Southland Times Building, 67 Esk St, Invercargill, 9810

DETAILED SEISMIC ASSESSMENT REPORT

Table of Contents:

Revision	Date	Description/Reason for Issue		
A	23/02/2018	Formal DSA Issue		
		Prepared by	Reviewed by	Approved by
	Name	Warren Holt	Andrew Marriott	Graham McDougall
	Signature			G.R.McDougall
	Post Nominals	BEng(Hons),CMEngNZ, CPENG, IntPE(NZ), MIStructE(UK), CEng(UK)	BE(Civil), CMEngNZ, CPEng, IntPE(NZ), M.ICOMOS	Director, BE, CMEngNZ, CPEng, IntPE(NZ)

1.0 Executive Summary

The following report covers the detailed seismic assessment of the building at 67 Esk Street, Invercargill. The essentially free standing three storey building (with a relatively small basement), constructed circa 1908, is 37.3 m long and 17.25-15.6 m wide. The construction consists of URM walls of varying widths supporting steel roof trusses and timber joist floors with internal steel beams and column gravity supports. There are some reinforced concrete veirendeel frames trimming 'new' openings to the URM walls (generally internal). Reinforced concrete foundations of an undetermined nature support the perimeter and load bearing walls. The building layout and configuration is shown in section 5.0.

BMC have been commissioned by HWCP Management Ltd to advise them on the current condition and strength of the building. We have visited the building to view its current condition, take selected samples of building fabric, identify the structural systems within the building and carry out an analysis of these elements to determine their capacity with respect to the current Ultimate Limit State (ULS) demand.

For the purposes of this evaluation, the above described building has been assessed as a single monolithic structure of Importance Level 2. 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 MIBE et al, referred to as SAEB from herein.

The building has previously been strengthened in 1986 and BMC has reviewed these works as part of the assessment. The building is considered to have a capacity of 20% of New Building Standard. The capacity of the building is limited by inadequate diaphragm connections to some primary structure bracing walls (specifically @ 1st floor level).

Loading Direction	%NBS (IL 2)
N-S	20-25%
E-W	20%

BMC notes that the governing elements of the structure are the diaphragm connections into the URM bracing walls particularly where the joists run parallel to the wall where the 1984-6 strengthening works provided anchors at only 1.2m centres. The Out of Plane capacity of the parapets and second floor walls and their connections into the buildings diaphragms are also of concern with a capacity of 33%NBS.

Geotechnical input indicates the supporting high level soils to be less than "Good Ground" with an allowable bearing capacity of 120kPa (Good Ground = 150kPa). Little conclusive evidence of the foundation depth and width is available however there is no evidence of cracking / settlement indicating that this is a problem. The water table depth is approximately at 1.5m below ground level and it is noted that there is likely to be some liquefaction under a ULS event.

Structural works could be undertaken to increase the capacity of the building to raise the %NBS capacity as follows:-

- The installation of adequate fixings between the diaphragms and the walls to the required demands for tension and shear (particularly on the first floor and fixings of joists parallel to the walls.
- The installation of steel tie backs framing to the tops of the effected parapets into the roof structure and to the wall structure below roof level (to prevent uplift of the parapet).
- The installation of internal timber framing and wall ties into the second floor walls to be fixed back to the diaphragms at the top and bottom of the wall (to improve the out-of-plane capacity).

In summary the Southland Times building is considered earthquake prone and in terms of structural strengthening and condition, in our opinion, it is feasible that this can be strengthened without loss to the heritage fabric. The potential lifespan of the now 110 year old building is limited and we feel that given the rate of development of analytical techniques for URM buildings and their inherent instability under Seismic lateral loading it may have to undergo periodic reassessment as the guidance develops. In addition, in the case of a significant seismic event even if strengthened some of the structural integrity of the critical wall elements may well be damaged beyond economical repair and therefore require demolition of the building.

It is also possible to retain the Façade (arguably the primary feature of the Cat 2 Heritage status) as part of any masterplan design and / or new build behind. This would require a significant temporary support structure during the demolition and rebuild.

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 Southland Times Building at 67 Esk 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.

Note: Preparation of concept strengthening strategy is also provided in this report.

