

Technical Report No 11

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning


Geotechnical Engineering and Geo-hazard Report

November 2012



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Quality Assurance Statement			
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This Technical Report has been produced in support of the Assessment of Environmental Effects (AEE) for the Main South Road Four Laning and Christchurch Southern Motorway Stage 2 Project. It is one of 20 Technical Reports produced (listed below), which form Volume 3 of the lodgement document. Technical information contained in the AEE is drawn from these Technical Reports, and cross-references to the relevant reports are provided in the AEE where appropriate.

A Construction Environmental Management Plan (CEMP) has been prepared to provide the framework, methods and tools for avoiding, remedying or mitigating environmental effects of the construction phase of the Project. The CEMP is supported by Specialised Environmental Management Plans (SEMPs), which are attached as appendices to the CEMP. These SEMPs are listed against the relevant Technical Reports in the table below. This Technical Report is highlighted in grey in the table below. For a complete understanding of the Project all Technical Reports need to be read in full along with the AEE itself; however where certain other Technical Reports are closely linked with this one they are shown in bold.

Schedule of Technical Reports for the AEE

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For further information on the structure of the lodgement documentation, refer to the 'Guide to the lodgement documentation' document issued with the AEE in Volume 1.

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Executive Summary

A series of geotechnical assessment, investigation and testing has been completed in order to delineate the ground conditions and identify potential geotechnical constraints associated with the proposed motorway construction of the Christchurch Southern Motorway Stage 2 and the Main South Road Four-Laning project (the Project).

The ground conditions encountered during the investigations has proven the published geology for the proposed alignment, comprising the Yaldhurst and Halkett members of the Springston Formation. For civil engineering and geotechnical purposes, these soils have been subdivided into units comprising top soil, sandy silt, sandy gravel and silty sandy clay.

Assessment of the engineering geological characteristics of the Project site has identified the recent earthquake events in the Canterbury region as the most significant element that requires particular consideration at the detailed design stage. Other engineering considerations of settlement, bearing capacity and slope stability do not pose a significant hazard.

With respect to seismic activity, ground shaking and liquefaction remain the major geological hazard in the Christchurch area with ground accelerations (PGAs) of 0.2 g recorded in the vicinity of the Project corridor. Ground movement associated with the seismic events have recorded horizontal movement up to 900 mm and vertical movement of up to 320 mm in the Project corridor area. Whilst liquefaction was extensive in the eastern suburbs of the city, no liquefaction was recorded in the vicinity of the Project. However, the interpretation of the investigation data has proven that liquefiable soils do exist at depth, and this will require consideration at the detailed design stage in terms of impact to foundations and structures.

While some land within the Project alignment has been identified as contaminated, concentrations of contaminants within the designation boundaries are consistent with background concentrations and compliant with the National Environmental Standard¹, as detailed in Contaminated Land Assessment (Technical Report 16).

Groundwater levels have been investigated from the perspective of their influence on geotechnical parameters and foundation solutions. Groundwater levels appear to be relatively shallow towards Christchurch at 6 to 14 m below ground level (bgl), increasing in depth towards Rolleston at 13 to 16 m bgl. Groundwater monitoring throughout the investigation period indicates the earthquake events have had negligible effect on the recorded levels. In general terms, the groundwater is at sufficient depth not to impact on construction methodology or techniques in order to accommodate an elevated groundwater. To this extent, it is not envisaged that large scale dewatering systems will be required, although some localised dewatering during deeper trenching or excavations may be needed on a site by site basis.

¹ Resource Management (National Environmental Standard for Assessing and Managing Contaminants in Soil to Protect Human Health) Regulations 2011.

Impact to groundwater from construction is subject to on-going assessment. Groundwater and stormwater issues are assessed in detail within the Assessment of Stormwater Disposal and Water Quality (Technical Report 3).

Geotechnical testing has provided guideline geotechnical parameters for concept design, which are summarised as follows:

Classification:	Low plasticity inorganic silt and clay Medium dense to dense coarse gravels
Soil Strength:	Strengths within the soil profiles vary with lateral and vertical extent concordant with the lithological variations recorded.
Compaction:	Soils testing from the CSM2 alignment tend to be 'dry' of optimum moisture content. Soils from MSRFL are near optimum moisture content.
CBRs	Design values of 4% are to be adopted for CSM2 and 3.7% for MSRFL.

At this concept design stage, geotechnical design has taken into account the need to provide stable embankment, foundations and bridge structures in the event of seismic activity, whilst providing cost effective solutions to construction form and foundations. A design life of 100 years has been adopted. Ground conditions are conducive to traditional strip and pad foundations, but where higher structural loading is required or ground conditions are marginal, piled foundations can be considered. Where piled foundations are used, vibration during their construction must be considered and managed by employing the appropriate piling method when in proximity to structures and or residential property.

There are limited requirements for cut and fill along the proposed alignment due to the design being at or near current grade. In addition, the soil profiles indicate the majority of material won from site will comprise silts, which will potentially restrict its use to non-structural fill used in landscaping. The presence of the silt will necessitate careful control of earthworks to ensure that dust nuisance and silt runoff is kept to a minimum. This can be achieved by ensuring that the dust is suppressed by water spraying from water carts and silt runoff controlled by settlement ponds.

Where sands and gravels are won from any excavations, these will be suitable to augment any structural fill required for embankments and hard fill. The majority of fill required on site will be won from the various consented gravel pits in the region. However, there are opportunities to use alternate sources of fill such as waste glass and demolition hard fill, especially crushed concrete, as a result of the current recovery work being undertaken in Christchurch.

From a geotechnical and geological perspective, the design and construction of the Project is relatively straight forward with few inherent risks associated with ground conditions or the environment. The flat topography and relatively homogenous geology comprising granular alluvial soils provides a low risk environment.

The construction period may cause effects on the environment, arising from surface activities such as earthworks, stockpiling, noise and vibration. Good engineering and construction management practice will mitigate or control these activities and potential effects on the environment through

well-developed management plans and on-going monitoring so that any effects are no more than minor.

The specific risks identified for the proposed Project and its construction, their effects on the environment and recommended mitigation are summarised in Table 8 of Section 11 of this report.

1. Summary Report Plan

Table 1 below highlights the main topics covered in this report with a brief summary to inform the reader of the contents of each section.

Table 1 Summary Report Plan

Section	Description
Proposal Description	A summary of the proposal in terms of the rationale for its construction, description of the route, a description of the two main components of CSM2 and MSRFL, the associated structures and infrastructure and a summary of the approvals sought
Geotechnical Investigations	A summary of the previous and current data and information available
Geotechnical Domains	Route characterisation and the various associated structures in terms of their geotechnical setting
Engineering Geology	Description and summary of the geological conditions in the context of civil engineering construction
Geological Hazards and Seismicity	Discussions of the hazards that impact and influence the route and construction including recent seismic events in the Canterbury Region.
Groundwater	Summary of the currently available information and the investigations undertaken in order to confirm the current regime and impact to construction.
Geotechnical Parameters	Characterisation of the engineering properties including soil classification, strength, compaction, settlement and earthworks
Geotechnical Design Philosophy	Application of the collated data and its influence on foundation and earthworks design
Risks, Effects & Recommended Mitigation	Highlights the risks and effects due to construction and long term impacts to the environment. Discussion of potential mitigation and management techniques to reduce the impacts.

2. Proposal Description

2.1 Overview

The NZ Transport Agency (NZTA) seeks to improve access for people and freight to and from the south of Christchurch via State highway 1 (SH1) to the Christchurch City centre and Lyttelton Port by constructing, operating and maintaining the Christchurch Southern Corridor. The Government has identified the Christchurch motorway projects, including the Christchurch Southern Corridor, as a road of national significance (RoNS).

The proposal forms part of the Christchurch Southern Corridor and is made up of two sections: Main South Road Four Laning (MSRFL) involves the widening and upgrading of Main South Road (MSR), also referred to as SH1, to provide for a four-lane median separated expressway; and the construction of the Christchurch Southern Motorway Stage 2 (CSM2) as a four-lane median separated motorway. The proposed construction, operation and maintenance of MSRFL and CSM2, together with ancillary local road improvements, are referred to hereafter as 'the Project'.

2.2 MSRFL

Main South Road will be increased in width to four lanes from its intersection with Park Lane north of Rolleston, for approximately 4.5 km to the connection with CSM2 at Robinsons Road. MSRFL will be an expressway consisting of two lanes in each direction, a median with barrier separating oncoming traffic, and sealed shoulders. An interchange at Weedons Road will provide full access on and off the expressway. MSRFL will connect with CSM2 via an interchange near Robinsons Road, and SH1 will continue on its current alignment towards Templeton.

Rear access for properties fronting the western side of MSRFL will be provided via a new road running parallel to the immediate east of the Main Trunk rail corridor from Weedons Ross Road to just north of Curraghs Road. For properties fronting the eastern side of MSRFL, rear access is to be provided via an extension of Berketts Drive and private rights of way.

The full length of MSRFL is located within the Selwyn District.

2.3 CSM2

CSM2 will extend from its link with SH1 / MSRFL at Robinsons Road for approximately 8.4 km to link with Christchurch Southern Motorway Stage 1 (CSM1, currently under construction) at Halswell Junction Road. The road will be constructed to a motorway standard comprising four lanes, with two lanes in each direction, with a median and barrier to separate oncoming traffic and provide for

safety.² Access to CSM2 will be limited to an interchange at Shands Road, and a half-interchange with eastward facing ramps at Halswell Junction Road. At four places along the motorway, underpasses (local road over the motorway) will be used to enable connectivity for local roads, and at Robinsons / Curraghs Roads, an overpass (local road under the motorway) will be provided. CSM2 will largely be constructed at grade, with a number of underpasses where elevated structures provide for intersecting roads to pass above the proposed alignment.

CSM2 crosses the Selwyn District and Christchurch City Council boundary at Marshs Road, with approximately 6 km of the CSM2 section within the Selwyn District and the remaining 2.4 km within the Christchurch City limits.

