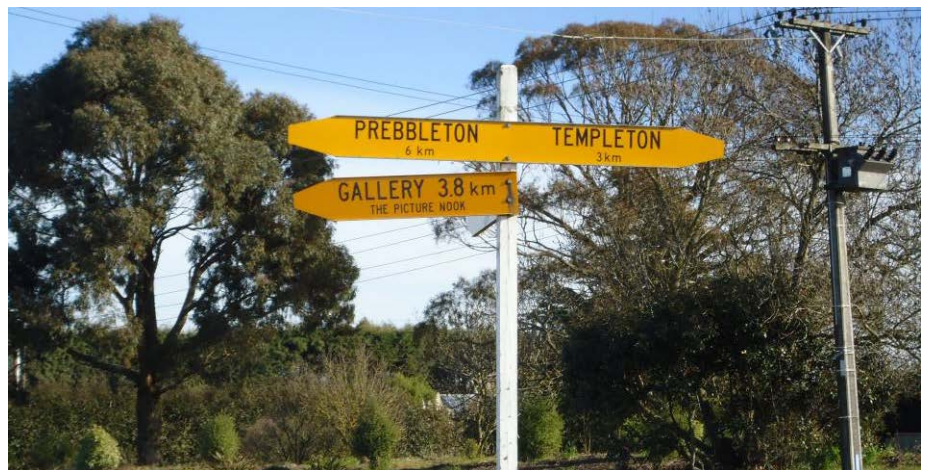


Technical Report No 1

Christchurch Southern Motorway Stage 2 and Main South Road Four Laning

Design Philosophy Statement

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

| No. | Technical Report Title | Primary AEE Chapter Reference | SEMPs |
|-----|---|-------------------------------|--|
| 1 | Design philosophy statement | 4 | |
| 2 | Traffic and transportation effects report | 11 | Construction Traffic Management Plan |
| 3 | Assessment of stormwater disposal and water quality | 19 | Erosion and Sediment Control Plan, Accidental Aquifer Interception Management Plan |
| 4 | Landscape and visual effects | 15 | Landscape Management Plan |
| 5 | Assessment of effects – urban design | 14 | Landscape Management Plan |
| 6 | Urban and landscape design framework | 14, 15 | Landscape Management Plan |
| 7 | Landscape context report | 15 | Landscape Management Plan |
| 8 | Assessment of operational noise effects | 17 | |
| 9 | Assessment of construction noise & vibration | 17 | Construction Noise and Vibration Management Plan |
| 10 | Assessment of air quality effects | 18 | Air Quality Management Plan |
| 11 | Geotechnical engineering and geo-hazards assessment | 3, 21 | |
| 12 | Assessment of archaeological effects | 24 | |
| 13 | Social impact assessment | 26 | |
| 14 | Economic impact assessment | 25 | |
| 15 | Cultural impact assessment | 23 | |
| 16 | Contaminated land assessment | 22 | |
| 17 | Aquatic ecology assessment | 20 | |
| 18 | Terrestrial ecology assessment | 20 | |
| 19 | Lighting assessment | 16 | |
| 20 | Statutory provisions report | 6, 28 | |
| - | Construction Environmental Management Plan | 5 | |

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

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- Marshs / Shands
- Springs / Halswell Junction
- SH1

1. Executive Summary

The Design Philosophy Statement (DPS) focuses on the philosophy of design for the key civil, structural and other related design components for the NZ Transport Agency's (NZTA) Christchurch Southern Motorway Stage 2 (CSM2) and Main South Road Four Laning (MSRFL) Project. The philosophy has been generated by the NZTA's scope of services and best practice design, with the key aspects outlined below.

- Land take required is minimised: within the corridor outlined in the Strategic Study land take and the number of land owners effected are minimised through considerations of the appropriate road cross-sections, interchange forms and stormwater treatments.
- Effects on adjacent residents is minimised: these effects include access, noise and visual elements.
- Effects on the environment are minimised: including stormwater, noise and landscaping.
- Connectivity of existing infrastructure is maintained: including local roads, property accesses, walking and cycling routes and stockwater races.
- All elements of the Project are designed to the relevant standards: including NZTA's RoNS design standards and guidelines.
- Route security: including recognition of secondary routes, design to standards, especially relating to stormwater (flooding) and structures (seismic effects) and the setting of the designation.
- Construction and maintenance costs are minimised.
- Continuity with Christchurch Southern Motorway Stage 1 (CSM1): including cross-sectional widths, pavement and structure types, levels of accessibility and landscaping and visual approach to design.

Along with the above philosophy, the Project design has been driven by the traffic modelling results and the receiving environment – particularly the existing flat terrain of the Canterbury plains, existing subdivisions and residences recent Plan Changes and proposed Plan Changes.

The Project has undergone two years of design consideration that led from strategic level studies through option considerations to more fine-tuned design. It has also been informed by consultation with key stakeholders including Selwyn District Council (SDC), Christchurch City Council (CCC) and Environment Canterbury (ECan), as well as landowner discussions around preferences.

This report does not consider how the Project will be constructed in any detail but it does reference staging and sequencing of the Project and on-going maintenance. The draft Construction Environmental Management Plan provides fuller detail on the construction process, particularly in relation to environmental performance.

2. Design Philosophy Introduction

The DPS describes the philosophy by which the Project has been developed and designed. The DPS It is a live document that outlines the current status of the process (the Scheme Assessment Design) and will change as the project progresses to the detailed design stage.

The DPS focusses on the key civil and structural elements of the design, referring in some instances to the technical reports for further detail on particular aspects of the design. These aspects are detailed in Table 1, with references to the relevant technical reports and sections of this report. The DPS sets out the philosophy that each of these aspects has been designed to.

Table 1 Key Civil, Structural and Other Design Aspects

| Design aspect | Section of the DPS | Technical Report Number |
|---|--------------------|-------------------------|
| Route Security | 4.4 | 2 |
| Design Standards | 5.1 | 2,3,4,6,7,8,9,11,19 |
| Design Traffic | 5.2 | 2 |
| Geometric Design | 6 | 1 |
| Interchanges and intersections | 7 | 2 |
| Walking and cycling | 8.1 | 2 |
| Railway Crossings | 8.2 | 2 |
| Commercial Vehicle Inspection unit facility | 8.3 | 2 |
| Street lighting | 8.4 | 19 |
| Utility and Public services | 8.6 | - |
| Geotechnical | 9 | 11 |
| Pavement | 10 | 1 |
| Stormwater | 11 | 3 |
| Structures | 12 | 1 |
| Property and designation | 13 | 1 |
| Sediment control | 14.1 | CEMP* |
| Noise | 14.2 | 8, 9 |
| Landscape | 14.3 | 4, 6, 7 |
| Construction and operation | 15 | CEMP* |
| Long term maintenance requirements | 16 | - |

* Construction Environmental Management Plan – this is not a Technical Report, but forms part of the lodgement and is contained in Volume 4.

A full list of technical reports associated with this project is listed on page ii of this report.

The design will be refined and finalised during the detailed design phase, based on designation and resource consent conditions and the philosophy for the civil and structural aspects contained in this report. This technical report should also be read in conjunction with the plan set (Volume 5 of the lodged documentation) and the Assessment of Environmental Effects.

3. Existing Southern Corridor

The existing southern corridor is defined as the route taken to travel from Rolleston to Brougham Street via the Christchurch Southern Motorway Stage 1 (CSM1) – this is described below to give context to the improvements discussed as part of the Project design in the remainder of the DPS. This existing corridor assumes that CSM1 has been constructed and is operational. CSM1 is currently under construction, due for completion in early 2013. The existing southern corridor route includes:

- Main South Road (MSR) from Rolleston to Templeton (rural section)
- MSR from Templeton to Halswell Junction Road (HJR) (urban section)
- HJR from MSR to CSM1, and
- CSM1 from Halswell Junction Road (HJR) to Brougham Street.

3.1 MSR – Rolleston to Templeton

MSR is a two lane undivided highway which forms part of State Highway 1 (SH1). The section between Rolleston and Templeton is approximately 8 km long and is generally in a rural environment.

The existing MSR horizontal alignment is characterised by long straights, with the exception of two large radius curves at the Weedons Road intersection and the southern end of the passing lanes respectively. The design speeds of these curves exceed the standard for a 110 to 120 km/h speed environment. The current posted speed is 100 km/hr in this section.

The vertical alignment is typical of the flat rural environment on the Canterbury Plains, with good forward visibility. Currently adjacent properties access MSR directly along the route, with 6 existing intersections within this section.

Between Rolleston and Templeton, there are two sets of passing lanes. The first set is immediately south of the Weedons Road intersection with passing lanes provided in both directions. The passing lanes are one kilometre in length and merge back to two lanes approximately 900 m north of the Hoskyns Road signalised intersection. The second set is located south of the Kirk/ Trents Road intersection at Templeton. These are also provided in both directions and merge back to two lanes prior to the Waterholes Road intersection.

3.2 MSR – Templeton to Halswell Junction Road (HJR)

Templeton denotes the southern entry into Christchurch City and a change from a rural to peri-urban environment. The MSR geometry and lane configuration is consistent with the rural section described above, with the addition of a flush median between Barbers Road and the Islington Tavern in recognition of the cluster of development along this section. North of the Tavern the median tapers back to a typical centreline through to the HJR traffic signals.

The section between Templeton and HJR is approximately 3 km long, containing 4 intersections and direct property accesses onto MSR. The current posted speed is 70 km/hr in this section.

3.3 HJR

HJR has recently been upgraded between MSR and Springs Road as part of the CSM1 project and will temporarily become a state highway for the period between the completion of CSM1 and opening of CSM2. This section is approximately 2.5 km in length.

The recent HJR upgrade included resurfacing and the addition of a flush median. It remains a two lane undivided road. This section of HJR passes through an industrial zone, with direct access from businesses onto HJR. The current posted speed is 70 km/hr in this section.

The upgraded work included traffic signals at the Shands Road intersection and an upgraded roundabout at Springs Road.

3.4 CSM1

CSM1 connects HJR at Springs Road, with the Brougham Street Arterial (SH 73) in the east. This section is a four-lane median separated motorway. This section is approximately 8 km in length, comprising two distinct sections:

- Four-lane median separated motorway between the Halswell Junction Road Springs Road intersection and the Curletts Road interchange, with underpasses at Aidanfield Drive and Awatea/Dunbars Roads.
- Upgrade to the existing Christchurch Southern Motorway to a four-lane median separated motorway between the Curletts Road interchange and Collins Street with full interchange access at both Curletts Road and Barrington Street.

The posted speed limit for this section is 100 km/hr.

4. Proposal Description

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'.

4.1 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.

4.2 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 motorway standard comprising four lanes, with two lanes in each direction, with a median and barrier to separate oncoming traffic and provide for safety.¹ 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,

¹ 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.

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.

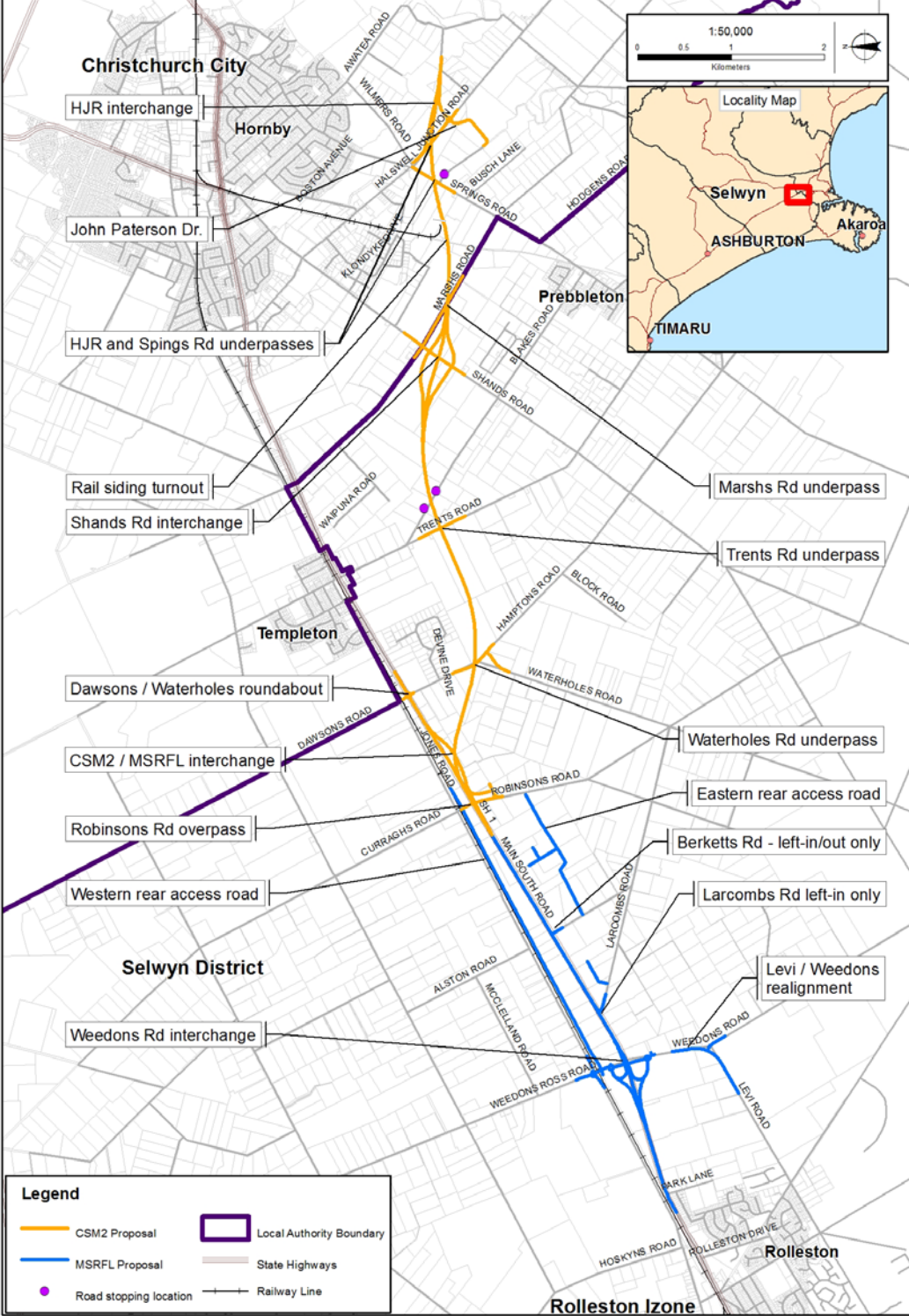
4.3 Key Design Features

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)
- 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



4.4 Route Security

As a RoNS, the Project is critical for supporting economic growth, reducing congestion and improving safety in the region. While not a Project objective set out by the NZTA, its construction will provide better security of access between the south of Christchurch and key destinations including Christchurch City centre and Lyttelton Port. The security of this route has been a key part of the design philosophy as is described further in individual sections below.

With the exception of Blakes Road within MSRFL, existing state highway and local road connectivity is maintained throughout the extent of the Project. The maintenance of these connections is important from a route security viewpoint, as they provide secondary routes should the MSRFL / CSM2 link be closed. In particular, the MSR / SH1 route which currently acts as the main access point to Christchurch from the South will remain operational following construction of the Project and can be used in the event of CSM2 being closed.

5. Design Standards & Traffic

5.1 Design Standards

Key standards, guidelines and manuals used on the key civil and structural design elements of the Project are shown below. The technical reports should be referred to for further details and descriptions.

Geometric Design

- Roads of National Significance (RoNS) Design Standards and Guidelines, 2009.
- NZTA (Transit) State Highway Geometric Design Manual (SHGDM) (Draft), 2000.
- Austroads AGRD Guide to Road Design (suite of standards), 2010.
- Christchurch City Council (CCC) Construction Standard Specification, 2010.
- NZTA 'Safe Systems' memorandum, 2012.

Traffic Signal Design

- Austroads AGRD04A-10 Guide to Road Design Part 4a: Unsignalised and Signalised Intersections, 2010.
- Land Transport NZ RTS 14 Guidelines for Installing Pedestrian Facilities for People with Visual Impairment, 2007.
- Signals New Zealand User Group (SNUG) National Traffic Signal Specification, 2005.

Signage and Pavement Marking Design

- NZTA Manual of Traffic Signs and Markings (MOTSAM), 2010.
- NZTA Traffic Control Devices Manual, 2008.
- CCC Construction Standard Specification, 2010.

Railway Level Crossing

- NZTA Manual of Traffic Signs and Markings (MOTSAM), 2010.
- NZTA Traffic Control Devices Manual Part 9: Level Crossings, 2008.

Street Lighting Design

- AS/NZS 1158.0:2005 Lighting for Roads and Public Spaces.

- AS 4282:1997 Control of the Obtrusive Effects of Outdoor Lighting.

Pavement Design

- Austroads AP-G17-04 Pavement Design Guide.
- NZ supplement to the 2004 Austroads pavement design guide, 2007.
- NZTA specifications for Pavement Design and Construction (B, M and P series).
- CCC Construction Standard Specification (parts 1 and 6), 2010.

Stormwater

- NZTA Stormwater Treatment Standard for State Highway Infrastructure, 2010.
- CCC Waterways Wetlands and Drainage Guide, 2003.
- ECan (report R11/19) – Natural Resources Regional Plan, 2011.
- SDC Code of Practice, 2010.
- Opus International Consultants Ltd – Selwyn District Design Rainfall, 2009.
- CCC Construction Standard Specification, 2010.

Structural Design

- NZTA SP/M/022 Bridge Manual, 2003.
- NZS 3404: Part 1:1997 Steel Structures Standard, 1997.
- NZS 3101: Part 1:2006 Concrete Structures Standard, 2006.
- NZS 1170.5 Earthquake Actions – New Zealand, 2004.

Sediment Control

- ECan (report R06/23) – Erosion and Sediment Control Guidelines, 2007.

Noise

- NZS 6806:2010 Acoustics – Road-traffic noise – New and altered roads.

Landscape

- NZTA Urban Design Policy, 2007.
- NZTA SP/M/020 Guidelines for Highway Landscaping, 2006.

5.2 Design Traffic

Traffic flows, the percentages of heavy vehicles and future travel demand have been assessed from the 2016, 2026 and 2041 traffic models developed for the Project – the CSM2 Project Model (CPM). This model has been derived from the Christchurch Transport Model (CTM version 2.) More detailed operational modeling of the Shands Rod interchange has been undertaken using VISSIM and all other intersections using SIDRA.

The models have been used to assess the performance of the transportation network in the Project area for a historic base year of 2006. This base year model has been subject to peer review in December 2010 and is considered fit for the purpose of assessing the future year traffic impacts of the Project.

The 2016, 2026 and 2041 future year models have been developed using demographic forecasts and planned growth consistent with the pre-earthquake Urban Development Strategy growth forecasts. The effects of the Canterbury earthquakes have therefore not been explicitly taken into account. However, it is expected that a slightly faster rate of growth will occur in the area served by the Project, and the overall changes in population and employment should not significantly alter the modeled outcomes.

Further details on the traffic modelling and outcomes are contained in Technical Report 2 – Assessment Traffic and Transportation Effects.

6. Geometric Design

The philosophy behind the geometric design has been determined by the Scope of Services document for the Project, in particular the project objectives. Geometric design of the mainline elements has largely been based on the RoNS 'Design Standards and Guidelines' and road classification as per the Scope of Services.

Refer to the Plan and longitudinal section drawings (62236-A-C100 to C133 and 62236-B-C101 to C163) contained within the Plan set for further geometric information.

6.1 Road Classification

MSRFL

MSRFL will be upgraded to a four laned median divided expressway, providing for on-road cyclists and retaining existing intersections at Berketts Road and Larcombs Road (with right turns prohibited). Direct private property access is removed from MSR and alternate rear access is provided. The road geometry is designed to motorway standard and will provide continuity with CSM1 and CSM2.

The philosophy for the MSRFL design is to provide a safe and consistent cross section with the CSM1 and CSM2 motorways, while retaining existing local road intersections to maintain connectivity and doing so within a minimal designation to minimise the impact on adjacent properties. For these reasons MSRFL has not been designated as a motorway.

MSRFL will remain as SH1 following completion of the Project.

CSM2

CSM2 is to be constructed to a motorway standard. The NZTA's motorway standard is achieved by a four lane median divided arterial road with no property access, intersections, or cycle and pedestrian access for CSM2.

The philosophy for the CSM2 design is to provide a safe and consistent motorway in accordance with recognised standards and to be in alignment with the CSM1 design philosophy that is currently under construction. Access onto the motorway is provided via the interchanges.

CSM2 will become SH76 following completion the Project (as per CSM1).

6.2 Mainline Design Speed

MSRFL

As per the RoNS and SHGDM standards for a rural state highway (in an open and flat environment), a design speed of 110 km/hr has been adopted for MSRFL from the proposed tie-in at the southern end,

just north of Rolleston, to the north bound major fork (bifurcation) of MSR (Northbound) and the start of CSM2. The current posted speed of MSR is 100 km/hr.

CSM2

As per the Austroads and RoNS standards for a motorway, a design speed of 110 km/hr has been adopted for CSM2 from the north bound bifurcation of MSR to the tie-in to CSM1, east of Springs Road/Halswell Junction Road intersection.

6.3 Mainline Horizontal Geometry

The horizontal geometry for both sections complies with the RoNS geometric standards, with key measures as outlined below.

- Desirable minimum radius 1100 m (110 km/h) 820 m (100 km/h). The minimum radius defines the minimum curvature within the alignment.
- Absolute minimum radius 720 m (110 km/h) 550 m (100 km/h).
- Desirable curve length 500 m.
- Minimum curve length 300 m.
- Maximum super-elevation (defined as banking of the roadway along the curve) six per cent (based on minimum radii and speeds).

MSRFL

MSRFL has been designed with the philosophy of retaining the existing MSR horizontal alignment as closely as possible to limit the designation required. With this in mind widening to the west has been proposed, as there is existing road designation available on this side of MSR (although additional designation for the widening is still required for the Project).

The MSRFL section comprises two horizontal curves as shown in Figure 2 and Table 2 below.

Figure 2 MSRFL Curve Geometry



Table 2 MSRFL Curve Geometry

| Parameter | Curve 1 | Curve 2 |
|---------------------|---------|---------|
| Design speed (km/h) | 100 | 100 |
| Radius (m) | 1200 | 1200 |
| Curve length (m) | 275 | 355 |

CSM2

From a horizontal point of view, CSM2 reflects the philosophy of providing a direct link from MSRFL to CSM, minimising negative effects on land owners and providing the best integration with local roads possible. This has been carried out within the corridor identified through the Strategic Study (Opus, 2009).

The CSM2 section comprises eight horizontal curves as shown in Figure 3 and Table 3. Although the first curve is only 105 m in length, it has a large radius (7000 m) and therefore is effectively regarded as a straight.

Figure 3 CSM2 Alignment

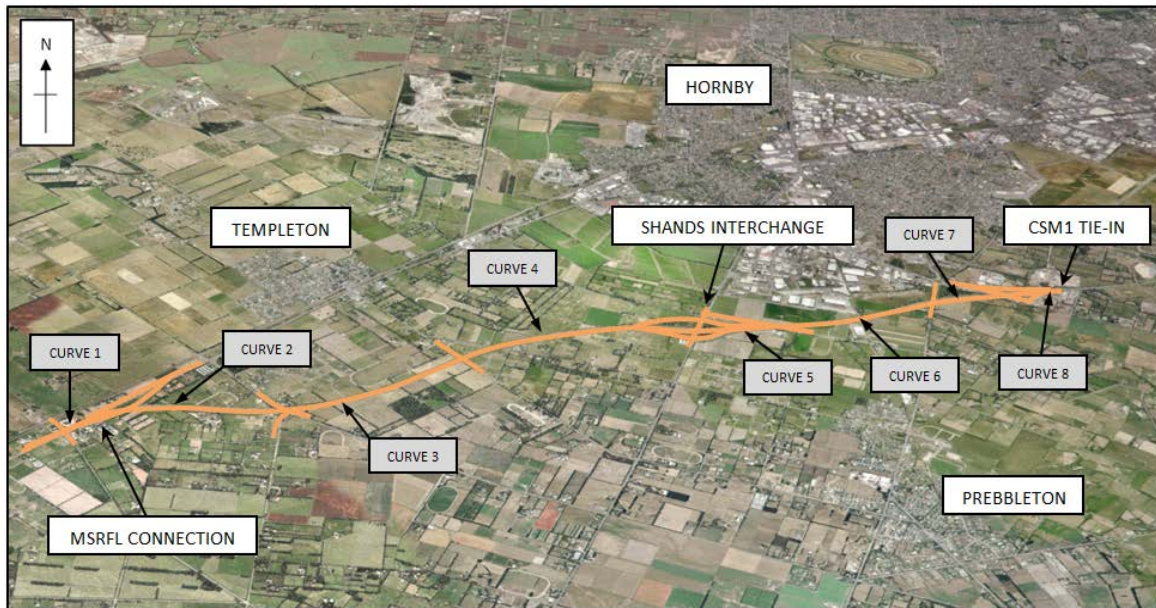


Table 3 CSM2 Curve Geometry

| Parameter | Curve 1 | Curve 2 | Curve 3 | Curve 4 | Curve 5 | Curve 6 | Curve 7 | Curve 8 |
|---------------------|---------|---------|---------|---------|---------|---------|---------|---------|
| Design speed (km/h) | 110 | 110 | 110 | 110 | 110 | 110 | 110 | 110 |
| Radius (m) | 7000 | 800 | 1800 | 2400 | 2400 | 2000 | 2400 | 3500 |
| Curve length (m) | 105 | 645 | 1235 | 1780 | 630 | 730 | 1080 | 325 |

6.4 Mainline Vertical Geometry

MSRFL

The vertical design philosophy was to provide connectivity and existing infrastructure, stockwater races, side roads and other local access wherever possible. Therefore it was established early in the design process that the existing vertical profile on the MSRFL section of carriageway needed to be maintained.

