URS

Volume 2B

RESOURCE CONSENT APPLICATION FOR THE STAGE 4 EXTENSION OF THE SOUTHERN LANDFILL, WELLINGTON

August 2013

Absolutely POSITIVELY Wellington

Me Heke ki Pōneke
Wellington City Council

Report

Southern Landfill Stage 4

Phase 1 Geotechnical Report

AUGUST 2013

Prepared for

Wellington City Council

101 Wakefield St
PO Box 2199
Wellington 6143

42787470

URS

Project Manager:

....
Greg Haldane
Civil Engineer

Reviewer:

....
Debbie Fellows
Principal

URS New Zealand Limited

Lambton House, 160 Lambton Quay
Wellington 6011
PO Box 3367, Wellington 6140New Zealand

T: 64 4 496 3750
F: 64 4 496 3755

Date:	**August 2013**
Reference:	42787470/R002/F
Status:	FINAL

Document delivery

URS New Zealand provides this document in either printed format, electronic format or both. URS considers the printed version to be binding. The electronic format is provided for the client's convenience and URS requests that the client ensures the integrity of this electronic information is maintained. Storage of this electronic information should at a minimum comply with the requirements of the Electronic Transactions Act 2002.

Where an electronic only version is provided to the client, a signed hard copy of this document is held on file by URS and a copy will be provided if requested.

Table of Contents

Table of Contents

Table of Contents

Tables

Figures

Plates

Table of Contents

Appendices

Introduction

1.1 Introduction

The Wellington City Council (WCC or the Council) is proposing to extend the existing Southern Landfill site, located off Landfill Road, Happy Valley, between Brooklyn and Owhiro Bay. WCC has engaged URS New Zealand Ltd (URS) to prepare this Phase 1 Geotechnical Report (Geotechnical Report) in support of the Resource Consent Application for the Stage 4 Extension of the Southern Landfill.

The geotechnical risks associated with the proposal have been identified by undertaking a geotechnical desk study, and preliminary ground investigation. Preliminary geotechnical analysis has been undertaken to determine suitable mitigation measures and to develop Stage 4 area concept design.

The geotechnical works were undertaken in two phases, in 2007 and 2010 / 2011.

1.2 Description of Proposed Stage 4 Extension

Filling of the proposed Stage 4 landfill extension would begin at the top of Carey's Gully and progress downwards to meet the existing Stage 3 landfill. The total area of the Stage 4 landfill footprint would be approximately 28 ha and the estimated airspace would be approximately 10 million cubic metres. The final landform is proposed to rise from the landfill periphery at a slope of approximately 1V:4H to a ridge or small plateau at the centre of the landfill. The landform would fall from the upper reaches of the valley to meet the final level of Stage 3.

The proposed landfill would receive general municipal, commercial waste, cleanfill, construction and demolition debris, biosolids, and special wastes consistent with those suitable for Class A landfill disposal in accordance with the Hazardous Waste Guidelines. The Council would continue to implement existing programmes to segregate greenwaste, cleanfill, recyclables, whiteware and household hazardous waste from the waste stream.

Access roads would be provided on both the southern and northern sides of the valley around the periphery of the proposed footprint. Access to the tip face would generally be from the southern access road with sideling roads down the gully slopes.

Filling of the landfill would be in discrete cells progressing down the valley. Fill would be placed and compacted and provided with daily cover. An engineered final cover system would be provided in areas where landfilling was complete. The final cover surface would be provided with contour drains directing stormwater runoff to the open drainage at the periphery of the landfill.

Stormwater would generally be diverted around the Stage 4 landfill operations. Stormwater from the catchment above the landfill footprint would be conveyed around the Stage 4 footprint through a cleanwater diversion system and ultimately discharge downstream of the overall landfill. Stormwater from areas of the Stage 4 landfill with final cover would also report to this cleanwater diversion. Within the landfill footprint cutoff drains would be provided around cells and report to Carey's Stream downstream of the landfill.

Carey's Stream within the landfill footprint would continue to be diverted beneath the Stage 2 and Stage 3 landfill areas through an existing tunnel system. However, the catchment area for this diversion would be reduced by 80% (due to the cleanwater diversion) and reduced further as filling progresses down the gully. Upon completion of the Stage 4 filling operations the diversion tunnel would no longer convey stormwater as the entire upper catchment would be conveyed by the cleanwater diversion. Upon completion of the landfill new access to the tunnel would be provided through a lateral tunnel and vertical tunnel at the access road. The tunnel would continue to convey leachate and subliner drainage.

Discharges to groundwater and land would be controlled through the inferred natural hydraulic containment of the valley and engineered liner and leachate management systems.

In areas where competent greywacke is not present, that is in significant shear/fracture zones along valley sideslopes and at the bottom of the gully where colluvial material is present, an engineered liner system would be provided. Due to the steepness and natural containment characteristics of the valley sidewalls, lining of these areas is not proposed. However, a drainage system, comprising inclined chimneys or a drainage layer on the side slopes, is proposed in these areas, which would reduce the hydraulic head at the interface with the natural slope, and drain to the engineered liner at the base of the landfill.

A leachate collection system is proposed at the valley floor above the engineered liner. The system would comprise a fully redundant perforated pipe system bedded in a coarse drainage aggregate layer capable of conveying full leachate flow should the pipe system fail. A drainage/filter layer would be provided above the coarse drainage layer and be tied into the sidewall drainage system described above. Leachate discharge would be reticulated to the existing Stage 3 system and ultimately discharged to tradewaste. Attenuation of leachate would be provided through lined ponds at each landfill cell.

To protect the proposed liner system at the landfill base and to reduce groundwater intrusion resulting from artesian conditions, a subliner drainage system is proposed. This drainage system would include a perforated pipe system beneath the liner bedded in aggregate drainage material. This system would normally discharge to Carey's Stream beyond the toe of the active landfill cell. However, this discharge could be redirected to the leachate system should groundwater become impacted. As a result, the subliner system would provide a fourth level of leachate control/containment in addition to the natural containment, engineered liner, and leachate management systems described above.

A landfill gas management system is proposed to control emissions to air. The system would comprise the following:

- Landfill gas collection wells
- Landfill gas conveyance system including headers, laterals and ring mains
- Condensate drainage and traps
- A low permeability cover system with a gas collection layer (described above)

Landfill gas would be reticulated to a landfill gas combustion system.

1.3 Summary of works completed

The following geotechnical work has been undertaken by URS in the period since 2007:

- Desk study, including:
  - Published geology
  - Stereographic photo analysis and digital terrain mapping
  - Fault movement literature review
  - Existing geotechnical reports
- Ground investigation, including:

1 Introduction

- Geological mapping of existing rock outcrops
- Boreholes and test pits
- Permeability testing

- Geotechnical concept design
- Geotechnical risk assessment

Desk Study

Drawings G-001 through **G-004** referred to below are provided in **Appendix A**.

2.1 Site Description

The proposed site for Southern Landfill Stage 4 is located in Carey's Gully, approximately 5 km to the southwest of central Wellington.

The topography comprises a series of steep sided gullies that join Carey's Gully, which has the only permanent stream. Carey's Gully winds its way southwards through the Stage 4 area until the boundary with the Stage 3 area, where it enters a culvert.

The lowest point on the site (120 m above ordnance datum (AOD)) is situated at the southern boundary of the site (Stage 3 area) where Carey's Gully stream enters the culvert. The site is bounded by densely vegetated high ridges (250 m AOD to >350 m AOD) to the north, east and west. A public roadway is situated upon the north and west ridges.

Dense vegetation is present across the majority of the site, which combined with steep soil and rock slopes limits access across the site. Access roads have been cut into the eastern and western valley sides, and part of the way up Carey's Gully.

Earthworks and extensive rock cut slopes are present where the Stage 4 area overlaps with the Stage 3 area and in areas where the access roads have been cut.

2.2 Geology

2.2.1 Superficial Geology

Information on the superficial deposits is derived from:

- Yetton, M. & Robertson, M. (1994): Geology and Hydrogeology of The Carey's Gully Landfill Wellington;
- Begg J.G. & Mazengarb, C., (1996) Geology of the Wellington Area, 1:50,000. Institute of Geological and Nuclear Science Geological Map 22.

These sources indicate that superficial deposits comprise Makara Soils. This typically consists of brown silty sand with some gravel and clay. Thicknesses are stated to vary widely from 0.5 m to 3 m and are generally thickest in mid slope areas of the valley. Angular greywacke gravel and scree slopes are common in steeper areas and there are no superficial deposits in the steepest areas, which comprise of bedrock.

Alluvium consisting of sub-rounded sandstone gravel occupies a thin strip in the bottom of the gullies.

2.2.2 Bedrock Geology

Information on the bedrock geology is derived from:

