24 April 2013

UNDERPASS LATERAL SPREADING

Lateral Spreading Assessment and Mitigation Requirements

UND-02-DES-RP-009

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Name	Position	Date	Signature
[redacted] s9(2)(a)	Geotechnical Manager	24/04/2013	

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1. INTRODUCTION

This report summarises an assessment of likely lateral spreading risk for the underpass structure. It also provides recommendations for the mitigation of lateral spreading effects on the underpass structure.

1.1 Liquefaction Susceptibility

Geotechnical investigations have been completed for the project. Geological observations, lab testing and industry standard prediction methods indicate that some of the alluvial soils below groundwater may be susceptible to liquefaction. However a variable, layered deposition sequence including layers of medium plasticity, indicate liquefaction is likely to occur only within isolated layers.

The geological setting, specific investigation data and a specific assessment of liquefaction risk (for the underpass) are described in the following documents:

- Geotechnical Factual Report (GFR) UND-02-DES-RP-001
- Geotechnical Interpretive Report (GIR) UND-02-DES-RP-002
- Geological Assessment of Lateral Spreading Memo UND-Memo-02-014 (Appendix C)
- Liquefaction susceptibility assessment UND-02-DES-RP-014 (Appended to GIR).

1.2 Slope Geometry

The Underpass alignment bisects a deeply weathered greywacke ridge. The ridge falls south to north and is centred some 65m west of Tory Street. Each shoulder of the ridge is mantled by mainly alluvial deposits, consisting of silts, sands and silty gravels.

The eastern ground slope, falls down to the Basin reserve at between 2 to 4°.

The western ground slope falls to the shallow Taranaki Street basin floor at approximately 2°.

Groundwater within the ridge and shoulders is typically at 3 to 4m depth. Groundwater conditions are described in UND-02-DES-RP-010.

1.3 Lateral Spreading

The potential for lateral spreading of the eastern and western slopes has been considered due to the liquefaction susceptibility of alluvial and marginal marine soils underlying each end of the underpass and trenches. Historic silt filled cracks have been observed on the site that are considered to be related to historic lateral spreading of the eastern slope only.

Lateral spreading is where land displaces towards a free edge i.e. a stream. This occurs due to a loss of strength (liquefaction) combined with lateral forces exerted by earthquake. Movement can occur on slopes at very low gradients.

Evidence of previous lateral spreading is described in UND-Memo-02-014, appended.

1.4 Determining Lateral Spreading Risk for Underpass

Three methods have been considered to assess the Lateral Spreading risk for the underpass. These are:

- Site Precedent, based on total lateral displacement of ground since deposition.
- Back Analysis of 1855 Wairarapa Earthquake where negligible displacement occurred.
- Empirical Assessment.

The site precedent approach is based on the measurement and dating of historic lateral displacement preserved within the geological sequence at the site. This combination of displacement history and frequency allows a prediction to be made of likely future risk.

Back Analysis of the slope has been undertaken as a sensitivity check. The slope did not suffer any significant lateral displacement in the Wairarapa earthquake event. The regional uplift in this event was 1.5m. An approximation (based on empirical relationships) is required to apply back analysis derived strengths to larger design events where liquefied soil strengths will inevitably be reduced. For this reason the back analysis method is considered only as a sensitivity check to the site precedent approach.

Empirical based analysis has also been considered. However these methods are unlikely to be applicable to the combination of the site conditions and large design accelerations for this project. The results of the empirical assessment do not correlate with the historical performance of the site, predicting significantly higher displacements than are evident in the geological deposits.

This report discusses each of the methods above and provides recommendations for the design of underpass with respect to lateral spreading risk and effects.

2. SITE PRECEDENT

Evidence of lateral displacement has been observed on the site. The upper alluvial / marginal marine layers on each side of the rock ridge contain sufficient organic materials that allow us to put some date bounds (by Radiocarbon dating) on the deposits and the infill of the cracks. The total displacement of the observed cracks has been recorded. These measures provide some indication of how the site has behaved in previous seismic events.

Site precedent is considered a robust method to estimate future risk provided:

- Dates of deposits and displacement can be reasonably bounded.
- Displacement can be reasonably measured.
- The slope profile at the time of historic displacement is known (or is at least no flatter now, than it was at time of historic displacement).
- The ground water conditions are the same or higher (relative to ground surface) than when the historic displacement occurred.

2.1 Method

The following is a summary of tasks completed to estimate lateral spreading risk:

- Geological investigations to assess the geological sequence and material on the site.
- Mapping, Investigation and measurement of the observed silt filled fissures (cracks).
- Radio carbon dating of organic materials within specific alluvial layers and within the crack infill material.
- Investigate the likely seismic history of the site (published data) including historic uplift by regional tilting.
- Assess how the topography and groundwater conditions may have changed over the observed history of displacement, and provide conservative estimates of potential variations.
- Back analyse the slope under likely seismic loading scenarios (of various return periods) to assess sensitivity to topographic changes and provide bounds for likely liquefied soil strengths.
- Assess possible historic scenarios that may have caused the cracking.

- Use the likely history of the site to estimate likely future lateral spreading risk for the underpass structure.

2.2 Evidence of past lateral spreading at this site

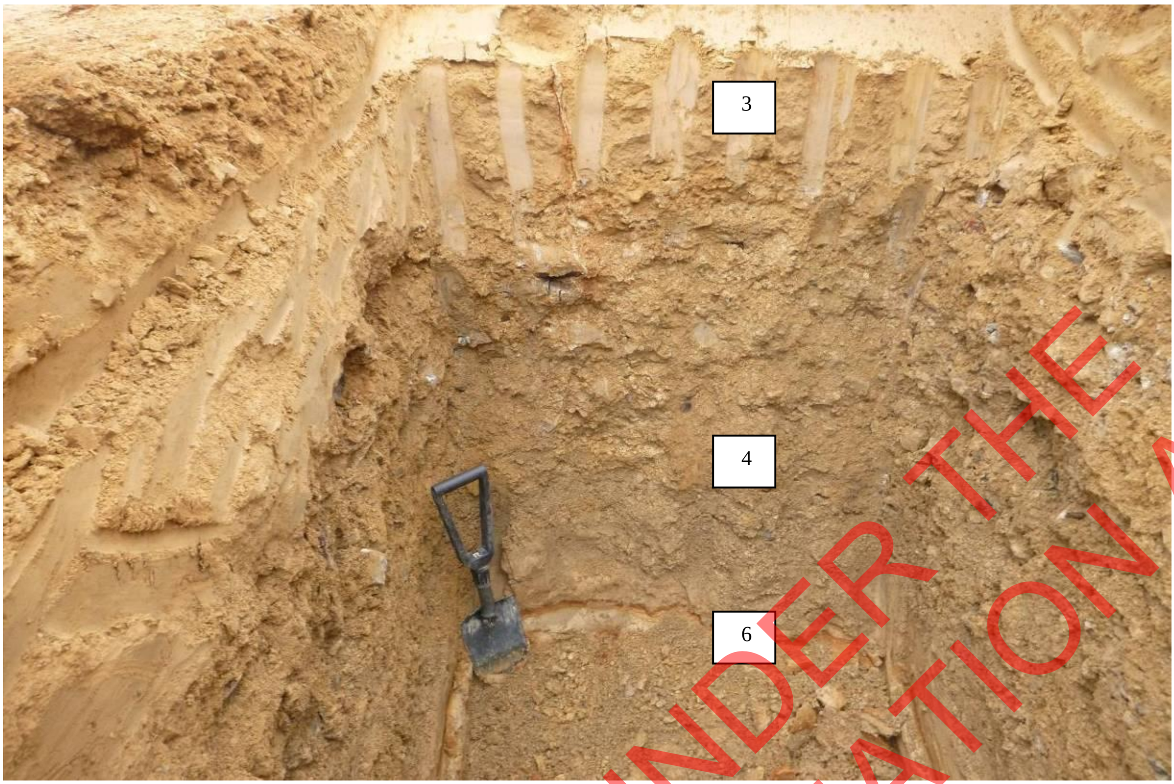
Excavations for the temporary road (to the north of the proposed underpass and to the east of Tory Street), uncovered silt filled cracks within several of the alluvial / marginal marine soil layers between approximately 0.8 to 2.2 metres depth. Subsequent investigations have been completed to track these features in the vicinity of the eastern end of the underpass and approach trench. Appendix A contains UND-Memo-002-014; a specific geological assessment of the features.