The vertical alignment generally falls from south to north on a very slight downhill grade of approximately 1 in 500. There are no significant vertical curves along MSRFL, with design levels and vertical curves being dictated by a mix of designation width constraints and maximising the use of the existing pavement. The MSRFL vertical design speed is 110 km/hr.

CSM2

The philosophy for this section of works has been to minimise the costs and effects of the works retaining route security for the alignment. The Strategic Study proposed that CSM2 would be placed in a trench.

Flooding was identified as a significant risk to the route security with this approach and, following groundwater analysis and consideration of discharge into Montgomery's Drain and Upper Knights Stream, the option of placing CSM2 into a trench was not considered feasible as it would severely restrict the ability to discharge road stormwater runoff.

Elevating CSM2 was not considered to be practical given the potential for increased environmental effects on the surrounding area, including noise, landscape and visual impacts. A raised motorway would also lead to a significant increase in construction costs associated with structures and substantial embankments.

Due to the above constraints, the proposed alignment is typically at or near grade to allow the disposal of stormwater above design groundwater levels, minimise the depth of stormwater crossings and stock-water race siphons, minimise the risk of road closure due to flooding and minimise the effects on the surrounding area. Allowing for cross fall across CSM2, a verge, swale and soak pit, the invert of the disposal system is typically 1–2 m above the design groundwater level.

The CSM2 section crosses an existing KiwiRail corridor between Marshs Road and Springs Road. KiwiRail has agreed to allow the motorway to pass across the railway corridor at-grade on the undertaking that the line is reconfigured to allow shunting and access into the existing industrial area sidings.

The vertical alignment generally falls from west to east on a subtle downhill grade of approximately 1 in 300. As a result, there are no significant vertical curves along CSM2, with a resulting 110 km/hr vertical design speed. The minimum vertical curve length is 120 m, which along with grades and curve radii fall within the RoNS guidelines.

6.5 Local Road Connection Geometry

The local road connections and associated structures have been designed to minimise negative effects on local residents. This has been achieved by keeping the structures and local roads on the existing alignment where possible to minimise the designation required and retain existing connectivity. The mainline (MSRFL and CSM2) has been kept at-grade, with adjustments proposed to the local roads (over or under the mainline). Retaining the local roads at grade would have resulted in increased negative effects and costs arising from the longer approach embankments required for CSM2 or MSRFL.

The majority of the local road connections are straight horizontal alignments. Therefore the proposed design speeds are based on the vertical geometry of proposed underpasses (local road over the mainline) and where relevant, the influence of intersection layouts.

All local roads in the vicinity of the Project are currently posted at 100 km/hr. However, it is anticipated that the current posted speeds will be revised with likely speed limit reductions as a result

of the Project and surrounding developments. Confirmation of these reductions and the resultant posted speeds will be undertaken in consultation with SDC and CCC during the detailed design phase.

Proposed design speeds and crest curve criteria for local roads crossing MSRFL and CSM2 are listed in Table 4.

Table 4 Design Speeds of Local Roads

| Road Section | Existing Posted Speed (km/hr) | Design Speed (km/hr) | Reason |
|---|-------------------------------|----------------------|--|
| Weedons Road / Weedons Ross Road underpass | 100 | 60 | Part of interchange layout with roundabouts located either side of the underpass. The roundabouts will act as a speed threshold and to minimise impact on adjacent properties, a design speed of 60 km/h has been adopted. |
| Robinsons Road / Curraghs Road overpass (local road under the mainline) | 100 | 60 | Due to the close proximity of the railway line and Jones Road intersection to the west, the vertical geometry has been restricted to a 60 km/hr design speed. The speeds to the east are managed by the inclusion of a roundabout on Robinsons Road. |
| Waterholes Road / Hamptons Road underpass | 100 | 80 | To minimise the impact on adjacent properties and also extent of approach embankments, the design speed has been reduced to 80 km/h. |
| Trents Road underpass | 100 | 80 | To minimise the impact on adjacent properties and also extent of approach embankments, the design speed has been reduced to 80 km/h. |

| Road Section | Existing Posted Speed (km/hr) | Design Speed (km/hr) | Reason |
|----------------------------------|--------------------------------------|----------------------|--|
| Shands Road underpass | 100 | 60 | Shands Road is currently posted at 100 km/hr up to Sir James Wattie Drive (300 m north of Marshs Road) where the posted speed reduces to 70 km/hr. As part of the Shands Road interchange, SDC has identified that upgrades are required to their local road network including new intersection controls at Shands / Blakes intersection, located 500 m south of the proposed Shands Road interchange. Following discussion with SDC it is anticipated that the 100 km/hr posted speed along Shands Road will be reduced to 60 k/hr in the vicinity of the Shands Road interchange. This 60 km/hr speed limit will continue northwards to the 70 km/hr speed limit at Sir James Wattie Drive and has been adopted for this vertical design |
| Marshs Road underpass | 100 (from Sir James Wattie Drive 70) | 80 | To minimise the impact on adjacent properties and also extents of approach embankments, the design speed has been reduced to 80 km/h. This will also encourage vehicles to slow when approaching the Marshs Road signalised intersection. |
| Springs Road underpass | 80 | 80 | To minimise the impact on adjacent properties and also extents of approach embankments, the design speed has been reduced to 80 km/h. The existing 80 km/h zone stops near Marshs Road however it is anticipated this will extend northwards to the Springs Road roundabout approaches. |
| Halswell Junction Road underpass | 100 | 60 | The post construction safety audit on CSM1 has recommended that posted speeds on HJR are reduced. This is currently being investigated as part of the CSM1 project. It is expected that the outcome will be to reduce the posted speed on Halswell Junction Road from 70 km/hr to either 50 km/hr or 60 km/hr. This will be consistent with the proposed design speeds along Shands Road in the vicinity of the interchange. |

6.6 Typical Cross Sections

MSRFL and CSM2

The typical MSRFL and CSM2 mainline cross section philosophy has been based on the RoNS standards, thereby ensuring consistency with the CSM1 project for continuity. These comprise the following components:

- 2.5 m wide outside shoulder
- Four x 3.5 m wide traffic lanes
- 1.5 m wide inner shoulder and
- 3.0 m wide central median (inclusive of wire rope barrier).

This equates to a carriageway width of 25 m (inclusive of the central median).

Figure 4 Mainline (CSM2 and MSRFL)

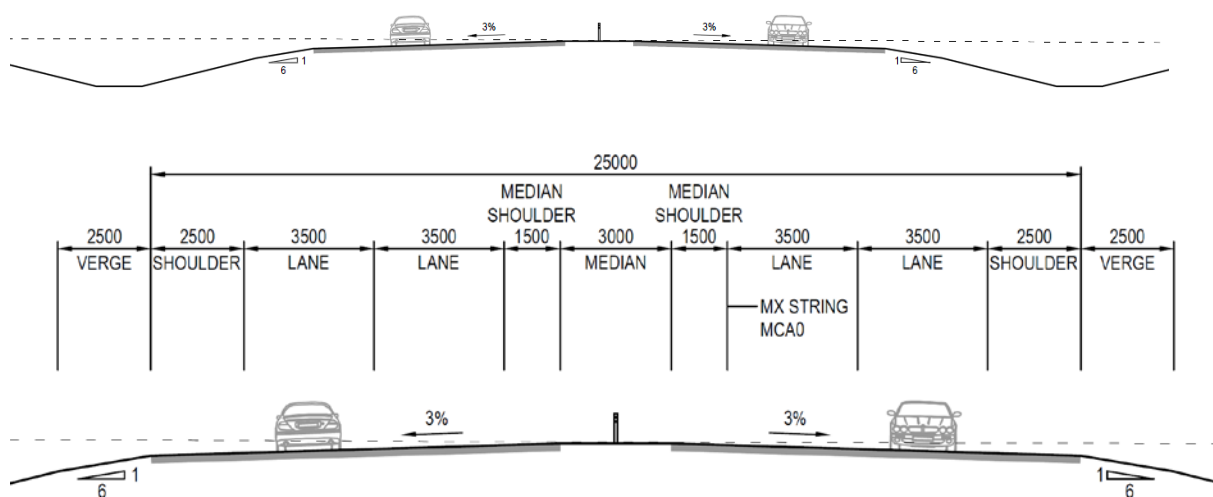


Figure 4 above is attached to show the extent of the cross section including the roadside swales proposed and is enlarged to illustrate the relevant cross sectional widths for the mainline.

Local Roads

Typical cross-sections for the rear access road, the underpass configurations and the interchange ramps are shown below. These are designed with the philosophy of retaining existing road widths as a minimum (for the underpasses) and ensuring connectivity for all road users.

Figure 5 Rear Access Road

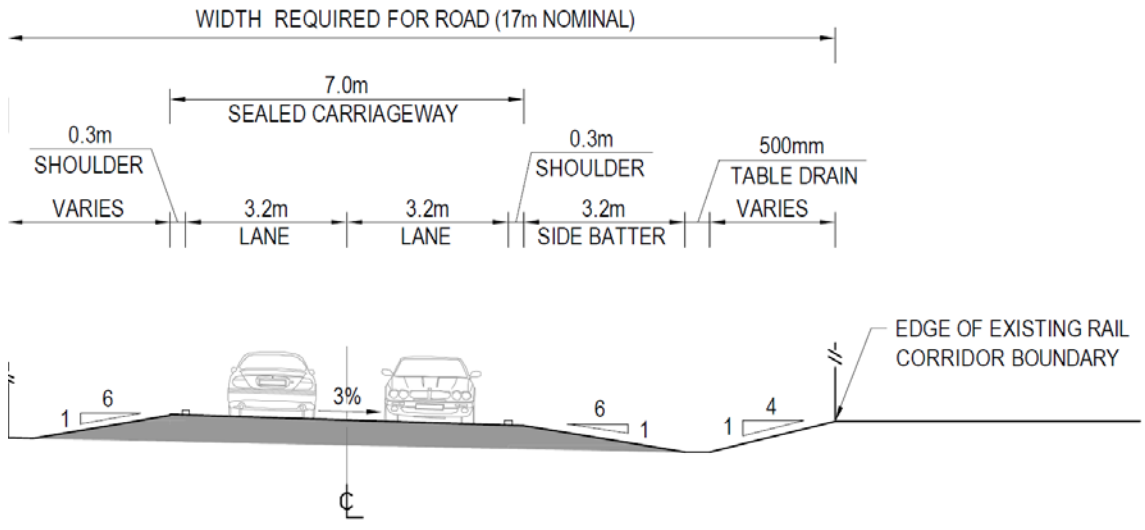


Figure 6 Typical underpass configuration

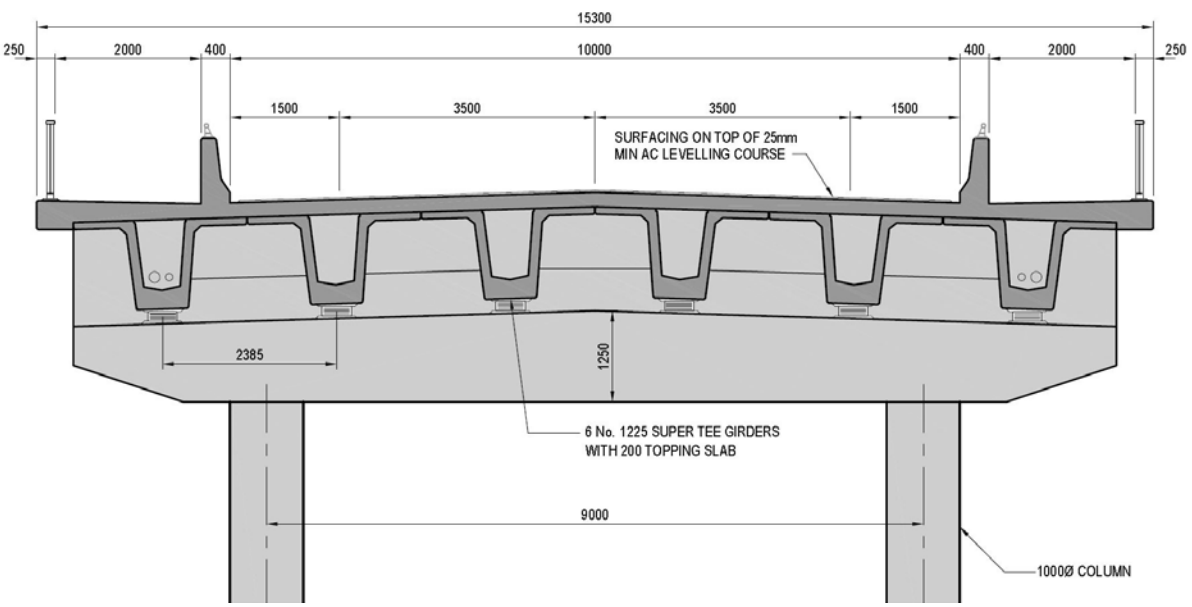
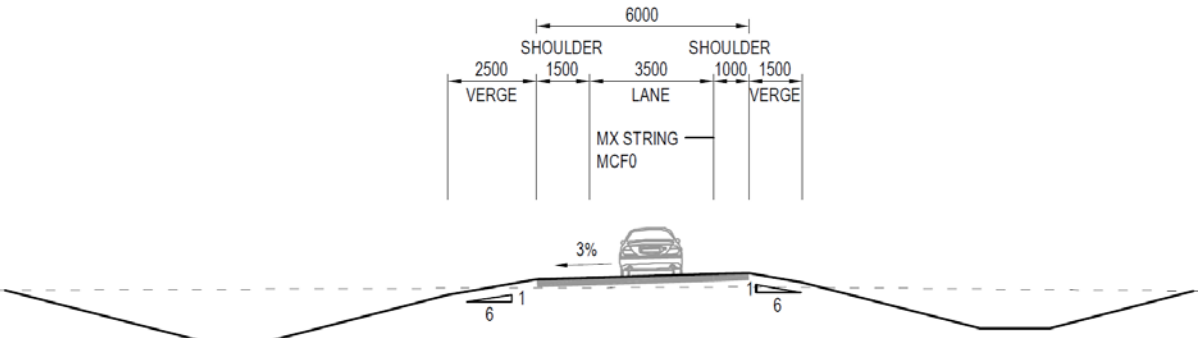


Figure 7 Typical interchange ramp configuration



6.7 Safe System Approach

At the commencement of the CSM2 and MSRFL scheme design the clear zone concept was the NZTA's guideline for the treatment of the roadway environment adjacent to the carriageway. The general principle was to adopt a 9.0 m wide clear zone that is free of hazards which could cause harm to errant vehicles. The design philosophy was initially based around this concept with frangible lighting poles and small scale landscaping the key features located within the 9.0 m clear zone.

During 2011/12 the NZTA adopted the 'safe systems' approach to provide an even safer roadside environment which ideally removes any risk of potential death or serious injury for vehicles that may leave the carriageway, This has since become the overarching philosophy with regards to creating a safe cross section. This approach means there is no fixed safe clear zone width for errant vehicles and in many situations, roadside safety barriers should be considered to protect the motorist from roadside hazards. The 'safe systems' approach is therefore somewhat subjective.

MSRFL

The MSRFL section consists of a 42.5 m wide cross section, resulting in a typical clear zone width of 11.75 m each side of the carriageway's edge line. The width of this designation is restricted by the desire to minimise the land required as a result of the designation along MSR.

The existing overhead power lines will be placed underground and the existing open water race will be culverted along the length of four laning to maximise the clear zone adjacent to MSRFL.

The 'safe systems' treatment is particularly important south of Weedons interchange where there are a row of oak trees along the eastern road reserve boundary. There is a strong desire from local residents, landscapers and SDC to retain as many of these trees as possible. It is therefore proposed to erect a TL4 wire rope barrier along this section to protect errant vehicles from hitting the oak trees.

CSM2

While the road cross section is identical to MSRFL, the lack of constraint to the designation means that the effective clear zone width is larger for CSM2 than MSRFL. The CSM2 design provides effective clear zone widths of 20.0 m from the outside edge of traffic lane to the top of the swales back slope, comprising:

- 2.5 m wide sealed shoulder
- 2.5 m wide 6:1 verge
- swale fore slope at 4 h:1v, typically 6.0 m wide
- swale base varying from 0.5 m to 2.0 m in width
- swale back slope at 4h:1v, typically 6.0 m to 7.0 m wide.

This cross section has been presented and discussed with the NZTA's National Design Manager where it was agreed that throughout the main alignment of CSM2 the proposed cross section was adequate from a 'safe systems' approach. Consideration should be given to flattening the swale fore slope to

6:1, if this can be accommodated within the designation following finalisation of the stormwater and landscaping treatments at detailed design stage.

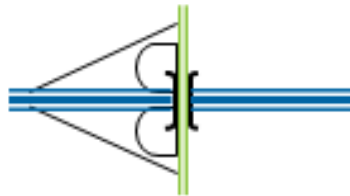
7. Interchanges / Intersections / Access provision

7.1 Interchanges

The Strategic Study and Christchurch and Rolleston Environs Transportation Study (CRETS) have been the basis for the location of the proposed interchanges. The proposed form of the interchanges have been informed by the results of the traffic modelling to ensure future route security and a philosophy to minimise land take and effects on adjacent land owners.

The Project includes interchanges at four locations as described below.

7.1.1 Weedons Road / Weedons Ross Road – partial clover leaf with south facing ramps



The construction of a full interchange at Weedons was recommended as a key component of the MSRFL Project through the CRETS study (refer to Chapter 7 of the AEE and Chapter 2 of Technical Report 2 – Assessment of Traffic and Transportation Effects). This was on the basis of the interchange functioning as the main access point into Rolleston (via Levi Road) and the Izone (via Jones Road), with the existing Weedons and Weedons Ross Road route becoming a district arterial between West Melton and Lincoln. This was supported by the NZTA and SDC.

At scoping stage, three interchange configurations were identified a partial clover leaf (parclo) and closed and open diamonds. The parclo option was preferred as it has the least impact on properties and businesses relative to both diamond options and a less prominent elevated structure than the closed diamond. All options maintained the highway alignment at-grade with Weedons Ross/ Weedons Road crossing overhead via an elevated bridge structure. As a result of the above, only the parclo interchange was carried forward to scheme design.

In terms of the parclo ramp terminal intersections, the preliminary traffic modelling identified that an acceptable operational performance could be achieved using two lane roundabouts. Roundabouts are supported by SDC and are the preferred form of control from a safety perspective.

The proposed form of the Weedons Road / Weedons Ross Road Interchange is illustrated in Figure 8.

Figure 8 Visualisation of Weedons / Weedons Ross Road parclo interchange



7.1.2 MSRFL /CSM2 Y layout with elevated southbound connection



The MSRFL and CSM2 connection is located on the northern side of the intersection of Robinsons and Curraghs Road. The design philosophy is for CSM2 to remain at-grade and function as the primary route, retaining connectivity to and from Templeton (north) via MSR on and off ramps.

Just south of the interchange, the two northbound lanes on MSR deviate on a large radius right hand curve to form the start of CSM2. A third outside northbound lane (northbound off-ramp) will continue straight to merge back into the existing MSR south of the intersection with Dawsons and Waterholes Road. In the southbound direction, MSR is proposed to diverge on a left hand curve to pass over the top of the CSM2 alignment, before merging back into the MSR alignment south (southbound on-ramp) of the Robinsons/ Curraghs Road intersection.

An option for an exit lane has also been included on the southbound lane of MSR to provide access to adjacent properties that will have their existing access severed by the motorway alignment. The exit lane will also link to the local road network via a new roundabout with Robinsons Road.

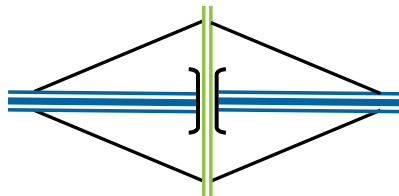
During the scheme development stage, the Y-interchange also considered providing for a U-turn facility between the northbound off-ramp and southbound on-ramp. However, this was later discounted after it was raised as a concern in the road safety audit due to the tight radius and steep grade required for this link. An alternative option for the U-turn movement is now proposed by installing a roundabout further north at the SH1 intersection with Dawsons and Waterholes Road. Refer to section 7.2.4 for further discussion on this roundabout.

The proposed form of the MSRFL / CSM2 Interchange is illustrated in Figure 9.

Figure 9 Visualisation of CSM2/ MSR connection²



7.1.3 Shands Road – full diamond



A full grade-separated interchange is proposed at Shands Road to provide access from Rolleston, the south of Lincoln and southern portions of the Hornby industrial area. High traffic volumes and safety considerations ruled out any at-grade intersection option at this location.

Given the close spacing of the Shands Road/Marshs Road intersection to the northbound ramps, traffic signals are considered to be the only practical solution for the ramp terminal intersections. Traffic signals provide a greater ability to control, synchronise and co-ordinate movement. Detailed micro-simulation modelling has demonstrated that an acceptable operating performance can be achieved. The signals also offer a better form of control for pedestrians and cyclists using the Shands and Marshs Road section of the Little River Rail Trail.

A further sub-option for a tighter closed diamond layout was also considered to provide increased spacing from Marshs Road, as well as moving the southern ramp terminal intersection further away from the Aberdeen subdivision (a rural residential development on the outskirts of Prebbleton).

² This photo simulation shows an earlier design. Refer to the Plan Set for the correct design details.

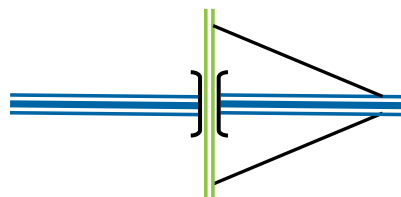
However, this would require the ramps to be raised on substantial embankments, including the construction of elevated ramp terminal intersections. Therefore, consistent with the philosophy to reduce effects on adjacent land owners, the option was discounted from further investigations.

On this basis, a spread diamond interchange with traffic signal control is being proposed as illustrated in Figure 10.

Figure 10 Visualisation of Shands Road Interchange



7.1.4 Halswell Junction Road – half diamond with east facing ramps



The CRETS study (refer to Chapter 7 of the AEE and Chapter 2 of Technical Report 2 – Assessment of Traffic and Transportation Effects) did not favour a direct motorway connection around Springs and Halswell Junction Road, rather that the CSM2/ Shands Road interchange be the primary connection in this vicinity to access the motorway from the Hornby industrial estate and local road network.

Full connectivity at HJR would have the effect of promoting traffic to use Springs Road with a potential for decreased level of service and associated amenity issues arising from increased traffic volumes through Prebbleton Township.

The motorway options presented in the first consultation newsletter in October 2010 (refer to Chapter 8 of the AEE) did not include an interchange at HJR. This generated considerable feedback for the NZTA to consider local road connectivity at this location.

The CSM2 Strategic Study identified that east facing, freight only ramps should be considered in the Springs Road area to enable Heavy Commercial Vehicles (HCVs) generated by adjacent industrial areas to quickly and efficiently access CSM1, Lyttelton Port and Christchurch City. This option retained the benefits to Prebbleton as described above, however issues around the policing required to restrict access on these ramps to only HCV's remained unresolved.

This option was presented in the second consultation newsletter in August 2011 (refer to Chapter 8 of the AEE) which included east facing, freight only ramps linking Halswell Junction Road with CSM1. Considerable feedback was received requesting that the NZTA consider full access ramps in this location. In addition the safety audit raised the freight only ramps as a significant concern, due to driver confusion, potentially complicated signage and the promotion of traffic to use less safe routes to access the city.

Based on preferences from the local community and CCC, the safety audit concerns and subsequent approval by NZTA, full access ramps are now proposed. A schematic plan showing the layout for options with eastward facing ramps is presented in Figure 11.

Figure 11 Visualisation of Halswell Junction Road interchange³



7.2 MSRFL intersections

Along with the philosophy to retain existing connectivity and accessibility throughout MSRFL, the below intersections have been designed to minimise land take and improve safety.

There are five existing intersections within the extent of the MSRFL Project:

³ This photo simulation shows an earlier design. The John Paterson Drive link to Halswell Junction Road is not shown. Refer to the Plan Set for the correct design details.

- Weedons Ross Road / Jones Road
- Levi Road / Weedons Road
- Larcombs Road
- Berketts Road and
- Dawsons Road/ Waterholes Road.

Details of the options considered at each of these crossings are provided below.

7.2.1 Weedons Ross Road / Jones Road

CRETS identified Jones Road as the main access to the Izone industrial area from the north (Christchurch) via MSRFL and the Weedons Road interchange.

Currently Weedons Ross Road forms a crossroads with Jones Road, with Weedons Ross Road having priority. In line with the CRETS recommendation a dual laned roundabout is proposed to replace the crossroads to improve provision for the left turn movement from Weedons Ross Road onto Jones Road (travelling towards the Izone). This will be constructed to the west of the railway line and requires land take to both the northwest and southwest of the current intersection.

This roundabout is proposed to be constructed as part of this Project.