2.4 Key Design Features

The key design features and changes to the existing road network (from south to north) proposed are:

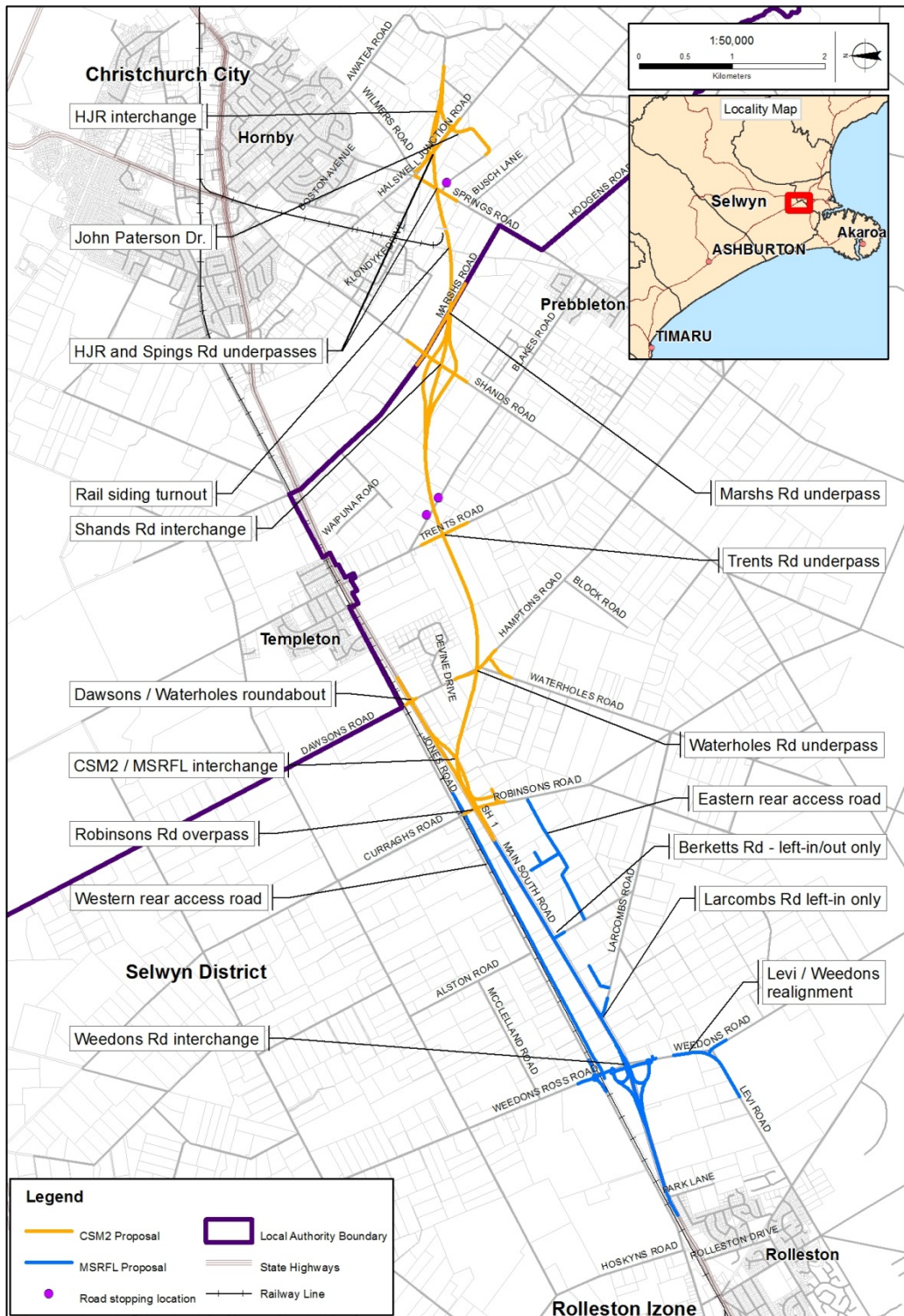
- a new full grade separated partial cloverleaf interchange at Weedons Road;
- a new roundabout at Weedons Ross / Jones Road;
- a realignment and intersection upgrade at Weedons / Levi Road;
- a new local road running to the immediate east of the rail corridor, to the west of Main South Road, between Weedons Ross Road and Curraghs Road;
- alterations and partial closure of Larcombs Road intersection with Main South Road to left in only;
- alterations to Berketts Road intersection with Main South Road to left in and left out only;
- a new accessway running to the east of Main South Road, between Berketts Road and Robinsons Road;
- an overpass at Robinsons and Curraghs Roads (the local roads will link under the motorway);
- construction of a grade separated y-junction (interchange) with Main South Road near Robinsons Road;
- a link road connecting SH1 with Robinsons Road;
- a short new access road north of Curraghs Road, adjacent to the rail line;
- a new roundabout at SH1 / Dawsons Road / Waterholes Road;
- an underpass at Waterholes Road (the local road will pass over the motorway);
- an underpass at Trents Road (the local road will pass over the motorway);

² CSM2 will not become a motorway until the Governor-General declares it to be a motorway upon request from the NZTA under section 71 of the Government Rounding Powers Act 1989 (GRPA). However, for the purposes of this report, the term “motorway” may be used to describe the CSM2 section of the Project.

- the closure of Blakes Road and conversion to two cul-de-sacs where it is severed by CSM2;
- a new full grade separated diamond interchange at Shands Road;
- an underpass at Marshs Road (the local road will pass over the motorway);
- providing a new walking and cycling path linking the Little River Rail Trail at Marshs Road to the shared use path being constructed as part of CSM1;
- an underpass at Springs Road (the local road will pass over the motorway);
- a new grade separated half interchange at Halswell Junction Road with east facing on and off ramps linking Halswell Junction Road to CSM1; and
- closure of John Paterson Drive at Springs Road and eastern extension of John Paterson Drive to connect with the CSM1 off-ramp via Halswell Junction Road roundabout (east of CSM2).

The proposed alignment is illustrated in Figure 1 and encompasses the MSRFL and CSM2 alignments between Rolleston and Halswell Junction Road.

Figure 1 Proposal Location Map



3. Geotechnical Investigations

3.1 General

The Geotechnical Investigations for this Project were undertaken in sequence following the NZTA's Standard Professional Services Specifications for Investigation and Reporting. The investigations were undertaken in the following order:

- Preliminary Geotechnical Appraisal (comprising desk study review);
- Geotechnical Scoping Report (identifying likely ground conditions and required physical investigations);
- Phase 1 Geotechnical investigations (comprising physical ground investigations along MSRFL and CSM2 alignment and being reported as Factual and Interpretative Reports); and
- Phase 2 Geotechnical investigations (comprising physical ground investigations to complement and infill the Phase 1 investigation).

The Phase 1 and Phase 2 investigations were predicated by the need to advance investigations to gain preliminary data on those parts of the route where access could be arranged. The Phase 2 investigations were undertaken once land owner access was available for investigational drilling and test pitting.

3.2 Preliminary Geotechnical Appraisal

The Preliminary Geotechnical Appraisal was issued on 30 August 2010. The purpose of the investigation was to identify the likely geotechnical characteristics of the site including risk areas and to recommend suitable field and laboratory testing requirements.

3.3 Field Investigations and Laboratory Testing

These were undertaken in two phases with Phase 1 between October to November 2010 and Phase 2 in April to May 2011. The cone penetrometer tests (CPTs) programmed under Phase 1 were found to be ineffective on a concurrent project in the same area, due to their inability to progress through the gravels. Air flush rotary boreholes were substituted to complement the cable tool boreholes.

A total of 31 air flush boreholes and 11 cable tool boreholes were drilled. Eight auger holes and associated scala penetrometer testing was carried out, in which falling head tests were conducted. 40 test pits were excavated with a number of infiltration tests carried out to assess ground soakage.

Laboratory testing consisted of determination of natural moisture contents, Atterberg limits (5), particle size distributions (8), soaked laboratory CBR tests (12) and standard compaction tests (14).

4. Geotechnical Domains and Settings

The Project route has been divided into its two main sections comprising MSRFL and CSM2, based on the type of construction required in each section, which has dictated the type and nature of geotechnical investigations undertaken. Whilst their geological and geotechnical settings are very similar, the division of the route allows the characterisation in terms of geotechnical domain. In addition, a number of sub sections exist across both domains that relate to specific structures and infrastructure.

4.1 Main South Road Four Laning

MSRFL comprises two lane widening on the north western side of the current Main South Road (SH1) including an over bridge structure for Weedons Road and an underpass structure for Robinsons Road, where it ties in with CSM2. The main emphasis of the investigations was for the construction of new pavement and at grade road infrastructure, with less emphasis on the need to investigate deep strata. The pavement surface will be close to existing surface levels which will necessitate cuts to accommodate the pavement structure, side water table drains and soakage ponds.

4.2 Main at Grade Carriageway (CSM2)

The construction of the CSM2 section consists of a new four lane motorway at grade between Robinsons Road and Halswell Junction Road, where it ties in with Stage One of the Christchurch Southern Motorway. The route is across 'greenfield' sites with the exception of the intersections with existing local roads and tie-ins. The emphasis of the investigation in this area was to identify ground conditions at grade for general road construction but also to undertake deeper investigation for the construction of bridge and embankment structures. The bridge structures will require spread or piled foundations.

4.3 Intersections and Structures

The Intersections and associated bridge structures will be built at grade and the difference in levels will be accommodated by embankments on the roads crossing the motorway and four laning sections. The bridge structures will require spread or piled foundations. The structures are described as:

- Overpass at Robinsons Road with an associated interchange at the CSM2/MSRFL junction.
- Underpass at Waterholes Road; and
- Underpass at Trents Road; and
- Underpasses at Marshes Road and Shands Road, the latter with an associated interchange;

5. Engineering Geology

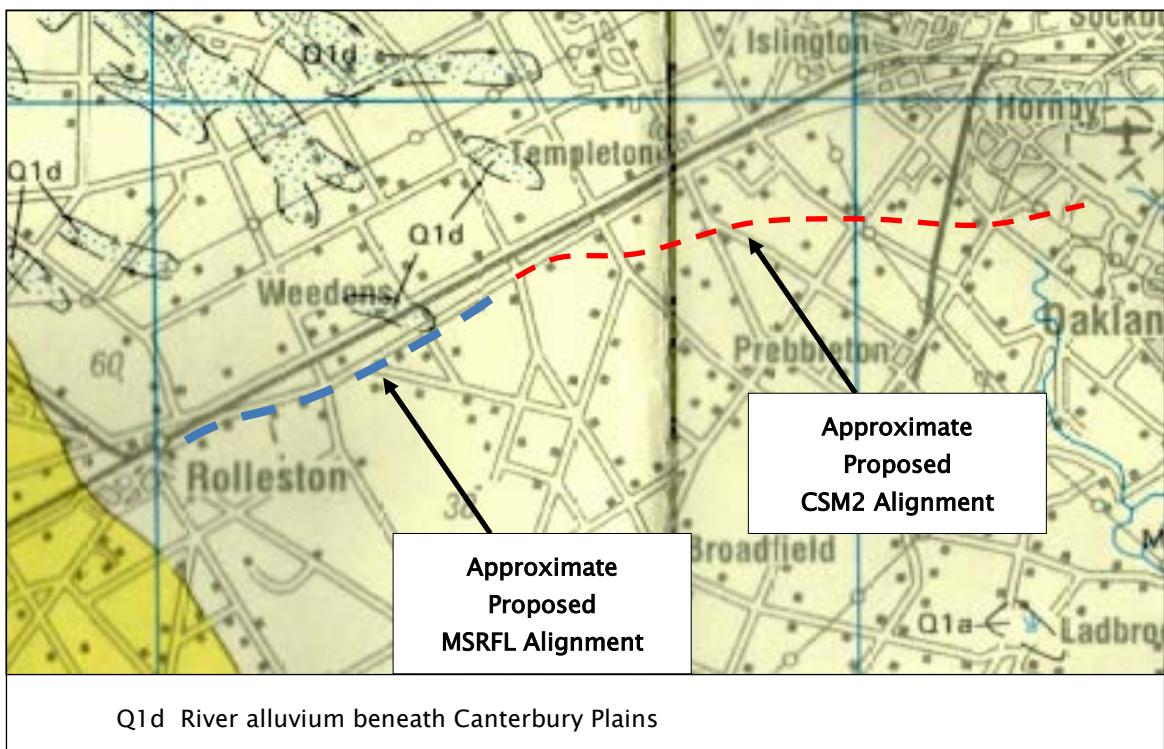
5.1 Topography

The topography of the Project alignment is described as generally flat with no significant undulations or depressions and a gradual increase in elevation towards Rolleston. Topographical relief is measured as 18.8 m above sea level at the CSM1/2 junction and 55.5 m at Rolleston. The MSRFL alignment gently undulates, with the majority of the fall in elevation recorded along the CSM2 alignment.

5.2 The Geological Model

The ground model has been developed during the investigation of the Project, and is based on published geological maps as well as the ground conditions and geology encountered during the ground investigations. An extract from the published geological mapping is presented as Figure 2.