In summary, the geological sequence of materials in the area of cracking is as shown in Table 01:

Table 01 Geological Sequence in area of observed historical lateral spreading

Layer No	Material Description	Inferred Origin	Depth below pre construction surface	Thickness
1	Grey gravely SILT / silty gravel	Man Made Fill	0m	0.2m
2a	Light grey and orange mottled gravely SILT with completely weathered greywacke clasts (orange sand)	Reworked Colluvium / Alluvial Fan	0.2m	0.4m
2b	Becoming silty gravel with fine to coarse, moderately weathered sub angular gravels at base. Heavy iron staining at base. Undulating basal surface.	Less weathered Colluvium / Alluvial Fan	0.6m	0.2m
3	Uniform, brown silty SAND / sandy SILT. Undulating basal surface	Marginal Marine Environment (backwater, estuarine, lacustrine)	0.8m	0.4m
4	Silty fine to coarse GRAVEL with minor light grey plastic silt in pockets. Becoming sandier towards the base. Undulating basal surface.	Alluvial Fan	1.2m	1.0m
5	Silty, brown fine SAND. The layer is locally disturbed and deformed.	Alluvium / Marginal Marine Environment	2.2m	0.1m
6	Light grey / brown SILT with heavy iron staining at top and bottom of layer	Alluvium / Marginal Marine Environment	2.3m	0.2m
7	Sandy silty fine to coarse GRAVEL	Alluvial Fan	2.5m	1m

Layers containing cracks



Photograph 1: Alluvium and Marginal Marine Deposit sequence – TPTT7

Photograph 2: Alluvium and Marginal Marine Deposit sequence – TPTT8

Observations of note include:

- Cracks are laterally extensive over tens of metres and are in filled with light grey, cohesive, plastic, organic silt. The cracks are up to 40mm wide and are sub vertical.
- There is negligible off set of the alluvial / marginal marine soil layers in either a horizontal or vertical plane, i.e. they are a pull apart or tension feature.

Photograph 3: In filled historic tension cracks.

- In filled tension cracks extend sub vertically through alluvial layers approximately between 0.8 and 2.2m depth.

- The cracks are very similar in appearance to excavated lateral spreading cracks investigated post Christchurch Earthquake.
- The cracks through the alluvial layers that show the most prominent cracks have higher relative plasticity than the surrounding material and smaller cracks, and they are likely to have been maintained as an open tension crack for a period of time. This is supported by the observation that the cracks are uniformly filled with silt at the top, with only minor side collapse of the cracks.
- Within the alluvial gravel layers, the silt filled crack becomes less pronounced with depth. Towards the bottom, the silts have accumulated around the gravel clasts, indicating more open (or partially collapsed) gravel that has been in filled, rather than a discrete silt filled crack.
- The cracks are aligned sub parallel. They all trend in the same relative direction. Cracking is orientated between approximately 270°-90°, or 290°-110°. The cracks are generally aligned at 270° at the eastern end of the temporary road, and become orientated to 290° at the western end. The location and distribution of cracks is shown on sketch EW-02-902 (Appendix A).
- Crosscutting and dislocation of cracks is visible in several places. This infers that two separate displacement events could have occurred since these alluvial soils have been deposited. However, the bifurcated and sinuous nature of some of the cracks suggests that the cracks are likely to be from one event, or two close events (i.e. perhaps an event and an aftershock).
- The cracks are variable in length, width and vertical extent. Not all the cracks extend to the same depth.
- The middle of the cracks are light grey in colour, however in nearly all cases, there is dark orange iron staining on both side walls, indicating water movement. On drying, the light grey crack infill silts turn brown, highlighting an organic component.

At the eastern end, total displacement of the cracks equates to a total strain of less than 3% in a north to south direction. There is no evidence of any lateral displacement at the eastern end in an east-west direction. There is no evidence of lateral spreading movement at the western end of the site.

2.2.1 Dating of deposits and crack infill

Radio carbon dating has been completed on organic samples obtained from various deposits above and below the cracking, as well as from the crack infill material itself. Table 02 sets to the results of the carbon dating and provides comment on the results.

Table 02 Radio carbon dating results

Layer (or equivalent layer)	Layer Description	Depth of sample in material sequence (see Table 1)	Age (rounded)	Comment
2a	Light grey and orange mottled gravelly silt	0.4m	10,000yrs	Reworked colluvium / alluvium contains reworked organic deposits. Thus age of organics in the layer do not necessarily represent the age of deposition in the current location.
6	Brown silt with trace organics	2.7m	25,000yrs	Alluvial / Marginal marine silts. Possibly contains some reworked alluvium like Layer 2a. Thus age of organics in the layer do not necessarily represent the age of layer deposition.
(3)	Crack infill within Layer 3 (marginal marine sands)	Approximately 0.9m	5,000yrs	Light brown cohesive organics within crack. Cracking orientation approximately 270°
(3)	Crack infill within Layer 3 (marginal marine sands)	0.9m	Testing in progress	Light brown cohesive organics within crack. Crack orientation approximately 278°
(3)	Crack infill within Layer 3 (marginal marine sands)	Approximately 0.9m	Testing in progress	Cracking orientation approximately 292°
3	Marginal marine Sands	Approximately 0.9m	Testing in progress	Brown silty sand. Main layer of cracking.
4	Alluvium. Silty fine to coarse gravel	Approximately 2m	Testing in progress	Sample was taken from the gravel layer rather than crack infill silts within the gravel layer.
Equivalent layer 3	Crack infill. Cohesive light grey and brown silt.	-	Testing in progress	Crack infill from west side of ridge where organics in the crack are visible.

As the assessment has proceeded it is recognised that further dating information would be useful to provide improved confidence in the age of the deposits and the crack infill. Further dating is currently underway on organic samples from layers 3 and 4, and also crack infill. The results will be revisited when this information is received (Expected 15/05/13).

Results to date indicate the cracks are in the order of 5000 years old.

2.2.2 Regional uplift & relative groundwater level

Prior to 1855, the basin reserve was a low lying lagoon or swampy area. This area was proposed to be a dock, connected by a canal cut along Kent / Cambridge Terrace. The Wellington CBD including Basin Reserve was uplifted by 1.5m in the 1855 earthquake and this proposal was abandoned as the area became drained.

Observed uplift of coastal wave cut platforms at the harbour entrance indicates a series of discrete uplift events of the Wellington area. There is evidence of uplift of terraces all around the Wellington coastline, for example at Pencarrow Head, Baring Head Tongue Point and most famously at Turakirae Head.

The uplift at Turakirae Head in the 1855 and 1460 earthquakes is measured to be approximately 2.5m and 6m respectively. Older uplifts are also recorded here, with three beaches recording uplift of 8.2m (approximately 3,100 years ago), 5.5m (approximately 4,900 years ago) and 2.7m (approximately 6,500 years ago). 24.9m of uplift has therefore occurred over 6500 years, equating to an average of 5m per event. Using a crude relationship (ratio of Basin uplift to Turakirae uplift) the Underpass site is likely to have risen in the order of 15m over the last 6500 years.

As sea level has remained relatively stable over the last 7000 years, the site has been rising relative to sea level since the deposit was formed.

As such, the relative groundwater levels in the deposits will have dropped with each uplift event. It is therefore reasonable to assume that relative groundwater levels were at higher levels when the liquefaction and lateral spreading occurred.

This hypothesis is also supported by the type of material deposition. For example the marginal marine deposits – (Layer 3), which is now above groundwater was deposited within water (i.e. the sea).

Back analysis of historic events using the Site Precedent method relies on the assumption that the historic groundwater levels will be lower post construction than it was when the historic movement occurred. This assumption is strongly supported by the above observations.

2.2.3 Changes in site topography

The site precedent method relies on the assumption that the site topography will be no steeper post construction than it was when the displacement occurred (or conservative estimates of slope profile can be supported).

The following observations indicate that the upper to mid sections of the eastern slope are no steeper than they are currently:

- Indicator layers (those with consistent cracks) are relatively consistent in thickness and are draped by layer 2a which is also of a relatively consistent thickness. This indicates there has not been significant erosion of the slope since its deposition.
- The top of the ridge has been excavated down (west of Tory Street) to form the current Buckle Street.
- There have been no significant changes to the site topography since the 1855 Wairarapa earthquake and there is no evidence of any displacement in layer 2 which predates the Wairarapa earthquake (more certainty of age will be confirmed by additional radiocarbon dating).

There is the possibility (although considered unlikely) that the toe of the slope (to the north east of the crèche has eroded down since the historic deformation (approximately 5000 years ago) occurred. Investigations indicate that the layers of Holocene deposits are potentially truncated to the north east of the crèche. This may have occurred due to down cutting of the Cambridge Terrace/Basin stream or the formation of a wave cut platform during a higher relative sea level. Analysis of events older than recorded history should conservatively assume the toe of the Buckle Street slope projected at a consistent 2° slope further out onto the basin floor.

A summary of some of the key observations and level of certainty associated with these are provided in Appendix B.

2.2.4 Peak Ground Accelerations

Peak ground accelerations for historic and future events (based on GNS site Specific hazard assessment is shown in Table 3 below.

Table 03 Peak Ground Accelerations

Earthquake Event	Return period	Felt Intensity / Magnitude	Likely peak ground acceleration	Potential Lateral spreading displacement as inferred from observed cracking
Probabilistic*	1:500	N/A	0.50	N/A
Probabilistic*	1:1000	N/A	0.67	N/A
Probabilistic*	1:2500	N/A	0.90	Less than 0.25% in east-west direction, based on no apparent north-south movement since deposition >5000 yrs.