7.2.2 Levi Road / Weedons Road

CRETS identified Levi Road as the main access into Rolleston from the north (Christchurch) via MSRFL and the Weedons Road interchange.

Currently Levi Road forms a T intersection with Weedons Road to the west of MSR, with Weedons Road having priority. In line with the CRETS recommendation, it is proposed that traffic travelling east from MSR on Weedons Road and turning right onto Levi Road, has priority. This involves realigning Levi and Weedons Roads and a small amount of land take to the west of the existing Levi / Weedons intersection.

This intersection improvement work is proposed to be constructed as part of this Project.

7.2.3 Larcombs and Berketts Road

At present Larcombs Road and Berketts Road are priority T intersections located on the eastern side of the alignment. The scoping investigations were based on retaining the intersections, but restricting the access to left-in and out movements as a result of the proposed central median barrier on Main South Road.

This concept has been carried through; the proposed provides a segregated left turn deceleration lane to enable left turning traffic to move clear of high speed southbound traffic on MSR at both Larcombs and Berketts intersections. An associated flush painted island is also proposed to improve lane discipline on the highway approach.

For Larcombs Road, access has been further restricted to left turns in only. Initially left turn out movements were considered, but the proximity of the Weedons interchange and the associated MSRFL off-ramp does not provide sufficient separation for a left turn acceleration lane. Consideration was given to realigning the Larcombs Road approach to increase the separation to the exit ramp however the desired separation could not be achieved within the land available. Moving Larcombs Road any further north would also impact Berketts Road. The close proximity of Berketts Road will provide adequate access for traffic wanting to turn left onto MSR from Larcombs Road.

A road safety audit proposed fully closing both intersections to fulfil the function of MSR as a limited access strategic road. This was met with considerable opposition from local landowners, businesses and SDC. Therefore, the Project has maintained a degree of access to/from the side roads as outlined above.

7.2.4 Dawsons Road/ Waterholes Road

Discussions with directly affected landowners around the CSM2 / MSRFL interchange identified demand for a U-turn movement close to the interchange, as the proposed MSRFL central median barrier restricted right turns to or from adjacent properties. The original proposed layout for the CSM2/ MSRFL interchange therefore included a U-turn facility between the northbound off-ramp and southbound on-ramp.

This facility was raised as a concern in the road safety audit due to a tight radii and steep grade, which suggested considering an alternative option for providing the U-turn movement further north by installing a roundabout at the MSR intersection with Dawsons and Waterholes Road. The intersection is currently a priority controlled crossroads layout and a roundabout would also offer additional advantages by allowing safer side road access.

The inclusion of the roundabout at this location will also function as a threshold for northbound traffic approaching Templeton. In the southbound direction it will signify the transition from the built-up Templeton area into the higher speed rural environment.

The single proposed option is for a large diameter, dual lane roundabout suitable for a high speed environment. Consideration was given to a single lane roundabout however this was discounted on the basis of providing a more sustainable, long term design solution consistent with the philosophy of long term route security.

7.3 CSM2 local road crossings

The philosophy for the local roads affected by CSM2 is to maintain connectivity wherever possible, but also to reduce accessibility to CSM2 as per its motorway classification and the approach used in CSM1.

The proposed CSM2 alignment crosses the local roads listed below.

- Robinsons/ Curraghs Road
- Waterholes Road
- Trents Road
- Blakes Road

- Marshs Road
- Springs Road
- Halswell Junction Road
- John Patterson Drive.

With the exception of Robinsons/ Curraghs Road and Blakes Road, it is proposed to construct underpasses at each location to carry the local road over the new motorway. The following sections outline the approach at each local road.

7.3.1 Robinsons/ Curraghs Road

Robinsons Road and Curraghs Road form a priority crossroads intersection with SH1 on the southern side of the proposed connection between CSM2 and MSRFL. Both are local roads running between Ellesmere Road and SH73.

Given the close proximity to the CSM2 and MSR interchange, the initial option was to partially close the intersection with consideration to restricting movements to left-hand turns in and out. However, there were safety implications identified with this option related to the design of adequate merge and diverge areas so close to the interchange.

The first round of consultation raised local road connectivity concerns as a key issue among the directly affected land owners and community, therefore a further option was therefore considered for Robinsons and Curraghs Road to pass underneath the highway on the existing alignment, thereby maintaining a local road connection at this location. This received positive feedback during the second round of consultation and so this was taken forward as the single preferred option.

An underpass (local road over the motorway) was considered for this connection; however was not possible as the proximity of the railway line to the immediate north of the MSRFL alignment and the required mainline vertical clearance meant that the grade required for the western bridge approach would have been too steep.

The stormwater design in this location has required special attention due to the depth below ground that is required to get under the mainline. Refer to Technical Report 3 for further information on this aspect of the design.

7.3.2 Waterholes Road

Waterholes Road is a local road in the Selwyn network providing a link from Springston to Main South Road at Templeton. The CSM2 alignment would cross Waterholes Road near its intersection with Hamptons Road. It is proposed to modify Waterholes Road with a reverse curve alignment to minimise the impact on adjacent private property and accesses. A minor realignment of the existing Waterholes Road/ Hamptons Road intersection is also required to increase the separation from the new bridge structure. This is the single option considered at this location, as illustrated below in Figure 12.

Figure 12 Visualisation of Waterholes Road underpass



7.3.3 Trents Road and Blakes Road

Trents Road is classified as a collector road in the Selwyn network and provides an important community link between Prebbleton and Templeton. An offline bridge solution⁴ to minimise impact on adjacent properties was considered however this was later discounted following the road safety audit where concerns were raised about the introduction of reverse curves on the Trents Road approaches.

CSM2 crosses Blakes Road just east of its intersection with Trents Road. Blakes Road is proposed to be closed either side of the motorway to become two cul-de-sac roads. No other alternatives have been considered, given the low traffic demand and the nearby availability of Trents Road as an alternative route. The skewed alignment across Blakes Road would also require a significant structure to keep the road open. Closing Blakes Road has received general support from the local community and has been endorsed by SDC.

This arrangement is shown below in Figure 13.

⁴ An offline bridge means that the bridge is located off the existing road alignment, meaning that the entire construction of the bridge can take place while keeping the existing road open.

Figure 13 Visualisation of Trents Road underpass and Blakes Road



7.3.4 Marshs Road, Springs Road and Halswell Junction Road

Marshs Road, Springs Road and Halswell Junction Road will all have an important function in the modified road network and it is considered important to maintain connectivity across the proposed motorway for all three roads. Marshs Road passes over the CSM2 alignment just to the east of the Shands Road interchange. An online⁵ structure can be accommodated without significant property impacts, so an offline solution was not considered at this location. The philosophy to retain the current straight alignment (i.e. an on-line bridge) was reinforced by the findings of the safety audit.

Springs Road will cross over the new motorway approximately 300 m west of Halswell Junction Road. An offline design was initially considered for the new structure to improve constructability and provide increased distance grading down the approach to the Halswell Junction Road intersection. However, this option was later eliminated following concerns raised in the road safety audit for a reverse curve alignment at this location. The single option for a straight alignment has therefore been taken forward.

Halswell Junction Road crosses CSM2 to the east of the existing Springs Road roundabout, approximately half way between this intersection and the proposed off ramp roundabout. Given the relatively close spacing of these intersections, only one option was been considered, namely an online structure in order to retain the straight horizontal alignment as it passes over the motorway. This can be seen in the earlier Halswell Junction Road interchange visualisation presented in section 7.1.4 above.

⁵ An online bridge means that the bridge is located on the existing road alignment.

7.4 Main South Road rear access roads

There are numerous property accesses located along the state highway frontage. These accesses vary from residential to commercial business. The majority of these are located on the 3.2 km section of MSR between the intersections of Weedons/ Weedons Ross Road and Robinsons/ Curraghs Road.

With the introduction of a central median to divide opposing lanes, all property accesses along MSRFL would be restricted to left-hand turns in and out. The dividing median is proposed to reduce the crash rate and crash severity from right turning movements and head on collisions which are typically higher on four lane highways.

As above, a key philosophy for MSRFL is retaining connectivity. One of the key issues raised during consultation was the effects of a left-in, left-out arrangement on property users. Under the above option, right turning vehicles would essentially have to travel to the nearest interchange and make a legal U-turn manoeuvre. The scoping stage investigations raised potential measures to mitigate the effects of this including:

- U-turn slots in the central median
- A new mid-block link perpendicular to MSR connecting to Jones Road
- Rear access roads.

There are significant safety concerns with U-turn arrangements in high speed environments and any new, centrally located link road would require a new level crossing of the rail line and introduce a new intersection to the highway. KiwiRail was not in favour of creating a new rail crossing due to potential safety risks. On this basis these options were discounted and the Project has moved forward based on the provision of rear access roads.

Layouts for access roads on the western and eastern sides of the MSR alignment are presented in Figure 14.

Figure 14 MSR rear access roads



The proposed Western rear access road provides a continuous road between Weedons Ross Road and 250 m north of Curraghs Road, located between the rear of the properties and the railway line. The

proposed Eastern rear access road utilises existing local roads, private right of ways and a proposed extension of Berketts Drive.

Consultation to confirm these routes with adjacent land owners is on-going and has received positive feedback from land owners, KiwiRail and SDC. The land required for the western rear access road and Berketts Drive and extension is included in the designation and land requirement drawings. While the existing section of Berketts Drive is currently privately owned, the proposal is to purchase the road and upgrade it as required to SDC standards as part of the Project, then vest the road back to SDC ownership at the completion of the works.

7.5 John Paterson Drive

John Paterson Drive currently forms a priority T intersection with Springs Road approximately 300 m south of Halswell Junction Road. This is at the location of the proposed CSM2/ Springs Road underpass where Springs Road will pass over the new motorway.

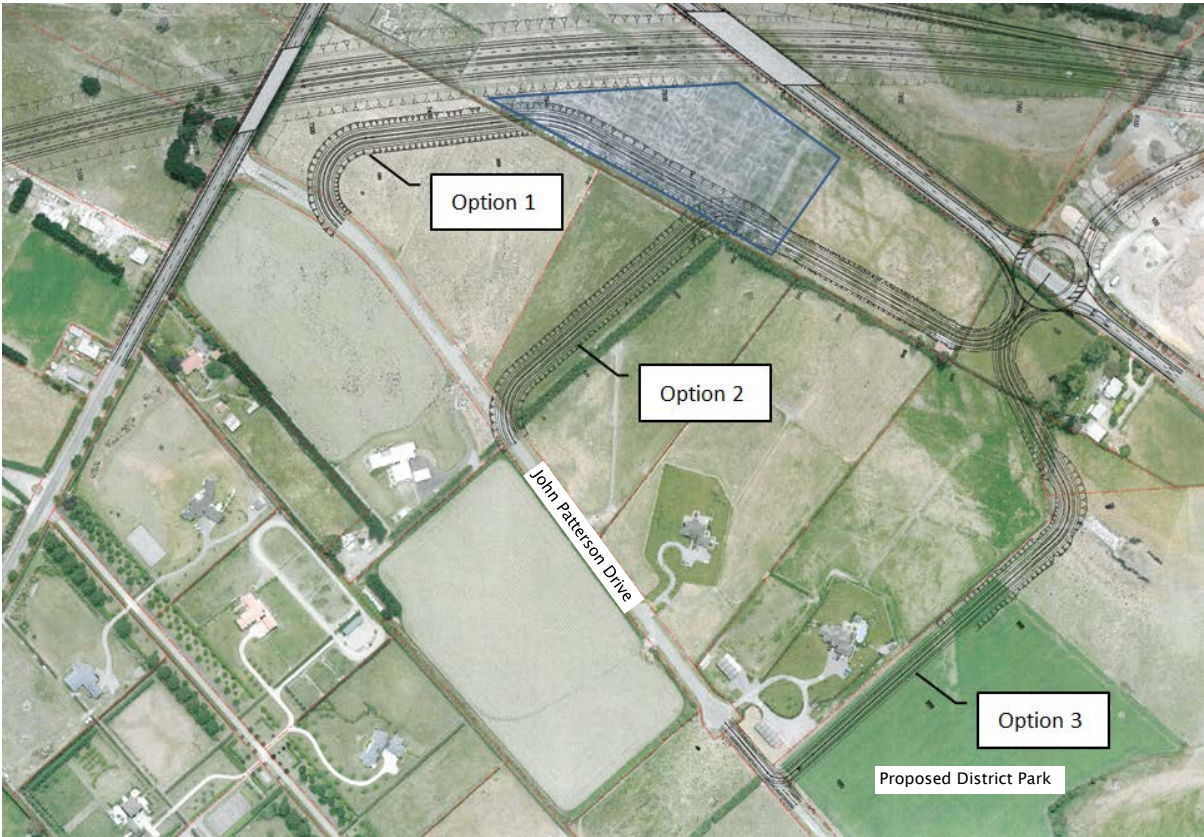
Due to the approach embankment required for the Springs Road underpass, the existing John Patterson Drive intersection would either need to be raised, or relocated to avoid the embankment. Raising the intersection increases the effects of the road on nearby residents, as well as introducing visibility issues due to the curvature of the bridge. Relocating the intersection on Springs Road to avoid the embankment would impact directly on an existing house to the east of Springs Road. As these two options were not favoured, additional options were considered to realign John Patterson Drive to connect with Halswell Junction Road instead of Springs Road.

Several options were considered for this alternative connection as presented in Figure 15. The preferred option (Option 3 below) extends John Paterson Drive east before running north along the future district park boundary identified in the adjacent Fulton Hogan plan change (refer to Chapter 6 of the AEE in relation to context). The road would then turn back west to tie into the off-ramp roundabout.

This alignment has been chosen as:

- It will provide good access to the proposed district park and residents of John Paterson Drive
- It will allow CCC to provide parking along the park edge
- The alignment avoids an area of land adjacent to the motorway that has been identified for stormwater treatment which would otherwise be affected if Options 1 and 2 were to proceed
- The alignment allows for future connectivity to the Fulton Hogan subdivision immediately to the east, although it is noted that the subdivision is not reliant on this link being provided
- It has the shortest length
- The option has the endorsement of the CCC as it supports plans to develop public access to the proposed district park.

Figure 15 John Paterson Drive options



8. Other Design Features

8.1 Walking and Cycling Facilities

Walking and cycling facilities have been designed with the philosophy of retaining safe and attractive connections for all road users across the MSRFL and CSM2 alignments. Walking and cycling along the alignment is prohibited on CSM2 due to its classification as a motorway, however as an expressway walking and cycling is allowed on MSRFL as discussed further below.

8.1.1 MSRFL

Several options were considered to provide safe, comfortable, direct and attractive facilities for pedestrians and cyclists travelling along or across the MSRFL corridor. These included:

- Using the sealed shoulder along the state highway
- Using Jones Road
- Using the proposed rear access road on the western side of MSR.

The first option was to enable access along the MSR corridor for cyclists. The number of access points along the corridor will be virtually eliminated as a result of the proposed MSRFL scheme, with only Larcombs Road and Berketts Road intersecting this section of the route. A 2.5 m wide sealed shoulder is provided along the extent of MSR and this would provide a coherent and direct route for cycle trips.

The second alternative was to provide a cycle route along the western rear access road. The lower speed environment and reduced traffic flow would make this attractive for less confident cyclists. However, by itself, it does not provide a coherent, safe or direct route option and still requires the use of either MSR or Jones Road north of Curraghs Road and south of Weedons Ross Road.

The third alternative was for the development of a cycle route using Jones Road, which is identified in the SDC Walking and Cycling Strategy. This could be in the form of widened seal on the carriageway. Relative to the rear access road, this is a less desirable option due to the higher speed environment (100 km/hr posted speed) and higher volume of through traffic. However, relative to MSR this route would be considerably more comfortable and attractive to some cyclists due to the significantly reduced traffic flows, but would not be as direct or intuitive as MSR.

The NZTA's preferred solution is to allow cyclists to use the sealed shoulder along MSR, with the proposed rear access road and existing Jones Road identified as secondary routes for those who choose to use them.

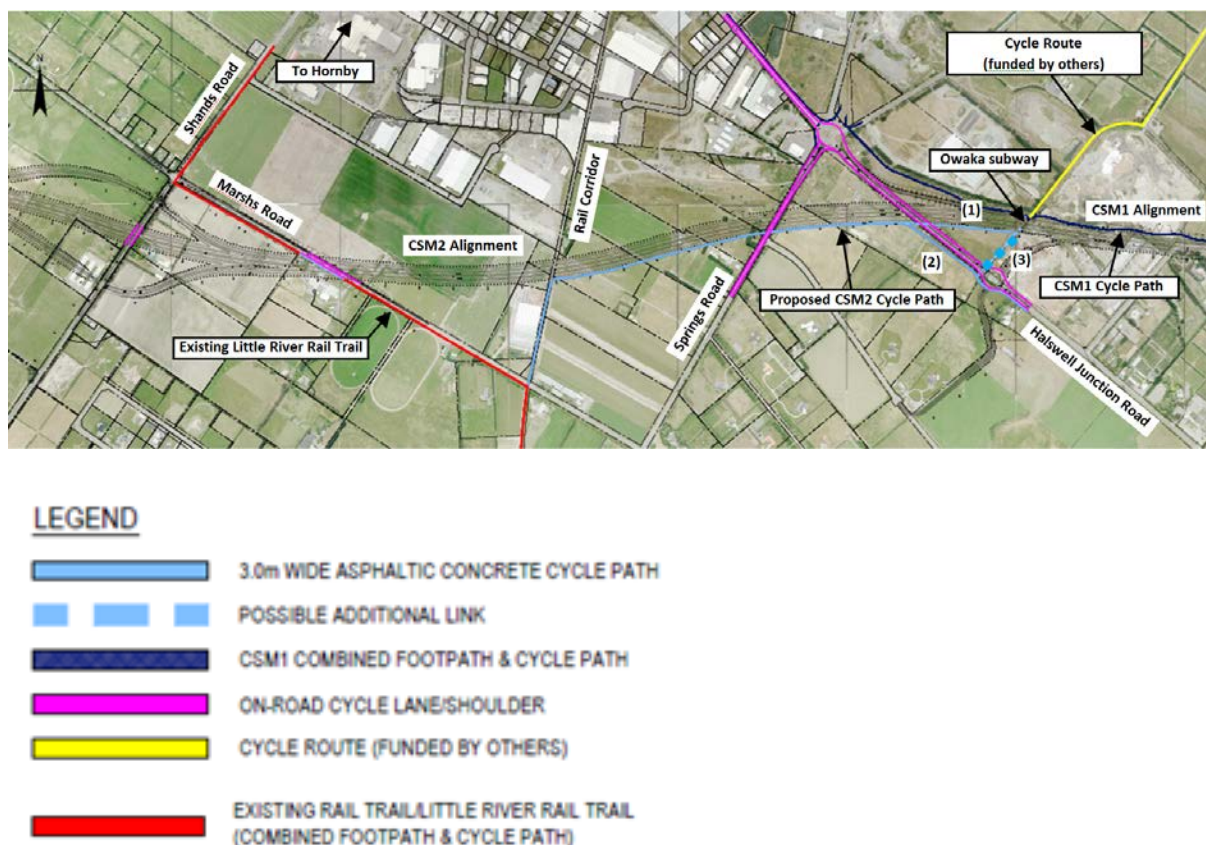
Pedestrian provision will be more limited along MSR and pedestrians are not specifically catered for in the existing situation. However, the primary pedestrian access is catered for by using the Robinsons/ Curraghs Road and Weedons Road/ Weedons Ross Road links. There will also be the option for pedestrians to cross MSR at each of the key intersections north of CSM2 including Waterholes Road, Trents Road and Barbers Road.

8.1.2 CSM2

A concept for a shared use walking and cycling facility has been identified for the scheme. This has been developed through a series of workshops with SDC and CCC and has primarily focused on providing a link between the CSM1 shared use path currently under construction and the Little River Rail Trail.

In collaboration with the key stakeholders, the proposed option involves an extension from the CSM1 route at the Owaka subway. This is proposed to continue to the south of the CSM2 alignment, west along the route within the new designation. The path would then pass under bridges at Halswell Junction Road and Springs Road before continuing along the disused section of rail corridor to Marshs Road, whereupon it connects with the existing Little River Rail Trail. The proposed route alignment (highlighted in light blue) is shown in Figure 16.

Figure 16 CSM2 proposed shared use path alignment



A second link is also highlighted in light blue on the southern side of Halswell Junction Road to the new CSM2 west bound off-ramp roundabout to create a link with the main CSM2 cycleway discussed above. In addition, a third link (highlighted in dashed light blue), from the Owaka subway to Halswell Junction is being built as part of CSM1, and will remain in place after CSM2 is completed. Access for cyclists across the new roundabout on Halswell Junction Road will be provided based on current standards.

The Marshs Road section of the Little River Rail Trail will be maintained from Shands Road and the signalised intersection associated with the Shands Road interchange will facilitate a cyclist crossing from Marshs Road to Shands Road. This maintains connectivity for cyclists to Hornby via Shands Road.

8.1.3 Provision for pedestrians and cyclists at bridge structures

Pedestrian and cycle movements will be accommodated at each of the nine local road connections across the Project alignment. This will maintain walking and cycling connectivity and safeguard for the development of any local walking and cycling facilities in the area. The current proposed design at the nine bridges is described below.

- Weedons interchange – the Weedons Road/ Weedons Ross Road bridge includes 2.0 m wide separated footpaths on both sides of the carriageway. 1.5 m wide shoulders are provided along the carriageway
- Robinsons/ Curraghs Road overpass – provides 1.5 m on-road shoulders and 2.0 m wide separated footpaths on both sides of the road carriageway
- CSM2/ MSRFL interchange – 2.5 m wide on-road shoulders provide for northbound cyclists on MSR. MSR southbound on-ramp bridge provides a 2.0 m wide footpath and 2.0 m wide shoulders along the carriageway
- Waterholes Road underpass – provides 1.5 m on-road shoulders and 2.0 m wide separated footpaths on both sides of the road carriageway
- Trents Road underpass – provides 1.5 m on-road shoulders and 2.0 m wide separated footpaths on both sides of the road carriageway
- Shands Road interchange – the Shands Road bridge includes 2.0 m wide separated footpaths on both sides of the carriageway. 1.5 m wide shoulders are provided along the carriageway
- Marshs Road underpass – provides 1.5 m on-road shoulders and 2.0 m wide separated footpaths on both sides of the road carriageway
- Springs Road underpass – provides 1.5 m on-road shoulders and 2.0 m wide separated footpaths on both sides of the road carriageway
- Halswell Junction Road underpass – provides 1.5 m on-road shoulders on both sides of the road carriageway. A 2.0 m wide footpath is provided on the northern side.

Continuity of cycle routes

The development of the Wigram Skies area (north of the CSM route at Owaka subway) will provide good pedestrian and cycle connections to local facilities. It is also anticipated that the Wigram Magdala Bridge will be constructed by CCC in the short to medium term providing an additional walking and cycling link in the area. Overall, the cycle links to and from Hornby and Christchurch City will be well connected by shared use routes and lower trafficked roads. One strategic link has been identified along Trents Road within the Selwyn District to provide access to MSR. Additional access options will be available at Waterholes Road and Barters Road. It is important to note that local roads currently are available for cycling and provide a comprehensive network on more lightly trafficked roads. This is consistent with the SDC Walking and Cycling Strategy.

8.2 Railway Crossings

8.2.1 Level Crossing

There is one road rail level crossing located within the Project – the Weedons Ross Road crossing. The road crossing has a single rail track controlled by flashing signals and barrier arms. The approach is to design the level crossing layout, marking and signage in accordance with the NZTA (Transit) Manual of Traffic Signs and Markings (MOTSAM) and Traffic Control Devices Manual Part 9: Level Crossings. This will be a matter of detailed design to be finalised in consultation with KiwiRail.

8.2.2 Hornby rail siding

The Hornby Industrial Line runs from the Main Trunk Line at Carmen Road heading in a southerly direction across Halswell Junction Road to just north of Marshs Road. The proposed CSM2 alignment passes across the southern end of this rail line, which is currently used for shunting trains into the Watties factory located north of Sir James Wattie Drive (accessed off Shands Road). To enable CSM2 to remain at-grade while continuing to cater for the shunting of carriages into Watties, it is proposed to realign the rail tracks.

Two options were considered a western turnout and an eastern turnout, which were presented to KiwiRail in February 2011. KiwiRail confirmed that both options appeared feasible and that the turnout to the west would cost less, but the eastern turnout may provide more development opportunities. The eastern turnout was preferred as it allowed the CSM2 alignment to remain further north increasing the separation to the Aberdeen subdivision and had less impact on the industrial land situated to the west. On this basis, the eastern turnout has been taken forward.

KiwiRail has advised that it has no current intention of extending the rail line further south for future commuter rail or similar. However, it has stated that the rail corridor would not be sold and that it would expect a Deed of Grant would be required for the NZTA to pass across the corridor. If in the future a rail extension to Prebbleton was justified, any associated upgrade works for the rail to pass across CSM2 would be undertaken at the NZTA's cost. Subsequent discussions with KiwiRail have indicated that the NZTA purchase of the affected section of rail corridor is an alternative option that can be progressed. Discussions on the exact mechanism are still occurring.

8.3 Commercial vehicle inspection unit facility

There has been a request from the NZ Police to include a Commercial Vehicle Inspection Unit (CVIU) within the length of the CSM2 Project.