Figure 2 Extract from the Published Geological Mapping (GNS 2008)



The general topography of the Project alignment is characterised by flat alluvial plains (Q1a). The alluvial material has been subdivided by Brown (1992) into alluvial sand and silt of historic river flood channels and underlying alluvial gravel and sand (and silt overbank deposits), both of the

Yaldhurst Member of the Springston Formation. These have been laid by alluvial processes over the past 10,000 years. The Halkett member of the Springston Formation is more coarse in nature and underlies the Yaldhurst Member.

5.2.1 Springston Formation – Yaldhurst Member

This unit underlies the majority of the CSM2 alignment. The unit consists of shallow low plasticity silts and clays, intermixed with fine sands. These soils are typically overlain by 0.1 – 0.3 m of topsoil and generally extend to a depth of between 0.1 to 2.2 m below ground level, although these soils were consistently encountered to depths of 1.5 to 3.5 m between the Project chainage 4500 m to 6600 m.

The silts have variable clay content, being defined as medium to low plasticity silt based upon Atterberg Limit tests. As they generally behave as fine to medium grained soils they have been described as predominantly non cohesive with a loose density based upon typical dynamic cone penetrometer values (DCP) of between 1 – 4 blows per 100 mm and drillhole SPT N60 values of $N = 5$. Where more cohesive material was encountered, it was described as having soft to firm consistency. The delineation between the lower boundary of this unit and underlying sandy gravels is clearly defined.

5.2.2 Springston Formation – Halkett Member

Sandy gravel and sandy silty gravel of the Halkett Member underlie the whole site area, either from surface or below the shallow Yaldhurst Member where encountered. The Halkett Member was encountered at depths from between 0.1 – 2.2 m below ground level to the full depth of the investigation holes at 21.5 m below ground level. Typical SPT N60 values for this unit were between $N = 25$ to $N = 50+$.

5.2.3 Adopted Soil Profiles

With the geological ground model developed, a simplified soil profile has been adopted for the purposes of developing geotechnical parameters and design philosophies. These soil profiles vary according to the specific section of the alignment and are derived for the exploratory hole suite located in that section. The profiles adopted are generally described as

- Top Soil
- Sandy Silt
- Sandy Gravel
- Silty Sandy Gravel

5.3 Active and Known Faulting

5.3.1 Pre September 2010

Before recent earthquake events commencing in September 2010, most ground shaking events were related to active faults situated in west and north Canterbury with few known faults within the Project area³.

Active faults (defined as those on which there is evidence of surface displacement over the last 50,000 years and have the potential to rupture in the future) have been mapped along the foothills of the Alps (e.g. Porters Pass Fault) and in the north of the region in Hanmer (Hope Fault) and in the area north of Motunau. The Ashley Fault, situated north and parallel with the Ashley River, was the closest active fault known in the region, some 30km north of the Project site.

Figure 3 below (taken from the Institute of Geological and Nuclear Sciences (GNS)) illustrates the distribution of active faulting in the north of the South Island, with a further summary of the active faults and their distances to the Project presented in Table 2 below.

³ For the purposes of this report "Project area" includes the rural area around the Project and the urban areas of Rolleston, Templeton, Prebbleton and Hornby.

Figure 3 Active Faults of the Northern South Island (after GNS)

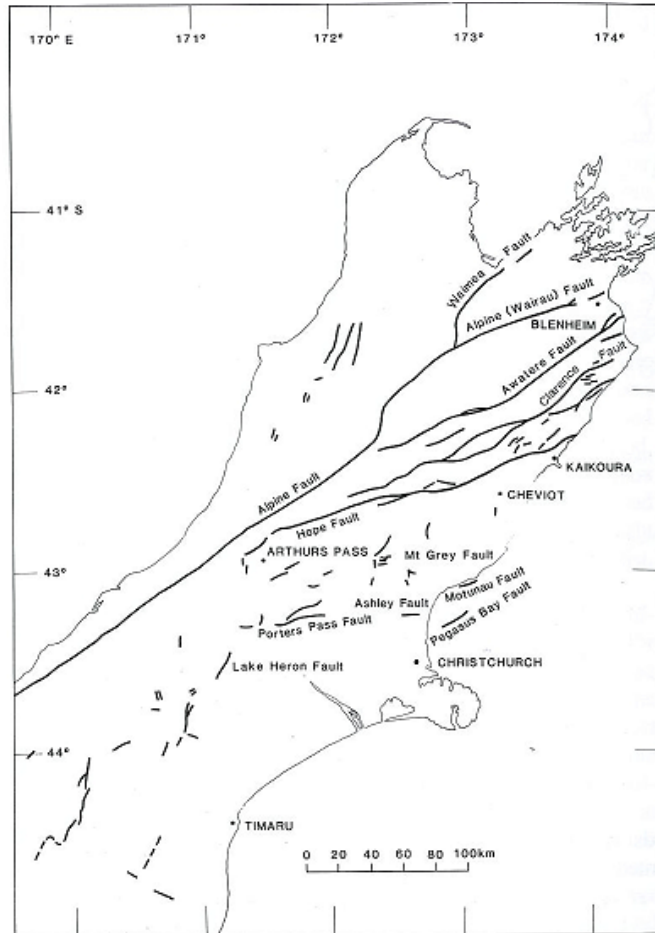


Table 2 Known Active Faults in the North of the South Island

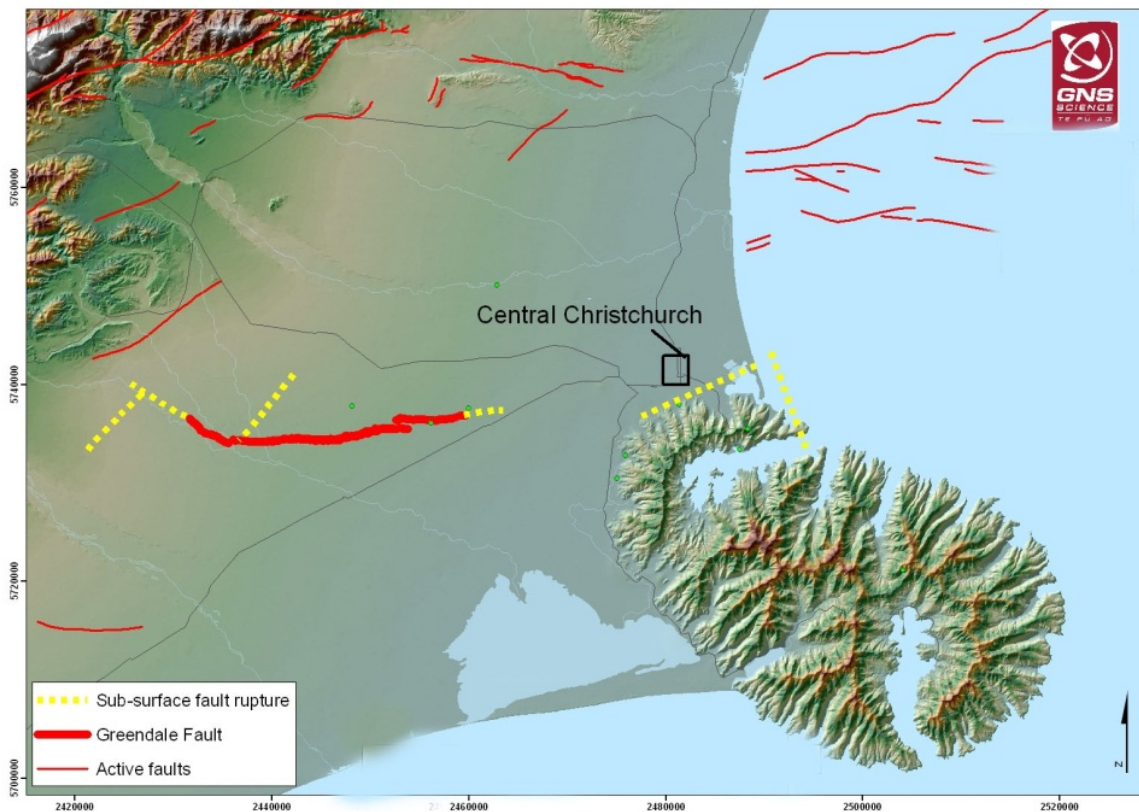
Known Active Fault	Distance from Site (km)	Max Likely Magnitude	Avg Recurrence Interval
Alpine Fault	144	8.3	~300 years
Greendale (2010) Fault	28	7.1	~15,000 years
Ashley Fault	30	7.0	~3,500 years
Hope Fault	121	7.2~7.5	120~200 years
Kelly Fault	127	7.2	~150 years
Porters Pass Fault	101	7.0	~1100 years

Christchurch will continue to experience effects from earthquakes centred on these and other South Island faults, which is further discussed in Section 6.

5.3.2 Current GNS Mapping

Following the rupture of the Greendale Fault in September 2010, GNS has subsequently published new data identifying active faults within the proximity of Christchurch. Current mapping is presented as Figure 4.

Figure 4 Current GNS Mapping of the Greendale Fault



5.3.3 Extension of Greendale Fault

The Greendale Fault has been mapped with its eastern end terminating approximately 1 km north of Rolleston. Propagation and extension of this fault eastwards would result in the active fault crossing the Project area⁴ east of Weedons and trending towards Prebleton. However, recently generated data and interpretation from GNS is indicating that seismic activity is moving eastwards and north away from the Project area, becoming centred offshore in near shore Pegasus Bay.

⁴ For the purposes of this report “Project area” includes the rural area around the Project and the urban areas of Rolleston, Templeton, Prebleton and Hornby.

5.4 Settlement

5.4.1 Static Settlement

Static settlement is due to the consolidation of the foundation soil in response to loading and dissipation of pore pressures. Settlement of a structural foundation comprises three parts, namely 'immediate', 'consolidation' and 'secondary'. The final settlement is the combination of all three components, with secondary settlement the long term, time dependant settlement, associated with soil creep.

5.4.2 Immediate Settlement

The materials on site generally comprise cohesive silts and granular sands and gravels. The latter soils experience relatively small volumes of settlement which is immediate during the application of loading. For design purposes, sands and gravels are anticipated to have static settlement of 0–25 mm. The cohesive silt materials, where used as founding strata, will experience both immediate and consolidated static settlement of magnitude 25–50 mm.

5.4.3 Potential Settlement issues

Whilst ground conditions do not pose significant risks to construction in terms of static settlement, design should be cognisant of settlement of embankments during and post construction and differential settlement beneath any structure due to lateral variations in strata at founding level or differential structural loading on a footing. Seismic settlement may also occur as a consequence of a seismic event.

5.5 Slope Stability

5.5.1 General Topography

Because of the flat topography it is considered that natural slope stability is not a risk to the Project and nor will the Project have any adverse effects in this regard. For this reason, no further assessment of effects in relation to land topography is included in this report.

5.5.2 Cuttings and Embankments

Slope stability of any required cutting and or embankment is essentially mitigated through appropriate design and application of geotechnical parameters. The parameters chosen are dependent on the materials selected for embankments and mechanically stabilised earth (MSE) walls, which will be granular in nature with a relatively high angle of friction. At this Investigation and Reporting stage, it is anticipated that embankments will comprise imported river run gravels. However, detailed design stage may accommodate alternative materials or sources, if deemed appropriate and/ or economical.

There are few cut slope requirements along the route. Slope stability of cuttings can again be accommodated in the design for cut slopes. Cut materials are likely to be within the silty gravels with parameters that should allow 2:1 batter angles.

5.6 Regional Groundwater Model

The groundwater of the Quaternary alluvial gravels of the Canterbury Plains typically extend within shallow (<20 m depth) unconfined aquifers, with hydraulic connection with any nearby surface water courses.

Groundwater yields tend to vary laterally over short distances indicating that more permeable gravel horizons heavily influence the groundwater flow. Groundwater movement below the plains and the Project site is generally downward and towards the coast.