* based on GNS Seismic Hazard Assessment

2.2.5 Minimum liquefied Soil Shear Strengths to limit lateral spreading to observed magnitude

The slope has been back analysed to assess the minimum residual liquefied soil shear strengths that would have had to exist to limit historical slope displacements to those observed in the geological record. The following conservative assumptions have been made:

- Ground water levels were above current groundwater levels, assuming swampland in the Basin Reserve area.
- The toe of the Buckle Street slope extended at a constant 2° further out into the floor of the Basin valley.
- The total displacement of the eastern slope in an easterly direction was less than 200mm in a single 1 in 2500 year return period event. This is based on:
  - There will have been a minimum of 2 x 1 in 2500 year earthquake events since the deposits formed (and more likely 8 x 2500 year events over 20,000 years since deposition.
  - Displacement 200mm or greater in an east-west direction would be identifiable in the indicator geological layers (this will be validated during excavation of the underpass).

Using the methods of back analysis using Slope/W limit equilibrium and Jibson/Newmark sliding block analysis (2007) for displacements indicates that minimum (average over potential shear plane) liquefied soil shear strengths (in 1:2500 event) are 69kPa.

Using these liquefied soil shear strengths (1:2500 year) and reanalysing with a steeper toe (i.e. current & post WICI Buckle Street profile suggest that having a steeper toe than in the past has no effective increase on the potential lateral spreading risk compared to historical events and ground profiles.

Based on the geological evidence and assessment described above it is very unlikely that there has been more than 200mm of lateral displacement in an easterly or westerly direction.

The future risk is unlikely to be different from the historical risk. Given there has been no significant east-west lateral displacement over the age of these deposits (approx 20,000 years) it is very unlikely there will be any significant (more than 200mm) future lateral spreading displacement in the ULS design earthquake.

2.2.6 Why did the slope displace in the north-south direction and not to the east?

We have yet to determine if the observed historic cracking relates to displacement of the slope in:

- A northerly direction down slope to what would have been the foreshore or,
- A southerly direction to a now in filled gully parallel and to the south of Buckle Street.

Regardless of the direction, the total cumulative displacement (north-south direction) over 20,000 years has been less than 3.5%. This would have occurred on a steeper slope than currently exists and during times of significantly higher ground water. Therefore a ULS 2500 year event is likely to result in less than 0.5% strain (north-south).

It will therefore be conservative to design the underpass to tolerate up to 0.5% total permanent strain in the north-south alignment.

3. BACK ANALYSIS OF HISTORIC SEISMIC EVENTS

As a sensitivity check we have also undertaken back analysis of the slope under the likely loading scenarios of the 1855 earthquake. There are two recent historic events of interest where the response of the site was observed post European settlement:

- 1942 Masterton Earthquakes. Magnitude 6.8 and 7.2 events. These would have caused ground acceleration in Wellington roughly equivalent to a 1 in 25 year return period earthquake. There is no reported or observed evidence of ground displacement at this site, as a result of the 1942 events.
- 1855 Wairarapa Earthquake, while difficult to estimate, it is likely that this was a Magnitude 8.2 – 8.3 event. It is estimated to have had a felt intensity in Wellington of MMI 10. For the Underpass site this would have resulted in ground accelerations equivalent to at least a 1 in 500 year return period earthquake. There were general reports of "Numerous slump cracks occurred in flat areas of Wellington. Sand craters and liberation of ground water occurred in the same areas".

There are no known records of any damage or displacement occurring at this site in the 1855 or 1942 events. The area was well developed for the period. There is also no evidence of any slope movement in the geological record. The observed cracking in layers 2B to 4 is covered by layer 2A. The deposition of this layer 2A definitely occurred after the slope displacement occurred which created the displacement record in the lower layers.

The deposition of layer 2A predates development of the site from the mid 1800s. This is confirmed by archaeological finds particularly brick and timber foundations into this layer from former buildings from the military occupation and the Catholic precinct.

As no observable cracks formed on the site or through layer 2A in these two earthquake events there is a high likelihood that related permanent strain was negligible. It is reasonable to assume a maximum strain of 0.3% or 30mm displacement for the 84th percentile displacement (estimated from Jibson 2007);

The likely felt intensity and peak ground accelerations for historic events is summarised in Table 04.

Table 04 Historical earthquake summary

Earthquake Event	Return period	Felt Intensity / Magnitude	Likely peak ground acceleration	Potential Lateral spreading displacement as inferred from observed cracking
Masterton	1:25?	MM8+ / Mw6.8-7.2	0.13	nil
Wairarapa	1:500?	MM10 / Mw8.2-8.3	0.50	nil

3.1 Lateral spreading back analysis (eastward displacement)

Analysis has been undertaken using Slope/W limit equilibrium and the Jibson/Newmark sliding block analysis (2007) for displacements.

Lateral spreading is the result of a reduction in soil strength (due to liquefaction) combined with lateral forces exerted by earthquake.

The following parameters are conservatively assumed for the back analysis (i.e. no more destabilising post construction than they were at time of historic displacement):

- Groundwater levels are at current worst case measured levels (approximately 2.5m below current ground level at the memorial park and near ground level in the basin reserve);
- Slope gradients on upper and mid slopes are at current slope angles (underpass excavation will reduce driving forces further but this has been ignored), assumed current eastern slope cross section profile;
- The 1855 Wairarapa earthquake is assumed to have a similar PGA to the 1 in 500 year earthquake;
- It is reasonable to assume a maximum strain of 0.3% or 30mm displacement occurred in the 1855 earthquake event.
- The top 15m below ground level is assumed to have liquefied/strain softened in this event;

This model was used to back analyse the possible liquefied/strain softened undrained shear strength for the soil mass. This conservatively confirmed the liquefied/strain softened undrained shear strength of the soil must have been no less than 87kPa.

The back analysis of the slope has been undertaken to derive the likely Liquefied strengths of the slope materials for the 1855 Wairarapa Earthquake. The liquefied strengths will have been different for each scale of historic event. I.e. more soils will have liquefied in a 1:500 Year event than a 1:25 year Event. The predicted total liquefied thickness does not vary greatly in the CPT profiles, however the CPT profiles generally reached refusal around 15-16m deep, it is possible that some layers beyond this depth may liquefy in greater events. Note site observations indicate all the observed lateral spreading has occurred within the upper 2.2m of the soil profile. However for the purposes of conservatism, greater depths have been investigated.

3.1.1 Lateral spreading predictions

We have used the results of the back analysis of the 1855 performance to conservatively predict how the site may perform in design earthquake events. This is conservative as the overall observations indicate no significant lateral spreading will occur to the east in the ULS event (refer 2.2.5).

For the back analysis of the 1855 earthquake and projected design earthquake events we have used the following assumptions for the lateral spreading predictions:

- The analysis assumes that the depth of liquefied soil increases with the increasing PGA load, borehole logs beyond the depth of the CPTs suggest that there are layers of soil with SPT(N) values around 30 that may potentially liquefy or strain soften under greater magnitude earthquakes.
- Idris and Boulanger (2008) suggest that with a greater number of cycles the undrained shear strength of soil reduces. The analysis assumes a greater number of cycles for a greater PGA and uses a soil strength reduction factor of 1.0 for 1 in 500 year event (1855 Wairarapa back analysis), 0.9 for 1 in 1000 year event and 0.8 for 1 in 2500 year event.

The results of the SlopeW stability analysis (east slope in east direction) and Jibson (2007) Newmark sliding block displacement estimates are set out in Table 05. Refer Appendix D for the calculation summary.

Table 05 Design earthquake event displacement estimates

Earthquake Event	Liquefied Soil Depth (mbgl)	Liquefied Su	50th Percentile Displacement	84th Percentile Displacement	95th Percentile Displacement
Back analysis of 1855 Wairarapa Earthquake (1 in 500 year)	15m	87kPa	10mm	29mm	57mm
1 in 1000 year	17.5m	78.3kPa	58mm	164mm	323mm
1 in 2500 year	20m	69.6kPa	200mm	570mm	1125mm

This analysis indicates that the eastern slope is likely to displace approximately 200mm in the 1:2500 ULS design event. This is consistent with the results of the Site Precedent method described in section 2 above.

4. EMPIRICAL ASSESSMENT

Established and industry recognised methods such as Olson and Stark (2003) are based on back analysing previous events in numerous locations and geological settings. A review of lateral spreading post the Christchurch event indicates these methods were relatively accurate for the Christchurch geological conditions. However, they are based on the performance of sites with relatively clean sands. These methods are likely to be over conservative for silty soils with modest plasticity such as at this site.

The calculated displacements using Olson and Stark are not consistent with the observation of the historic performance of the site. Using the Olson and Stark average liquefied soil strength of $0.09\sigma_v$ for the potential liquefied soil mass suggests that the site would move under static conditions (yield acceleration of zero), the empirical calculations break down under such conditions suggesting very high levels of seismic movement.

The site is likely to have multiple layers of liquefied sand/silt (up to 7m thick), strain softened silts/clays and layers that do not liquefy or strain soften significantly in a large event. Layers of liquefaction susceptible materials may also be of limited lateral extent. It is difficult to isolate these layers and model how these individual layers and lenses would perform accurately due to the variability in the Holocene and Pleistocene alluvial deposits.