Currently there is only one permanent weigh station in Canterbury located on SH1 in Glasnevin (south of the Waipara River) and one weigh-in-motion site at the Waipara River Bridge. There are other sites set up throughout Canterbury, however these consist of a concrete pit adjacent to the road and require the CVIU to supply portable scales.

Weigh stations serve not only the state highway network, but also the local roads and are generally provided where the route choice is limited. Two areas in Christchurch have been identified as being suitable for a CVIU facility also tying in with current RoNS construction projects. They are:

- north of Belfast, associated with either the Western Belfast Bypass or Northern Arterial RoNS projects
- in the Hornby area associated with the CSM2 RoNS Project.

The CSM2 weigh station would include a weigh bridge catering for 25 m long vehicles with room for parking, inspection and unloading and would also allow for drink driver testing. It is desirable that the facility is able to capture both northbound and southbound movements, with the potential need for associated variable message board signage to direct traffic.

Initially an area just south of Halswell Junction Road on the NZTA land was identified for a CVIU facility. However, this land is now required for stormwater purposes. The revised indicative location is within the area of land east of Shands Road between Marshs Road and the Shands Road interchange city bound on ramp, shown on Figure 17. This parcel of land is proposed for total purchase.

Further liaison between the NZ Police and the NZTA is required to confirm location, funding and operational details of this CVIU facility. The CVIU facility is not proposed for construction as part of this Project.

Figure 17 Proposed Indicative CVIU location



8.4 Street Lighting

The lighting philosophy on this Project is to ensure that the proposed motorway is safely lit for the users and that the lighting is designed to manage the effects of that lighting on the surrounding environment.

The Project interchanges, junctions and intersection(s) will be lit with appropriate highway lighting designed to the NZTA standards. Lighting will be designed as part of the detailed design stage and is addressed in Technical Report 19.

As the Project is in a semi-rural environment, full lighting of the motorway and MSR is not proposed. Throughout the alignment, the minimum gap in lighting is 300 m. In some sections, no lighting is

required to meet the relevant standards, for example on MSRFL, from 150 metres south of Larcombs Road to 450 metres south of Robinsons Road and CSM2 from 500 metres west of Waterholes Road to the start of the Shands road interchange. The relevant standard will generally be applied to all lengths of the alignment and all connections, underpasses and interchanges. The following specific sections of the proposed design are noted:

- CSM2: Waterholes and Hamptons Road intersections – isolated lit section
- CSM2: Hamptons and Trents overbridges – no lighting required however ducting will be installed for future use
- MSRFL: Berketts and Larcombs intersections – intersection flag lighting will be used.

Lighting of adjoining local CCC and SDC roads will also be carried out as required. Refer to the Technical Report 19 for further details.

Cycleways are proposed to be lit to comply with the relevant standard as are all underpasses.

8.5 General Civil Components

8.5.1 Traffic Signals

There are currently no existing traffic signal facilities within the area of CSM2 and MSRFL and the philosophy of design was to maintain this approach given the preference of SDC not to have traffic signals to retain the rural environment (preferring roundabouts).

This is achieved in the MSRFL section, however the CSM2 section includes the design of three signalised intersections as part of the Shands Road interchange one at the existing Shands Road / Marshs Road intersection, one at the proposed Shands Rd / eastbound off Ramp / eastbound on ramp intersection and one further south on Shands Road at the proposed intersection of Shands Rd / westbound off Ramp / westbound on ramp. Signals are preferred at Shands Road due to the limited space between the western ramp intersection and the Marshs Road intersection and the safety benefits provided for cyclists and pedestrians when compared to roundabouts.

Shands Road is currently posted at 100 km/hr up to Sir James Wattie Drive (300 m north of Marshs Road) where the posted speed reduces to 70 km/hr. Following discussion with SDC it is anticipated that the 100 km/hr posted speed along Shands Road will be reduced to 60 km/hr in the vicinity of the Shands Road interchange. This 60 km/hr speed limit will continue northwards to the 70 km/hr speed limit at Sir James Wattie Drive.

8.5.2 Kerb and Channel

Kerb and channel is not generally proposed along the mainline alignment, due to the cost associated with this work, because of the additional water treatment the roadside swales provide and because the standard kerb and channel profile is a potential safety hazard. Kerb and channels are required in some instances however generally at structures, intersections and where width is restricted.

All new kerb and channel profiles will be based on the CCC and SDC standards. New kerbs will typically be:

- semi mountable kerbs
- kerb and flat channel
- in-situ mountable median kerb, or
- 600 mm wide concrete dish channel.

The location and type of kerb profile to be used will be confirmed at detailed design stage, following finalisation of the stormwater and geometric designs.

8.5.3 Barriers

Barriers are proposed in certain locations along the alignment as per the 'safe systems' approach discussed in section 6.7. Barriers are typically proposed at and around structures and in the central median of both MSRFL and CSM2.

All new barriers will typically be:

- Test Level 4 (TL4) wire rope barrier (main alignment central median)
- TL4 nu-guard W-section roadside barrier (bridges and immediate approaches) or
- TL4 concrete 'F-Shape' edge barrier (bridges and immediate approaches).

The location and type of barrier to be used will be confirmed at detailed design stage, where features including emergency cross over points in the central median will be considered.

8.5.4 Signage

Key motorway signage locations have been identified and are illustrated on the signage plans (62236-A-C501 to C509 and 62236-B-C501 to C517) contained within the Plan set. Local road signage (e.g. give way and stop signs) will be detailed at the next design stage. Given the adoption of the safe system approach discussed earlier, breakaway sign support systems will be included where necessary.

8.5.5 Vehicle Tracking and Over-Dimensional Route

Tracking paths have been undertaken on heavy vehicle turning movements using AutoTURN to check there is adequate room provided. A minimum 600 mm clearance has been allowed for, in addition to the tracking path to cater for driver error or misjudgement. The design vehicle is the RTS 18 m long quad rear axle semi-trailer as this provides the worst case tracking path out of the heavy vehicle group.

MSRFL and CSM2 are 'over dimensional' and 'Overweight Permit' routes. Through the investigation of options, the required 10.5 m wide x 6.1 m high over dimensional envelope has been allowed for. Median island traffic signal poles (and potentially overhead masts) may need to be collapsible to allow for continuity of the 'over dimensional' route. This will be investigated further at detailed design stage.

8.6 Utility and Public Services

Up to date location plans and records of known utilities have been obtained for inclusion in the assessment phase of the works. Each utility provider has been consulted with to understand the various requirements and limitations in the local area that may impact relocation or protection of the services. The necessary written approvals and agreements to enable the works will need to be obtained at a future stage, where required.

8.6.1 Transpower

The proposed CSM2 alignment passes under Transpower's Islington to Springston 66 kV transmission lines to the southwest of the Shands Road and Marshs Road intersection. The alignment falls within the transmission line clearance envelope and Transpower has confirmed that modification of this line is necessary to achieve the required clearance standards. The preferred option is to raise one, possibly two, of the existing towers, one of which may also require minor relocation to achieve lateral clearances.

The preferred solution will be identified when the Project advances to detailed design and it is recommended that the modifications to these 66 kV lines are undertaken prior to the construction of CSM2. This will allow the contractor a clearer and safer working space during the construction of the CSM2/ Shands Road interchange.

The alignment also passes underneath the 220 kv Bromley to Islington line to the east of the Shands Road interchange. Transpower has confirmed that there are no clearance or proximity issues for this line.

8.6.2 Other Existing services

A mixture of electronic and hard copy drawings were received from the relevant service authorities. Table 5 and Table 6 summarise the services within the MSRFL and CSM2 sections. It should be noted that no depths were provided with the service plans received.

Table 5 MSRFL existing services

| Location | Orion (overhead) | TelstraClear | Water Races |
|-------------------|---|---|----------------------------------|
| Main South Road | Eastern side crossing the road periodically | Western side | Eastern side (from Weedons Road) |
| Jones Road | Western side | Eastern side (south of Weedons Ross Rd) | |
| Weedons Ross Road | Both sides | Southern side (south of Jones Rd) | Northern side |
| Weedons Road | Southern side | Southern side (doesn't extend to Levi Rd) | Northern side |
| Levi Road | Shown on the plans but not observed on site | - | - |

In addition to the services within the MSRFL section presented in Table 5, there is a Chorus cable running parallel to MSR to the east of the railway reserve from around 500 m south of Robinsons Road to around 300 m north of Robinsons Road.

Table 6 CSM2 existing services

| Location | Orion (overhead) | Chorus | Water Supply | Water Races |
|---|--|---|--|--|
| Robinsons/ Curraghs Road | Western side and an additional line crossing CSM2 east of Robinsons Rd | North of Robinsons Rd | - | Western side and north of Robinsons Rd |
| Waterholes/ Hamptons Road | Both sides | - | - | Eastern side |
| Trents Road | Western side | West of Trents Rd and crosses Trents Rd to the north of CSM2 | - | Eastern side and west of Trents Rd |
| Blakes Road | Eastern side | - | - | Western side |
| Shands Road | Western side | - | - | - |
| Marshs Road | Northern side | South of Marshs Rd through the CSM2/ Shands interchange. Crosses Marshs Rd and CSM2 east of their intersection. | Runs around the NW corner of Marshs/ Shands intersection | Northern side |
| Railway Corridor | Eastern side | - | - | - |
| Springs Road | Western side | - | Both sides | Western side |
| John Patterson Drive (existing alignment) | Northern side | - | Northern side | - |
| Halswell Junction Rd | Eastern side | - | Western side | Western side |

In addition to the services within the CSM2 section presented in Table 6, there are:

- Overhead Orion lines and TelstraClear services run along MSR, on the eastern and western sides respectively (further north from the MSRFL section)
- Two lines of Transpower pylons intersect just north of the proposed Shands Road/ Marshs Road interchange
- Sewer pipes on the eastern side of Shands Road and the western side of Springs Road.

9. Geotechnical Investigations

The philosophy of approach for the geotechnical investigations has focussed on route security with the need to provide stable embankment, foundations and bridge structures in the event of seismic activity, while providing cost effective solutions to construction form and foundations.

In general it is considered there are limited geological hazards along the alignment, other than seismicity and liquefaction. The design and construction of the Project from a geological perspective is relatively straight forward due to the flat topography and relatively homogeneous geology of granular alluvial soils.

Further details relating to the geotechnical investigations and geo-hazards are included as part of Technical Report 11 (Geotechnical Engineering and Geo-hazards Assessment).

9.1 Seismicity

The Project alignment is south of the Marlborough fault zone and east of the Alpine fault. While a significant aftershock associated with the recent earthquakes centred around Christchurch (2010 to 2012) pose the highest current risk, the Marlborough fault zone and the Alpine faults are also capable of generating significant seismic activity in the Project area. In the recent Christchurch earthquakes, ground accelerations of 0.2 g and movement of up to 900 mm horizontally and 320 mm vertically were recorded in the project area.

Given the flat topography of the site earthquake induced landslides are not a risk to the Project.

Despite extensive liquefaction in Christchurch's eastern suburbs, no liquefaction was recorded in the project area. However, the interpretation of the investigation data has proved that liquefiable soils do exist at depth and will require consideration at the detailed design stage in terms of impact to foundations and structures.

9.2 MSRFL

The overall site for the proposed MSRFL comprises flat alluvial plains underlain by two distinct geological units comprising discrete areas of shallow loose sands and soft to firm silts typically up to 2.0 m thick overlying medium dense to very dense sandy and silty gravels. Recorded groundwater levels are generally in the range of 14 – 16 m below ground level.

The approach embankment assessments are based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

The proposed road stormwater management comprises swales and soakaway drainage. The permeability testing and general ground conditions allow the use of the proposed swales and soakaway drainage provided stormwater infiltration zones should be located within the high permeability sand and gravel soils.

In-situ and laboratory CBR (California bearing ratio – a penetration test that evaluates the mechanical strength of road subgrades and base courses) testing on natural soils at approximate proposed subgrade level returned values of 1.5% to > 30%. Design CBRs of 4% and 3.7% were determined for the pavement greenfield and widening sections respectively.

9.3 CSM2

The overall site for the proposed Christchurch Southern Motorway comprises flat alluvial plains underlain by two distinct geological units comprising discrete areas of shallow loose sands and soft to firm silts up to 3.5 m thick overlying medium dense to very dense sandy and silty gravels. Recorded groundwater levels are generally in the range of 6 m – 15 m below ground level, becoming deeper towards the south west and away from Christchurch.

The approach embankment assessments for all the intersections are based on well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

The proposed road stormwater management comprises swales and soakaway drainage. The permeability testing and general ground conditions allow the use of the proposed swales and soakaway drainage provided stormwater infiltration zones are located within the high permeability sand and gravel soils.

In-situ and laboratory CBR testing on natural soils at approximate proposed subgrade level returned values of 1.5% to > 30%. A design CBR of 4% was determined to inform the pavement design.

10. Pavement Design and Surfacing

The pavement design philosophy has been based on meeting the Austroads requirement for a modified aggregate base to reduce the risk of rutting, provide a safe and quiet pavement surface and acknowledge the RoNS standards. Consistency with the CSM1 project has also been acknowledged in the design.

The design for the mainline is presented below. Works to the local roads have not had a pavement designed carried out as part of the scheme design phase and are not covered in this section. The design for the local roads will be based on the relevant SDC and CCC standards and will be detailed in the next phase of the design.

MSRFL and CSM2 pavement design are treated separately below.

The pavement design is based on inferred design CBR values. Any softer spots located along the route will need to be dug out further and the subgrade improved by replacing with additional granular material.

10.1 Traffic calculations

Traffic calculations are discussed in detail in Technical Report 2 – Assessment Traffic and Transportation Effects. These are summarised below as a basis for the pavement design.

10.1.1 MSRFL

The two-way average annual daily traffic (AADT) for MSRFL is forecast to be 26,750 in 2016 West of Robinsons Road and Curraghs Road and 27,000 West of Weedons Road and Weedons Ross Road. The higher AADT has been used in the design traffic calculation.

10.1.2 CSM2

Modelling data indicates that the two-way AADT for CSM2 is forecast to be 16,000 between MSR and the Shands interchange and 19,750 in 2016 between the Shands interchange and HJR. For pavement design purposes, CSM2 has been broken into two sections Subsection A from Halswell Junction Road to Shands Road interchange and Subsection B from Shands Road interchange to Main South Road.

10.2 MSRFL Pavement Design

While some segments of MSR already have four lanes, other segments require widening of the existing state highway. Two designs are outlined below – one for the widening section where completely new pavement is required and one for the section where the existing road requires strengthening.

10.2.1 MSRFL Widening Design

Where road widening is required the pavement design presented in Table 7 has sufficient capacity for the design traffic and the observed subgrade conditions. Pavement surfacing of either OGPA or SMA is assumed.

Table 7 Pavement design for road widening of State Highway 1

| Layer | Depth (mm) |
|--|------------|
| OGPA or SMA | 35 |
| Grade 4 chipseal membrane | – |
| Foamed bitumen stabilised NZTA modified M/4 AP40 | 180 |
| CCC AP65 sub-base | 380 |

10.2.2 MSRFL Strengthening

Modelling using Circlly (pavement design software) incorporating the test pit thicknesses and the Falling Weight Deflectometer⁶ (FWD) analysis both indicate that a number of sections in the existing pavement have insufficient strength for the design traffic. Therefore, granular overlays are required in these sections. An appropriate solution is the application of granular overlay to produce a total constructed granular thickness of 560 mm, followed by foamed bitumen stabilised to a depth of 180 mm. Table 8 illustrates the maximum overlay required where the existing pavement is thinnest (being 180 mm thick). The overlay design is effectively modelled exactly the same as the pavement design for road widening. A pavement surfacing of either OGPA or SMA is assumed.

Table 8 Pavement design for overlay of State Highway 1

| Layer | Depth (mm) |
|--|---------------|
| OGPA or SMA | 35 |
| Grade 4 chipseal membrane | – |
| Overlay foamed bitumen stabilised NZTA modified M/4 AP40 | 180 |
| Overlay CCC AP65 sub-base | 200 |
| Existing pavement | 180 (minimum) |

⁶ FWD - a testing device used to evaluate the physical properties of an existing pavement

Based on the pavement thicknesses observed in the test pits and homogeneous FWD results, the required overlay and stabilisation treatment requirements are given in Table 9 where the presumed existing pavement thicknesses are also tabulated. In all cases, the granular basecourse is to be stabilised to a depth of 180 mm.

Table 9 MSRFL pavement strengthening requirements

| | Existing pavement thickness (mm) | GAP65 Overlay thickness (mm) | M/4 Overlay thickness (mm) | Foam bitumen stabilisation depth (mm) |
|---------------------------------|----------------------------------|------------------------------|----------------------------|---------------------------------------|
| Hoskyns Road to Weedons Road | 600 | None | None | 180 |
| Weedons Road to Larcombs Road | 300 | None | 260 | 180 |
| Larcombs Road to Berketts Road | 180 | 200 | 180 | 180 |
| Berketts Road to Robinsons Road | 280 | None | 280 | 180 |

In each case the overlay material shall be a combination of a suitable sub base course material and a modified NZTA basecourse material. The proposed basecourse material will be tested prior to construction to ensure that the assumed mechanical strength shall be achieved by the foam bitumen stabilisation process.

10.3 CSM2 Pavement Design

CSM2 is a greenfields section and a suitable pavement configuration for both design traffic levels is presented in Table 10. Pavement surfacing of either OGPA or SMA is assumed.

Table 10 Pavement design for CSM2 Subsection A and Subsection B

| Layer | Depth (mm) |
|--|------------|
| OGPA or SMA | 35 |
| Grade 4 chipseal membrane | - |
| Foamed bitumen stabilised NZTA modified M/4 AP40 | 180 |
| Sub-base | 350 |

10.4 Pavement Surfacing

On the basis of the design philosophy stated above, OGPA and SMA have been chosen as the surfacing materials for the mainline, as per the surfacing drawings (62236-A-C250 to 253 and 62236-B-C250

to 255) contained within the Plan set in Volume 5, which should be referred to for details of the extent of the different types of surfacing.

Net present value calculations for the 25 year design life of the pavement (which excludes the milling costs) show that the chipsealing is the cheapest option. However, these calculations do not include the cost of traffic disruptions of each treatment; the chipsealing option requires five treatments over the 25 year design life while the SMA requires two treatments and the OGPA requires three. Furthermore, the noise reductions of the OGPA and SMA surfaces make these options attractive (OGPA is 0.8 dB quieter than SMA and 2.3 dB quieter than chipseal).

Given the lower noise impact of the OGPA and SMA surfacing as well as the increased lives (and consequent reduction in traffic disruption relative to chipseal), OGPA and SMA surfacing is recommended for use on the CSM2 and MSRFL mainline.

OGPA has been chosen as the predominant pavement type due to its superior noise reduction properties relative to SMA. It also has positive effects on the reduction of surface water on the road. SMA is only utilised in areas of high braking, typically at the approach to intersections, on corners and underpasses as it is more durable and provides more friction than OGPA. This approach has been taken on CSM1 hence the design proposed above maintains continuity.

For the local roads, the use of chipseal is recommended except where noise impacts need to be mitigated or the SMA is required to provide a superior surfacing in high stress situations such as approaches to intersections, overpasses and on and off ramps.

11. Stormwater Drainage Design

The stormwater design is a significant part of the Project design and can be split into five sections.

1. Groundwater analysis
2. Discharge of surface water runoff
3. Stormwater treatment
4. Stockwater races
5. Overland flow paths.

The philosophy behind the approach to these sections has been to achieve the standards for water quality and disposal, to ensure the route security from a flooding perspective and to retain connectivity for stockwater races and overland flow. Construction and operation costs have also been taken into account. The stormwater design has been driven by the flat nature of the site, the groundwater levels and the lack of natural water courses in the area as well as political and physical constraints such as ECan's NRRP and CCC's South West Area Plan infrastructure.

Significant further discussion on stormwater and drainage is included in Assessment of Stormwater Disposal and Water Quality (Technical Report 3 and in Chapters 19 and 20 of the AEE).

11.1 Groundwater Analysis

GHD has investigated the groundwater levels along the alignment of MSRFL and CSM2 to establish a design groundwater level.

Existing ECan and Project specific bores were monitored throughout the design phase and this information together with a review of historical records allowed the determination of design groundwater levels. The design groundwater levels have then been modified to take account of the Central Plains Water Scheme and the effects of proposed changes to the CCC stormwater network at the HJR end of the Project.

This analysis allowed the determination of a design groundwater level which has informed:

- the stormwater disposal design: a design objective of the Project is to ensure the effective disposal of stormwater runoff while achieving the 1 m clearance between the disposal system and the design groundwater level as specified in the NRRP
- the vertical geometry of the Project: the design groundwater level was one of the key reasons the CSM2 section could not be set in a trench.

11.2 Discharge of surface water runoff

Options considered for conveyance and discharge of runoff have included kerb and channelling with sumps and piped conveyance system, grassed swales and steep sided open channels. Given the rural environment, the availability of land and water quality objectives, swales were selected as the

preferred option. Swales are a low cost and effective treatment solution. They also provide storage of stormwater prior to discharge to land. Steep sided open channels have only been used in a limited capacity due to road safety considerations and kerb and channels were limited to short lengths typically around interchanges due to the cost.

Kerb and channelling has been proposed on ramps and structures and around Halswell Junction Road due to the limited depth between the edge of seal and historical high groundwater levels. Typically run-off from structures will drain to swales to provide treatment prior to disposal.

Disposal to land via soakpits and soakage areas was considered the only viable option for disposing of stormwater due to the flat topography and absence of natural water courses in the vicinity of the Project (other than at the Halswell Junction Road end). The nearest natural water course to the south is the Selwyn River and to the east is the Halswell River and options such as constructing a piped discharge network to dispose of stormwater to these rivers was discounted early in the design process due to the significant costs associated with such a system.

Due to the lesser depth to groundwater in the vicinity of Halswell Junction Road (less than 6 metres), treatment is required prior to discharge to land hence a series of treatment ponds are proposed. Halswell Junction Road runs parallel to Montgomery's Drain and the overflow point of these ponds is into this drain which connects to the Upper Knights Stream.

A pond is also proposed opposite Berketts Road, with water piped from the eastern side of the road to the pond on the western side. This is necessitated by the inclusion of the acceleration and deceleration lanes at Berketts Road which mean there is no longer the cross sectional width required for swales and soakpits.

Pumping options were only considered in very rare instances where other solutions were not possible, such as at Robinsons Road/ Curraghs Road. Conveying runoff to ponds and dedicated larger disposal fields was considered but rejected due to having to designate and purchase larger blocks of land to accommodate these features.

The preferred option for regular soakage devices was selected due to increased redundancy and reduced land take.

11.3 Stormwater treatment

Generally collection and treatment in swales was considered an acceptable methodology as this is used throughout the region and discharge to land solely from a road without treatment is permitted under the NRRP. Increased treatment of stormwater is required by the NRRP in areas mapped by ECan as less than 6 m to groundwater – specifically the area east of Springs Road relative to this Project. In the areas where this increased treatment is required, first flush basins with organic filter media are also proposed prior to discharge to land. The methods set out in the NRRP are prescriptive to achieve a permitted activity status. Almost the entire project is compliant with the NRRP rules regarding stormwater treatment and disposal. The exceptions to the rule are being addressed as part of the AEE which is being prepared to support the resource consent application.

Rule WQL6 of the NRRP requires a 1 m buffer between the disposal elevation and historical groundwater maximas, which provides some treatment within the sub-strata prior to meeting

groundwater. It is not possible to maintain this buffer in all areas without raising the road, which introduces increased adverse environmental effects and increased costs.

Alternative treatment systems using proprietary stormwater treatment devices have not been considered due to high cost and high maintenance requirements. Consideration was given to concentrating stormwater to storage/ attenuation ponds prior to discharge to land along the entire route but was discounted due to increased land take.

Overall, the proposed dispersed treatment and disposal system was considered to have the least risk and lowest environmental impact.

11.4 Stockwater races

Nine existing stock-water races cross the proposed alignment. A range of considerations were examined including: closure, part closure, pumping and realignment. Overall the function of the race network needed to be maintained thus wider closures were not considered. The stockwater races also have a dual function of providing land drainage during heavy rain and providing environmental flows to the Halswell River.

Therefore stockwater race siphons have been designed to carry the flow under the Project alignment, maintaining connectivity of the network. Alterations and closures to some stockwater races are required as part of the Project, however these have been minimised as per the approach outlined above.

11.5 Overland flow paths

The Project crosses approximately 12 overland flow paths (OLFPs) in addition to the stockwater race flow paths set out above. Along MSRFL the OLFPs do not typically pass through/ beneath SH1 currently and have the potential to flood the upstream landowners. Protection of the drainage system via earth bunds and shallow timber flood walls was preferred over reshaping of the existing land and wider flood bunds due to land constraints.

For CSM2, options investigated to deal with these included:

- Ignoring the OLFPs in terms of the drainage design: this relies on natural soakage to ground upstream of the alignment, increasing the risk of flooding to property upstream and the alignment itself
- Allowing overland flow to enter the Project drainage swales: this option carried the risk that the Project swales would become overloaded resulting in flooding of the alignment
- Storing the overland flow: this option would require large increases in the designation area to provide the required area to store the necessary volumes
- Passing it beneath the alignment via siphon arrangements: this is the preferred solution as it limits the environmental impacts described above and reduces the design disposal and storage footprint.

