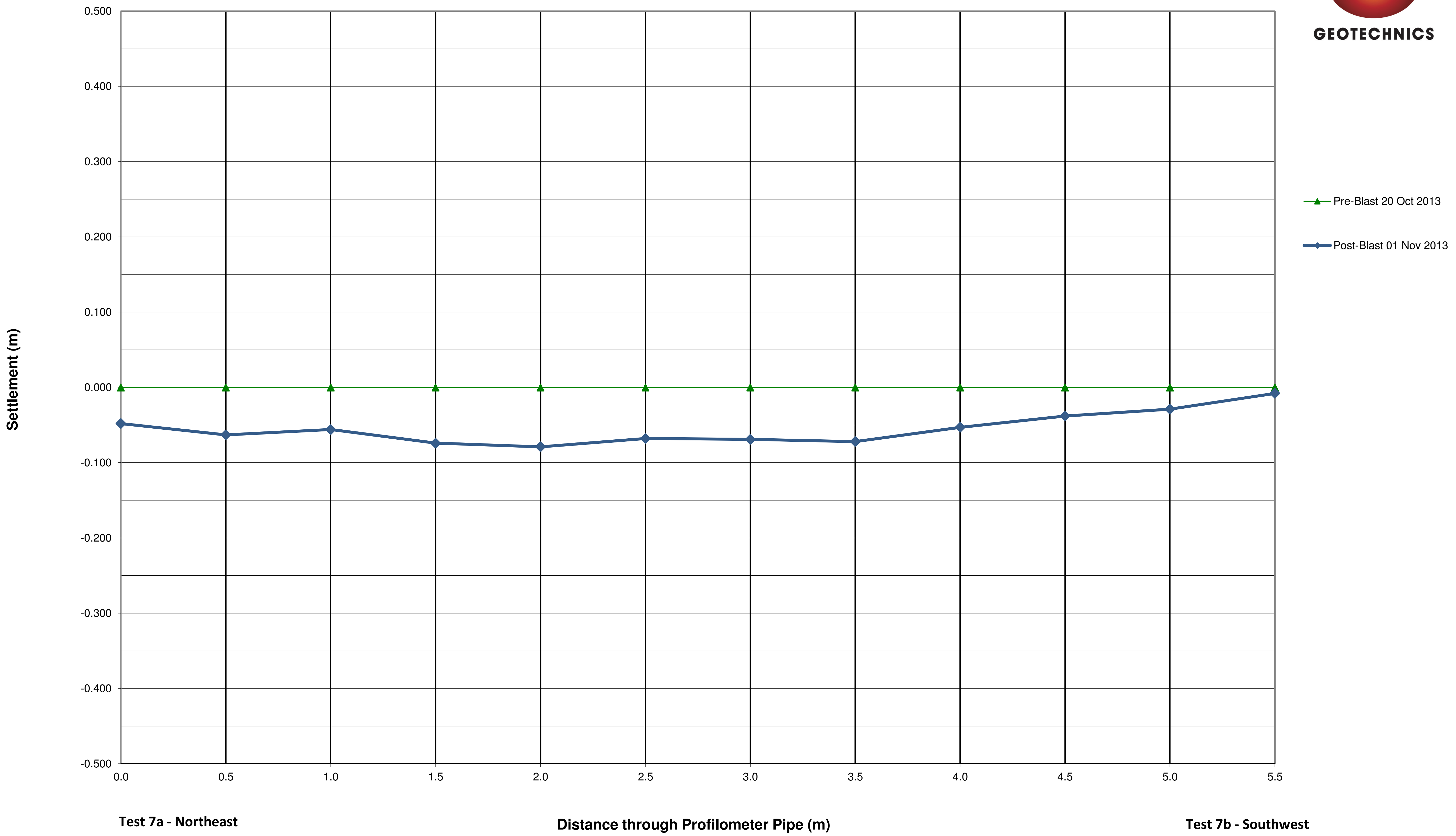


Job number: 720313.000

H:\Red\SCIRT\Geotechnical\Explosive induced liquefaction trial\08 SCIRT Liquefaction Trial - Data (raw)\04 Geotechnics - Profilometer\2 Post-blast\Test_2

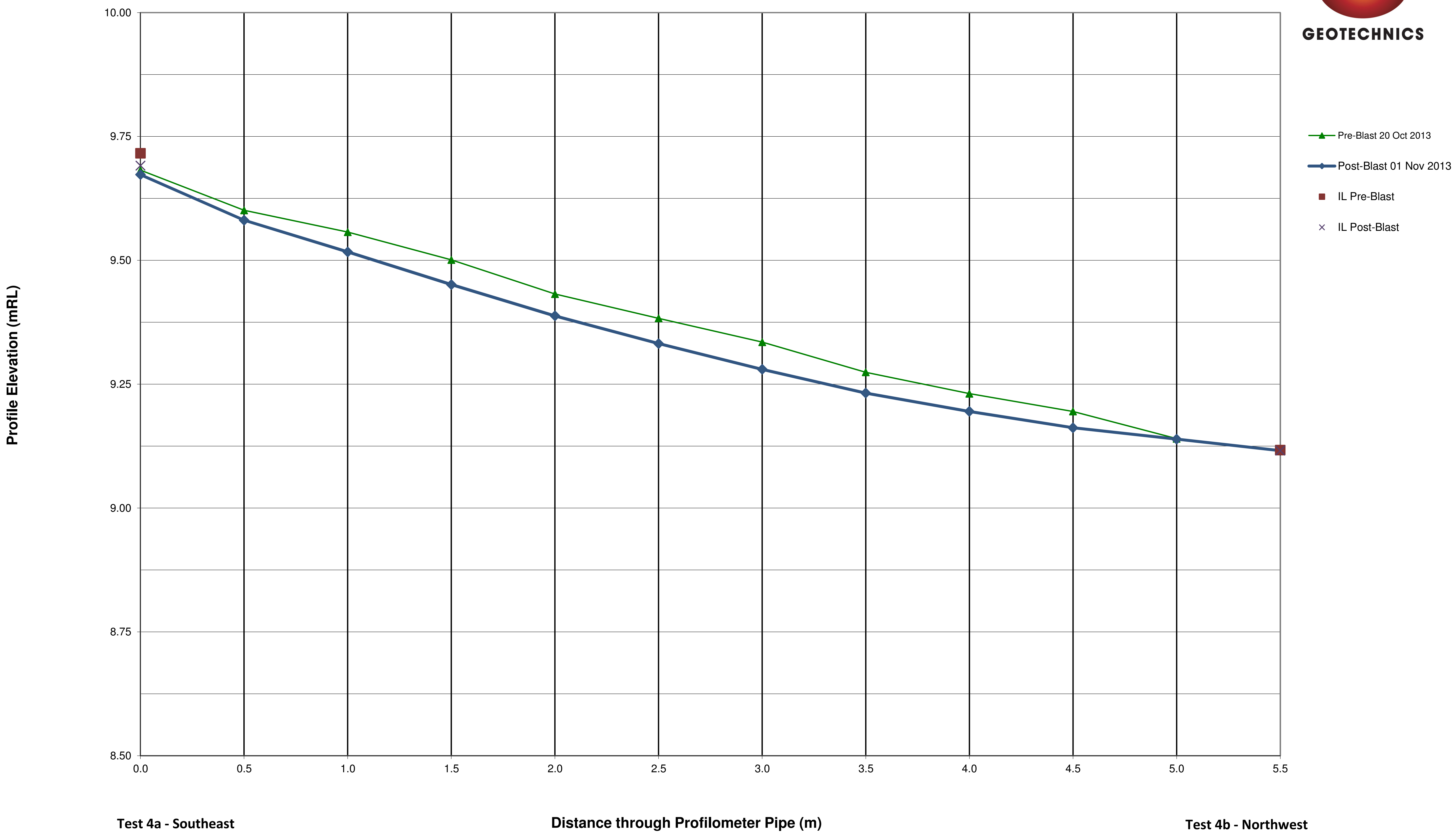

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 2 - pipeline with AP20 haunching
GEOTECHNICS
Settlement (m)
0.500
0.400
0.300
0.200
0.100
0.000
-0.100
-0.200
-0.300
-0.400
-0.500
0.0
1.0
2.0
3.0
4.0
5.0
6.0
7.0
8.0
9.0
10.0
11.0
12.0
13.0
14.0
15.0
16.0
17.0
Pre-Blast 20 Oct 2013
Post-Blast 31 Oct 2013
Test 2a - Southeast
Distance through Profilometer Pipe (m)
Test 2b - Northwest

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 3 - pipe lateral connecting to pressure sewer chamber with permeable backfill

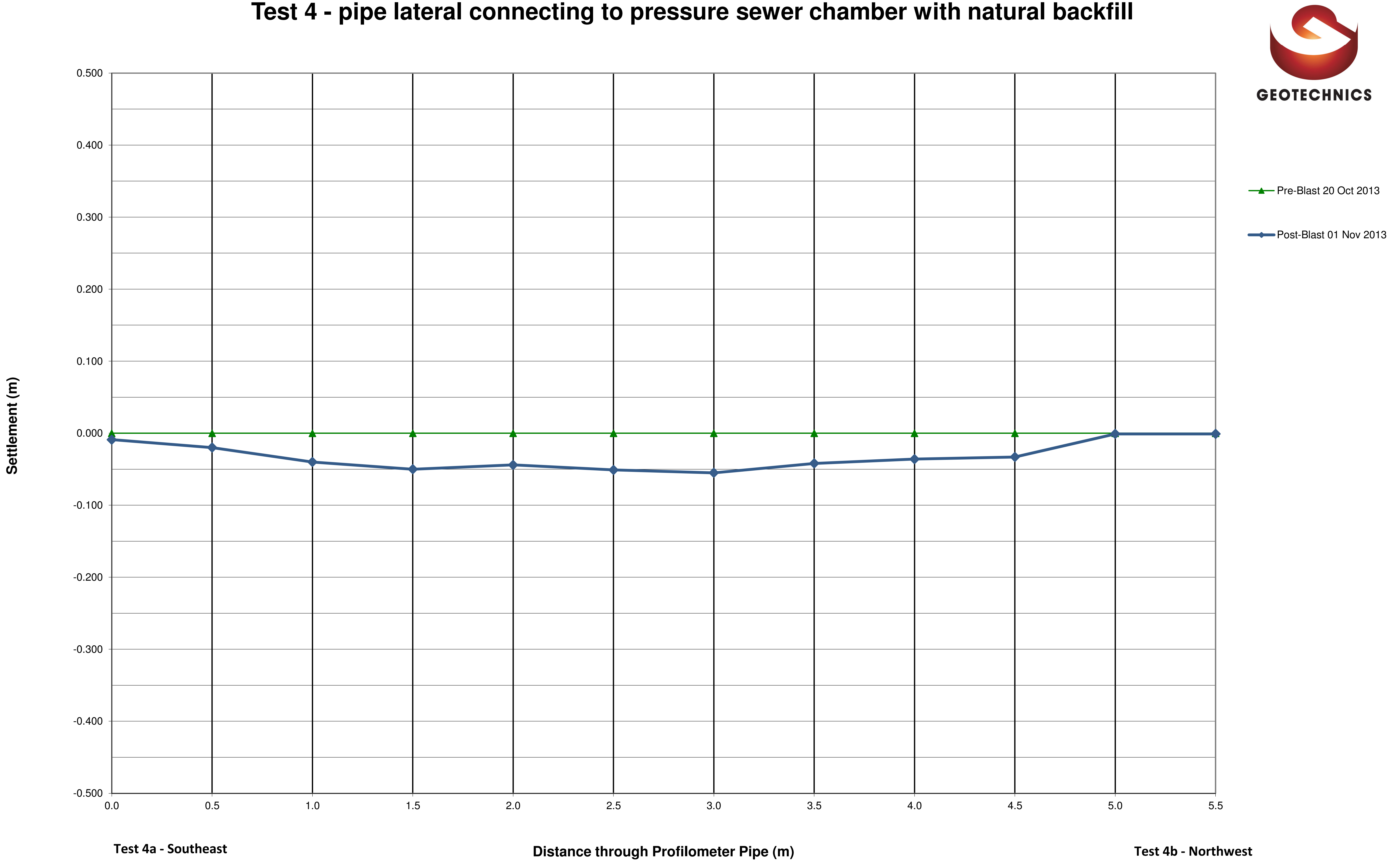


Job number: 720313.000

Profilometer Monitoring - 31 Ardrossan St, Christchruch
Test 4 - pipe lateral connecting to pressure sewer chamber with natural backfill

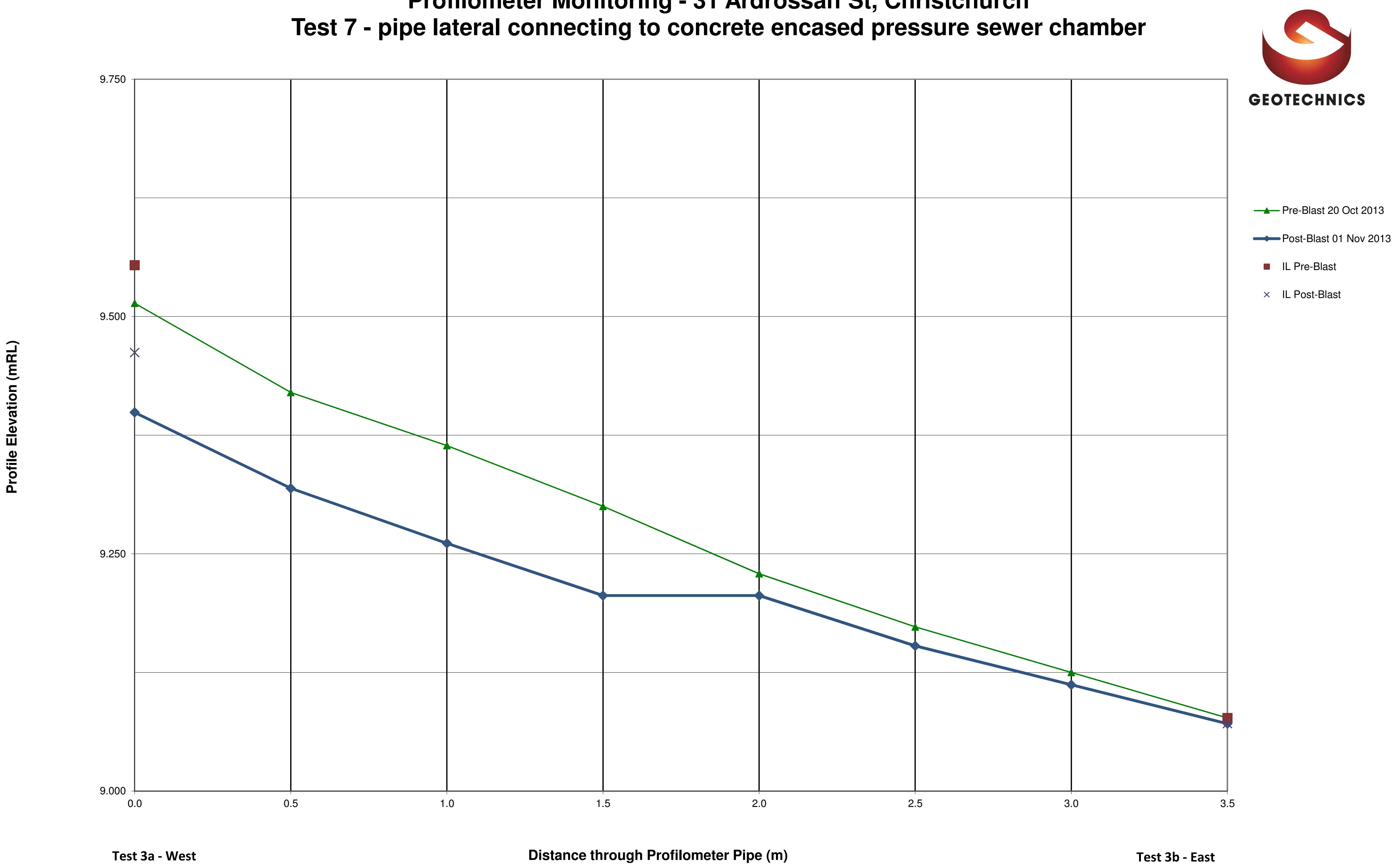


Profilometer Monitoring - 31 Ardrossan St, Christchruch
Test 4 - pipe lateral connecting to pressure sewer chamber with natural backfill

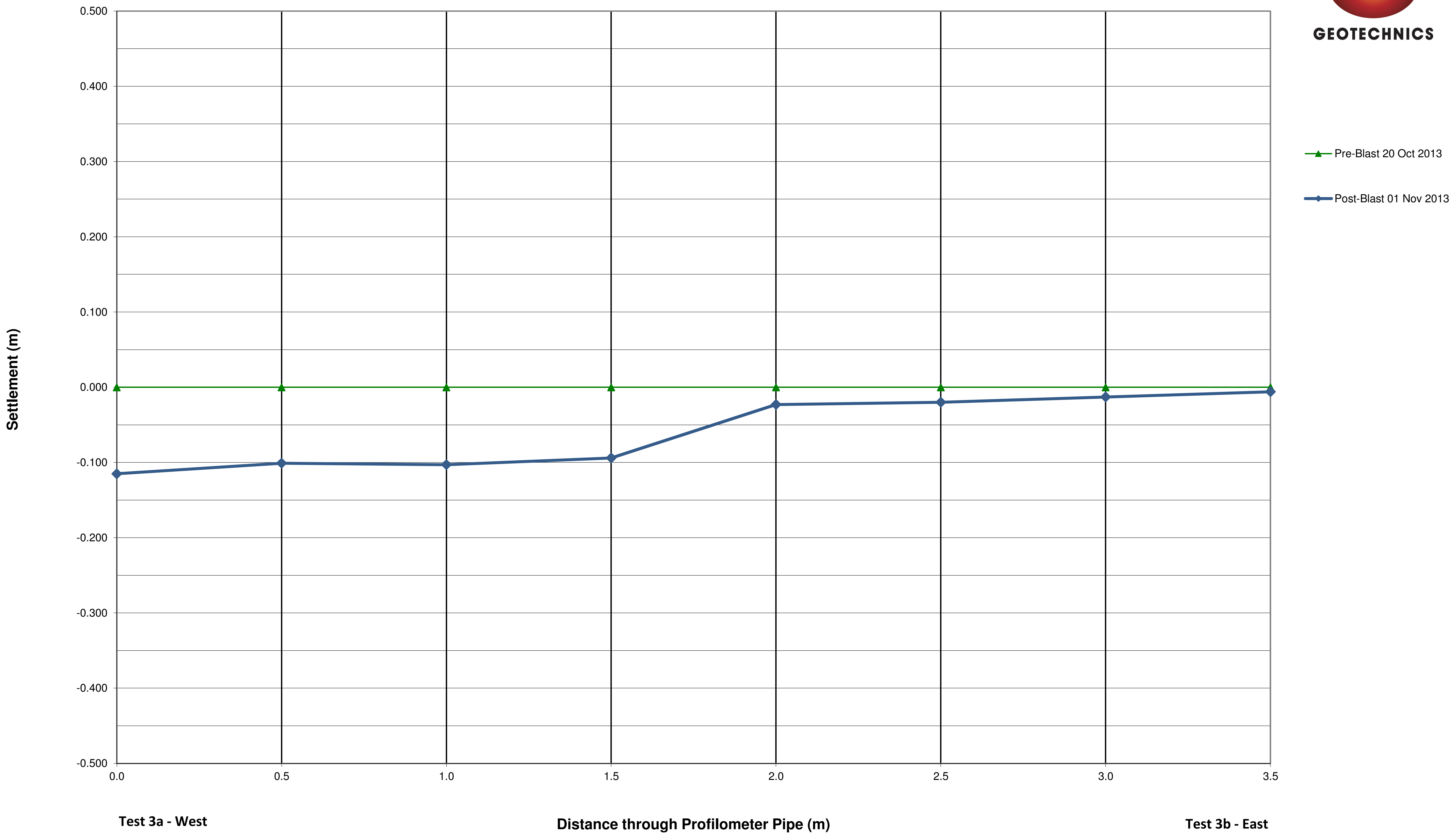


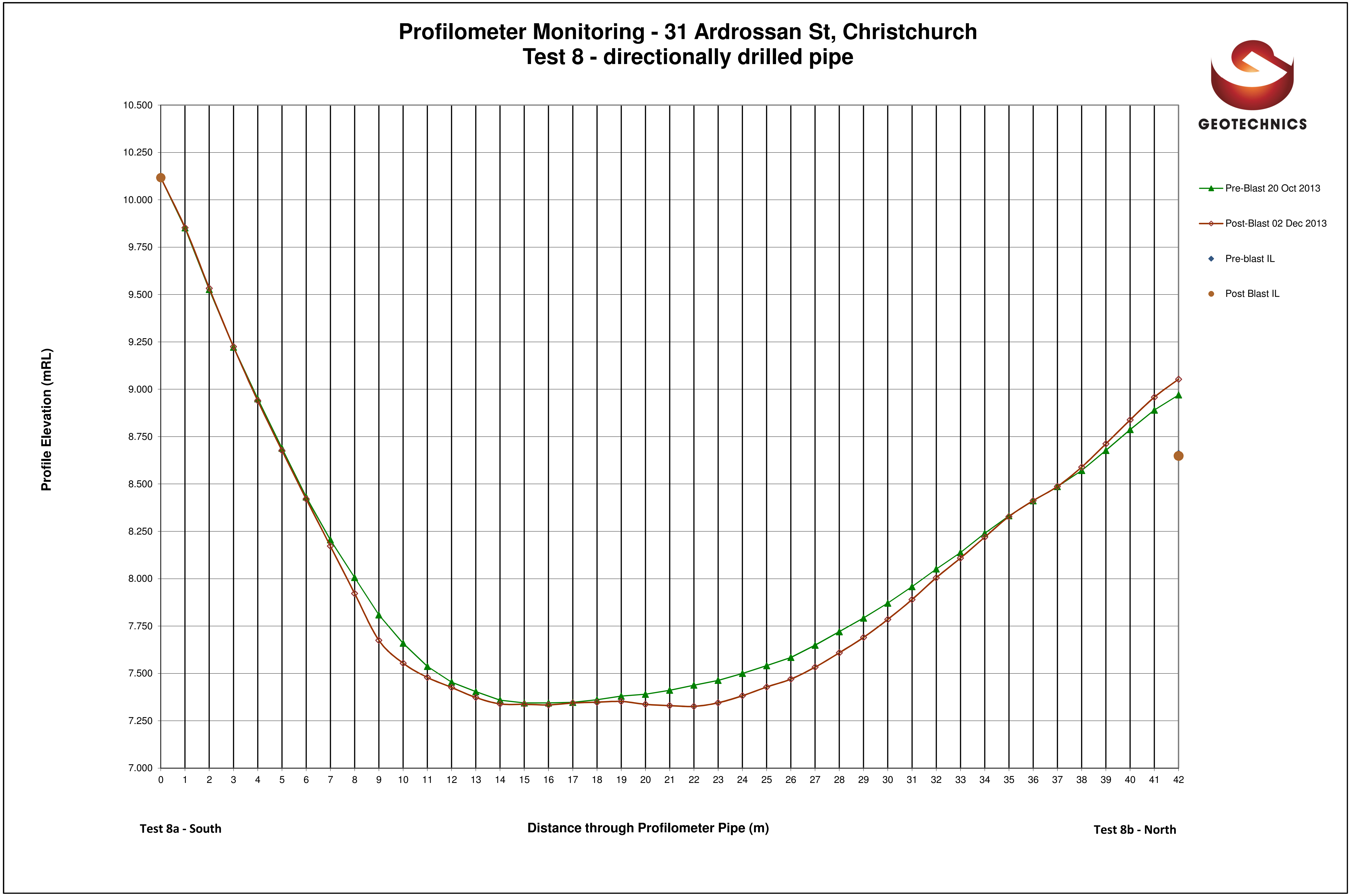
Job number: 720313.000

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 7 - pipe lateral connecting to concrete encased pressure sewer chamber