To provide a comparative assessment to the Site Precedent and back analysis methods we have assumed an average Olson and Stark Su = $0.25\sigma_v$ (assumed value based on a mix of sand like, clay like and non-liquefied dense soil behaviour in the liquefied soil zone) using the same model layout as the site precedent Slope W analysis.

The critical difference is that the Olson and Stark models will tend to push failure surfaces closer to the ground surface (due to the reducing undrained shear strength near ground level), one change to the models is the switch to grid and radius failure surfaces instead of the deep block failures with a minimum 3m deep failure surface to exclude shallow failures where there is fill material present.

The results are summarised in Table 06 below. The yield acceleration is the same for each earthquake load case as the soil strength is the same for each model and the failure surfaces are generally within the top 10m.

Table 06 Empirical lateral spreading displacement estimates

Earthquake Event	Liquefied Soil Depth (mbgl)	Yield Acceleration	50th Percentile Displacement	84th Percentile Displacement	95th Percentile Displacement
1 in 500 year	15m	0.048g	744mm	2118mm	4178mm
1 in 1000 year	17.5m	0.048g	1221mm	3473mm	6851mm
1 in 2500 year	20m	0.048g	1977mm	5622mm	11092mm

As this method clearly over estimates lateral spreading risk (based on site precedent of historical site performance), empirical methods (Olson and Stark) is not considered to be appropriate for the estimation of future risk at this site.

5. CONCLUSIONS AND RECOMMENDATIONS

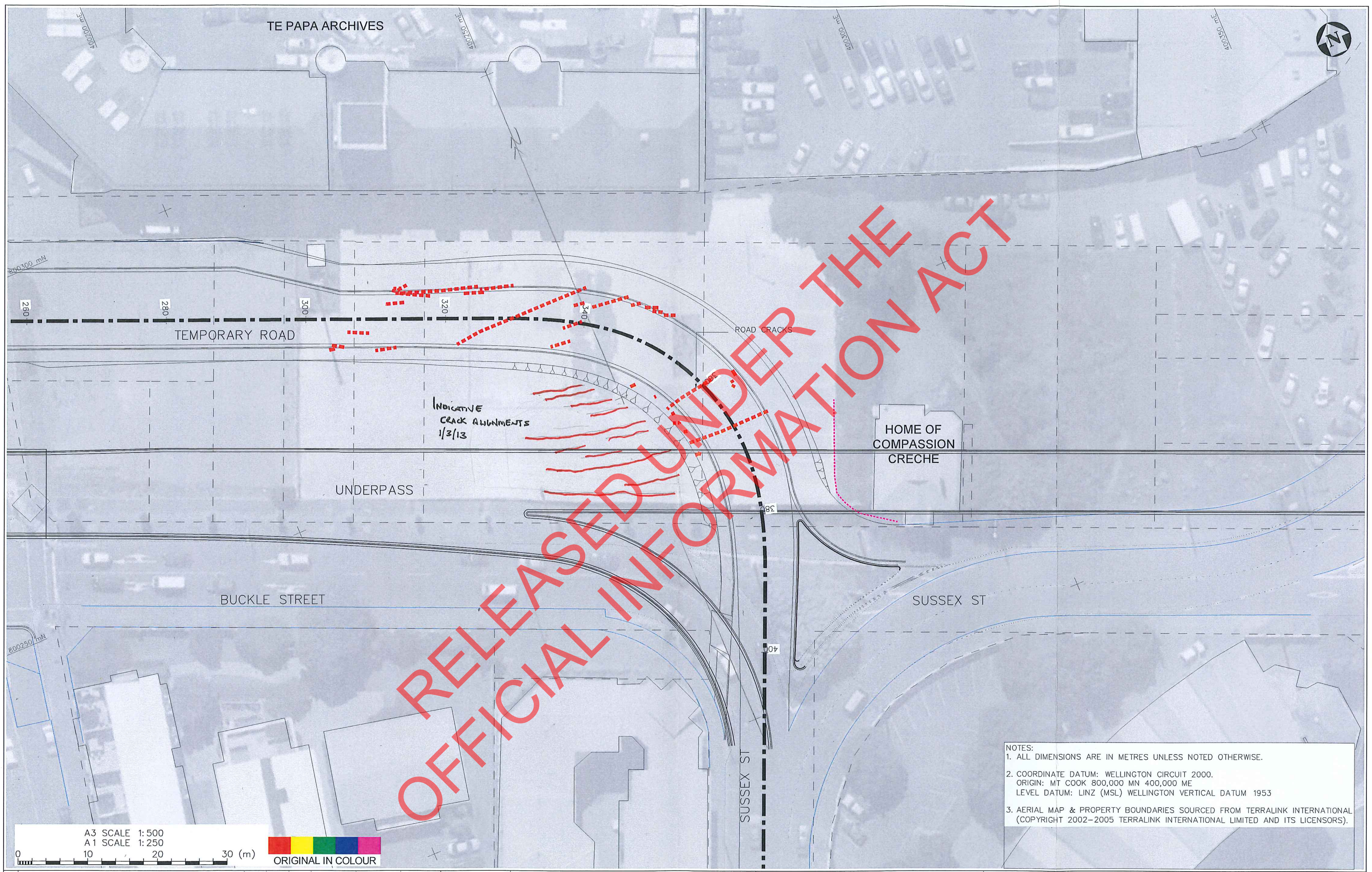
The Site Precedent and Backanalysis methods are consistent and are likely to provide a reliable prediction of likely Lateral spreading risk.

Empirical methods particularly that of Olson and Stark do not reconcile with the geological record preserved in the soil deposits, or the known history of the site (i.e. it did not fail in the 1855 Wairarapa earthquake). These methods are unlikely to provide a credible prediction tool due to the relative high plasticity of the soils, limited lateral extents of liquefiable materials and the limits (asymptotic behaviour) of the method at high ground accelerations.

Based on Site Precedent and Back analysis, the underpass is likely to be subject to negligible lateral spreading risk in the ULS design event. However, it is considered prudent to provide sufficient capacity within the underpass structure to tolerate the following lateral spreading scenarios:

- Eastside
  - Design for lateral spreading in easterly direction (ULS event) of up to 200mm total displacement.
  - Consider what would occur with a worse case 1.0m of lateral displacement.
  - Design for lateral spreading in north-south direction up to 0.5% strain.
- Westside
  - Design for lateral spreading in west direction up to 200mm total displacement.
  - Design for lateral spreading in north-south direction up to 0.5% strain.

Appendix A – Lateral Spreading Crack Plan



NOTES:
1. ALL DIMENSIONS ARE IN METRES UNLESS NOTED OTHERWISE.
2. COORDINATE DATUM: WELLINGTON CIRCUIT 2000.
ORIGIN: MT COOK 800,000 MN 400,000 ME
LEVEL DATUM: LINZ (MSL) WELLINGTON VERTICAL DATUM 1953
3. AERIAL MAP & PROPERTY BOUNDARIES SOURCED FROM TERRALINK INTERNATIONAL (COPYRIGHT 2002–2005 TERRALINK INTERNATIONAL LIMITED AND ITS LICENSORS).

A3 SCALE 1:500
A1 SCALE 1:250
0 10 20 30 (m)

ORIGINAL IN COLOUR

Rev	Revision Description	Drawn	Drawing Checked	Designed	Design Checked	Approved	Date
A	FIRST ISSUE	FFAN		NCP			

Tab EW-02-902
Scales 1:500 (A3) 1:250 (A1)
Original Size A1

SH1 WELLINGTON INNER CITY IMPROVEMENTS

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WELLINGTON INNER CITY IMPROVEMENTS

ENABLING WORKS - 02 GEOTECHNICAL
ROAD CRACKS
PLAN

Approved

Status: TARGET COST DESIGN

Drawing Number: EW-02-902

Revision: A

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Appendix B – Summary of Key Observations and Assumptions

Conclusion or Hypothesis	Level of Certainty	Comment
Lateral displacement has occurred and when it has it leaves a clear and definitive geological record.	High	Where the indicator layers have displaced a clear definitive record is left.
There has been a total strain on the eastern slopes (probably due to lateral spreading) of no more than 3% in a north-south direction.	High	Cracking is well preserved and laterally extensive. If significant displacement had occurred in an east-west orientation, this would have been observable.
There has been no significant lateral spreading on the eastern slopes in an easterly direction (i.e. parallel to the underpass alignment).	High	Indicator layers show only displacement to the south – cracks are pull apart and not shear. So majority of displacement is likely to have been perpendicular to the cracking.
There has been no significant lateral displacement at the western end of the site for age of deposits (20K +)	High	Indicator layers are present and show no evidence of lateral displacement
The observed cracking represents at least two events.	High	Cross cutting of two cracks has been observed indicating one must have formed and been in filled prior to the second forming in a separate event. The material infill is however similar indicating the movements is likely to have occurred within the same depositional environment.
Lateral spreading in a north-south direction has occurred at the eastern end of the site. This is likely to have occurred approximately 5000 years ago during a period when the groundwater levels were at or close to the surface.	Moderate	The layers with cracking are also buried by layer 2a which shows no evidence of displacement. This is supported by both radio carbon dating and geological observation of weathering and mineralisation.
3% cumulative strain has occurred in the north-south direction over age of deposits.	Moderate	Some small strains may have occurred that was not sufficient to form full tension cracks and so is not able to be recorded. This could feasibly add in the order of 0.5% strain to total cumulative strain predictions.
No significant lateral displacement has occurred in the material since the indicator layers have been covered, definitely younger than 5000 yrs (date in crack), but predates European habitation.	Moderate	
No significant lateral displacement in Wairarapa Earthquake.	High	There was no written record of specific lateral spreading in this area and there is no displacement in the upper layers associated with this event.