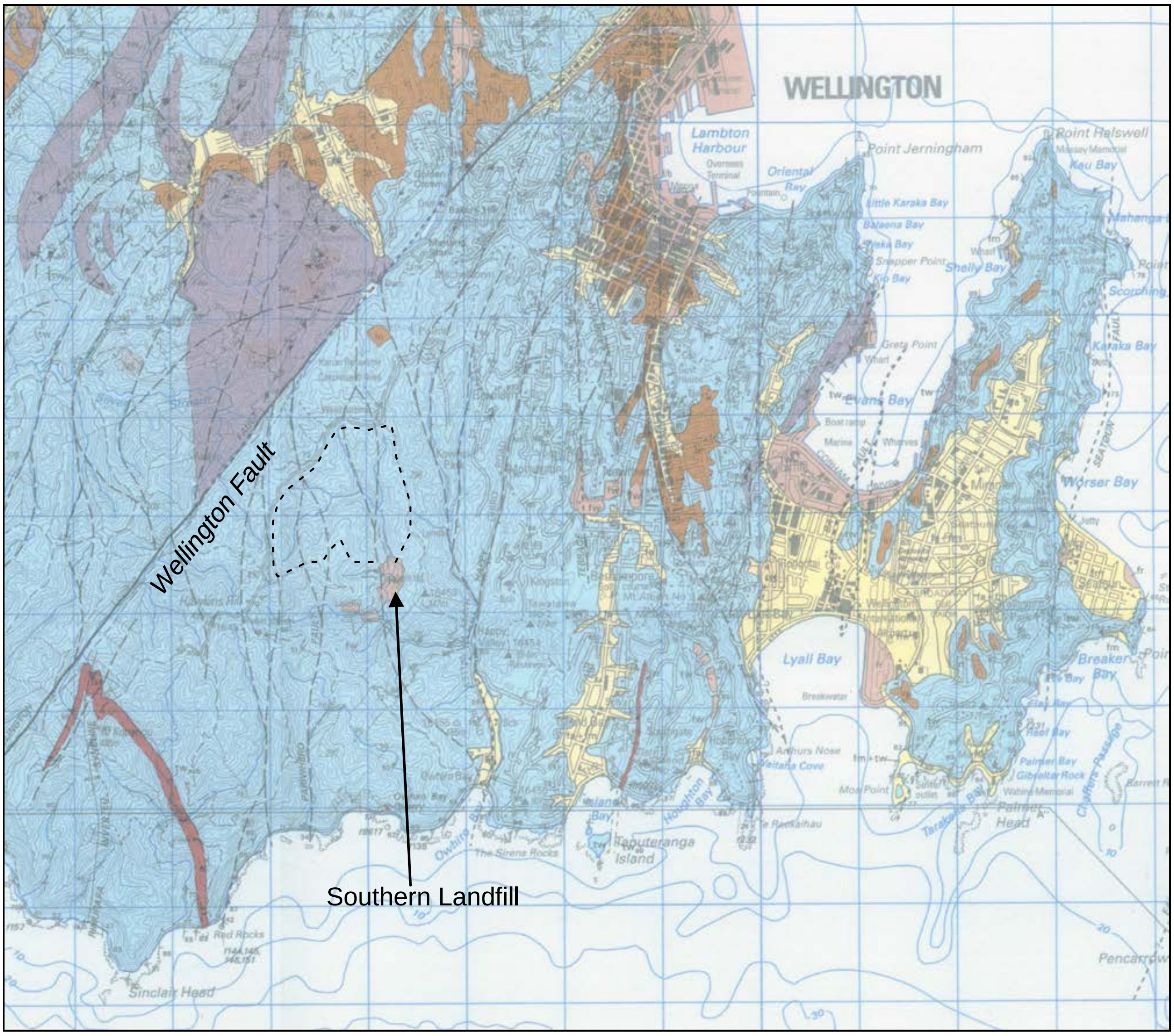
- Begg J.G. & Mazengarb, C., (1996) Geology of the Wellington Area, 1:50,000. Institute of Geological and Nuclear Science Geological Map 22.

This shows the Stage 4 area to be underlain by Rakaia Terrane (Torlesse Group) indurated sandstones and mudstones, often referred to as "greywacke". The local geology is summarised in **Figure 2-1**, with blue areas underlain by greywacke.

2 Desk Study

The major structural elements in the region include the Wellington Fault, which strikes approximately northeast and reaches within 1 km of the north-eastern boundary of the site. In addition, several north striking faults (mapped as possibly inactive) are inferred to pass through the site (See **Figure 2-1** and **Drawing G-002**). The regional bedding orientation shown is a moderately steep dip towards the west and northwest.

Figure 2-1 Geological map of the Wellington area (after Begg and Mazengarb 1996). The blue area underlying Stage 4 area outline is Greywacke (Sandstone and mudstone).



2.3 Aerial Photography

Digital and stereographic aerial photographs were viewed to determine whether there is historical slope instability and any lineaments indicating possible faults.

Wellington City Council digital aerial photographs flown in 2008 and 2010 were viewed to determine whether historical slope instability is present. Several head scarps and scree slopes were identified (refer **Drawing G-001**). Positive identification of other historic slope instability was difficult due to the presence of dense vegetation.

An analysis of stereographic aerial photographs (1:25000 scale, 2004) of this site was undertaken to provide a three dimensional view of the site. The intent of this exercise was to identify lineaments in the topography which may indicate the location of possible fault hazards. A summary of this work is presented on **Drawing G-002**. A series of “secondary”, possibly inactive faults are identified in the area, with a north-south orientation. Lineaments with northeast-southwest and northwest-southeast trends were observed.

2.4 Digital Terrain Mapping

A digital elevation model (DEM) has been compiled for the Southern Landfill Stage 4 area undertaken using LiDAR (Light Detection and Ranging) data flown in 2008. The DEM is presented upon **Drawing G-003** in **Appendix A** and shows the relationship between slope angles and topography across the catchment.

This DEM shows that slope angles vary across the site, generally between 20^o to 50^o. There are significant areas steeper than 50^o, correlating to the steep gully sides, the existing landfill cut slopes and access road cut slopes. Areas with slopes shallower than 20^o correlate to roads, hill tops, gully bottoms and the existing landfill landform.

Field Work

3.1 Geological Mapping

Geological mapping and site walkovers were undertaken in July 2007 and December 2010. This was to identify and characterise geomorphological features, superficial geology, bedrock geology, and discontinuities. The results are discussed below and summarised on **Drawing G-001**.

3.1.1 Geomorphology

The site comprises a deeply incised valley with steep slopes and natural slope angles typically between 30° to 50°. Slopes steeper than 50° are present, which generally comprise rock, typically encountered in the bottom of the main gully, the access road cut slopes and where Stage 4 area overlaps with Stage 3 area. These steeper slopes comprise either very thin soils over rock or rock, with little to no soil observed on slopes >55°. There is approximately 200 m of relief between natural valley floors and ridges. The majority of the slopes are densely vegetated with scrub and trees, which obscures the landforms (See **Plates 3-1 to 3-3**).

Plate 3-1 View across proposed Southern Landfill Stage 4 area to eastern road

Plate 3-2 View across to western road cut. **View up valley from Stage 3 area.**

3 Field Work

Plate 3-3 View up valley into northern and western areas of the proposed Southern Landfill Stage 4 area, from east and west access roads respectively.

3.1.2 Superficial Geology

The superficial deposits were observed to consist of colluvium, alluvium and fill. Colluvium was observed at the top of road cuttings along the eastern and western valley sides and in Carey's Gully. Alluvium was observed in Carey's Gully stream bed.

The colluvium is typically described as silty GRAVEL or gravelly SILT, with minor sand, cobbles and clay. The gravel and cobbles are angular to sub-angular sandstone. The thickness of colluvium varies across the site and appears to depend upon slope angle, gullies and faulting. This is summarised on **Drawing G-001** and in **Table 3-1**, below.

Table 3-1 Typical colluvium thicknesses based upon geological mapping observations

Area	Colluvium Thickness
Very steep slopes (>40^{o})	0.0m to 0.5m
Steep slopes (25^{o} to 40^{o})	0.5m to 1.0m
Gullies	0.5m to >2.5m
Fault Zones	1.0m to >2.0m

The alluvium is typically sub-rounded, gravel, cobbles and boulders of sandstone and is restricted to the bottom of Carey's Gully.

Fill is present in some areas of the eastern access roadway, especially in the area from BH4A up to BH3A. The fill comprises sandstone gravel, cobbles and boulders excavated during the access road construction. The strength and density of this fill is unknown and the road embankment slope angles in these areas are likely to be at the angle of repose of the fill as a result of the end tipped road construction method.

3.1.3 Bedrock Geology

Rock exposure is mainly available in steep cut slopes around the existing landfill, alongside the incised stream bed of Carey's Gully and along the road cuttings along the eastern and western valley sides.

Highly weathered rock was only occasionally observed, forming a thin band (0.0 m to 0.5 m thick) at the top of road cuttings, and is typically weak and brown in colour.

Where moderately weathered rock is exposed in the road cuttings along the eastern and western valley sides, these rocks are typically brownish orange to brownish grey, moderately strong to strong, fracture spacing 50 mm to 200 mm; interbedded with dark grey, weak to moderately strong, mudstone with typical joint spacing 10 mm to 50 mm. The thickness of this material was observed to range from about 0.0 m to about 7.0 m

Where slightly weathered rock is exposed in the valley floor and occasionally in the road cuttings, these rocks are typically characterised as light grey, strong or very strong, fine to medium grained sandstone with typical fracture spacing of 50 to 150 mm; interbedded with dark grey, strong, mudstone with typical joint spacing <10 mm.

3.1.4 Discontinuities

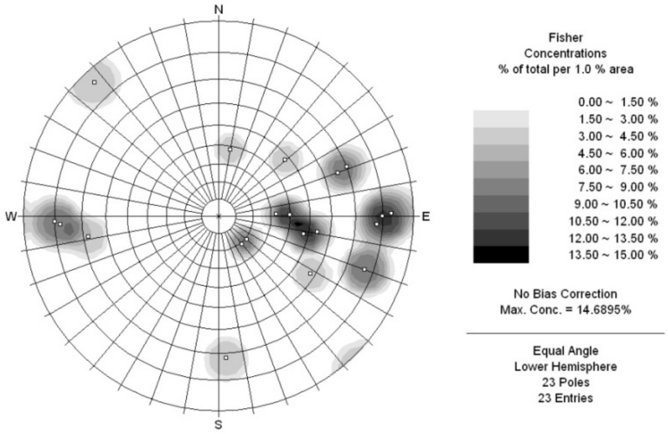
Bedding

Bedding is evident in the outcrops but can be hard to distinguish due to weathering, shearing and other deformation. The dip and strike of bedding is shown on **Drawing G-001** and can be summarised as steeply inclined to vertical, dipping from northwest to southwest and east through to southeast. Due to multiple folding events causing micro, meso and macro scale folding steeply dipping beds are common in the Rakaia Terrane sandstones and mudstones and the results presented here are consistent with those in published literature (Begg & Mazengarb, 1996). The rock mass often forms discontinuities (partings or shears) along bedding.

Bedding attitudes have been measured using a structural compass and typical orientations have been plotted on stereonets (**Figure 3-1**). Note that only easily accessible locations were assessed, thus there is bias in the data set.

3 Field Work

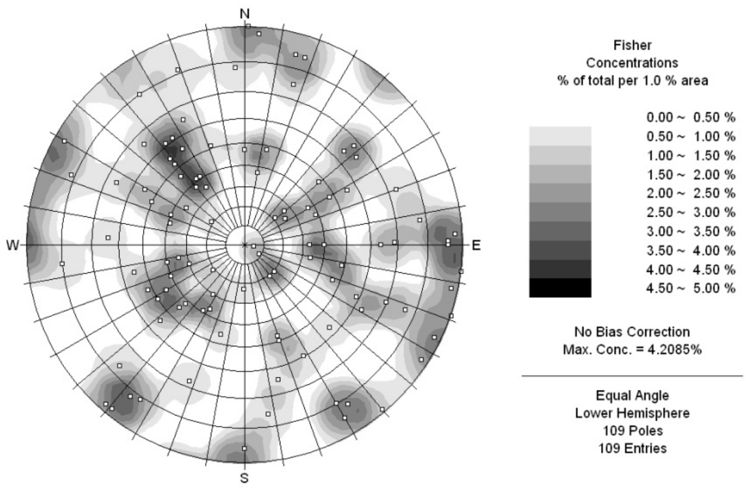
Figure 3-1 Lower hemisphere equal area projection of poles to all bedding measured in outcrop. Projection generated by DIPS 5.1.