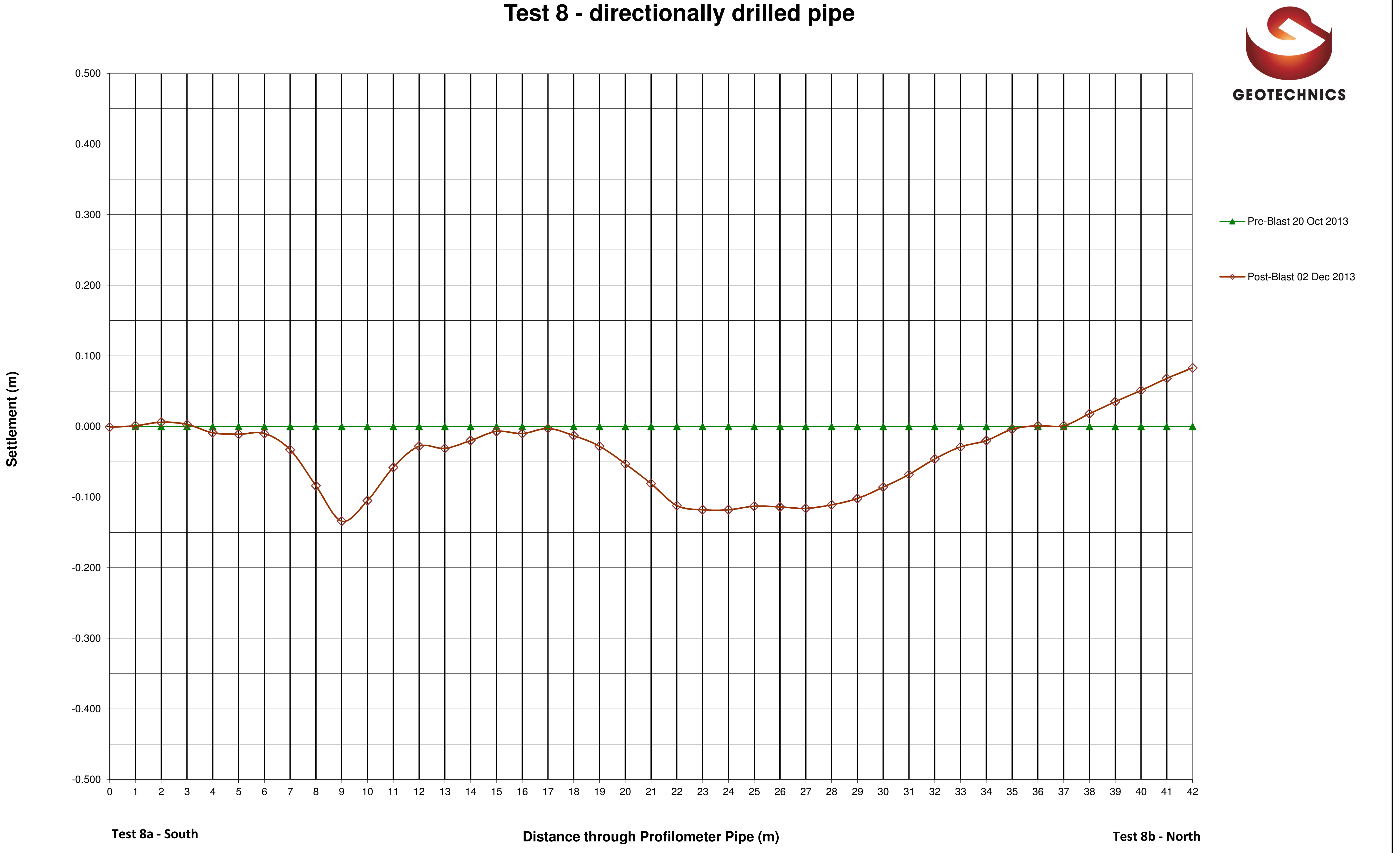


Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 7 - pipe lateral connecting to concrete encased pressure sewer chamber



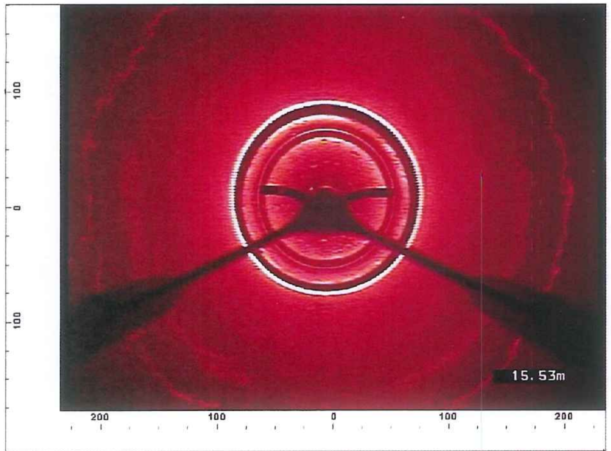

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 8 - directionally drilled pipe
GEOTECHNICS
Pre-Blast 20 Oct 2013
Post-Blast 02 Dec 2013
Pre-blast IL
Post Blast IL
Profile Elevation (mRL)
10.500
10.250
10.000
9.750
9.500
9.250
9.000
8.750
8.500
8.250
8.000
7.750
7.500
7.250
7.000
0 1 2 3 4 5 6 7 8 9 10 11 12 13 14 15 16 17 18 19 20 21 22 23 24 25 26 27 28 29 30 31 32 33 34 35 36 37 38 39 40 41 42
Test 8a - South
Distance through Profilometer Pipe (m)
Test 8b - North

Profilometer Monitoring - 31 Ardrossan St, Christchurch
Test 8 - directionally drilled pipe

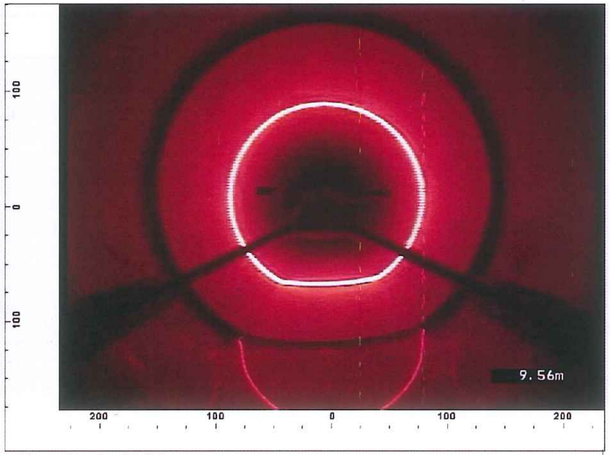


Ovality Observations Report

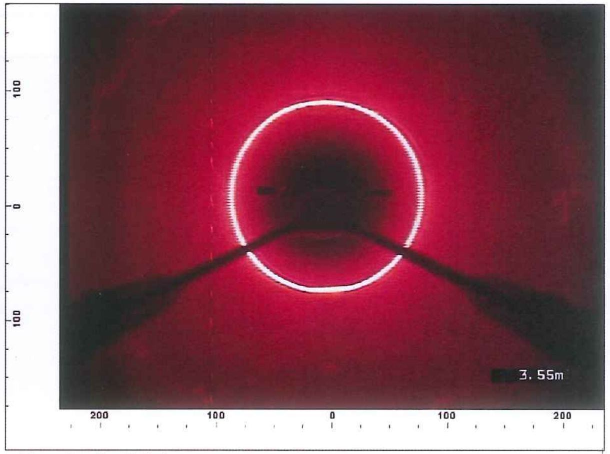
Site ID		Asset No.	Test 1	Date	17/10/2013
City				Material	PVC
Start No	MH1.1	Finish No	MH1.2	Pipeline Length	15.6 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm



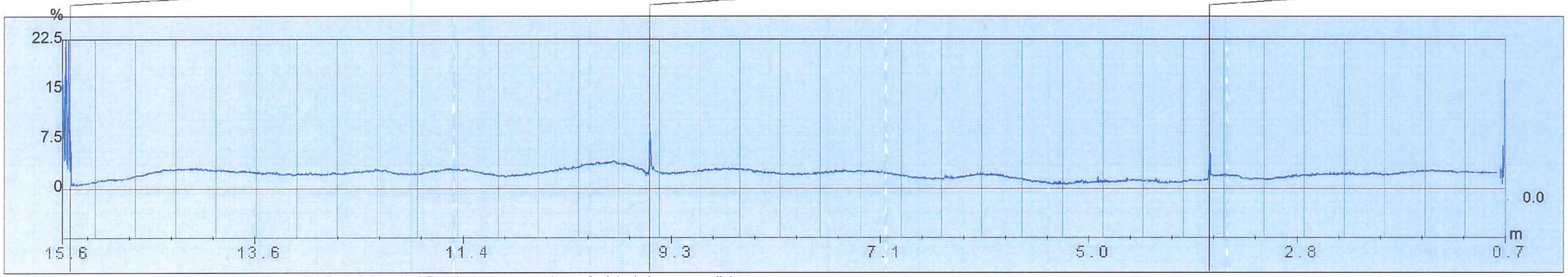
Abnormal readings due to plug at start of inspection.



Non standard joint.



Non standard joint.



90% - Fractile: 2.9%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Hydrotech Drainage
Ph: 03 366 1568

www.cleanflowsystems.com

Printed: 11/12/2013

HydroTech

Ovality Summary Report

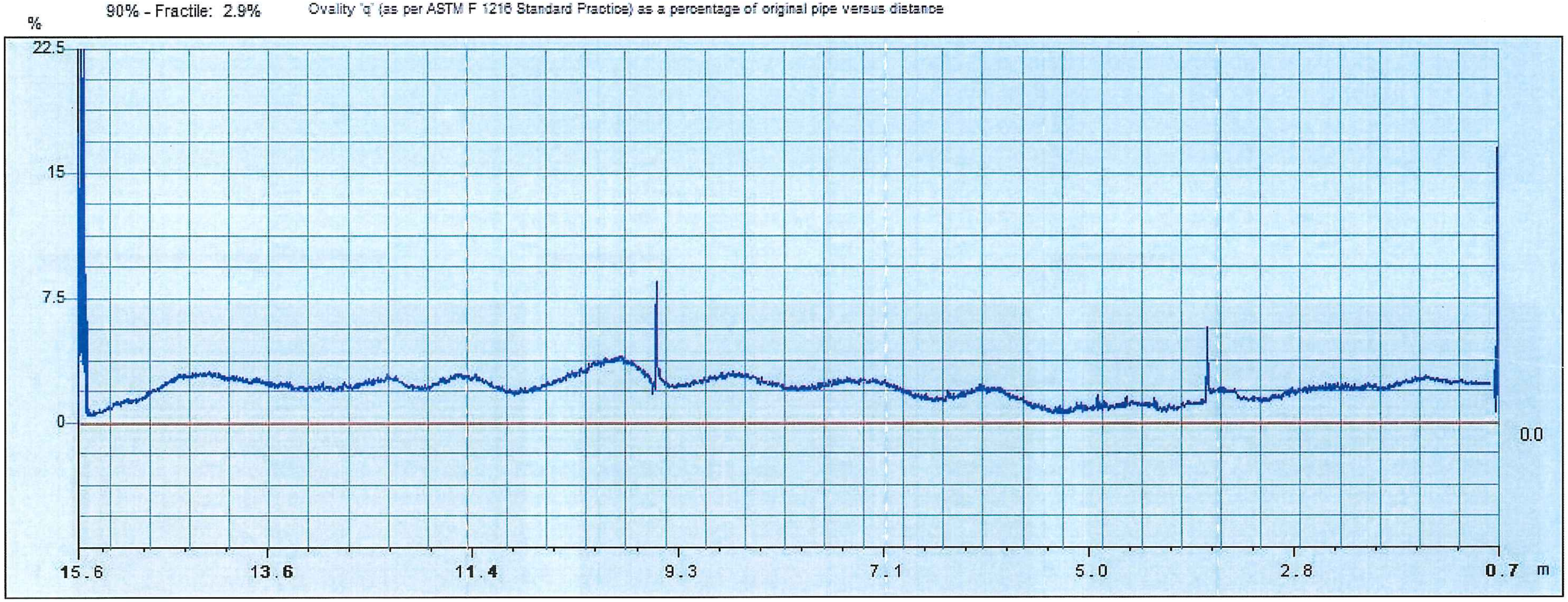
Site ID		Asset No.	Test 1	Date	17/10/2013
City				Material	PVC
Start No	MH1.1	Finish No	MH1.2	Pipeline Length	15.6 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

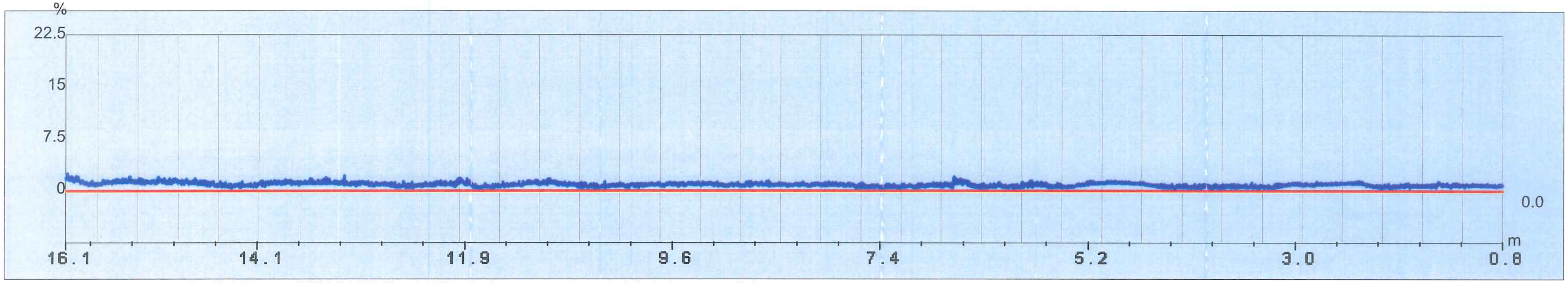
90% - Fractile: 2.9%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



Ovality Observations Report

Site ID		Asset No.	Test 2	Date	19/10/2013
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.11 m
Location	Ardrossan St, Avondale	Location	Ardtossan St, Avondale	Internal Diameter (Expected)	150 mm



90% - Fractile: 1.3%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

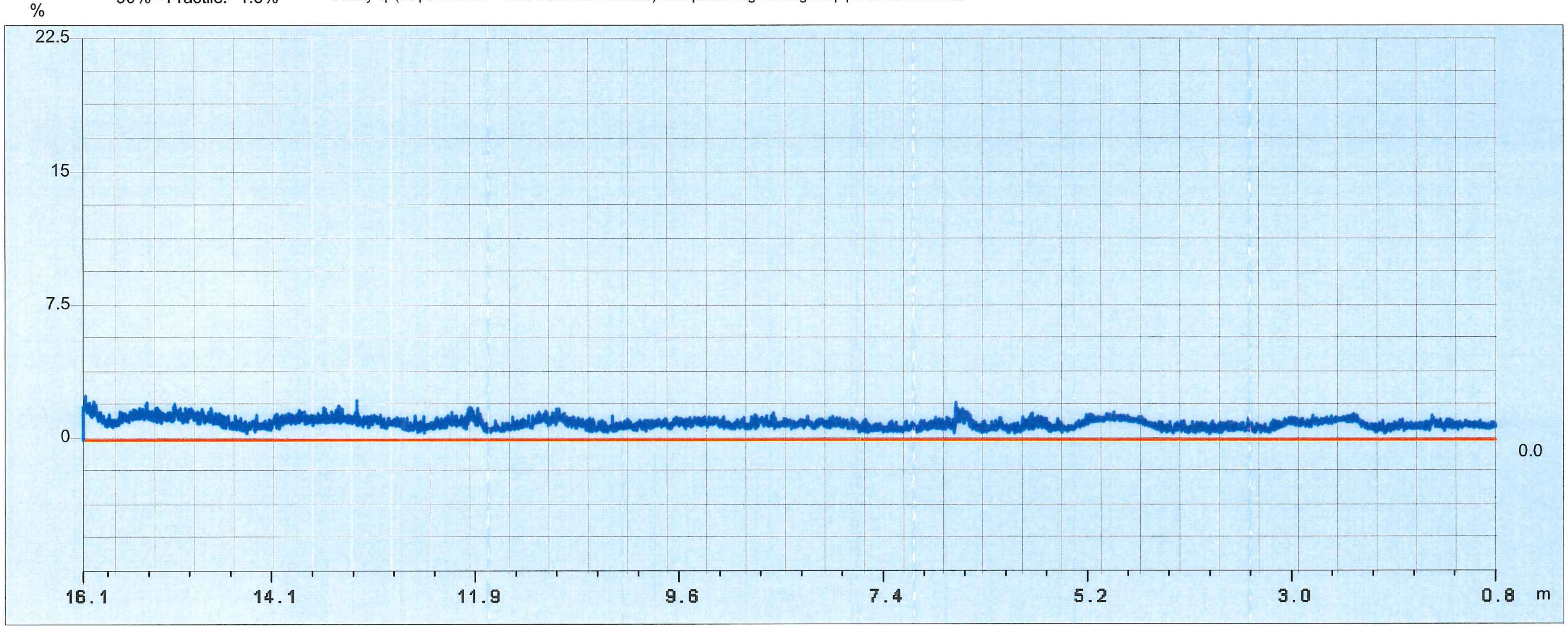
Site ID		Asset No.	Test 2	Date	19/10/2013
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.11 m
Location	Ardrossan St, Avondale	Location	Ardtossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

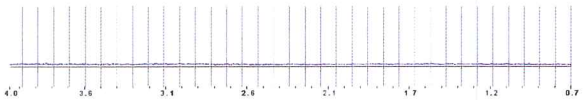
90% - Fractile: 1.3%

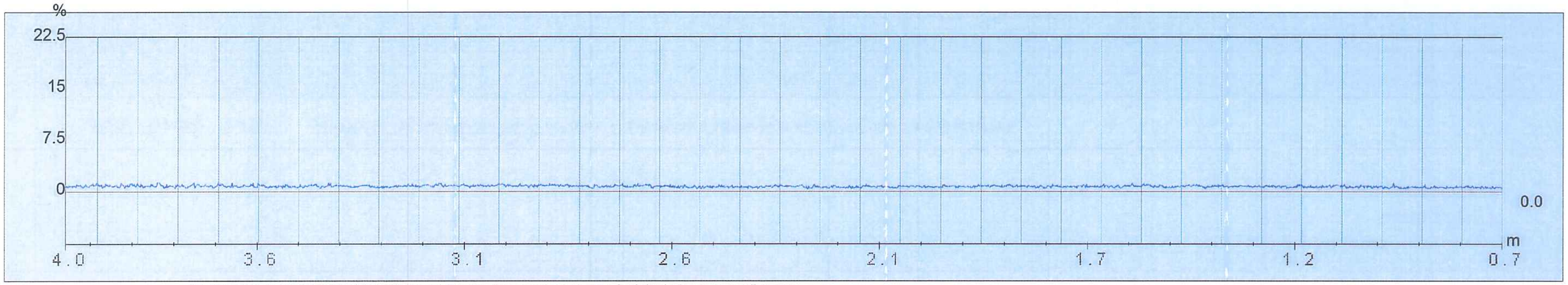
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



Ovality Observations Report

Site ID		Asset No.	Test 3	Date	17/10/2013
City				Material	PVC
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm





90% - Fractile: 1.0%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

HydroTech

Ovality Summary Report

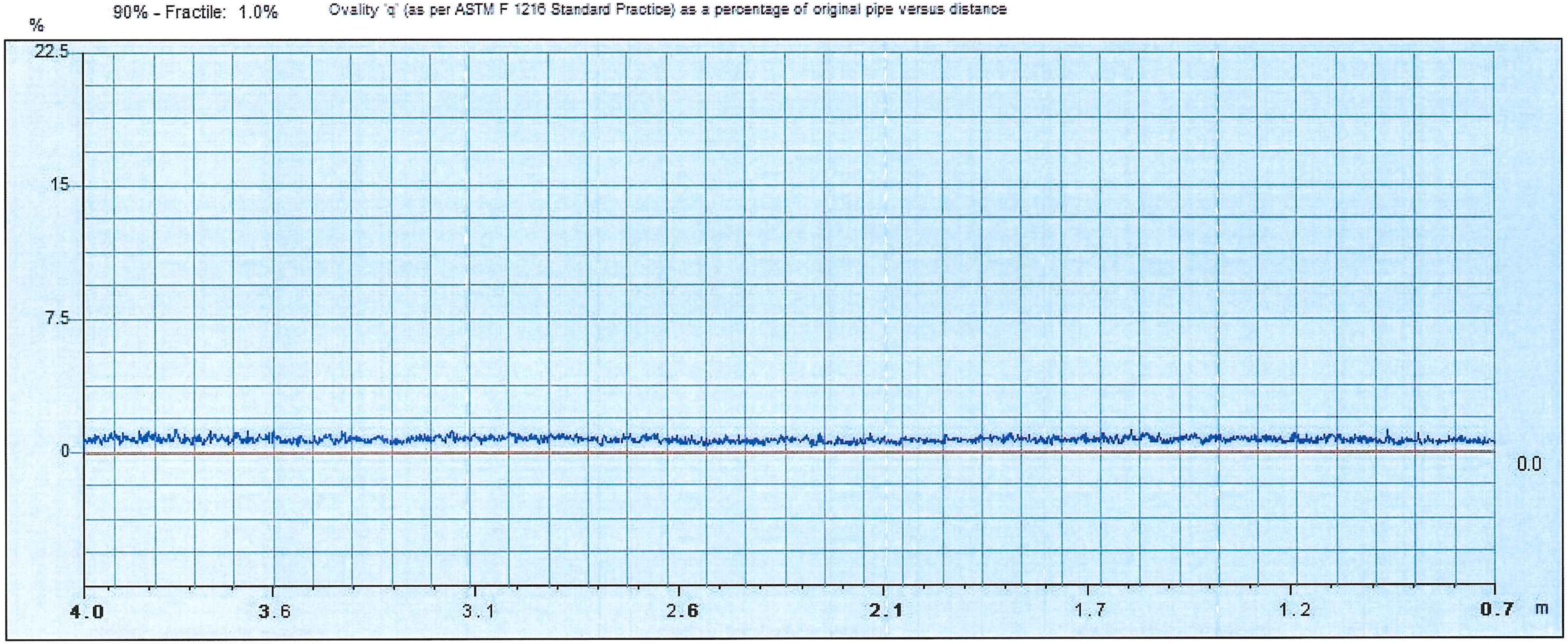
Site ID		Asset No.	Test 3	Date	17/10/2013
City				Material	PVC
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

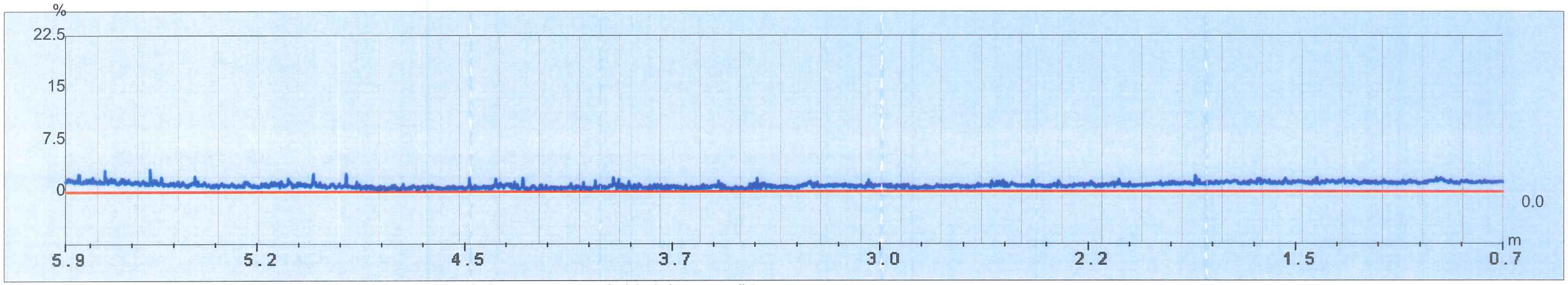
90% - Fractile: 1.0%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



Ovality Observations Report

Site ID		Asset No.	Test 4	Date	17/10/2013
City				Material	PVC
Start No	CE4.1	Finish No	MH4.1	Pipeline Length	5.9 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm



90% - Fractile: 1.5%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

HydroTech

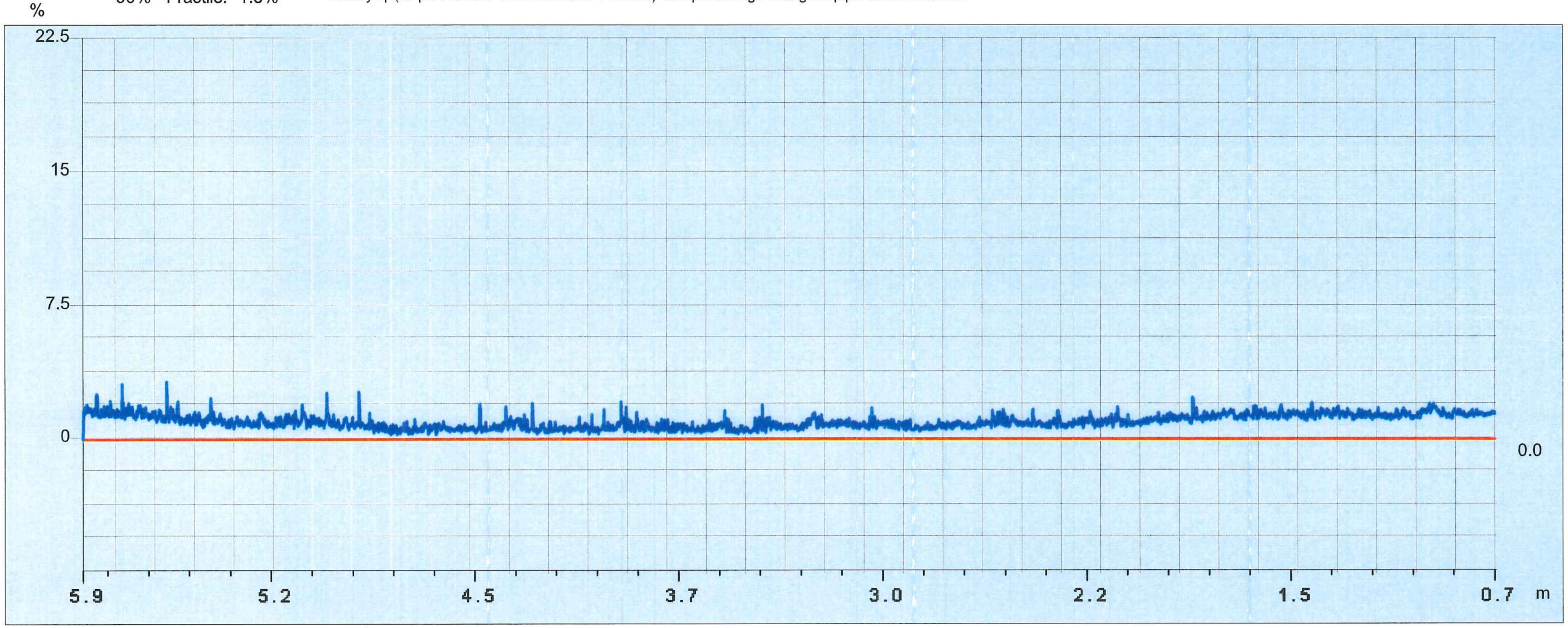
Site ID		Asset No.	Test 4	Date	17/10/2013
City				Material	PVC
Start No	CE4.1	Finish No	MH4.1	Pipeline Length	5.9 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

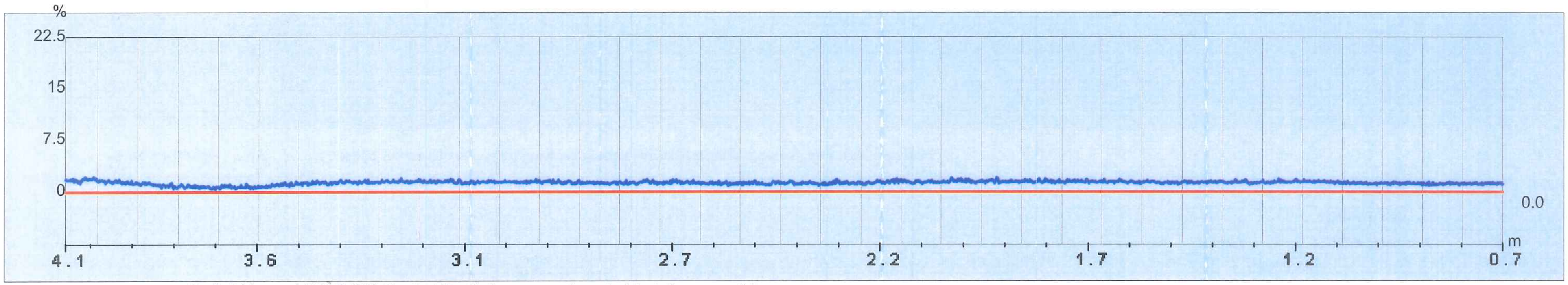
90% - Fractile: 1.5%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



Ovality Observations Report

Site ID		Asset No.	Test 5	Date	17/10/2013
City				Material	pvc
Start No	CE5.2	Finish No	MH1.2	Pipeline Length	4.08 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm



90% - Fractile: 1.7%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

Site ID		Asset No.	Test 5	Date	17/10/2013
City				Material	pvc
Start No	CE5.2	Finish No	MH1.2	Pipeline Length	4.08 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

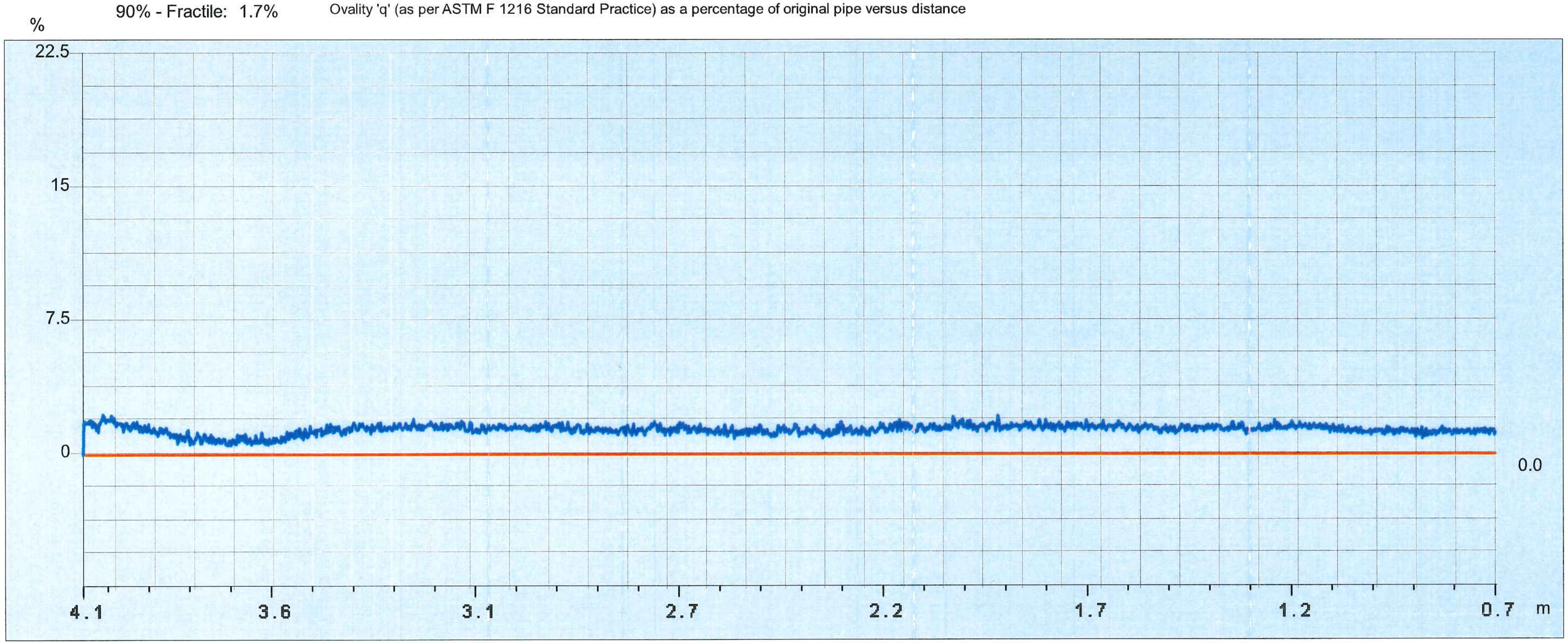
Limit Lines

Upper limit = 0

Lower Limit= 0

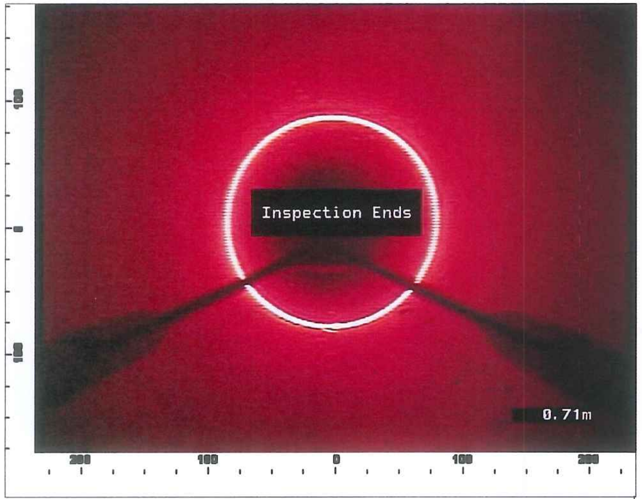
90% - Fractile: 1.7%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

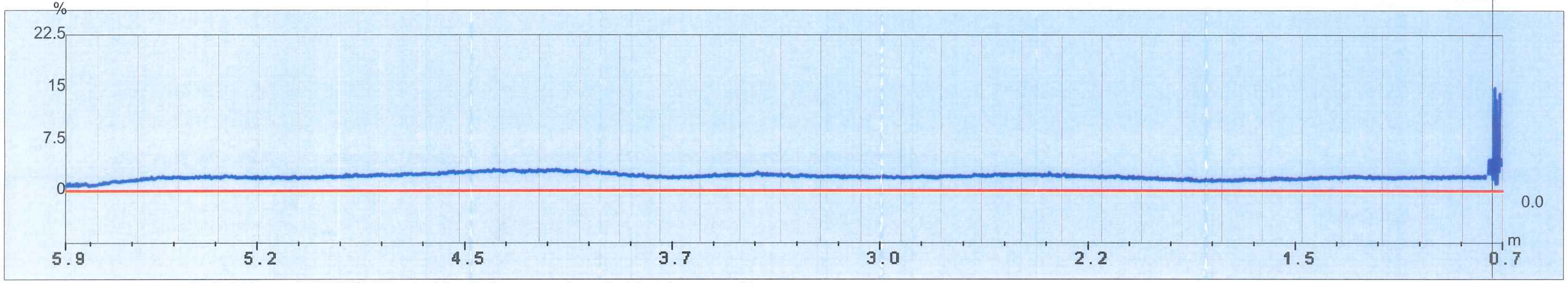


Ovality Observations Report

Site ID		Asset No.	Test 7	Date	17/10/2013
City				Material	PVC
Start No	CE7.1	Finish No	MH7.2	Pipeline Length	5.92 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm



Abnormal readings due to end of inspection screen.



90% - Fractile: 2.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

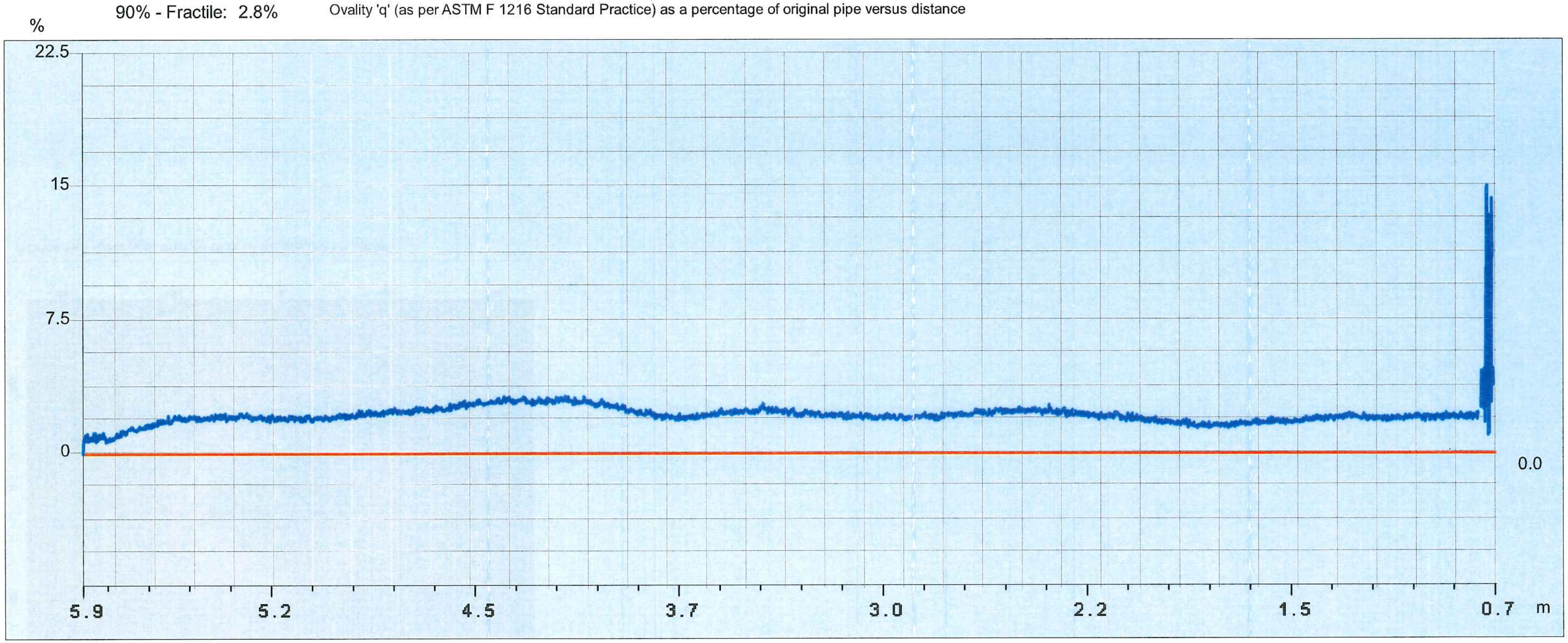
Site ID		Asset No.	Test 7	Date	17/10/2013
City				Material	PVC
Start No	CE7.1	Finish No	MH7.2	Pipeline Length	5.92 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

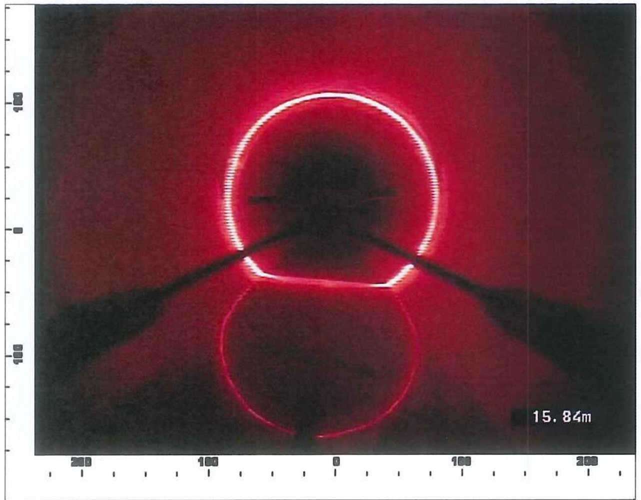
90% - Fractile: 2.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

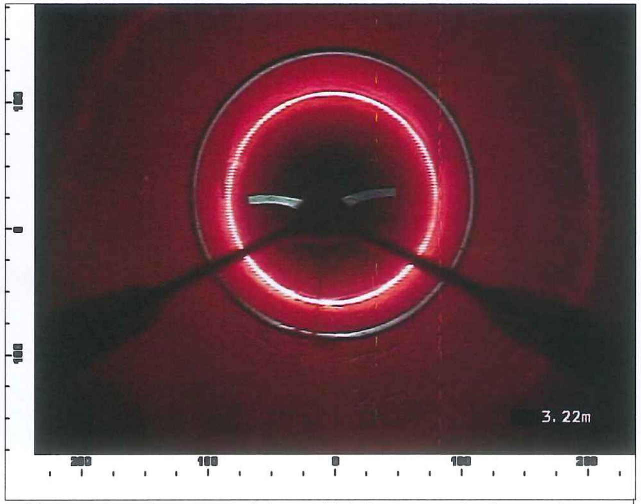


Ovality Observations Report

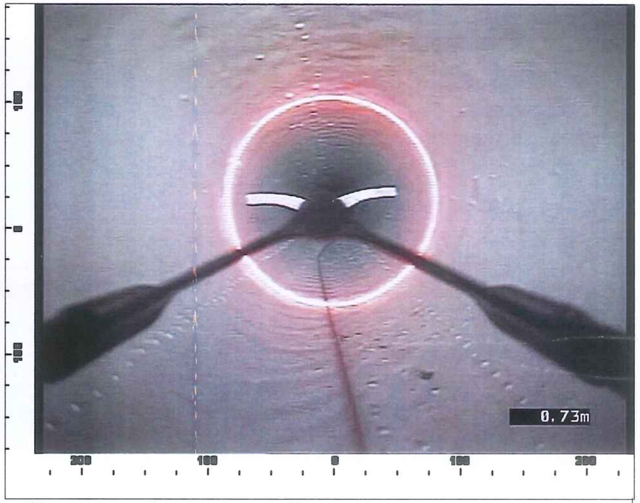
Site ID		Asset No.	Test 8	Date	19/10/2013
City				Material	PVC
Start No	8.2	Finish No	8.1	Pipeline Length	41.52 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm



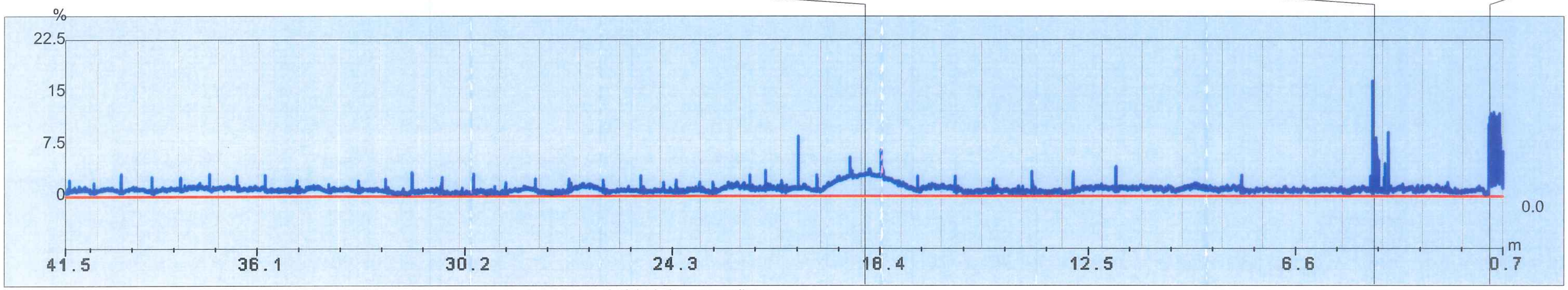
Abnormal reading due to small dip in pipe.