Appendix C – Geological Assessment of Lateral Spreading Memo

NZ TRANSPORT AGENCY
WAKA KOTAHI
Memorial Park Alliance

28th March 2013

MEMORANDUM

UND-02-Memo-014

To	[redacted] s9(2)(a)
From	[redacted] s9(2(a)
CC	
Date	27 March 2013
Subject	Underpass – Historic Lateral Spreading Cracks – Geological Observations

Background

Sub parallel orientated, infilled cracks are observed in surfical soils at the eastern end of the MPA Underpass site. These cracks are considered to have been formed as a result of seismic activity.

Observed Geology

Based on our observations, and the excavation of two test pits through the cracks (TPTT7 and TPTT8), the geological sequence of materials in the vicinity of the cracking can be summarised as follows:

Layer No	Material Description	Inferred Origin	Depth below pre construction surface	Thickness
1	Grey gravelly SILT / silty gravel	Man Made Fill	0m	0.2m
2a	Light grey and orange mottled gravelly SILT with completely weathered greywacke clasts (orange sand)	Reworked Colluvium / Alluvial Fan	0.2m	0.4m
2b	Becoming silty gravel with fine to coarse, moderately weathered sub angular gravels at base. Heavy iron staining at base. Undulating basal surface.	Less weathered Colluvium / Alluvial Fan	0.6m	0.2m
3	Uniform, brown silty SAND / sandy SILT. Undulating basal surface	Marginal Marine Environment (backwater, estuarine, lacustrine)	0.8m	0.4m
4	Silty fine to coarse GRAVEL with minor light grey plastic silt in pockets. Becoming sandier towards the base. Undulating basal surface.	Alluvial Fan	1.2m	1.0m

5	Silty, brown fine SAND. The layer is locally disturbed and deformed.	Alluvium / Marginal Marine Environment(backwater, estuarine, lacustrine)	2.2m	0.1m
6	Light grey / brown SILT with heavy iron staining at top and bottom of layer	Marginal Marine Environment (backwater, estuarine, lacustrine)	2.3m	0.2m
7	Sandy silty fine to coarse GRAVEL	Alluvial Fan	2.5m	1m?

Layers containing cracks

These layers are indicated in Photograph 1 and 2 below.

Photograph 1: Alluvium and marginal marine deposits in site excavation

Photograph 2: Alluvium and marginal marine deposits in TPTT8

Crack Description

A summary of crack information is as follows:

Extent	See attached Sketch EW-02-902. To date, cracks have only been observed at the eastern end of the underpass alignment, between approximate underpass chainage 480-520. This is the only location where the ground has been cleaned back to layers containing cracks. Other areas are either cleaned back to fills, other alluvium layers, or rock.
Width	Typically 10mm (smaller cracks) to 30mm (larger cracks)
Spacing	Smaller cracks are approximately 300-500mm spaced. Larger cracks are approximately 1.5-2m spaced.
Length	Shorter cracks are 2-3m long. Longer cracks are 10-20m long.
Distribution within geological sequence	Layer 1 – None Layer 2a – None visible Layer 2b – Large cracks only Layer 3 – All cracks Layer 4 – Small cracks terminate in the layer, larger cracks continue to base Layer 5 – Large cracks only Layer 6 – None visible Layer 7 – None visible
Colour	Smaller cracks are light grey brown with some orange iron staining on side walls Larger cracks are light grey with dark orange brown iron staining on side walls.
Infill	In layer 3 and upper layer 4, large cracks contain light grey, plastic, cohesive, clayey silt with brown organics.

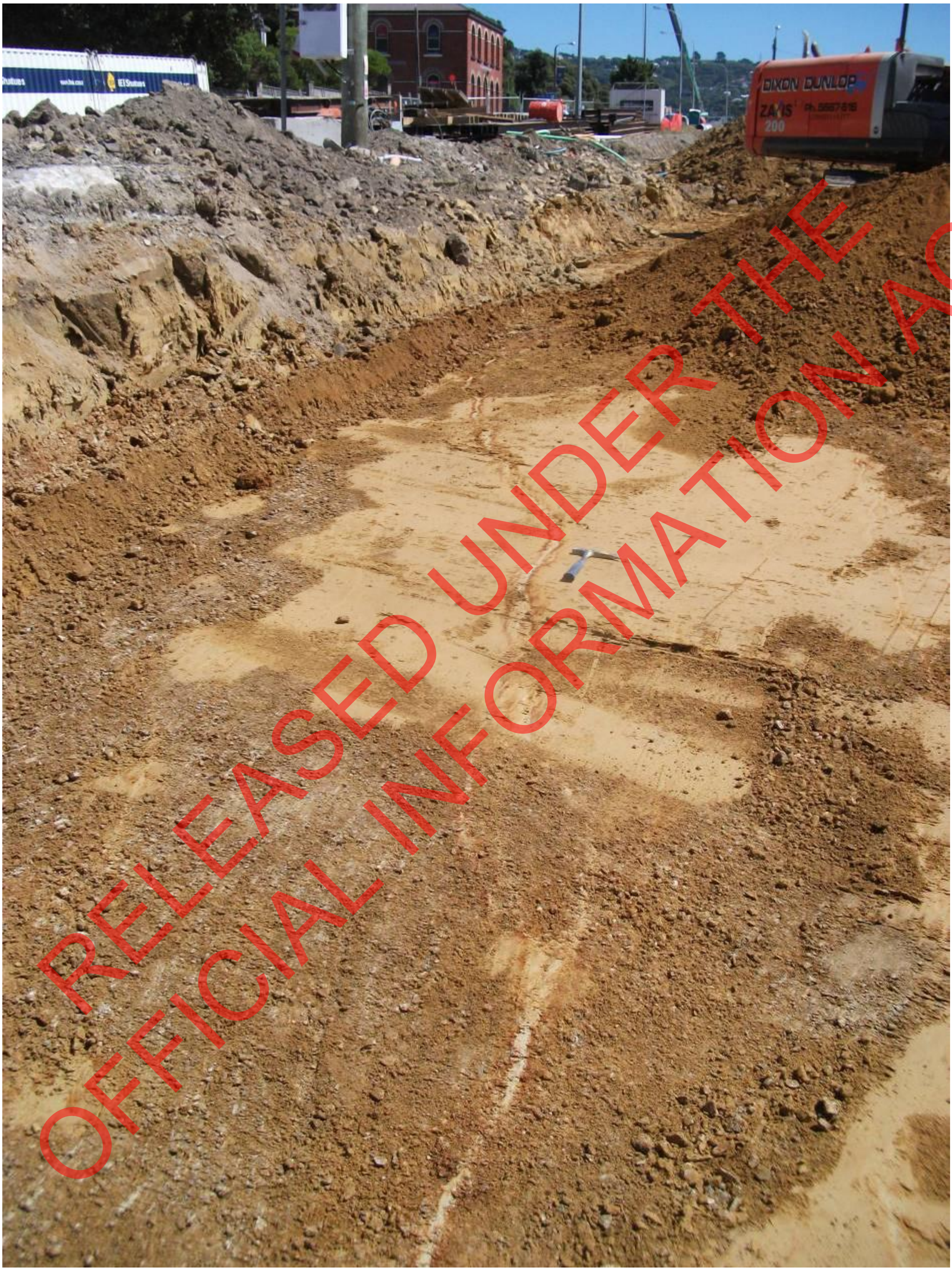
	In lower Layer 4, large cracks are defined by light grey clayey silt and gravel. There is no discernible crack in the lower layer 4. The silts are inferred to have washed down and accumulated into a more open texture within the gravels. This infers that layer 3 and upper layer 4 supported an open tension crack that was filled with silt, whereas lower gravel layer did not form a crack, but rather a more open texture. Smaller cracks contain sandy silt. The cracks are slightly siltier than the surrounding Layer 3. They do not contain the cohesive, plastic light grey silts like the larger cracks. A common feature is iron staining on the side walls. It is more pronounced and thicker on the larger cracks. There is also trace evidence of precipitation / layering parallel to the sidewalls.
Orientation	The cracks are generally orientated as follows: • 265-275° - at the eastern end of the temporary road (c. Chainage 500-520). • 285-295° - at approximate Chainage 480-500.
Shape	The cracks are essentially sub parallel. They are locally sinuous and also there is evidence that along the length of the larger cracks they curve round from c.270° to 290° orientation as the cracks track in a westerly direction. Some cracks bifurcate (split), both horizontally and vertically. Other cracks appear to cross cut each other. Some cracks are relatively uniform in width from top to bottom. Others are wider at the top, being 50-80mm wide, compared to 30mm wide lower down. The vertical dip of the cracks is variable, with some dipping steeply to the north, others to the south. The cracks are typically at 75-90° from horizontal. The dip of the crack varies with depth. Some are slightly sinuous, whilst others are curved.
General observations and comments	- Cracks appear to have opened from top down (some cracks do not penetrate all the way through Layer 4). - Crack infill has probably entered from the top, probably as a flood plain silt deposit. - The plasticity of the crack infill is considered to be the result of post-depositional alteration - Precipitation of the iron minerals has occurred at the interface between the sand / gravel and relatively impermeable crack infill. - Cracks follow desiccation cracking on surfaces. - The silts that infill the cracks are different to the silts that form Layer 2a. The crack infill is more plastic. - The crack infill silts also infill low spots on the upper surface of Layer 3 (i.e. into the bottom of Layer 2b).
Age	Carbon dating of organics in the cracks at eastern end of the temporary road indicates the infill is approximately 5000 years old. This is based on 1 carbon dating result. There is no dating evidence to suggest cracks of different ages. Organics in the colluvium / alluvium layers above and below the cracks (Layer 2a and Layer 6 respectively) are dated at 10,000 and 25,000 yrs respectively. The dating of Layer 6 therefore suggests it is of Pleistocene Age, and dates back to the last ice age. This infers that the cracks are younger than Layer 2a which is deposited above them. This is likely explained as Layer 2a being a reworked colluvium deposit, carrying organics in

