Government Life Building, 33 Dee St, Invercargill, 9810

DETAILED SEISMIC ASSESSMENT REPORT

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Revision	Date	Description/Reason for Issue		
B	23/05/2018	Final issue to client – additional information regarding east section		
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Rev. No	Date	Issue Description	Prepared by	Reviewed by
A	23/02/2018	Initial issue to client - final	CC	AM

1.0 Executive Summary

The following report covers the detailed seismic assessment of the Government Life building at the corner of Dee and Esk Streets in Invercargill. The building consists of two distinct structural systems, described as West and East and was constructed circa 1929. The Government Life West section of the building is five storeys high (with no basement) and is approximately 16 m long and 17 m wide giving an approximate footprint of 275 m² at ground floor level. The construction largely comprises steel encased concrete internal columns and reinforced concrete walls with reinforced gravity concrete beams supporting concrete floors.

The Government Life East section of the building is five storeys high (including a basement) with a similar ground floor area to the West section. The overall footprint of the building is therefore approximately 550 m². The basement of Government Life East has a floor area approximately the same as the ground floor. URM parapets cantilever approximately 3m above roof level to a height similar to the fifth floor of Government Life West. The construction of Government Life East consists of largely URM walls with timber floors and steel gravity frames.

For the purposes of this evaluation, the above described building has been assessed as a two monolithic structures of Importance Level 2 (IL2). This assessment has been carried out in accordance with the guidelines as prescribed in 'The Seismic Assessment of Existing Buildings, Technical Guidelines for Engineering Assessments' (July 2017), issued by MBIE et al, referred to as SAEB from herein.

The West section of the building is considered to have a capacity of 10-15% of New Building Standard (NBS). The capacity of the building is limited by concrete wall spandrels.

Loading Direction	%NBS (IL2)
N-S	10-15%
E-W	10-15% (estimated)

The East section of the building is considered to have a capacity of 10-20% of New Building Standard. The capacity of the building is limited by URM spandrels and the parapets

Loading Direction	%NBS (IL2)
N-S	15-20%
E-W	10-20%

BMC notes that the governing elements of the structure are weak poorly detailed concrete spandrels on the West building and URM wall elements on the East building. It was also noted that there is no connection between the walls and timber floor diaphragms on the East building, increasing the vulnerability of the URM elements.

Geotechnical input indicates bearing capacity in the very soft to firm alluvial silt underlying the site is expected to be significantly lower than "good ground. Some areas of the site are expected to liquefy below the water table under ULS loading, but not at SLS loading.

Due to the age (and condition) of the building being in the order of 90 years, we have core drilled a significant number of concrete samples from structural elements and had them tested for strength, chlorides and carbonation by Opus Laboratories in Christchurch and Wellington. In summary the results were: -

- Concrete strengths are very low and vary from 6.5MPa to 20MPa.
- Chloride concentrations are somewhat elevated above normal levels. The chloride concentrations are likely to be from the use of poorly washed marine aggregate within the concrete mix.
- Variable carbonation through the cores, reflects the quality of the workmanship. The maximum carbonation depths likely exceed typical cover depths, indicating depassivation of at least some fraction of the reinforcing steel and hence a current vulnerability to corrosion.

In summary the Government Life Building is earthquake prone and in terms of structural strength and condition is in our opinion not able to be repaired or strengthened without the loss of most of the heritage fabric and values of the building. The ornate cornices and column treatments to the façade all appear to have been formed in reinforced concrete, plastered and painted. The building has not been occupied above ground floor for approximately 35 years and has significant structural and non-structural damage caused by lack of maintenance.

2.0 Introduction

2.1 Objective

Batchelar McDougall Consulting (BMC) Ltd has been engaged by HWCP Management Ltd to carry out a detailed structural assessment (DSA) for the Government Life Building at 33 Dee Street, Invercargill. The assessment has been undertaken in accordance with the Ministry of Business, Innovation and Employment's (MBIE) Technical Guidelines for Engineering Assessments titled 'The Seismic Assessment of Existing Buildings' (SAEB) and dated July 2017.

2.2 Scope of Work

BMC have been engaged to carry out the following scope of works:

- Review available drawings for the building to determine the nature of the design, primary structural characteristics, and adequacy of the lateral load resisting systems.
- Walk around the building to familiarise ourselves with the structure, visually assess its condition, observe important structural and seismic characteristics, and note obvious deficiencies.
- Undertaken intrusive investigations to determine floor slab thicknesses, element sizes and scan concrete elements for reinforcement provisions.
- Engage OPUS Laboratories to undertake concrete core compression testing and chloride and carbonation testing.
- Carry out a DSA to determine the likely seismic performance of the building
- Identify concept strengthening strategy (if appropriate) or justify demolition
- Provide a DSA report documenting our findings and recommendations

2.3 Information used for the assessment

The information used for this assessment is summarised in bullet point format as follows:

- Photos of some of the original drawings of variable quality by B J Ager dated 1929
- Alteration drawings by Barham & Barham Architects dated 1966
- Alteration drawings by Gray Hesselin & Baxter Architects dated 1982
- Structural drawings of prior strengthening by G M Designs: McMillan Consulting Engineers Ltd dated 2002
- Visual survey undertaken and indicators of defects present at the time (including opening up of some hidden critical areas)
- Test results for core samples taken from the building

2.4 Inspection

A team of BMC Engineers visited the site on 27rd November 2017, 11th December 2017 and again on the 15th December 2017. During these visits BMC engineers undertook a damage assessment and undertook a limited site measure to provide information not found in limited drawings.

2.5 Limitations

Findings presented as a part of this report are for the sole use of HWCP Management Ltd in its evaluation of the subject property. The findings are not intended for use by other parties, and may not contain sufficient information for the purposes of other parties or other uses.

This assessment has been restricted to structural aspects only. Waterproofing elements, electrical and mechanical equipment, fire protection and safety systems, service connections, water supplies and sanitary fittings have not been reviewed, and secondary elements such as windows and fittings have not generally been reviewed.

Limited documentation was provided to BMC therefore assumptions have been made based on site observations and era of construction.

Assumptions have been made as to the likely connections used, based on the observed area of construction. Further invasive investigations would be required to observe all of these hidden connections.

Our professional services are performed using a degree of care and skill normally exercised, under similar circumstances, by reputable consultants practicing in this field at this time. No other warranty, expressed or implied, is made as to the professional advice presented in this report.

BMC have commissioned GeoSolve to provide a Desktop Study for the entire CBD redevelopment block, refer to Section 7.0 of this report for recommendations and the Geotechnical Report that we have obtained.

Assessment on earthquake only loads. No other load cases have been considered.

3.0 Statutory Requirements

3.1 Building Act incorporating The Building (Earthquake-prone Buildings) Amendment Act 2016

3.1.1 Earthquake Prone Building Policy - Section 133

The Building (Earthquake-prone Buildings) Amendment Act was passed into law by Parliament on the 10th of May 2016 and came into effect on 1 July 2017 (now embedded in the Building Act). Some of the significant changes from the previous requirements are outlined below.

3.1.1.1 Definition of 'Earthquake-prone'

The Building Act changes the definition of 'Earthquake-prone Building' by:

- Clarifying that an Earthquake-prone Building can be one that poses a risk to people on adjoining properties and not just those within the building itself;
- Excluding from the definition of Earthquake-prone Building certain residential housing, farm buildings, retaining walls, wharves, bridges, tunnels and monuments;
- Included in the definition of Earthquake-prone Building are hostels, boarding houses and residential housing that is more than two stories and contains three or more household units.

3.1.1.2 Seismic Risk

Different locations are assigned different 'seismic risk' as shown in Figure 1. The new regulations identify three different categories defined by the seismic hazard factor (Z) in the New Zealand Loadings Code (NZS 1170):

- High seismic risk – Z greater than or equal to 0.30
- Medium seismic risk – Z between 0.15 and 0.30
- Low seismic risk – Z lower than 0.15

The seismic risk relates to timeframes for strengthening and identification of potentially Earthquake-prone buildings. The Government Life Building is in a medium Seismic Risk Area.

3.1.2 Priority Buildings

Priority buildings are defined as buildings that:

- Are generally used for health or emergency services or used as educational facilities.
- Contain unreinforced masonry that could fall on to busy thoroughfares in an earthquake – such as parapets.
- The Territorial Authority has identified as having the potential to impede strategic transport routes after an earthquake.

Priority buildings have shorter timeframes for identification and strengthening of Earthquake-prone Buildings. The Government Life Building is classed as a priority building as it comprises unreinforced masonry parapets which may potentially fall onto busy thoroughfares in an earthquake.

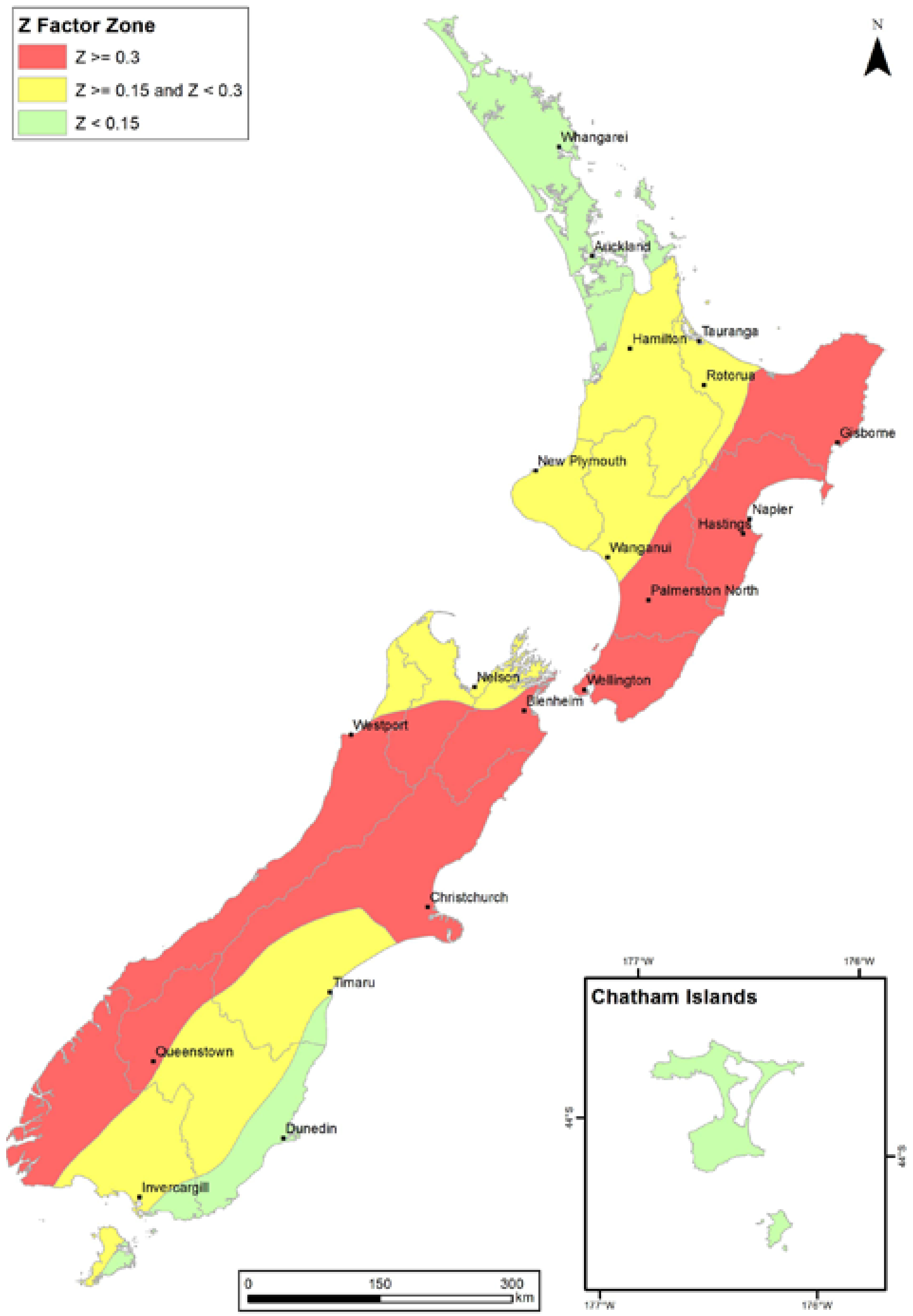

Figure 1 - Three seismic risk areas in map format (map produced by GNS Science)

3.1.2.1 Timeframes for Identifying Earthquake-prone Buildings

The Building Act contains maximum timeframes for Territorial Authorities to assess and identify potentially Earthquake-prone Buildings as outlined below.

High seismic risk areas:

- High Priority buildings 2.5 years
- All other buildings 5 years

Medium seismic risk areas:

- High Priority buildings 5 years
- All other buildings 10 years

Low seismic risk areas:

- All buildings 15 years

The timeframes set out above and in Figure 2 commenced on 1st July 2017.

Seismic risk area	TAs must identify potentially earthquake-prone buildings within:		Owners must strengthen or demolish earthquake-prone buildings within:	
	Priority	Other	Priority	Other
High	2 ½ years	5 years	7 ½ years	15 years
Medium	5 years	10 years	12 ½ years	25 years
Low	n/a	15 years	n/a	35 years

Figure 2 - Time frames for the identification and remediation of earthquake-prone buildings

Following identification by the Territorial Authorities, building owners are required to provide an engineering assessment of the building within twelve months. Upon receipt of the engineering assessment the Territorial Authority decides whether the building should be classified as Earthquake-prone. The ICC must issue an Earthquake-prone Building notice when it determines that a building or part of a building is earthquake-prone.

The Government Life Building will be required to be demolished or strengthened in 12.5 years according to the building act.

3.1.2.2 Timeframes for Strengthening Earthquake-prone Buildings

The amended Act contains maximum timeframes for strengthening Earthquake-prone Buildings after notice has been issued by the Territorial Authority as outlined below.

- High seismic risk areas 15 years
- Medium seismic risk areas 25 years
- Low seismic risk areas: 35 years

3.1.3 Building Alterations (Section 112)

Under the Building Act:

- Alterations to Earthquake-prone Buildings may be allowed even if after those alterations the building will not comply with the provisions of the Building Code that relate to means of escape from fire and disabled access. The Territorial Authority must be satisfied that the proposed alteration would contribute towards making the building no longer Earthquake-prone and that carrying out other upgrades would be unduly onerous on the owner;
- The Territorial Authority will be able to require the owner to carry out strengthening works in addition to other alterations where the alterations are 'substantial alterations'. The definition of 'substantial alterations' is more than 25% of the ratable value.

3.1.4 Change of Use (Section 115)

This section requires that the territorial authority is satisfied that the building with a new use complies with the relevant sections of the Building Code 'as near as is reasonably practicable'.

This is typically interpreted by territorial authorities as being 100% of the strength of an equivalent new building or as near as practicable.

3.1.5 Heritage Status

The building is listed by Heritage New Zealand Pouhere Taonga as a Historic Place Category 2. It is also listed in the Proposed Invercargill City District Plan as part of the heritage record. Section 3.8 of the District Plan sets out the planning requirements for repairs and maintenance of the building which is a permitted activity, demolition, however is a non-complying activity.

3.2 Building Code

The Building Code outlines performance standards for buildings and the Building Act requires that all new buildings comply with this code. Compliance Documents published by The Department of Building and Housing can be used to demonstrate compliance with the Building Code.

4.0 The Site

4.1 Site Location

The Government Life building is situated at the corner of Esk Street and Dee Street, Invercargill (refer Figure 3). The site has two street frontages, (Dee Street, SH6) to the West and (Esk Street) to the North. The East and South sides of the building are directly adjacent to other buildings.

4.2 Site Description

The site is rectangular in shape and is approximately 35 m long and 17 m wide, thereby occupying a footprint of approximately 550 m². The site is flat and sits approximately 10 m above sea level.

The site is developed by a five storey, category two heritage building. The building comprises numerous retail stores at the ground floor while the upper floors have remained unoccupied over the last 35 years. Ground floor occupancies can be accessed from the street and access to upper floors is solely through the back of the corner convenience store. Numerous retail and commercial style buildings of similar construction are located on the same block bounded by Esk Street, Kelvin St, Dee St (SH6) and Tay St (SH1).

The site can be accessed from both Esk Street and Dee Street.