12. Structures

12.1 Introduction

The preferred option has a number of bridge structures on its alignment where the Project crosses local roads requiring the motorway to be carried over the local road or the local road over the motorway. The nine major bridges required for this Project are:

- Weedons Road Underpass
- Robinsons Road Overpass
- SH1 Southbound On-Ramp Bridge
- Waterholes Road Underpass
- Trents Road Underpass
- Shands Road Underpass
- Marshes Road Underpass
- Springs Road Underpass and
- Halswell Junction Road Underpass.

The individual bridges are described below and for each bridge the factors influencing the design are outlined, together with the options considered and the recommended option. Overall the design philosophy has considered construction and operational cost, route security through design to the appropriate standards and visual aspects of the bridges providing consistency in approach where possible (both within the Project and with CSM1). Detail regarding the foundation options for each structure is contained within Appendix A.

As an example, Figure 18 below shows a bridge visual produced by for the CSM1 project, illustrating a treatment option for the abutments and embankments. The treatment options for the bridges described below will be finalised during detailed design. For further details on the appearance of the bridges please refer to Technical Report 6 – Urban and landscape design framework.

Figure 18 Example visualisation from CSM1



It is recognised that bridge design is also important in a number of other reports, including:

- Technical Report 4 – Landscape and visual effects
- Technical Report 5 – Assessment of effects – urban design
- Technical Report 6 – Urban and landscape design framework
- Technical Report 7 – Landscape Context Report
- Technical Report 11 – Geotechnical engineering and geo-hazards assessment
- Technical Report 13 – Social impact assessment.

This section focuses on the engineering and design issues that are relevant.

12.2 Design Standards

The Transit New Zealand Bridge Manual (TNZBM) Second edition 2003 and the material design standards specified therein define the general design criteria to be adopted for the structures. This includes the June and September 2004 amendments and the Provisional Amendment dated December 2004.

The Roads of National Significance (RoNS) Design Standards and Guidelines include a number of bridge related requirements. These include:

- Use of specific concrete edge barriers on bridges
- Full width 3.0 m outer shoulders to be taken over full bridge length

Where the Transit Bridge Manual does not address the specific requirements the appropriate Australian or UK bridge design standards are to be referenced. The key design criteria are shown in Table 11.

Table 11 Design Criteria

| Design Criteria | Standard Proposed |
|--|---|
| Vertical clearance at bridges over local roads (overpass) | 6.1 m |
| Vertical clearance at bridges over motorway (underpass) | 6.1 m |
| Shoulder widths for bridges that carry the motorway (overpass) | 3.0 m outer shoulders 1.5 m inner shoulders |
| Longitudinal span for CSM2 over Robinsons Road | 21.0 m clear span to suit local road, footways and cyclists at Robinsons Road Overpass |
| Footway/shoulder/lane widths on underpasses | 2.0 m footways 10.0 m total width between barriers (1.5 m shoulders with 2 x 3.5 m lanes) except: 17.3 m total width between barriers for Shands Road Underpass with 1.5 m outer shoulders, 4 x 3.5 m traffic lanes and 0.3 m median. 20.0 m total width for Halswell Junction Road Underpass with 4 x 3.5 m lanes, 1.8 m shoulders and 2.4 m median |
| Edge barriers | TL 4 flexible barriers for Robinsons Road Overpass TL4 concrete barriers elsewhere Pedestrian handrails on footways |

12.3 Weedons Road Underpass

12.3.1 Description of bridge

The Weedons Road Underpass is the only bridge structure which spans MSRFL. The structure links Weedons Road to Weedons Ross Road. The bridge is not directly connected to MSR, but the approach roads to the structure link back into the motorway on-ramps and off-ramps.

12.3.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two 3.5 m traffic lanes with 1.5 m wide shoulders for cyclists and has 2.0 m wide footways on each side.

Geometrics

The bridge carrying Weedons Road is on straight horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

Weedons Road has a small skew to Main South Road of about 7 degrees, requiring the bridge to also be skewed to reduce span lengths.

Foundation conditions

Ground conditions at the bridge comprise a 3 m layer of sandy silt overlying sandy gravel. The groundwater is approximately 14 to 15 m below ground level.

The liquefaction assessment has indicated liquefaction potential in layer between and 15 m and 19 m below ground, which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundations options have been considered and it is recommended that a shallow footing founded on the sandy gravel layer is adopted.

Constraints on span arrangements and clearances

The bridge is required to span the Main South Road Four Laning which at this location comprises a four lane highway with a diverging off-ramp lane on the south side and a separate on-ramp lane on the north side.

Span lengths of 22.0 m and 18.0 m are required over the main carriageway of Main South Road and a span of 24.0 m is required for the northern side span to span the on-ramp and embankment batter slope.

Constraints on construction methods and materials

The bridge is required to be constructed over the existing Main South Road with live traffic which will require a bridge form that can be erected during limited lane closures.

Suitable superstructure forms are precast hollow core and Super Tee beams and steel composite I girders for the spans required.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in Technical Report 6.

For Weedons Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footways and to provide steel handrails on the outside of the footways. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on top to give a total height of 1100 mm.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from Main South Road below and from Weedons Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.3.3 Design options considered

The two design options considered for this bridge are:

- Four span bridge with open side spans using Double Hollow Core, Super Tee beams or steel composite beams supported on (reinforced concrete (RC) piers on spread foundations and bank seats supported on the filled embankments
- Three span bridge using Double Hollow Core, Super Tee beams or steel composite beams supported on RC piers on spread footings and MSE wall abutments.

12.3.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams

- 1025 mm deep Super Tee beams with 180 mm deck slab
- 1000 mm deep steel composite beams with 250 mm thick deck slab.

12.3.5 Cost considerations

On the basis of construction cost, the option to use 900 mm deep double hollow core beams gives the lowest cost. Options that provide either three spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments.

12.3.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of these factors, the preferred option is a four span bridge with double hollow core beams supported on RC piers with spread footings and bank seats supported on the approach filled embankments. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the hollow core beams. It also satisfies the urban design requirements to provide an open bridge structure.

12.3.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle of approximately 7 degrees. The bridge is a four span bridge with internal skewed span lengths of 22 m and 18 m and end spans of 20 m and 24 m. The central pier is located in the middle of SH1 with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is 15.3 m to the outer edge of the footpath. This provides for a carriageway width of 10.0 m, 2 No. 2.0 m wide footpaths, 2 No. 0.4 m wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure consists of 12 No. 900 mm deep precast prestressed concrete double hollow core (DHC) units. An in-situ topping concrete of 180 mm minimum thickness at either edge of the deck forms the raised footpath. A surfacing course will overlay the DHC units across the width of the road carriageway.

The three piers comprise three 1000 mm diameter columns on top of 18 m x 6 m x 1.2 m thick spread footings. The pier crossheads provide seating for the DHC units. The abutments comprise a 1.8 m wide spread footing. A 250 mm thick, 3.0 m long settlement slab is provided to minimise post-construction differential settlement.

12.4 Robinsons Road Overpass

12.4.1 Description of bridge

CSM2 passes over Robinsons Road at approximate chainage 400 metres. The structure is required to carry the motorway over the local road and to allow vehicular access beneath the motorway.

12.4.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide for traffic, pedestrians and cyclists on Robinsons Road and has a clear width between abutments of 21.0 m to allow for 2 No 3.5 m traffic lanes, shoulders and footways.

The bridge carries CSM2 and provides for two traffic lanes northbound and two traffic lanes southbound plus two traffic lanes and a merge median from the southbound on ramp. It also provides for a 6.0 m median including 1.5 m shoulders and wide outer shoulders.

Geometrics

The bridge over Robinsons Road is on a curved horizontal alignment and has a straight vertical alignment. The bridge is required to be tapered on plan due to the merging on-ramp.

Robinsons Road has a small skew to the motorway of about 8 degrees, requiring the bridge to also be skewed to reduce the span length.

Foundation conditions

Ground conditions at the bridge comprise a 1 m layer of sandy silt overlying sandy gravel. The groundwater is approximately 12 to 13 m below ground level.

The liquefaction assessment has indicated no liquefaction potential in the soil profile.

Shallow footings and driven piled foundations options have been considered and it is recommended that a shallow footing founded on the sandy gravel layer is adopted.

Constraints on span arrangements and clearances

The bridge is required to span Robinsons Road which at this location comprises a two lane road with shoulders and wide verges.

A clear square span length of is required to allow for the road and verges.

Constraints on construction methods and materials

The bridge is required to be constructed over the existing Robinsons Road with live traffic which will require a bridge form that can be erected during limited road closures.

Suitable superstructure forms are precast hollow core beams, Super Tee beams and steel composite I girders for the spans required.

Earthquake design requirements

As the road is a state highway it is classified as Importance Level 3 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For Robinsons Road Overpass the use of vertical MSE walls is considered appropriate given the low traffic volumes and pedestrian use on this route and the clear width below the bridge.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide W-section TL4 barriers on the edge of the outer shoulders of the motorway.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from Main South Road below and from Robinsons Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.4.3 Design options considered

The two design options considered for this bridge are:

- Single span bridge using double hollow core, Super Tee beams or steel composite beams supported on MSE abutments
- A three-span bridge using double hollow core, super tee beams or steel composite beams supported on RC piers on spread footings and bank seats supported on filled embankments.

12.4.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams
- 1025 mm deep Super Tee beams with 180 mm deck slab and
- 1000 mm deep steel composite beams with 250 mm thick deck slab.

12.4.5 Cost considerations

On the basis of construction cost, the option to use 900 mm deep double hollow core beams gives the lowest cost.

The single span option has a lower cost than the three span option.

12.4.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of these factors, the preferred option is a single span bridge with double hollow core beams supported on MSE abutments. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the hollow core beams.

12.4.7 Description of recommended option

The proposed bridge abutments are skewed with respect to the centreline of the bridge at an angle of approximately 8 degrees. The single span bridge spans 24.0 m between the abutment centrelines. The abutments are located on top of mechanically stabilised earth walls.

The overall deck width is 45.8 m to the outer edge of the bridge beams. This provides for a carriageway width of approximately 40.0 m with TL4 W-section flexible barriers on either side of the bridge. The structure consists of 40 No. 900 mm deep precast prestressed concrete DHC units. An in-situ topping concrete of 200 mm minimum thickness at either edge of the deck forms the raised kerb slab. A surfacing course will overlay the DHC units across the width of the road carriageway.

The abutments comprise an 1800 mm wide reinforced concrete footing supported on top of vertical mechanically stabilised earth retaining walls. The footing has an upstand beam and the overall height of the abutment is 1700 mm. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.5 SH1 Southbound On-Ramp Bridge

12.5.1 Description of bridge

The SH1 southbound on-ramp bridge is a flyover above the CSM2 alignment that provides southbound connection onto the CSM2 alignment from MSR.

12.5.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two traffic lanes with wide shoulders and has a 2.0 m wide footway on one side.

Geometrics

The bridge carrying the SH1 southbound on-ramp is on curved horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

The on-ramp has a high skew to the motorway of about 60 degrees.

Foundation conditions

Ground conditions at the bridge comprise a 3 m layer of sandy silt overlying sandy gravel. The groundwater is approximately 14 m below ground level.

The liquefaction assessment has indicated no liquefaction potential in the soil profile.

Shallow footings and driven piled foundations options have been considered and it is recommended that a shallow footing founded on the sandy gravel layer is adopted.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises two traffic lanes in each direction, a median and outer shoulders and drainage swales.

Because of the high skew, two spans of 42.0 m are required over the main carriageway of the motorway and side spans of 30.0 m are required to suit the verges, swales and embankment slopes. This span arrangement allows the spans to be square to the piers and abutments and caters for the horizontal curve, with a single column pier provided for the centre median pier to fit within the proposed motorway median width.

Constraints on construction methods and materials

The bridge is required to be constructed over the greenfields motorway alignment which allows a variety of bridge forms to be considered, including precast prestressed beams, cast-in-situ concrete and steel composite I girders.

Suitable superstructure forms for the spans are steel composite I girders, steel box girders and concrete box girders.

Earthquake design requirements

As the road is part of State Highway 1 it is classified as Importance Level 3 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For the SH1 Southbound On-ramp Bridge the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footway and to provide a steel handrail on the outside of the footway. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on the side of the footway to give a total height of 1100 mm. The concrete barrier on the outside of the bridge remote from the footway will be 800 mm high.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from the SH1 Southbound On-ramp. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.5.3 Design options considered

The design options considered for this bridge are:

- Four span bridge using steel I girders, steel box girders or concrete box girders with open side spans using steel composite beams supported on RC piers on spread foundations and bank seats supported on the filled embankments and
- Two-span bridge using steel I girders, steel box girders or concrete box girders supported on RC piers on spread footings and MSE wall abutments with highly skewed abutments.

12.5.4 Materials considered

Materials considered for the superstructure are:

- 1800 mm deep steel composite beams with 250 mm thick deck slab
- 2100 mm deep steel box girders and
- 2500 mm deep concrete box girders.

12.5.5 Cost considerations

On the basis of construction cost, the option to use 1800 mm deep steel composite girders the lowest cost.

Options that provide either two spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments. The two-span option also has a high skew which will increase the cost rate compared to a perpendicular.

12.5.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches and
- Urban design considerations.

On the basis of the above factors, the preferred option is a four span bridge with steel composite I girders supported on RC piers with spread footings and bank seats supported on the approach filled embankments. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the steel I girders compared to the concrete box girder option. It also satisfies the urban design requirements to provide an open bridge structure.

12.5.7 Description of recommended option

The proposed bridge abutments and piers are in line with the centreline of the bridge. The bridge however is skewed at an angle of approximately 60 degrees to the CSM2 proposed alignment. The bridge is a four span bridge with the two internal span lengths of 42.0 m and end spans of 30.0 m.

The central pier is located in the middle of the motorway with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is 13.05 m to the outer edge face of the footpath. This provides for a carriageway width of 10.0 m, 1 No. wide footpath, 2 No. 0.4 m wide rigid barriers and allowance for a pedestrian handrail at the southern edge of the deck. Each span of the structure comprises 4 Steel I-girders with an in situ topping slab. The I-girders are typically 1800 mm deep with 500 mm wide top and bottom flanges. The in situ deck topping is a constant concrete thickness is 250 mm. A surfacing course will overlay the deck across the width of the roadway.

The central pier comprises a single 1500 mm diameter column on top of a 9 m x 7.5 m x thick spread footing, the outer piers consist of 2 No. 1250 mm diameter concrete columns on top of 17 m x 6.5 m x spread footings. For the central pier the I-girders are cast integral with the pier crosshead while the end pier crossheads provide seating for the I-girders. The abutments comprise a 2.75 m wide spread footing. The overall height of the abutment from the top of the deck to the underside of the footing is approximately. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.6 Waterholes Road Underpass

12.6.1 Description of bridge

The Waterholes Road Underpass spans the new CSM2 alignment linking the northern end of Waterholes road to the southern end. The bridge is not connected to the motorway and therefore there are no on-ramps or off-ramps.

12.6.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two traffic lanes with wide shoulders for cyclists and has wide footways on each side.

Geometrics

The bridge carrying Waterholes Road is on curved horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

Waterholes Road has a skew to the motorway of about 40 degrees, requiring the bridge to also be skewed to reduce span lengths.

Foundation conditions

Ground conditions at the bridge comprise a layer of sandy silt overlying sandy gravel. The groundwater is approximately 13 to below ground level.

The liquefaction assessment has indicated liquefaction potential in a soil layer at 17 m below ground which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundations options have been considered and it is recommended that a shallow footing founded on the sandy gravel layer is adopted.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises a four lane divided highway with a median including inner shoulders and 2.5 m outer shoulders.

Span lengths of are required over the main carriageway of the motorway and a span of is required for each side span the embankment batter slope.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered, including prestressed concrete beams, cast in-situ concrete and steel composite I girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For Waterholes Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footways and to provide steel handrails on the outside of the footways. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on top to give a total height of 1100 mm.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Waterholes Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.6.3 Design options considered

The design options considered for this bridge are:

- Four span bridge with open side spans using Double Hollow Core, Super Tee beams or steel composite beams supported on RC piers on spread foundations and bank seats supported on the filled embankments and
- A two-span bridge using double hollow core, Super Tee beams or steel composite beams supported on RC piers on spread footings and MSE wall abutments.

12.6.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams
- 1025 mm deep Super Tee beams with 180 mm deck slab and
- 1000 mm deep steel composite beams with 250 mm thick deck slab.

12.6.5 Cost considerations

On the basis of construction cost, the option to use 900 mm deep double hollow core beams gives the lowest cost.

Options that provide either two spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments.

12.6.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches and
- Urban design considerations.

On the basis of the above factors, the preferred option is a four span bridge with double hollow core beams supported on RC piers with spread footings and bank seats supported on the approach filled embankments. This option has the lowest construction cost and reduces the earthwork volume on the

approaches due to the thinner construction depth of the hollow core beams. It also satisfies the urban design requirements to provide an open bridge structure.

12.6.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle 40 degrees. The bridge consists of four skewed span lengths of 24.0 m. The central pier is located in the middle of the motorway with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is 15.3 m to the outer edge face of the footpath. This provides for a carriageway width of 2 No. wide footpaths, 2 No. wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure consists of 12 No. 900 mm deep precast prestressed concrete DHC units. An in-situ topping concrete of 180 mm minimum thickness at either edge of the deck forms the raised footpath. A surfacing course will overlay the DHC units across the width of the roadway.

The three piers comprise three 1000 mm diameter columns on top of 20 m x thick spread footings. The pier crossheads provide seating for the DHC units. The abutments comprise a wide spread footing. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.7 Trents Road Underpass

12.7.1 Description of bridge

The Trents Road Underpass spans the new CSM2 alignment linking the north western end of Trents Road to the southern eastern end. The bridge is not connected to the motorway and therefore there are no on-ramps or off-ramps.

12.7.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two traffic lanes with wide shoulders for cyclists and has wide footways on each side.

Geometrics

The bridge carrying Trents Road is on straight horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

Trents Road has a skew to the motorway of about 5 degrees, requiring the bridge to also be skewed to reduce span lengths.

Foundation conditions

Ground conditions at the bridge comprise a 2 m layer of silt overlying a 5 m layer of sandy gravel, over sandy silty gravel over sandy gravel. The groundwater is approximately 12 to 13 m below ground level.

The liquefaction assessment has indicated liquefaction potential in a soil layer at 14 to 20 m below ground which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundations options have been considered and it is recommended that a shallow footing founded on the sandy gravel layer is adopted.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises a four lane divided highway with a median including inner shoulders and 2.5 m outer shoulders.

A span length of 30.0 m is required over the main carriageway of the motorway and side spans of 27.0 m are required for each side span the swales and embankment batter slopes.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered, including prestressed concrete beams, cast in-situ concrete and steel composite I girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For Trents Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footways and to provide steel handrails on the outside of the footways. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on top to give a total height of 1100 mm.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Trents Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.7.3 Design options considered

The design options considered for this bridge are:

- Four span bridge with open side spans using double hollow core beams, Super Tee beams or steel composite beams supported on RC piers on spread foundations and bank seats supported on the filled embankments
- Three span bridge using double hollow core, Super Tee beams or steel composite beams supported on RC piers on spread footings and bank seats supported on the filled embankments
- Two-span bridge using Super Tee beams or steel composite beams supported on RC piers on spread footings and MSE abutments.

12.7.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams for the four span option only
- 1225 mm deep Super Tee beams with 180 mm deck slab for two and three span options and
- 1200 mm deep steel composite beams with 250 mm thick deck slab for two and three span options.

12.7.5 Cost considerations

Options that provide either four spans with open side spans or two spans with MSE walls or four spans with open side spans are considered to be similar cost to the three span option due to the trade-off of additional deck area with the cost of the MSE abutments.

12.7.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost

- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of the above factors, the preferred option is a three span bridge with Super Tee beams supported on RC piers with spread footings and bank seats supported on the approach filled embankments. This option has the lowest construction cost and it also satisfies the urban design requirements to provide an open bridge structure.

12.7.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle of approximately 5 degrees. The bridge is a three span bridge with an internal skewed main span length of and skewed end spans of 27.0 m. The two piers are positioned clear of the back face of the edge barriers.

The overall deck width is to the outer edge face of the footpath. This provides for a carriageway width of two x wide footpaths, two x wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure comprises 6 precast prestressed concrete Super Tee beams with an in situ topping slab. The precast Super Tee beams are 1225 mm deep and 2400 mm wide across the top flanges. The in situ deck topping concrete thickness is a minimum of 200 mm. A surfacing course will overlay the deck across the width of the roadway.

The two piers comprise two x 1000 mm diameter columns on top of spread footings that are 14.5 m thick. The pier crossheads provide seating for the Super Tee beams. The abutments comprise a wide spread footing. The overall height of the abutment from the top of the deck to the underside of the footing is approximately 5.0 m. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.8 Shands Road Underpass

12.8.1 Description of bridge

The Shands Road Underpass spans the new CSM2 alignment linking the north eastern end of Shands Road to the south western end. The bridge is not directly connected to CSM2, but the approach roads to the structure link back into the motorway on-ramps and off-ramps.

12.8.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide four traffic lanes with wide shoulders for cyclists and has wide footways on each side. There is a 300 mm narrow median between the two carriageways.

Geometrics

The bridge carrying Shands Road is on straight horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

Shands Road has a skew to the motorway of about 18 degrees, requiring the bridge to also be skewed to reduce span lengths.

Foundation conditions

Ground conditions at the bridge comprise a layer of sandy silt overlying a layer of sandy gravel, over 3 to of gravelly sand over sandy gravel. The groundwater is approximately 9 to 11 m below ground level.

The liquefaction assessment has indicated liquefaction potential in a soil layer at 10 to 16 m below ground which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundations options have been considered and it is recommended that driven piled foundations are founded in the Riccarton Gravels.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises a four lane divided highway with a median including inner shoulders and outer shoulders.

A span length of is required over the main carriageway of the motorway and side spans of are required for each side span the swales and embankment batter slopes.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered, including prestressed concrete beams, cast insitu concrete and steel composite I girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For Shands Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footways and to provide steel handrails on the outside of the footways. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on top to give a total height of 1100 mm.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Shands Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.8.3 Design options considered

The design options considered for this bridge are:

- Four span bridge with open side spans using double hollow core beams, Super Tee beams or steel composite beams supported on RC piers on driven piled foundations and bank seats supported driven piles
- Three span bridge using Super Tee beams or steel composite beams supported on RC piers on driven piled foundations and bank seats supported on driven piles
- Two-span bridge using Super Tee beams or steel composite beams supported on RC piers on driven piled foundations and bank seats with driven piles behind the MSE abutments.

12.8.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams for the four span option only
- 1225 mm deep Super Tee beams with 180 mm deck slab for two and three span options
- 1200 mm deep steel composite beams with 250 mm thick deck slab for two and three span options.

12.8.5 Cost considerations

Options that provide either four spans with open side spans or two spans with MSE walls or four spans with open side spans are considered to be similar cost to the three span option due to the trade-off of additional deck area with the cost of the MSE abutments.

12.8.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of these factors, the preferred option is a three span bridge with Super Tee beams supported on RC piers with driven piled foundations and piled bank seats. This option has the lowest construction cost and it also satisfies the urban design requirements to provide an open bridge structure.

12.8.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle of approximately 18 degrees. The bridge is a three span bridge with an internal skewed span length of 30.0 m and end spans of 27.0 m. The two piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is 22.6 m to the outer edge face of the footpaths. This provides for a carriageway width of 17.3 m, 2 No. wide footpaths, 2 No. wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure comprises 9 No precast prestressed concrete Super Tee beams with an in situ topping slab. The precast Super Tee beams are 1225 mm deep and 2400 mm wide across the top flanges. The in situ deck topping concrete thickness is 200 mm. A surfacing course will overlay the deck across the width of the roadway.

The two piers comprise three 1000 mm diameter columns on top of 22.0 m x 1.1 m thick pile caps. Each pile cap is supported by 48 No. 14.0 m long 310UC 118 driven piles. The piles are founded in the competent Riccardo gravels. The pier crossheads provide seating for the Super Tee beams. The abutments comprise a 2.75 m wide capping beam which have 30 No. 20.0 m long 310UC 118 driven piles piled down to competent Riccardo gravels. The overall height of the abutment from the top of the deck to the underside of the footing is approximately 4.0 m. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.9 Marshs Road Underpass

12.9.1 Description of bridge

The Marshs Road Underpass spans the new CSM2 alignment linking the north western end of Marshs Road to the south eastern end. The bridge is not connected to the motorway and therefore there are no on-ramps or off-ramps.

12.9.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two traffic lanes with wide shoulders and has a wide footway on each side.

Geometrics

The bridge carrying Marshs Road has a straight horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

The road has a high skew to the motorway of about 70 degrees.

Foundation conditions

Ground conditions at the bridge comprise a layer of sandy silt overlying a layer of sandy gravel, over 3 to of gravelly sand over sandy gravel. The groundwater is approximately 7 to 8.5 m below ground level.

The liquefaction assessment has indicated liquefaction potential in a soil layer at 9 to 19 m below ground which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundation options have been considered and it is recommended that driven piled foundations are founded in the Riccardo Gravels.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises two traffic lanes in each direction, a median and outer shoulders and drainage swales.

Because of the high skew, two spans of 48.0 m and 54.0 m are required over the main carriageway of the motorway and side spans of are required to suit the verges, swales and embankment slopes. This span arrangement allows the spans to be square to the piers, with a single column pier provided for the centre median pier to fit within the proposed motorway median width.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered cast-in-situ concrete and steel composite I girders.