Shallow groundwater levels vary seasonally and respond to winter recharge and summer irrigation use. Connectivity of surface waters with shallow groundwater increases the vulnerability of groundwater to contamination. However, with reference to Section 6.4 of this report and to the Contaminated Land Assessment (Technical Report 16), there is no contamination issue within the designated route and as such, groundwater is not seen as susceptible to impacts from in situ contaminated ground.

6. Geological Hazards

6.1 Geological Hazards along the Route

In general terms it is concluded that limited geological hazards are presented along the alignment. The significant hazard as evidenced by recent earthquakes is seismicity and liquefaction. These are specifically discussed in the following section.

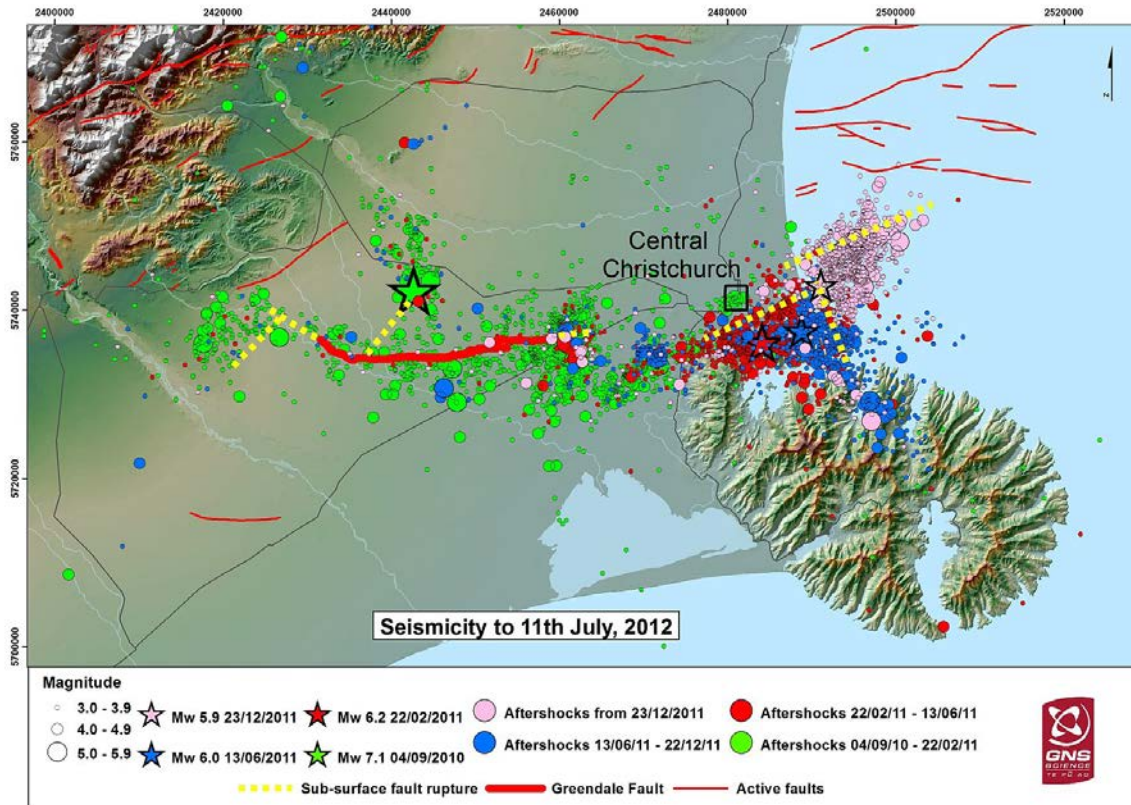
6.2 Seismicity

6.2.1 Regional Setting

The route lies within the Canterbury Region south of the Marlborough Fault Zone and to the east of the Alpine Fault. The earthquakes that have occurred since September 2010 lie with a localised region centred around Christchurch and to the west of the city. The significant M5 earthquake events (and greater) have migrated east with time, with the latter events occurring off the New Brighton coast. The smaller aftershocks have likewise shown a general eastward migration but can occur anywhere in the region. This migration of aftershocks to the east is clearly illustrated in GNS's annotated map showing seismicity to 11 July 2012. This map has been reproduced below and presented as Figure 5.

The highest current risk is from a significant aftershock from the current sequence, but both the Marlborough Fault zone and the Alpine Fault are capable of generating large earthquake events which could result in significant ground shaking in the Christchurch area (see Figure 3 above).

Figure 5 Seismicity in the Canterbury Region to 11 July 2012 (after GNS)



6.2.2 Ground Deformation and Fault Rupture

The September 2010 Darfield Earthquake was centred on the previously unmapped Greendale Fault located to the west of the route as indicated in Figure 4 above. The movement on the fault that generated the earthquake was accompanied by extensive ground rupture. Subsequent mapping has disclosed subsurface rupture, one extension of which approaches the route immediately north east of Rolleston.

The following earthquakes of 26 December 2010, 22 February 2011, 13 June 2011 and 23 December 2011 have not been accompanied by surface rupture, but significant ground deformation from either tectonic movement or earthquake induced settlement has been disclosed by surveys undertaken by GNS and generally reported in technical presentations and journals. Settlements of up to 1.2 metres have been noted outside the Project area in the eastern suburbs of Christchurch.

In the Project area⁵ significant ground movement occurred following the September 2010 event. Horizontal displacements of up to 900 mm towards the west occurred on Main South Road near Rolleston with horizontal displacements reducing further east e.g. 300 mm at Berketts Road. To the

⁵ For the purposes of this report "Project area" includes the rural area around the Project and the urban areas of Rolleston, Templeton, Prebbleton and Hornby

east of Berketts Road, the vector of displacement changes direction to the north east with displacements of 190 to 230 mm being measured.

The vertical displacements measured are all negative (i.e. movements downwards with respect to previous levels). The maximum movements noted were 230 mm west of Weedons Road and 320 mm near the Larcombs Road intersection. Smaller negative displacements of 20 mm to 40 mm were noted around the Main South Road/Motorway intersection.

The subsequent aftershocks have not generated tectonic movements in the Project area.

6.2.3 Ground Shaking

Unusually high levels of ground shaking were noted in the 22 February 2011 and 13 June 2011 earthquakes with maximum peak ground accelerations (PGA) of 2.2 g recorded in the Heathcote Valley and 1.6 g in parts of the Eastern suburbs. In the CBD of Christchurch, PGAs of up to 0.6g were recorded. These values are significantly higher than Building Code requirements. The high accelerations recorded were a consequence of the following factors:

- The close proximity of the earthquakes to the city
- The direction that the energy was released
- The interaction of the subsurface strata under the city
- The high energy released by the fault

In the Project area, PGA values were in the order of 0.2 g arising from both the 4 September and 22 February events.

The interaction of the subsurface strata is significant with respect to the Project area. In the Heathcote Valley and Eastern Suburb's soft alluvial sediments overlying harder basement rock (the subsurface extension of the Lyttelton Volcano) has resulted in the amplification of the arriving earthquake waves i.e. they refract and "bunch up". The subsurface conditions underlying the Project area are somewhat different, with stiff soils to some considerable depth. There is unlikely to be any amplification effects.

6.2.4 Earthquake Induced Landslides

The topography of the Project area is of low relief and land sliding induced by earthquakes and seismic activity will not be a feature.

6.3 Liquefaction

6.3.1 Previous Hazard Mapping

Environment Canterbury had previously (to 2010) carried out liquefaction susceptibility studies in Christchurch to identify area of particular risk. The Project area was identified as having a low risk of liquefaction. For a soil to liquefy three concurrent conditions need to be met.

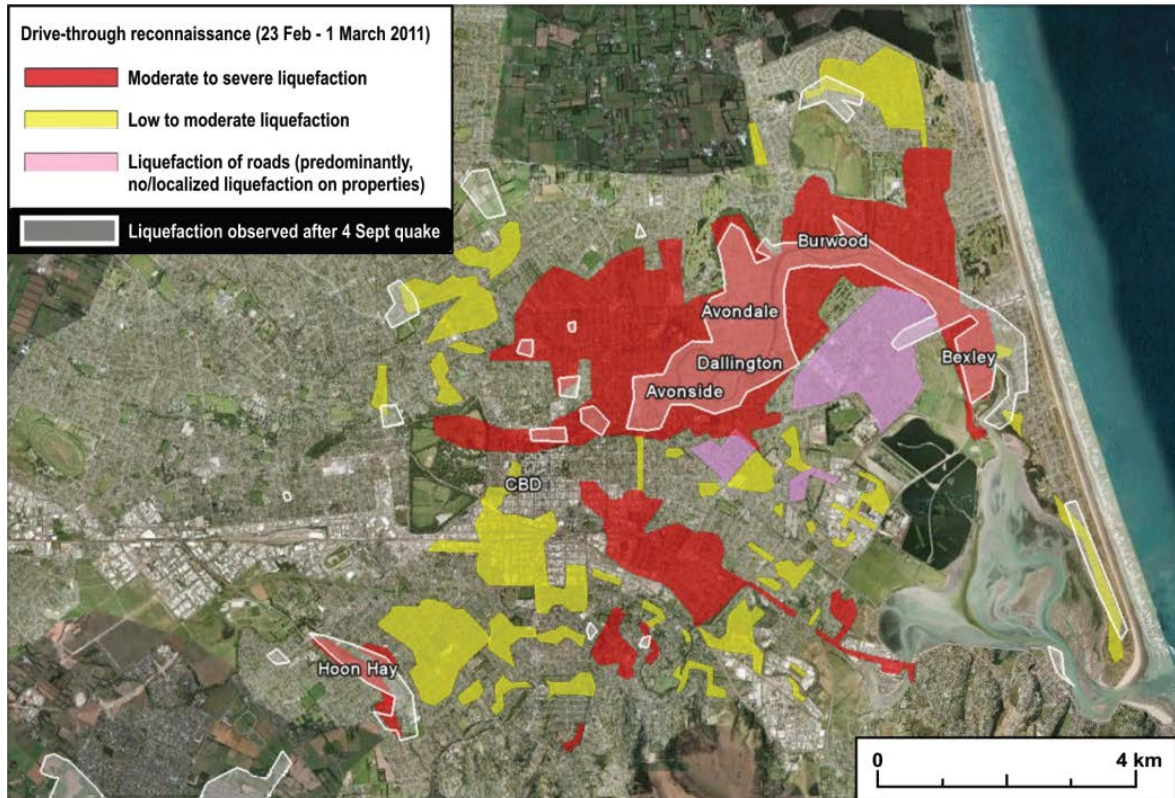
- An earthquake source developing sufficient ground acceleration to trigger liquefaction
- A high groundwater level sufficient to saturate near surface soil
- Soils with a susceptibility to liquefaction – usually fine sands or coarse silts which are uniformly graded and of low to medium density.

The earthquake sequence starting with the Darfield earthquake of 4 September 2010 provided the first of these conditions.

6.3.2 Actual and Observed Events

The earthquakes of 4 September 2010, 22 February 2011, 13 June 2011 and 23 December 2011 generated liquefaction in the Christchurch area. By far the most extensive liquefaction was generated in the 22 February event. Figure 6 illustrates the liquefaction that occurred following this event.

Figure 6 Liquefaction Extent Post 22 February (From Kaiser et al, 2012)



Areas of observed liquefaction in Christchurch due to the 22 February M_w 6.2 earthquake (coloured areas) and the M_w 7.1 Darfield main shock (white contours) based on drive-through reconnaissance and surface manifestation of liquefaction visible on aerial photographs (Cubrinovski & Taylor, 2011).