4.3 Surrounding Land Use

The site is located within the Invercargill central business district (CBD). The vast majority of the surrounding buildings are a mixture of single and double storey unreinforced masonry (URM) retail and commercial buildings.

Immediately behind the building (to the South) is a single storey URM building currently occupied by a Reading Cinemas. Directly to the East are a series of single storey URM retail stores. Esk Street to the North of the building is a largely pedestrian street.

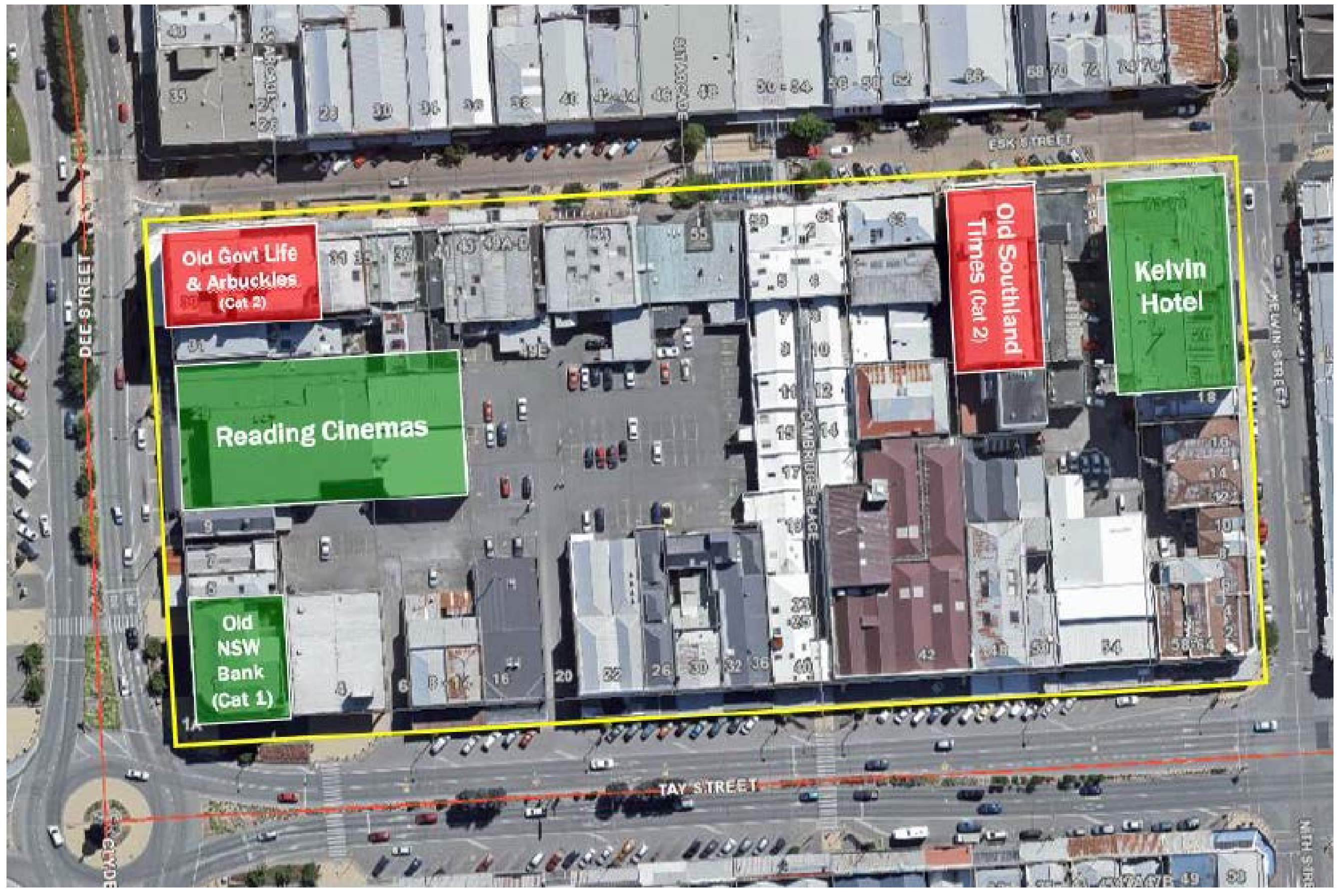

Figure 3 - Google maps satellite image of the site

Figure 4 – View of site from the south on SH6

5.0 Building Description

The Government Life Building is a five-storey building holding a prime location on the Esk and Dee St corners and was constructed circa 1929. The structure comprises two distinct areas; Government Life West and Government Life East as shown in Figure 5.

5.1 Building Form

The Government Life West section of the building is five storeys high (with no basement) and is approximately 16 m long and 17 m wide giving an approximate footprint of 275 m² at ground floor level. The construction largely comprises steel encased concrete columns and reinforced concrete walls, with reinforced concrete gravity beams supporting concrete floors.

The Government Life East section of the building is five storeys high (including a basement) with a similar ground floor area to the West section. The overall floor area of the building is therefore approximately 550 m². The basement of Government Life East has a floor area approximately the same as the ground floor. URM parapets, approximately 3m high project above roof level to match the fifth-floor height of Government Life West. The construction consists of URM walls with timber floors and gravity steel frames.

5.2 Secondary Features

The secondary building structural systems are described in the following section of the report but some of the key features are described as follows.

5.2.1 Stairs

The building incorporates two stairwells. One of timber construction located centrally within the structure. This stair wraps around a steel framed lift void. The second is the main fire egress stair comprising and concrete stair and landing supported on URM walls and located at the far South East corner of the structure.

5.2.2 Fire Escape

An external fire escape is located on the West side of the building. The fire escapes are constructed from steel members. These were deemed unsafe onsite and hence are no longer in use.

5.2.3 Fifth Floor Safe

A safe comprising 170 mm thick reinforced concrete walls is located in the East section of the West building. The safe walls land on the 330 mm thick reinforced concrete floor. Observations of the supporting structure revealed no additional beams or supporting structure beneath the concrete safe walls.

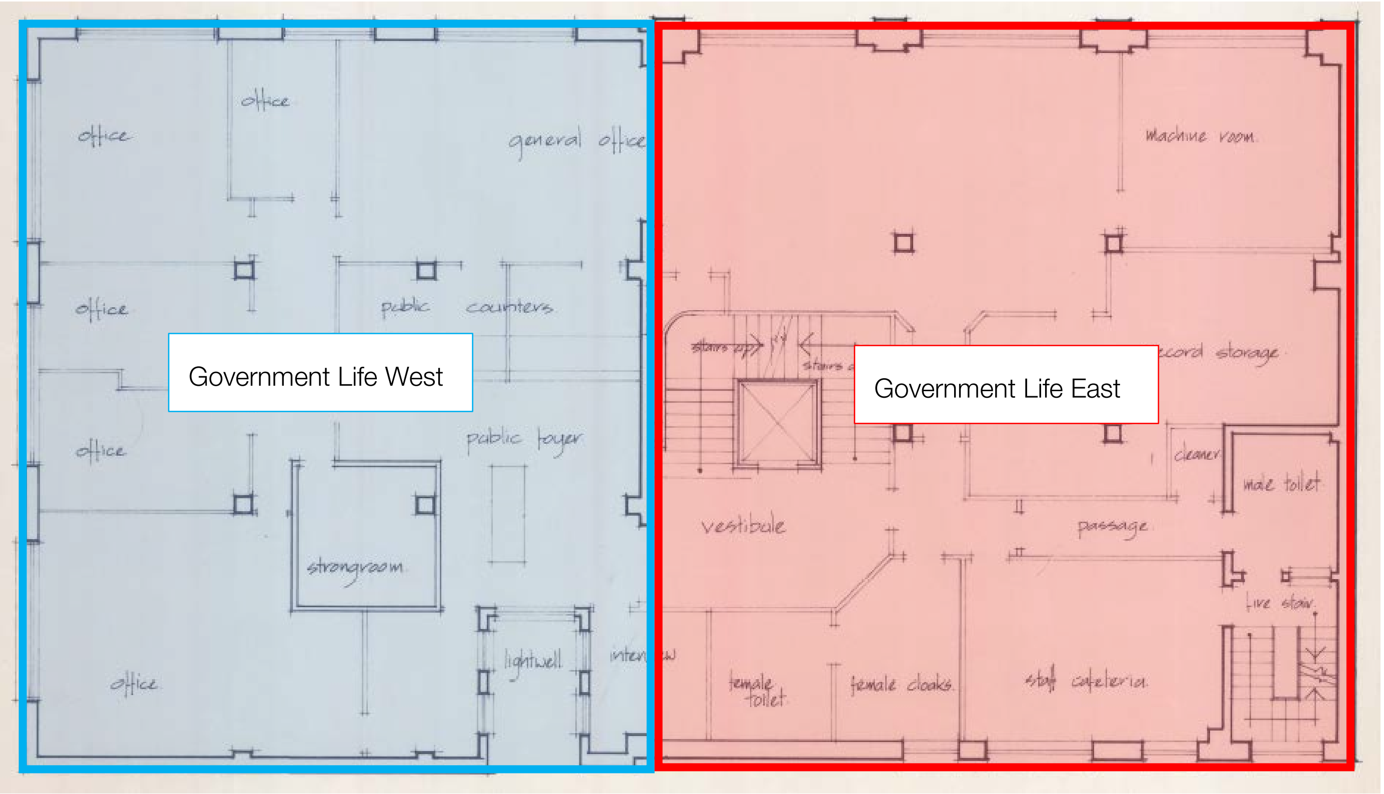

Figure 5 – Government Life building general arrangement plan

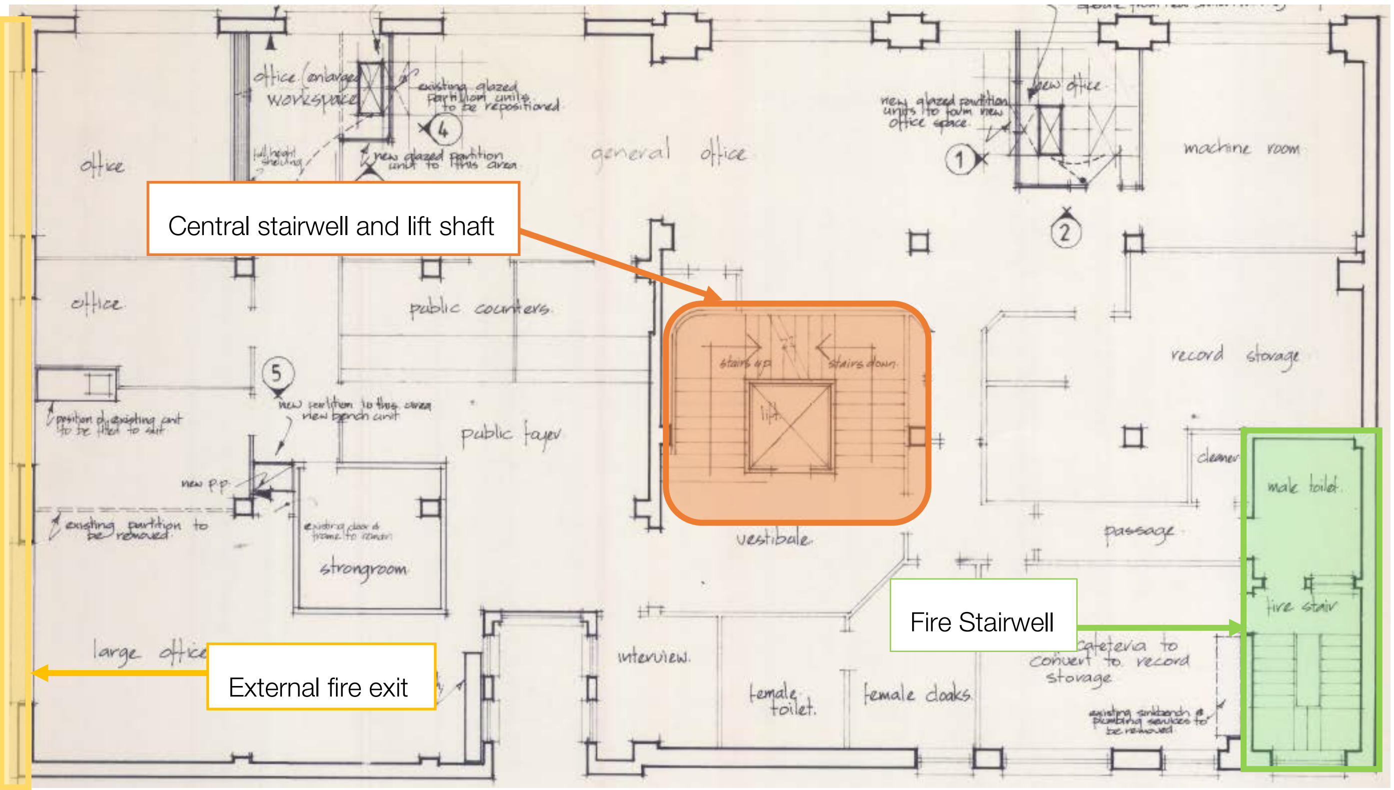

Figure 6 - Location of secondary features

6.0 The Structure

6.1 Gravity load resisting system

6.1.1 Government Life West

Roof Level

→ URM parapet
→ Steel portal frames

Fifth Floor

→ 330 mm thick insitu reinforced concrete floor
→ Reinforced concrete beams along length of building in both directions
→ Insitu reinforced concrete walls and steel CHS posts supporting roof structure

First, Second, Third and Fourth Floor

→ 150 mm thick insitu reinforced concrete floor
→ 250 mm wide by 450 mm deep reinforced concrete beams across building
→ 300 mm wide by 330 mm deep reinforced concrete beams along building
→ Reinforced concrete wall and concrete encased steel columns to interior and reinforced concrete columns to external walls
→ 200mm wide by 1390 mm deep reinforced concrete spandrels to west and north elevations
→ 700 mm wide by 700 mm deep concrete encased steel beam across the shop fronts

Ground Floor

→ Steel posts/columns and SHS strut bracing

6.1.2 Government Life East

Roof Level

→ URM parapets
→ Steel beams with timber purlin roof structure

Ground, First, Second, Third and Fourth Floor

→ 350 mm by 50 mm timber joists at 400 mm centres
→ 360 mm by 152 mm steel I beams across building
→ 250 mm by 130 mm steel I beams along building
→ 225 mm by 175 mm steel I columns and URM walls

Basement

→ Insitu reinforced concrete columns and concrete masonry walls

6.1.3 Foundations

Due to the lack of documentation, assumptions have been made in regards to the foundation system. The foundation system is assumed to comprise reinforced concrete strip footing beneath columns and walls.