Joint Sets

Joints are typically very closely to moderately widely spaced, impersistent (<1 m to 3 m), planar smooth to undulating smooth, very narrow to tight, weathered and minor silt and clay infill. As is typical of Rakaia Terrane sandstones and mudstones, joint orientations do not fall into simple joint sets but there may be in excess of three to five joint sets with additional random joints. The high variation in the joint set orientation may be partly due to the extensive faulting that has taken place in this area. Identifiable joint sets in outcrops were measured during this evaluation and orientations have been plotted on stereonets (See **Figure 3-2**). Note that only unrestricted accessible locations were assessed, thus there is a degree of bias in the data set.

Figure 3-2 Lower hemisphere equal area projection of poles to joints measured in outcrop. Projection generated by DIPS 5.1



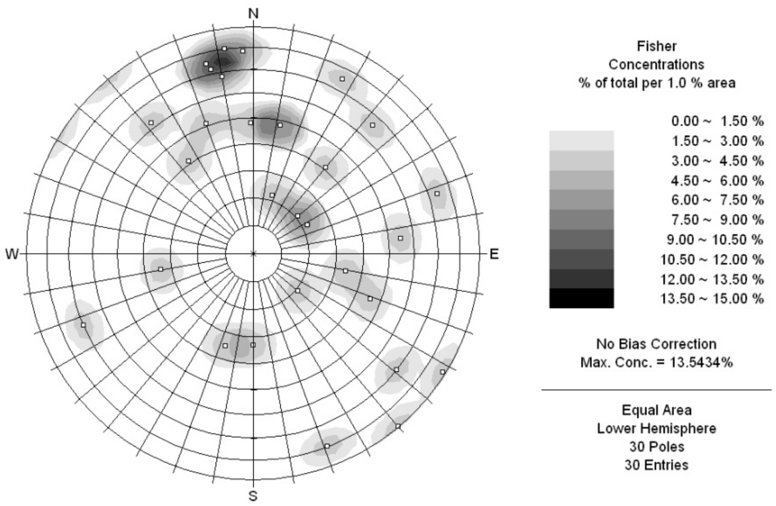
3 Field Work

Faults and shear zones

Shears or faults up to about 0.5 m in thickness are common in outcrop (See **Plate 3-4** and **Drawing G-001**). These comprise a zone of intensely fractured rock and contain seams of sheared rock or clay 5 mm to 50 mm in thickness. The shears exhibit variable orientations and have an average spacing of 3 m to 10 m. The orientations of dominant shears or faults exposed in outcrop were measured and have been plotted on **Figure 3-3**. Continuity of the shears or faults was observed to be at least 20 m in places and some features are expected to be continuous for hundreds of metres. Note that only easily accessible locations were assessed, thus there is bias in the data set.

The widest shears or faults observed (refer **Plate 3-5**) have less than 1 m to 3 m thickness of sheared material or fault gouge (angular sandstone gravel and cobbles in silty matrix) and groundwater seepage was observed.

Figure 3-3 Lower hemisphere equal area projection of poles to faults and shear zones measured in outcrop. Projection generated by DIPS 5.1



3 Field Work

Plate 3-4 Fault shear zone on western access road cut slope

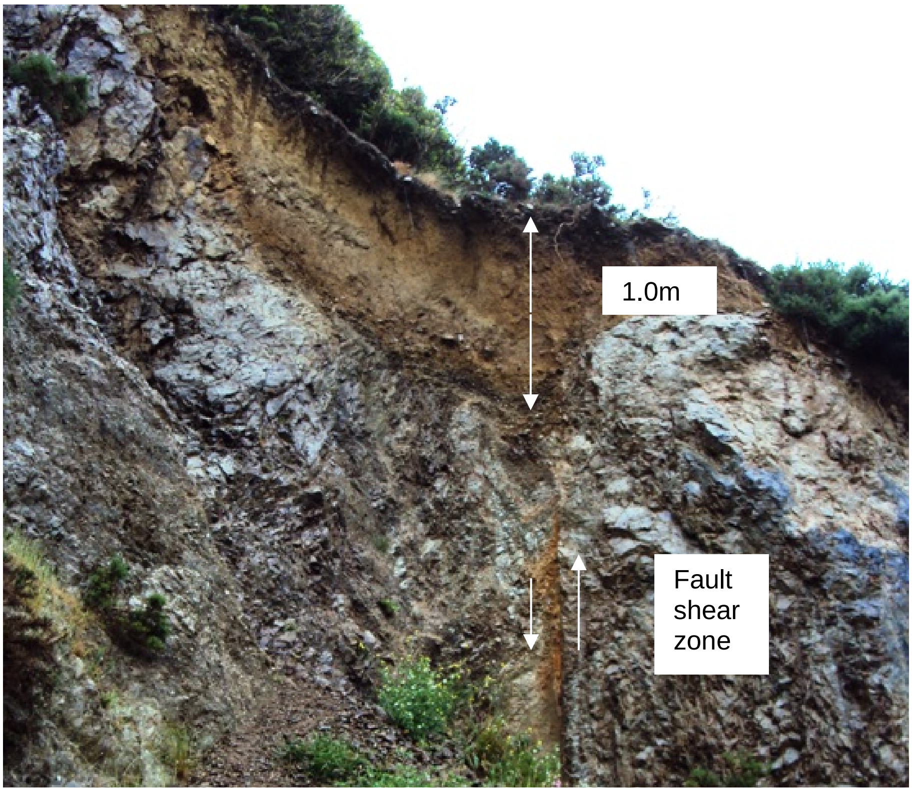
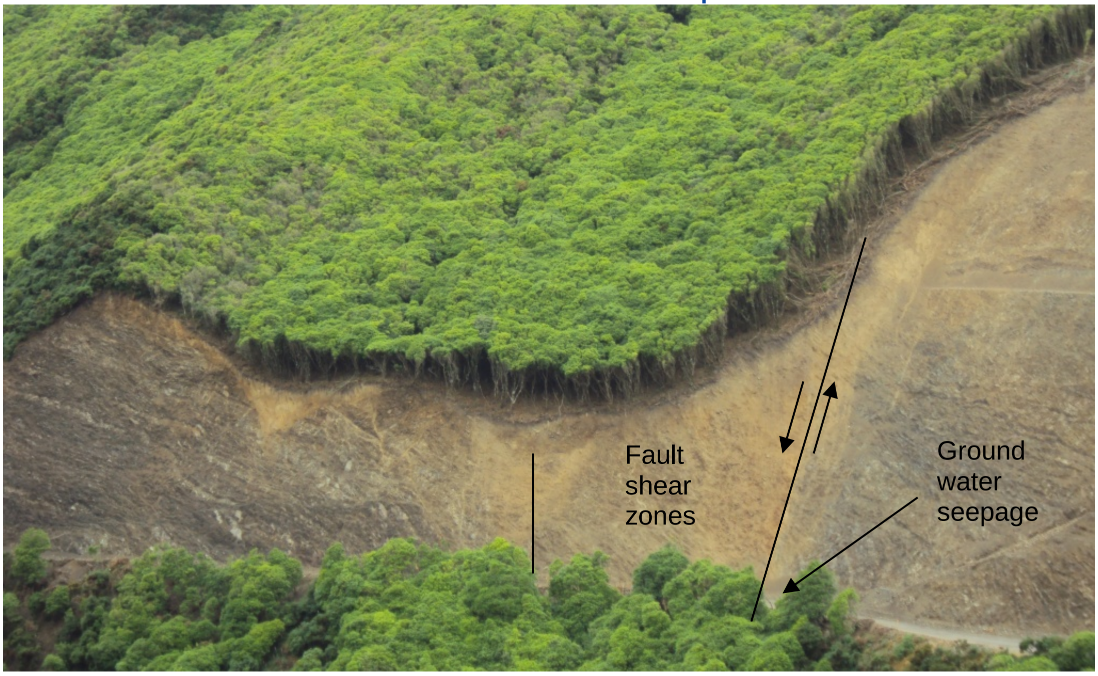


Plate 3-5 Fault shear zone on eastern access road cut slope



3.1.5 Slope Stability Observations

The slopes across the site have a range of angles, typically 20^o to 50^o with some areas 50^o to 70^o and occasionally steeper slopes (up to 80^o). Slopes in the 20^o to 50^o range are typically densely vegetated, with colluvium (0.2 m to 2.4 m thickness) overlying sandstone and mudstone rock.

Natural slopes appear stable where from 0^o to about 40^o, and marginally stable / unstable where slopes greater than 40^o. Colluvium >1 m thickness was not observed on natural slopes steeper than 40^o and no colluvium observed on natural slopes >55^o.

Slope failures were observed where roadways have cut into the natural slopes, where both rock and soils have similar cut angles. A summary of slope height, slope angle and failure mechanism from site observations is presented in **Table 3-2**, below. The majority of slope failures were observed in the areas of newly cut slopes on the eastern access road, where cut slopes range from 55^o to 80^o. The predominant modes of failure include small rotational, wedge and ravelling failures (gravel, cobbles and small boulders (<0.5 m diameter). The older rock slopes on site appear to be more stable than the younger rock slopes, this probably because the majority of rock slope failure occurred during or shortly after construction. The overlying colluvium is generally cut to the same angle as the rock slopes and shallow rotational and translational slides are evident. Small scale slope failure was observed during site works in both wet and dry periods, but the frequency noted to increase during wet periods.

Table 3-2 Summary table of cut slope stability based upon visual inspection during 2007 and 2010 / 2011 site works.