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.
- Inspection of the building to familiarise ourselves with the structure, visually assess its condition, observe important structural and seismic characteristics, and note obvious deficiencies.
- Undertake intrusive investigations to determine; floor constructions, existing strengthening provisions and wall thickness at various locations and levels.
- Engage OPUS Laboratories to undertake brick compression testing.
- Carry out a DSA to determine the likely seismic performance of the building (including the 1986 strengthening works).
- Provide concept information on strengthening works required to provide >67%NBS capacity.
- 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:

- Engineering alteration / strengthening drawings prepared by E R Garden & Partners Ltd 1981-1986
- Alteration drawings prepared by L. F. Simpson
- Visual survey undertaken & indicators of defects present at the time of inspection (including opening up of some hidden critical areas).

2.4 Inspection

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

The scope of this evaluation is limited to the assessment of the potential performance of the building in an earthquake only. No assessment has been made of other load cases such as wind, snow and gravity (although it is likely that floor live load capacity of 3.0kPa for office/retail currently required is less than the original design value and therefore is adequate). The assessment is made in the context that the building may potentially be affected by the Earthquake Prone Building (EPB) provisions of the Building Act (2004) (Incorporating the Building (Earthquake Pro ne Building) Amendment Act 2016).

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 and therefore some allowance or contingency is required if strengthening is considered.

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 Ltd to provide a "Desktop" study for the entire CBD redevelopment block. Refer to Section 7 of this report for recommendations and Geotechnical Reports that we may have obtained.

Assessment of earthquake loadings only, no other load cases have been assessed.

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 Southland Times Building is in a medium Seismic Risk Area.

3.1.1.3 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 Southland Times Building is classed as a priority building as it comprises unreinforced masonry parapets which may potentially collapse onto a busy street frontage.