The effects included extensive “sand boils”, discharge of groundwater, lateral spreading of liquefied soils and associated cracking of overlying soils and settlement of ground and structures founded on surficial soils. Little or no liquefaction was observed in the Project area. This was due to:

- Lower Peak Ground Accelerations (PGA) in the Project area
- Lower ground water levels (4–5 metres below surface)
- Dominant soils e.g. gravels, which are not particularly susceptible to liquefaction.

6.3.3 Susceptibility of Site Soils to Liquefaction

Based on the site investigation data obtained and the observed effects and data from the recent earthquakes, the susceptibility of the soils to liquefaction is low and limited to particular horizons of more silt and fine sand rich material.

The major effects from liquefaction, if it were to occur on the alignment, would be concentrated on the structures. It is unlikely given the encountered soils of sands and gravels that liquefaction

would disrupt either the pavement or buried services such as occurred in the eastern suburbs of Christchurch.

Table 3 summarises the analysis undertaken to determine the liquefaction potential at the intersections along the route.

Table 3 Liquefaction Analyses Summary

Intersection	Liquefaction potential under ULS*	Horizon	Influence
Weedons Road	Yes	15 –19 m	Minor #
CSM2/MSRFL	None	-	-
Waterholes/Hampton	Yes	17 m	Minor #
Trents/Blakes	Yes	14–20 m	Minor #
Marshs Road	Yes	9 –19 m	Minor #
Shands Road	Yes	10 –16 m	Minor #
Springs Road	Yes	15 –19 m	Minor #
Halswell Junction Road	None ^^	-	-

* Ultimate Limit State

Settlements occurring at depths greater than 12 m below ground surface will have limited influence on the behaviour of the structure, assuming appropriately designed shallow foundations are adopted.

^^ The liquefaction assessment at Halswell Junction Road is based on one borehole (AF20), the SPTs in which do not indicate there is liquefaction potential. As the design develops it will be necessary to carry out further investigation to confirm this.

Based on this analysis, it is concluded that there is little risk from liquefaction to major structures on the alignment that prudent design would not mitigate. However, if detailed design requires deep foundations, then the effect of liquefaction may be realised and more detailed investigation will be required at the detailed design phase to ascertain if liquefiable soils are present beneath the structure.

6.4 Contaminated Ground

The assessment of soil and ground contamination has been subject to specific investigations and is reported under separate cover, in the Contaminated Land Assessment (Technical Report 16).

There are no contamination issues or any need for remediation within the proposed designation.

7. Groundwater

7.1 Current Available Data

Groundwater levels have been investigated along the Project alignment to establish a design groundwater level, and to establish if any issues will arise with drainage swales and an underpass being set below existing ground level. Various sources of information have been used to determine the design level. More detailed analysis is covered in the Assessment of Stormwater Disposal and Water Quality (Technical Report 3).

7.1.1 Environment Canterbury Monitoring

Six existing ECan boreholes (refer to Figure 5 of the Assessment of Stormwater Disposal and Water Quality (Technical Report 3)) have been identified near the alignment with monitoring records of varying frequency and record length, the oldest dating back to the 1950s. These records have helped establish a baseline groundwater regime and identify seasonal and long term fluctuations. These logs indicate the groundwater is typically 3 to 5 m below ground level close to Christchurch increasing to 12 to 20 m below ground level near to Rolleston.

7.1.2 CSM1 Investigation Data and Monitoring

Groundwater data from the 'Mushroom Pond', located to the north of the east bound Halswell Junction Road on ramp, from the CSM1 investigations has been used for conservative design parameters for CSM2 detention basins. No other CSM1 data was applicable to CSM2/MSRFL.

7.1.3 Groundwater Studies for Central Plains Water

Evidence⁶ provided by Central Plains Water (CPW) at the hearing considering the irrigation effects of its scheme, indicates that there will be mounding of the groundwater. This is predicted to be in the order of 4 m towards the Rolleston end and 1 m towards Christchurch. An allowance has been made for the CPW effects of the operation of the CPW irrigation scheme in development of the design ground water level.

⁶ Weir, October 2009, Supplementary Evidence of Julian James Weir in the matter of applications by Central Plains Water Trust to: Canterbury Regional Council for resource consents to take and use water from the Waimakariri and Rakaia Rivers and for all associated consents required for the construction and operation of the Central Plains Water Enhancement Scheme Selwyn District Council for resource consents to construct and operate the Central Plains Water Enhancement Scheme.

7.2 Groundwater Investigations

7.2.1 Borehole Construction Observations

Groundwater strikes were recorded in all boreholes during their construction. Whilst this gives an initial indication of groundwater levels, the mechanics of drilling, disturbing the ground and the introduction of drilling mediums, such as compressed air, will influence groundwater levels. To this extent, the water strike levels facilitate the piezometer design and provide drilling information only, with more emphasis placed on results of longer term groundwater monitoring of piezometers.

7.2.2 Piezometer Installations

Ten piezometers were installed during Phase 1 Investigation cable tool boreholes CT01 to CT10. The boreholes CT01 to CT03 are on the MSRFL alignment, the remainder distributed along CSM2. One groundwater monitoring well was installed during the Phase 2 investigation, located at AF27-2011. Each piezometer was installed with a data logger to allow regular high frequency monitoring. Groundwater monitoring wells CT03, CT04, CT05, and CT06 were damaged by the Christchurch Earthquake, re-drilled in July 20, and reactivated in August 2011. However, piezometers in CT05 and CT06 are still no longer serviceable due to damage to the wires.

7.2.3 Auger Holes and Falling Head Tests

Eight falling head tests were performed during Phase 1 investigations in auger holes AH01 - AH08 along the northern verge of SH1 Main South Road. Falling head permeability rates were determined using the calculations outlined in Section 4, BS 5930:1999. The falling head tests were scheduled to provide permeability values for drainage design purposes adjacent to the MSRFL alignment.

7.2.4 Constant Head Tests

Constant head testing was carried out along the CSM2 alignment during Phase 2 investigations in purpose installed standpipes, constructed of a 2.5 m deep piezometer with 0.5 m slotted PCV pipe installed into filter sands at the base of the stand pipe. Flow rates and constant head measurements were undertaken by McMillans Drilling. Constant head permeability rates were determined using the calculations outlined in Section 4, BS 5930:1999. The constant head tests were scheduled to provide permeability values for drainage design purposes.

7.2.5 Soakaways

Seven soakaway tests were carried out in test pits along the CSM2 alignment and in strategic locations on Main South Road. Soakaway tests were carried out in accordance with BRE Digest 365 (1991) method for soakaway design. Soakaways were undertaken to provide an alternative method for falling and constant head tests in determining permeability and percolation rates for drainage design.

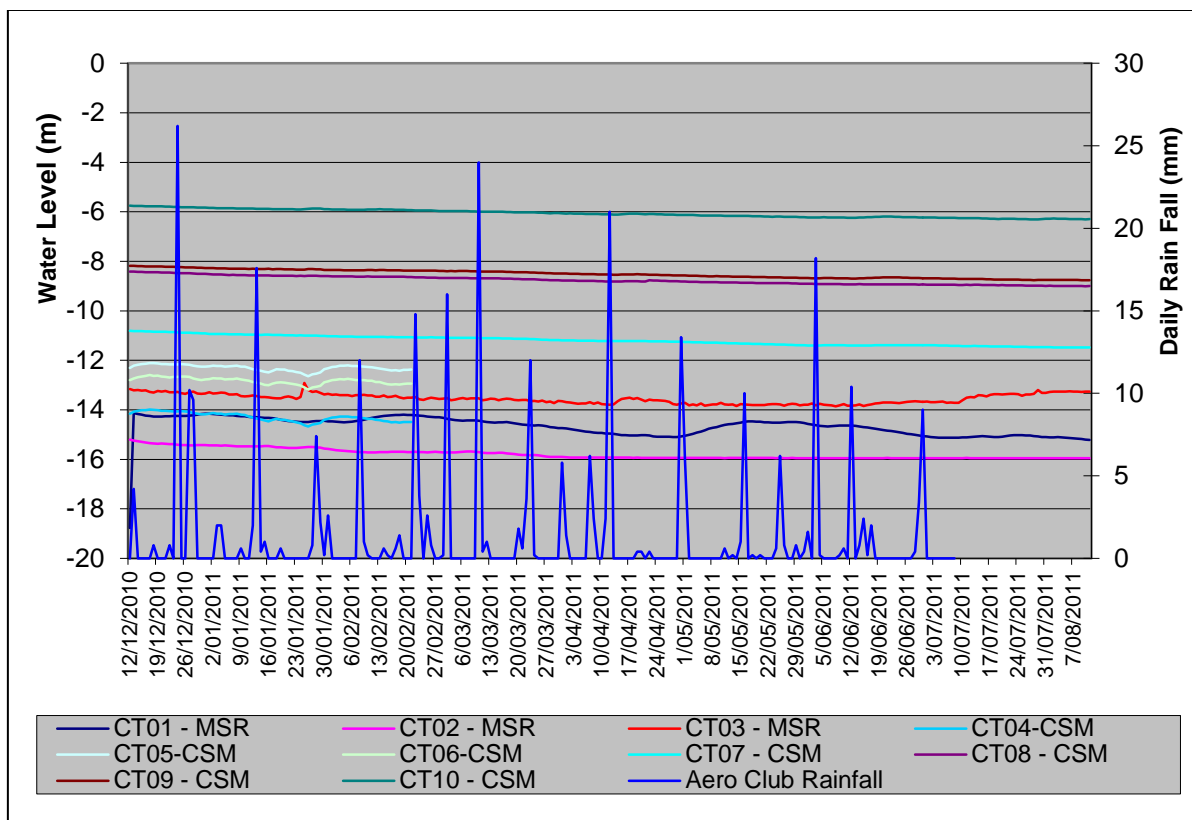
7.2.6 Groundwater Monitoring

Groundwater was monitored using data loggers recording levels at 6 hourly intervals from 7 January 2011 and is still capturing data at the time of reporting. Manual dipping was also undertaken as a calibration exercise and as a baseline.

Groundwater was encountered in the boreholes at depths ranging from approximately 13.0 m to 16.0 m below ground level on the MSRFL alignment. Typical ground water levels fluctuated between 0.7 m and 1.9 m during the period of monitoring. CSM2 boreholes recorded groundwater levels ranging from 5.8 m to 13.9 m.

Figure 7 presents the groundwater monitoring data with rainfall data overlain for the period December 2010 to August 2011. The graph illustrates there is very little variation in groundwater levels of any significance.

Figure 7 Groundwater Monitoring Results for All Piezometers



7.2.7 Earthquake Effects

Groundwater monitoring wells CT03, CT04, CT05, and CT06 were damaged by the Christchurch Earthquake, and re-drilled in July 2011. Monitoring at these sites was periodically undertaken until monitoring devices were reinstalled and reactivated in August 2011, although as previously stated, CT05 and CT06 remain unserviceable. Groundwater monitoring has indicated the recent seismic

events have not adversely affected the regional groundwater regime, with little evidence of any shift or fluctuation in levels. General observations of elevated groundwater levels post February and June 2011 events has not manifested itself in the monitoring records for the installations for this Project.

7.3 Groundwater Regime⁷

Groundwater depth is in the order of 12 to 15 m below ground level towards Rolleston becoming gradually shallower with proximity to the city, measuring 6 to 7 m below ground level at the Springs Road junction. The historical highs have been recorded at 2 to 3 m below ground level. The gradient is generally towards the south east, reflecting the same gradient as ground levels across the same area. No artesian water was encountered during the investigations. The long term effect of the seismic activity in the region remains an unknown and can only be reconciled with specific long term monitoring and investigation.