Abnormal readings due to joint and brief camera misalignment.



Abnormal reading due to end of inspection.



90% - Fractile: 1.5%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

HydroTech

Site ID		Asset No.	Test 8	Date	19/10/2013
City				Material	PVC
Start No	8.2	Finish No	8.1	Pipeline Length	41.52 m
Location	Ardrossan St, Avondale	Location	Ardrossan St, Avondale	Internal Diameter (Expected)	150 mm

Comments

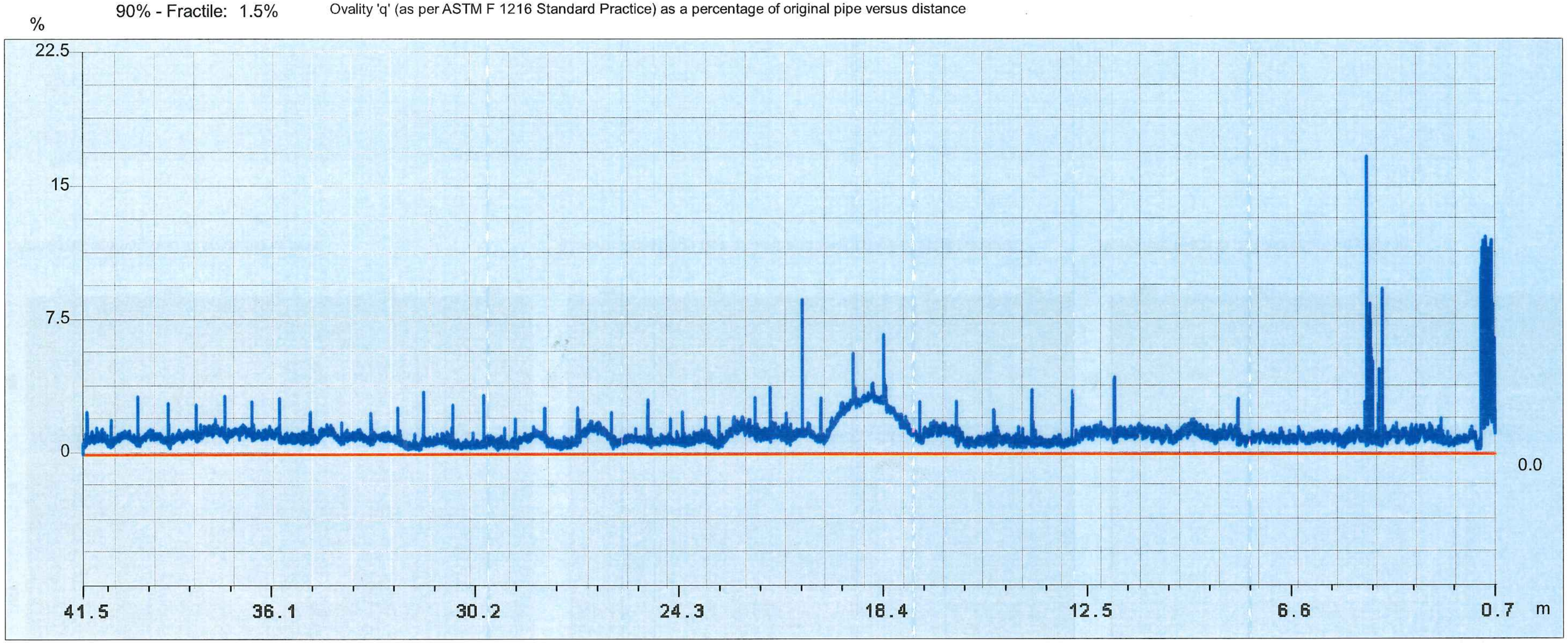
Limit Lines

Upper limit = 0

Lower Limit= 0

90% - Fractile: 1.5%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



HydroTech

Ovality Summary Report

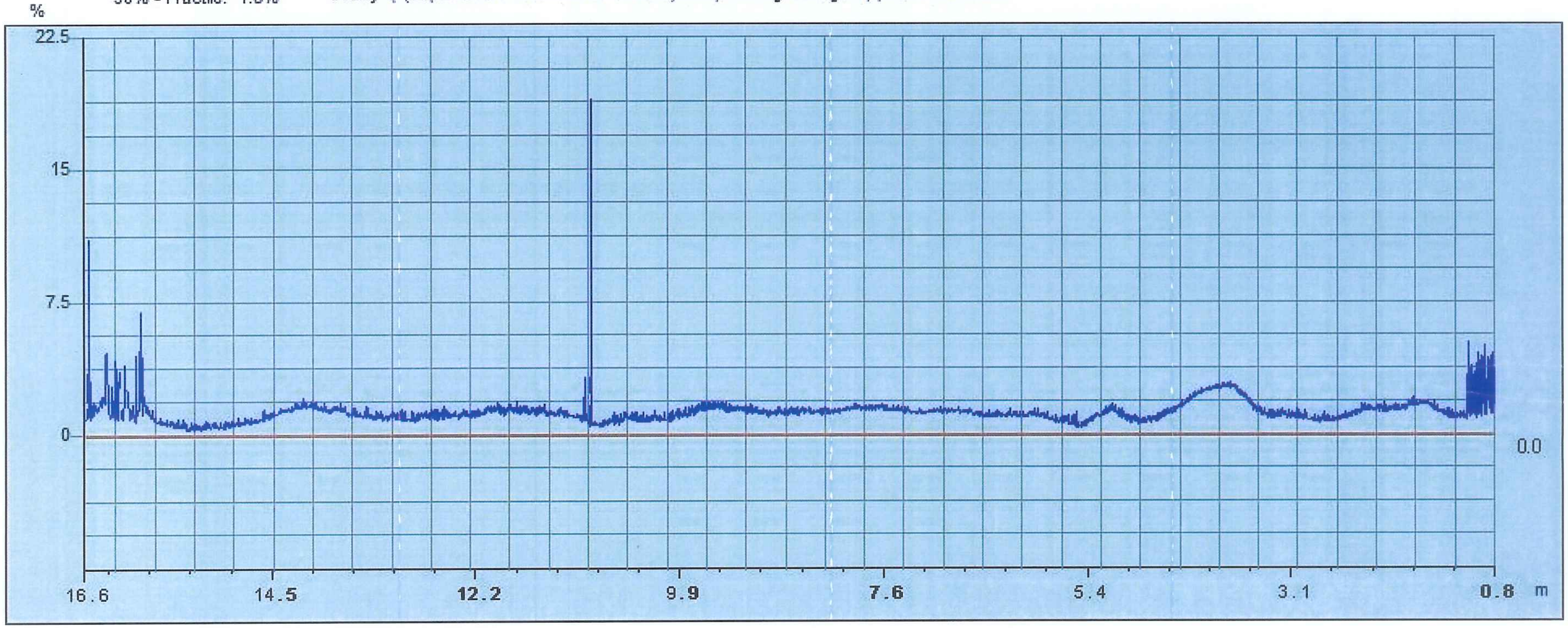
Site ID		Asset No.	Test 2 - Post	Date	30/10/13
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.61 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

90% - Fractile: 1.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



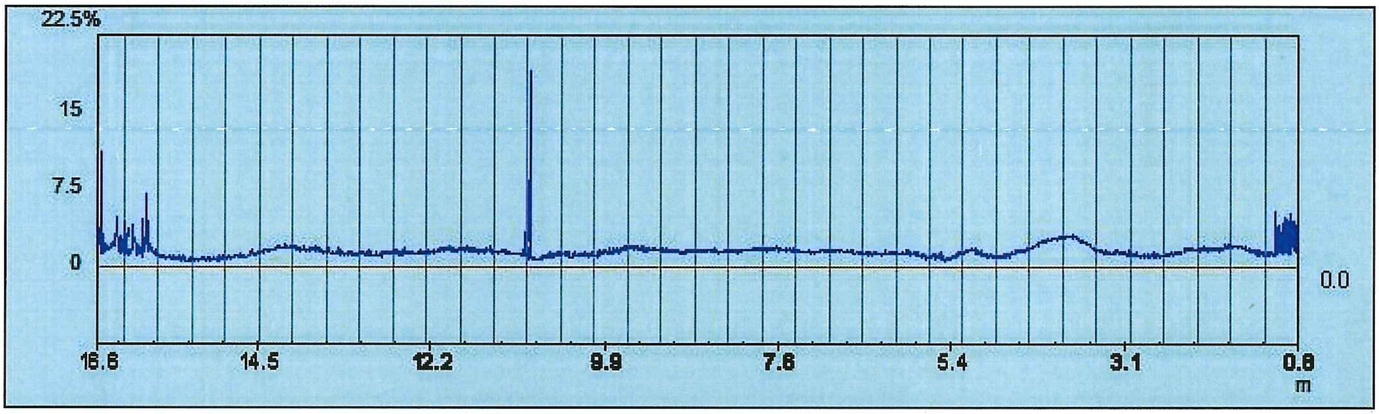
HydroTech

Ovality Analysis Report

Site ID		Asset No.	Test 2 - Post	Date	30/10/13
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.61m
Location	Ardorssan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150mm

90% - Fractile: 1.8%

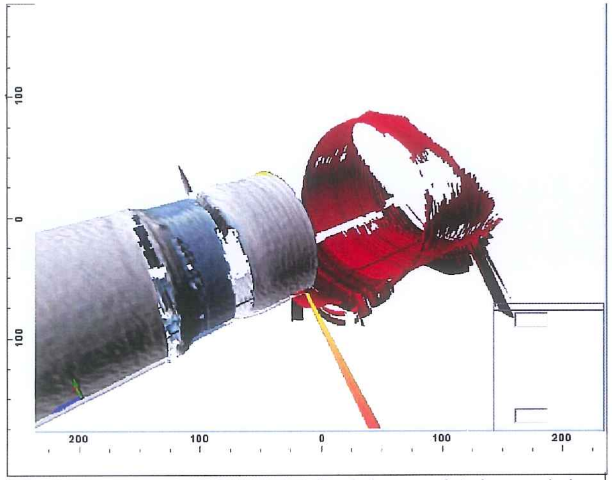
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



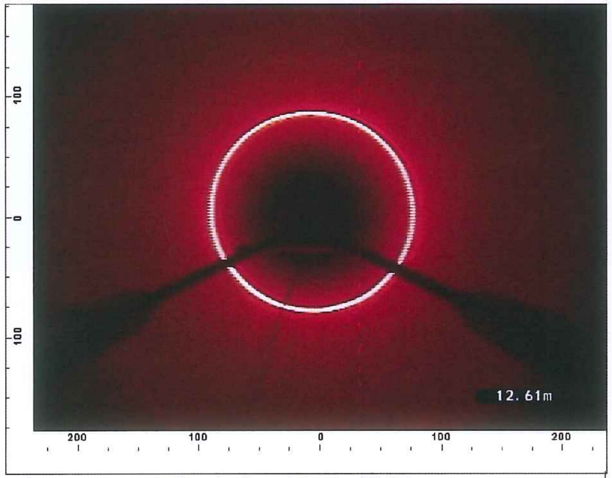
Comments

Ovality Observations Report

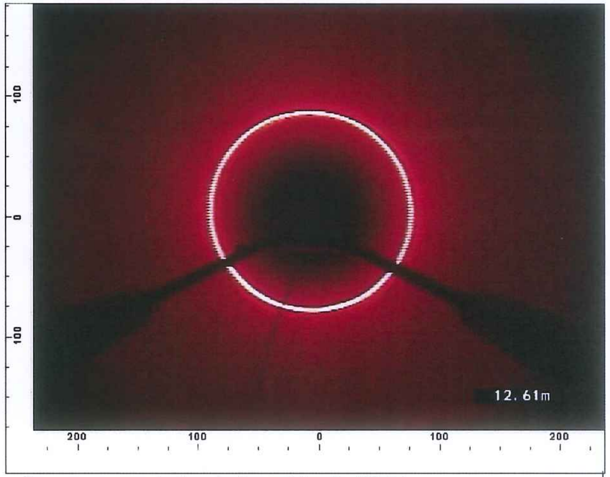
Site ID		Asset No.	Test 2 - Post	Date	30/10/13
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.61 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm



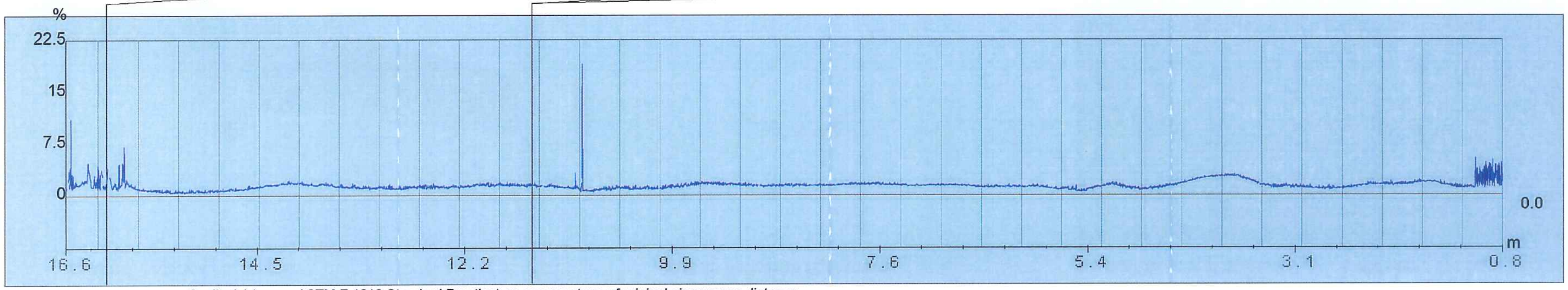
Abnormal readings at start of pipe due to laser registering manhole.



Camera momentarily misaligned due to JO (L)



Camera momentarily misaligned due to JO (L)



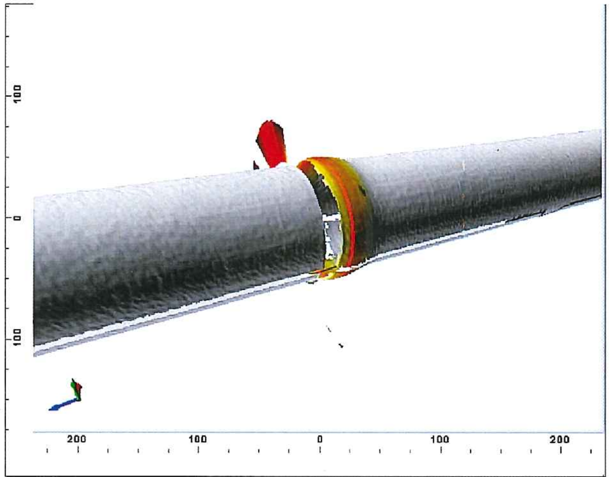
90% - Fractile: 1.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

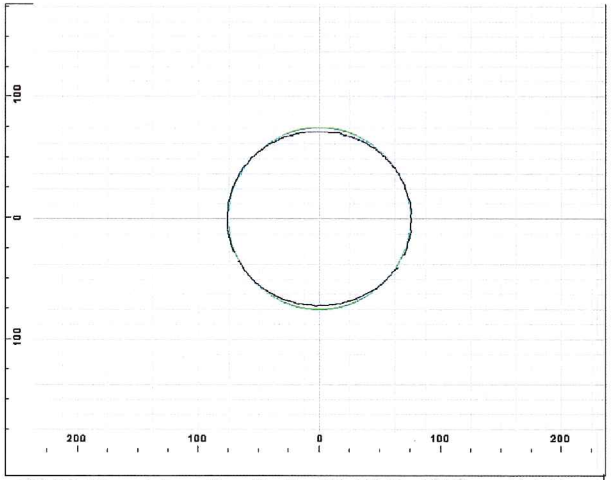
HydroTech

Ovality Observations Report

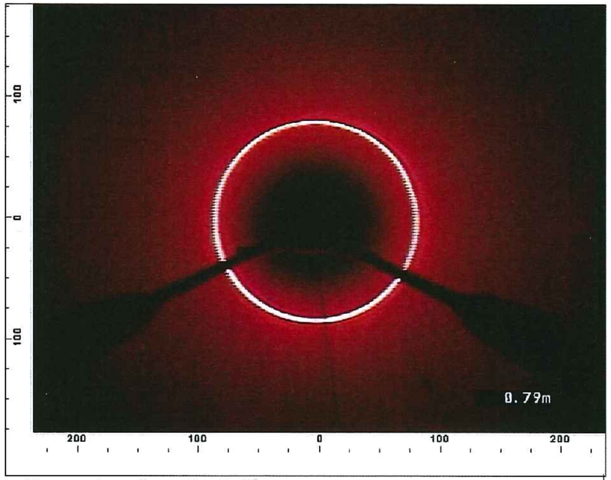
Site ID		Asset No.	Test 2 - Post	Date	30/10/13
City				Material	PVC
Start No	MH2.2	Finish No	MH2.1	Pipeline Length	16.61 m
Location	Ardorssan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm



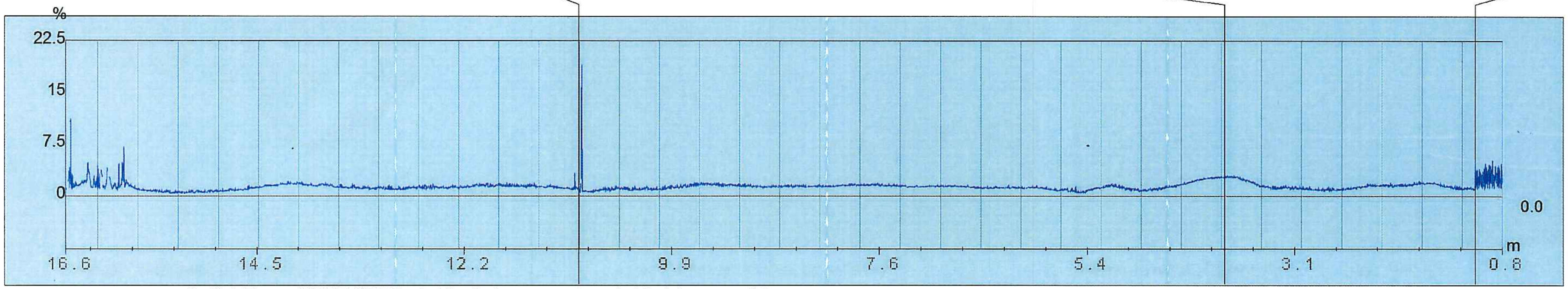
JO (L)



PF (S). Minor deformation. Ovality: 3%. Width: 151.7mm. Height: 142.9mm



Abnormal readings due to IE.