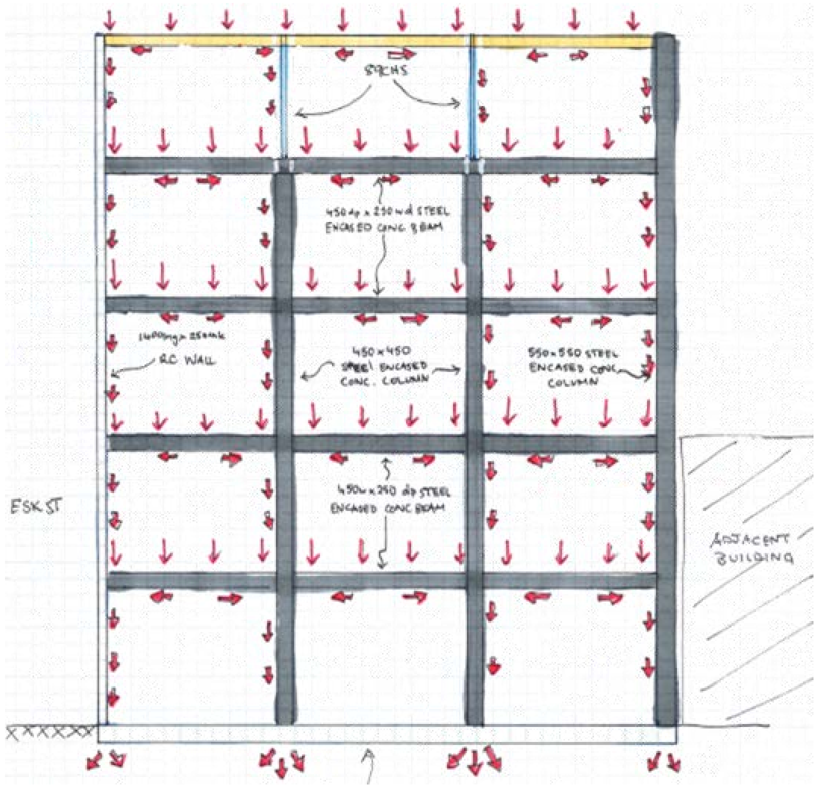

Figure 7 – Government Life West typical gravity load path

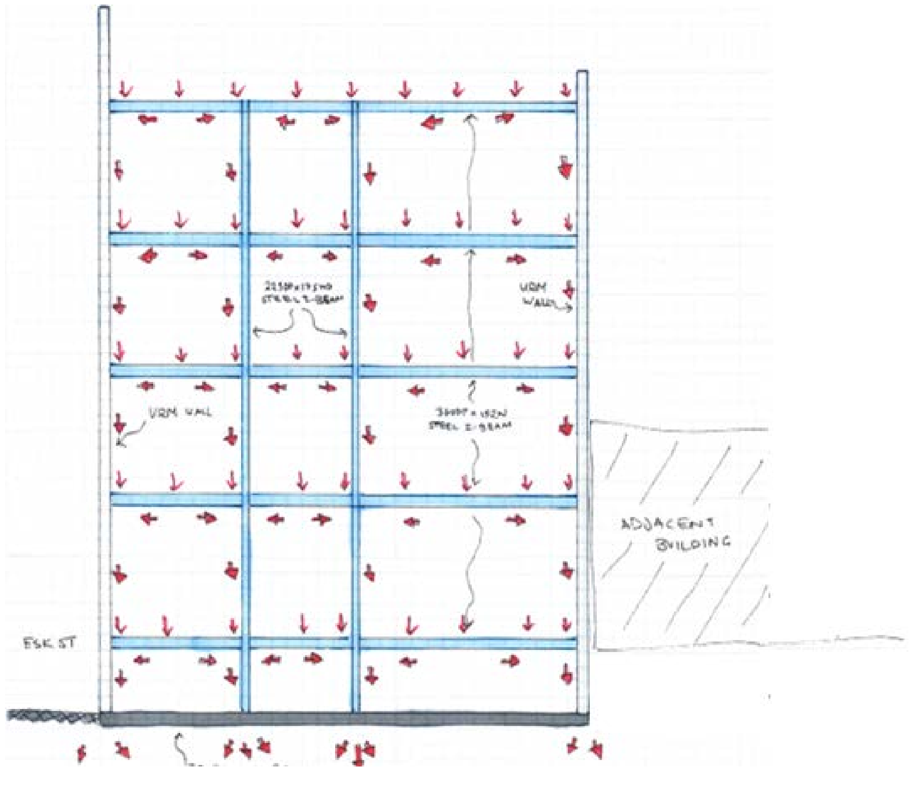

Figure 8 – Government Life East typical gravity load path

6.2 Lateral load resisting system

6.2.1 Government Life West

6.2.1.1 Transverse direction

Roof Level

→ Plasterboard ceiling diaphragms
→ Steel portal frames
→ Insitu reinforced concrete walls

Fifth Floor

→ Rigid diaphragm formed by 330 mm thick insitu reinforced concrete floor
→ Moment frames comprising concrete encased steel beams/columns and reinforced concrete walls
→ Spine shear wall comprising URM

First, Second, Third and Fourth Floor

→ Rigid diaphragm formed by 150 mm thick insitu reinforced concrete floor
→ Moment frames comprising concrete encased steel beams/columns and reinforced concrete walls
→ Spine shear wall comprising URM

Ground Floor

→ Rigid diaphragm formed by 150 mm thick insitu reinforced concrete floor
→ Steel posts/columns and SHS strut bracing

6.2.1.2 Longitudinal direction

The longitudinal lateral load resisting system is similar to the transverse lateral load resisting system other than there are only concrete encased steel frames and reinforced concrete shear walls. The URM shear wall provides insignificant lateral resistance in the longitudinal direction

6.2.2 Government Life East

The lateral load resisting system comprises URM walls in the transverse direction and a URM wall and frame in the longitudinal direction. No diaphragm action can be relied on due to the lack of positive fixing between the timber floors and the URM elements. Lateral load will be transferred to the URM elements based on tributary width. The stiffer elements (URM walls) will likely attract a larger proportion of lateral load, as indicated in Figure 10. The contribution of the steel frames to the lateral load resisting system will be insignificant due to their flexible nature. This therefore induces eccentricity to the structure with the potential for failure. The URM walls are required to span out-of-plane between walls or frames but are limited by their fixing capacities.

This section of the building lacks a resilient lateral load path due to the lack of sufficient diaphragm action (stiffness and fixity) when out-of-plane loads are applied. This creates the potential for wall collapse out from the building.

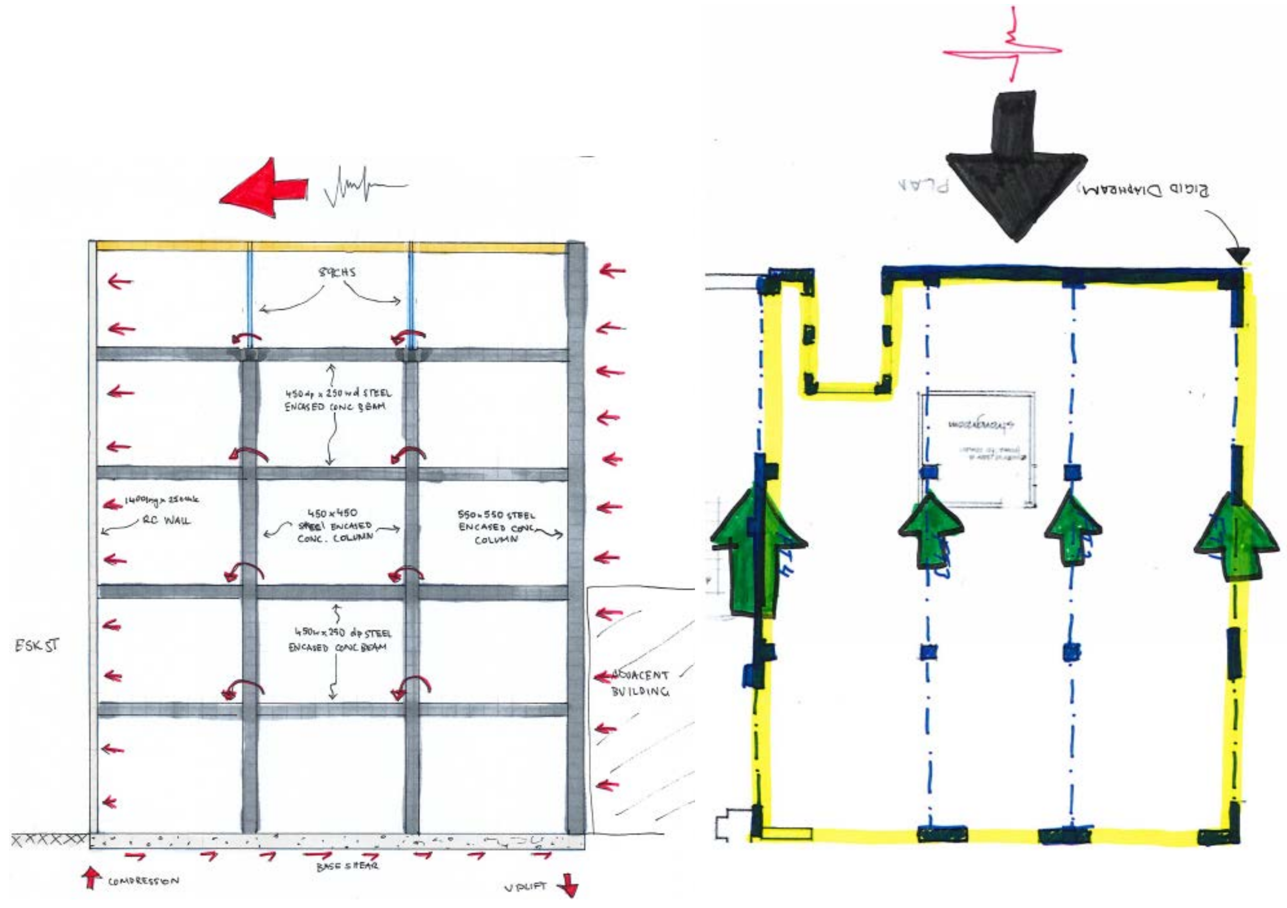

Figure 9 – Government Life West typical transverse lateral load path (elevation and plan view)

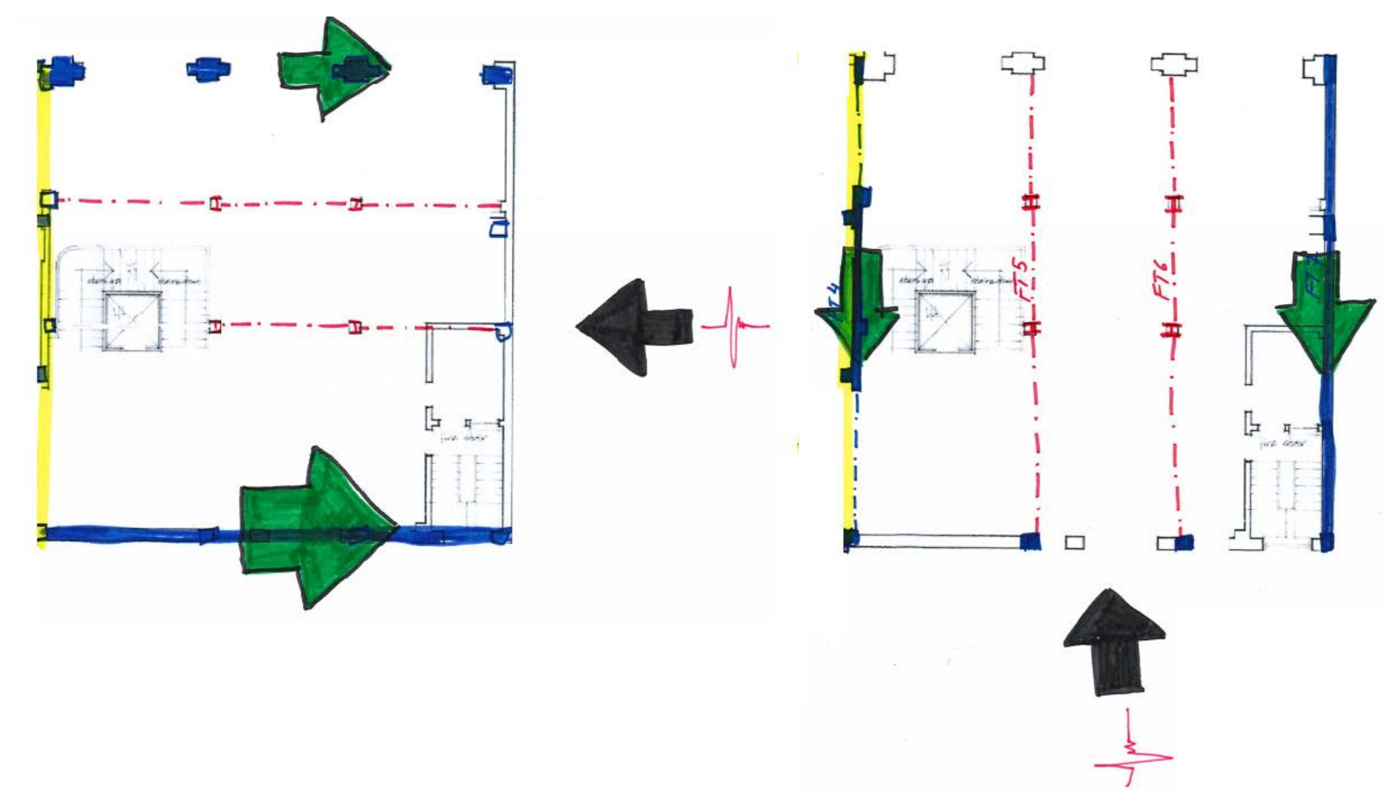

Figure 10 – Government Life East longitudinal lateral load resisting system (left) and transverse lateral load resisting system (right)

7.0 Geotechnical Considerations

A geotechnical desktop study was carried out by GeoSolve Ltd during February 2018 (reference: 171019), see Appendix D.

The report was written for the Invercargill CBD Project involving both the Government Life Building and the Southland Times Building. A desktop study was deemed sufficient to assist with the structural assessment undertaken by Batchelar McDougall Consulting Ltd.

No site-specific investigations have been undertaken for the purpose of this report. GeoSolve have completed a review of shallow and deep site investigations in close proximity to the sites in central Invercargill to infer the underlying geological model. Class D soil type with 'susceptibility to liquefaction' at ULS seismic events have been used in the analysis.

7.1 Ground Conditions

The subsurface soils underneath the Government Life Building are inferred to comprise:

- Uncontrolled fill/ engineered fill, overlying;
- Alluvial silt, overlying;
- Alluvial sand, overlying;
- Alluvial gravel.

The groundwater level was observed between 1.4 m and 3.3 m bgl in the area. Further site-specific investigations would be required to confirm the groundwater levels.

7.2 Liquefaction Assessment

The liquefaction analysis from surrounding sites indicates there is typically no potential for liquefaction or lateral spreading under SLS seismic loading, however minor liquefaction is predicted under ULS loading at some sites in the area i.e. loose sand lenses overlying or within the alluvial gravel unit have the potential to liquefy below the water table under ULS seismic loading.

7.3 Foundations

It is understood the Government Life Building's foundations are likely to comprise strip footings bearing upon alluvial silt. Bearing capacity within the very soft to firm alluvial silt underlying the site is expected to be significantly lower than "good ground". The basement foundation is expected to bear on the underlying alluvial gravel or a thin layer of alluvial silt overlying alluvial gravel.

Strip footings (500 mm wide by 500 mm deep) within the alluvial silt are expected to have a geotechnical ultimate bearing capacity of 120 kPa. Footings (400 mm wide by 400 mm deep) upon the alluvial gravel have an expected higher geotechnical ultimate bearing capacity of 300 kPa, see Figure 11 and Figure 12. Note low bearing for wider footings.