7.4 Impact to Construction

7.4.1 Influence on Construction and Form

In general terms the groundwater is at sufficient depth not to impact on construction methodology or require techniques in order to accommodate an elevated groundwater. To this extent, it is not envisaged that large scale dewatering systems will be required, although some localised dewatering during deeper trenching or excavations may be needed on a site by site basis.

Localised groundwater peaks around the CSM1 connection during wet weather events or during the wet season may influence the laying of foundations for the road and disposal field for the local stormwater ponds.

7.4.2 Impact to Groundwater

The impact to groundwater from construction is not considered significant for the following reasons:

- Run-off and sediment control during construction will be appropriately managed and maintained within the temporary controlling infrastructure in accordance with the Erosion and Sediment Control Plan.
- The construction form is not effecting nor changing the hydraulic properties of the underlying materials.
- The most compressible materials are situated above the groundwater level.

⁷ Current groundwater regime at the time of reporting

Consequently, the impact to groundwater during the construction period is considered less than minor. However, there is the possibility that the NZTA will adopt a number of wells in the area and draw water from these wells for use in construction. At this point of the investigation, there is no data or testing in terms of the suitability of the wells to perform at efficient rates, or the effect any drawdown may have on the groundwater levels. Whilst this presents itself as a risk to the NZTA, it can be managed at the detailed design phase and alternative sources of water can be used.

Post construction, the run-off and mobile sediments from the pavement areas will be directed to the stormwater infrastructure. However, the residual risk remains, as with any road construction, that the down gradient users may be impacted should the first flush infrastructure fail to sufficiently treat the stormwater run-off before infiltration to ground. The risk is considered minor and is common to all projects of this nature.

7.4.3 Stormwater

It is anticipated that the stormwater ponds at the CSM1 junction are likely to be inundated during extreme groundwater events with the inclusion of the effects of the CPW irrigation scheme. The design groundwater level is at sufficient depth to allow the construction of swales and other stormwater infrastructure along the remainder of the alignment to function appropriately based on the disposal to ground philosophy. This is discussed further in the Assessment of Stormwater Disposal and Water Quality (Technical Report 3).

8. Geotechnical Parameters

8.1 Soil Classification

The results of the soil classification tests are shown in the Casagrande charts for MSRFL and CSM2 (Figure 8 and Figure 9 respectively). A Casagrande chart shows the soil distribution relative to the A line (which depicts the boundary between silts, plotting below the A-line and clays above the line). The figures below show for CSM2 and MSRFL that the soils are low plasticity inorganic silts and clays.

Figure 8 MSRFL Classification of Soils

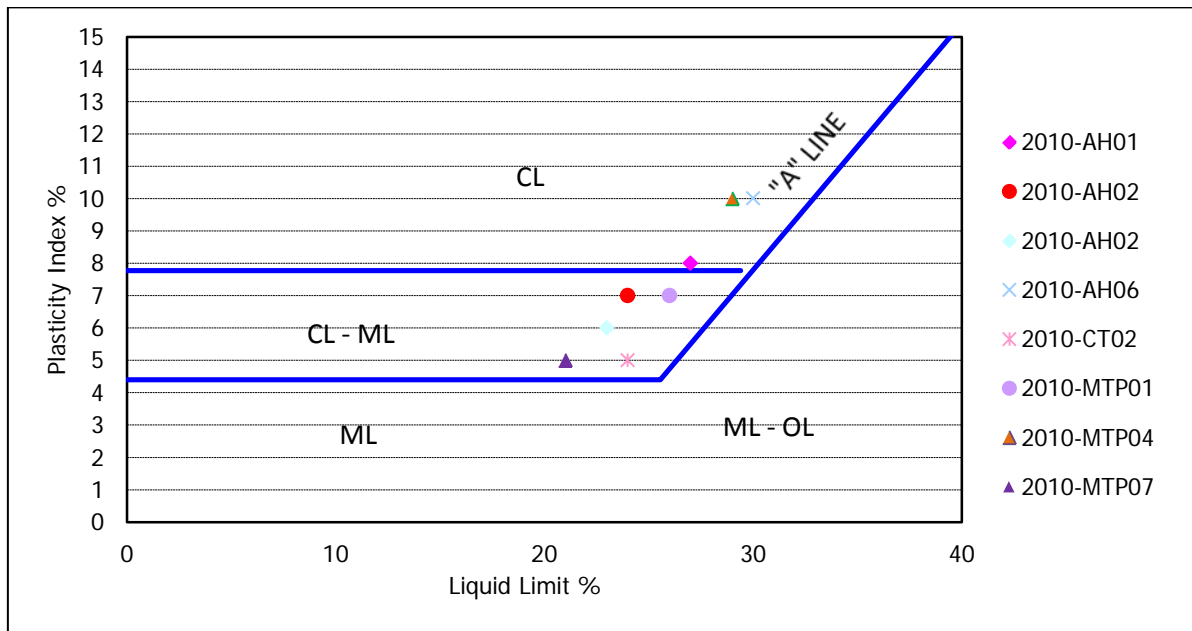
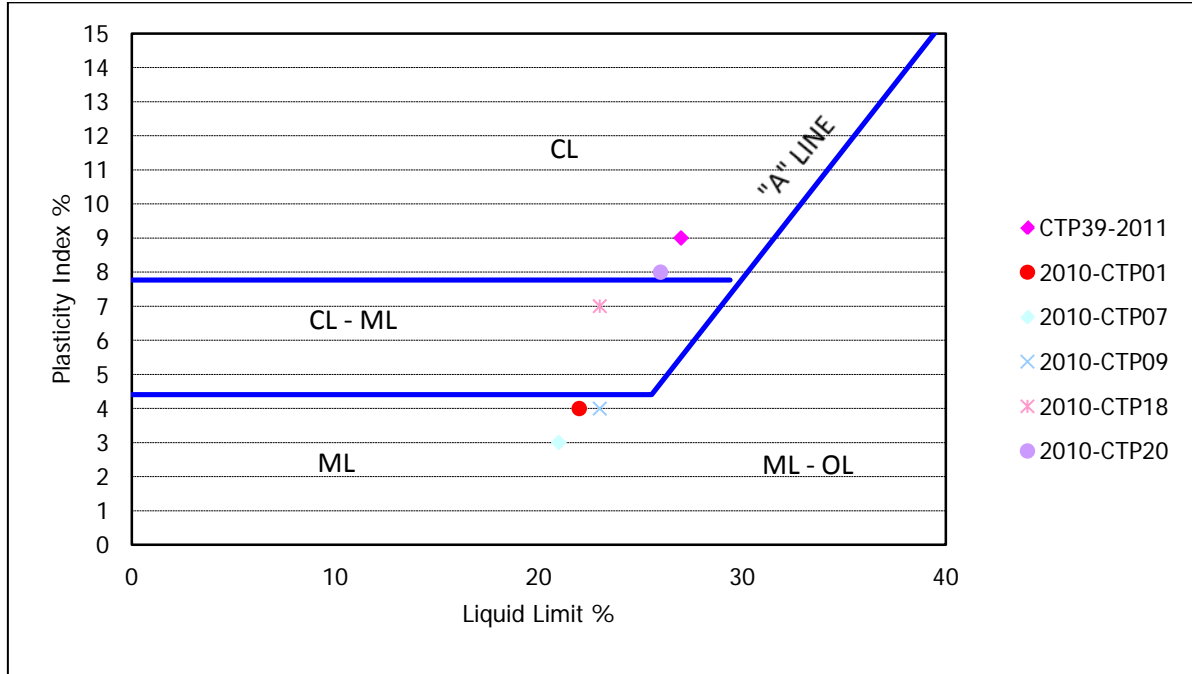


Figure 9 CSM2 Classification of Soils

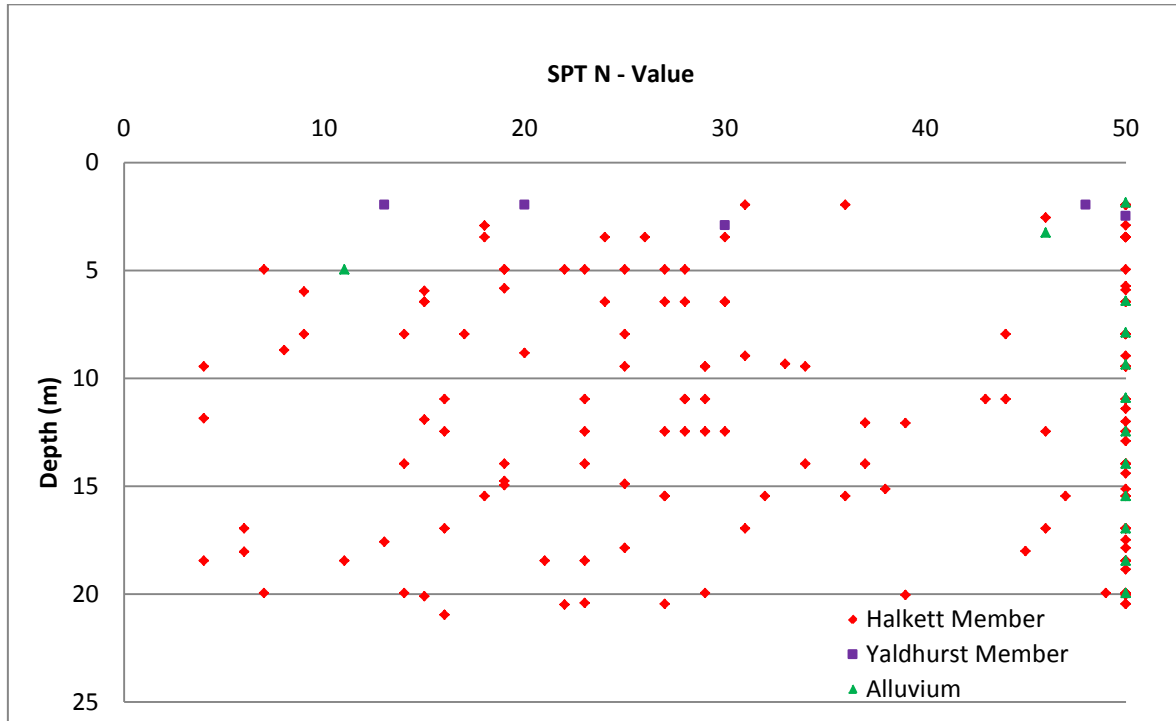


8.2 Soil Strength

8.2.1 Standard Penetrometer Tests (SPTs)

SPTs were completed during the construction of the boreholes with the resultant N-Values recorded on the borehole logs presented in the Factual reports for MSRFL and CSM2. The results of the SPTs have been plotted with depth in the soil profile across the Yaldhurst, Halkett and Alluvial soils and are presented in Figure 10 below.