	material that mobilised in an older event. Likewise the trace organics found in Layer 6. It is likely that the yellow brown sand and gravel deposits of this area are actually Holocene in age. And were deposited less than 12,000 years ago. Further carbon dating testing is in progress.
Environment of Deposition	Layer 3 is likely to have been deposited in a terrestrial marginal marine (e.g. estuary / backwater / lacustrine) environment. This low energy, quiet environment would have been periodically interrupted by high energy alluvial fans from the surrounding hillsides and up the valley. These alluvial fans would have been triggered by extreme storms or earthquake events. They would have deposited thick gravels (layers 4 and 7). Between these events, sands and silts (Layers 5 and 6) would have been deposited. These indicate a quieter, lower energy alluvial / marginal marine environment. Some of these deposits are disturbed. It is believed that the rapid emplacement of alluvial gravels over them (i.e. Layer 4) has caused the relatively fluid sands and silts below to deform into the overlying gravel.
Sea Level Changes and Uplift	Sea level rose rapidly up to approximately 6500 years ago, and since then it has remained relatively constant (give or take 1m or so). In conjunction with sea level rise, the area has been subject to uplift from fault movements for a long time, including the last 6500 years. Raised beaches around the coast of Wellington (i.e. Turakirae Head) indicate that there have been multiple events in the last 6500 years that have raised the ground level in the region. Uplift at Turakirae Head in the 1855 and 1460 earthquakes is measured to be approximately 2.5m and 6m respectively. Older uplifts are also recorded here, with three beaches recording uplift of 8.2m (approx 3,100 years ago), 5.5m (approx 4,900 years ago) and 2.7m (approx 6,500 years ago). 24.9m of uplift has therefore occurred over 6500 years, equating to approximately 5m per event. This is why the marginal marine deposits are elevated above sea level despite no significant sea level change in the last 6500 years.

Photograph 3: Example of a marginal marine environment (Cloudy Bay)

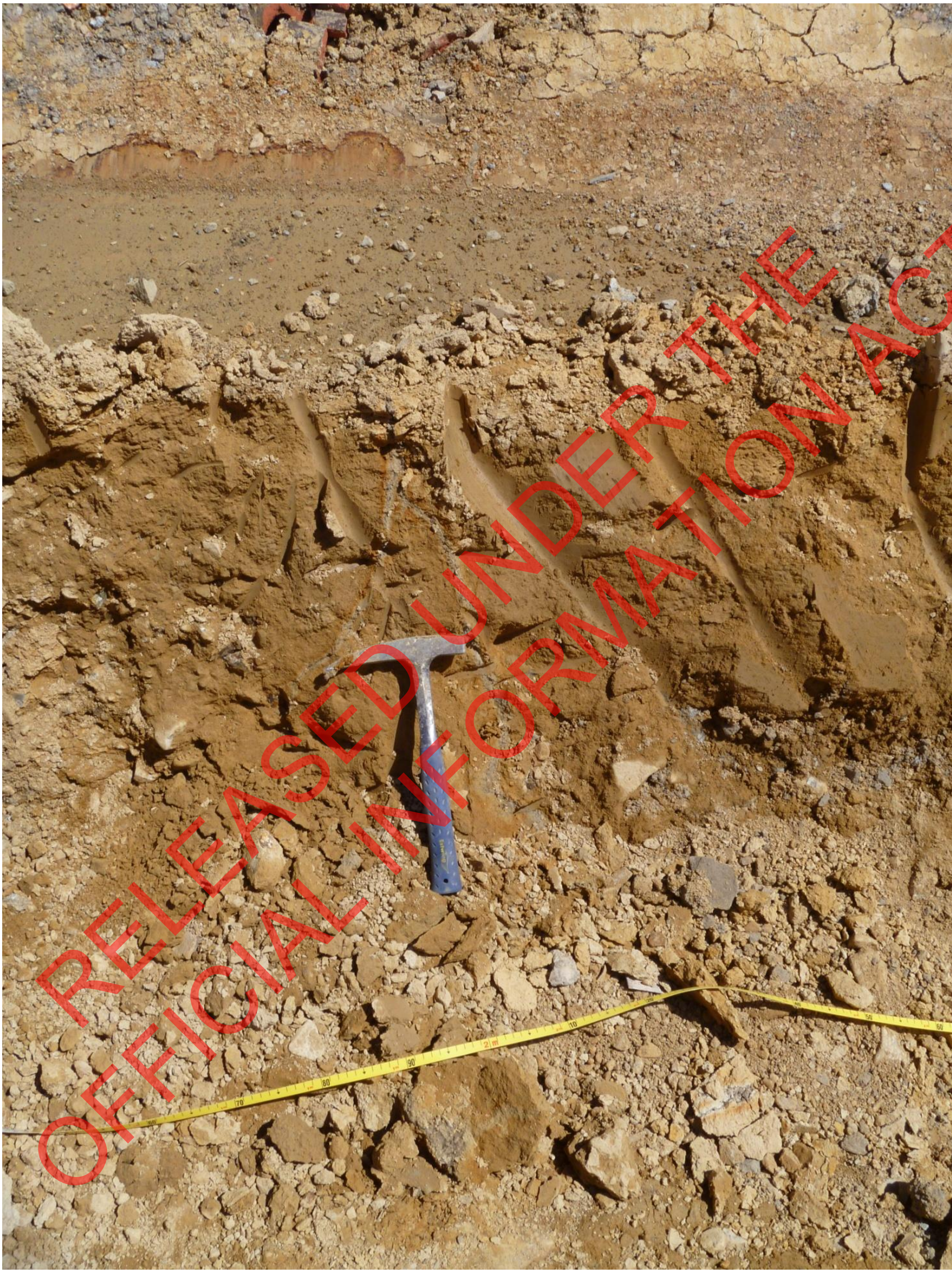
Photograph 4: Geological sequence in TPTT8 with large crack through middle of face

Cracking Photographs



Photograph 5: Large bifurcating crack passing through Layer 2b (gravel) and 3 (sand)

Photograph 6 & 7: Indicative spacing and distribution of large cracks (1.5-2m)