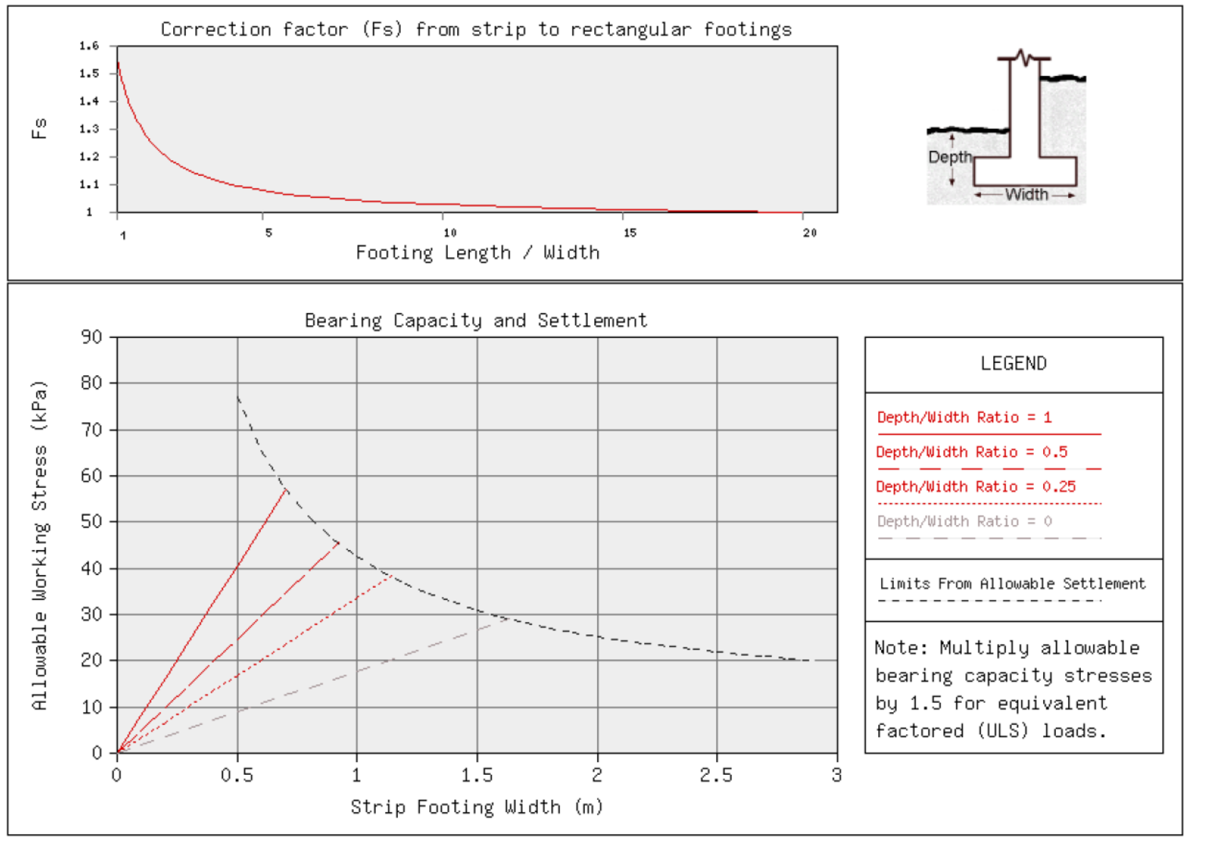

Figure 11 – Typical bearing for shallow footings on alluvial silt (excerpt from GeoSolve report)

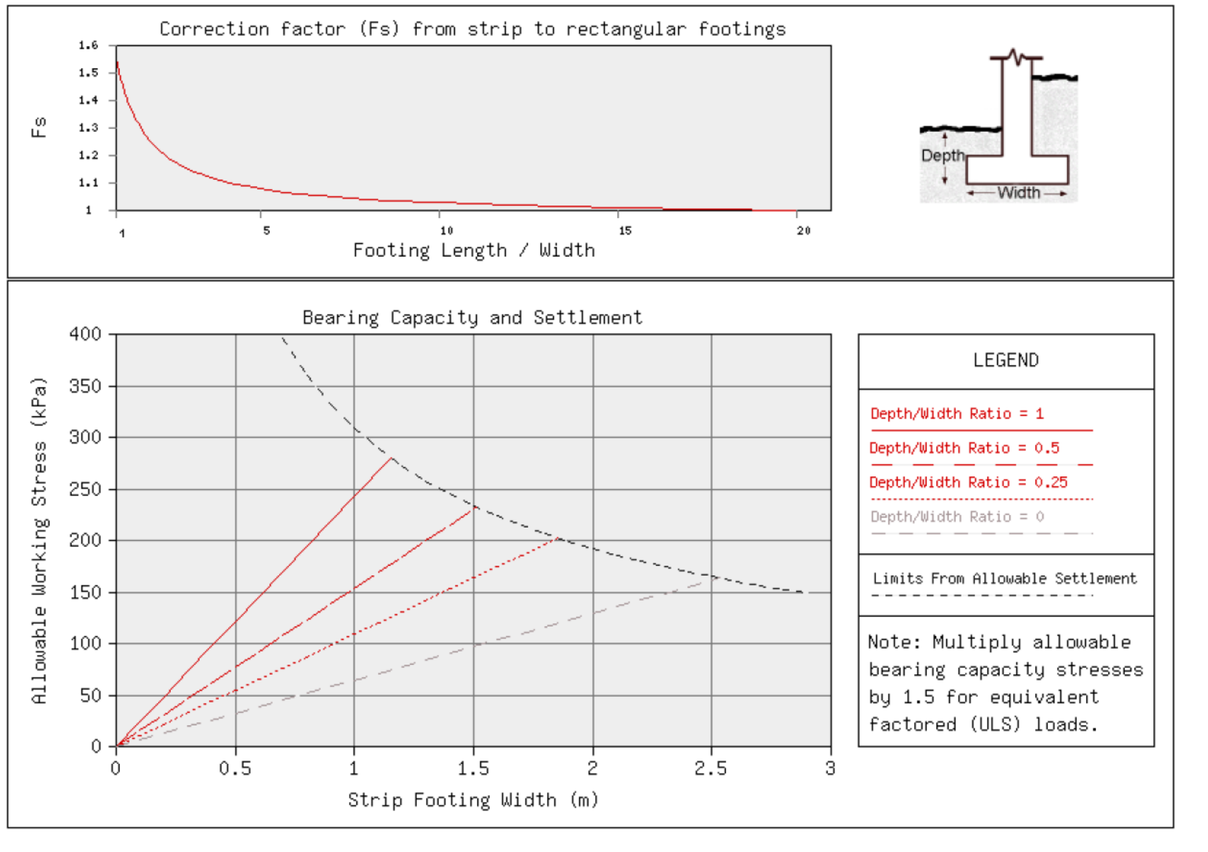

Figure 12 - Recommended bearing for shallow footings on alluvial gravel (excerpt from GeoSolve report)

8.0 Seismic Assessment Parameters

8.1 Material Properties

The following structural and geotechnical material properties have been used to carry out this seismic assessment. No structural specification for the original construction has been made available to BMC, so parameters have typically been taken from industry guidance and testing, see references below:

Material	Element	Property	Assigned Value	Notes/comments/assumptions
Concrete	Steel encased concrete columns	28 day compressive strength, f'c	6.5 MPa	OPUS Concrete compression test report, ref CH3667, dated 08/01/2018
	Reinforced concrete beams	28 day compressive strength, f'c	8.5 MPa	
	Wall elements	28 day compressive strength, f'c	16 MPa	
	Spandrel elements	28 day compressive strength, f'c	20 MPa	
	Floor elements	28 day compressive strength, f'c	12 MPa	
	All concrete members	Modulus of elasticity	13-20 GPa	NZS 3101: Part 1:2006 clause 5.2.3
Steel Reinforcement	R bars	Lower characteristic yield strength	227 MPa	SAEB Part C Appendix C5-19 table C5C.1
		Probable yield strength	272 MPa	SAEB Part C Section 5 C5.4.3.2
	All reinforcing steel	Modulus of elasticity	200 GPa	SAEB Part C Section 5 C5.5.4.3.3
Structural Steel	Existing frame members	Lower characteristic yield strength	210 MPa	SAEB Part C Appendix C6-10 table C6B.10
		Probable yield strength	231 MPa	SAEB Part C Section 6 table C6.2
	Strengthening frame members	Lower characteristic yield strength	230 MPa	SAEB Part C Appendix C6-10 table C6B.10
		Probable yield strength	264 MPa	SAEB Part C Section 6 table C6.2
	All structural steel	Modulus of elasticity	205 GPa	NZS 3404: Part 1:1997
Unreinforced Masonry (Brick)	All URM elements	Probable compressive strength, f'm	12 MPa	SAEB Part C Section 8 table C8.5
		Modulus of elasticity	3.6 GPa	SAEB Part C Section 8 C8.7.6

8.1.1 Importance Level

For the purposes of consideration of loading, the structure been classified as Importance Level 2 (IL2) in accordance with AS/NZS 1170.0:2002.

8.1.2 Design Working Life

The Government Life building has been assumed to have been constructed with a Design working life of 50 years. Together with the Importance Level assigned above, this has been used to determine the annual probability of exceedance for ultimate limit states, including earthquake loads, in accordance with NZS 1170.0:2002, table 3.3.

8.2 Seismic Loading

The seismic loads used in this assessment are based on the provisions of the current loadings standard NZS1170.5:2004.

Seismic Parameter	Values	Notes/References/Comments
Soil category:	D	NZS1170.5.2004 Table 3.1
Hazard factor Z:	0.17	NZS1170.5.2004 Clause 3.1.4
Return period factor Ru:	1.0	NZS1170.5:2004 Clause 3.1.5
Near-fault factor N(T,D):	1.0	NZS1170.5:2004 Clause 3.1.6

Please note: The performance of the building under 'Serviceability' (SLS) seismic loads has not been addressed. A review of GNS Strong Motion Data for Invercargill (earliest record 1994) shows a Peak Ground Acceleration (PGA) of 0.03g (20 mm displacement) in the 2009 Milford Quake which is less than the expected ULS event.

Recent research under a ULS Alpine Fault event the expected strong motion shaking duration in Invercargill is approximately 45 seconds.

8.2.1 Seismic Weight

The seismic weight has been calculated in accordance with NZS 1170.5:2004 clause 4.2 based on a load combination of dead plus seismic live load.

Building Area	Seismic Weight (kN)	Area of ground floor footprint (m2)	Equivalent area load (kPa)
Government Life West	10,000	270	37
Government Life East	6,900	270	25

9.0 Seismic Assessment Procedure

9.1 Analysis Procedure overview

9.1.1 Government Life West

The structural analysis was completed in accordance with C5 of the SAEB technical guidelines, section C5.8 Global capacity of Dual Frame-Wall Concrete Buildings.

9.1.1.1 Modelling assumptions

The following assumptions have been made when modelling the building:

1. The rigid diaphragm at each floor has adequate strength to transfer these loads
2. The steel encased concrete and reinforced concrete elements and masonry wall elements are in good condition (i.e. no cracking). This assumption is not entirely valid with some cracking and spalling evident.
3. The soils which the walls are founded on have adequate bearing capacity to resist over turning. This assumption would need to verified by a site specific Geotechnical Engineering assessment of soil bearing capacity.

9.1.1.2 Primary transverse system

The primary transverse system of the existing Government Life West building consists of three concrete frames and one URM wall. Loads are distributed at each floor through rigid diaphragm action provided by reinforced concrete floors. The concrete frames comprise a mixture steel encased concrete beams/ columns and reinforced concrete beams, columns and walls. Additional reinforced concrete wall elements, such as window mullions, have been excluded from the system as the lateral load capacity they provide is insignificant. The seismic weight and rigid diaphragm associated with this building section is outlined in yellow, in Figure 13.

Two separate two-dimensional (2D) SAP2000 computer models were constructed to investigate the behaviour of each frame type in the transverse lateral load resisting system. The lateral seismic forces are assumed to be distributed over the building height in accordance with Section 6 of NZS 1170.5:2004 and the corresponding internal forces and building displacements are determined using a linear elastic static analysis. Computer model extracts of the 2D frame models' displacements are provided adjacently in Figure 14.

<u>Torsional Analysis</u>

For buildings with rigid diaphragms it is necessary to consider the torsional amplification effect arising from the demand and resistance eccentricities and the location of the centre of strength. Method A: Elastic torsion response from the SAEB technical guidelines Section C2F.2 was used to assess the rigid reinforced concrete diaphragm. This method uses the elastic force-based procedure and linear analysis techniques, therefore only the consideration of accidental torsion is required.

The torsional assessment determined the proportion of load required to be resisted by each frame line. This assessment determined the demand on the SAP2000 frame model.

Component examination was carried out to determine the capacity of specific elements. These checks outlined critical structural weaknesses in the building. These checks were undertaken using in-house BMC spreadsheets and the assumed component detailing is outlined in Appendix A and the following sections summarise these results.

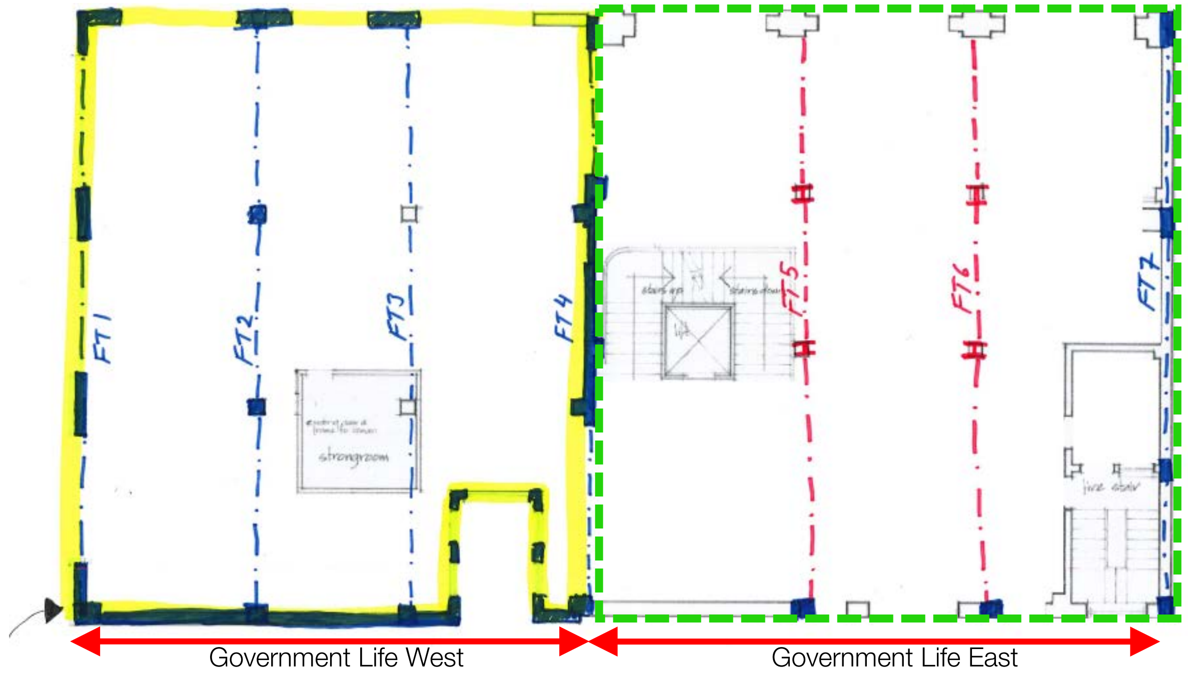

Figure 13 - Transverse lateral load resisting system

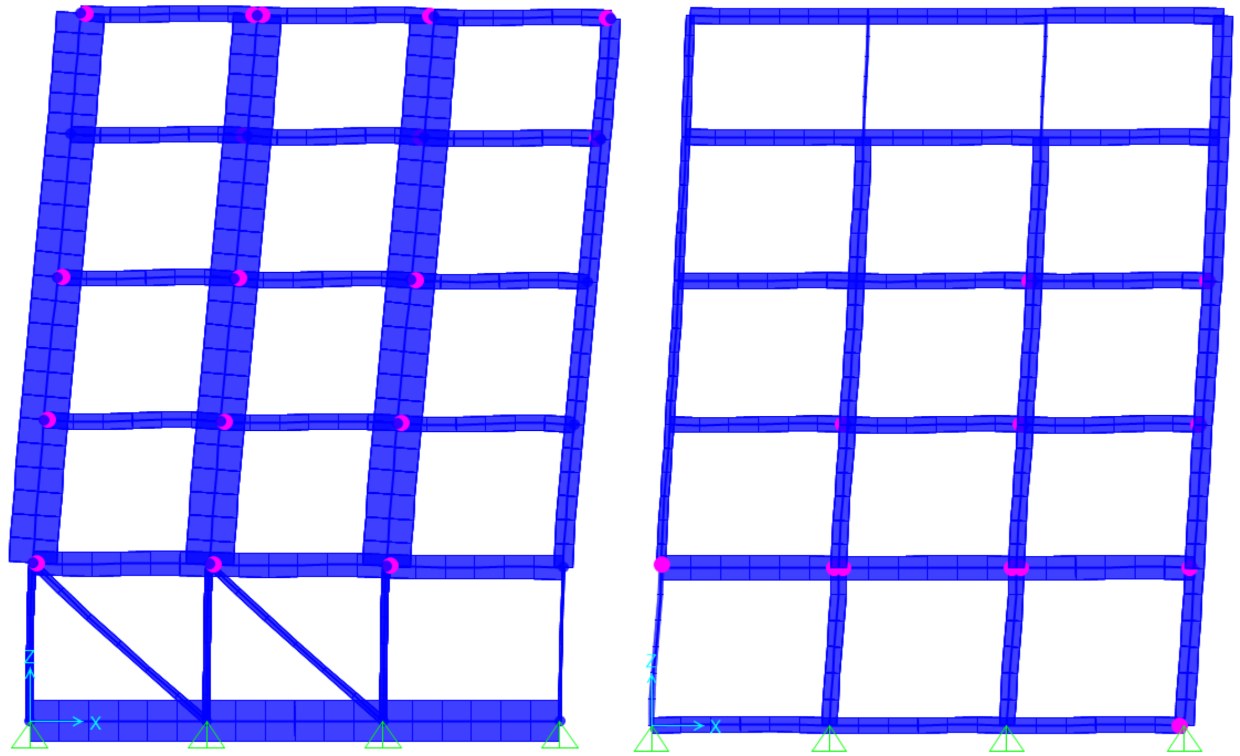

Figure 14 - SAP2000 frame displacement computer extract (External frame left, internal frame right)

External Dee Street Frame

The frame along Dee St, Figure 15, comprises reinforced concrete wall, column and spandrel sections and a steel encased concrete beam above ground floor. Below ground floor the frame is constructed of steel columns and struts. The strength of this external frame under earthquake loading is limited by the flexural capacity of the reinforced concrete spandrels and the axial capacity of the out-of-plane 200PFC post at ground floor. Based on these specific component checks the frame capacity is 10-15%

Internal Frames

These frames comprise concrete encased steel columns, reinforced concrete beams and one reinforced concrete wall section acting out-of-plane at the north end of the frame above ground floor. Below ground floor there is a steel column as the wall section stops at first floor. The strength of the internal frame under earthquake loading is limited by the concrete encased steel columns and the reinforced concrete beams and column. Each of these elements has a similar relative capacity. The capacity of these internal frames in 25-30%.