Suitable superstructure forms for the 48.0 m and 54.0 m spans are steel composite I girders, steel box girders and concrete box girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For the Marshs Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footway and to provide a steel handrail on the outside of the footway. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on the side of the footway to give a total height of 1100 mm. The concrete barrier on the outside of the bridge remote from the footway will be 800 mm high.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Marshes Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.9.3 Design options considered

The design options considered for this bridge are:

- Four span bridge using steel I girders, steel box girders or concrete box girders with open side spans using steel composite beams supported on RC piers on driven piled foundations and bank seats supported on driven piles
- Two-span bridge using steel I girders, steel box girders or concrete box girders supported on RC piers on driven piled foundations and bank seats with driven piles behind the MSE wall abutments which are highly skewed.

12.9.4 Materials considered

Materials considered for the superstructure are:

- 2400 mm deep steel composite beams with 250 mm thick deck slab
- 2400 mm deep steel box girders
- 2800 mm deep concrete box girders.

12.9.5 Cost considerations

On the basis of construction cost, the option to use 2400 mm deep double steel I girders gives the lowest cost.

Options that provide either two spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments. The two-span option also has a high skew which will increase the cost rate compared to a perpendicular alignment.

12.9.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of these factors, the preferred option is a four span bridge with steel composite I girders supported on RC piers with driven piled foundations and piled bank seats. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the steel I girders compared to the concrete box girder option. It also satisfies the urban design requirements to provide an open bridge structure.

12.9.7 Description of recommended option

The proposed bridge abutments and piers are in line with the centreline of the bridge. The bridge however is skewed at an angle of approximately 70 degrees to the CSM2 main alignment. The bridge is a four span bridge with the internal span lengths of 48.0 m and 54.0 m and the end spans of 40.0 m. The central pier is located in the middle of the motorway with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is to the outer edge face of the footpath. This provides for a carriageway width of two wide footpaths, two wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure comprises 4 Steel I-girders with an in situ topping slab. The I-girders are typically 2400 mm deep with 500 mm wide top and bottom flanges. The in situ deck topping is a thickness is 250 mm. A surfacing course will overlay the deck across the width of the roadway.

The central pier comprises a single 1500 mm diameter column on top of 10 m x 8.75 m x 1.6 m thick pile caps. The outer piers consist of 2 No. 1250 mm diameter concrete columns on top of 16.25 m x

7.5 m x 1.4 m pile caps. The pile caps are supported by 18.0 m long 310UC 118 driven piles. The piles are founded in the competent Riccardo gravels. For the central pier the I-girders are cast into the pier crosshead while the end pier crossheads provide seating for the I-girders. The abutments comprise a wide capping beam with long 310UC 118 driven piles piled down to competent Riccardo gravels. The overall height of the abutment from the top of the deck to the underside of the footing is approximately 3.8 m. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.10 Springs Road Underpass

12.10.1 Description of bridge

The Springs Road Underpass spans the new CSM2 alignment linking the north eastern end of Springs Road to the southern western end. The bridge is not connected to the motorway and therefore there are no on-ramps or off-ramps.

12.10.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide two traffic lanes with wide shoulders and has a wide footway on each side.

Geometrics

The bridge carrying Marshs Road has a straight horizontal alignment and has a curved vertical alignment with a large radius hog curve provided.

The road has a high skew to the motorway of about 40 degrees.

Foundation conditions

Ground conditions at the bridge comprise a layer of sandy silt overlying a 4 m layer of sandy over saturated sandy gravel. The groundwater is approximately 6 to 7 m below ground level.

The liquefaction assessment has indicated liquefaction potential in a soil layer at 15 to below ground which is considered to have limited influence on the structural behaviour of the bridge.

Shallow footings and driven piled foundation options have been considered and it is recommended that driven piled foundations are founded in the Riccardo Gravels.

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises two traffic lanes in each direction, a median and outer shoulders and drainage swales.

Because of the skew, two spans of are required over the main carriageway of the motorway and side spans of are required to suit the verges, swales and embankment slopes.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered including precast prestressed beams, cast-in-situ concrete and steel composite I girders.

Suitable superstructure forms for the spans are double hollow core beams, Super Tee beams and steel composite I girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For the Springs Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL 4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footway and to provide a steel handrail on the outside of the footway. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on the side of the footway to give a total height of 1100 mm.

The outer pedestrian handrails will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Springs Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.10.3 Design options considered

The design options considered for this bridge are:

- Four span bridge using double hollow core beams, Super Tee beams or steel composite I girders with open side spans supported on RC piers on piled foundations and bank seats supported on piled foundations and
- Two-span bridge using double hollow core beams, Super Tee beams supported on RC piers on driven piled foundations and bank seats with driven piles behind MSE wall abutments which are skewed.

12.10.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams
- 1025 mm deep Super Tee beams with 180 mm deck slab and
- 1000 mm deep steel composite beams with 250 mm thick deck slab.

12.10.5 Cost considerations

On the basis of construction cost, the option to use 900 mm deep double hollow core beams gives the lowest cost.

Options that provide either two spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments. The two-span option also has a high skew which will increase the cost rate compared to a perpendicular alignment.

12.10.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches and
- Urban design considerations.

On the basis of the above factors, the preferred option is a four span bridge with double hollow core beams supported on RC piers with driven piled foundations and piled bank seats. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the double hollow core beams. It also satisfies the urban design requirements to provide an open bridge structure.

12.10.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle of 40 degrees. The bridge consists of four skewed span lengths of 20.0 m. The central pier is located in the middle of the motorway with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is to the outer edge face of the footpath. This provides for a carriageway width of 2 No. wide footpaths, 2 No. wide rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure consists of 12 No. 90 mm deep precast prestressed concrete DHC units. An in-situ topping concrete of 180 mm minimum thickness at either edge of the deck forms the raised footpath. A surfacing course will overlay the DHC units across the width of the road carriageway.

The three piers comprise three 900 mm diameter columns on top of 21 m x 1.1 m thick pile caps. Each pile cap is supported by 36 No. long 310UC 118 driven piles. The piles are founded in the competent Riccardo gravels. The pier crossheads provide seating for the DHC units. The abutments comprise a wide capping beam with 22 No. long 310UC 118 driven piles piled down to competent Riccardo gravels. The overall height of the abutment from the top of the deck to the underside of the footing is approximately. A 250 mm thick, long settlement slab is provided to minimise post-construction differential settlement.

12.11 Halswell Junction Road Underpass

12.11.1 Description of bridge

The Halswell Junction Road Underpass spans the new CSM2 alignment linking the north western end of Halswell Junction Road to the south eastern end. The bridge is not directly connected to CSM2, but the approach roads to the structure link back into the motorway on-ramp and off-ramp.

12.11.2 Factors which influence the design

Provision for traffic, pedestrians and cyclists

The bridge is to provide four traffic lanes with wide outer shoulders and has a wide footway on one side. The two carriageways are divided by a 2.4 m wide median including 0.3 m shoulders.

Geometrics

The bridge carrying Halswell Junction Road has a straight horizontal alignment and has a straight vertical alignment.

The road has a high skew to the motorway of about 53 degrees.

Foundation conditions

Ground conditions at the bridge comprise a layer of sandy silt overlying a layer of sandy over saturated sandy gravel. The groundwater is approximately 6 to below ground level.

The liquefaction assessment has indicated no liquefaction potential in the soil profile.

Shallow footings and driven piled foundation options have been considered and it is recommended that driven piled foundations are founded in the Riccardo Gravels

Constraints on span arrangements and clearances

The bridge is required to span the motorway which at this location comprises two traffic lanes in each direction, a median and outer shoulders and drainage swales.

Because of the skew, two spans of 25.0 m are required over the main carriageway of the motorway and side spans of 25.0 m are required to suit the verges, swales and embankment slopes.

Constraints on construction methods and materials

The bridge is required to be constructed over the new motorway which allows a variety of bridge forms to be considered including precast prestressed beams, cast-insitu concrete and steel composite I girders.

Suitable superstructure forms for the 25.0 m spans are double hollow core beams, Super Tee beams and steel composite I girders.

Earthquake design requirements

As the road is a local road it is classified as Importance Level 2 and it has a design life of 100 years.

The seismic design parameters for the bridge are derived from the Transit Bridge Manual and the December 2004 Provisional Amendment.

Urban design considerations for the visual appearance of the bridge

Requirements for urban design are addressed in the Technical Report 6.

For the Halswell Junction Road Underpass the use of open side spans to give an open feel for road users is preferred.

Side protection requirements

TL4 edge barriers are required for the traffic flows and risk factors at this site based on the Transit Bridge Manual requirements.

It is proposed to provide concrete TL4 barriers on the edge of the carriageway inside the footway and to provide a steel handrail on the outside of the footway. The concrete barriers will be 800 mm high concrete with a 300 mm high steel handrail on the side of the footway to give a total height of 1100 mm. The concrete barrier on the outside of the bridge remote from the footway will be 800 mm high with a 300 mm steel handrail.

The outer pedestrian handrail will be 1100 mm high to suit pedestrians.

Access for inspection and maintenance

Access for inspection and maintenance will be carried out from the motorway below and from Halswell Junction Road. Traffic management will be required for these activities, with lane closures during off-peak periods be sufficient for most activities.

12.11.3 Design options considered

The design options considered for this bridge are:

- Four span bridge using double hollow core beams, Super Tee beams or steel composite I girders with open side spans supported on RC piers on driven piled foundations and bank seats supported on piled foundations and
- Two-span bridge using Super Tee beams or steel I girders supported on RC piers on driven piled foundations and piled bank seats, with skewed MSE wall abutments.

12.11.4 Materials considered

Materials considered for the superstructure are:

- 900 mm deep double hollow core prestressed beams
- 1025 mm deep Super Tee beams with 180 mm deck slab
- 1000 mm deep steel composite beams with 250 mm thick deck slab.

12.11.5 Cost considerations

On the basis of construction cost, the option to use 900 mm deep double hollow core beams gives the lowest cost.

Options that provide either two spans with MSE walls or four spans with open side spans are considered to be similar cost due to the trade-off of additional deck area with the cost of the MSE abutments. The two-span option also has a high skew which will increase the cost rate compared to a perpendicular alignment.

12.11.6 Option selection

The preferred option has been selected on the basis of:

- Construction cost
- Whole of life cost and future maintenance
- Overall bridge height to reduce earthworks volumes on approaches
- Urban design considerations.

On the basis of the above factors, the preferred option is a four span bridge with double hollow core beams supported on RC piers with driven piled foundations and piled bank seats. This option has the lowest construction cost and reduces the earthwork volume on the approaches due to the thinner construction depth of the double hollow core beams. It also satisfies the urban design requirements to provide an open bridge structure.

12.11.7 Description of recommended option

The proposed bridge abutments and piers are skewed with respect to the centreline of the bridge at an angle of approximately 53 degrees. The bridge consists of four equal skewed span lengths of 25.0 m. The central pier is located in the middle of the motorway with rigid barriers either side. The two outer piers are positioned clear of the back face of the edge barriers at the shoulders of the road.

The overall deck width is 23.2 m to the outer edge of the bridge. This provides for a carriageway width of 1 No. 2.0 metre wide footpath, 2 No. rigid barriers and allowance for pedestrian handrails at the outer edges of the deck. Each span of the structure consists of 20 No. 900 mm deep precast prestressed concrete DHC units. An in-situ topping concrete of 180 mm minimum thickness at one edge of the deck forms the raised footpath. A surfacing course will overlay the DHC units across the width of the roadway.

The three piers comprise five 900 mm diameter columns on top of 37.5 m x 1.1 m thick pile caps. Each pile cap is supported by 60 No. long 310UC 118 driven piles. The piles are founded in the competent Riccardo gravels. The pier crossheads provide seating for the DHC units. The abutments comprise a wide capping beam with 40 No. long 310UC 118 driven piles piled down to competent Riccardo gravels. The overall height of the abutment from the top of the deck to the underside of the footing is approximately. A 450 mm thick, long approach slab is provided to assist in resisting longitudinal loads from the structure.

13. Property Requirements and Designation Width

While investigating the design options, the philosophy has been to minimise the overall impact on property acquisition whilst still achieving the best Project outcome. With respect to the designation, a minimal philosophy has been adopted, route security has also been considered in ensuring there is adequate designation allowed for the construction of the Project. The design contained within the Plan set provides the NZTA with preliminary land requirement plans for consultation and the property acquisition required to develop this Project to the next stage.

Chapter 11 of the AEE concerning land use and property discusses land requirements, the approach to acquisition and processes under the Public Works Act 1981 in more detail. The following drawings contained within the Plan set in Volume 5 will also provide further detail:

- Land Requirement Plans: 62236-A-C1101 to C1110 and 62236-B-C1101 to C1118
- Designation Plans: 62236-A-C1001 to C1010 and 62236-B-C1001 to C1018.

13.1 Property Access

The rear access roads discussed in section 7.4 above detail the property access proposals for MSRFL.

The CSM2 alignment predominantly passes through existing rural land, thereby minimising the effect on property accesses. Where alterations are proposed to local roads (for realignments or structures) property access are affected particularly at Robinsons Road, Waterholes Road, Trents Road and Springs Road. Details of these access alterations will be confirmed at detailed design stage based on further consultation, however initial discussions with effected residents have been positive.

13.2 KiwiRail

As stated in section 8.2.2 above, KiwiRail has agreed in principle for the CSM2 alignment to cross the existing industrial rail siding corridor at grade, with the existing siding to be realigned. A formal Deed of Grant or land purchase agreement will need to be obtained for this option in the next design phase.

The western rear access road discussed in section 7.4 above utilises part of the KiwiRail corridor between Weedons Ross Road and Robinsons Road. Again, KiwiRail has agreed to this in principle a formal purchase agreement will need to be obtained for this option in the next design phase.

13.3 Accommodation Works

The Crown's property agents (The Property Group and Opus) will negotiate any property acquisition requirements on behalf of the Crown. Other accommodation works may be identified as the design is developed in the next phase.

As a minimum, affected fencelines, gates, accesses and signs that require relocation will be safely relocated or replaced to match the existing standard in consultation with the effected land owners.

Quantification of the required accommodation works will be confirmed at detailed design stage, following further consultation with effected land owners.

As per the drainage layout plans (62236-A-C401 to C406 and 62236-B-C401 to C417) included in the Plan set in Volume 5, alterations to existing water races are required, which generally will be carried out in advance of the main contract works.

13.4 Designation Width

The proposed designation width has been based on the typical cross section widths described in section 6.7 and the 'safe systems' approach. It was determined during two workshops and additional discussions between the NZTA's technical and planning personnel and Consultants.

MSRFL

The MSRFL section is constrained by the existing properties adjacent to MSR, with the designation set at typically 42.5 m wide. However this widens in the vicinity of Weedons Road to incorporate the grade separated interchange and associated ramps. There are also two locations along the western side of MSRFL where additional landscaping is proposed to help mitigate views for the motorist. In these locations, an additional two metre wide strip has been included to allow for the enhanced landscaping.

CSM2

Following the first designation workshop, the proposed CSM2 designation was generally 90 m to 100 m wide which allowed for future off-road cycle paths, maintenance access away from the main carriageway, additional bunding and landscaping provisions and design refinement. This width was considered overly conservative, especially after the Project safety audit confirmed that the CSM2 alignment and footprint was appropriate. Therefore a full review was undertaken in a second workshop held during April 2012. In this workshop it was agreed to adopt a typical designation width of 70 m but generally widening to 80 m in the vicinity of bridges to allow for increased landscaping. This is similar in approach to the designation philosophy for CSM1, providing continuity for the road user.

The minimum 70 m designation width was based on the following components:

- 25.0 m total carriageway width (pavement edge to pavement edge)
- 35.0 m made up of 17.5 m width each side of carriageway incorporating grass drainage swales with 4:1 batter slopes (clear zone from carriageway edgeline)
- allowing for a landscape buffer each side of the carriageway located behind the drainage swales.

As noted above the 70 m was adopted as the base designation width and widened where necessary to accommodate additional landscaping at key locations, primarily in response to landowner consultation and feedback. Key locations where the designation has been widened with additional landscaping/screening includes the northern side of the CSM2 alignment adjacent to Claremont

subdivision and the southern side of CSM2 between Blakes Road through to Shands Road ramps catering for the residents primarily located along Blakes Road and the Aberdeen subdivision.

14. Environmental Considerations

The effect of the Project on the environment has been assessed as part of the AEE and is addressed in detail in the associated Technical Reports. The design philosophy in respect to environmental considerations is to minimise the effects of the Project on the environment, with the focus of the technical reports to highlight the resultant effects and recommend appropriate mitigation. In particular the technical reports listed in Table 12 are applicable.

Table 12 Environmental Considerations – Technical Report references

| No. | Technical Report Title | Primary AEE Chapter Reference |
|-----|---|-------------------------------|
| 2 | Traffic and transportation effects report | 10 |
| 3 | Assessment of stormwater disposal and water quality | 1 |
| 4 | Landscape and visual effects | 14 |
| 5 | Assessment of effects – urban design | 13 |
| 6 | Urban and landscape design framework | 13, 14 |
| 7 | Landscape Context Report | 14 |
| 8 | Assessment of operational noise effects | 16 |
| 9 | Assessment of construction noise & vibration | 16 |
| 11 | Geotechnical engineering and geo-hazards assessment | 21, 22 |
| 13 | Social Impact Assessment | 27 |
| 19 | Lighting assessment | 15 |
| – | Construction Environmental Management Plan | 5 |

There are three areas of particular note where environmental design consideration is important.

14.1 Sediment Control

For most of the Project area sediment potential is limited due to flat grades, gravel subgrades and separation from watercourses. In the area around Springs Road and Halswell Junction Road, more substantial measures will be required due to the proximity of existing water courses. Technical Report 3 (Assessment of stormwater disposal and water quality) and the Construction Environmental Management Plan considers sediment control practices and controls in detail.

14.2 Noise

An assessment of the existing and anticipated road traffic noise levels has been undertaken and identifies appropriate mitigation measures. Refer to Technical Reports 8 and 9 for further details. In addition, Technical Report 6 – Urban and Landscape Design Framework considers the effects of the proposed mitigation measures.

14.3 Landscape Design

Full landscape design has been undertaken as part of this Project – refer to the Plan set and Technical Report 7 for further details. In addition, Technical Report 4 – Landscape and Visual Effects assesses visual effects and the proposed landscape mitigation for the Project which has been incorporated into the landscape design.

15. Project Construction

A key part of the design philosophy has been to ensure the Project is able to be constructed in a safe and cost effective manner. This has been achieved through designing to best practice standards and consultation with a construction industry leader at key stages of the design.

In addition to the information below, the Construction Environmental Management Plan (CEMP) and associated Specialised Environmental Management Plans (SEMP) covers conceptual staging, management of traffic and identify areas of risk / concern for the construction phase of the Project. The CEMP can be found in Volume 4.

15.1 Enabling Works

Several packages of work are recommended to be carried out in advance of the main contract to provide more efficient and safer construction during the main contract and to minimise disruption throughout the construction phase. These include:

- Transpower infrastructure works at Shands Road (refer to section 8.6.1 above)
- Orion overhead lines undergrounding along MSRFL
- Rear access roads on the MSRFL section (refer to section 7.4 above)
- The Nursery Access adjacent to Robinsons Road (refer to section 7.1.2 above).

Refer to section 5.3 in the AEE for further details regarding enabling works.

15.2 Construction Methodology

The AEE (Chapter 5) and CEMP (Chapter 6) both discuss the approach to construction. This will be further developed once a Contractor has been appointed. The overall philosophy however is to construct local road connections first, along with the associated structures and embankments in order to maintain local connectivity. This will necessitate temporary diversions at some locations while structures are being built, possibly supplemented by local road closures at times.

The motorway mainline is generally proposed to be constructed in conjunction with the structures as it is through greenfield land and would not be open to traffic. The widening of MSR principally to the north western side of the current alignment enables the existing carriageway to remain operational, so construction of the additional lanes can take place offline.

15.3 Construction Traffic Management

It is foreseen that the objectives of the temporary traffic management for the construction of the MSRFL and CSM2 Project will be:

- adherence to the standards set out in COPTTM wherever practical

- minimise disruption on state highways and local roads as far as is practical and maintain existing flows and travel times
- minimise the number of construction vehicle trips and their effects on local roads and seek to avoid residential areas where practical
- minimise the effects of construction vehicle parking.

Construction traffic management is the subject of a SEMP (Construction Traffic Management Plan) which should be referred to for further detail.

16. Long Term Maintenance Requirements

A philosophy of approach has been taken in the preliminary designs for MSRFL and CSM2 to minimise on-going maintenance costs wherever possible by considering industry best practice throughout the design process. This philosophy should be continued through the detailed design process.

On-going maintenance will be required for the bulk of assets installed as part of this Project, including:

- line marking
- pavements and surfacing
- landscaping and grassed areas
- signs, signals, lighting and barriers
- bridges and structures
- stormwater infrastructure.

The boundaries for maintenance responsibility between the NZTA and local councils will be confirmed at detailed design stage in particular around the interchange ramps, bridges and approach embankments.

Appendix A – Bridge Foundation Options

- Weedons
- Robinsons / Waterholes
- Trents / Blakes
- Marshs / Shands
- Springs / Halswell Junction
- SH1

Memorandum

To: Nik Stewart/Vincent Lobendahn
From: Greer Gilmer / Grant Newby
Copy: Dave Aldridge
Subject: Weedons Rd Underpass – Revision B

Date: 12 August 11
Our Ref: 3390691

This memo provides soil parameters/stiffness for design of the foundation options at the Weedons Rd Underpass Structure. These have been based on the information provided in emails from V Lobendahn dated 27th June 27 and 5th July 2011. The memo also includes estimates of settlement and liquefaction information following assessment of the structures based on loading information provided on 19th July 2011.

Options on the structural form and foundation type to be adopted for Weedons Rd Underpass Structure should be included in the Scheme Assessment Report. This will need to be reviewed as part of the scheme design.

Our assessment provided below has considered the current preferred structural form option with abutments and piers founded on shallow footings.

The soil model has been based on borehole information at this location, which will need to be reviewed when the project wide model is completed.

1.1 Soil Profile

A moderately conservative soil profile has been adopted for Weedons Rd Underpass, with particular emphasis on the geotechnical investigations listed below which are located closest to the structure:

- Cable Tool Boreholes (CT): CT3, 2, 4
- Air Flush Boreholes (AF): AF1 - 3
- Test pits (TP): TP2 – 4, 10, 11

Table 1 – Design Soil Profile

| Soil Layer | Assumed Material Type ⁽¹⁾ | Depth below GL to top of layer (m) | Top of layer RL (m) | Layer Thickness (m) | Design SPT N ₆₀ Values |
|------------|--------------------------------------|------------------------------------|---------------------|---------------------|-----------------------------------|
| 1 | Sandy SILT | 0 | 51 | 3 | - |
| 2 | Sandy GRAVEL | 3 | 48 | - | 22 (10 – 50) |

SPT N₆₀ Values based on AF and CT noted above and values in brackets indicate range.

1.2 Groundwater

The groundwater level was measured at RL 36m at the proposed location of the Weedons Rd Structure (approximately 14 to 15m below existing ground level).

Memorandum

2 Proposed Structural Arrangement

The Weedons Rd Underpass Structure provides local road access (2 lanes) over the proposed Main South Road Four Lining (MSRFL). The proposed structure incorporates approach embankments in the order of 7.0m high. A four span concrete bridge is being considered, with three central piers and abutments at each approach.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 5, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 2 and a design life of 100 years. The PGA for liquefaction at the Weedons Rd Underpass Structures are summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

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

¹ NZS 1170.5: 2004, Table 3.5 and as advised by N Stewart.

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

The liquefaction potential for the site underlying the Weedons Rd Underpass Structure was assessed using SPT data from the AF1, 2 and CT2, 4 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.44g and SLS PGA 0.11g. Assessment of CT4 indicates potential for liquefaction under ULS seismic conditions, in a layer between 15 and 19m below ground surface. However, at depths greater than 12m below the ground surface the effects of settlement on the structure are anticipated to be minor.

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed that settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided Table 6, and discussed in Section 4.1.

3.3.2 Seismic Settlement

Under a ULS seismic event potential settlement is expected to be negligible at the Weedons Road Underpass, with soils assessed as not to be susceptible to liquefaction.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 Sandy SILT, and with localised removal of this silt layer down to Layer 2 Sandy GRAVEL and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread through lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within immediate vicinity of each bridge, are assumed to be approximately 7.5m high. The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 – Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the Weedons Rd Underpass Structures will be founded within the approach embankments. The embankment fill material is assumed to be well graded, alluvial sandy gravels, as described above.

Assessment has considered abutments of 14 m length (from preliminary structural drawings) with a footing width ranging from 1.5m to 2.5m.

Memorandum

4.1.1 Abutment Bearing Capacity

Design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing founded within fill at a height of 3m the above existing ground. Allowable bearing capacities on the embankment fill, Layer 1 Sandy SILT, and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 6-1a and 1b. The ultimate geotechnical bearing pressures are equal to three times the allowable bearing pressures.

4.1.2 Abutment Footing Settlement

Table 6-1a and 6-1b summarises anticipated settlement beneath the abutment. Settlements of between 5 to 15mm are anticipated.

4.2 Piers

A spread footing option at the piers has been considered for the Weedons Rd Underpass, considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 Sandy SILT.
- Removal of the Layer 1 Sandy SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

Excavation beneath the groundwater table is not anticipated for either of these sub options. Refer Table 6-2a and 6-2b.