90% - Fractile: 1.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

HydroTech

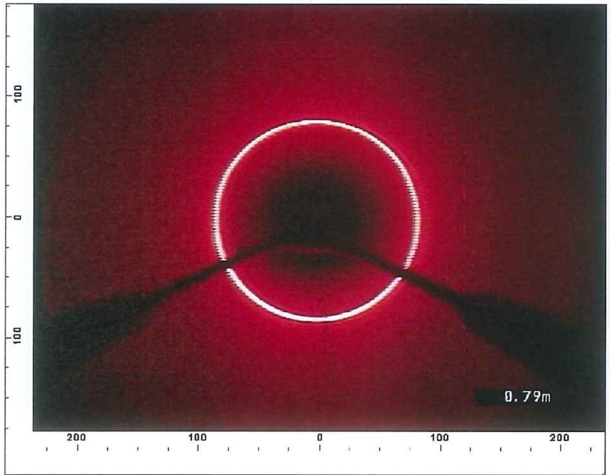
Ovality Observations Report

Site ID
City
Start No MH2.2
Location Ardorssan St, Wainoni

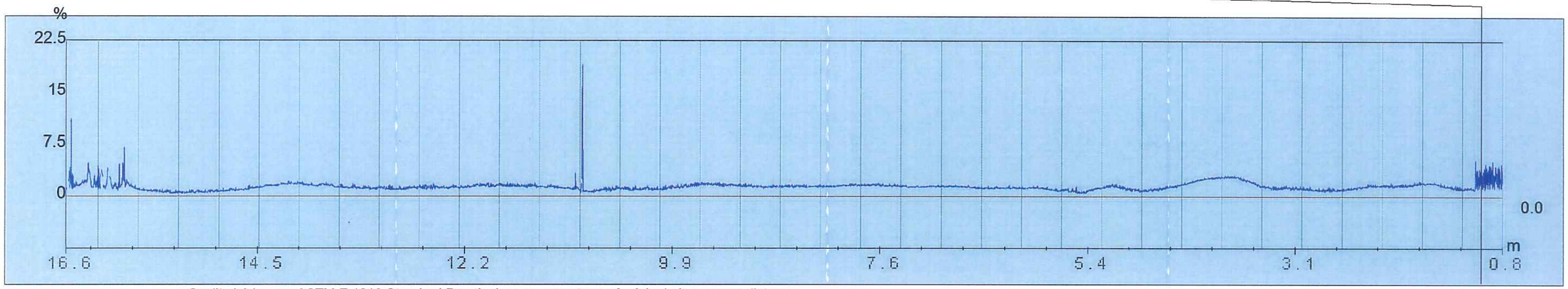
Asset No. Test 2 - Post

Finish No MH2.1
Location Ardrossan St, Wainoni

Date 30/10/13
Material PVC
Pipeline Length 16.61 m
Internal Diameter (Expected) 150 mm



Abnormal readings due to inspection end.



90% - Fractile: 1.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

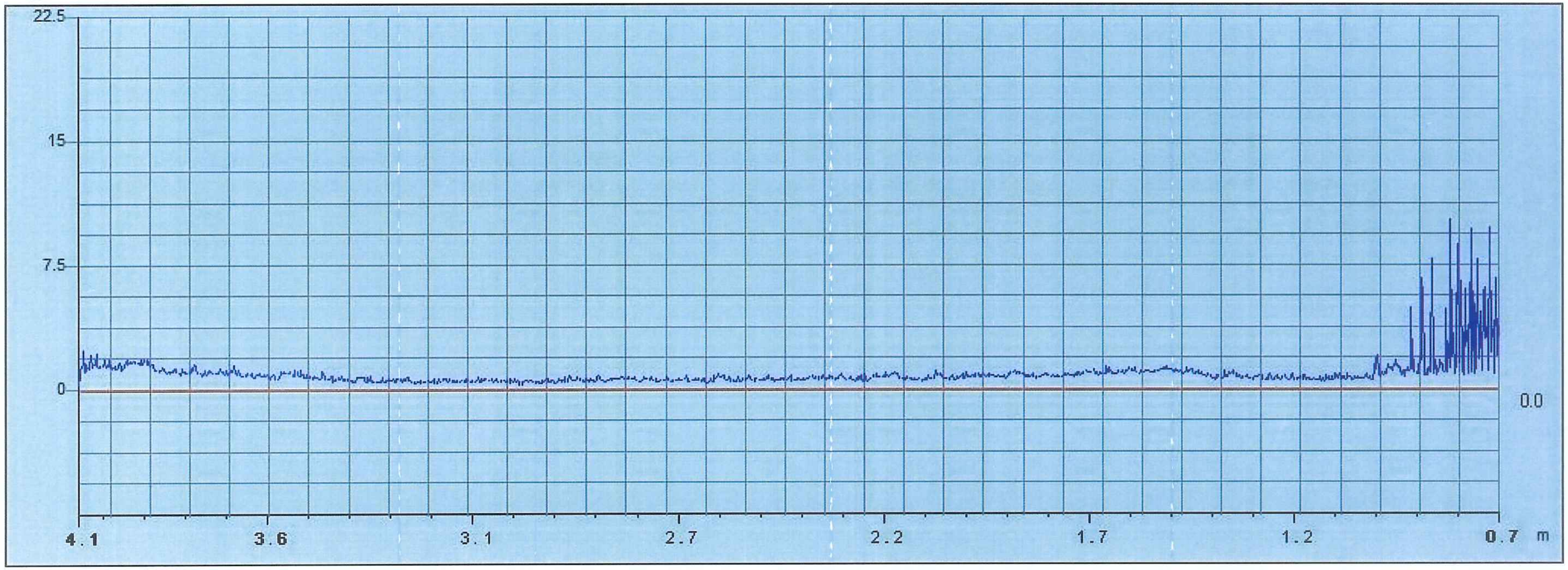
Site ID		Asset No.	Test 3 - Post	Date	30/10/2013
City				Material	
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4.07 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

90% - Fractile: 1.4%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



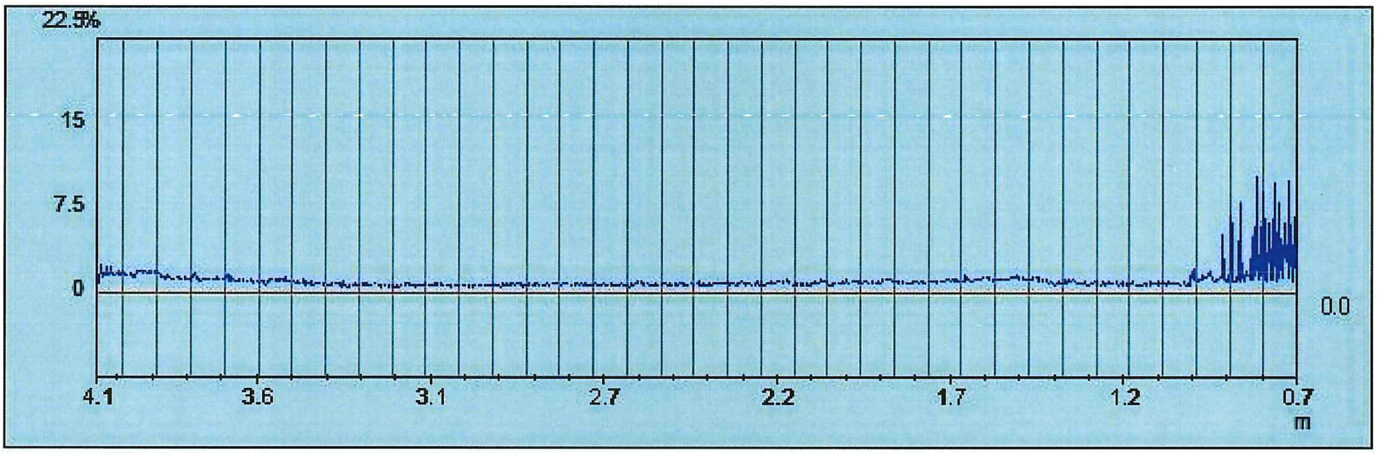
HydroTech

Ovality Analysis Report

Site ID		Asset No.	Test 3 - Post	Date	30/10/2013
City				Material	
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4.07m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Waind	Internal Diameter (Expected)	150mm

90% - Fractile: 1.4%

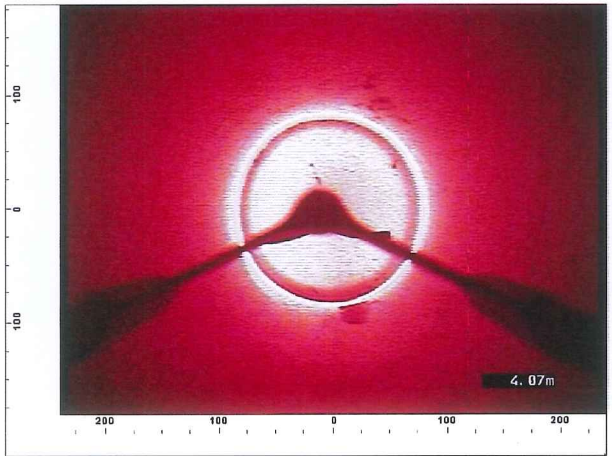
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



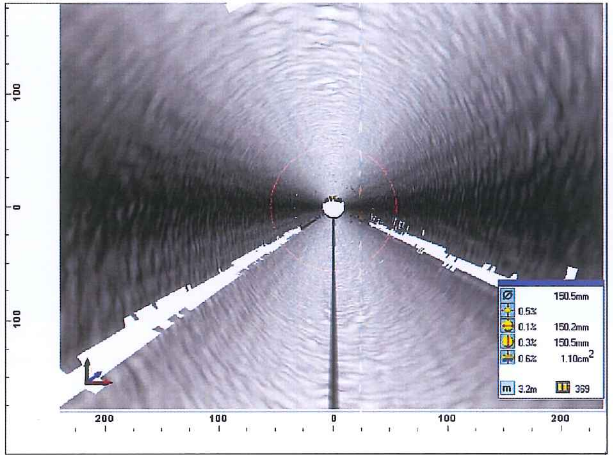
Comments

Ovality Observations Report

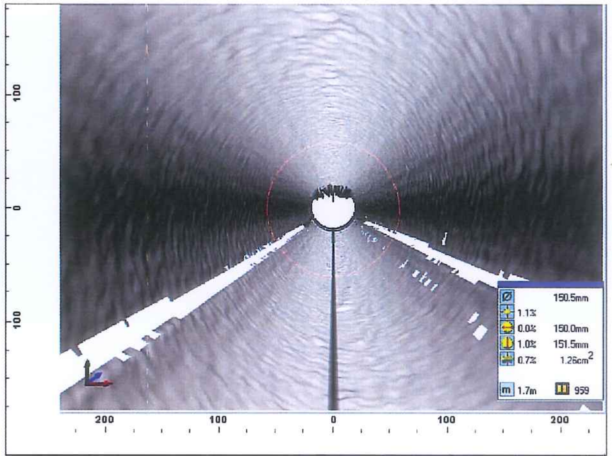
Site ID		Asset No.	Test 3 - Post	Date	30/10/2013
City				Material	
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4.07 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm



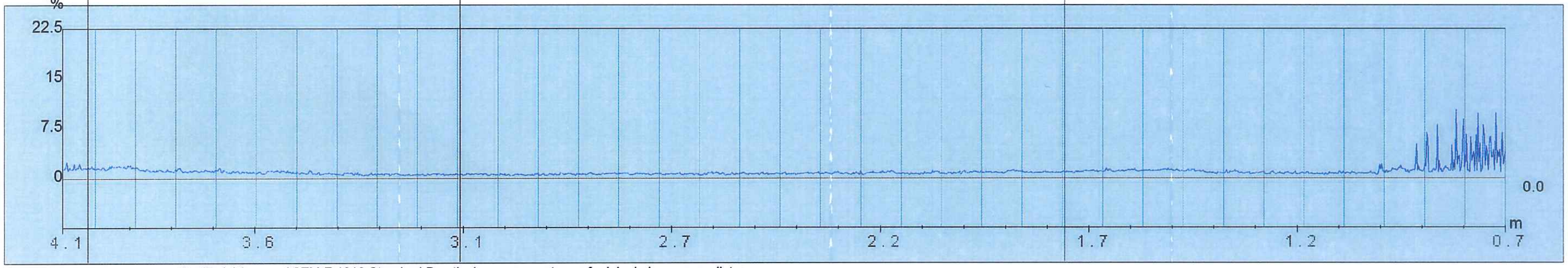
Abnormal readings due to pipe end.



General pipe condition: Diameter: 150.6mm. Ovality: 0.6%



General pipe condition: Diameter: 150.5mm. Ovality: 1.1%..



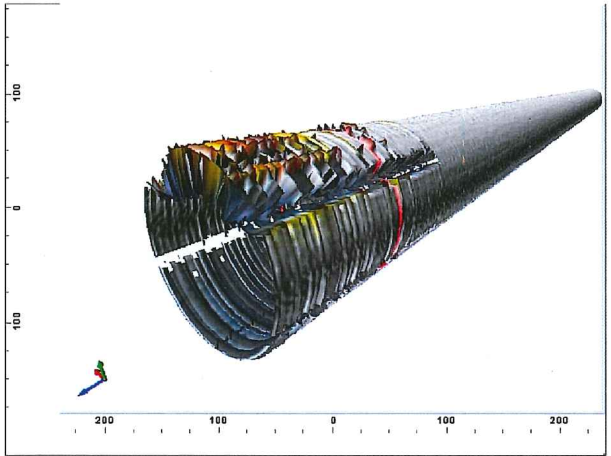
90% - Fractile: 1.4%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

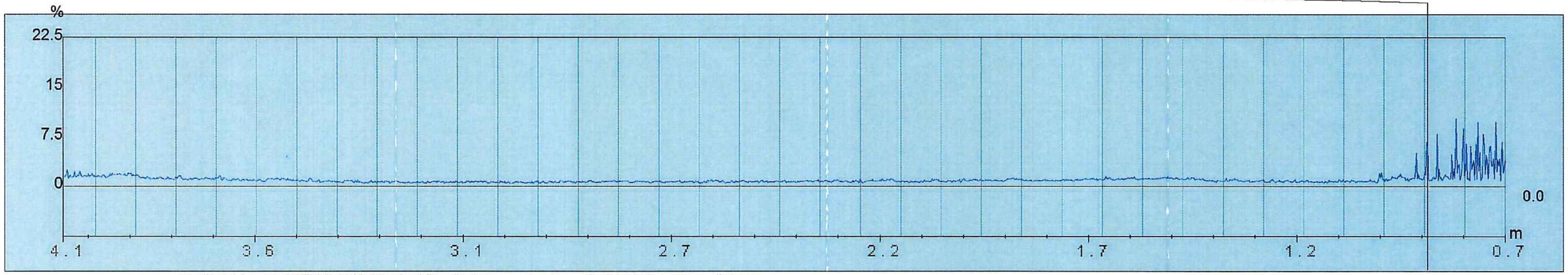
HydroTech

Ovality Observations Report

Site ID		Asset No.	Test 3 - Post	Date	30/10/2013
City				Material	
Start No	CE3.1	Finish No	MH3.2	Pipeline Length	4.07 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm



Abnormal readings due to IE.



90% - Fractile: 1.4%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

HydroTech

Ovality Summary Report

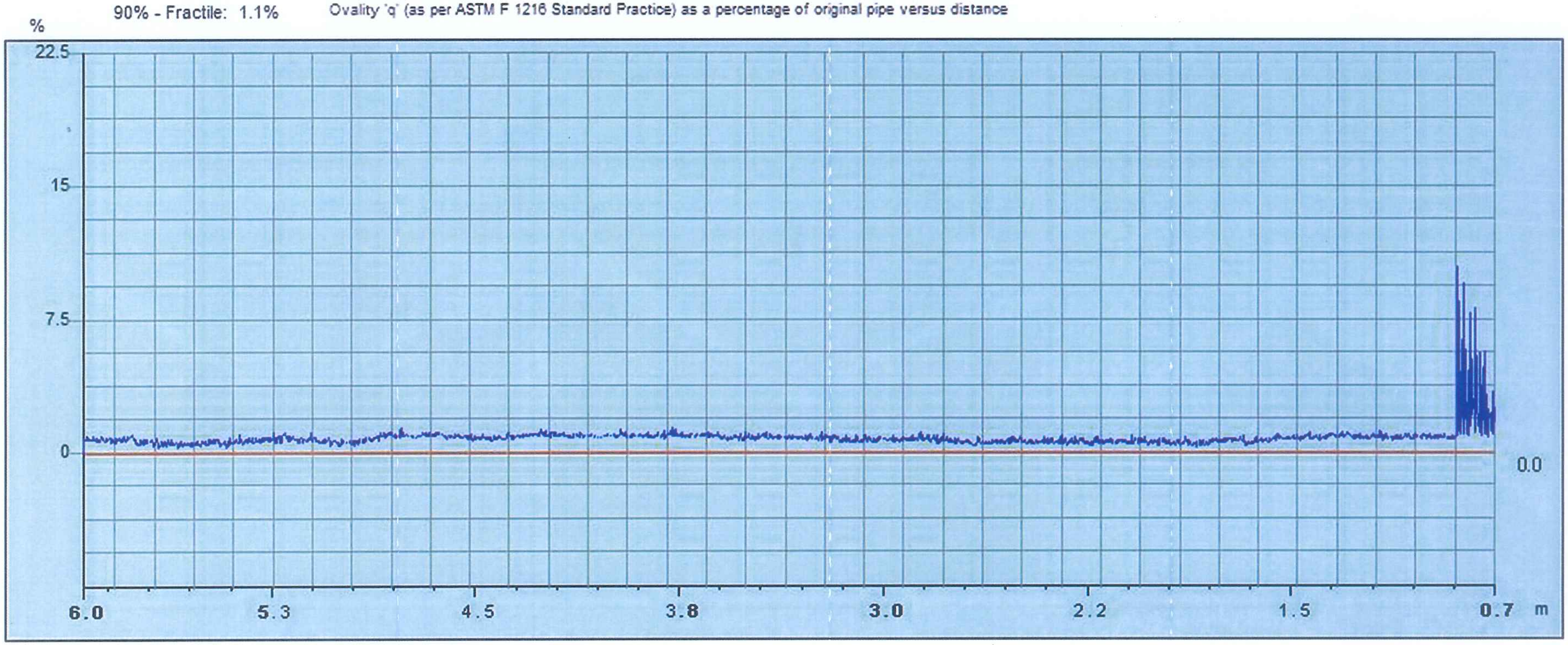
Site ID		Asset No.	Test 4 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE4.1	Finish No	MH4.3	Pipeline Length	5.98 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm

Comments

Limit Lines
Upper limit = 0
Lower Limit= 0

90% - Fractile: 1.1%

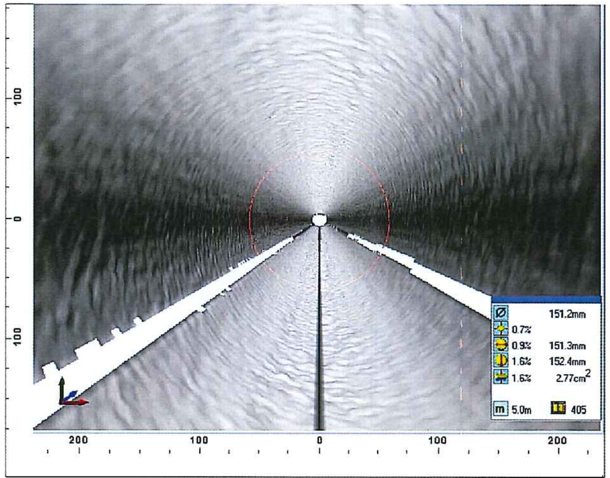
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



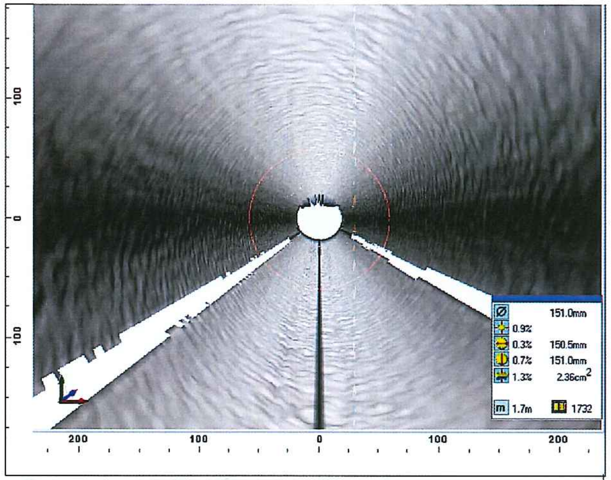
HydroTech

Ovality Observations Report

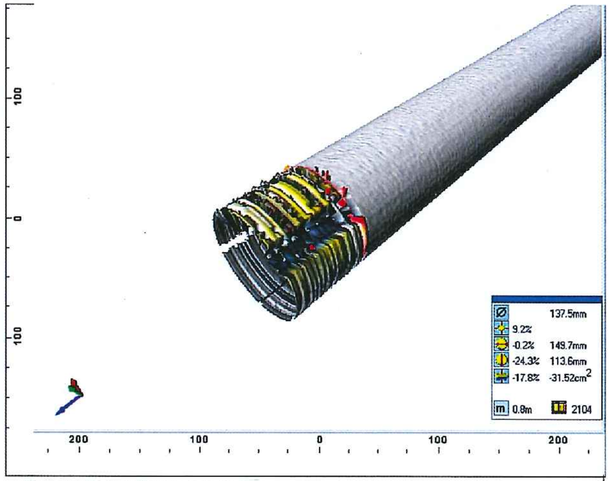
Site ID		Asset No.	Test 4 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE4.1	Finish No	MH4.3	Pipeline Length	5.98 m
Location	Ardrossan St, Wainoni	Location	Ardrossan St, Wainoni	Internal Diameter (Expected)	150 mm



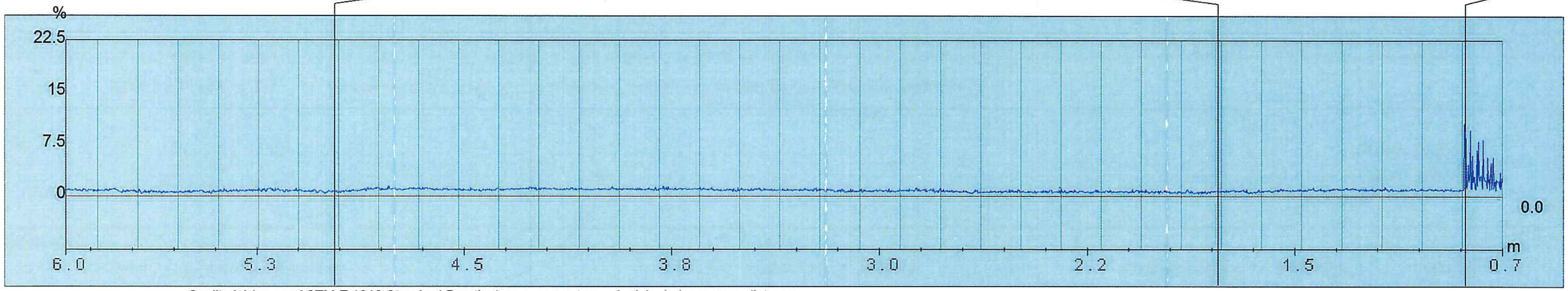
General pipe condition: Diameter: 151.2mm Ovality: 0.7%



General pipe condition: Diameter: 151mm Ovality: 0.9%



Abnormal readings due to IE



90% - Fractile: 1.1%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

HydroTech

Ovality Summary Report

Site ID		Asset No.	Test 5 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE5.1	Finish No	MH1.2	Pipeline Length	4.03 m
Location	Ardrossan St. Wainoni	Location	Ardrossan St. Wainoni	Internal Diameter (Expected)	150 mm

Comments

Post test.