Figure 10 SPT N-Value Distribution with Depth in the Profile

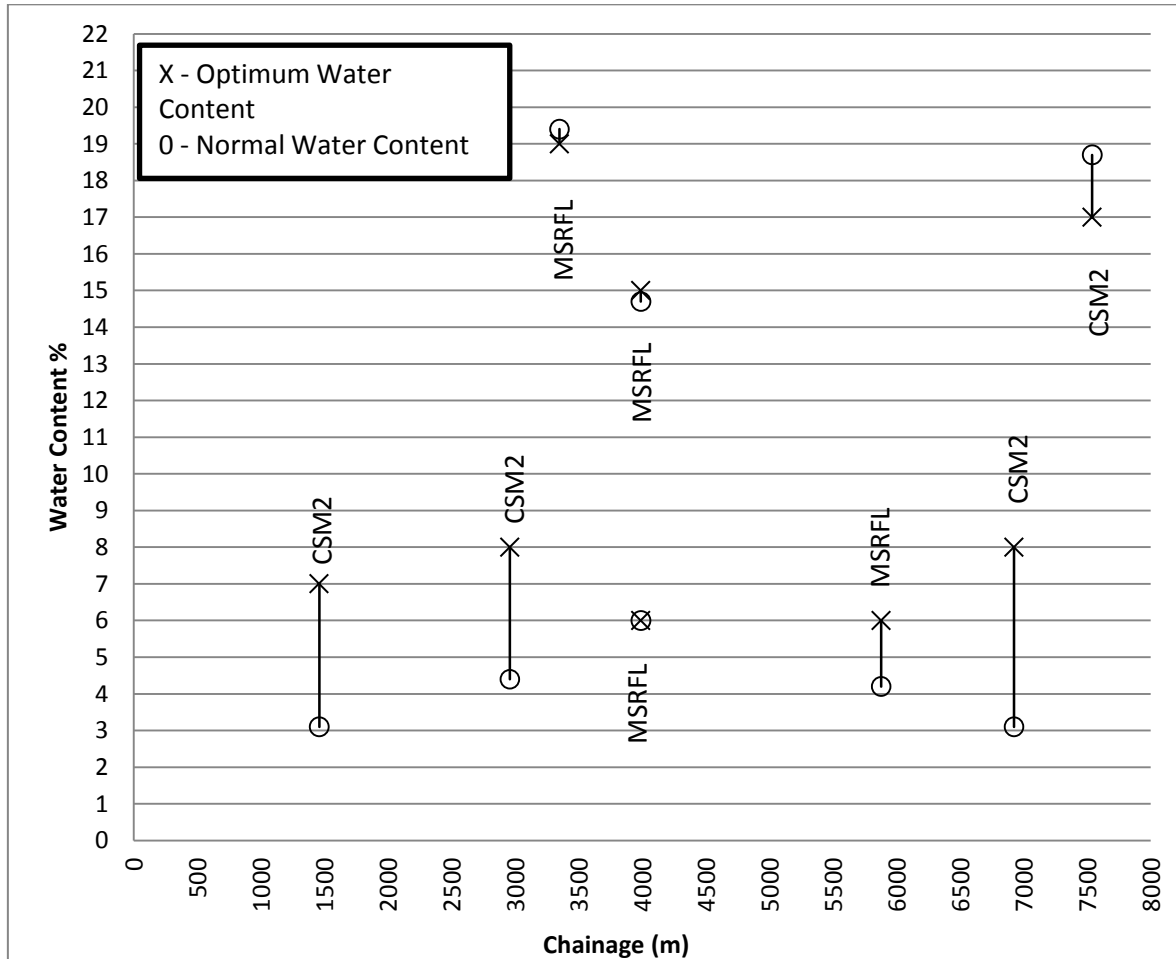


The scattered distribution indicates there is little correlation of SPT with depth, which would be expected for more homogenous soils. The results therefore reflect that strength properties of the Springston Formation do vary with both lateral and vertical extent, which is concordant with the lithological variations recorded.

8.2.2 Compaction

Compaction tests were carried out to the New Zealand Standards (1986). Figure 11 shows the natural moisture content and the optimum moisture contents for each sample for the MSRFL and CSM2 sections. The results indicate that the majority of samples tested are 'dry' of optimum moisture content for CSM2 and near optimum moisture content for MSRFL. The compaction tests indicate a range of dry densities from 1.73 t/m³ to 2.29 t/m³.

Figure 11 Soil Compaction Test Results



8.2.3 CBRs

Soaked CBR tests were carried out on bulk samples excavated from test pits for MSRFL and CSM2, the soaked CBR test produced CBR values ranging from 3.5 to 125%. For design purposes, a CBR value of 4% has been selected for CSM2 and a value of 3.7% for MSRFL.

9. Geotechnical Design Philosophy

9.1 General

At this concept design stage, geotechnical design has taken into account the need to provide stable embankment, foundations and bridge structures in the event of seismic activity, whilst providing cost effective solutions to construction form and foundations. A design life of 100 years has been adopted.

9.2 Seismic Criteria

Reference has been made to the New Zealand Standard NZS 1170.5:2004 Structural Design Actions – Part 5 Earthquake Actions in order to determine the Seismic Importance Level for any structure being considered. In addition, Table 3.5 of this standard has been used to determine the Annual Probability of Exceedance in terms of Ultimate Limit State (ULS) and Serviceability Limit State (SLS).

As required in verification method B1/VM1 of the New Zealand Building Code, in the Canterbury Region, the risk factor (R) for the Serviceability Limit State (SLS) has been taken as 0.33 and the Hazard factors Z for Ultimate Limit State as 0.3.

For structures assessed for CSM2 and MSRFL, the seismic criteria is summarised in Table 4 below:

Table 4 Seismic Criteria for Design

Importance Level (NZS 1170)	Annual Probability of Exceedance ¹		PGA (as proportion of g)	
	ULS ²	SLS ³	ULS ³	SLS ³
2	1/1000	1/50	0.44	0.11
3	1/2500	1/50	0.6	0.11

1 NZS 1170.5:2004, Table 3.5

2 ULS – Ultimate Limit State

3 SLS – Serviceability Limit State

Liquefaction assessments have been undertaken using the Idriss and Boulanger (2006) method, considering a Magnitude 7.5 earthquake, and the appropriate ULS and SLS PGAs depending on the Seismic Importance level of the structures.

9.3 Current Review of Structural and Building Codes

GNS are currently investigating and reporting on the change in seismic setting for the Canterbury Region, which will have a bearing on structural codes adopted for the engineering industry. Therefore any structural elements have to be subject to the confirmed peak ground accelerations published by GNS.

There is current concern and dialogue within the engineering community that the potential for liquefaction exists within the soils of Christchurch and the surrounds to a depth of 20 m. As a consequence, the depth of investigation should be reviewed at the detailed design stage and be cognisant of the changes in the seismic setting of the region.

9.4 Traditional Strip and Pad Footings

Where traditional strip and pad footings can be utilised for abutments, piers, street furniture and associated ground structures, the bearing capacities can be developed from the undrained shear strength characteristics of the cohesive silt material and drained shear strength parameters for the granular material. Table 5 below summarises the values that can be assumed at this stage of design.

Table 5 Soil Strength Parameters

Assumed Material Type	Cu (kPa)	ϕ' (°)
Embankment Fill	-	34
Sandy SILT	60-120	28
Gravelly SAND	-	33
Sandy Silty GRAVEL	-	34
Sandy GRAVEL	-	36
Sandy GRAVEL (saturated)	-	36

9.5 Seismic Settlement

Seismic settlement (from future events) has been considered for ultimate limit state conditions for all of the main intersections. In most cases, liquefaction settlement is expected to be negligible although some general densification and rearrangement of soil particles is anticipated. However, settlement associated with a seismic event has been calculated for Trents / Blakes, Marshs / Shands and Springs/Halswell junctions as summarised below:

Table 6 Settlement Estimates under Seismic Event

Intersection	Settlement (mm)		Comment
	SLS Event	ULS Event	
Weedons/MSR	-	Negligible	-
Robinsons	-	Negligible	-
CSM2/MSRFL	-	Negligible	-
Waterholes/Hampton	-	Negligible	-
Trents/Blakes	-	70 – 300	100 mm differential settlement also possible across site
Marshs/Shands	-	40 – 400	Within Layer 3 Gravel SAND and the upper part of Layer 4 Sandy GRAVEL; differential settlements between the abutments and piers is expected to be in the order of 300 mm
Springs/Halswell	-	20 – 500	Within Layer 3 Gravel SAND and the upper part of Layer 4 Sandy GRAVEL; differential settlements between the abutments and piers is expected to be in the order of 200 – 300 mm

N.B. The above settlement magnitudes are based on limited data and are consequently estimated in terms of the magnitude of settlement under an ULS event. This would be addressed at detailed design stage and more accurate assessment made with further, targeted site investigation.

9.6 Piled Foundations

Where soil conditions provide insufficient bearing capacity for shallow pad and strip foundations for the larger structural loads, for example around bridge piers and abutments, then small diameter piles are considered appropriate to take the increased loads to adequate founding strata with the following recommendations:

- Steel driven H piles or tube piles shall be driven to a target minimum embedment of 5 pile diameters into the underlying competent dense gravel layer and designed to carry the vertical loads in end bearing;
- The driven piles are to be founded in the upper portion of the layer to reduce the influence of the potentially weaker underlying material. The final proposed arrangement should be confirmed as the design develops;
- For steel driven H piles or tube piles spaced at centres of not less than 4 pile diameters, pile tip settlements in the order of less than 5 mm are expected under SLS loads;

- The individual junctions/intersections have an ultimate end bearing capacity in the range of 10 – 18 MPa which can be adopted and reduced by an appropriate strength reduction factor to derive a dependable capacity;
- The pile capacities discussed above are for a single pile. If pile groups are adopted, pile group effects would need to be considered; and
- The estimated ultimate skin friction available on the piles is likely to be nominal.

As the design develops there are various other considerations / risks for this option, which will need to be assessed, including:

- Variation of layer thickness and strength;
- Vibration effects due to driving on adjacent properties;
- Noise effects due to driving; and
- Settlement of piles due to influence of lower strength underlying materials.

Construction vibration effects associated with piling have been assessed through on-site measurement, as well as the review of data from relevant standards and previous measurements in the Assessment of Construction Noise and Vibration (Technical Report 9). This data has been analysed and processed to establish setback distances for building vibration risk assessments. The results indicate that it is unlikely for there to be any buildings along the CSM2 alignment with a high vibration risk. Predictions of construction vibration levels indicate there is a degree of risk that the Project criteria may be exceeded at dwellings within 20 m of the Main South Road Four-Laning alignment. A detailed assessment of building vibration risk will be conducted once detailed information is known on the construction methods. If piling activities exceed the adopted criteria other pile types may be suitable and could be adopted if appropriate, subject to the detailed design stage, particularly if vibration during the pile construction poses an unacceptable risk or nuisance to adjacent structures or residential properties.

9.7 Slope Stability

Slope stability has been assessed for approach embankments and the single cut slope at the Trents/Blakes intersection. Embankment batters, based on the performance of CSM1 and with the geotechnical parameters determined from these investigations, have a design of 2:1 slope batter. The single cut slope has a batter of 4:1. The slope batter geometry is summarised below in Table 7.

Table 7 Slope Stability Geometries

Intersection	Approach Geometry		Embankment		Cut Slope Geometry	
	Height	Carriageway Width	Slope Batter	Max Height	Slope batter	
CSM2 – MSRFL	~7 m	13 m	2:1	-	-	
Waterholes Rd	~7 m	14 m	2:1	-	-	
Trents Rd & Blakes Rd	~4.5 m	14 m	2:1	~4.5 m	4:1	
Marshs Rd & Shands Rd	~7 m	14 m	2:1	-	-	
Springs Rd & Halswell Junction Rd	7–8 m	22–42 m	2:1	-	-	

For stability of foundations under seismic conditions, abutment foundations placed within embankment fill should be located below a 5H:1V projected line from the toe of the slope or a maximum of 3 m above ground.

10. Earthworks Considerations

10.1 General Cut/Fill Requirements

The proposed vertical alignment of CSM2 and MSRFL are generally at, or close to, grade with only limited amounts of cut and fill. These limited cuts are proposed at various locations along the alignment, including potentially at the intersections with Main South Road and Trents/Blakes Roads and Weedons Road. Elsewhere the cuts are associated with areas of slightly higher ground. Generally the vertical alignment is within + 1 m of existing ground level. Swales are proposed adjacent to the carriageway and these will be cut below the carriageway level.