Assessment has considered foundations for the central piers of 14m in length (from preliminary structural drawings) with a footing ranging in width from 6 to 8m.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing pressures for shallow footings founded within the Layer 1 Sandy SILT and Layer 2 Sandy GRAVEL exceed the design pressure. Refer Table 6-2a and 2b.

4.2.2 Pier Footing Settlement

Table 6-2a and 6-2b summarises anticipated settlement beneath the central piers. Settlements of between 10 to 50mm are anticipated where founded in the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles 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 final proposed pile arrangement should be confirmed following the preliminary structural design.
- For steel driven H piles or tube piles spaced at centres of no less than 4 pile diameters, pile tip settlements of less than 5mm are expected under SLS loads.

Memorandum

- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|---------------------------|-------------------------------------|
| Waterholes Rd Underpasses | 9 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials. To be reviewed following receipt of the additional geotechnical investigation data.
- Other pile types may be suitable and can be considered if required.

5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5) and geometry as described below.

Approach Embankment Geometry:

- Height = ~7m
- Carriageway Width = 14m
- Slope Batter = 2:1

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|-------------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 2 | 1.56 | - | - |
| | Static - undrained | 2 | 1.53 | - | - |
| | Seismic | 2 | 0.63 | 0.19 | ≤150 |
| No Movement (3:1) | Seismic | 2 | 1 | 0.44 | Nil |

¹ Importance Level 2, Design PGA=0.44g

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement.

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

7 Recommendations

- Shallow footings are recommended for the Weedons Rd Underpass Structures based on the geotechnical conditions indicated by the investigations at the site.
- At the piers, removal of the typically 3m thick Layer 1 Sandy SILT is recommended beneath shallow footings to mitigate bearing failure and to minimise static settlement.
- At the abutments, the shallow footings may be constructed within the fill embankment without removal of the Layer 1 Sandy SILT, though removal would mitigate risk of differential settlement between the piers and abutments.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ' (°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 ¹ | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy silty GRAVEL | - | 32 | 0 | 30 | - | 0.26 | 4.5 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2

³ $K = n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$).

Memorandum

Table 6 –Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | SILT | 14 | 1.5 – 2.5 | 2900 | 10-15 | 16 | 250 |
| | 1b ⁴ | Sandy GRAVEL | 14 | 1.5 – 2.5 | 2900 | 5 - 10 | 10 | 750 |
| Central Piers | 2a | SILT | 14 | 6 - 8 | 5764 | 30 - 50 | 50 - 70 | 250 |
| | | SILT | 14 | 6 - 8 | 6317 | 30 - 50 | 50 - 80 | |
| | 2b | Sandy GRAVEL | 14 | 6 - 8 | 5764 | 10 - 20 | 50 - 70 | 750 |
| | | Sandy GRAVEL | 14 | 6 - 8 | 6317 | 10 - 20 | 50 - 80 | |

¹ Dead Load only considered for settlement analysis.

² Using foundation width 1.5m for abutments and 6 - 8m for central piers

³ Assuming the silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill

Memorandum

To: Nik Stewart/Vincent Lobendahn **Date:** 12 August 11
From: Greer Gilmer / Grant Newby **Our Ref:** 3390691
Copy: Dave Aldridge
Subject: Waterholes Rd and Robinsons Rd Structures– Revision B

This memo provides soil parameters/stiffnesses for design of foundation options at the Waterholes Rd and Robinsons Rd Structures. These have been based on the information provided in emails from V Lobendahn dated 27th June and 5th July 2011. The memo also includes estimates of settlement and liquefaction effects following assessment of the structures based on loading information provided on 19th July 2011.

Options for the structural form and foundation type to be adopted for Waterholes Rd and Robinsons Rd Structures should be included in the Scheme Assessment Report. This will need to be reviewed as part of the scheme design.

Our assessment provided below has considered the current preferred structural form option with abutments and piers founded on shallow footings.

The soil model has been based on borehole information at this location, which will need to be reviewed when the project wide model is completed.

1.1 Soil Profile

A moderately conservative soil profile has been adopted for Waterholes Rd and Robinsons Rd Structures, with particular emphasis on the geotechnical investigations listed below which are located closest to the structure:

- Cable Tool Boreholes (CT): CT4
- Air Flush Boreholes (AF): AF4 – 5, 7 - 9
- Test Pits (TP): TP2 – 8, 12 – 15, 23 - 25

Table 1 – Design Soil Profile

| Soil Layer | Assumed Material Type ⁽¹⁾ | Depth below GL to top of layer (m) | Top of layer RL (m) | Layer Thickness (m) | Design SPT N ₆₀ Values |
|------------|--------------------------------------|------------------------------------|---------------------|---------------------|-----------------------------------|
| 1 | Sandy SILT | 0 | 40 - 45 | 1 | 22 (11 – 40) |
| 2 | Sandy GRAVEL | 1 | 39 - 44 | - | 25 – 30 (12 -50) |

SPT N₆₀ Values based on AF and CT noted above and values in brackets indicate range.

1.2 Groundwater

The groundwater levels were measured at between RL 33 - 34m at the proposed location of the Robinsons Rd Structure (approximately 12 to 13m below existing ground level) and RL 26 - 27m at the proposed location of the Waterholes Rd Structure (approximately 13 to 14m below existing ground level) during 2010 and 2011.

2 Proposed Structural Arrangement

The Waterholes Rd Underpass Structure provides local road access (2 lanes) over the proposed Christchurch Southern Motorway Stage 2/ Main South Road Four Laning (CSM2/MSRFL). The Robinsons Rd Overpass Structure provides local road access (2 lanes) under the proposed MSRFL. The proposed structures incorporate approach embankments in the order of 7.0m high.

At Waterholes Rd a four span concrete bridge is being considered, with three central piers and abutments at each approach. At Robinsons Rd a single span concrete bridge on a Mechanically Stabilised Earth (MSE) Wall with abutments at each approach is being considered.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 5, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 2 and 3 (for the Robinsons Rd Overpass) and a design life of 100 years. The PGA for liquefaction at the Waterholes and Robinsons Road are summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

| Importance Level (NZS 1170) | Annual Probability of Exceedance | | PGA (as proportion of g) | |
|--------------------------------|----------------------------------|------------------|-----------------------------|------------------|
| | ULS ² | SLS ³ | ULS ² | SLS ³ |
| 2 | 1/1000 | 1/50 | 0.44 | 0.11 |
| 3 | 1/5500 | 1/50 | 0.6 | 0.11 |

¹ NZS 1170.5: 2004, Table 3.5 and as advised by N Stewart

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

The liquefaction potential for the Waterholes Rd Underpass Structure was assessed using SPT data from the AF7 – 9, CT4 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.44g and SLS PGA 0.11g. Assessment indicated liquefaction potential under ULS seismic conditions, at 17m below ground surface in AF8. However, at this depth influence on the structure is anticipated to be minor.

Within this layer liquefaction has been assessed to initiate for seismic PGA greater than 0.27g.

The liquefaction potential for the Robinsons Rd Overpass Structure was assessed using SPT data from the AF4, 5 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.44g and SLS PGA 0.11g. Assessment indicates none of the soil profile is susceptible to potential liquefaction under both ULS and SLS seismic conditions.

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided in Table 6 and 7, and discussed in Section 4.1.

3.3.2 Seismic Settlement

Under a ULS event settlement is expected to be negligible at the Waterholes Rd and Robinsons Rd Structures, with the soils assessed to be not susceptible to liquefaction.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 Sandy SILT and, with localised removal of this silt layer, down to the Layer 2 Sandy GRAVEL and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread though lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within the immediate vicinity of each bridge, are assumed to be approximately 6.5m high (Waterholes Road) and 7.5m (Robinsons Rd). The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

Memorandum

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 – Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the Waterholes Rd Underpass structure would be founded within the approach embankments. The embankment fill material is assumed to be well graded, alluvial sandy gravels as described above.

Assessment has considered abutments of 14m in length at Waterholes Rd (from preliminary structural drawings) with a footing width ranging from 1.5m to 2.5m

The abutments of the Robinsons Rd Overpass structure will be founded on or within the approach embankments behind the MSE wall. The embankment fill material is assumed to be well graded, alluvial sandy gravels as described above.

Assessment has considered abutments of 45m in length (from preliminary structural drawings) with a footing width ranging from 1.5m to 2.5m

4.1.1 Abutment Bearing Capacity

At Waterholes Rd the design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing is founded within fill at a height of 3m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 Sandy SILT, and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 6-1a and 1b.

At Robinsons Rd the design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing founded within fill at a height of 5.5m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 Sandy SILT, and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 7-1a and 1b.

The ultimate geotechnical bearing capacity is equivalent to three times the allowable bearing capacity.

4.1.2 Abutment Footing Settlement

Table 6-1a and 1b, and Table 7-1a and 1b, summarises anticipated settlement beneath the abutments. Settlements of <5 to 20mm are anticipated at Robinsons Rd and Waterholes Rd.

4.2 Piers

A spread footing option at the piers has been considered for the Waterholes Rd Underpass, considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 Sandy SILT.
- Removal of the Layer 1 Sandy SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

Excavation beneath the groundwater table is not anticipated for either of these sub options. Refer Table 6-2a and 6-2b.

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Assessment has considered foundations for central piers of 14m in length (from preliminary structural drawings) with a footing width ranging from 6m to 8m.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing pressures for shallow footings founded within the Layer 1 Sandy SILT satisfy design. Allowable bearing capacities for the Layer 2 Sandy GRAVEL however, exceed the design pressures. Refer Table 6-2b.

4.2.2 Pier Footing Settlement

Table 6-2a and 6-2b summarises anticipated settlement beneath the central piers at Waterholes Rd. Settlements of 5 to 40mm anticipated at Waterholes Rd where founded on the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles 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 final proposed arrangement should be confirmed following preliminary structural design.
- For steel driven H piles or tube piles spaced at centres of not less than 4 pile diameters, pile tip settlements of less than 5mm could be expected under SLS loads.
- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|---------------------------|-------------------------------------|
| Waterholes Rd Underpasses | 10 MPa |
| Robinsons Rd Overpass | 14 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials.
- Other pile types may be suitable and can be considered if required.

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5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5) and geometry as described below.

Approach Embankment Geometry:

- Height = ~7m
- Carriageway Width = 14m
- Slope Batter = 2:1

MSE Wall Geometry

- Height = 5.5m
- Depth = 5.5m

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|-------------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 2 | 1.64 | - | - |
| | Static - undrained | 2 | 2.64 | - | - |
| | Seismic | 2 | 0.78 | 0.45 | ≤20 |
| No Movement (5:1) | Seismic | 2 | 0.94 | 0.56 | Negligible |
| MSE Wall | Static | 3 | 2.48 | - | - |
| | Static - undrained | 3 | 2.71 | - | - |
| | Seismic | 3 | 0.88 | 0.5 | ≤10 |

¹ Importance Level 2, Design PGA=0.44g. Importance Level 3, Design PGA=0.6g

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement

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

7 Recommendations

- Shallow footings are considered suitable for the Waterholes Rd Underpass and Robinsons Rd Overpass based the geotechnical conditions indicated by investigations at each site.
- At the central piers on Waterholes Rd, removal of the upper typically 1m Layer 1 Sandy SILT is recommended beneath shallow footings to mitigate excessive static settlements.
- At the abutments of both structures, the shallow footings may be constructed within the fill embankment without removal of the Layer 1 Sandy SILT, provided settlements due to embankment fill have occurred, though removal would mitigate risk of differential settlement between the piers and abutments.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ' (°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 ¹ | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy GRAVEL | - | 36 | 0 | 30 | - | 0.28 | 5 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2.

³ $K=n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$).

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Table 6 – Waterholes Rd Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Proposed Structure Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|---|---|
| Abutments | 1a ³ | Sandy SILT | 14 | 1.5 – 2.5 | 2912 | 10 - 20 | 23 | 200 |
| | 1b ⁴ | Sandy GRAVEL | 14 | 1.5 – 2.5 | 2912 | <5 | 14 | 750 |
| Central Piers | 2a | Sandy SILT | 14 | 6 - 8 | 5211 | 20 - 40 | 40 - 70 | 250 |
| | 2b | Sandy GRAVEL | 14 | 6 - 8 | 5211 | 10 - 15 | 40 - 70 | 750 |

¹ Dead Load only considered for settlement analysis.

² Using foundation width 1.5m for abutments and 6 - 8m for central piers.

³ Assuming the silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill

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Table 7 - Robinsons Rd Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | Sandy SILT | 45 | 1.5 – 2.5 | 7550 | 10 - 15 | 60 - 120 | 250 |
| | 1b ⁴ | Sandy GRAVEL | 45 | 1.5 – 2.5 | 7550 | 5 - 10 | 60 - 120 | 750 |

¹ Dead Load only considered for settlement analysis.

² Using foundation width 1.5m for abutments.

³ Assuming silt has not been removed beneath embankment

⁴ Assuming silt has been removed and replaced with hardfill

Memorandum

To: Nik Stewart/Vincent Lobendahn
From: Greer Gilmer / Grant Newby
Copy: Dave Aldridge
Subject: Trents-Blakes Rd Underpass – Revision B

Date: 12 August 11
Our Ref: 3390691

This memo provides soil parameters/stiffnesses for the design of foundation options at the Trents-Blakes Rd Underpass Structure. These have been based on the information provided in email from V Lobendahn on 27th June and 5th July 2011. The memo also includes estimates of settlement and liquefaction effects following assessment of the structures based on loading information provided on 19th July 2011.

Options on the structural form and foundation type to be adopted for Trents-Blakes Rd Underpass Structure should be included in the Scheme Assessment Report. This will need to be reviewed as part of the scheme design.

Our assessment provided below has considered the current preferred structural form option with abutments and piers founded on shallow footings.

The soil model has been based on borehole information at this location, which will need to be reviewed when the project wide model is completed.

1 Ground Conditions

1.1 Soil Profile

A moderately conservative soil profile has been adopted for Trents-Blakes Rd Underpass, with particular emphasis on the geotechnical investigations listed below which are located closest to the structure:

- Cable Tool Boreholes (CT): CT9, CT5, CT6,
- Air Flush Boreholes (AF): AF10-12, AF1-2,
- Test pits (TP): TP26-29, TP9-10

Table 1 – Design Soil Profile

| Soil Layer | Assumed Material Type ⁽¹⁾ | Depth below GL to top of layer (m) | Top of layer RL (m) | Layer Thickness (m) | Design SPT N ₆₀ Values |
|------------|--------------------------------------|------------------------------------|---------------------|---------------------|-----------------------------------|
| 1 | SILT some sand | 0 | 36 | 2 | 22 (22 – 50) |
| 2 | Sandy GRAVEL | 2 | 34 | 5 | 50 (45 – 50) |
| 3 | Sandy silty GRAVEL | 7 | 29 | 2 | 20 (18 – 32) |
| 4 | Sandy GRAVEL | 9 | 27 | - | 50 (20 – 50) |

SPT N₆₀ Values based on AF and CT noted above and values in brackets indicate range.

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1.2 Groundwater

The groundwater levels were measured at between RL 23m and RL 24m during 2010 and 2011 (approximately 12 to 13m below existing ground level) at the proposed location of the Trents-Blakes Rd underpass.

2 Proposed Structural Arrangement

The Trents-Blakes Rd Underpass Structure provides local road access (2 lanes) over the proposed Christchurch Southern Motorway Stage 2 / Main South Road Four Laning (CSM2/MSRFL). A three span concrete bridge is being considered, with two central piers and abutments at each approach. The proposed structure incorporates approach embankments in the order of 7.0m high.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 4, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 2 and a design life of 100 years. The PGA for liquefaction at the Trents-Blakes Underpass are summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

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

¹ NZS 1160.5:2004, Table 3.5 and as advised by N Stewart.

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

The liquefaction potential for the Trents-Blakes Underpass Structure was assessed using SPT data AF1, 2, 10, 11, 12, 15 and CT5, 6, 9 from the site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.44g and SLS PGA 0.11g. Assessment of AF1 indicated liquefaction potential under ULS seismic conditions, in a layer between 14 and 20m below ground surface, although settlements occurring at depths greater than 12m below the ground surface will have limited influence on the behaviour of the structure.

Within this layer liquefaction has been assessed to initiate for seismic PGA greater than 0.25g.

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed that settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided Table 6, and discussed in Section 4.1.

3.3.2 Seismic Settlement

Settlement due to seismic shaking is expected to be in the order of 70-300mm for a ULS event. As potential for liquefaction has only been identified at AF1, potential exists for differential seismic throughout the site of up to 100mm. Negligible seismic induced settlements are expected for a SLS event.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 SILT, and with localised removal of this silt layer down to Layer 2 Sandy GRAVEL and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread though lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within immediate vicinity of the bridge, are assumed to be approximately 4.5m high. The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed

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following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 - Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the Trents-Blakes Underpass will be founded within the approach embankments. The embankment fill material is assumed to be well graded, alluvial sandy gravels, as described above.

Assessment has considered abutments of 14m length (from preliminary structural drawings) with a footing width ranging from 1.5m to 3.5m

4.1.1 Abutment Bearing Capacity

Design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing is founded within fill at a height of 3m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 SILT, and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 6-1a and 1b. The ultimate geotechnical bearing pressures are equal to three times the allowable bearing pressures.

4.1.2 Abutment Footing Settlement

Table 6-1a and 6-1b summarises anticipated settlement beneath the central piers. Settlements of between 5 and 20mm are anticipated.

4.2 Piers

A spread footing option at the piers has been considered for the Trents-Blakes Underpass, considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 SILT and sand.
- Removal of the Layer 1 SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

Excavation beneath the groundwater table is not anticipated for both of these sub options. Refer Table 6-2a and 6-2b.

Assessment has considered foundations for the central piers of 14m in length (from preliminary structural drawings) with a footing ranging in width from 6 to 8m.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing capacities for shallow footings founded within the Layer 1 SILT and Layer 2 Sandy GRAVEL exceed the design pressures. are adequate to satisfy design. Refer Table 6-2a and 6b.

4.2.2 Pier Footing Settlement

Table 6-2a and 6-2b summarises anticipated settlement beneath the central piers. Settlements of the order of 20mm are anticipated where founded on the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles 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 following preliminary structural design.
- For steel driven H piles or tube piles spaced at centres of not less than 4 pile diameters, pile tip settlements of less than 5mm could be expected under SLS loads.
- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|---------------------------|-------------------------------------|
| Trents-Blakes Underpasses | 18 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials.
- Other pile types may be suitable and can be considered if required.

5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

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6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5) and geometry as described below.

Approach Embankment Geometry:

- Height = ~4.5m
- Carriageway Width = 14m
- Slope Batter = 2:1

Cut Slope Geometry

- Max Height = 4.5m
- Slope Batter = 4:1

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 2 | 1.85 | - | - |
| | Static - undrained | 2 | 2.7 | - | - |
| | Seismic | 2 | 0.95 | 0.4 | Negligible |
| Cut | Static | 2 | 3.82 | - | - |
| | Static - undrained | 2 | 4.44 | - | - |
| | Seismic | 2 | 0.87 | 0.31 | ≤20 |

¹ Importance Level 2, Design PGA=0.44g.

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement.

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

7 Recommendations

- Shallow footings would be suitable for the Trents-Blakes Rd Underpass provided the post-earthquake differential settlements can be accommodated by the structure.
- If these settlements cannot be tolerated by the structure then pile foundations would be required.
- At the piers, removal of the typically 2m thick Layer 1 SILT is recommended beneath shallow footings to mitigate bearing failure and to minimise static settlement.
- At the abutments, the shallow footings may be constructed within the fill embankment without removal of the Layer 1 SILT, provided settlements due to embankment fill have occurred, though removal would mitigate risk of differential settlement between the piers and abutments.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ' (°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 ¹ | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy GRAVEL | - | 41 | 0 | 40 | - | 0.22 | 6 | - | - | 10 - 50 | 20 - 100 |
| 3 | Sandy silty GRAVEL | - | 34 | 0 | 30 | - | 0.25 | 4.5 | - | - | 10 - 50 | 20 - 100 |
| 4 | Sandy GRAVEL | - | 41 | 0 | 40 | - | 0.22 | 6 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2.

³ $K=n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$) for piles.

Table 6 - Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | SILT | 14 | 1.5 – 2.5 | 4406 | 15 - 20 | 27 | 250 |
| | 1b ⁴ | Sandy GRAVEL | 14 | 1.5 – 2.5 | 4406 | 5 - 10 | 16 | 750 |
| Central Piers | 2a | SILT | 14 | 6 - 8 | 7590 | 30 - 40 | 60 - 100 | 250 |
| | 2b | Sandy GRAVEL | 14 | 6 - 8 | 7590 | 10 - 20 | 60 - 100 | 750 |

¹ Dead Load only considered for settlement analysis

² Using foundation width of 1.5m for abutments and 6 - 8m for central pier

³ Assuming silt has not been removed beneath embankment

⁴ Assuming silt has been removed and replaced with hardfill

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1.2 Groundwater

The groundwater levels were measured at between RL 17m and RL 20m during 2010 and 2011. Groundwater is approximately 7 to 8.5m below existing ground level at the proposed location of the Marshs Road Structure, and 9 to 11m below existing ground level at the proposed location of the Shands Road.

2 Proposed Structural Alignment

The Marshs Rd and Shands Rd Underpass Structures provide local road access (2 lanes) over the proposed Christchurch Southern Motorway Stage 2/Main South Road Four Laning (CSM2/MSRFL). The proposed structures incorporate approach embankments in the order of 7.0m high.

At Marshs Rd a four span composite steel I-girder bridge is being considered, with three central piers and abutments at each approach. At Shands Rd a three span concrete bridge, with two central piers and abutments at each approach is being considered.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 5, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 2 and a design life of 100 years. The PGA for liquefaction at the Marshs and Shands Underpass Structures are summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

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

¹ NZS 117.5:2004, table 3.5 and as advised by N Stewart

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

The liquefaction potential for the site underlying the Marshs Rd Underpass Structure was assessed using SPT data from the AF4, 8 and CT8, 11 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.48g and SLS PGA 0.11g. The assessment indicates liquefaction potential under ULS seismic conditions in a layer between 9 and 19m below ground surface, although settlements occurring at depths greater than 12m below the ground surface will have limited influence on the behaviour of the structure.

Within this layer liquefaction has been assessed to initiate for seismic PGA greater than 0.2g.

The liquefaction potential for the site underlying the Shands Rd Underpass Structure was assessed using SPT data from the AF3, 16, 17 and CT7, 10 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.48g and SLS PGA 0.11g. Assessment of AF3, 16, 17 and CT7 indicated liquefaction potential under ULS seismic condition, in a layer between 10m and 16m below ground surface although, at depths greater than 12m below ground surface the differential effects on the structure are anticipated to be minor.

Within this layer liquefaction has been assessed to initiate for seismic PGA greater than 0.2g.

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed that settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing option was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided in Table 6 and 7, and discussed in Section 4.1.

3.3.2 Seismic Settlement

At Marshs Rd, settlement due to seismic shaking is expected to occur due to liquefaction within Layer 2 Gravelly SAND and the upper part of Layer 3 Sandy GRAVEL with a magnitude of 40 – 400mm for a ULS event. This could lead to differential settlements between abutments and piers in the order of 300mm. In a SLS event negligible settlements are anticipated.

At Shands Rd these settlements are expected to be in the order of 40 – 360mm for the ULS event and resulting differential settlements in the order of 200mm. Negligible seismic induced settlements are expected for a SLS event.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

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Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 Sandy SILT, and with localised removal of this silt layer down to the Layer 2 Gravelly SAND and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread through lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within immediate vicinity of the bridge, are assumed to be approximately 7.5m high. The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 – Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the Marshs and Shands Underpass Structures would be founded within the approach embankments. The embankment fill material is assumed to be well graded, alluvial sandy gravels, as described above.

Assessment has considered abutments of 14m (Marshs Rd) and 22m (Shands Rd) in length (from preliminary structural drawings) with a footing width ranging from 1.5m to 3.5m

4.1.1 Abutment Bearing Capacity

Design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing is founded within fill at a height of 3m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 Sandy SILT, and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 6-1a and 1b and Table 7-1a and 1b.

The ultimate geotechnical bearing pressure is equivalent to three times the allowable bearing pressure.

4.1.2 Abutment Footing Settlement

Table 6-1a and 1b and Table 7-1a and 1b summarises anticipated settlement beneath the abutment. Settlements ranging from <10 to 60mm are anticipated.

4.2 Piers

A spread footing option at the piers has been considered for the Marshs and Shands Underpasses, considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 SILT and sand.
- Removal of the Layer 1 SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

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Excavation beneath the groundwater table is not anticipated for both of these sub options. Refer Table 6-2a and 2b, and 7-2a and 2b.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing pressures for shallow footings founded within the Layer 1 Sandy SILT and Layer 2 Sandy GRAVEL exceed the design pressures. Refer Table 6-2a, 6-2b, 7-2a and 7-2b.

4.2.2 Pier Footing Settlement

Table 6-2a, 6-2b, 7-2a and 7-2b summarises anticipated settlement beneath the central piers. Settlements of 10 to 20mm are anticipated where founded on the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles can 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 following preliminary structural design.
- 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 5mm are expected under SLS loads.
- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|-----------------------|-------------------------------------|
| Marshs Rd Underpasses | 18 MPa |
| Shands Rd Underpass | 18 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials.
- Other pile types may be suitable and can be considered if required.