Location	Approximate Age of Cut Slope	Approximate Slope Heights (m)	Slope Angle	Apparent Soil Slope Stability	Apparent Rock Slope Stability
Western access road cutting	Pre-2004	3 m to 8 m	55^o to 80^o	Minor soil slope rotational and translational failure to a few meters depth probably occurred during wet periods.	Rock failure includes ravelling (gravel, cobbles and small boulders <0.5 m) during wet periods and small wedges (~1 m).
Tie in to existing Stage 3 area	Pre-2004 with regrading of some slopes in	Up to 80 m. Benched in some areas, highest unbenched slope is ~35 m high.	50^o to 75^o	Generally no soils present or unable to observe soils due to height of cut.	Where moderately weathered rock, failure includes ravelling (gravel, cobbles and small boulders <0.5 m) and small wedges (~1 m) probably occurred during wet periods. Where unweathered rock lower down the valley slope angles of up to 75^o appear stable.
Eastern access road cutting (Stage 3 area to BH4A)	2006 to 2009	10 m to 40 m. Benched in some areas, highest unbenched slope is ~25 m high.	50^o to 80^o	Minor soil slope translational slides at top of cut during wet periods.	Rock failure includes ravelling (gravel, cobbles and small boulders <0.5 m) during wet periods and small wedges (~1 m).

Location	Approximate Age of Cut Slope	Approximate Slope Heights (m)	Slope Angle	Apparent Soil Slope Stability	Apparent Rock Slope Stability
Eastern access road cutting (From BH4A to BH3A)	2010	5 m to 15 m.	55^{o} to 80^{o}	Minor soil slope translational slides at top of cut during wet and dry periods.	Rock failure includes ravelling (gravel, cobbles and small boulders <1.0 m) and small wedges (<2 m) during wet and dry periods.

3.1.6 Hydrology and Hydrogeology

The only perennial stream is situated in Carey's Gully, other streams were noted to flow only after rainfall events in the tributary gullies.

In general, it was after intense or prolonged rainfall events that groundwater was observed seeping from minor shear zones and faults. The only groundwater seepage observed from a fault in dry weather was minor (film of water / wetting of 2 m^2 area at base of fault) from the significant fault shown in **Plate 3-5** on the eastern access road.

Ponding of water was observed at the base of the access road rock cut slopes where situated on rock.

3.2 Ground Investigation

3.2.1 Site Works

Geotechnical ground investigation field work was undertaken by URS over two periods, including July 2007 to August 2007 and December 2010 to January 2011. Due to the steep topography and dense vegetation the field work was limited to the areas of the eastern and western access roads and the access road in Carey's Gully. The extent of this outcrop is shown in **Plate 3-1** through **Plate 3-5**.

July 2007 to August 2007

The field work included two fully cored HQ triple tube drill holes, BH1A and BH2A, to depths of 30.1 m below ground level (bgl) and 104.5 m bgl respectively. A total of six insitu permeability tests (packer tests) were undertaken and results recorded. Upon completion of the boreholes 50 mm slotted standpipes were installed for groundwater sampling and further permeability testing.

An additional two open hole HQ sized drill holes, BH1B and BH2B, were progressed to depths of 11.0 m bgl and 47.5 m bgl respectively. No rock core was obtained and upon completion the boreholes 50 mm slotted standpipes were installed for groundwater sampling and further permeability testing.

Borehole locations are shown on **Drawing G-001**, borehole logs in **Appendix B** and permeability testing in **Appendix C**.

3 Field Work

December 2010 to January 2011

The field work included two fully cored HQ triple tube drill holes, BH3A and BH4A, to depth of 60.0 m bgl. A total of five insitu permeability tests (packer tests) were undertaken during drilling and results recorded. Upon completion of the boreholes 50 mm slotted standpipes were installed for groundwater sampling and further permeability testing.

An additional three open hole HQ sized drill holes, BH3B, BH3C and BH4B, were progressed to depths between 10.5 m bgl and 26.5 m bgl. No rock core was obtained and upon completion of the boreholes 50 mm slotted standpipes were installed for groundwater sampling and further permeability testing.

Borehole locations are shown on **Drawing G-001**, borehole logs in **Appendix B** and permeability testing in **Appendix C**.

A total of four test pits (TP1 to TP4) were excavated to maximum depth of 2.4 m bgl to obtain soil samples for geotechnical testing. Test pit positions TP1 to TP3 comprised existing exposed soil slopes and were progressed by hand digging. Test pit position TP4 was dug by an excavator. Shear vane tests were carried out at approximately 0.5 m intervals where fine grained soils were encountered. Scala penetrometer tests were carried out to refusal (maximum depth 1.4 m bgl) in undisturbed ground adjacent to test pits TP1, TP2 and TP3.

Test pit locations are shown on **Drawing G-001** and test pit logs in **Appendix B**.

Rock and soil samples obtained for geotechnical testing were delivered to Central Laboratories for testing. Geotechnical testing to characterise the rock and soil geotechnical properties was specified by URS, including:

- Six particle size distribution tests were undertaken by wet sieving method and four particle size distribution tests were undertaken by hydrometer method on soil samples;
- Two Atterberg limits tests on soil samples;
- Five unconfined compressive strength tests were undertaken on rock.

Laboratory test results are included in **Appendix C**.

3.2.2 Superficial Geology

Superficial deposits encountered in test pits TP1 to TP4, typically had 0.05 m to 0.2 m topsoil overlying fine and coarse grained colluvium with a maximum depth of 2.4 m. The colluvium was observed to fine upwards and details summarising the soil characteristics are provided in **Table 3-3**.

Table 3-3 Summary table of soil characteristics

Superficial deposit	Location	Description	Density	Plasticity	Strength and density test results
Colluvium – fine grained	TP1: 0.05 m-0.7m TP2: 0.1 m-0.4 m	Sandy gravelly SILT with some clay; brownish orange.	Firm to very stiff with increasing stiffness with depth	Low plasticity	Vane: 168kPa *Scala: 2 to 6 blows / 100 mm penetration.
Colluvium – coarse grained	TP2: 0.4 m-1.2 m TP3: 0.2 m-1.8 m TP4: 0.2 m-2.4 m	GRAVEL with some sand, minor silt / clay and cobbles; brownish orange.	Loose to dense with increasing density with depth	-	*Scala: 2 to 9 blows / 100 mm penetration.

* Higher scala penetrometer test results than those shown were recorded. These have been discounted as they are likely related to cobble obstructions or bedrock.

3.2.3 Bedrock Geology

Rock recovered from boreholes BH1A to BH4A, comprised moderately weathered to unweathered grey sandstone and dark grey to black mudstone, weak to very strong, highly fractured and with fault and shear zones present. A summary of the rock mass characteristics is presented in **Table 3-4**.

Table 3-4 Summary of rock mass characteristics

Characteristic	BH1	BH2	BH3	BH4
Weathering	Unweathered	Moderately to slightly weathered to 30m bgl, then unweathered.	Moderately to slightly weathered 0.13 m to 13.7 m bgl, then unweathered.	Moderately to slightly weathered 0.30 m to 18.10 m, slightly weathered 18.10 m to 37.90 m, then unweathered.
Lithology	70% sandstone and 30% mudstone (greywacke)	70% sandstone and 30% mudstone (greywacke)	93% sandstone and 7% mudstone (greywacke)	83% sandstone and 17% mudstone (greywacke)
Recovery	0.0 m - 10 m: 50% 10 m – 30 m: 95%	0.0 m - 4.5 m – 0% 4.5 m – 39 m: >95% 39 m – 45 m: ~50% 45 m – 68 m: > 95% 68 m - 74 m: ~50% 74 m – 92 m ~90% 92 m – 97 m ~80% 97 m – 104 m ~30%	0.0 m – 0.13 m 0% 0.13 m – 60.0 m: 91% to 100% but occasional core loss zones (<0.2 m) due to drilling.	0.0 m – 0.30 m 0% 0.30 m – 60.0 m: 92% to 100% but occasional core loss zones (<0.55 m) due to drilling. 46.02 m - 46.86 m: 83%

Characteristic	BH1	BH2	BH3	BH4
Faults and shears	11.2 m - 11.9 m crush zone 21.1 m - 22.0 m crush zone with 2 mm clay seam at 21.9 m	Many crush zones with clay seams observed with thickness <0.05 m. More significant zones (<0.2 m thickness) observed at depths (bgl): 21.20 m; 30.70 m; 31.50 m; 34.70 m; 36.60 m; 37.60m; 38.00 m; 63.98 m; 62.73 m; 77.83 m; 82.13 m; 87.30 m; 86.80 m; 92.00 m; 97.00 m; 98.00 m; 104.00 m.	Many crush zones with fault gouge (clay and gravel) observed with thickness <0.3 m at depths (bgl): 16.88 m; 17.65 m; 20.10 m; 22.95 m; 59.45 m; 58.55 m.	Many crush zones with fault gouge (clay and gravel) observed with thickness <0.3 m at depths (bgl): 21.60 m; 23.65 m; 30.20 m; 32.35 m; 42.15 m.
Unconfined compressive strength (MPa)	No testing	No testing	All tests in sandstone: 5.2 m –93MPa; 52.35 m –87MPa.	All tests in sandstone: 5.19 m - 11.5MPa (failed along joint at 50^{o}); 18.25 m -122MPa; 38.11 m – 103.5MPa.
Bulk Density (kN/m^3)	No testing	No testing	All tests in sandstone: 5.2 m – 2.70kN/m^3; 52.35 m – 2.77kN/m^3.	All tests in sandstone: 5.19 m – 2.59kN/m^3; 18.25 m – 2.67kN/m^3; 38.11 m – 2.680kgN/m^3.
Permeability Tests	25.00 m - 30.00 m – No seal achieved	37.18 m - 41.68 m: No seal achieved 58.8 m - 66.89 m: No seal achieved 73.18 m - 78.16 m: 0Lu 89.30 m - 95.10 m: 0Lu 96.00 m - 100.0 m: 1Lu	No seal achieved	No seal achieved

3.2.4 Groundwater

Groundwater monitoring of the boreholes was undertaken post site works between 7 and 9 March 2011. Details of groundwater levels encountered are presented in **Table 3-5**, below. Groundwater depths levels vary significantly across the site from 0.2 m bgl in BH3C to 28.96 m bgl in BH2A.

The groundwater monitoring indicates that BH1A & B, BH3A to C have high total head, which could be expected from boreholes situated in the bottom of gullies with sizeable catchment area. Boreholes BH4A & B has moderate head that could be typically associated with boreholes on a valley side. Boreholes BH2A & B have low to moderate head that could be typically associated with boreholes on a ridge line.

3 Field Work

Groundwater flow is expected to vary across the site and likely to be controlled by the geomorphology and the discontinuity characteristics of the rock mass. The most important characteristics are likely to be the discontinuities aperture and interconnectivity especially where continuous structures such as shear zones and faults are present.