Unreinforced Masonry Spine Wall

The URM spine was wall was assessed as 350 mm thick and approximately 8.5 m long. This was the stiffest lateral load resisting element in the building and hence attracted the largest proportion of lateral load. The demand on this element was derived from the torsional analysis in addition to half the seismic weight of the URM section of the building. The capacity of the masonry wall was determined using the method outlined in SAEB, Section C8. The wall here will be able to perform to a capacity within the range of 15-25 %NBS.

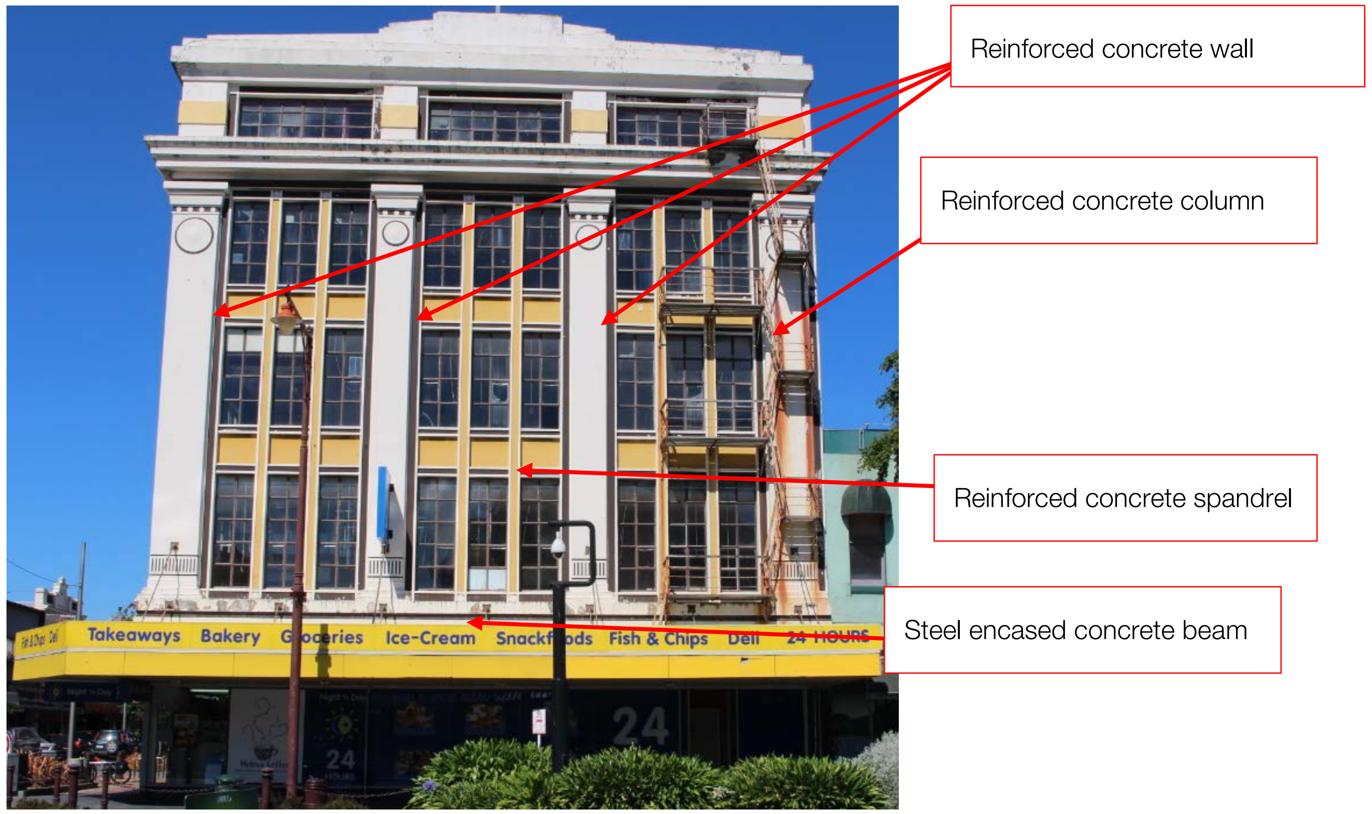

Figure 15 - External Dee Street Elevation

9.1.2 Government Life East

The structural analysis of the Government Life East building assessed the major lateral load resisting elements and areas deemed critical structural weaknesses. Structural elements were investigated according to section C8 of the technical guidelines.

9.1.2.1 Modelling assumptions

The following assumptions have been made when modelling the building:

1. No diaphragm action (due to lack of fixity to wall elements)
2. The URM elements are in good condition (i.e. no cracking). This assumption is not entirely valid with some cracking and spalling evident.
3. The soils which the walls are founded on have adequate bearing capacity to resist over turning. This assumption would need to verified by a site specific Geotechnical Engineering assessment of soil bearing capacity.
4. No ductility, elastic behavior only (μ=1). There are no ductile elements or mechanisms in the structure.

9.1.2.2 Primary transverse system

The primary transverse system of the existing Government Life East building consists of URM walls and one URM frame elevation. Loads are distributed to these elements based on tributary width analysis. The steel frames have been excluded from the system as they provide insignificant lateral load capacity. The seismic weight associated with this building section is outlined in green, in Figure 13.

Eastern External Unreinforced Masonry Wall

The eastern most wall is constructed of URM bounded by insitu concrete columns. The URM is required to span between two floors (between the 2^{nd} and 4^{th} floor), see Figure 17. This section of the wall was determined to be the most critical out-of-plane. The capacity of this wall over the full 2 storeys is approximately 15-20 %NBS(IL2).

Diaphragm action could be introduced as a remedial strategy. Diaphragm action would be introduced by providing fixity between the timber floors and URM wall. This would increase the capacity of the URM wall to 40-50 %NBS.

The in-plane capacity of the URM wall was assessed and determined to meet 60-70 %NBS. This capacity was determined under the assumption that this wall was required to support 50% of the seismic weight of the URM section of the building.

Parapet

The cantilever URM parapet above roof level was analysed also according to section C8 of the technical guidelines as a vertical spanning cantilever wall. The maximum height parapet cantilevers 3.0 m. The out-of-plane capacity of this parapet was calculated at approximately 15-20%NBS, see Figure 17.

North Elevation Unreinforced Masonry Frame

A two-dimensional (2D) SAP2000 computer model was constructed to investigate the behaviour of the URM frame in the longitudinal direction (E-W). The lateral seismic forces are assumed to be distributed over the building height in accordance with Section 6 of NZS 1170.5:2004 and the corresponding internal forces and displacements are determined using a linear elastic static analysis. Computer model extracts of the 2D frame model and the displaced shape is provided in Figure 16.

An upper bound and lower bound solution was undertaken to provide a level of sensitivity analysis.

Lower Bound

The seismic load applied comprised a third of the seismic weight of Government Life East. From engineering judgement, it was determined that the south wall would attract a larger proportion of load due to its inherently stiff nature and this north frame would attract a smaller proportion of load. The capacity of this frame under this level of loading was 10-20%NBS(IL2). The critical elements were the first-floor spandrels.

Upper Bound

The seismic load applied comprised half of the seismic weight of Government Life East based on the tributary width between lateral load resisting elements. The capacity of this frame under this level of loading was 10-15%NBS(IL2). The critical elements were the first-floor spandrels and the middle columns above first floor.

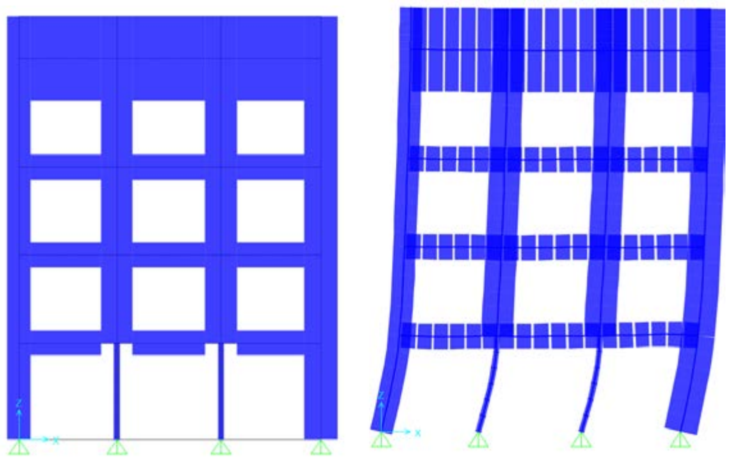

Figure 16 - Computer extract of URM frame (left) and displaced shape of URM frame (right)

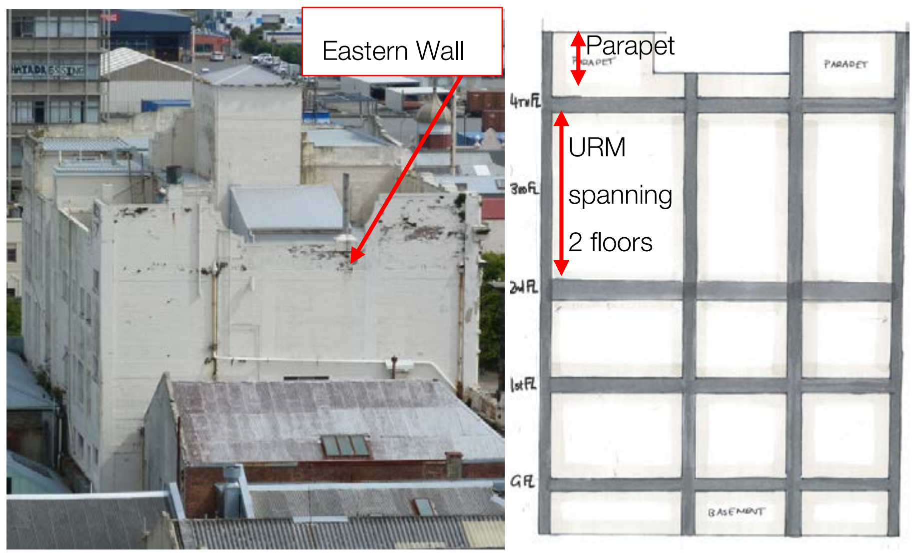

Figure 17 - Location of eastern URM wall (left), sketch of approximate configuration of eastern URM wall (right)

10.0 Quantitative Results Summary

A summary of the results from the quantitative assessment is provided in the table below. These ratings represent an estimate of the original seismic load resistance of the building prior to any earthquakes/damage.

Building area	Loading direction		Specific review element	%NBS Upper Bound	%NBS Lower Bound	Notes/Description of limiting criteria
Government Life West	Transverse (N-S)	In-plane	Reinforced concrete external wall sections	15-20%	15-20%	Flexural capacity based on μ=1 loads.
			550mm² reinforced concrete column	15-20%	15-20%	Flexural capacity based on μ=1 loads.
			450mm² steel encased concrete column	25-30%	100%	Flexural capacity based on μ=1 loads.
			Central URM spine wall	15-25%	15-25%	Shear capacity based on demand determined from torsional analysis
			350 mm deep x 300 mm wide internal reinforced concrete beam	25-30%	70-75%	Flexural capacity based on μ=1 loads.
			Wall spandrel	10-15%	5-10%	Flexural capacity based on μ=1 loads.
		Out-of-plane	Steel Brace – 200SHS9	40-45%	45-50%	Axial capacity based on μ=1 loads.
Government Life East	Longitudinal (E-W)	Out-of-plane	Eastern URM wall	10-20%	40-45%	Capacity based on vertical spanning wall SAEB C8.8.5
			3.0 m high parapet	10-20%	10-20%	Capacity based on cantilever wall section SAEB C8
		In-Plane	North URM frame	10-15%	10-20%	Shear capacity based on μ=1 loads.
	Transverse (N-S)	Out-of-plane	South URM wall	15-20%	40-45%	Capacity based on vertical spanning wall SAEB C8.8.5
		In-plane	Eastern URM wall	60-70%	60-70%	Capacity based on cantilever wall section SAEB C8

11.0 Building Condition Assessment

11.1 Site Visits and Overview

BMC carried out site assessments of the building on 11th December 2017 and 15th December 2017. This involved obtaining a photographic and written record of the structural systems of the building along with areas of damage or decay. The observations made were visual only (i.e. non-intrusive) and limited to obtaining representative samples of the concrete for strength, carbonation and chloride testing.

The first site visit on 11th December 2017 BMC involved inspection of the following areas of the building:

- Basement
- Levels 1-4 (site measure of level 2)
- Exterior where accessible.
- Roof and parapets
- Site measure of critical wall and floor thicknesses.
- Opening up of spalled concrete areas to determine the cause of the damage

The second site visit on the 15th December 2017 involved inspection of the following areas of the building:

- Ground Floor
- Site measure of basement
- Lift Shaft

The site observations described below relate to structural damage only i.e. damage to structural elements which form part of either the lateral or gravity load resisting systems or both. Cosmetic damage i.e. damage that only affects the appearance of something is purposefully not described here.

Concrete samples were obtained from various areas and elements of the building and sent to Opus Laboratories in Christchurch and Wellington for testing. The results of the testing are attached in Appendix B and Appendix C.

It is believed that the basement and levels 1 through 4 of the building have been unoccupied for approximately 35 years. The building does not appear to have been maintained to a good standard during the period it has not been occupied and currently suffers from moisture ingress through the roof and exterior walls. We bought to the attention of HWCP Management Ltd the fact that the fire escape at the Southwest corner of the building was likely to collapse if it was used. The fire escape was immediately isolated from use from off the veranda to Dee Street and signage placed inside to building prevent use.

The building was tested for the presence of asbestos which was identified in two specific and isolated locations.

Figure 18 - Spalling concrete adjacent to window in light well area.

Figure 19 - Spalling to window mullion on West wall

Figure 20 - Severe spalling to window mullion on West wall

Figure 21 - Cracking extends all the way through the window mullion

11.2 Site Observations

Structurally the main observation was the porosity of the concrete to the exterior walls allowing moisture into the building. This moisture combined with the presence of poorly washed marine aggregate leading to elevated chloride concentrations, and weak concrete strengths, has led to the concrete spalling in a significant number of areas particularly on the West and South elevations being the predominant wind directions during wet weather. The spalling of the concrete is caused by the reinforcing rusting and expanding by up to five times the size of the original steel. This generates forces within the concrete, leading to it cracking towards the surface generally in the shortest distance possible.

Figure 20 through Figure 22 show that some parts of the structure have failed due to corrosion of the reinforcing reaching a point where the mullion in this case has failed. Temporary stabilisation of this particular mullion has been undertaken to prevent it or debris from falling onto SH6 below.

The parapets to Government Life East appear to be unreinforced brick masonry covered in plaster which is spalling off in some areas as shown in Figure 25. The mortar strength is weak due in the main to weathering and age.

Pounding of the adjacent building on Esk St is evident as shown in Figure 23.1 and Figure 23.2.

The roof is leaking in a number of areas, leading to degradation of the structure as shown in Figure 26.

Figure 22 - Close up photo of severe cracking to mullion

Figure 23 - SW corner column showing cracking to least exposed corner of the column

Figure 23.1 – Pounding close up of eastern end of structure and adjacent building

Figure 23.2 – Shear cracking inside adjacent building due to pounding with GL

Figure 24 - Crack extends back to main column reinforcing which is rusting

Figure 25 - East parapet wall showing brick below plaster surface

Figure 26 - Roof leaks causing structural damage to beam support.