10.2 Soil Profiles

The boreholes and test pits give subtly different indications of the composition of the soils within two metres of ground level, as follows:-

- The generalised soil profile developed from the boreholes indicates a minimum of 2 m of silt, the exception being at Waterholes Road, where the silt was only approximately 1 m thick with sandy gravel below. It was generally described as being non-plastic to low plasticity and being firm, although on occasion it was described as stiff or very stiff.
- In the test pits the silt is recorded as being between approximately 0.5 and 1.0 m depth to the west, thickening to greater than 2 m in the east. It was generally described as having low plasticity and being firm to very stiff, with a tentative trend of increasing stiffness with increasing thickness.

Based on the mass exposure and relative lack of disturbance, the descriptions of the silt are taken to be more reliable from the test pits than from the air flush or cable tool boreholes.

10.3 Geotechnical Parameters

10.3.1 Sandy Silts

The material that is to be obtained from cutting is expected to consist predominantly of silt, with varying proportions of sand, the exception being to the west, where sandy gravel may be encountered. Laboratory testing of the silt recorded an average Plasticity Index of 6%, with a range of 3 to 10%, confirming its low plasticity. Three compaction tests on the silt gave consistent results, with an optimum moisture content of 15% (range 13 to 16%) compared to a natural moisture content of 15% (range 10 to 19%) and a maximum dry density of 1.79 Mg/m³ (range 1.73 to 1.84 Mg/m³).

Generally the silt is likely to have a high degree of sensitivity, being stiff or very stiff on initial excavation, but being susceptible to softening if it becomes wet. Due to the lower drained strength of the silt and its susceptibility to wetting, it is not recommended that it be used as structural fill beneath foundations or on the external faces of embankments and slopes. It could potentially be utilised within the core of embankments, subject to further assessment.

10.3.2 Sand and Gravels

The laboratory testing of the sand and gravel indicates that this material can be used as engineered hardfill within the works, including beneath foundations and on the external faces of embankments and slopes, subject to detailed design. It has been assumed that the approach embankments for underpasses will be constructed of imported pit run or river run gravel, comprising a well graded sand and gravel, with minimal fines content.

10.4 Cut to Fill

Based on the materials expected to be encountered close to ground level, there appears to be limited benefit in considering a lower vertical alignment to source material which could be used for filling elsewhere. If silt were to be used for structural earthworks it would likely necessitate the need for flatter embankment slopes and, due to its susceptibility to wetting, restrict the periods when earthworks could take place.

If silts are to be used, they could be placed within the core of any landscape bunds that are constructed, as these are not structural in nature. In addition, silts can be used within the core of embankments to overbridges, so long as controlled placement procedures are adhered to.

10.5 Alternative Sources of Fill

The current estimation of required fill is approximately one million cubic metres, of which one third will be won from cut material. The balance will likely comprise clean river run gravels from the various consented gravel pits and approved sources from within the region. However, there are opportunities to use alternative sources of fill, with experience gained from the recent construction of CSM1:

- Waste glass has been used as a core fill to embankments and areas of non-structural fill.
- Crushed concrete from the on-going demolition of earthquake damaged buildings could be used as a sub base material for the construction of new pavement required for the greenfield development of the CSM2 motorway.

The use of both of these materials would be a beneficial use of otherwise waste materials, providing some relief to the landfills of Canterbury.

10.6 Disposal Sites

The cut fill balance is such that little waste material will require disposal. However, where necessary, waste materials may have to be taken to landfill where other suitable uses cannot be found within the construction process.

Temporary stockpiles of fill and waste material will be required on site at strategic locations to minimise traffic movements and haulage and to reduce impact to the working site and immediate surroundings. Suitable and appropriate management of the waste areas will be required, including run off control to reduce impact to surface waters and or drainage and also dust suppression if fine materials are being stockpiled, such as silts and fine sands.

10.7 Other Construction Considerations

The presence of the silt will necessitate careful control of earthworks to ensure that dust nuisance and silt runoff is kept to a minimum. This can be achieved by ensuring that the dust is suppressed by mitigation such as water spraying from water carts and silt runoff controlled by settlement ponds. Stormwater control and dust emissions are addressed in the Assessment of Stormwater Disposal and Water Quality (Technical Report 3) and Assessment of Air Quality (Technical Report 10).

The four laning of Main South Road will require new construction to be merged in to the existing pavement. The interface should be stepped to ensure that a permanent construction joint is not created at the interface that could result in differential settlement.

Whilst the site investigations have not identified any ground impacted by contamination within the designated route, encountering unforeseen contaminated ground during the construction period cannot be discounted. Should contaminated ground be encountered during construction, its management should be carefully considered. If the impacted area does not support structural form, then a capping solution whereby the contaminated soils remain in situ, should be considered. If however, the material compromises foundations and or pavement, then the material should be excavated and removed to appropriately licensed landfill after a full characterisation of the contaminants is concluded. Contamination is addressed in the Contaminated Land Assessment (Technical Report 16).

11. Risks, Effects and Recommended Mitigation

11.1 General Comment

From a geotechnical and geological perspective, the design and construction of the second stage of Christchurch Southern Motorway and the four laning of Main South Road, are relatively straight forward with few inherent risks associated with ground conditions. The flat topography and relatively homogenous geology comprising granular alluvial soils provides a low risk environment where at grade construction will not incur any slope stability issues. The strength of the soils are sufficient, such that traditional strip and pad foundations or simple driven piles offer cost effective and low impact solutions.

The change in seismic activity and setting for the Canterbury region as a consequence of the earthquake events from September 2010 onwards does pose an element of risk in terms of elevated PGAs, ground shaking, ground rupture and liquefaction (at depth in the soil horizon). The current work being undertaken by GNS will influence the scope, extent and parameters implemented through the Building Code and Structural Engineering Standards. Detailed design may require a review of the current SLS and ULS parameters used at this preliminary design stage, should a new seismic regime be implemented.

Mitigation of risks and effects associated with ground conditions and geological hazards will be largely addressed through detailed and commensurate investigation for the detailed design of the structures and implementation of the appropriate geotechnical parameters which will ensure the risk is 'designed out'.

Effects of the Project on the environment that relate to ground conditions are more pertinent to the actual construction period and are more concerned with the type of methods employed during construction, for example the type of foundation solution, which could have adverse effects, such as noise and vibration. Engineering and construction management practice will mitigate or control these activities. This can be addressed through management plans and on-going monitoring to ensure compliance and methodologies are adhered to minimise the impact to the environment.

11.2 Specific Risks, Effects and Mitigation

Table 8 provides a summary of the main identified risks associated with the Project from a geotechnical and geological hazard perspective. Where possible, specific mitigation methods are proposed in the table. The most appropriate mitigation will be subject to the final construction form that is developed, and the aspects (e.g. foundations) to be adjusted and identified in the design or construction phase, with contractor management input.

Table 8 Summary of considerations, risks, effects and their mitigation relating to ground conditions

Category	Risk	Effect	Mitigation
Ground Conditions.	Bearing Capacity.	Insufficient soil strength may result in an increase in the structure/foundation footprint, with potential impact to designation.	Further investigation at the detailed design stage to ensure ground conditions are commensurate with the foundation solution.
	Settlement.	Excessive settlement beyond design capacity.	Further investigation at the detailed design stage to ensure ground conditions are commensurate with the foundation solution.
	Liquefaction.	Seismic settlement and liquefaction of deeper soil horizons may result in soil strength reduction and differential settlement.	Appropriate investigation at detailed design stage cognisant of the GNS changes in the regional seismic setting. Investigations at specific structures to extend at least 20 m below ground level. Foundations designed in line with the design philosophy.
	Slope Stability.	Very limited due to at grade construction. Minor instability issues related to cuttings and embankments.	Ensure appropriate material is used for embankments with cognisance of geotechnical parameters.
	Permeability of Soils.	Too low permeability impacts on design of stormwater infrastructure with potential redesign required or impact on designation and aesthetic value due to increased capacity requirements.	On-going monitoring of the existing piezometers and groundwater levels and further permeability testing to increase confidence in the design parameters. Ensure the design of any infrastructure is commensurate with permeability and Design Philosophy Statement (Technical Report 1).

Category	Risk	Effect	Mitigation
	Unforeseen ground conditions.	Increased volumes of cut material unsuitable for foundation strata or fill. Will require imported material to back level.	Increased volume of investigation to reduce risks associated with unforeseen ground conditions.
Seismicity.	Fault Rupture and Ground Displacement.	Displacement and damage to infrastructure at ground level.	Difficult to mitigate against with no certainty of the magnitude or direction of any potential fault rupture.
	Regional Seismicity.	Change in agreed seismicity levels for the region by GNS will affect the building codes and levels of importance for structural analysis with impact on preliminary designs.	Adhere to Standards and adjust any seismic design parameters as necessary.
	Peak ground Accelerations (PGAs).	Ground shaking and distress to structures.	Adhere to Standards and adjust any seismic design parameters as necessary.
	Liquefaction and Settlement.	Seismic settlement and liquefaction of deeper soil horizons may result in soil strength reduction and differential settlement.	Appropriate investigation at detailed design stage cognisant of the GNS changes in the regional seismic setting. Investigations at specific structures to extend at least 20 m below ground level. Foundations designed in line with the design philosophy.
Groundwater.	Shallow (near ground) water table.	Impact on bearing capacity and foundation design Instability to trenching during construction Disposal to ground capacity reduced.	Appropriate foundation design applied. Dewatering implemented during construction. Further testing for specific sites to ensure confidence in design.

Category	Risk	Effect	Mitigation
	CPW Groundwater mounding.	Increased groundwater levels above design groundwater level in the CSM1 /2 intersection area. This may influence the laying of foundations to the road and disposal field for the local stormwater ponds.	Stormwater ponds designed to have sufficient capacity to be inundated on occasion. Foundations to infrastructure to be cognisant of influence of high groundwater on design.
	Seismic activity.	On-going seismic activity may adversely affect the groundwater regime.	Continue groundwater monitoring prior to construction and correlate with any significant seismic event to identify trends or effects.

Category	Risk	Effect	Mitigation
Construction.	Pile Driving.	<p>Environmental impact in terms of noise nuisance to surrounding area.</p> <p>Vibration of immediate environment with potential distress or shaking to adjacent structures/ground.</p>	<p>Careful selection of type of pile and plant used for their construction depending on engineering design and the proximity of sensitive environments and structures. The Assessment of Construction Noise & Vibration (Technical Report 9) outlines predictions of construction vibration levels and distances from foundation solution where buildings may be at risk. It was assessed that it is unlikely for there to be any buildings along the CSM2 alignment with a High vibration risk, Predictions of construction vibration levels indicate there is a degree of risk that the Project criteria may be exceeded at dwellings within 20 m of the Main South Road Four-Laning alignment. This will be refined and supported by site-specific measurements once construction begins.</p> <p>Implementation of the Construction Noise and Vibration Management Plan (SEMP004) identifying active noise and vibration management and mitigation measures. Monitoring to ensure levels of noise and vibration are within acceptable limits.</p>

12. Conclusions

The geological and geotechnical investigations undertaken for the Project, together with an assessment of ground related hazards has demonstrated there are no unusual ground conditions along the proposed alignment.

There are no significant apparent risks that cannot be readily managed through appropriate design and construction management, with standard foundation and engineering solutions.

Given the recent seismic activity and changes to the regional setting in terms of seismic design parameters, it is essential to ensure that any detailed design for structures and embankments/slopes is appropriate to the seismic regime. The seismic setting is subject to potential change so it will need to be checked at the time of detailed design.

13. Technical References

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