Table 3-5 Groundwater Monitoring Results

Borehole	Total Depth (m bgl)	Response Zone (m bgl)	7/03/11 Depth to Water (m bgl)	8/03/11 Depth to Water (m bgl)	9/03/11 Depth to Water (m bgl)	Comments
BH1A	29.50	22.80 to 30.10	NM	NM	0.00	Situated adjacent to Carey's Gully Stream (permanent flow in stream). Approx. 25 m from Secondary fault. Shear zones present in RZ.
BH1B	10.85	1.30 to 11.00	NM	NM	0.30	Situated adjacent to Carey's Gully Stream (permanent flow in stream). Approx. 25 m from Secondary fault. Likely that fractures are open from 1.3 m to 3.0 m indicated by no core recovery in BH1A. Shear zones present in RZ.
BH2A	104.50	82.00 to 104.50	28.75	28.91	28.96	Situated upon ridgeline, approx. 60 m from Secondary Fault. Poor core recovery over RZ. Shear zones present in RZ.
BH2B	45.50	25.00 to 47.50	16.42	16.40	16.42	Situated upon ridgeline, approx. 60 m from Secondary Fault. Shear zones present in RZ.
BH3A	60.30	44.50 to 60.30	0.42	0.48	0.54	Situated in a gully (surface water flow after rainfall events). In area of secondary fault. Shear zones present in RZ.
BH3B	25.83	16.00 to 26.5	1.86	1.92	1.51	
BH3C	9.89	1.00 to 10.00	0.20	0.20	0.31	Situated in a gully (surface water flow after rainfall events). In area of secondary fault.
BH4A	55.10	41.00 to 60.00	7.33	7.75	7.85	Situated in a gully. In area of possible fault (URS Lineation 2011). Shear zone at top of RZ.
BH4B	24.10	11.50 to 24.50	2.74	2.78	2.77	

RZ: Response zone NM: Not measured. NA: Not applicable.

Geotechnical Parameters

4.1 Geotechnical parameters

Design parameters for the concept design have been derived from historic investigations and this ground investigation, engineering geology mapping and published literature. These are summarised in the **Table 4-1**, below.

Parameters for the soils have been assumed from values provided by Bowles (1997) supported by site observations of colluvium on high slope angles (stable slopes up to 40^{o}).

The strength of the slightly to unweathered intact rock bedrock in the area is in the order of 100 MPa. However, the strength of the rock mass is influenced by discontinuities as shown by Read et al (2003) and a detailed analysis of discontinuities on proposed rock cut faces will be required to confirm the validity of the assumed rock strengths.

To progress the concept design, the following material parameters have been assumed for the rock.

Table 4-1 Summary of concept design geotechnical parameters

Material Type	Bulk Density (kN/m^3)	Effective Angle of Friction, **Φ**' (degrees)	Effective Cohesion, c' (kPa)	Geological Strength Index (GSI)	UCS (MPa)
Colluvium (sandy gravelly SILT)	16	30*	2	-	-
Colluvium (sandy GRAVEL)	17	40*	0	-	-
Moderately weathered bedrock (sandstone and mudstone - greywacke) 0.0 m to 7.0 m thickness	22	35	15	10**	-
Slightly to unweathered bedrock (sandstone and mudstone - greywacke) >2.0 m depth	27	39	50	40**	100

* From Bowles, J.E., (1997) Foundation analysis and design. ** From Read et al, (2000) Assessment of New Zealand Greywacke Rock Masses with Hoek-Brown Failure Criterion.

4.2 Groundwater

Gauging information indicates that in rock areas the groundwater level in low areas of the site and in the gullies is likely to be near rock surface. For the ridge lines and higher up the slopes the groundwater table is likely to be encountered at depth. In the soils no groundwater flow was observed during the test pit excavation. This information does not account for seasonal effects and storm events.

For design purposes groundwater has been assumed at about 20% of soil thickness. For areas at the bottom of gullies a 2.4 m thick soil layer has been assumed with groundwater level at 0.5 m above the

4 Geotechnical Parameters

top of rock (groundwater depth of 1.9 metres below the soil surface) and for 1.0 m thick soils groundwater level of 0.2 m above top of rock (groundwater depth of 0.8 m below the soil surface) was assumed. Bedrock was assumed to be fully saturated for the lower areas.

Faults

5.1 Faults Affecting Stage 4 Area

The Wellington area is well known for the presence of extensive faulting, both active and inactive. Assessment of the likelihood of fault hazard being present and its effect on the proposed Stage 4 area is presented below.

The main hazards that faults could present to the proposed development, include:

- Unfavourable orientation reducing stability of slopes;
- Shaking or horizontal and vertical ground acceleration reducing stability of slopes;
- Surface fault rupture that may damage the proposed development and / or provide preferential flow paths for landfill leachate.

The first two hazards are discussed in relation to slope stability in **Section 6** of this report. Determining whether surface rupture is likely and its effect is discussed below.

The expected fault locations, which may affect **Stage 4** are indicated on **Drawing G-002** which shows information from Begg & Mazengarb (1996) including the Wellington Fault (Principal fault), NE to SW trending, dextral strike slip, downthrow to the southeast and is situated <1.5 km to the north west of the site. Secondary faults, possibly active and inactive, are indicated to trend north to south through the Stage 4 area with downthrow to the east. During field work there was little evidence of the second order faults presence on site. However, dense vegetation and steep terrain restricted access to the western part of the site.

Other faults were identified during site works and have been discussed in **Section 3.1.4**. Many minor faults are present across the site and a more significant fault identified to be present on the east access road. There is evidence that displacement has occurred on these faults but the timing and extent of displacement is unknown.

Studies have been undertaken by Van Dissen et al. (1992) to determine palaeoseismicity of the Wellington – Hutt Valley segment of the Wellington Fault, with data collected from trench sites in Long Gully / Karori Reservoir (approximately 2 km-4 km from Stage 4 area). They support the idea that the Wellington – Hutt Valley Segment of the Wellington Fault ruptures as a single fault segment. They state that the horizontal slip rate in Long Gully is c. 5 mm/yr which is lower than fault movements further north on the fault at Emerald Hill, which are c.6.0 mm to 7.6 mm/yr, suggesting a difference in slip rate along the fault. They state that the difference in rate may be the result of "off fault" deformation on the low slip, more northerly faults that extend through Wellington City. This indicates that it is possible the difference in movement rates is being taken up by Secondary faults and there are a number of Secondary faults indicated to be 'possibly inactive' in the Stage 4 area. Studies by Langridge et al. (2007) indicate that in most cases, but certainly not all, Secondary faults have low slip rates (<1 mm/yr), and therefore have long rupture recurrence intervals. Although the Wellington Fault does not present a specific rupture hazard to the site, it can generate rupture along Secondary faults within the Stage 4 area.

A summary of the Wellington Fault horizontal displacement, vertical displacement, elapsed time since last rupture event, return period and likely time until next rupture event is presented in **Table 5-1**.

5 Faults

Table 5-1 Wellington Fault Displacement Summary

Author	Horizontal displacement (mm/yr)	Horizontal displacement – single rupture event (m)	Vertical displacement – single rupture event (m)	Elapsed time since last rupture event (yr BP)	Return period (yr BP)
Van Dissen *et al*. (1992)	5.0 (Long Gully) and 6.0 to 7.6 (Emerald Hill)	3.2 to 4.7	No data	360 to 510	420 to 780
Langridge *et al*. (2007)	6.0 to 7.6	3.8 to 4.6	<1.0	295 to 445	500 to 770
Little et al. (2010)	>4.5 to <8.2	5.0	No data	No data	*610 to 1100

* Little *et al*. (2010) revised horizontal displacement rate assuming Wellington-Hutt Valley segment as a whole, implying longer mean earthquake recurrence in comparison to earlier estimates.

Although there is insufficient published literature on Secondary fault displacements due to a single rupture event, we propose displacements on the Secondary faults in the Stage 4 area could be up to 10% of that observed on the Wellington Fault from the geological record. Surface rupture on a secondary fault in the Stage 4 area, generated during the design lifetime (i.e.1:500 year return period), from a single rupture event on the Wellington Fault, may cause displacements in the order of 0.5 m horizontally and 0.1 m vertically. Earthquake return periods for the Wellington Fault have been recently updated by Little *et al*. (2010) who provide return period of ~610-1100 yrs for a magnitude 7.5 earthquake. This in conjunction with elapsed time since last rupture event from Langridge et al. (2007) and Van Dissen *et al*. (1992) indicates an event of similar magnitude is likely to occur within the next 800 yrs.

Slope Stability Assessment

6.1 Introduction

Slope stability assessment is required to determine the risk that existing natural and cut slopes pose to the proposed development in the Stage 4 area. It is also required to determine safe angle of soil cut, rock cut and landfill slopes for the concept design to be developed.

The stability assessments considered include:

- Soils (colluvium) using Slope/W software and geotechnical parameters provided in **Table 4-1**;
- Rock (sandstone and mudstone – greywacke) using information detailed in **Section 3.1**;
- Landfill waste slope and design of buttresses with reference to URS (2010) Stage 3 area report on Slope Stability Assessment, Southern Landfill, Happy Valley, Wellington.

The slope stability assessment presented below would be refined during detailed design.

6.2 Seismic Loading

URS (2010) provides the Peak Horizontal Ground Acceleration (PGA) estimates, expressed as a fraction of earth gravitational acceleration, for the Stage 3 area. The seismic loading philosophy and PGA provided are relevant to the risk assessment and concept design in the Stage 4 area.

Landfills are not specifically referenced in AS/NZS1170.0:2002, however the landfill has been assumed to have an importance level of 2 (*Normal structures and structures not in other importance levels*) for this report to give some guidance as to possible design lifetimes and resultant return periods. Structures at importance level 2 are required to be designed to resist earthquake loadings with return periods of 1: 500 years for a design working life of 50 years.

Peak Horizontal Ground Acceleration (PGA), expressed as a fraction of earth gravitational acceleration, for the site has been calculated using NZS1170.5:2004 as a guideline. Expected Richter magnitude for a 1:500 year return period event in the Wellington region is given by Cousins *et al*. (2009) and provides a PGA of 0.53g for magnitude 7.4 earthquakes.