Photograph 8: Vertical bifurcation of cracks in Layer 3

Photograph 9: Crack infill also infilling low spots in top of Layer 3

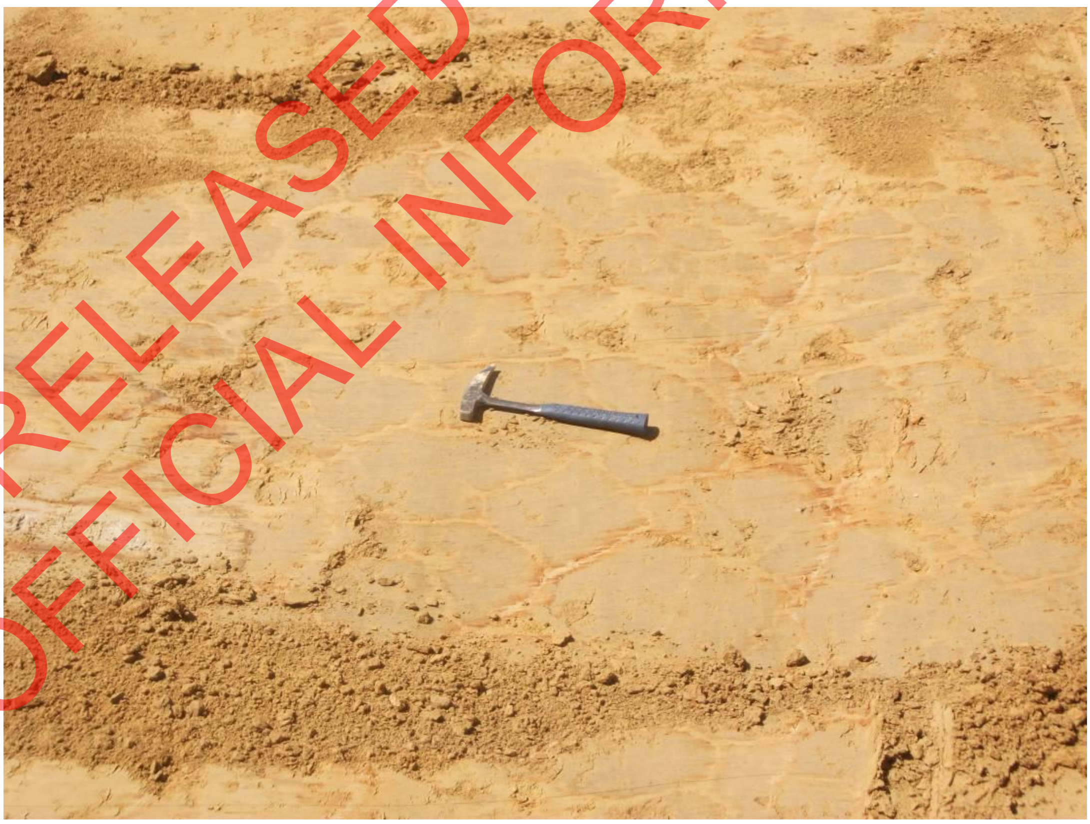
Photograph 10: Large crack extending into Layer 4

Photograph 11: Large crack extending from Layer 3 into Layer 4

Photograph 12: Bifurcating / cross cutting cracks

Photograph 13: Large cracks


Photograph 14: Clay infill at top of Layer 3


Photograph 15: Clay infill around desiccation cracks at top of Layer 3


Photograph 16: Bifurcating cracks

Photograph 17: Cross cutting cracks

28 March 2013
P:\85700\Geotech\WorkingMaterial\02 Underpass\04 Design Lateral Spreading\UND-MEMO-002-014 - Lateral spreading cracks - Geological Observations.docx

Appendix D – Underpass Lateral Spreading Estimates Calculations

Memorial Park Alliance

DESIGN CALCULATION

Underpass - Lateral Spreading Estimates - Back Analysis of 1855 Wairarapa Earthquake.

UND/CALC/02/015

REVISION HISTORY					
Rev	Prepared By	Status	Checked by	Date	Comments/Updates
A	[redacted] s9(2)(a)	Design	[redacted] s9(2(a)	28/03/13	

DESIGN REVIEWS/APPROVALS					
Critical Input/output	Relevant Sections	Relevant Appendices	Reviewed by	Signature	Date
Design Criteria, Ground Model and Method of Analysis	1-4		[redacted] s9(2(a)	………………	
Material Properties	3		[redacted] s9(2)(a)	………………	
SlopeW Analysis	4-5		[redacted] s9(2(a)	………………	
Displacement Estimates	4-5		[redacted] s9(2(a)	………………	

Key Design Conclusions		
Item	Description	Relevant Attachment/Section
1	Refer Section 8 for design conclusions	8
2		

Key Design verification requirements		
Item	Description	Monitor/verify?
3	<u>Ground Conditions</u> – exposed during underpass excavation– In particular: • Depth to rock • Lose silt or sand layers • Ground Water • Lateral Spreading Cracks	Verify

Contents

1. BACKGROUND

1.1 Purpose/Scope

This document provides estimates of the lateral spreading risks affecting the underpass for different design earthquake events. This calculation outlines the methods used and results for the back analysis of the existing slope and future predictions for the site.

1.2 Relevant Reports

1 MPA Lateral Spreading Assessment (UND-RPT-02-009)
2 MPA Interpretative Geotechnical Report (UND-RPT-02-002)
3 MPA Factual Geotechnical Investigation Report (UND-RPT-02-001)
4 National War Memorial Underpass Design Statement

2. DESIGN CRITERIA AND LOADING CONDITIONS

The limits for allowable lateral spreading displacement are outlined in the Design Statement. The design earthquake loads are outlined in the Interpretative Geotechnical Report (UND-RPT-02-002).

3. GROUND MODEL

3.1 Model assumed for design

The site is consists of Holocene and Pleistocene alluvium overlying weathered greywacke bedrock. The CPT based liquefaction analysis suggests that significant layers in the upper 15m will liquefy in a relatively low PGA event. Therefore it is conservatively assumed that the upper 15m of alluvium could liquefy/strain soften in all events. Deeper investigations suggest that there are deeper layers that may potentially liquefy in higher PGA events. Therefore the depth of liquefiable material has been assumed to increase with PGA load.

Existing lateral spreading cracks on site suggest that only the eastern section of the site is susceptible to lateral spreading in a large earthquake.

General material properties were used for the non-liquefied soil as outlined in the MPA Interpretative Geotechnical Report (UND-RPT02-002)

4. METHOD OF ANALYSIS

4.1 Back Analysis of 1855 Wairarapa Earthquake

The following assumptions were used for the back analysis of the 1855 Wairarapa Earthquake:

- Groundwater levels are at current worst case measured levels (approximately 2.5m below current ground level at the memorial park and near ground level in the basin reserve);
- Slope gradients on upper and mid slopes are at current slope angles (underpass excavation will reduce driving forces further but this has been ignored), assumed current eastern slope cross section profile;
- The alluvium up to 15m below ground level is assumed to have liquefied/strain softened in this event;

- The 1855 Wairarapa earthquake is assumed to have a similar PGA to the 1 in 500 year earthquake;
- A pseudo static horizontal load has been used to model the seismic loading to allow calculation of a yield acceleration in SlopeW;
- Lateral spreading in the 1855 Wairarapa earthquake was not reported at the site (and there is no evidence in the surface layers), while not reported it is reasonable to assume a maximum strain of 0.3% or 30mm displacement for the 84th percentile displacement (estimated from Jibson 2007);

This model conservatively confirmed that the liquefied/strain softened undrained shear strength of the soil is a minimum of 87kPa.

4.2 Predictions for Future Earthquakes

We have used the results of the back analysis to predict how the site may perform in future design earthquake events. We have used the following assumptions for the lateral spreading predictions:

- The analysis assumes that the depth of liquefied soil increases with the increasing PGA load.
- Idris and Boulanger (2008) suggest that with a greater number of cycles the undrained shear strength of soil reduces. The analysis assumes a greater number of cycles for a greater PGA and uses a soil strength reduction factor of 1.0 for 1 in 500 year event (1855 Wairarapa back analysis), 0.9 for 1 in 1000 year event and 0.8 for 1 in 2500 year event.

5. ANALYSIS RESULTS

5.1 Predicted Displacements

The SlopeW models are included in Appendix A and the Jibson (2007) Newmark block displacement spreadsheets are included in Appendix B. The predicted displacements and assumed soil conditions for each load case are summarised in Table 1 below:

Table 1: Design Earthquake Event Displacement Estimates

Earthquake Event	Liquefied Soil Depth (mbgl)	Liquefied Su	50th Percentile Displacement	84th Percentile Displacement	95th Percentile Displacement
1 in 500 year/ Back analysis	15m	87kPa	10mm	29mm	57mm
1 in 1000 year	17.5m	78.3kPa	58mm	164mm	323mm
1 in 2500 year	20m	69.6kPa	200mm	570mm	1125mm

5.2 Comparison to Design Statement Requirements

The predicted lateral spreading displacements fall within the performance requirements of the design statement, therefore, no specific lateral spreading retaining measures are required.

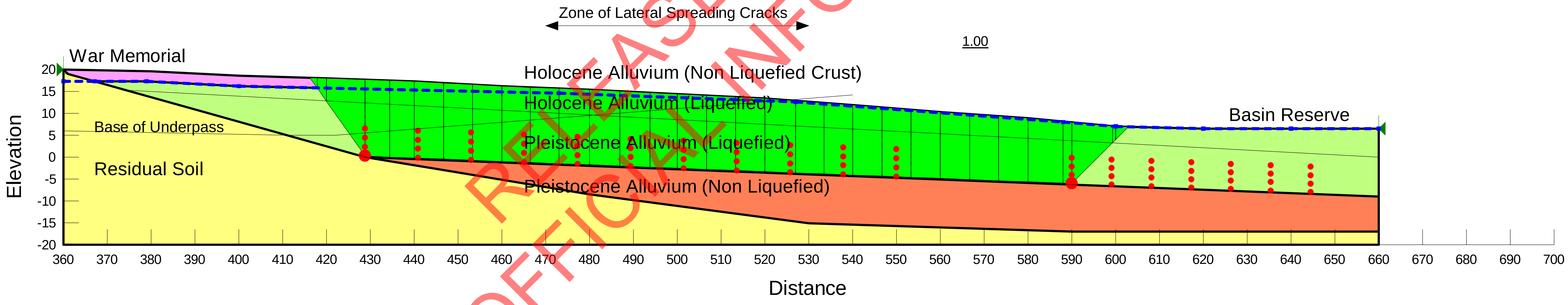
Appendix A –SlopeW Models

Name: Residual Soil
Model: Bedrock (Impenetrable)