Memorandum

5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

Memorandum

6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5), with a geometry which is shown below.

Approach Embankment Geometry:

- Height = ~7m
- Carriageway Width = 14m
- Slope Batter = 2:1

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 2 | 1.75 | - | - |
| | Static - undrained | 2 | 2.5 | - | - |
| | Seismic | 2 | 0.75 | 0.29 | 25 |

¹ Importance Level 2, Design PGA=0.44g.

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement.

7 Recommendations

- Shallow footings would be suitable if post-earthquake settlements can be tolerated by the structure.
- Where these settlements cannot be accommodated within the structures pile foundations will be required.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ'(°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 ¹ | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy GRAVEL | - | 36 | 0 | 30 | - | 0.26 | 6 | - | - | 10 - 50 | 20 - 100 |
| 3 | Gravelly SAND | - | 33 | 0 | 20 | - | 0.3 | 4.5 | - | - | 10 - 50 | 20 - 100 |
| 4 | Sandy GRAVEL | - | 36 | 0 | 30 | - | 0.26 | 6 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2

³ $K=n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$).

Memorandum

Table 6 – Marshs Road Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q_d | Allowable Bearing Pressure (kPa) ² q_a |
|----------------------|-----------------|------------------|-----------------------|----------------------|--------------------|-----------------|---|--|
| Abutments | 1a ³ | SILT | 14 | 1.5 – 2.5 | 4603 | 25 | 29 | 200 |
| | 1b ⁴ | Sandy GRAVEL | 14 | 1.5 – 2.5 | 4603 | <10 | 17 | 750 |
| Central Piers | 2a | SILT | 14 | 6 - 8 | 9208 | 40 - 60 | 80 - 110 | 250 |
| | | SILT | 14 | 6 - 8 | 9837 | 40 - 60 | 80 - 120 | |
| | 2b | Sandy GRAVEL | 14 | 6 - 8 | 9208 | 10 - 20 | 80 - 110 | 750 |
| | | Sandy GRAVEL | 14 | 6 - 8 | 9837 | 10 - 20 | 80 - 120 | |

¹ Dead Load only considered for settlement analysis

² Using foundation width 1.5m for abutments and 6-8m for central piers

³ Assuming silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill

Memorandum

Table 7 – Shands Road Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | SILT | 22 | 1.5 – 2.5 | 6478 | 25 | 26 | 200 |
| | 1b ⁴ | Sandy GRAVEL | 22 | 1.5 – 2.5 | 6478 | <10 | 15 | 750 |
| Central Piers | 2a | SILT | 22 | 6 - 8 | 11,113 | 20 - 40 | 60 - 90 | 250 |
| | 2b | Sandy GRAVEL | 22 | 6 - 8 | 11,113 | 10 - 15 | 60 - 90 | 750 |

¹ Dead Load only considered for settlement analysis

² Using foundation width 1.5m for abutments and 6 - 8m for central piers

³ Assuming the silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill

Memorandum

To: Nik Stewart / Vincent Lobendahn
From: Greer Gilmer / Grant Newby
Copy: Dave Aldridge
Subject: Springs and Halswell Underpass

Date: 1 September 11
Our Ref: 3390691

This memo provides soil parameters/stiffnesses for the design of the foundation options at the Springs and Halswell Junction Roads Underpass Structures. These have been based on the information provided in emails from V Lobendahn dated 27th June and 5th July 2011. The memo also includes estimates of settlement and liquefaction effects following assessment based on structures loading information provided on 19th August 2011.

Options for the structural form and foundation type to be adopted for the Springs and Halswell Junction Roads Underpass Structures should be included in the Scheme Assessment Report. This will need to be reviewed as part of the scheme design.

Our assessment provided below has considered the current preferred structural form option with abutments and piers founded on shallow footings.

The soil model has been based on borehole information at this location, which will need to be reviewed when the project wide ground model is completed.

1.1 Soil Profile

A moderately conservative soil profile has been adopted for the Springs and Halswell Junction Rd Underpasses, with particular emphasis on the geotechnical investigations listed below which are located closest to the structures:

- Cable Tool Boreholes (CT): CT10
- Air Flush Boreholes (AF): AF20, 21, 22, 23, 24, 25
- Test pits (TP): TP21 – 22, 36 - 39

Table 1 – Design Soil Profile

| Soil Layer | Assumed Material Type ⁽¹⁾ | Depth below GL to top of layer (m) | Top of layer RL (m) | Layer Thickness (m) | Design SPT N ₆₀ Values |
|------------|--------------------------------------|------------------------------------|---------------------|---------------------|-----------------------------------|
| 1 | Sandy SILT | 0 | 21 - 24 | 3 | 40 (30 – 50) |
| 2 | Sandy GRAVEL | 3 | 18 - 21 | 4 | 50 |
| 3 | Sandy GRAVEL (saturated) | 7 | 14 - 17 | - | 30 (10 – 50) |

SPT N₆₀ Values based on AF and CT noted above and values in brackets indicate range.

1.2 Groundwater

The groundwater levels were measured at between RL 14m and RL 17m during 2010 and 2011. Groundwater is approximately 6 to 7m below existing ground level at the proposed location of the Springs Road Structure. A moderately conservative groundwater level of 5m below ground level has been adopted.

Memorandum

2 Proposed Structural Alignment

The Springs Rd Underpass Structure provides local road access (2 lanes) over the proposed Christchurch Southern Motorway Stage 2 (CSM2). The proposed structure incorporates approach embankments in the order of 8.0m high. A four span concrete bridge is being considered with three central piers and abutments at each approach.

The Halswell Junction Rd Underpass Structure provides local road access (4 lanes) over the Christchurch Southern Motorway Stage 2 (CSM2). A four span concrete bridge with approach embankments in the order of 7.0m high is being considered, with three central piers and abutments at each approach.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 5, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 2 and a design life of 100 years. The PGA for liquefaction at the Springs and Halswell Junction Rd Underpass Structures is summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

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

¹ NZS 117.5:2004, table 3.5 and as advised by N Stewart

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

Assessments have been undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.44g and SLS PGA 0.11g.

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The liquefaction potential for the site underlying the proposed Springs Rd Underpass Structure was assessed using SPT data from the AF21 - 25 and CT10 site investigations. The assessment indicates liquefaction potential under ULS seismic conditions in a layer between 15 and 19m below ground surface, although settlements occurring at depths greater than 12m below the ground surface will have limited influence on the behaviour of the structure.

Within this layer liquefaction has been assessed to initiate for seismic PGA greater than 0.2g.

The liquefaction potential for the site underlying the Halswell Junction Rd Underpass Structure was assessed using SPT data from the AF20 site investigation. This indicates none of the soil profile is susceptible to liquefaction under ULS and SLS conditions. However, the limited amount of data suggests confirmatory investigations should be undertaken during detailed design.

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed that settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing option was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided in Table 6 and 7, and discussed in Section 4.1.

3.3.2 Seismic Settlement

At Springs Rd, settlement due to seismic shaking is expected to occur due to liquefaction within Layer 2 sandy GRAVEL and Layer 3 Sandy GRAVEL (saturated) with estimates ranging from 20 to 500mm, for the soil profiles identified at investigation locations, under a ULS event. This could lead to differential settlements between abutments and piers in the order of 200 to 300mm. In a SLS event negligible settlements are anticipated.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 Sandy SILT, and with localised removal of this silt layer down to the Layer 2 Sandy GRAVEL and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread through lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within immediate vicinity of the bridge, are assumed to be approximately 8m high at Springs Rd, and 7m high at Halswell Junction Rd. The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed

Memorandum

following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 – Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the Springs and Halswell Junction Underpass Structures would be founded within the approach embankments. The embankment fill material is assumed to be well graded, alluvial sandy gravels, as described above.

Assessment has considered abutments of approximately 22m (Springs Rd) and 42m (Halswell Junction Rd) in length (from preliminary structural drawings) with a footing width ranging from 1.5m to 3.5m

4.1.1 Abutment Bearing Pressure

Design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing is founded within fill at a height of about 5m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 Sandy SILT, and Layer 2 Sandy GRAVEL typically exceed the design pressures. Refer Table 6-1a and 1b and Table 7-1a and 1b.

The ultimate geotechnical bearing pressure is equivalent to three times the allowable bearing pressure.

4.1.2 Abutment Footing Settlement

Table 6-1a and 1b and Table 7-1a and 1b summarises anticipated settlement beneath the abutment. Settlements ranging from <10 to 30mm are anticipated.

4.2 Piers

A spread footing option at the piers has been considered for the Springs and Halswell Junction Underpasses, considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 Sandy SILT and sand.
- Removal of the Layer 1 Sandy SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

Excavation beneath the groundwater table is not anticipated for both of these sub options. Refer Table 6-2a and 2b, and 7-2a and 2b.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing pressures for shallow footings founded within the Layer 1 Sandy SILT and Layer 2 Sandy GRAVEL typically exceed the design pressures. Refer Table 6-2a, 6-2b, 7-2a and 7-2b.

4.2.2 Pier Footing Settlement

Table 6-2a, 6-2b, 7-2a and 7-2b summarises anticipated settlement beneath the central piers. Settlements of 10 to 20mm are anticipated where founded on the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles can 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 following preliminary structural design.
- 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 5mm are expected under SLS loads.
- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|--|-------------------------------------|
| Springs and Halswell Junction Rd Underpasses | 18 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials.
- Other pile types may be suitable and can be considered if required.

5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

Memorandum

6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5), with the geometry described below.

Approach Embankment Geometry:

- Height = 7 - 8m
- Carriageway Width = 22 - 42m
- Slope Batter = 2:1

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 2 | 1.8 | - | - |
| | Static - undrained | 2 | 2.5 | - | - |
| | Seismic | 2 | 0.94 | 0.41 | Negligible |

¹ Importance Level 2, Design PGA=0.44g.

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement.

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

7 Recommendations

- Shallow footings would be suitable if post-earthquake settlements can be tolerated by the structure.
- Where these settlements cannot be accommodated by the structures, piled foundations will be required.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ' (°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy GRAVEL | - | 41 | 0 | 40 | - | 0.22 | 6 | - | - | 10 - 50 | 20 - 100 |
| 3 | Sandy GRAVEL (sat) | - | 36 | 0 | 40 | - | 0.22 | 5 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2

³ $K=n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$).

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Table 6 – Springs Road Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | Sandy SILT | 22 | 1.5 – 2.5 | 4675 | 10 - 15 | 170 | 600 |
| | 1b ⁴ | Sandy GRAVEL | 22 | 1.5 – 2.5 | 4675 | 5 - 10 | 170 | 6868 |
| Central Piers | 2a | Sandy SILT | 22 | 5 | 10,810 | 20 - 30 | 105 | 200 |
| | 2b | Sandy GRAVEL | 22 | 2 | 10,810 | 10 - 20 | 105 | 3004 |

¹ Dead Load only considered for settlement analysis. Includes the weight of pad plus soil above the pad.

² Using foundation width 1.5m for abutments and 5m for central piers

³ Assuming silt has not been removed beneath embankment

⁴ Assuming silt has been removed and replaced with hardfill

Memorandum

Table 7 – Halswell Junction Road Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load (kN) ¹ | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | Sandy SILT | 22 | 1.5 – 2.5 | 7585 | 10 - 15 | 190 | 200 |
| | 1b ⁴ | Sandy GRAVEL | 22 | 1.5 – 2.5 | 7585 | 5 - 10 | 190 | 7704 |
| Central Piers | 2a | Sandy SILT | 22 | 5 | 22,060 | 10 - 20 | 120 | 750 |
| | 2a | Sandy SILT | 22 | 5 | 22,915 | 10 - 20 | 120 | |
| | 2b | Sandy GRAVEL | 22 | 5 | 22,060 | 10 -15 | 120 | 2900 |

¹ Dead Load only considered for settlement analysis. Includes the weight of pad plus soil above the pad.

² Using foundation width 1.5m for abutments and 5m for central piers

³ Assuming the silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill

Memorandum

To: Nik Stewart/Vincent Lobendahn
From: Greer Gilmer / Grant Newby
Copy: Dave Aldridge
Subject: SH1 Interchange – Revision B

Date: 12 August 11
Our Ref: 3390691

This memo provides soil parameters/stiffness for design of the foundation options at the SH1 Interchange Structure. These have been based on the information provided in emails from V Lobendahn dated 27th June and July 5th 2011. The memo also includes estimates of settlement and liquefaction effects following assessment of the structures based on loading information provided on 19th July 2011.

Options for the structural form and foundation type to be adopted for SH1 Interchange Structure should be included in the Scheme Assessment Report. This will need to be reviewed as part of the scheme design.

Our assessment provided below has considered the current preferred structural form option with abutments and piers founded on shallow footings.

The soil model has been based on borehole information at this location, which will need to be reviewed when the project wide model is completed.

1.1 Soil Profile

A moderately conservative soil profile has been adopted for SH1 Interchange Rd Underpass, with particular emphasis on the geotechnical investigations listed below which are located closest to the structure:

- Cable Tool Boreholes (CT): CT1
- Air Flush Boreholes (AF): AF6
- Test Pits (TP): TP1, 40

Table 1 – Design Soil Profile

| Soil Layer | Assumed Material Type ⁽¹⁾ | Depth below GL to top of layer (m) | Top of layer RL (m) | Layer Thickness (m) | Design SPT N ₆₀ Values |
|------------|--------------------------------------|------------------------------------|---------------------|---------------------|-----------------------------------|
| 1 | Sandy SILT | 0 | 43 | 3 | 12 (12 – 40) |
| 2 | Sandy silty GRAVEL | 3 | 40 | - | 30 (25 – 50) |

SPT N₆₀ Values based on AF and CT noted above and values in brackets indicate range.

1.2 Groundwater

The groundwater levels were measured at RL 29m at the proposed location of the SH1 Interchange (approximately 14m below existing ground level).

2 Proposed Structural Arrangement

The SH1 Interchange Structure provides linking road access (2 lanes) over the proposed Christchurch Southern Motorway Stage 2 / Main South Road Four Laning (CSM2/MSRFL). The proposed structures incorporate approach embankments in the order of 7.0m high.

At the SH1 Interchange a four span composite steel I-girder bridge is being considered, with three central piers and abutments at each approach.

3 Geotechnical Considerations

3.1 Geotechnical Parameters

The geotechnical parameters used for design have been derived from a range of in situ and laboratory tests. Generally moderately conservative design parameters have been assigned to the soils, which are considered appropriate for the concept design. Where sufficient site specific testing is unavailable, parameter determination has considered data from similar materials across the site.

The geotechnical parameters are provided in Table 5, which is appended.

3.2 Liquefaction Potential

3.2.1 Seismic Assessment

The loads have been assessed based on a Seismic Importance Level of 3 and a design life of 100 years. The PGA for liquefaction at the SH1 Interchange are summarised in Table 2 below, for assessed soil Class D.

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 shall be taken as 0.33.

Table 2 – Seismic Design Criteria

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

¹ NZS 1170.5:2004, Table 3.5 and as advised by N Stewart

² ULS – Ultimate Limit State

³ SLS – Serviceability Limit State

3.2.2 Liquefaction Assessment

The liquefaction potential for the site underlying the SH1 Interchange Structure was assessed using SPT data from the CT1 and AF6 site investigations.

Assessment was undertaken using the Idris and Bollinger (2006) method, considering a Magnitude 7.5 earthquake, ULS PGA 0.6g and SLS PGA 0.11g. Assessment indicates none of the soil profile is susceptible to potential liquefaction under both ULS and SLS seismic conditions.

Memorandum

3.3 Settlement

3.3.1 Static Settlement

Assessment of static settlement has assumed that settlement induced by the embankment fill is complete prior to construction of structure foundations. Estimate of total embankment settlement is beyond the scope of this memo.

Static settlement of the pier and abutment shallow footing was assessed using the method developed by Burland and Burbidge (1985). A summary of the estimated static settlement is provided in Table 6, and discussed in Section 4.1.

3.3.2 Seismic Settlement

Under a ULS seismic event settlement is expected to be negligible at the SH1 Interchange, with soils assessed to be not susceptible to liquefaction.

3.4 Bearing Capacity

The allowable bearing pressures beneath the abutments and central piers were calculated in accordance with the New Zealand Building Code B1/VM4 method for shallow foundations.

Assessment of bearing capacity has considered both shallow footings founded on or within the Layer 1 Sandy SILT and, with localised removal of this silt layer, down to the Layer 2 Sandy Silty GRAVEL and backfilling with granular embankment fill up to footing level.

Assessment of allowable bearing capacity has assumed load spread though lower material layers at an angle of 30° from vertical.

3.5 Approach Embankment

The approach embankments, within the immediate vicinity of the bridge, are assumed to be approximately 7.5m high. The embankment fill material is assumed to be alluvial sandy gravels, either from a river source or run of pit material from a quarry.

The assessment is based on a well graded, sandy gravel fill material, with adopted parameters of friction angle of 34 degrees and no apparent cohesion. These parameters will need to be reviewed following confirmation of the intended fill source. Higher grade materials will need to be considered where high passive resistance is required from abutment fills.

4 Foundation Option 1 – Shallow Footing on Existing Ground

4.1 Abutments

The abutments of the SH1 Interchange structure will be founded within the approach embankments. The embankment fill material is assumed to be alluvial sandy gravels, as described above and is assumed to be a well graded, sandy gravel fill material.

Assessment has considered abutments of 13m length (from preliminary structural drawings) with a footing width ranging from 1.5 to 2.5m.

Memorandum

4.1.1 Abutment Bearing Capacity

Design pressures exerted by the structure to the ground underlying the footing were assessed assuming that the footing is founded within fill at a height of 3m above the existing ground. Allowable bearing capacities on the embankment fill, Layer 1 SILT, and Layer 2 sandy GRAVEL exceed the design pressures. Refer Table 6-1a and 1b. The ultimate geotechnical bearing pressure (q_u) is equal to three times the allowable bearing pressure.

4.1.2 Abutment Footing Settlement

Table 6-1a and 6-1b summarises anticipated settlement beneath the abutment. Settlements in the order of 10mm are anticipated.

4.2 Piers

A spread footing option at the piers has been considered for the SH1 Interchange considering the following sub options for founding the shallow footing:

- Founded within the Layer 1 SILT and sand.
- Removal of the Layer 1 SILT, founding at the top of the Layer 2 Sandy GRAVEL, or at a higher elevation on an engineered platform of hardfill.

Excavation beneath the groundwater table is not anticipated for either of these sub options. Refer Table 6-2a and 6-2b.

Assessment has considered foundations for the central piers of 13m in length (from preliminary structural drawings) with a footing width ranging from 6m to 8m.

4.2.1 Pier Bearing Capacity

Assessment of bearing capacities identified that the allowable bearing pressures for shallow footings founded within the Layer 1 Sandy SILT, while adequate would not satisfy design settlement criteria.

Allowable bearing capacities for the Layer 2 Sandy Silty GRAVEL however, exceed the design pressures. Refer Table 5-2b.

4.2.2 Pier Footing Settlement

Table 6-2a and 6-2b summarises anticipated settlement beneath the central piers. Settlements of between 10mm and 20mm are anticipated where founded on the Layer 2 Sandy GRAVEL.

5 Foundation Option 2 - Driven Pile Foundation Option

Small diameter piles may be driven into the Layer 2 - Sandy Silty GRAVEL, with the following recommendations:

- Steel driven H piles or tube piles 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.

Memorandum

- 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 following preliminary structural design.
- 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 4mm are expected under SLS loads.
- An ultimate end bearing capacity can be adopted for design as outlined in Table 3 with a strength reduction factor of 0.5 based on undertaking Hiley assessment on all piles. This reduction factor is in accordance with Clause A5.8 and the Transit NZ Bridge Manual. The pile capacities provided are based on a single pile. If pile groups are adopted, pile group effects are to be considered.

Table 3 - Gravel End Bearing Capacity

| Structure | Ultimate End Bearing Capacity (MPa) |
|-----------------|-------------------------------------|
| SH1 Interchange | 12 MPa |

Other considerations/ risks include:

- Variation of layer thickness and strength.
- Vibration effects due to driving on adjacent properties.
- Noise effects due to driving.
- Settlement of piles due to influence of lower strength underlying materials. To be reviewed following receipt of the additional geotechnical investigation data.
- Other pile types may be suitable and can be considered if required.

5.1.1 Pile Parameters

a. Pile Vertical Stiffness

Recommended values of vertical stiffness for the design of the bridge structure are to be calculated using expected soil deflections of 3 mm to 5 mm under working load, and 5 mm to 15 mm under ultimate loads.

b. Pile Vertical Uplift Capacity

The estimated ultimate skin friction available on the piles is likely to be nominal and for preliminary design purposes a value of 10kPa can be adopted.

c. Pile Lateral Stiffness

The passive resistance of the soil is to be modelled using soil springs based on the Horizontal Modulus of Subgrade Reaction parameters provided in Table 5.

6 Slope Stability

The slope stability models were generated using information from the design soil profile (Table 1), geotechnical parameters (Table 5) and geometry as described below.

Approach Embankment Geometry:

- Height = ~7m
- Carriageway Width = 13m
- Slope Batter = 2:1

The results of the slope stability analysis are shown in Table 4 below.

Table 4 - Slope Stability Summary

| | Design Case | Importance Level ¹ | FOS for Design PGA | PGA for FOS=1 | Displacement ² (mm) |
|-------------------|--------------------|-------------------------------|--------------------|---------------|--------------------------------|
| Embankment | Static | 3 | 1.7 | - | - |
| | Static - undrained | 3 | 1.5 | - | - |
| | Seismic | 3 | 0.65 | 0.32 | ≤100 |
| No Movement (5:1) | Seismic | 3 | 0.94 | 0.56 | Negligible |

¹ Importance Level 3, Design PGA=0.6g

² From Ambraseys and Menu (1988). Using a Probability of Exceedence of 5% and unsymmetrical displacement.

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

7 Recommendations

- Shallow footings are considered suitable for the SH1 Interchange based on the geotechnical conditions indicated by the investigations at the site.
- At the piers, removal of the upper 1m Layer 1 Sandy SILT is recommended beneath shallow footings to mitigate excessive static settlements.
- At the abutments, the shallow footings may be constructed within the fill embankment without removal of the Layer 1 Sandy SILT, provided settlements due to embankment fill have occurred, though removal would mitigate risk of differential settlement between the piers and abutments.

Regards,

Greer Gilmer / Grant Newby

Table 5 – Geotechnical Parameters

| Soil Layer | Assumed Material Type | Undrained Strength Parameters ¹ | Drained Strength Parameters | | Elastic Properties | | Pressure Coefficient (Horizontal) ² | | Coefficient Horizontal Subgrade Reaction (MN/m ³) | | | |
|------------|-----------------------|--|-----------------------------|----------|--------------------|----------------------|--|----------------|---|---------|-----------------------------|----------|
| | | | Φ' (°) | c' (kPa) | E' (MPa) | E _u (MPa) | K _a | K _p | K _h | | n _h ³ | |
| | | | | | | | | | Static | Seismic | Static | Seismic |
| 0 | Embankment Fill | - | 34 | 0 | - | - | 0.24 | 6.5 | - | - | - | - |
| 1 | Sandy SILT | 60 - 120 ¹ | - | - | 30 | 60 | - | - | 5 - 10 | 10 - 20 | - | - |
| 2 | Sandy silty GRAVEL | - | 36 | 0 | 40 | - | 0.22 | 5 | - | - | 10 - 50 | 20 - 100 |

¹ Design shear strength from pavement pit investigations and SPT N values (based on the silt not becoming wet).

² Refer BS8002:1994 figures A1 and A2.

³ $K = n_h * z$ (where z=depth)

A range of values are given for the Horizontal Modulus of Subgrade Reaction k_h under static and seismic loading and we recommend sensitivity analyses be undertaken to determine the effect of changes in these parameters based of 50% and 200% of the values provided.

The resistance of the horizontal springs are to be limited in magnitude to three times the passive soil pressure ($3K_p$).

Memorandum

Table 6 –Static Settlement and Bearing Pressure Summary

| | | Geological Layer | Foundation Length (m) | Foundation Width (m) | Net Dead Load ¹ (kN) | Settlement (mm) | Structure Dead Load Bearing Pressure (kPa) q _d | Allowable Bearing Pressure (kPa) ² q _a |
|----------------------|-----------------|------------------|-----------------------|----------------------|---------------------------------|-----------------|--|---|
| Abutments | 1a ³ | Sandy SILT | 13 | 1.5 – 2.5 | 3383 | 5 - 10 | 20 | 150 |
| | 1b ⁴ | Sandy GRAVEL | 13 | 1.5 – 2.5 | 3383 | <5 | 15 | 750 |
| Central Piers | 2a | Sandy SILT | 13 | 6 - 8 | 6990 | 20 - 30 | 60 - 90 | 250 |
| | | Sandy SILT | 13 | 6 - 8 | 7350 | 20 - 30 | 70 - 100 | |
| | 2b | Sandy GRAVEL | 13 | 6 - 8 | 6990 | 15 - 20 | 60 - 90 | 750 |
| | | Sandy GRAVEL | 13 | 6 - 8 | 7350 | 15 - 20 | 70 - 100 | |

¹ Dead Load only considered for settlement analysis

² Using foundation width 1.5m for abutments and 6 - 8m for central piers.

³ Assuming the silt has not been removed beneath embankment

⁴ assuming silt has been removed and replaced with hardfill