6.3 Required Safety Factors

Industry practice requires the factors of safety (FOS) for long term stability to be 1.5 for static conditions and for seismic slope stability 1.2. This may be appropriate for natural and cut slopes in soil and rock. However, it is accepted practice for landfill design (refer United States Environment Protection Agency Seismic Design Guidance for Municipal Solid Waste Landfill Facilities) that displacement criteria may be used for the design of landfills under seismic loading. Displacement criteria are considered appropriate for the design of this landfill.

6.4 Soil Slope Stability Assessment

6.4.1 Soil Slope Stability Assessment – Static Analysis

A range of slope stability analyses were carried out for the natural soil slopes (colluvium) on site using the Slope/W V7.17 software and the geotechnical parameters as given **Section 4.1**. The soil slopes were designed to provide assessments of stability for likely typical ranges of slope angles and soil thicknesses. Result outputs are presented in **Appendix D**. Thickness of soil strata has been assumed as 1.0 m for both fine and coarse colluvium, reflecting the most prevalent thickness observed on site. Assessment on a thicker (2.4 m thick) unit of coarse colluvium as encountered in TP4 which had a natural slope angle of 40^o has also been undertaken. This was to identify whether strata thickness has a significant effect on the stability of the slope.

Dense vegetation is expected to improve the stability of soil slopes but has been conservatively excluded from stability analysis. Conversely, the presence of vegetation can reduce the stability of rock slopes due to root jacking action on joints and the additional weight of the vegetation on rock wedges or blocks.

Critical safety factors are summarised in **Table 6-1** below. From the results it can be seen that 30^o to 40^o slopes are marginally stable (FoS ~1.0) and do not satisfy the required factor of safety for long term stability. A 25^o slope satisfies the safety factor requirements for static conditions.

The models show that sensitivity to soil thickness in the coarse colluvium is not significant.

Table 6-1 Summary of Static Safety Factors for Soil (Colluvium)

Strata	Thickness	Water level (~20% strata thickness)	Slope angle	FoS	Acceptable FOS >1.5
Coarse Colluvium (sandy GRAVEL)	2.4 m	0.5 m	40^o	1.00	No
			30^o	1.45	No
			25^o	1.80	Yes
Coarse Colluvium (sandy GRAVEL)	1 m	0.2 m	40^o	0.95	No
			30^o	1.38	No
			25^o	1.73	Yes
Fine Colluvium (sandy gravelly SILT)	1 m	0.2 m	40^o	1.04	No
			30^o	1.38	No
			25^o	1.69	Yes

6.4.2 Soil Slope Stability Assessment – Seismic Analysis

Horizontal seismic loadings were applied to the model and additional slope angles were assessed down to 10^o. The slope stability analysis outputs are presented in **Appendix D**. Critical safety factors are summarised in **Table 6-2** below.

The acceptable safety factor condition of 1.2 under seismic loading is not satisfied for 1:500 year return period for slopes as shallow as 10^o. Marginal stability (FoS ~1) is indicated for a 10^o slope of coarse colluvium and for a 15^o slope in fine colluvium subjected to a 1:500 year return period earthquake.

Table 6-2 Summary of Seismic Safety Factors for Soil (Colluvium), 500 year return period

Strata	Thickness	Water level (~20% strata thickness)	Slope angle	FoS	Acceptable FOS >1.2
Colluvium (sandy GRAVEL)	2.4 m	0.5 m	40^{o}	0.61	No
			25^{o}	0.83	No
Colluvium (sandy GRAVEL)	1 m	0.2 m	40^{o}	0.64	No
			25^{o}	0.81	No
			15^{o}	1.03	No
Colluvium (sandy gravelly SILT)	1 m	0.2 m	40^{o}	0.58	No
			25^{o}	0.79	No
			10^{o}	1.07	No

6.5 Rock slope stability assessment

6.5.1 Rock Slope Stability Assessment – Static Analysis

Failure of rock slopes can occur by several different mechanisms, including planar failure, wedge failure, rotational failure, toppling failure and ravelling failure. To determine the likelihood of which failure mechanism will occur, detailed assessment of the rock mass is required for each proposed cut face as identified by Read *et al* (2003). Such detailed information is not available at this stage in the design. However, engineering geology mapping and site observations of the existing rock cut allows for an observational design approach commonly used in steep or difficult ground conditions.

Discontinuity information presented in **Section 3.1.4** and in borehole logs (See **Appendix B**) indicates the rock to have many discontinuities and the main failure mechanisms could be expected to include:

- Plane failure where bedding, fault or shear zone orientation is unfavourable in relation to the cut slope
- Wedge failure where bedding, fault, shear zone and joint sets intersect to form a wedge and the orientation is unfavourable to the cut slope. Minor wedge failure is expected where joints and small shear intersect. Significant wedge failure is expected where bedding, faults or continuous shear zones intersect.
- Ravelling failure is expected due to the close spacing and number of intersecting discontinuities in the rock mass.

The apparent rock slope stability observed on site is discussed in **Section 3.1.5** with key points summarised as follows:

- Existing rock cut slope angles vary between 50^{o} and 80^{o};
- Older rock cut slopes on the western access road appear to be reasonably stable with only minor ravelling (gravel, cobbles and small boulders <0.5 m) and wedge failure (~1 m) appearing to have occurred during wet periods. Bedding appears to have favourable dip;
- Older rock cut slope at the tie in with Stage 3 area where rock is unweathered appear reasonably stable with slope angles up to ~75^{o}. Where rock is moderately weathered, further up the valley slope, ravelling (gravel, cobbles and small boulders <0.5 m) and minor wedge (~1 m) failure is evident. Bedding appears to have favourable dip;

- Younger rock cut slopes on the eastern access road appear to be unstable with continuous ravelling failure (gravel, cobbles and small boulders <1.0 m) and minor wedge (<2 m) failure during wet and dry periods. Bedding appears to have favourable dip.

It is expected that the proposed cut slopes (stormwater swale and access road) will encounter moderately to slightly weathered sandstone and mudstone. For concept design purposes it is recommended that cut slopes have an angle of 68^{o} (1H:2.5V) with a 2 m wide bench every 10 m in height, which results in an overall average slope angle for a 20 m high slope of 63^{o} (1H:2V). It is anticipated that some areas will encounter less favourable conditions such as where fault shear zones may be present, moderately to highly weathered rock, likelihood of plane or significant wedge failures, and in these areas flatter slopes will be required.

Further ground investigation and rock mass assessment would be addressed fully in detailed design.

6.5.2 Rock Slope Stability Assessment – Seismic

There is currently insufficient information available regarding specific geotechnical parameters of the cut rock slopes to assess horizontal seismic loading effects on the slopes. Further ground investigation and rock mass assessment would be addressed fully during the detailed design and appropriate mitigation developed (e.g., rock bolting, shotcrete, mesh netting) if required.

6.6 Landfill Landform

Based on previous work completed by URS (2010) at this site, the following slope angles are recommended for the purposes of concept design; maximum 1V:4H slope for the final landform (top surface); maximum 1V:3H slope for the landfill operating faces. Further work would be undertaken during detailed design to optimise slope angles, develop mitigation measures for an acceptable risk, and assess the effect of bioslolids on slope stability if they are proposed to be incorporated in to the fill.

Geotechnical Risk

7.1 Geotechnical Hazard Assessment

The geotechnical hazards within the proposed Stage 4 area have been identified by undertaking a desk study, geological mapping, ground investigation, fault hazard assessment and preliminary slope stability analysis. The geotechnical risk has been assessed qualitatively using a risk register, in accordance with the DETR/ICE Guidance (2001), see **Section 7.2**.

The Geotechnical hazard plan is included on **Drawing G-004** (see **Appendix A**).

The main geotechnical hazards identified at the site with their potential mitigation measures are discussed below. This assessment and proposed mitigation would be confirmed and refined as required during detailed design.

7.1.1 Unstable soil slopes

Soil (colluvium) slopes, both natural and cut slopes with insufficient FoS resulting in slope failure may cause blockage of adjacent stormwater swales, damage to infrastructure and/or health and safety hazard.

The likelihood of the hazard being present has been shown in its simplified form to relate to slope angle (see **Tables 6-1 and 6-2**). Soil slopes 25^o to 40^o with a marginal FOS 1.0 to 1.5 present 'negligible' hazard where >50 m from the Stage 4 area, this is increased to 'unlikely' where hazard is <50 m of the Stage 4 area. Soils slopes >40^o with FOS<1 are 'likely' to present a hazard to the Stage 4 area landfill. As soils were generally not observed on slope angles >55^o no hazards iare expected to be present in these areas. Seismic analysis for a 1:500 year return period indicates that slopes >10^o -15^o have a factor of safety <1 and these are 'likely' to present a hazard to the Stage 4 area.

The likely effect of the hazard, shown by slope stability analysis and site observations, is the occurrence of shallow small scale slope failures.

Where slope instability is identified during the detailed design due consideration would be given to the following:

- Assessment of mitigation measures, which could include some or all of the following: reduce soil slope angles; implement drainage measures; retain vegetation; provide catch benches at the top of rock cut slopes.
- Acceptance of risk and hazard is not mitigated, resulting in potential maintenance/remedial works following slope instability

It is noted that in such steep topography that cost effective mitigation measures may not be available in all areas to prevent slope failure during a 1:500 year seismic event. During detailed design mitigation would be developed for an appropriated level of failures.

7.1.2 Unstable rock slopes

For both natural and cut rock slopes failure mechanisms could include unfavourable joint orientation, bedding, joint sets, faults and other discontinuities. The concept design recommendations are based on an observational approach and include the adoption of 68^o (1H:2.5V) slope angles with a 2 m wide bench every 10 m assuming rock quality and discontinuities are favourable.

Observations on site indicate that small scale failure will likely occur with this proposed slope angle, including ravelling failure (gravel, cobbles and small boulders <0.5 m) and minor wedge failure (<2 m).

In areas of unfavourable discontinuity orientation localised larger failures could occur, especially where discontinuities are persistent for large distances (greater than 10 m), such as in bedding, shear zones and fault areas. Slope failures could cause blockage of stormwater surface drains, damage to infrastructure, and / or possible loss of life. The effect of 1:500 year return period horizontal ground acceleration would be assessed during detailed design but is anticipated to result in a higher occurrence of small and large scale failures.

Rock slopes >68^{o}, unfavourable discontinuity orientation, highly weathered and weak rock are present on site. The likely effect of the hazard is slope failure causing blockage of stormwater swale, damage to infrastructure, and / or health and safety issues.