Figure 27 - Spalling plaster over brick substrate

11.3 Building Condition Discussion

The Government Life building is 90 years old and is exhibiting signs that the concrete structure has exceeded its lifespan. This is manifested in the spalling of the concrete on the exposed West and South walls of the building. It is also backed up by the results of the concrete testing undertaken on the wall elements that are damaged but also the interior concrete element strengths. New structures are normally designed using the current material codes for a durability of 50 years. The concrete code, NZS 3101: Part 2:2006, C3.3 states "Durability is indirectly defined as the ability to withstand the expected wear and deterioration throughout the intended life of the structure without the need for undue maintenance. The expected wear and deterioration may include the influences of weathering chemical attack and abrasion." Clearly with the spalling that is evident within the building, the building has exceeded its life expectancy and is likely to rapidly deteriorate within a short time frame.

Some of the concrete samples were tested for carbonation and chloride ion contamination and the results are attached in Appendix C. In summary the results were: -

- Chloride concentrations are somewhat elevated above normal levels.
- The chloride concentrations are likely to be from the use of poorly washed marine aggregate within the concrete mix.
- Variable carbonation through the cores, reflects the quality of the workmanship. The maximum carbonation depths likely exceed typical cover depths, indicating depassivation of at least some fraction of the reinforcing steel and hence a current vulnerability to corrosion

The mullion in figures 19-21 above has clearly failed due to the effects of the environment and the age of the concrete. This effect is also noted in the spandrel beams and other structural elements and will eventually, given time lead to widespread damage and potentially localised collapse of these elements.

It is clear from figure 19 and figure 22 that the vertical bars have minimal cover of approximately 20mm which exacerbates the issues as there is very little concrete outside the bar to resist the forces due to rusting.

The brickwork to the parapets is exposed to the weather over a large area where the plaster and paint have degraded. The mortar is weak in these areas leading to a reduction in strength in these already weak elements.

The water ingress into the roof and walls on level 4 are degrading the structural elements and will lead to loss of support to the roof structure.

11.3.1 Demolition Methodology

As a result of site investigations and structural analysis, BMC has investigated potential methods of demolition. Preliminary discussions were held with demolition contractor, Ryal Bush, to discuss the implications and practicality of the demolition of the Government Life building. Concerns were raised regarding the building (or parts of) falling onto adjacent buildings to the south and SH1 to the west. Any works protruding onto SH1 would require SH1 road closures. Any SH1 road closure will be restricted by NZTA regulations.

Following these preliminary discussions, demolition was indicated as only possible if it started from the east elevation. Government Life West could consequently not be removed without demolishing Government Life East.

12.0 Conclusions

The Government Life Building at the corner of Dee and Esk streets was constructed in 1929 in two distinct halves which we have called the West and East sections. The West section was built using mainly concrete construction with concrete floor diaphragms. Whilst the East building is URM with timber floors that have no diaphragm connections. Both sections have cantilever URM parapets above roof level.

The West section of the building is considered to have a capacity of 10-15% of New Building Standard. The capacity of the building is limited by concrete wall spandrels.

Loading Direction	%NBS (IL2)
N-S	10-15%
E-W	10-15% (estimated)

The East section of the building is considered to have a capacity of 10-20% of New Building Standard. The capacity of the building is limited by URM spandrels and parapet.

Loading Direction	%NBS (IL2)
N-S	10-20%
E-W	15-20%

BMC notes that the governing elements of the structure are weak poorly detailed concrete spandrels on the West building and URM wall elements on the East building. It was also noted that there is no connection between the walls and timber floor diaphragms on the East building

Geotechnical input indicates bearing capacity in the very soft to firm alluvial silt underlying the site is expected to be significantly lower than "good ground. Some areas of the site are expected to liquefy below the water table under ULS loading, but not at SLS loading.

Our intrusive investigations of the concrete and testing for strength, chlorides and carbonation by Opus Laboratories have found concrete strengths are very low and vary from 6.5MPa to 20MPa. Chloride concentrations are somewhat elevated above normal levels. The chloride concentrations are likely to be from the use of poorly washed marine aggregate within the concrete mix. Variable carbonation through the cores, reflects the quality of the workmanship. The maximum carbonation depths likely exceed typical cover depths, indicating depassivation of at least some fraction of the reinforcing steel and hence a current vulnerability to corrosion.

The Government Life Building is earthquake prone and in terms of structural strength and condition is in our opinion not able to be repaired or strengthened without the loss of most of the heritage fabric and values of the building. The building has not been occupied above ground floor for approximately 35 years and has significant structural and non-structural damage caused by lack of maintenance.

Appendix A – Detailing Assumptions

- Typical floor slab
  - Depth = 150 mm
  - Reinforcing = 16 mm ϕ bars @ 178 mm cnrs
- Typical transverse beam
  - Depth = 350 mm
  - Width = 300 mm
  - Reinforcing = 5x22 mm ϕ bars top and bottom
- Typical internal steel encased concrete column
  - 450 mm wide x 450 mm deep
  - 200UC steel column encased
- Typical external column along south elevation
  - 550 mm wide x 550 mm deep
  - 8x22 mm ϕ bars
- Typical external wall section along Esk St/ Dee St
  - 250 mm thick reinforced concrete
  - 10x16 mm ϕ bars
- Only steel encased concrete beam between ground floor and first floor along Dee St elevation
  - 700 mm wide x 700 mm deep
  - 200UC steel beam encased
- Spandrels along Esk St/ Dee St elevations
  - Thickness spandrel = 200 mm
  - Total height spandrel = 1390 mm
  - Reinforcing = 8x12 mm ϕ bars and 2x6 mm ϕ bars
- Rear wall is RC columns with cavity brick infill
  - 2 layer brick with cavity

Figure 27 - Typical transverse beam section

Figure 28 - Typical external column reinforcing

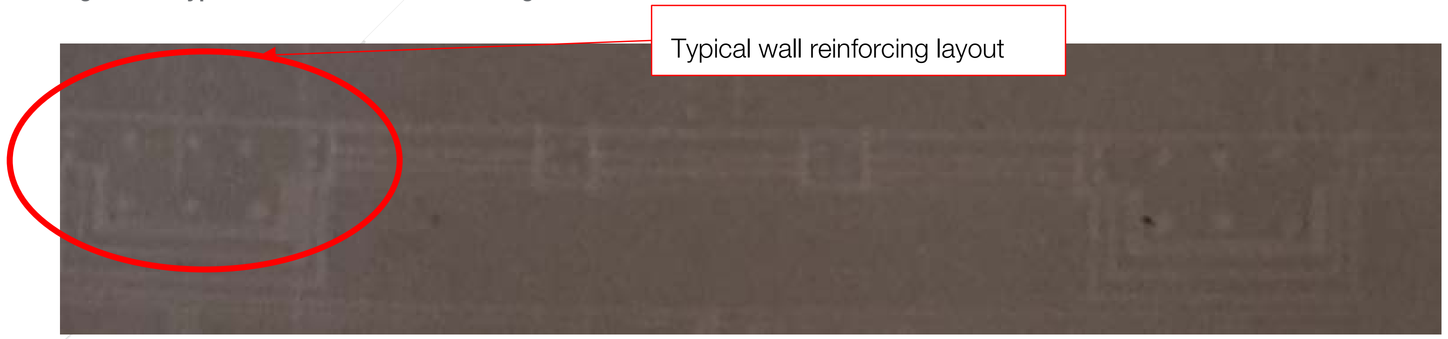

Figure 29 - Typical external wall section along Esk St/ Dee St

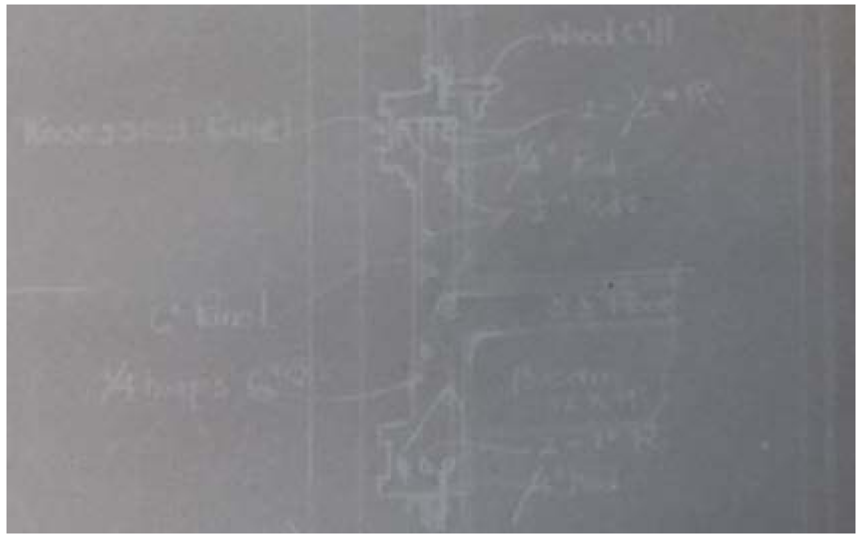

Figure 30 - Typical spandrel reinforcing layout

Appendix B – Concrete Compression Test Report

CONCRETE COMPRESSION TEST REPORT

OPUS

Project:	**Concrete Quality Assurance**
Location:	**Old Govt Life & Southland Times**
Client:	**Batchelar McDougall Consulting**
Contractor:	**Batchelar McDougall Consulting et al**
Sampled by :	**Batchelar McDougall Sub-Contractor**
Date sampled :	**23 December 2017**
Sampling method :	**Rotary Core Drill of Various Diameter's**
Sample Conditioning:	**Tested as Received**
Source :	**Corner Esk & Dee Street, Invercargill**
Grade :	**Circa 1928**
Date received:	**8 January 2018**

Project No:	**6-JBMCL.16/6LC**
Lab Ref No:	**CH3667**
Client Ref No:	**29 Esk Street**

Test Results

Lab reference no		40/1	40/2	40/3	40/4
Client reference no		**Level 2 Column**	**Level 2 Beam**	**Level 2 Floor**	**Level 3 Floor**
Date made		1928	1928	1928	1928
Date tested		2018	2018	2018	2018
Age of material	(years)	**90**	**90**	**90**	**90**
Average diameter	(mm)	94.2	94.4	68.6	68.6
Length	(mm)	185.0	120.0	123.5	115.5
Mass of cylinder in air	(g)	3019	1954	1094	1037
Design strength	(MPa)	-	-		
Density	(kg/m³)	**2400**	**2390**	**2430**	**2470**
Height diameter ratio		1.96	1.27	1.80	1.68
Compressive strength	(MPa)	**7.0**	**9.5**	**12.5**	**12.5**
Adjusted compressive stress	(MPa)	-	*8.5	*12.0	*12.0
Number of ends capped		Two	Two	Two	Two
Defects prior to capping		Irregularities	Irregularities	Irregularities	Irregularities

Comments

Test Methods	Notes
Compression, NZS 3112 : 1986, Pt 2 Section 6	*H/D outside 2:1, 'D' factor applied
Density, NZS 3112 : 1986, Pt 3 Section 5	
Capping NZS 3112 : 1986, Pt 2 Section 4 (amendment No 2 2000)	

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

Date reported : 22 January 2018

IANZ Approved Signatory

Designation : Laboratory Manager
Date : 22 January 2018

IANZ ACCREDITED LABORATORY

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

PF-LAB-092 (19/02/2015)

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CONCRETE COMPRESSION TEST REPORT

OPUS

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Location:	**Old Govt Life & Southland Times**
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Grade :	**Circa 1928**
Date received:	**8 January 2018**

Project No:	**6-JBMCL.16/6LC**
Lab Ref No:	**CH3667**
Client Ref No:	**29 Esk Street**

Test Results

Lab reference no Client reference no		40/5 **Level 3 Spandrel**	40/6 **Level 4 Spandrel**	40/7 **Level 1 Column**	40/8 **Level 1 Spandrel**
Date made		1928	1928	1928	1928
Date tested		2018	2018	2018	2018
Age of material	(years)	**90**	**90**	**90**	**90**
Average diameter	(mm)	94.3	93.7	94.3	94.0
Length	(mm)	131.5	185.0	186.5	186.5
Mass of cylinder in air	(g)	2111	3093	2976	3093
Design strength	(MPa)	-	-		
Density	(kg/m³)	**2320**	**2410**	**2370**	**2400**
Height diameter ratio		1.40	1.98	1.98	1.98
Compressive strength	(MPa)	**20.5**	**21.5**	**6.5**	**27.5**
Adjusted compressive stress	(MPa)	*18.5	-	-	-
Number of ends capped		Two	Two	Two	Two
Defects prior to capping		Irregularities	Irregularities	Irregularities	Irregularities

Comments

Test Methods	**Notes**
Compression, NZS 3112 : 1986, Pt 2 Section 6	*H/D outside 2:1, 'D' factor applied
Density, NZS 3112 : 1986, Pt 3 Section 5	
Capping NZS 3112 : 1986, Pt 2 Section 4 (amendment No 2 2000)	

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Sample Conditioning:	**Tested as Received**
Source :	**Corner Esk & Dee Street, Invercargill**
Grade :	**Circa 1928**
Date received:	**8 January 2018**

Project No:	**6-JBMCL.16/6LC**
Lab Ref No:	**CH3667**
Client Ref No:	**29 Esk Street**

		Test Results			
Lab reference no		40/9	40/10	40/11	
Client reference no		**Level 1 Beam**	**Level 1 Sth Wall**	**Lev 1 Light Well**	
Date made		1928	1928	1928	
Date tested		2018	2018	2018	
Age of material	(years)	**90**	**90**	**90**	
Average diameter	(mm)	94.4	94.1	68.4	
Length	(mm)	187.5	133.0	129.5	
Mass of cylinder in air	(g)	2660	2132	1156	
Design strength	(MPa)	-	-		
Density	(kg/m³)	**2430**	**2410**	**2440**	
Height diameter ratio		1.99	1.41	1.89	
Compressive strength	(MPa)	**12.5**	**17.5**	**26.5**	
Adjusted compressive stress	(MPa)	-	*16.0	*26.0	-
Number of ends capped		Two	Two	Two	
Defects prior to capping		Irregularities	Irregularities	Irregularities	

Comments

Test Methods	Notes
Compression, NZS 3112 : 1986, Pt 2 Section 6	*H/D outside 2:1, 'D' factor applied
Density, NZS 3112 : 1986, Pt 3 Section 5	
Capping NZS 3112 : 1986, Pt 2 Section 4 (amendment No 2 2000)	

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Appendix C – Durability Analysis of Cores

2 February 2018

Opus International Consultants Ltd

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PO Box 30 845, Lower Hutt 5040
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Charlotte Corston
Batchelar McDougall Consulting
PO Box 9440
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Christchurch 8149

Ref: 6-MBMCL.16 / 6CL

Durability Analysis of Concrete Cores – Government Life / Southland Times Building

Dear Charlotte

This letter summarises the results of the analyses undertaken on your behalf of four concrete cores[1] supplied by Geoff Jones from our Christchurch laboratory for the purpose of determining the depth of the carbonation front and possible presence of chloride ion contamination.

All of the cores had a nominal diameter of 100 mm and had been cut through the full depth of the sampled element. Both ends of the each concrete core were rendered with a solid plaster finish of varying thickness and overpainted with a membrane-forming coating. The orientation of the cores was not indicated but could be inferred by the presence of a thin skim coat of gypsum plaster beneath the paint on one face, which was assumed to correspond to the interior end of the core. One core was observed to intersect a single ¼″ diameter reinforcing bar, which had a total depth of cover from the exterior face of 30 mm, including ca. 10 mm of plaster.

1. Methodology

The as-received cores were initially prepared for testing by slicing each specimen longitudinally in half using a water-cooled diamond saw.

The presence of any carbonation through the concrete was determined by spraying phenolphthalein indicator solution on the freshly-cut surface of one half of the specimens after thorough rinsing with water remove any residual contamination from the cutting process. This procedure is based upon RILEM Recommendation CPC-18.[2] The measured depth below the surface of the core that remains colourless on application of the phenolphthalein reagent, rather than becoming stained a magenta colour, indicates the region of concrete with a pH of less than 9.0 – 9.3. This region is assumed to correspond to the total depletion of $Ca(OH)_2$ in the concrete through reaction with atmospheric CO_2; any reinforcement that lies within this zone is potentially vulnerable to corrosion.