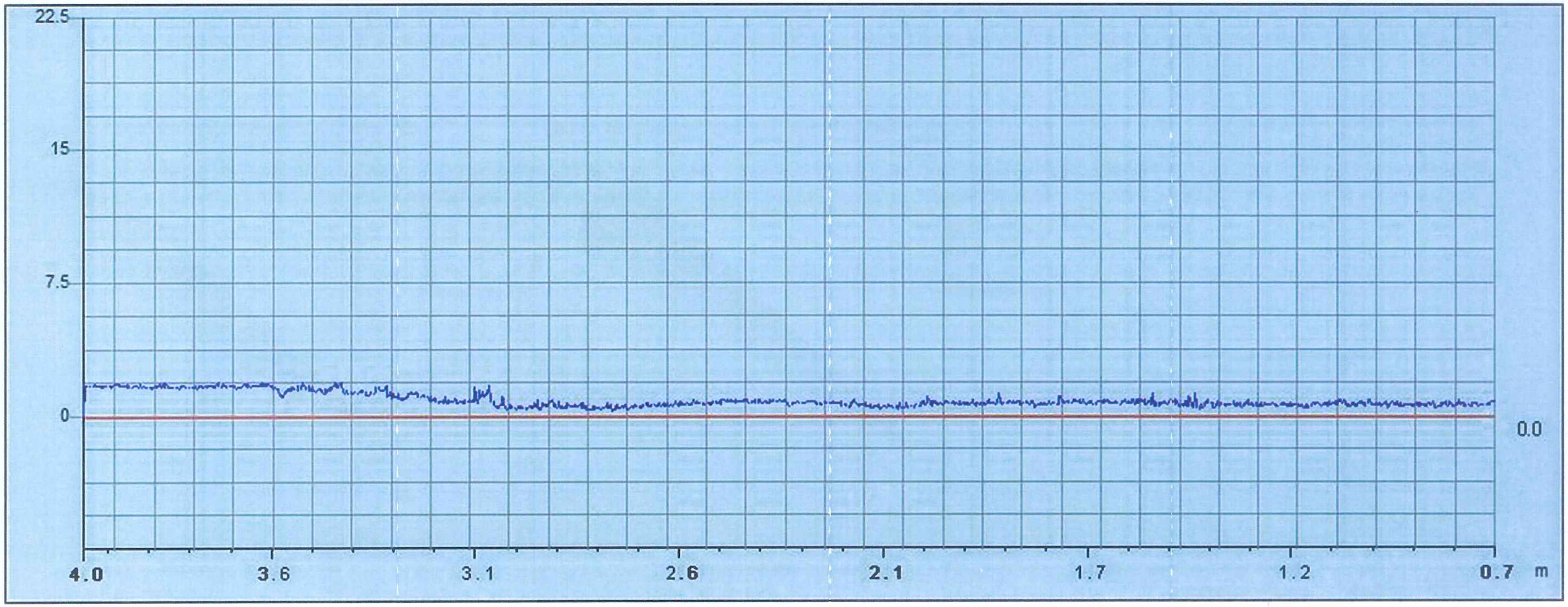
Limit Lines

Upper limit = 0

Lower Limit= 0

90% - Fractile: 1.8%

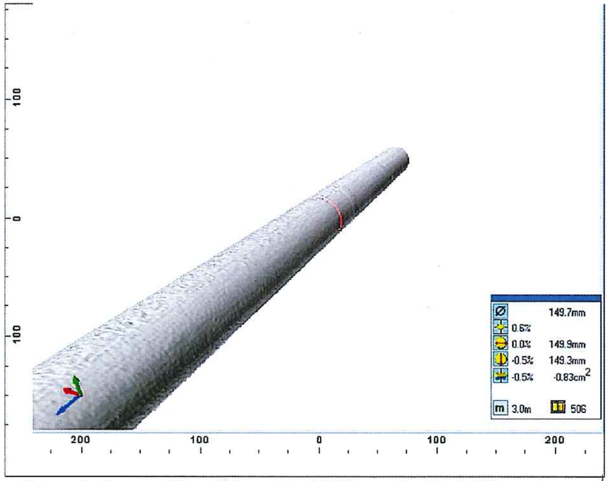
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



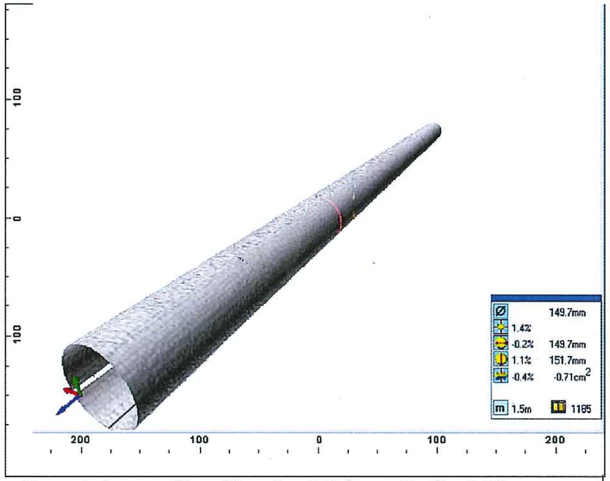
HydroTech

Ovality Observations Report

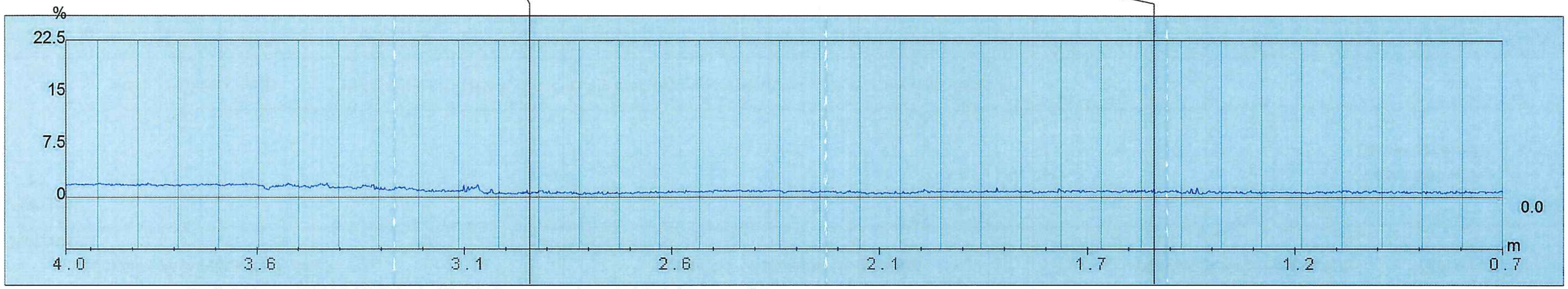
Site ID		Asset No.	Test 5 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE5.1	Finish No	MH1.2	Pipeline Length	4.03 m
Location	Ardrossan St. Wainoni	Location	Ardrossan St. Wainoni	Internal Diameter (Expected)	150 mm



General pipe condition: Diameter: 149.7mm Ovality: 0.6%



General pipe condition: Diameter: 149.7mm. Ovality: 1.4%



90% - Fractile: 1.8%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

Ovality Summary Report

Site ID		Asset No.	Test 7 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE7.1	Finish No	MH7.2	Pipeline Length	5.92 m
Location	Ardrossan St. Wainoni	Location	Ardrossan St. Wainoni	Internal Diameter (Expected)	150 mm

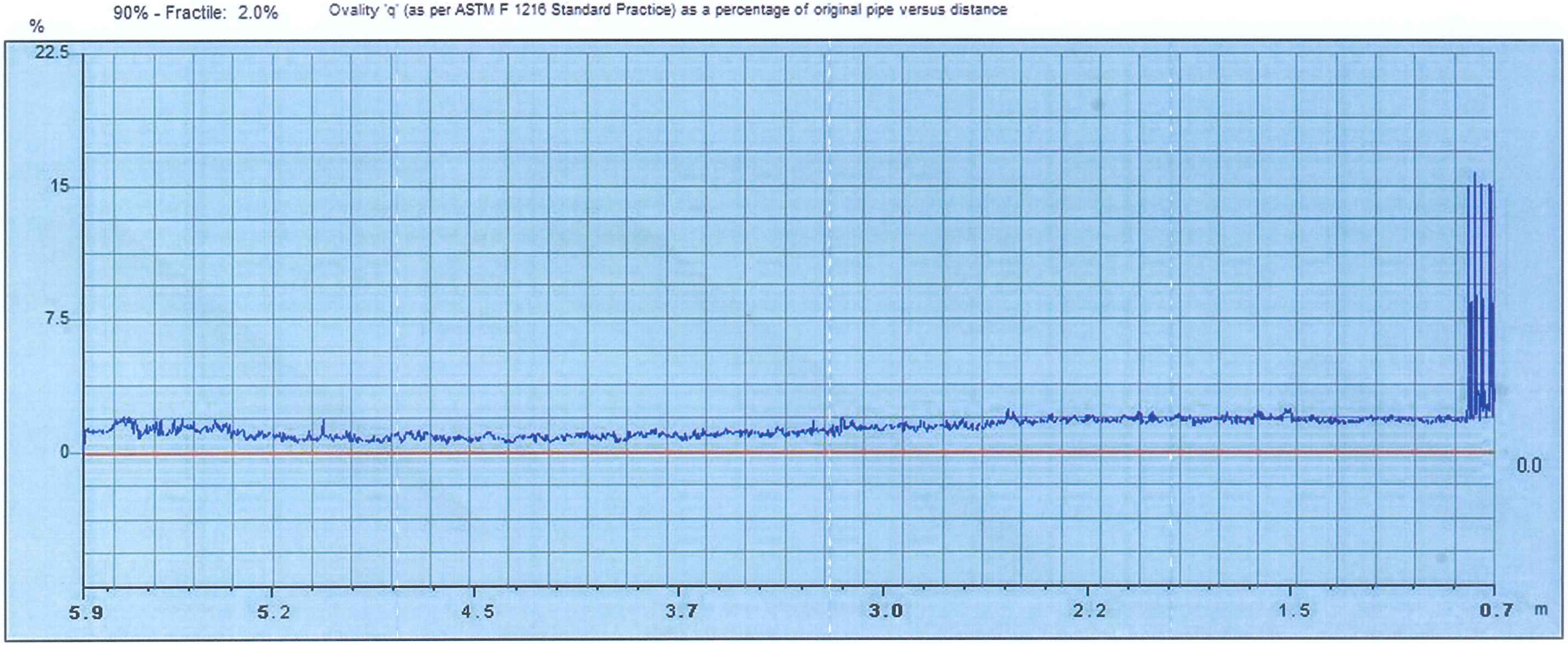
Comments

Post test.

Limit Lines
Upper limit = 0
Lower Limit= 0

90% - Fractile: 2.0%

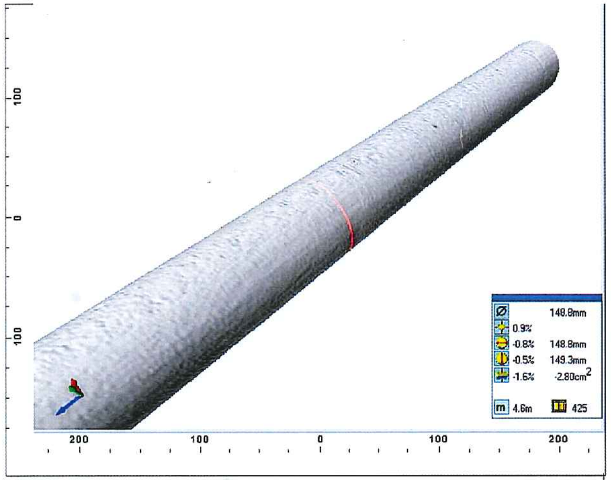
Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance



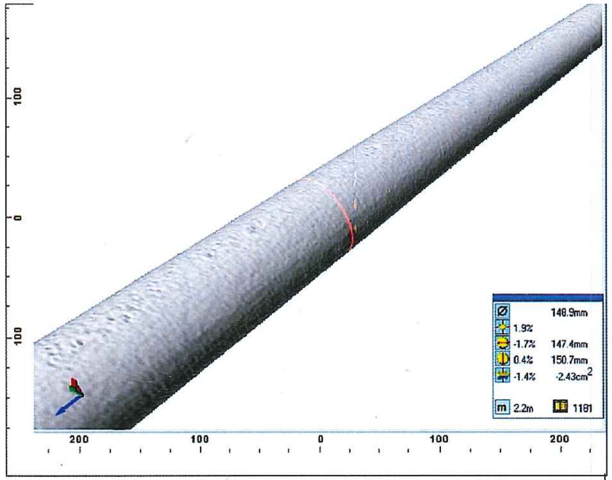
Hydro Tech

Ovality Observations Report

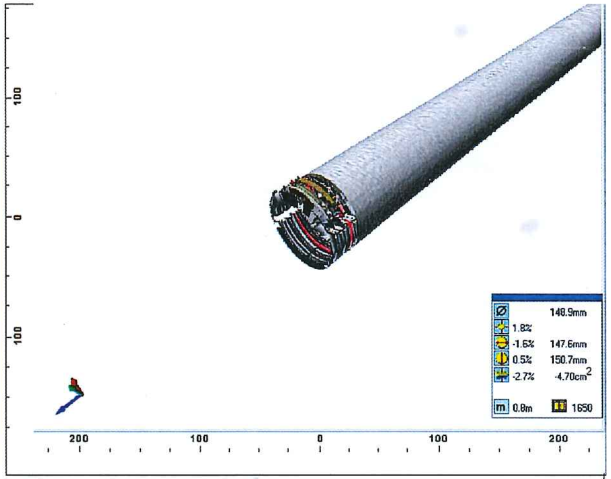
Site ID		Asset No.	Test 7 - Post	Date	30/10/2013
City				Material	PVC
Start No	CE7.1	Finish No	MH7.2	Pipeline Length	5.92 m
Location	Ardrossan St. Wainoni	Location	Ardrossan St. Wainoni	Internal Diameter (Expected)	150 mm



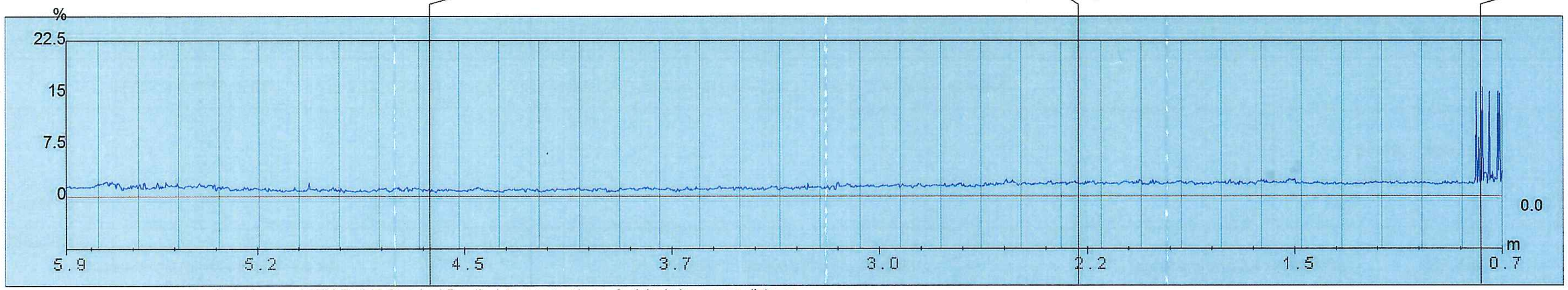
General pipe condition: Diameter: 148.8mm Ovality: 0.9%



General pipe condition: Diameter: 148.9mm Ovality: 1.9%



Abnormal readings due to IE



90% - Fractile: 2.0%

Ovality 'q' (as per ASTM F 1216 Standard Practice) as a percentage of original pipe versus distance

H

Appendix H Uplift Calculations

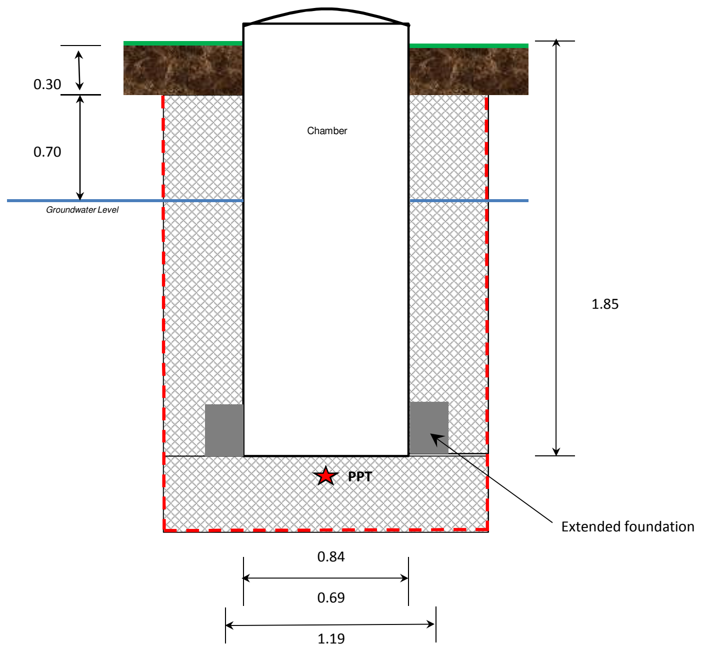
Assessment of Liquefaction Induced Uplift of Chambers Trial DESIGN assumptions

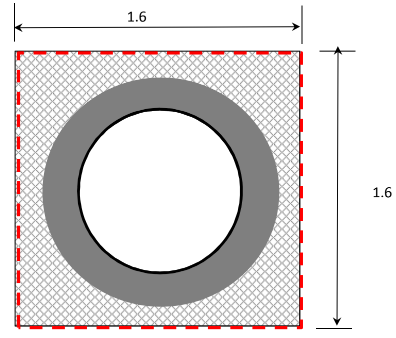
Assessment Trial No. 3 - Pressure sewer with permeable backfill

mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

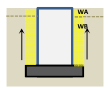
Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	0.47	m^3
Volume of concrete ring foundation	0.15	m^3
Unsaturated Unit weight of backfill	18	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Piezometer	1.97	m
Peak Net Piezometer Pressure	8.2	kPa
Equivalent Pore water Pressure within backfill	14.0	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	369	kg
Angle of friction of backfill	32	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.56	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.85	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	0.2	m
Diameter of base flange	1.19	m
Width of backfill a	1.60	m
Width of backfill b	1.60	m
Depth to ground water level	1.00	m
Liquefaction within Backfill (Y/N)	N	

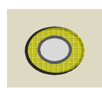




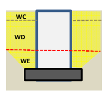
Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Static Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**6.09**
	Seismic: Excess pore pressure				**4.56**
	Seepage Force assumed to be 5% of Buoyant force				**0.53**
	Total				**11.18**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.56	0.17	**2.84**
	Wedge A (W) - topsoil below GWL	0.00	0.56	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.70	0.56	0.39	**7.01**
	Wedge B (W) - backfill below GWL	0.65	0.56	0.36	**1.46**
	Shear resistance within topsoil - above GWL				**0.83**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**8.76**
	Shear resistance within granular backfill - below GWL				**13.57**
	Weight of tank and concrete base (W)				**4.35**
	Total				**38.81**
Resisting force - Method B	Wedge C - topsoil above GWL	0.30	1.45	0.44	**7.42**
	Wedge C - Topsoil below GWL	0.00	1.45	0.00	**0.00**
	Wedge D - backfill above GWL	0.70	1.45	1.02	**18.33**
	Wedge D - Backfill below GWL	0.28	1.45	0.41	**1.64**
	Wedge E - Backfill below GWL	0.37	1.01	0.37	**1.50**
	Weight of tank and concrete base (W)				**4.35**
	Total				**33.23** *Critical Case*

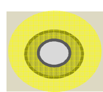
K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30
0.47	12.6	32
0.47	2.6	32