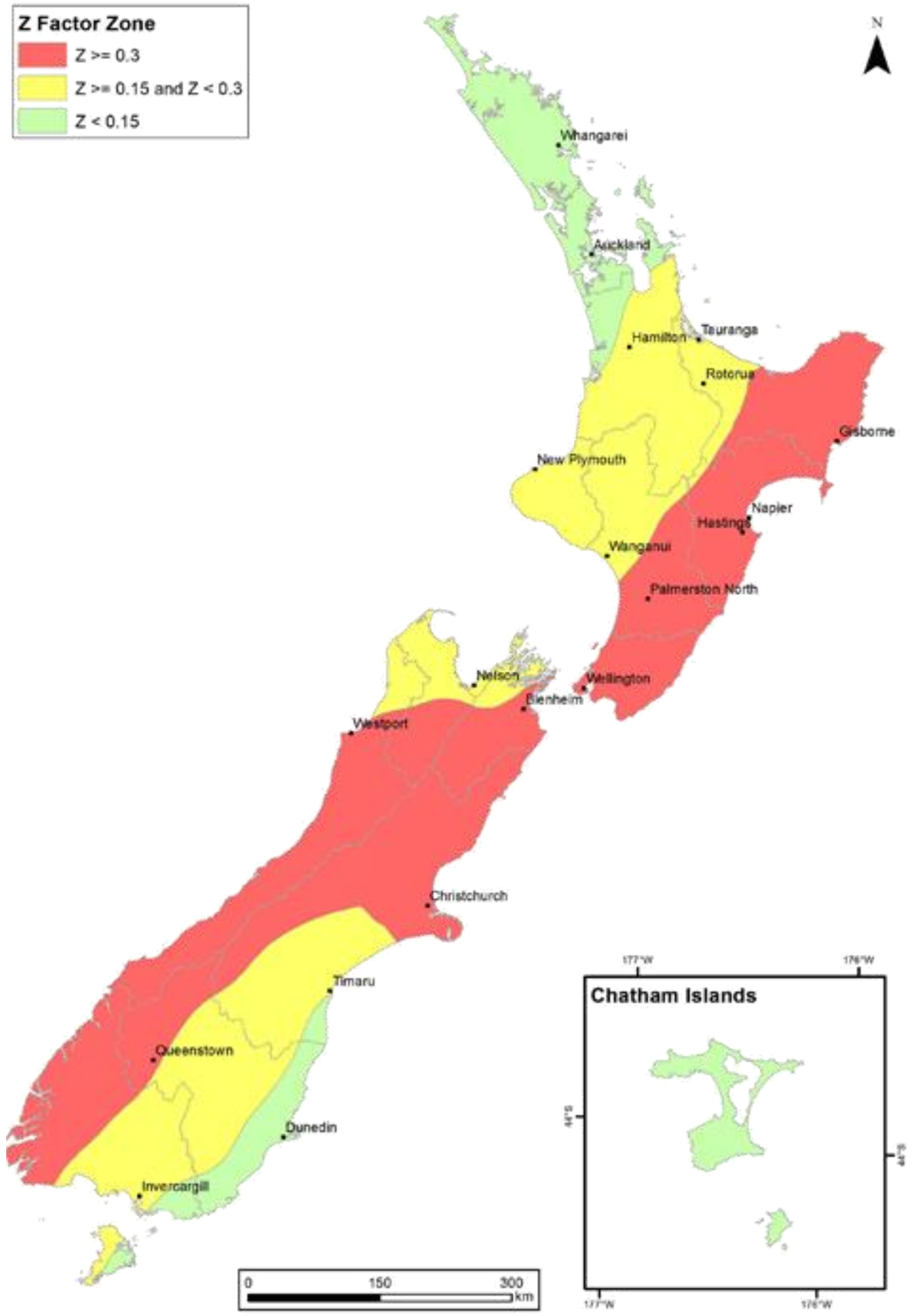

Figure 1 - Three seismic risk areas in map format (map produced by GNS Science)

3.1.1.4 Timeframes for Identifying Earthquake-prone Buildings

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

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

The above timescales commenced on the 1st July 2017.

The Southland Times building will be required to be demolished or strengthened within 12, 5 years. The ICC must issue an Earthquake-prone Building notice when it determines/confirms that a building or part of a building is earthquake-prone.

3.1.2 Building Alterations – Section 112

Under the Building Act:

- Alterations to Earthquake-prone Buildings may be allowed even if after those alterations to 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.

3.1.3 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.

4.0 The Site

4.1 Site Location

The Southland Times building is situated on Esk Street adjacent to the Kelvin Hotel at the corner of Esk Street and Kelvin Street, Invercargill (refer Figure 3). The site has one street frontage, (Esk Street) to the North. The East side of the building is isolated from the adjacent buildings and the East and South sides are directly linked to more recent extensions of the building as a whole which are not part of the heritage listing and are to be demolished as part of the proposed development works in the area. Beyond these extensions these elements are isolated from the adjacent buildings.

4.2 Site Description

The site is rectangular in shape and is approximately 37.3 m long and 17.25-15.6 m wide, thereby occupying and area approximately 602 m². The site is flat and sits approximately 10 m above sea level.

The site is developed by a three storey (with a part basement), category two heritage building. The building is currently vacant, originally housing the Southland Times newspaper offices (including the printing presses in the RC (non heritage) part of the complex).

The site can be accessed from both Esk Street, a pedestrian alleyway to the West and an access driveway between its East extension and the Kelvin Hotel.

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 multi-storey unreinforced masonry (URM) retail and commercial buildings.

Immediately behind the building (to the South) is a three storey concrete building which was an office extension to the Southland Times building and is currently unoccupied. Similarly directly to the East is the other extension element which is partly two storey office element to Esk Street and then to the rear a single storey (but a double storey height) printing press room. To the West across a pedestrian access are a series of single and double storey URM buildings which are a mixture of retail stores, offices and storage facilities. Esk Street to the North of the building is a largely pedestrian street.