To measure any potential chloride contamination present within the concrete, a 15 mm thick slice concrete was removed from the other longitudinal slice of each specimen for analysis. Because a determination of the chloride profile (i.e. the variation in concentration with depth from the surface exposed to the external environment) was not required to determine the origin or rate of accumulation of

[1] Opus Research sample registry # 4-18/030, received 23 January 2018.

[2] RILEM Recommendation CPC-18. 1998. 'Measurement of hardened concrete carbonation depth'. *Materials & Structures* 21 no 6, pp 453 – 455, November 1998.

any contamination present,[3] the slice was generally cut from the concrete immediately below any plaster finish on the externally-exposed face of the core. This potentially represents a worst-case location since the chloride ingress through the concrete from environmental sources is likely to be at a maximum at this depth and will concentration will also be superimposed on any cast-in contamination present.

The resulting slices were dried, crushed to a fine powder and analysed by x-ray fluorescence spectroscopy (XRF) to express the total chloride content as a percentage of the dry weight of concrete.

Reinforcement corrosion induced by carbonation or chloride ion contamination is recognised by NZS 3101 *'Concrete Structures'* as the durability-related deterioration mechanism most likely to control the service life of a concrete structure under typical NZ conditions.[4]

2. Results

Table 1 summarises the results from the carbonation testing and chloride ion analyses obtained. The carbonation results are also illustrated photographically by Figure 1.

Table 1. Summary of durability analysis of supplied cores.

Specimen Label	**Maximum Carbonation Depth**[&]		**Chloride Analysis** (%w/w by mass of concrete)
	From External Face (mm)	**From Internal Face** (mm)	
Core A *Level 2 Spandrel West Esk Street*	45	75	0.011
Core B *Level 2 Spandrel West Esk Street*	18	74	0.127
Core C *Level 2 Spandrel South Dee Street*	10	34	0.019
Core D *Level 2 Spandrel South Dee Street*	2	55	0.055

[&] Carbonation measurement includes the plaster thickness; this was typically a single 10 mm thick flanking coat plus a 2 – 3 mm skim coat of either cementitious material or gypsum, depending on the orientation of the face. However the reveals at the spandrel panel margins intersected by Cores C & D had been much more heavily plastered to fair the surface, as depicted by the annotated dotted yellow lines in Figure 1.

[3] Charlotte Corston, *personal communication*. Email to Geoff Jones dated 17 January 2018.

[4] Standards New Zealand. NZS 3101:2006. *Concrete Structures Standard. Part 1: The Design of Concrete Structures & Part 2: Commentary*. Wellington, New Zealand.

These results reveal:

- Chloride concentrations through the cores that are somewhat more elevated than would ordinarily be expected from routine background contributions from appropriate mix constituents.
- No obvious relationship between the measured chloride concentration in individual cores; the erratic pattern observed suggests the origin of the contamination is through incorporation of a poorly-washed marine aggregate as an integral component of the mix, rather than the result of environmental exposure. This is consistent with the presence of abundant bivalve fragments observed within the concrete matrix. Because of the nature of this contamination there are likely to be 'hotspots' of elevated chloride concentration, posing a high risk of reinforcement corrosion, which are somewhat randomly distributed through the concrete amongst comparatively benign areas.
- The mean contamination measured in the individual cores ranges from 0.011% to 0.127 % chloride by mass of concrete. The significance of this range is briefly discussed in the following section.
- Variable carbonation through the cores (Figure 1), which likely reflects differences in the quality of consolidation and local internal relative humidity due to micro-exposure environment. The maximum carbonation depths likely exceed typical cover depths, indicating depassivation of at least some fraction of the reinforcing steel and hence a current vulnerability to corrosion.

Figure 1. Longitudinal slices through the cores, photographed following application of the phenolphthalein reagent. The arrowed lines indicate the approximate maximum carbonation depth from each exposed end of the core. The approximate position from which the chloride sample was removed from the matching longitudinal slice is also indicated.

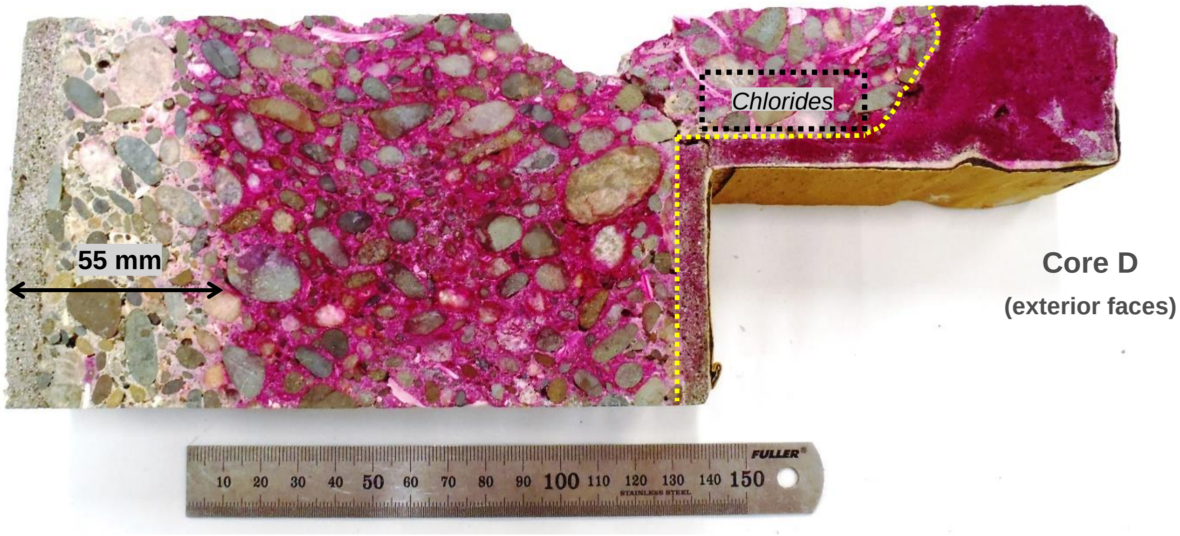

Figure 1 (continued).

3. Interpretation of Risk

The cast-in chloride contamination within the concrete is generally at levels that would be considered tolerable, particular where the concrete remains dry. However in combination with the advanced carbonation, the susceptibility to corrosion may be higher than the conventional risk thresholds would ordinarily suggest: In particular it is generally accepted that the corrosion risk is controlled by both the chloride and hydroxide ion concentrations within the pore fluid of concrete, with mild steel reinforcement vulnerable under conditions where $[Cl^-]/[OH^-] > 0.6$.[5]

Due to the absence of a convenient measurement technique and the fact that hydroxide ion concentration is, to a first approximation, constant in uncarbonated concrete, corrosion risk thresholds are ordinarily formulated in terms of chlorides values alone. By definition however, carbonated concrete is depleted of hydroxide ions, thus increasing the $[Cl^-]/[OH^-]$ ratio and hence the intrinsic corrosion risk at any given level of chloride contamination. The situation is rarely encountered except where chlorides are cast-in, since environments conducive to environmental ingress are seldom also favourable for carbonation. Additionally, the carbonation reaction has the unfortunate property of decomposing the C_3A (tri-calcium aluminate) phase in cement that ordinarily immobilises a certain fraction of the chloride ions, thus liberating them to participate in corrosion reactions. Carbonation may also result in a redistribution of the chloride ions in response to concentration gradients as the C_3A reacts.

Because of this synergistic coupling between carbonation and cast-in chlorides, the current and future reinforcement corrosion risk for the sampled concrete is potentially moderate to high where the carbonation has reached the reinforcement, particularly if the environment is not protected and dry (Figure 2). Because of the nature of cast-in contamination, the corrosion risk is likely to somewhat variable across the structure, with localised hotspots reflecting the inhomogeneity of chloride distribution in the source aggregate.

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5 Broomfield, J.P. 2007. *Corrosion of Steel in Concrete: Understanding, Investigation & Repair*. 2nd Edition. Taylor & Francis, United Kingdom.

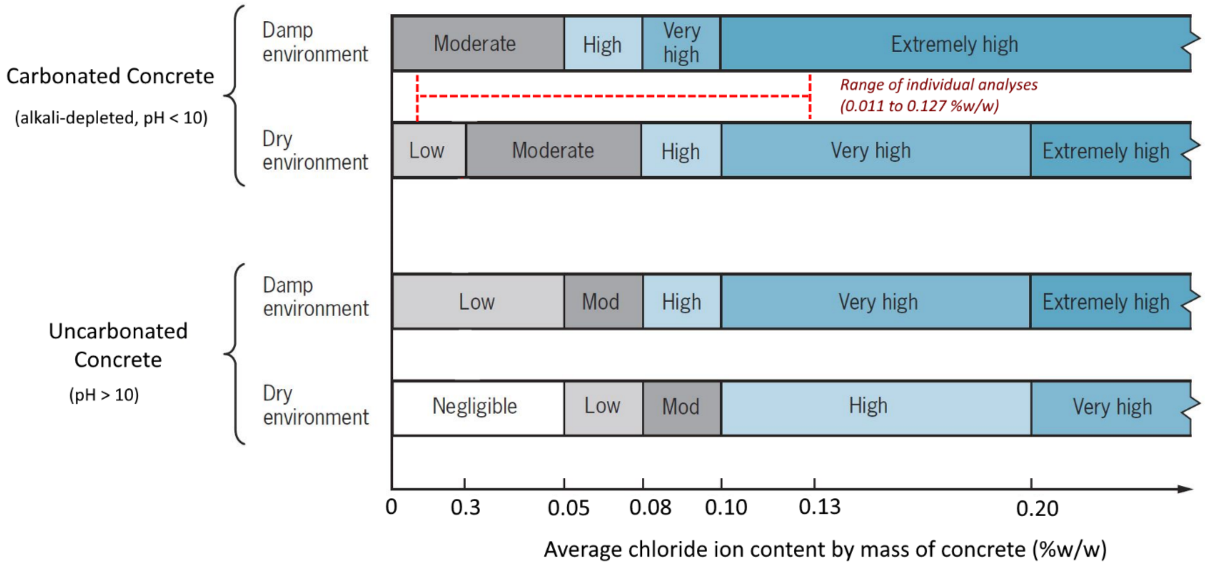

Figure 2. Comparison of chloride contamination measured in cores with estimated risk of reinforcement corrosion under differing environmental conditions for typical quality older structural concrete.[6]

I trust this information is helpful to your condition assessment. Please contact me if you have any queries regarding the contents of this report, or if we can assist you further in the future.

Kind Regards

Neil Lee

Neil Lee
Concrete Technologist

[6] Figure adapted from BRE Digest 444 Part 2 *Corrosion of Steel in Concrete: Investigation & Repair.* Building Research Establishment, Watford Junction, United Kingdom.

Appendix D – GeoSolve geotechnical report

Geotechnical Desktop Study

Invercargill CBD Project – Stage 1, Old Government Life/Arbuckles Building and old Southland Times Building

Invercargill

Report prepared for:

Batchelar McDougall Consulting

Report prepared by:

GeoSolve Limited

Distribution:

Batchelar McDougall Consulting

GeoSolve Limited (File)

February 2018

GeoSolve Ref: 171019

GEOTECHNICAL WATER RESOURCES PAVEMENTS

Table of Contents

1 Introduction

1.1 General

This report presents the results of a geotechnical desktop study carried out by GeoSolve Ltd in order to determine likely subsoil conditions and provide geotechnical inputs for a structural assessment of two buildings (the Old Southland Times building and the Old Government Life/Arbuckles building) in the Invercargill CBD.

Photo 1 – Old Southland Times Building, Looking southwest from Esk St (source - *maps.google.co.nz*)

Photo 2 – Old Government Life/Arbuckles Building, Looking southeast from corner of Esk St and Dee Street (source - *maps.google.co.nz*)

The desktop study was carried out for Batchelar McDougall Consulting in accordance with GeoSolve Ltd's proposal dated 23 December 2017, which outlines the scope of work and conditions of engagement.

1.2 Scope of Works

We understand that the two existing buildings above are to be structurally assessed by Batchelar McDougall Consulting and to assist the assessment a geotechnical desktop study is required, outlining:

- The likely ground conditions below the site;
- Preliminary seismic soil classification;
- Preliminary assessments of the likely bearing capacity of the existing building foundations at the sites and liquefaction and settlement susceptibility;
- Recommendations for likely foundations for any new development in this area for 3-4 story construction.

2 Site Description

2.1 General

The subject properties are located in central Invercargill as shown in Figure 1 below.

Figure 1: Site location plan, showing the location of the two buildings in red (Old Govt Life and Old Southland Times buildings) being assessed.

The buildings are accessed from Esk St and Dee St.

2.2 Topography and Surface Drainage

The building sites are situated on horizontal ground with an established drainage system in the area that is expected to control surface flows.

3 Geotechnical Investigations

No site specific investigations have been undertaken for the purpose of this report. GeoSolve have completed a review of shallow and deep site investigations in close proximity to the sites in central Invercargill to infer the underlying geological model.

4 Subsurface Conditions

4.1 Geological Setting

The site is expected to be underlain by shallow surface fill, which in turn overlies alluvial deposits with Tertiary-age marine sediments at depth. The alluvial deposits comprise Quaternary outwash gravels developed during former glaciation, which occurred inland. More recent silty/sandy floodplain or mudflat deposits overlie these gravels. The alluvial deposits merge with marine deposits at depth in the vicinity of Invercargill.

No active faults have any been reported in the vicinity of Invercargill. Strong earthquakes are common in Fiordland near the current tectonic plate boundary and consequently some moderate ground shaking can be expected to occur in Invercargill during such events. The nearest trace of any mapped active fault is the Hillfoot Fault, approximately 60 km to the north of the site.

Significant seismic risk exists in this region from potentially strong ground shaking, likely to be associated with a rupture of the Alpine Fault, located along the West Coast of the South Island. There is a high probability that an earthquake with an expected magnitude of over 8 will occur along the Alpine Fault within the next 50 years.

4.2 Stratigraphy

Subsurface soils beneath the two buildings being assessed are inferred to comprise:

- Uncontrolled fill/engineered fill, overlying;
- Alluvial silt, overlying;
- Alluvial sand, overlying;
- Alluvial gravel.

Uncontrolled fill was observed to underlie each lot where GeoSolve have completed investigations in the area. Uncontrolled fill was observed to comprise clayey SILT with some gravel and sand, sandy GRAVEL with minor silt, gravelly SILT with wood, ash and bricks and SAND. Engineered fill platforms may have been constructed under the existing building foundations.

The fill is predominately underlain by alluvial silt comprising very soft to firm, SILT with nil to some sand content and clayey SILT. The base of the alluvial silt was observed between 1.5-3 m bgl in the area.

In discrete locations an alluvial sand layer was observed to underlie the alluvial silt to between 2 and 4 m bgl. Alluvial sand was observed to comprise silty SAND with some fine gravel, and SAND with trace silt.

Alluvial gravel was observed to underlie the alluvial silt or sand in all cases. Alluvial gravel has been observed within 8 Boreholes and depths have been inferred from 24 Heavy Dynamic Probe (DPH) tests completed in the Invercargill CBD area. The depth to the top of the alluvial gravel in the area is inferred to be between 2 and 4 m bgl. Alluvial gravel was observed to predominately comprise medium dense to dense, sandy GRAVEL and silty GRAVEL with thin SAND lenses.

4.3 Groundwater

Groundwater was observed between 1.4 and 3.3 m bgl in the area. Investigations completed in closest proximity to the buildings being assessed indicate a water level of 3-3.3 m and 1.4-1.7 m at 16-24 Don Street (~150 m N of the site) and 65 Don St (~180 m NE of the site) respectively.

It is recommended that piezometers are installed on site to confirm the groundwater levels.

5 Liquefaction Analysis

5.1 Design Earthquakes

Two earthquakes scenarios have been assessed in accordance with NZS1170 – Structural Design Actions[1] for an Importance Level 2 structure with a 50-year design life.

Peak horizontal ground accelerations and effective magnitudes were calculated using the procedure from the NZTA Bridge Manual[2]. Table 5.1 summarises the scenarios considered.

The site has been assessed as subsoil category *Class D – Deep soil* site in accordance with NZS1170 – Structural Design Actions.