During detailed design mitigation would be developed for an appropriate level of failure. The following would be considered/completed during the detailed design phase:

- Further investigation, if required, to delineate likely failure mechanisms for the variously oriented rock cuts.
- Assessment of the seismic effects from a 1:500 year return period event.
- Assessment of mitigation measures such as the following: reduce rock slope angles; provide catch benching; rock bolting; shotcrete; and wire mesh / netting on rock face.
- Acceptance of risk and hazard is not mitigated, resulting in potential maintenance/remedial works following slope instability.

7.1.3 Landfill slope instability

The concept design of the solid municipal waste final landfill landform (top surface) of 1V:4H and landfill operating faces of 1V:3H will be stable in the static condition but displacements will ‘probably’ occur from a 1:500 year return period event.

During detailed design mitigation would be developed for an appropriate level of failure. The following would be considered/completed during the detailed design phase:

- optimal slope angles;
- design displacements;
- mitigation/protection measures; and,
- the effect on slope stability of biosolids, if they are proposed to be incorporated in to the fill.

7.1.4 Faults

Fault rupture hazard is likely to be present within the Stage 4 area. It is specifically restricted to potential displacement on the Secondary faults. These Secondary faults are potentially active and rupture could occur as a result of rupture of the Wellington Fault that is situated 1.5 km to the northwest. Horizontal displacement may be in the order of 0.5 mm/yr and rupture from a single event (610 to 1100 year return period, Magnitude 7.5) may cause 0.5 m horizontal and 0.1 m vertical (downthrow to the east). The principal risk is fault rupture damaging infrastructure (dams, access roads, stormwater swale, leachate ponds and the existing tunnel and the proposed access extension and the shear zones providing preferential flow paths for groundwater and /or leachate.

Other faults are present on site as shown on **Drawing G-002**; however, the effect of these hazards is significantly less than that presented by the Secondary faults. The hazard presented by these faults is principally expected to be preferential flow paths for groundwater and /or leachate.

During detailed design mitigation would be developed for an appropriate level of failure. The following would be considered/completed during the detailed design phase:

- Further ground investigation, as required, to identify and characterise the faults impacting upon the Stage 4 area.
- Assessment of mitigation measures, which are anticipated to primarily include hazard avoidance by locating critical infrastructure away from faults where possible and / or design measures to accommodate anticipated displacements (e.g., thickened soil liners).
- Acceptance of risk and hazard is not mitigated, resulting in potential maintenance/remedial works following a fault rupture

7.2 Geotechnical hazard register

The geotechnical risks have been assessed qualitatively using a risk register. In the hazard register the risk is assessed by multiplying the likelihood of the hazard arising and its impact on the proposed development (See **Table 7-1**), in accordance with the DETR/ICE Guidance (2001) on Managing Geotechnical Risk. The geotechnical risk assessment is presented in **Table 7-2**.

Table 7-1 Geotechnical Risk Matrix

Scale of Likelihood of Presence of Geotechnical Hazard	
Scale	Likelihood
4	Probable
3	Likely
2	Unlikely
1	Negligible

Scale of Effect of Presence of Geotechnical Hazard on Construction or Completed infrastructure	
Scale	Effect
4	Very high
3	High
2	Low
1	Very low

Degree of Risk = Scale of Likelihood x Scale of Effect	
Scale	Risk Level
13-16	Intolerable
9-12	Substantial
5-8	Significant
1-4	Trivial

7 Geotechnical Risk

Table 7-2 Qualitative Geotechnical Risk Assessment

Geotechnical Hazard	Possible Impact	Initial Risk			Either do nothing or implement mitigation measures to reduce risk.	Area where hazard is expected to be present	Residual Risk following Mitigation		
		Likelihood	Effect	Risk			Likelihood	Effect	Risk
UNSTABLE SOIL SLOPES									
Soil slope angles 25^{o} to 40^{o} (FoS 1.0 to 1.5)	Slope failure causing: blockage of adjacent open surface drainage; damage to infrastructure; and / or possible loss of life.	2	3	6	Reduce soil slope angles; implement drainage measures; retain vegetation; provide catch benches at the top of rock cut slopes.	See slope angles 25^{o} to 40^{o} presented on **Drawings G-004 and G-005**	1	3	3
Soil slope angles >40^{o} (FoS <1.0)		3	3	9		See slope angles >40^{o} presented on **Drawings G-004 and G-005**	1	3	3
Earthquake 1:500 year return period event causing surface shaking / ground acceleration.		3	3	9	-	Soil slope angles >10^{o} to 15^{o} are likely to fail.	-	-	-
UNSTABLE ROCK SLOPES									
Rock slope angles >68^{o}	Slope failure causing: blockage of adjacent open surface drainage; damage to infrastructure; and / or possible loss of life.	4	3	12	Further ground investigation to assess rock mass characteristics; reduce rock slope angles; provide catch benching; rock bolting; shotcrete; and wire mesh / netting on rock face.	Rock slope cut for stormwater swale and access roads. Also natural rock slopes in steep gully sides.	2	3	6
Unfavourable discontinuity orientation		4	3	12		Rock slope cut for stormwater swale and access roads. Also natural rock slopes in steep gully sides.	2	3	6
Highly weathered or weak rock		4	3	12		Rock slope cut for stormwater swale and access roads. Also natural rock slopes in steep gully sides and associated with fault shear zones. See **Drawings G-004 and G005** for locations.	2	3	6
Earthquake 1:500 year return period event causing surface shaking / ground acceleration.	Slope failure leading could lead to: blockage of adjacent open surface drainage; damage to infrastructure; and / or possible loss of life.	3	4	12	Further ground investigation to assess rock mass characteristics and seismic stability modelling; reduce rock slope angles; provide catch benching; rock bolting; shotcrete; and wire mesh / netting on rock face.	Site wide	3	3	9
LANDFILL LANDFORM									
Earthquake 1:500 year return period event causing surface shaking / ground acceleration.	Displacement of landfill material damaging the leachate ponds, stream diversion, dams and other critical infrastructure	3	4	12	Protection of stream diversions; dams; leachate ponds; and any other critical infrastructure.	Within and immediately adjacent to Stage 4 area	3	2	6
FAULTS									
Secondary faults (published maps)	Fault displacement or rupture damaging infrastructure (dams, access roads, surface drainage, leachate ponds and existing tunnel and proposed access	3	4	12	Further ground investigation is undertaken to identify and characterise the faults. Locate dams and other infrastructure away from faults where possible and / or design to account displacement and rupture fault.	Location of Secondary faults presented on **Drawings G-004 and G-005**	3	2	6
	Preferential flow path for groundwater or leachate	3	3	9	Further ground investigation is undertaken to identify and characterise the faults. Thickened liners or cover.	Location of Secondary faults presented on **Drawings G-004 and G-005**	2	3	6
Significant fault (URS 2010 / 2011 field work)	Preferential flow path for groundwater and / or leachate.	3	3	9	Further ground investigation is undertaken to identify and characterise the faults. Thickened liners or cover.	Within and immediately adjacent to Stage 4 area	2	3	6
Minor fault or shear zones (URS 2010 / 2011 field work)	Preferential flow path for groundwater and / or leachate.	3	2	6	Further ground investigation is undertaken to identify and characterise the faults. Thickened liners or cover.	Within and immediately adjacent to Stage 4 area	1	2	2

7 Geotechnical Risk

Geotechnical Hazard	Possible Impact	Initial Risk			Either do nothing or implement mitigation measures to reduce risk.	Area where hazard is expected to be present	Residual Risk following Mitigation		
		Likelihood	Effect	Risk			Likelihood	Effect	Risk
SURFACE WATER AND GROUNDWATER									
High groundwater table	Flooding of landfill or insufficient drainage.	3	3	9	Further investigation to identify areas of high groundwater table. Construction of dams, storm water swales, stream diversions to prevent flooding of landfill.	In gullies and areas of poor drainage.	1	3	3
High permeability rock	Preferential flow path for groundwater or leachate.	3	4	12	Further ground investigation to identify areas of high permeability rock. Liners to be used.	May be associated with shear zones and areas of highly fractured rock.	1	4	3

Conclusions and Recommendations

8.1 Conclusions

This report presents the findings of a geotechnical study to be used in support of Southern Landfill Stage 4 area resource consent, including an assessment of the geotechnical risks to the proposed Southern Landfill Stage 4 area. It also provides geotechnical parameters and recommendations to be incorporated into concept and detailed design.

The site comprises a deeply incised valley with steep slopes and natural slope angles typically between 20° to 50°. Slopes steeper than 50° are present, which generally comprise rock, typically encountered in the bottom of the main gully, access road cut slopes and where Stage 4 area overlaps with Stage 3 area. These steeper slopes comprise either very thin soils over rock or rock, with little to no soil observed on slopes >55°. There is approximately 200 m of relief between natural valley floors and ridges.

Geotechnical ground investigation field work was undertaken by URS over two periods, including July 2007 to August 2007 and December 2010 to January 2011. This included boreholes, test pits and engineering geology mapping. The test locations were restricted by the steep topography and dense vegetation but are considered appropriate for this Phase 1 Geotechnical Report.

Insitu testing (permeability) was carried out, samples obtained for geotechnical testing and monitoring wells installed in the boreholes for further permeability testing.

Colluvium, typically firm to stiff sandy gravelly silts and loose to medium dense sandy gravels of about 1m thickness, locally 2 m to 3 m thickness overlies bedrock where slopes <55^{o}, on steeper slopes bedrock is at surface. Rock generally comprises moderately to unweathered sandstone and mudstone (greywacke), strong (locally weak to very strong) and many discontinuities present (joint sets, faults, shears, bedding).

Groundwater table and flow is expected to vary across the site and likely to be controlled by the geomorphology and the discontinuity characteristics of the rock.

Geotechnical parameters have been provided for concept and detailed design. The sandy gravelly silts have bulk density of 16 kN/m^3; Φ' of 30^{o}; c' of 2kPa, and sandy gravel have bulk density of 17 kN/m^3; Φ' of 40^{o}. Groundwater level is assumed to be 20% soil layer thickness and rock is expected to be fully saturated.

The strength of the intact rock bedrock in the area is >100MPa; however, the strength of the rock mass is controlled by discontinuities as shown by Read *et al* (2003). Detailed analysis of discontinuities on each proposed rock cut face is required to determine whether defect patterns are favourable and thus provide realistic parameters for the sandstone and mudstone (greywacke).