H = 0.36983





Factor of Safety (FOS) against uplift FOS = 2.97

$$Fs = \frac{W+Q}{Us+Ud+F}$$

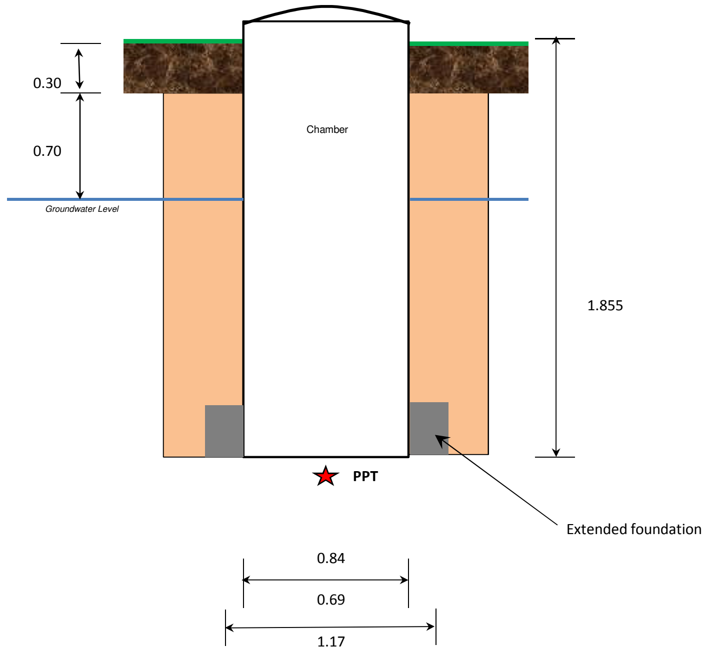
Assessment of Liquefaction Induced Uplift of Chambers   Trial DESIGN assumptions

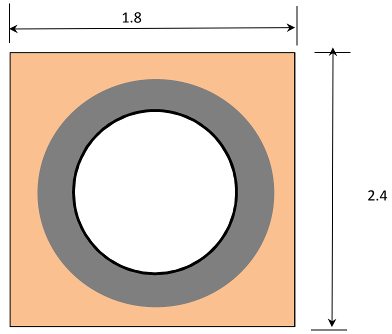
Assessment Trial No. 4 - Pressure sewer with natural backfill

mfg   22/01/2014

Assessment of FOS to Buoyant Uplift

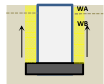
Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	0.47	m^3
Volume of concrete ring foundation	0.14	m^3
Unsaturated Unit weight of backfill	17	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Piezometer	1.9	m
Peak Net Piezometer Pressure	15.6	kPa
Equivalent Pore water Pressure within backfill	18.0	kN/m^3 *Liquefied = ~18kN/m^3*
Weight of water within Chamber	30	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	359	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.55	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.855	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	0.205	m
Diameter of base flange	1.17	m
Width of backfill a	1.80	m
Width of backfill b	2.40	m
Depth to ground water level	1.00	m
Liquefaction within Backfill (Y/N)	Y	

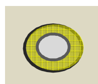




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**6.06**
	Seismic: Excess pore pressure				**8.65**
	Seepage Force assumed to be 5% of Buoyant force				**0.74**
	Total				**15.45**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.52	0.16	**2.65**
	Wedge A (W) - topsoil below GWL	0.00	0.52	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.70	0.52	0.36	**6.19**
	Wedge B (W) - backfill below GWL	0.65	0.52	0.34	**0.00**
	Shear resistance within topsoil - above GWL				**0.81**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**8.45**
	Shear resistance within granular backfill - below GWL				**0.00**
	Weight of tank and concrete base (W)				**4.55**
	Total				**22.66**
Resisting force - Method B					

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30





Factor of Safety (FOS) against uplift   FOS = 1.47

$$Fs = \frac{W+Q}{Us+Ud+F}$$

Assessment of Liquefaction Induced Uplift of Chambers Trial DESIGN assumptions

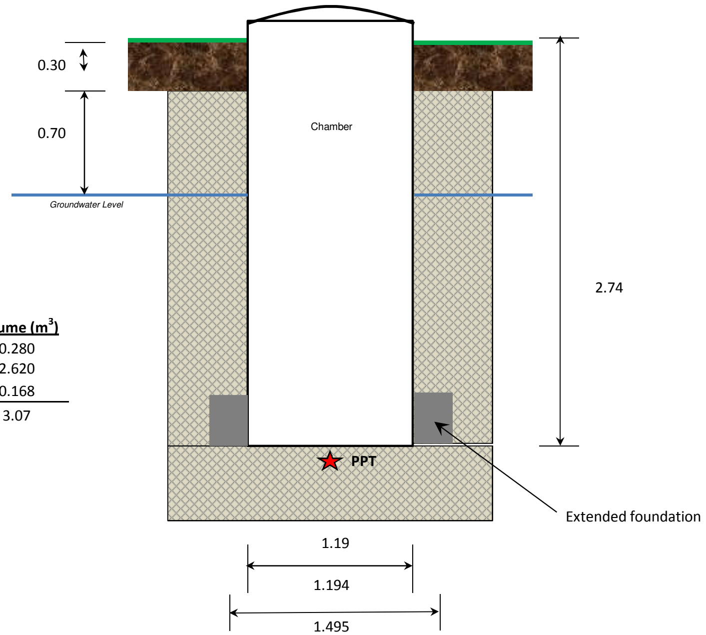
Assessment Trial No. 5 - Standard precast concrete manhole with CCC AP65 backfill

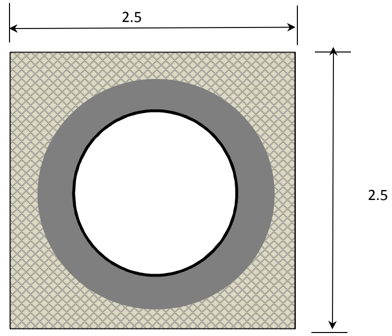
mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

Volume of chamber	3.07	m^3
Volume of chamber below GWL	1.95	m^3
Volume of concrete ring foundation	0.10	m^3
Unsaturated Unit weight of backfill	21	kN/m^3
Saturated Unit weight of backfill	22	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Peizometer	2.82	m
Peak Net Piezometer Pressure	23.2	kPa
Equivalent Pore water Pressure within backfill	18.0	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	2289	kg
Weight of concrete foundation	238	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	1.12	m^2
Typical width of chamber	1.194	m
Base width of chamber	1.194	m
Depth to base of chamber	2.74	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	0.15	m
Diameter of base flange	1.495	m
Width of backfill a	2.50	m
Width of backfill b	2.50	m
Depth to ground water level	1.00	m
Liquefaction within Backfill (Y/N)	N	

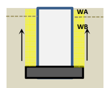
Item	Weight (kg)	Volume (m^3)
Lid	541	0.280
Riser	1328	2.620
Base	419	0.168
	2289	3.07

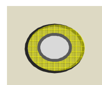


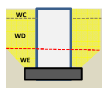


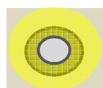
Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**20.05**
	Seismic: Excess pore pressure				**25.98**
	Seepage Force assumed to be 5% of Buoyant force				**2.30**
	Total				**48.33**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.64	0.19	**3.24**
	Wedge A (W) - topsoil below GWL	0.00	0.64	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.70	0.64	0.44	**9.34**
	Wedge B (W) - backfill below GWL	1.59	0.64	1.01	**4.00**
	Shear resistance within topsoil - above GWL				**1.04**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**12.17**
	Shear resistance within granular backfill - below GWL				**50.96**
	Weight of tank and concrete base (W)				**24.79**
	Total				**105.54** *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.30	3.79	1.14	**19.32**
	Wedge C - Topsoil below GWL	0.00	3.79	0.00	**0.00**
	Wedge D - backfill above GWL	0.70	3.79	2.65	**55.70**
	Wedge D - Backfill below GWL	0.64	3.79	2.44	**9.68**
	Wedge E - Backfill below GWL	0.95	2.21	2.09	**8.29**
	Weight of tank and concrete base (W)				**24.79**
	Total				**117.78**

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30
0.44	14.7	34
0.44	6.3	34









H = 0.945065

Factor of Safety (FOS) against uplift FOS = 2.18

$$Fs = \frac{W+Q}{Us+Ud+F}$$

Assessment of Liquefaction Induced Uplift of Chambers — Trial DESIGN assumptions

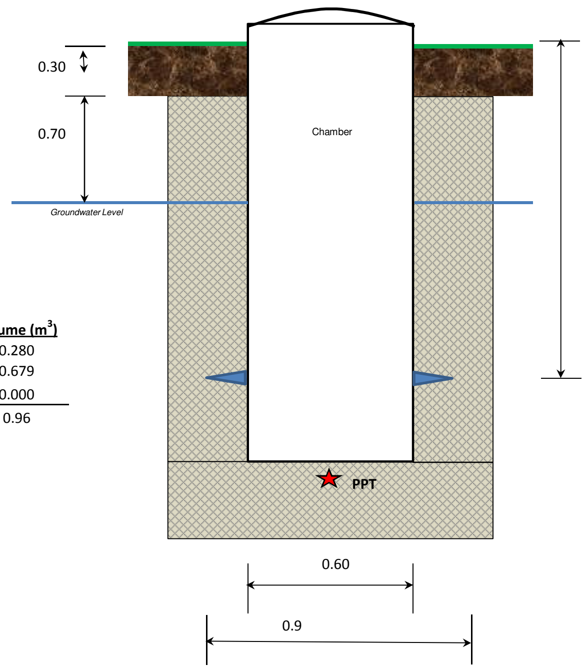
Assessment Trial No. 6 - PE manhole with CCC AP65 backfill

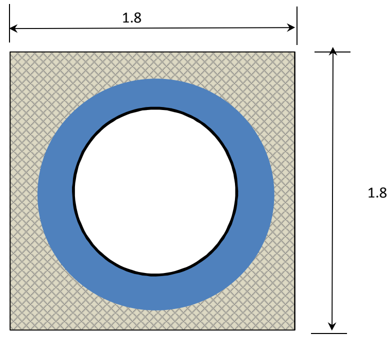
mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

Volume of chamber	0.96	m^3
Volume of chamber below GWL	0.47	m^3
Volume of concrete ring foundation	0.04	m^3
Unsaturated Unit weight of backfill	21	kN/m^3
Saturated Unit weight of backfill	22	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of PE flange	18	kN/m^3
Depth of Piezometer	2.53	m
Peak Net Piezometer Pressure	20.72	kPa
Equivalent Pore water Pressure within backfill	18.0	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	621	kg
Weight of concrete foundation	65	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.28	m^2
Typical width of chamber	0.600	m
Base width of chamber	0.6	m
Depth to base of ring	1.8	m
Depth to base of chamber	2.47	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	0.1	m
Diameter of base flange	0.9	m
Width of backfill a	1.80	m
Width of backfill b	1.80	m
Depth to ground water level	1.00	m
Liquefaction within Backfill (Y/N)	N	

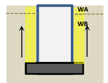
Item	Weight (kg)	Volume (m^3)
Lid	541	0.280
Riser	80	0.679
Base	0	0.000
	621	0.96

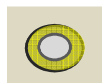




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				4.92
	Seismic: Excess pore pressure				5.86
	Seepage Force assumed to be 5% of Buoyant force				0.54
	Total				11.32
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.35	0.11	1.80
	Wedge A (W) - topsoil below GWL	0.00	0.35	0.00	0.00
	Wedge B (W) - backfill above GWL	0.70	0.35	0.25	5.20
	Wedge B (W) - backfill below GWL	0.70	0.35	0.25	0.99
	Shear resistance within topsoil - above GWL				0.62
	Shear resistance within topsoil - below GWL				0.00
	Shear resistance within granular backfill - above GWL				7.33
	Shear resistance within granular backfill - below GWL				12.48
	Weight of tank and concrete base (W)				6.73
	Total				35.14 *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.30	2.26	0.68	11.54
	Wedge C - Topsoil below GWL	0.00	2.26	0.00	0.00
	Wedge D - backfill above GWL	0.55	2.26	1.25	26.30
	Wedge D - Backfill below GWL	0.00	2.26	0.00	0.00
	Wedge E - Backfill below GWL	0.85	1.31	1.11	4.43
	Weight of tank and concrete base (W)				6.73
	Total				48.99

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30
0.44	14.7	34
0.44	2.8	34





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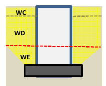
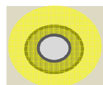




Factor of Safety (FOS) against uplift FOS = 3.10

$$Fs=\frac{W+Q}{Us+Ud+F}$$

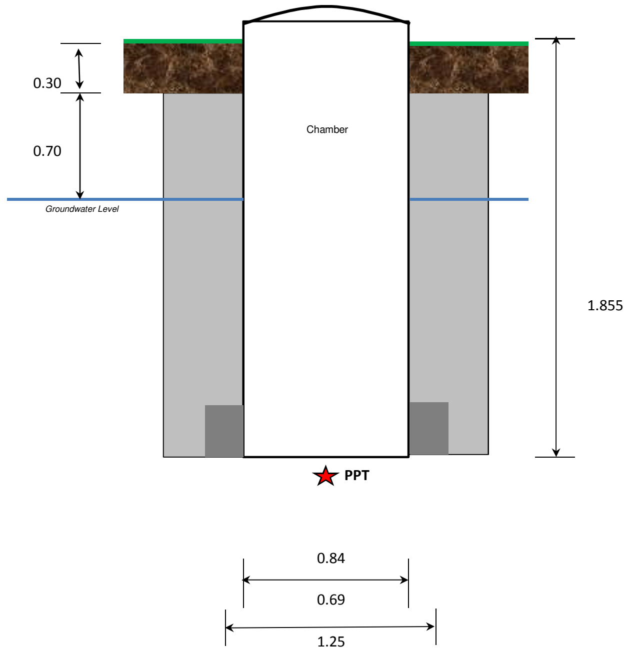
Assessment of Liquefaction Induced Uplift of Chambers   Trial DESIGN assumptions

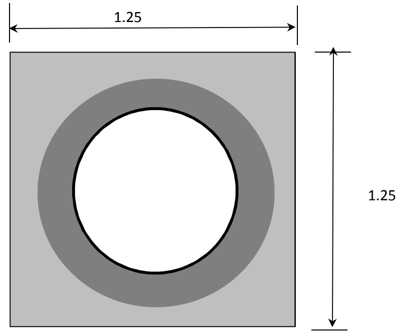
Assessment Trial No. 7 - Pressure sewer with concrete backfill

mfg   22/01/2014

Assessment of FOS to Buoyant Uplift

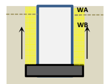
Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	0.47	m^3
Volume of concrete foundation	1.57	m^3
Unsaturated Unit weight of backfill	17	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	22	kN/m^3
Depth of Piezometer	1.9	m
Peak Net Piezometer Pressure	15.6	kPa
Equivalent Pore water Pressure within backfill	18.0	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	3514	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.55	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.855	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	1.555	m
Diameter of base flange	1.25	m
Width of backfill a	1.25	m
Width of backfill b	1.25	m
Depth to ground water level	1.00	m
Liquefaction within Backfill (Y/N)	N	

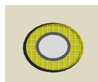




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**20.03**
	Seismic: Excess pore pressure				**8.65**
	Seepage Force assumed to be 5% of Buoyant force				**1.43**
	Total				**30.11**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.67	0.20	**3.43**
	Wedge A (W) - topsoil below GWL	0.00	0.67	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.70	0.67	0.47	**8.00**
	Wedge B (W) - backfill below GWL	-0.70	0.67	-0.47	**0.01**
	Shear resistance within topsoil - above GWL				**0.87**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**9.03**
	Shear resistance within granular backfill - below GWL				**0.00**
	Weight of tank and concrete base (W)				**35.21**
	Total				**56.55**
Resisting force - Method B	Wedge C - topsoil above GWL	0.30	0.67	0.20	**3.43**
	Wedge C - Topsoil below GWL	0.00	0.67	0.00	**0.00**
	Wedge D - backfill above GWL	0.00	0.67	0.00	**0.00**
	Wedge D - Backfill below GWL	0.00	0.67	0.00	**0.00**
	Wedge E - Backfill below GWL	0.00	0.67	0.00	**0.00**
	Weight of tank and concrete base (W)				**35.21**
	Total				**38.64** *Critical Case*

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30





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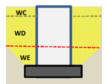
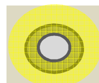




Factor of Safety (FOS) against uplift   FOS = 1.28

$$Fs=\frac{W+Q}{Us+Ud+F}$$

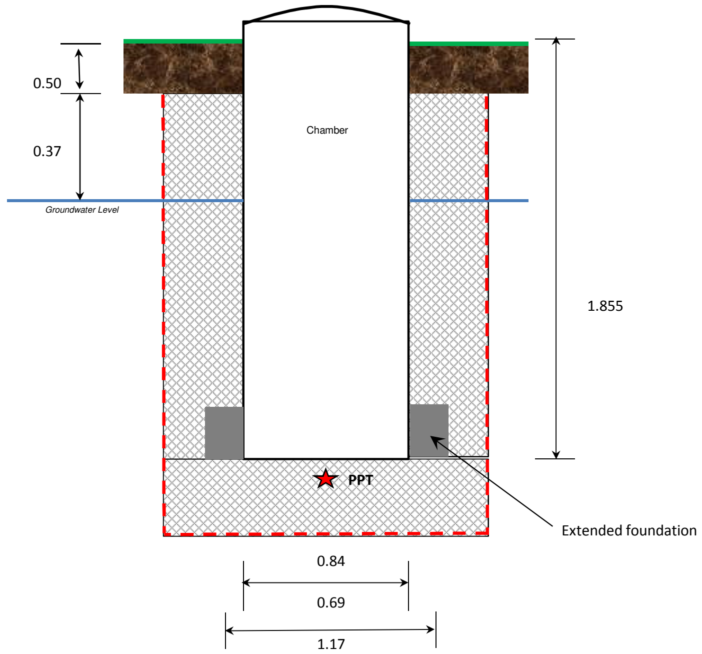
Assessment of Liquefaction Induced Uplift of Chambers **ASBUILT Condition and measured uplift pressure**

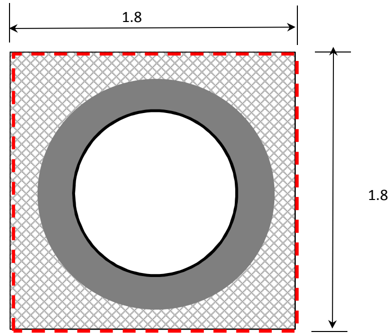
Assessment Trial No. 3 - Pressure sewer with permeable backfill - initial

mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

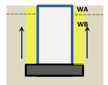
Volume of pressure sewer chamber	1.03	m^3	
Volume of pressure sewer chamber below GWL	0.55	m^3	
Volume of concrete ring foundation	0.14	m^3	
Unsaturated Unit weight of backfill	18	kN/m^3	
Saturated Unit weight of backfill	18	kN/m^3	
Unsaturated unit weight of topsoil	17	kN/m^3	
Saturated unit weight of topsoil	18	kN/m^3	
Unit weight of concrete	24.5	kN/m^3	
Depth of Piezometer	1.97	m	
Peak Net Piezometer Pressure	9.5	kPa	0.97 m
Equivalent Pore water Pressure within backfill	14.6	kN/m^3	*Liquefied = ~18kN/m^3*
Weight of water within Chamber	39	kg	0.38
Weight of Chamber and Equipment	75	kg	0.74
Weight of concrete foundation	359	kg	3.52
Angle of friction of backfill	32	°	
Angle of friction of topsoil	30	°	
Plan area of chamber	0.55	m^2	
Typical width of chamber	0.84	m	
Base width of chamber	0.69	m	
Depth to base of chamber	1.855	m	
Thickness of Topsoil	0.5	m	
Height of concrete base ring	0.205	m	
Diameter of base flange	1.17	m	
Width of backfill a	1.80	m	
Width of backfill b	1.80	m	
Depth to ground water level	0.87	m	
Liquefaction within Backfill (Y/N)	N		

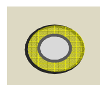




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Static Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**6.77**
	Seismic: Excess pore pressure				**5.27**
	Seepage Force assumed to be 5% of Buoyant force				**0.60**
	Total				**12.64**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.50	0.52	0.26	**4.42**
	Wedge A (W) - topsoil below GWL	0.00	0.52	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.37	0.52	0.19	**3.47**
	Wedge B (W) - backfill below GWL	0.78	0.52	0.41	**1.37**
	Shear resistance within topsoil - above GWL				**2.25**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**4.73**
	Shear resistance within granular backfill - below GWL				**13.87**
	Weight of tank and concrete base (W)				**4.64**
	Total				**34.75** *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.50	1.99	0.99	**16.91**
	Wedge C - Topsoil below GWL	0.00	1.99	0.00	**0.00**
	Wedge D - backfill above GWL	0.37	1.99	0.74	**13.25**
	Wedge D - Backfill below GWL	0.21	1.99	0.42	**1.42**
	Wedge E - Backfill below GWL	0.57	1.26	0.71	**2.40**
	Weight of tank and concrete base (W)				**4.64**
	Total				**38.63**

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	8.5	30
0.50	0	30
0.47	6.66	32
0.47	2.6	32





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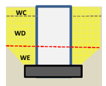
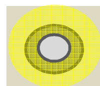