Name: Holocene/Pleistocene Alluvium (Liquefiable)
Model: Undrained (Phi=0)
Unit Weight: 18 kN/m³
Cohesion': 87 kPa

Name: Pleistocene Alluvium (Non Liquefiable)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 32 °

Name: Holocene Alluvium (Non Liquefied Crust)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 30 °



Method: Morgenstern-Price
PWP Conditions Source: Piezometric Line
Slip Surface Option: Block
Minimum Slip Surface Depth: 1 m
Horizontal Seismic Coefficient.: 0.28g

Memorial Park Alliance
Eastern Underpass Outlet - Lateral Spreading Analysis
Wairarapa 1855 Earthquake Back Analysis (1 in 500 year earthquake Ay/Amax=0.56)

Scale: 1:1000 (A3)
Created By: [redacted] s9(2(a)
Last Edited By: [redacted]
Date: 26/03/2013

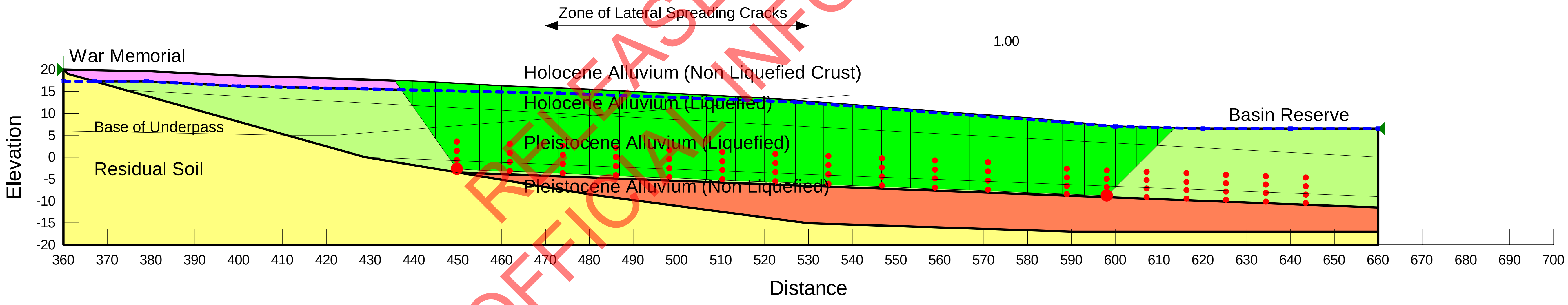
Directory: C:\Users\sbg\Desktop\ Lateral Spreading East - Wairarapa 1855 Backanalysis.gsz

Name: Residual Soil
Model: Bedrock (Impenetrable)

Name: Holocene/Pleistocene Alluvium (Liquefiable)
Model: Undrained (Phi=0)
Unit Weight: 18 kN/m³
Cohesion': 78.3 kPa

Name: Pleistocene Alluvium (Non Liquefiable)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 32 °

Name: Holocene Alluvium (Non Liquefied Crust)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 30 °



Method: Morgenstern-Price
PWP Conditions Source: Piezometric Line
Slip Surface Option: Block
Minimum Slip Surface Depth: 1 m
Horizontal Seismic Coefficient.: 0.224g

Memorial Park Alliance
Eastern Underpass Outlet - Lateral Spreading Analysis
1 in 1000yr Earthquake, 0.9Su, 17.5m deep liquefaction

Scale: 1:1000 (A3)
Created By: [redacted] s9(2(a)
Date: 26/03/2013

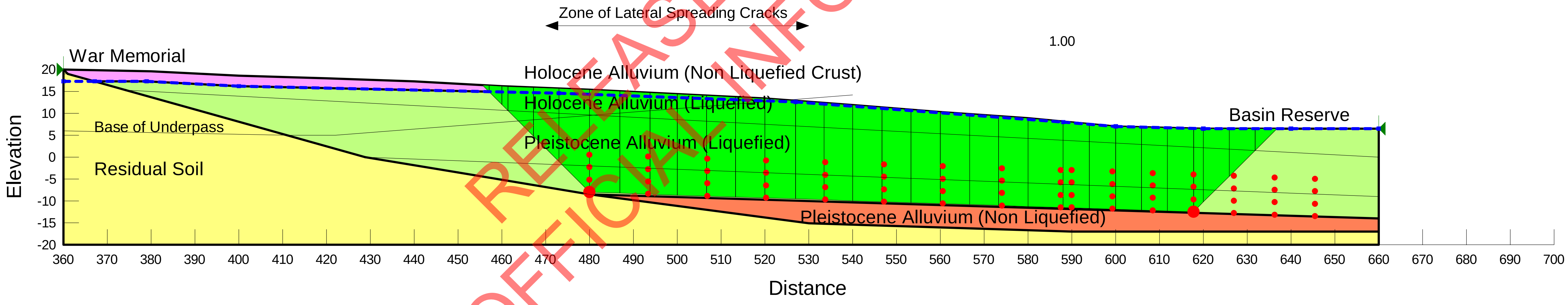
Directory: C:\Users\sbg\Desktop\Lateral Spread\ Lateral Spreading East - 1in1000yr 0.67g EQ 0.9Su.gsz

Name: Residual Soil
Model: Bedrock (Impenetrable)

Name: Holocene/Pleistocene Alluvium (Liquefiable)
Model: Undrained (Phi=0)
Unit Weight: 18 kN/m³
Cohesion': 69.6 kPa

Name: Pleistocene Alluvium (Non Liquefiable)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 32 °

Name: Holocene Alluvium (Non Liquefied Crust)
Model: Mohr-Coulomb
Unit Weight: 18 kN/m³
Cohesion': 4 kPa
Phi': 30 °



Method: Morgenstern-Price
PWP Conditions Source: Piezometric Line
Slip Surface Option: Block
Minimum Slip Surface Depth: 1 m
Horizontal Seismic Coefficient.: 0.175g

Memorial Park Alliance
Eastern Underpass Outlet - Lateral Spreading Analysis
1 in 2500yr Earthquake, 0.8Su, 20m deep liquefaction

Scale: 1:1000 (A3)
Created By: [redacted] s9(2(a)
Last Edited By: [redacted]
Date: 26/03/2013

Directory: C:\Users\sbg\Desktop\Lateral Spread\ Lateral Spreading East - 1in2500yr 0.67g EQ, 0.8Su.gsz

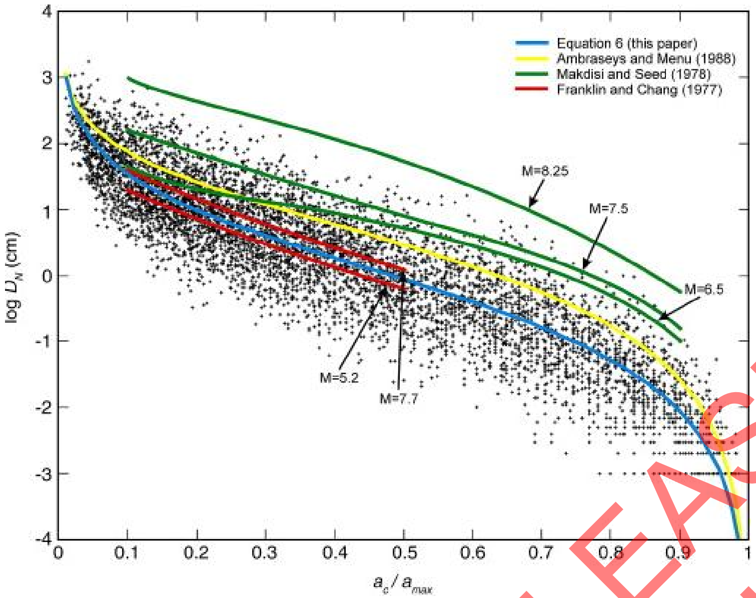
Appendix B – Jibson (2007) Newmark Block Spreadsheet

Estimate of Displacement For Seismic Event

All cases assume M=7.5 earthquake using Jibson Newmark Block Displacements

				95th Percentile		84th Percentile		50th Percentile		16th Percentile	
Load Case	Ay	Amax	Ay/Amax	Log(Dn) Max	Displacement (cm)	Log(Dn) Min	Displacement (cm)	Log(Dn) Min	Displacement (cm)	Log(Dn) Min	Displacement (cm)
East Side 1 in 500yr earthquake	0.28	0.5	0.56	0.758740096	5.7	0.463640096	2.9	0.009640096	1.0	-0.4443599	0.4
East Side 1 in 1000yr earthquake	0.224	0.67	0.33	1.509684218	32.3	1.214584218	16.4	0.760584218	5.8	0.306584218	2.0
East Side 1 in 2500yr earthquake	0.175	0.9	0.19	2.050993179	112.5	1.755893179	57.0	1.301893179	20.0	0.847893179	7.0

Log(Dn)=-2.710+log[(1-Ay/Amax)^2.335*(Ay/Amax)^-1.478]+0.424*M±0.454z
(Jibson 2007)



z	percentile
1.65	95th
1	84th
0	50th
-1	16th

M= 7.5