The Wellington Fault is located 1.5 km to the north west of the site and several Secondary faults are present in the Stage 4 area. Estimated displacements on the Secondary faults in the Stage 4 area could be up to 10% of that observed on the Wellington Fault from the geological record. Surface rupture on a secondary fault in the Stage 4 area, generated during the design lifetime (i.e., 1:500 year return period), from a single rupture event on the Wellington Fault, may cause displacements in the order of 0.5 m horizontally and 0.1 m vertically. Earthquake return periods for the Wellington Fault have been recently updated by Little *et al*. (2010) who provide return period of ~610-1100yrs for a magnitude 7.5 earthquake. This in conjunction with elapsed time since last rupture event from Langridge et al. (2007) and Van Dissen *et al*. (1992) indicates an event of similar magnitude is likely to occur within the next 800 yrs. Other smaller faults and shear zones are also present on site.

8 Conclusions and Recommendations

Soil slopes have been modelled using Slope-W indicating that slope angles in static condition: <25^o with FoS >1.5 are stable; 25^o to 40^o with FoS 1.0 to 1.5 are marginally stable; >40^o with FOS <1.0 are unstable. This analysis is supported by site observations. Seismic conditions for a 1:500 year return period event were applied to the model and indicated that slopes >10^o-15^o have FOS <1.0 and are unstable. Should they occur, soil slope failures are expected to consist of shallow small scale failures.

Rock slopes, both natural and cut, failure mechanisms could include unfavourable joint orientation including bedding, joint sets, faults and other discontinuities. For the purposes of the concept design a rock cut slope angle of 68^o (1H:2.5V) with a 2 m wide bench every 10 m in height was adopted. It is anticipated that some areas will encounter less favourable conditions and in these areas flatter slopes will be required. Observations on site indicate that small scale failure will likely occur with this proposed slope angle, including ravelling failure (gravel, cobbles and small boulders <0.5 m) and minor wedge failure (<2 m). A 1:500 year return period seismic event could be expected to result in higher occurrence of small and large scale failures.

Previous work by URS (2010) indicates that for disposal of solid municipal waste, final landfill landform (top surface) slope of 1V:4H and landfill operating faces slopes of 1V:3H will be stable in the static condition but displacements will probably occur from a 1:500 year return period event. This work did not consider the effect on the slope stability of the landfill landform if non-solid municipal waste / sludge is incorporated into the landfill.

A qualitative geotechnical risk assessment has been undertaken showing the geotechnical hazards to the Stage 4 area to include: unstable soil slopes; unstable rock slopes; faults; landfill landform; and surface water and groundwater. Preliminary mitigation measures have been proposed to reduce the likelihood of presence of the geotechnical hazard and subsequent residual risk assessed.

8.2 Recommendations

A Phase 1 geotechnical assessment has been undertaken in support of concept design and consenting. This assessment includes a preliminary analysis of geotechnical risks, hazards and mitigation measures.

This assessment and proposed mitigation would be confirmed and refined as required during detailed design. Further ground investigations would be undertaken, as required, fully assessing geotechnical hazards and providing geotechnical parameters for detailed design. Should these investigations identify secondary faults and shear zones within or adjacent to the proposed footprint of the proposed landfill cell appropriate mitigation would be developed and implemented. The following concept design slopes, parameters and assumptions would be optimised and/or confirmed during detailed design:

- Maximum soil cut slope of ~25° (1V:2H) to achieve the industry standard FOS of 1.5
- Maximum rock cut slopes adopt an angle of 68^o (1H:2.5V) with a 2 m wide bench every 10 m in height
- Maximum slope angle of the top surface of the municipal solid waste landfill landform of 1V:4H and embankment slope angle of 1V:3H
- Effect of biosolids or other non-municipal solid-waste material on landfill stability should it be incorporated in to the fill
- Location-specific mitigation measures for an appropriate level of failures
- Acceptable design displacements for slope stability

8 Conclusions and Recommendations

- Anticipated displacements during secondary fault rupture

References

AS/NZS 1170.0:2002. *Structural Design Actions, Part 0: General Principals.*

Begg J.G. & Mazengarb, C., 1996. Geology of the Wellington Area, 1:50,000. Institute of Geological and Nuclear Science geological Map 22.

Cousins. W., Power. W., Destegul. U., King. A., Trevethick. R., Blong. R., Weir. B., Miliauskas. B., 2009. *Earthquake and Tsunami Losses from Major Earthquakes Affecting the Wellington Region.* 2009 NZSEE Conference, Paper No. 24.

Clayton, C., 2001. *DETR/ICE Guidance: Managing Geotechnical Risk*.

Langridge, R.M., Van Dissen, R.J., Rhoades, D.A., Villamor, P., Wilson, K.J., Litchfield, N.J., Stirling, M.W., Clark, D., Lyman, A., Barnett, E., Carne, R., 2007. *Statistical assessment of earthquake recurrence for the W-HV segment of the Wellington Fault, New Zealand.* p. F1753 (abstract S14C-03) IN: *AGU Fall Meeting, 10-14 December 2007, San Francisco, California.* Washington, DC: American Geophysical Union.

Little, T.A., Van Dissen, R.J., Rieser, U., Smith, E.G.C., Langridge, R.M., 2010. *Coseismic strike slip at a point during the last four earthquakes on the Wellington Fault near Wellington, New Zealand*. Journal of geophysical research. Solid earth, 115: B05403,

NZS 1170.5:2004. *Structural Design Actions, Part 5: Earthquake Actions – New Zealand.*

Read, S.A.L., Richards, L., Cook, G.K., 2003. *Rock Mass Defect Patterns and the Hoek-Brown Failure Criterion. ISRM 2003 - Technology Roadmap for Rock Mechanics*, South African Institute of Mining and Metallurgy, 2003.

Read S.A.L., Richards, L., Perrin, N.D., 2000. Assessment of New Zealand Greywacke Rock Masses with Hoek-Brown Failure Criterion. p. 947-954 IN: Handley, M.; Stacey, D. (comps) *International Society for Rock Mechanics, 10th Congress: technology roadmap for rock mechanics.* Johannesburg: South African Institute of Mining and Metallurgy. Symposium series / South African Institute of Mining and Metallurgy S33

Richardson.G, Kavazanjian. E, Matasovi. N., 1995. *Seismic Design Guidance for Municipal Solid Waste Landfill Facilities.* RCRA Subtitle D (258), United States Environmental Protection Agency.

URS NZ Ltd., 2010. *Slope Stability Assessment Southern Landfill*, *Happy Valley Road, Wellington*.

Van Dissen, R.J.; Berryman, K.R., Pettinga, J.R., Hill, N.L., 1992. *Palaeosysmicity of the Wellington-Hutt Valley Segment of the Wellington fault, North Island, New Zealand.* New Zealand Journal of Geology and Geophysics, 1992, Vol. 35: 165-176.

Yetton, M. & Robertson, M., 1994. *Geology and Hydrogeology of The Carey's Gully Landfill Wellington.*

10

Limitations

10.1 Geotechnical Report

URS New Zealand Limited (URS) has prepared this report in accordance with the usual care and thoroughness of the consulting profession for the use of Wellington City Council and only those third parties who have been authorised in writing by URS to rely on the report. It is based on generally accepted practices and standards at the time it was prepared. No other warranty, expressed or implied, is made as to the professional advice included in this report. It is prepared in accordance with the scope of work and for the purpose outlined in the Proposal dated November 2009.

The methodology adopted and sources of information used by URS are outlined in this report. URS has made no independent verification of this information beyond the agreed scope of works and URS assumes no responsibility for any inaccuracies or omissions. No indications were found during our investigations that information contained in this report as provided to URS was false.

This report was prepared between February 2012 and August 2013 and is based on the conditions encountered and information reviewed at the time of preparation. URS disclaims responsibility for any changes that may have occurred after this time.

This report should be read in full. No responsibility is accepted for use of any part of this report in any other context or for any other purpose or by third parties. This report does not purport to give legal advice. Legal advice can only be given by qualified legal practitioners.

This report contains information obtained by inspection, sampling, testing or other means of investigation. This information is directly relevant only to the points in the ground where they were obtained at the time of the assessment. The borehole logs indicate the inferred ground conditions only at the specific locations tested. The precision with which conditions are indicated depends largely on the frequency and method of sampling, and the uniformity of conditions as constrained by the project budget limitations. The behaviour of groundwater and some aspects of contaminants in soil and groundwater are complex. Our conclusions are based upon the analytical data presented in this report and our experience. Future advances in regard to the understanding of chemicals and their behaviour, and changes in regulations affecting their management, could impact on our conclusions and recommendations regarding their potential presence on this site.

Where conditions encountered at the site are subsequently found to differ significantly from those anticipated in this report, URS must be notified of any such findings and be provided with an opportunity to review the recommendations of this report.

Whilst to the best of our knowledge information contained in this report is accurate at the date of issue, subsurface conditions, including groundwater levels can change in a limited time. Therefore this document and the information contained herein should only be regarded as valid at the time of the investigation unless otherwise explicitly stated in this report.

Appendix A Drawings

G001 Engineering Geology Summary Plan

G002 Fault Hazard Plan

G003 Slope Angles Plan

G004 Geotechnical Hazard Plan Option 1: Top Down

see separate pdf's

B

Appendix B Borehole and Test Pit Logs

BH1A

BH1B

BH2A

BH2B

BH3A

BH3B

BH3C

BH4A

BH4B

TP1

TP2

TP3

TP4

See separate pdf

C

Appendix C Insitu Testing and Laboratory Testing

see separate pdf

URS

D

Appendix D Slope Stability Analysis

Soil Slope Static Analysis

Soil Slope Seismic Analysis

see separate pdf

URS

WELLINGTON OFFICE
160 Lambton Quay
Wellington 6011
T: 04 496 3750
F: 04 496 3755

GOVERNMENT OIL & GAS INFRASTRUCTURE POWER INDUSTRIAL

URS is a leading provider of engineering, construction, technical and environmental services for public agencies and private sector companies around the world. We offer a full range of programme management; planning, design and engineering; systems engineering and technical assistance; construction and construction management; operations and maintenance; and decommissioning and closure services for power, infrastructure, industrial and commercial, and government projects and programmes.

NEW ZEALAND OFFICES
Auckland T: +64 9 355 1300
Tauranga T: +64 7 927 8171
Wellington T: +64 4 496 3750
Christchurch T: +64 3 374 8500

www.urscorp.co.nz

URSNZXXXX 22082013