Factor of Safety (FOS) against uplift FOS = 2.75

$$Fs = \frac{W+Q}{Us+Ud+F}$$

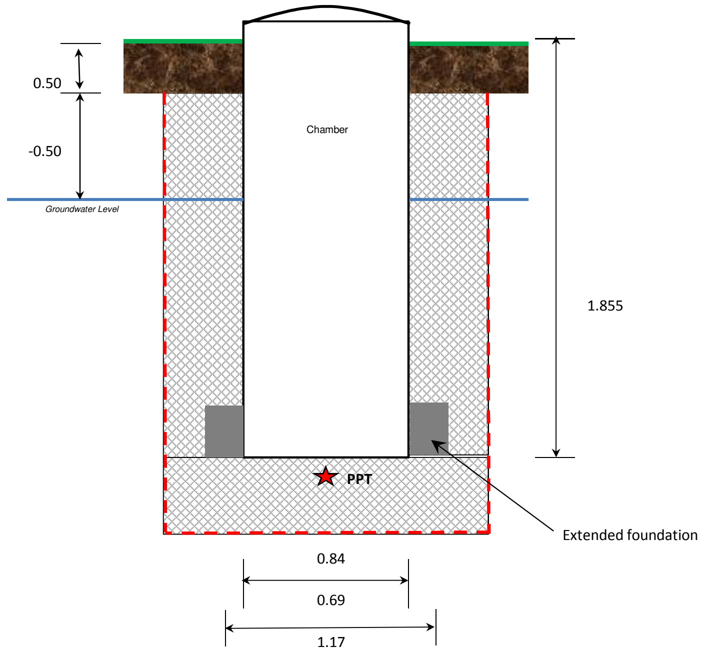
Assessment of Liquefaction Induced Uplift of Chambers   ASBUILT Condition and measured uplift pressure

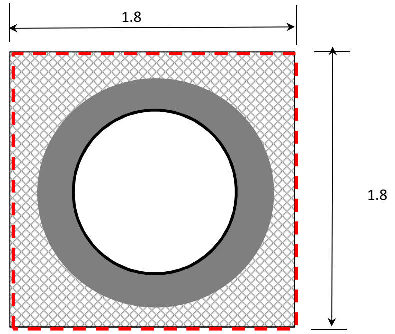
Assessment Trial No. 3 - Pressure sewer with permeable backfill - final, pressure equalisation

mfg   22/01/2014

Assessment of FOS to Buoyant Uplift

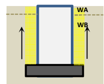
Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	1.03	m^3
Volume of concrete ring foundation	0.14	m^3
Unsaturated Unit weight of backfill	18	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Piezometer	1.97	m
Peak Net Piezometer Pressure	0	kPa
Equivalent Pore water Pressure within backfill	9.8	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	39	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	359	kg
Angle of friction of backfill	32	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.55	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.855	m
Thickness of Topsoil	0.5	m
Height of concrete base ring	0.205	m
Diameter of base flange	1.17	m
Width of backfill a	1.80	m
Width of backfill b	1.80	m
Depth to ground water level	0.00	m
Liquefaction within Backfill (Y/N)	N	

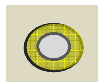


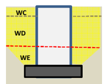


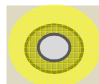
Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**11.50**
	Seismic: Excess pore pressure				**0.00**
	Seepage Force assumed to be 5% of Buoyant force				**0.00**
	Total				**11.50**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.00	0.52	0.00	**0.00**
	Wedge A (W) - topsoil below GWL	0.50	0.52	0.26	**2.13**
	Wedge B (W) - backfill above GWL	0.00	0.52	0.00	**0.00**
	Wedge B (W) - backfill below GWL	1.15	0.52	0.60	**4.90**
	Shear resistance within topsoil - above GWL				**0.00**
	Shear resistance within topsoil - below GWL				**1.09**
	Shear resistance within granular backfill - above GWL				**0.00**
	Shear resistance within granular backfill - below GWL				**13.47**
	Weight of tank and concrete base (W)				**4.64**
	Total				**26.23** *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.00	1.99	0.00	**0.00**
	Wedge C - Topsoil below GWL	0.50	1.99	0.99	**8.15**
	Wedge D - backfill above GWL	0.00	1.99	0.00	**0.00**
	Wedge D - Backfill below GWL	0.58	1.99	1.16	**9.48**
	Wedge E - Backfill below GWL	0.57	1.26	0.71	**5.84**
	Weight of tank and concrete base (W)				**4.64**
	Total				**28.11**

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	0	30
0.50	4.095	30
0.47	0	32
0.47	13.5	32









H = 0.568275

Factor of Safety (FOS) against uplift   FOS = 2.28

$$Fs = \frac{W+Q}{Us+Ud+F}$$

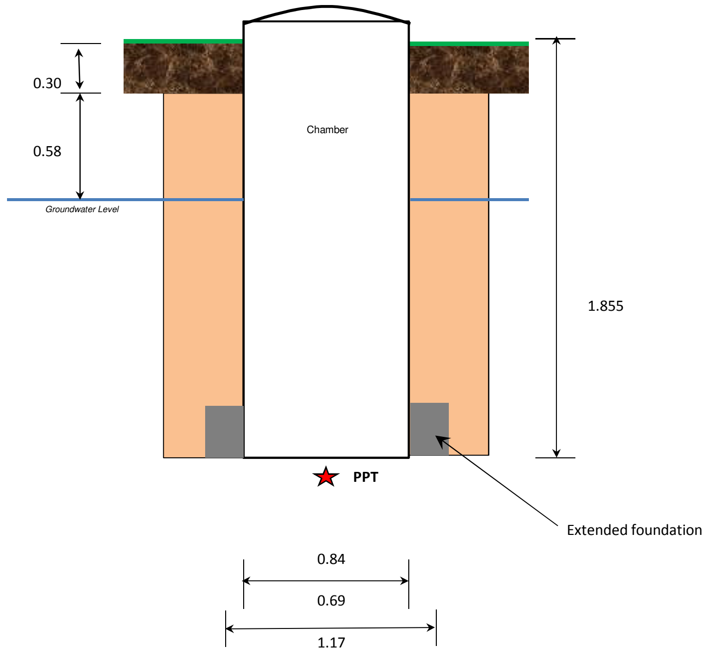
ASBUILT Condition and measured uplift pressure

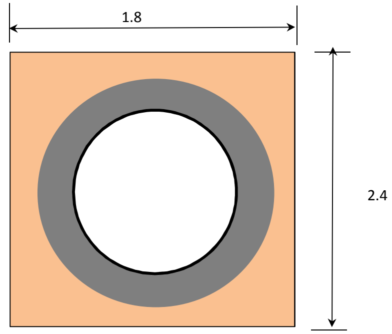
Assessment Trial No. 4 - Pressure sewer with natural backfill

mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

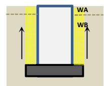
Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	0.54	m^3
Volume of concrete ring foundation	0.14	m^3
Unsaturated Unit weight of backfill	17	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Piezometer	1.9	m
Peak Net Piezometer Pressure	25	kPa
Equivalent Pore water Pressure within backfill	23.0	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	30	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	359	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.55	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.855	m
Thickness of Topsoil	0.3	m
Height of concrete base ring	0.205	m
Diameter of base flange	1.17	m
Width of backfill a	1.80	m
Width of backfill b	2.40	m
Depth to ground water level	0.88	m
Liquefaction within Backfill (Y/N)	Y	

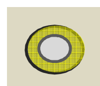




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**6.72**
	Seismic: Excess pore pressure				**13.87**
	Seepage Force assumed to be 5% of Buoyant force				**1.03**
	Total				**21.61**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.30	0.52	0.16	**2.65**
	Wedge A (W) - topsoil below GWL	0.00	0.52	0.00	**0.00**
	Wedge B (W) - backfill above GWL	0.58	0.52	0.30	**5.13**
	Wedge B (W) - backfill below GWL	0.77	0.52	0.40	**0.00**
	Shear resistance within topsoil - above GWL				**0.81**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**6.36**
	Shear resistance within granular backfill - below GWL				**0.00**
	Weight of tank and concrete base (W)				**4.55**
	Total				**19.51**
Resisting force - Method B					

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	5.1	30
0.50	0	30





Factor of Safety (FOS) against uplift FOS = 0.90

$$Fs=\frac{W+Q}{Us+Ud+F}$$

 ASBUILT Condition and measured uplift pressure

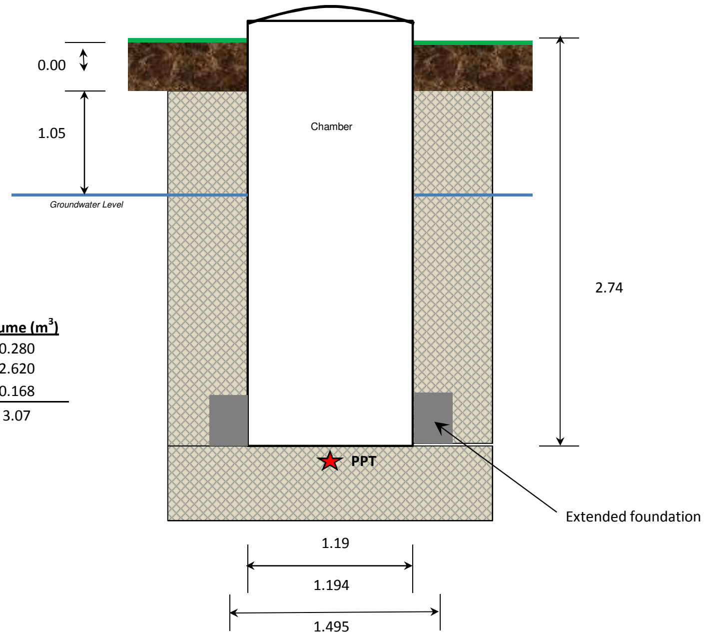
Assessment Trial No. 5 - Standard precast concrete manhole with CCC AP65 backfill

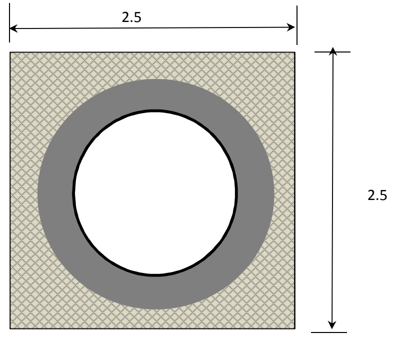
mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

Volume of chamber	3.07	m^3
Volume of chamber below GWL	1.89	m^3
Volume of concrete ring foundation	0.10	m^3
Unsaturated Unit weight of backfill	21	kN/m^3
Saturated Unit weight of backfill	22	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	24.5	kN/m^3
Depth of Peizometer	2.82	m
Peak Net Piezometer Pressure	8.8	kPa
Equivalent Pore water Pressure within backfill	12.9	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	2289	kg
Weight of concrete foundation	238	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	1.12	m^2
Typical width of chamber	1.194	m
Base width of chamber	1.194	m
Depth to base of chamber	2.74	m
Thickness of Topsoil	0	m
Height of concrete base ring	0.15	m
Diameter of base flange	1.495	m
Width of backfill a	2.50	m
Width of backfill b	2.50	m
Depth to ground water level	1.05	m
Liquefaction within Backfill (Y/N)	N	

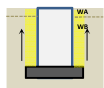
Item	Weight (kg)	Volume (m^3)
Lid	541	0.280
Riser	1328	2.620
Base	419	0.168
	2289	3.07

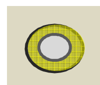




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**19.50**
	Seismic: Excess pore pressure				**9.85**
	Seepage Force assumed to be 5% of Buoyant force				**1.47**
	Total				**30.82**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.00	0.64	0.00	**0.00**
	Wedge A (W) - topsoil below GWL	0.00	0.64	0.00	**0.00**
	Wedge B (W) - backfill above GWL	1.05	0.64	0.67	**14.01**
	Wedge B (W) - backfill below GWL	1.54	0.64	0.98	**8.88**
	Shear resistance within topsoil - above GWL				**0.00**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**16.17**
	Shear resistance within granular backfill - below GWL				**62.44**
	Weight of tank and concrete base (W)				**24.79**
	Total				**126.28** *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.00	3.79	0.00	**0.00**
	Wedge C - Topsoil below GWL	0.00	3.79	0.00	**0.00**
	Wedge D - backfill above GWL	1.05	3.79	3.98	**83.54**
	Wedge D - Backfill below GWL	0.59	3.79	2.25	**20.44**
	Wedge E - Backfill below GWL	0.95	2.21	2.09	**18.96**
	Weight of tank and concrete base (W)				**24.79**
	Total				**147.74**

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	0	30
0.50	0	30
0.44	22.05	34
0.44	14.0	34





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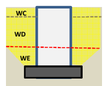
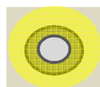




Factor of Safety (FOS) against uplift FOS = 4.10

$$Fs=\frac{W+Q}{Us+Ud+F}$$

Assessment of Liquefaction Induced Uplift of Chambers — ASBUILT Condition and measured uplift pressure

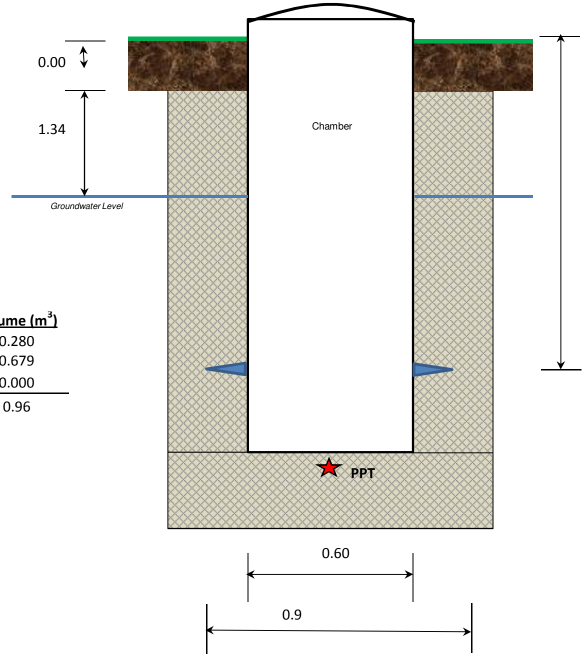
Assessment Trial No. 6 - PE manhole with CCC AP65 backfill

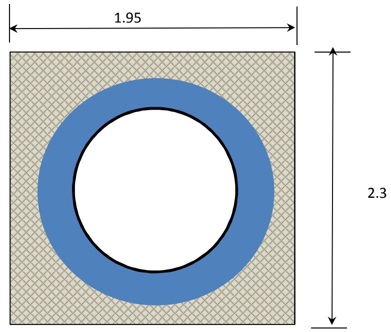
mfg 22/01/2014

Assessment of FOS to Buoyant Uplift

Volume of chamber	0.96	m^3
Volume of chamber below GWL	0.37	m^3
Volume of concrete ring foundation	0.04	m^3
Unsaturated Unit weight of backfill	21	kN/m^3
Saturated Unit weight of backfill	22	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of PE flange	18	kN/m^3
Depth of Piezometer	2.53	m
Peak Net Piezometer Pressure	14	kPa
Equivalent Pore water Pressure within backfill	15.3	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	0	kg
Weight of Chamber and Equipment	621	kg
Weight of concrete foundation	65	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.28	m^2
Typical width of chamber	0.600	m
Base width of chamber	0.6	m
Depth to base of ring	1.8	m
Depth to base of chamber	2.47	m
Thickness of Topsoil	0	m
Height of concrete base ring	0.1	m
Diameter of base flange	0.9	m
Width of backfill a	1.95	m
Width of backfill b	2.30	m
Depth to ground water level	1.34	m
Liquefaction within Backfill (Y/N)	N	

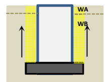
Item	Weight (kg)	Volume (m^3)
Lid	541	0.280
Riser	80	0.679
Base	0	0.000
	621	0.96

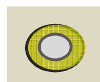




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**3.98**
	Seismic: Excess pore pressure				**3.96**
	Seepage Force assumed to be 5% of Buoyant force				**0.40**
	Total				**8.34**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.00	0.35	0.00	**0.00**
	Wedge A (W) - topsoil below GWL	0.00	0.35	0.00	**0.00**
	Wedge B (W) - backfill above GWL	1.34	0.35	0.47	**9.95**
	Wedge B (W) - backfill below GWL	0.36	0.35	0.13	**0.85**
	Shear resistance within topsoil - above GWL				**0.00**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**15.85**
	Shear resistance within granular backfill - below GWL				**8.88**
	Weight of tank and concrete base (W)				**6.73**
	Total				**42.25** *Critical Case*
Resisting force - Method B	Wedge C - topsoil above GWL	0.00	3.26	0.00	**0.00**
	Wedge C - Topsoil below GWL	0.00	3.26	0.00	**0.00**
	Wedge D - backfill above GWL	0.55	3.26	1.79	**37.56**
	Wedge D - Backfill below GWL	0.00	3.26	0.00	**0.00**
	Wedge E - Backfill below GWL	1.15	1.81	2.08	**13.87**
	Weight of tank and concrete base (W)				**6.73**
	Total				**58.16**

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	0	30
0.50	0	30
0.44	28.14	34
0.44	2.4	34





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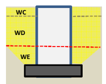
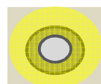




Factor of Safety (FOS) against uplift FOS = 5.07

$$Fs = \frac{W+Q}{Us+Ud+F}$$

Assessment of Liquefaction Induced Uplift of Chambers   **ASBUILT Condition and measured uplift pressure**

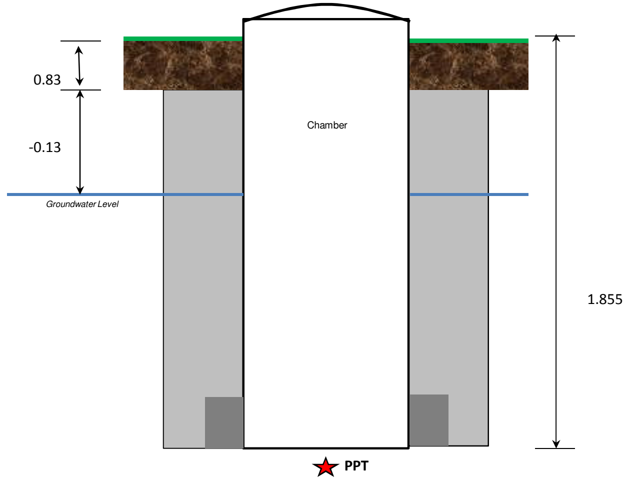
Assessment Trial No. 7 - Pressure sewer with concrete backfill

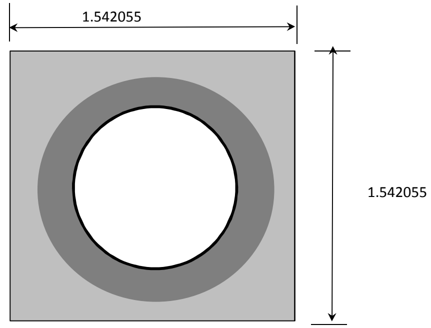
mfg   22/01/2014

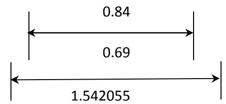
Assessment of FOS to Buoyant Uplift

Volume of pressure sewer chamber	1.03	m^3
Volume of pressure sewer chamber below GWL	0.64	m^3
Volume of concrete ring foundation	1.35	m^3
Unsaturated Unit weight of backfill	17	kN/m^3
Saturated Unit weight of backfill	18	kN/m^3
Unsaturated unit weight of topsoil	17	kN/m^3
Saturated unit weight of topsoil	18	kN/m^3
Unit weight of concrete	15.2	kN/m^3 Lab measured
Depth of Piezometer	1.9	m
Peak Net Piezometer Pressure	30	kPa
Equivalent Pore water Pressure within backfill	25.6	kN/m^3 *Liquefied = ~18kN/m³*
Weight of water within Chamber	19	kg
Weight of Chamber and Equipment	75	kg
Weight of concrete foundation	2085	kg
Angle of friction of backfill	34	°
Angle of friction of topsoil	30	°
Plan area of chamber	0.55	m^2
Typical width of chamber	0.84	m
Base width of chamber	0.69	m
Depth to base of chamber	1.855	m
Thickness of Topsoil	0.83	m
Height of concrete base ring	1.025	m
Diameter of base flange	1.54	m
Width of backfill a	1.54	m
Width of backfill b	1.54	m
Depth to ground water level	0.70	m
Liquefaction within Backfill (Y/N)	N	

Equivalent diameter (1.57 + 1.78)/2 by (1.13+1.10)/2
Area = 1.87 m^2
Equivalent diameter = 1.54 m

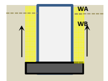


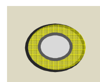




Action	Description	Layer Height (m)	Average Area (m^2)	Average Volume (m^3)	Force (kN)
Factored Uplift force	Buoyant Force: Weight of fluid displaced by impermeable components of chamber				**19.49**
	Seismic: Excess pore pressure				**16.64**
	Seepage Force assumed to be 5% of Buoyant force				**1.81**
	Total				**37.93**
Resisting force - Method A	Wedge A (W) - topsoil above GWL	0.70	1.31	0.92	**15.62**
	Wedge A (W) - topsoil below GWL	0.13	1.31	0.17	**0.00**
	Wedge B (W) - backfill above GWL	0.00	1.31	0.00	**0.00**
	Wedge B (W) - backfill below GWL	0.00	1.31	0.00	**0.00**
	Shear resistance within topsoil - above GWL				**1.08**
	Shear resistance within topsoil - below GWL				**0.00**
	Shear resistance within granular backfill - above GWL				**0.00**
	Shear resistance within granular backfill - below GWL				**0.00**
	Weight of tank and concrete base (W)				**21.38**
	Total				**38.08**
Resisting force - Method B	Wedge C - topsoil above GWL	0.70	1.31	0.92	**15.62**
	Wedge C - Topsoil below GWL	0.13	1.31	0.17	**0.00**
	Wedge D - backfill above GWL	0.00	1.31	0.00	**0.00**
	Wedge D - Backfill below GWL	0.00	1.31	0.00	**0.00**
	Wedge E - Backfill below GWL	0.00	1.31	0.00	**0.00**
	Weight of tank and concrete base (W)				**21.38**
	Total				**37.00** *Critical Case*

K_o	$\Delta\sigma_v'$ (kPa)	δ (°)
0.50	11.9	30
0.50	0	30





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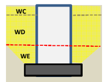

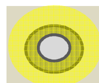

Factor of Safety (FOS) against uplift   FOS = 0.98

$$Fs = \frac{W+Q}{Us+Ud+F}$$