Table 5.1 – Earthquake accelerations and effective magnitudes for liquefaction assessment

Scenario	Performance Requirements	Annual Probability of Exceedance	Peak Horizontal Ground Acceleration (PGA)	Effective Magnitude
Serviceability Limit State (SLS)	*Avoid damage that would prevent the structure being used as originally intended without repair*	1/25	0.05 g	6.2
Ultimate Limit State (ULS)	*Avoid collapse of the structural system*	1/500	0.2 g	6

5.2 Liquefaction Summary

The liquefaction analysis from surrounding sites indicates there is typically no potential for liquefaction or lateral spreading under SLS seismic loading, however minor liquefaction is predicted under ULS loading at some sites in the area.

Typical liquefaction analysis from the surrounding area indicate the following:

- No liquefaction or cyclic softening is predicted for the SLS design earthquake;
- Minor liquefaction is predicted for the ULS design earthquake. Loose sand lenses overlying or within the alluvial gravel unit have the potential to liquefy below the water table under ULS seismic loading;
- CPT and DPH testing in the surrounding area predict liquefaction induced free field settlement of between 0-50 mm in an ULS seismic event.
- ULS settlement should be confirmed with site specific deep investigations comprising boreholes, DPHs and CPTs.

[1] NZS1170-5 (2004) Structural Design Actions, Part 5: Earthquake Actions – New Zealand.
[2] NZTA Bridge Manual (2014). SP/M/022, third edition amendment 1, Effective from September 2014.

6 Engineering Considerations

6.1 General

Data presented as part of this report is preliminary in nature and is only to be used to assist in the structural assessment of the old Government Life/Arbuckles and the old Southland Times buildings. No site specific investigations have been completed as part of this assessment.

6.2 Geotechnical Parameters

Table 6.1 provides a summary of the typical geotechnical design parameters for the soil materials expected to be encountered underlying the existing buildings.

Table 6.1 – Recommended geotechnical design parameters

Unit	Thickness (m)	Bulk Density γ (kN/m^3)	Effective Cohesion c´ (kPa)	Effective Friction ϕ´ (deg)	Elastic Modulus **E** (kPa)	Poissons Ratio ν
Uncontrolled Fill	0-1	16	N/A	N/A	N/A	N/A
Alluvial Silt (very soft to firm SILT with some sand and clayey SILT)	0.3-1.7	18	0	28-30	1-5,000	0.3
Alluvial Sand (loose to medium dense silty SAND with some gravel and SAND with trace silt)	0.5-2.5	18	0	31-32	3-10,000	0.3
Alluvial Gravel (medium dense to dense, sandy GRAVEL)	Not proven	19	0	35	20-30,000	0.3

6.3 Groundwater Issues

The groundwater table at the sites is expected to be within the alluvial sand/gravel unit. No artesian groundwater pressures are expected at the site.

During periods of heavy rainfall the existing stormwater system is expected to control surface flows across the site and drain appropriately.

6.4 Foundations

6.4.1 General

It is understood the old Southland Times and Government Life/Arbuckles building's foundations are likely to comprise of strip footings bearing upon alluvial silt. Bearing capacity within the very soft to firm alluvial silt underlying the site is expected to be significantly lower than "good ground".

It is however understood the Government Life/Arbuckles building has a basement which may result in the foundation loads being transferred to the underlying alluvial gravel or a thin layer of alluvial silt overlying alluvial gravel, this is unlikely to be the case for the old Southland Times building, where the foundation is understood to be constructed close to road level.

6.4.2 Shallow Foundations

Figure 2 below summarises typical working stresses for shallow footings, which bear upon alluvial silt. It should be noted the foundation working stresses presented on Figure 2 are governed by bearing capacity in the case of narrow footings and settlement in the case of wide footings.

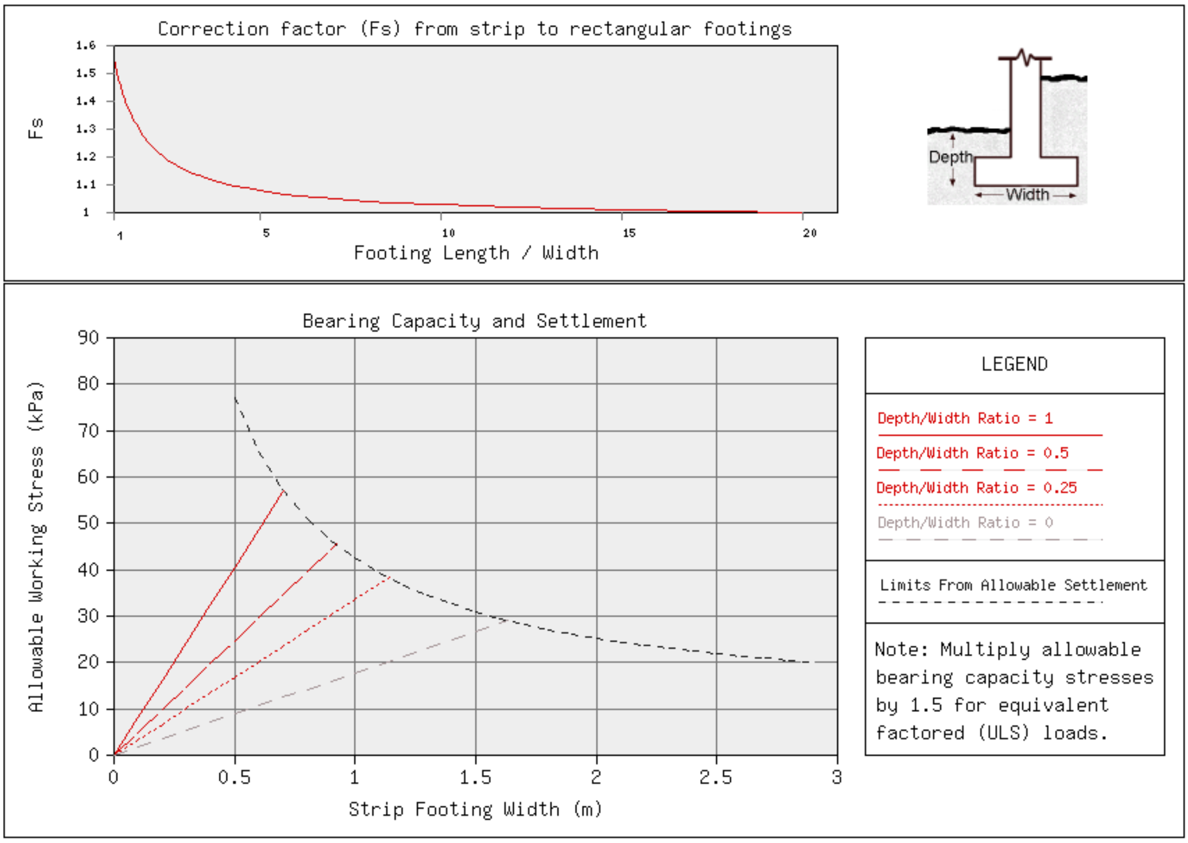

Figure 2: Typical Bearing for Shallow Footings on Alluvial Silt

From Figure 2 it can be seen an allowable working stress of approximately 40 kPa is recommended for a 500 mm wide by 500 mm deep strip footing founded within alluvial silt. This corresponds to a factored (ULS) bearing capacity of approximately 60 kPa and an ultimate geotechnical bearing capacity of 120 kPa. Note the low allowable bearing for larger footings.

Figure 3 summarises the recommended working stresses for shallow footings, which bear upon alluvial gravel. It should be noted the foundation working stresses presented on Figure 3 are governed by bearing capacity in the case of narrow footings and settlement in the case of wide footings.

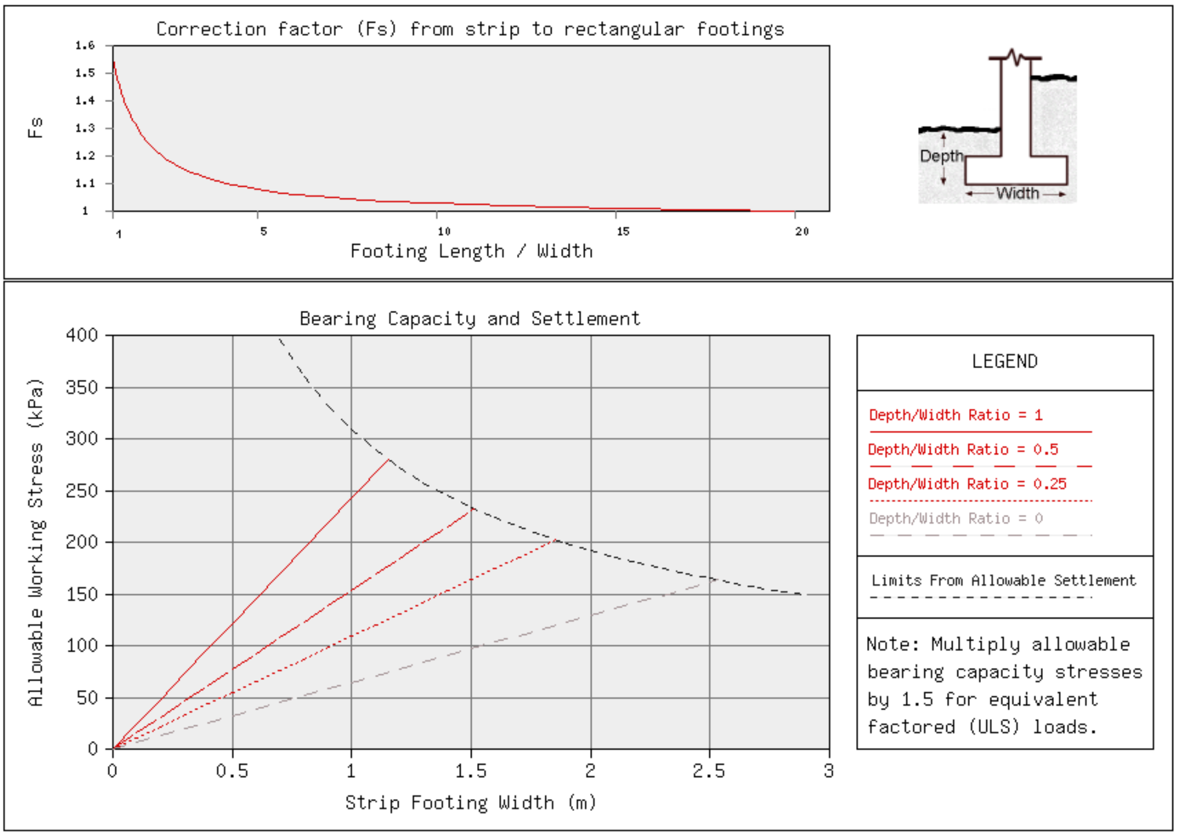

Figure 3: Recommended Bearing for Shallow Footings on Alluvial Gravel

From Figure 3 it can be seen an allowable working stress of approximately 100 kPa is recommended for a 400 mm wide by 400 mm deep strip footing founded within alluvial gravel. This corresponds to a factored (ULS) bearing capacity of approximately 150 kPa and an ultimate geotechnical bearing capacity of 300 kPa.

Minor liquefaction induced settlement could have some effect on an existing building with a shallow foundation; nearby testing estimates liquefaction induced settlement of 0-50 mm in a ULS seismic event.

In future construction the effects of liquefaction below the site is expected to be negligible as foundations are recommended to be constructed on piles bearing upon the non-liquefiable alluvial gravel unit below the site.

6.4.3 Foundations for 3 to 4 Storey Buildings

It is recommended that foundations for future multi-story development in this area are constructed on piles bearing within the underlying alluvial gravel. This has been observed between 2 and 4 m bgl at surrounding sites.

Screw piles, bored or driven piles can be considered for future construction. The recently constructed ICC building at 16-24 Don St, 150 m to the north, has 7 m long 800 mm diameter cased bored reinforced concrete piles supporting the structural loads. The foundation slab is supported on shorter, smaller diameter piles.

Bored and Franki pile rigs are available in Invercargill, whereas screw pile rigs will need to be established from Canterbury.

6.4.3.1 Bored Piles/ Franki Piles

Both traditional bored concrete reinforced piles and Franki piles are considered suitable for future construction.

The alluvial gravel below the two sites being assessed is estimated to be between 2 and 4 m bgl. However, a loose sand layer has been observed in discrete locations in the area surrounding the sites.

Piles should be installed a minimum of 3 pile diameters into the medium dense to dense gravel unit interpreted to underlie the sites to ensure that full end bearing is achieved.

Casing is likely to be required to support the pile bore during construction, due to the loose soils and relatively shallow groundwater.

6.4.3.2 Driven Timber Piles

A cost effective and relatively straightforward option may be to drive timber piles onto the gravels. The timber piles should be driven with a piling hammer to achieve a set determined using appropriate pile driving formula (e.g. wave equation analysis or Hiley formula). However the vibration effects of driven piles on nearby structures will have to be considered.

Trial piles should be carried out in advance of the main piling works to confirm pile depths.

Driven timber piles are more likely to be suitable to support minor structural loading or floor slabs.

6.4.3.3 Screw Piles

A screw pile consists of a steel circular hollow section with a helix welded tip and is installed by screwing it tip first into the ground. This piling method is advantageous as minimal vibration and noise is caused during construction, and it can be designed for both tension and compression forces. The design of screw pile is specialist and typically undertaken by the contractor who will be installing the piles. This design will require sonic boreholes to confirm design parameters and suitability of the installation and is a requirement of screw piling contractors.

6.5 Site Subsoil Category

For detailed design purposes it is recommended the magnitude of seismic acceleration be estimated in accordance with the recommendations provided in NZS 1170.5:2004.

Existing nearby drilling data suggests the site is Class D (deep soil site) in accordance with NZS 1170.5:2004 seismic provisions. A deep borehole contacting to bedrock would be required confirm whether Class C or D is appropriate.

6.6 Neighbouring Structures

The construction contractor should take the appropriate measures to control the construction noise, in accordance with Invercargill City Council requirements.

It is expected that conventional earthmoving equipment, such as hydraulic excavators, rollers and trucks as well as heavy piling equipment will be required during future building construction.

During fill compaction and pile driving/augering care should be taken to ensure that neighbouring properties are not adversely affected by ground vibrations, especially if fill and piles are being constructed in close proximity to neighbouring structures.

With regards to occupied properties in the wider area, the construction contractor should take appropriate measures to control the construction noise and vibration and ensure Invercargill City Council requirements are met.

7 Conclusions and Recommendations

- Data held on the GeoSolve database infers the geological model underlying the site areas comprise uncontrolled fill overlying alluvial silt, overlying discrete layers of alluvial sand, overlying alluvial gravel to moderate depth;
- The old Southland Times and Government Life building foundations are expected to comprise shallow strip footings, however the Government Life building does have a basement which decreases the thickness of alluvial silt underlying the foundations. Due to the basement that has been previously constructed the Government life building may be constructed upon alluvial gravel or a comparatively thin layer of alluvial silt overlying the alluvial gravel. This would have to be confirmed with site specific investigations;
- Shallow footings bearing upon alluvial silt are expected to provide an allowable bearing capacity of 40 kPa for a 500 mm wide and 500 mm deep footing. This is significantly below NZS 3604's definition of "good ground";
- Minor liquefaction induced settlement is predicted from testing completed on nearby sites in the Invercargill CBD. Between 0-50 mm of liquefaction induced settlement is predicted at nearby sites with the groundwater level predominately being within the alluvial sand and gravel underlying the area. Discrete lenses of loose alluvial sand are predicted to liquefy in a ULS seismic event;
- From existing nearby drilling the seismic soil classification for the site is considered likely to be Class D, however a deep borehole contacting to bedrock would be required to confirm whether class C or D is appropriate for design;
- Piles are recommended for future multi-level building foundation construction. Pile options are outlined in section 6.4 of this report. During the recent construction on

the ICC Building (16-24 Don St), 7 m long 800 mm diameter cased, bored concrete piles were installed.

- A risk of seismic activity has been identified for the region as a whole and appropriate allowance should be made for seismic loading during detailed design of the proposed building and foundations.

8 Applicability

This report has been prepared for the benefit of Batchelar McDougall Consulting with respect to the particular brief given to us and it may not be relied upon in other contexts or for any other purpose without our prior review and agreement.

It is important that we be contacted if there is any variation in subsoil conditions from those described in this report.

It is understood that site specific investigations will be undertaken for future building foundation design.

Report prepared by:

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Reviewed for GeoSolve Ltd by:

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