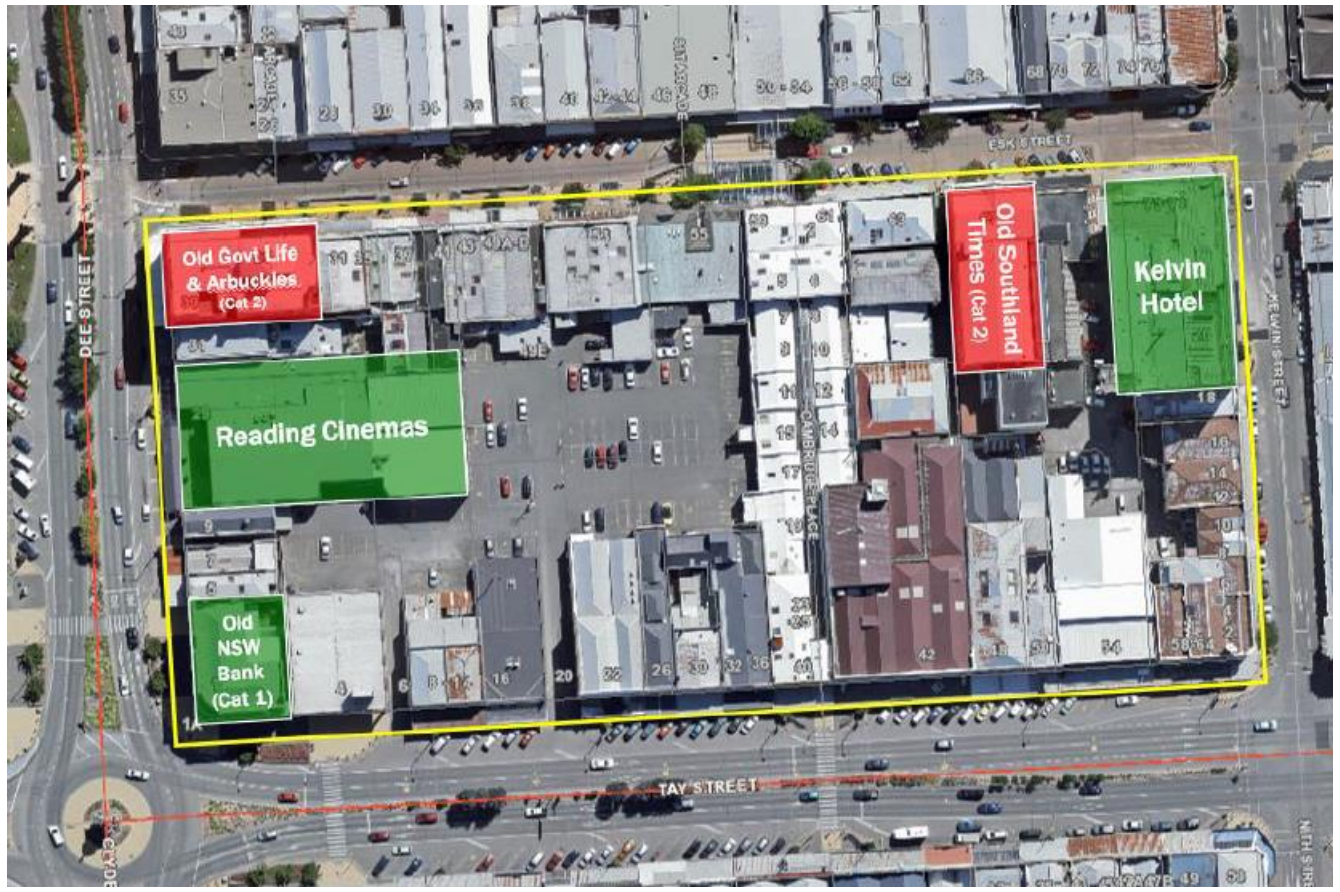

Figure 3 – ICC aerial image of the site. The buildings in 'green' are planned to remain, those in red (Category 2 heritage buildings) are to be demolished as part of the redevelopment plan.

Figure 4 – View of site from the North elevation on Esk Street

5.0 Building Description

The Southland Times Building is a three storey plus part basement building on the Esk Street and was constructed circa 1908. The structure shows evidence of being a two part construction with changes in width and roof forms between the front and rear elements as shown in Figure 5.

5.1 Building Form

The Southland Times Front (North) section of the building is three storeys high (with no basement) and is approximately 12.15 m deep (N-S) and 17.25 m wide (E-W) giving an approximate footprint of 210 m² at ground floor level. The construction largely comprises URM concrete brickwork walls with steel roof trusses.

The Southland Times Rear (South) section of the building is three storeys high (plus a small basement) and is approximately 25.15m deep (N-S) and 15.6m wide (E-W) giving an approximate footprint of 392m2 to ground floor level. The basement of Southland Times Rear is approximately 5m deep (N-S) and 11m wide (E-W) and is located to the North East corner of the rear section at its interface with the front section.

The roof comprised URM parapets throughout and is located at a height similar to both sections eaves. The construction consists of largely URM with timber floors. The overall floor area of the building is therefore approximately 602 m².

The 1954 rear office extension and the 1981 East office and print room extensions are to be removed as they are not part of the Heritage Listing leaving the remaining original structure as a freestanding structure as the basis of this assessment.

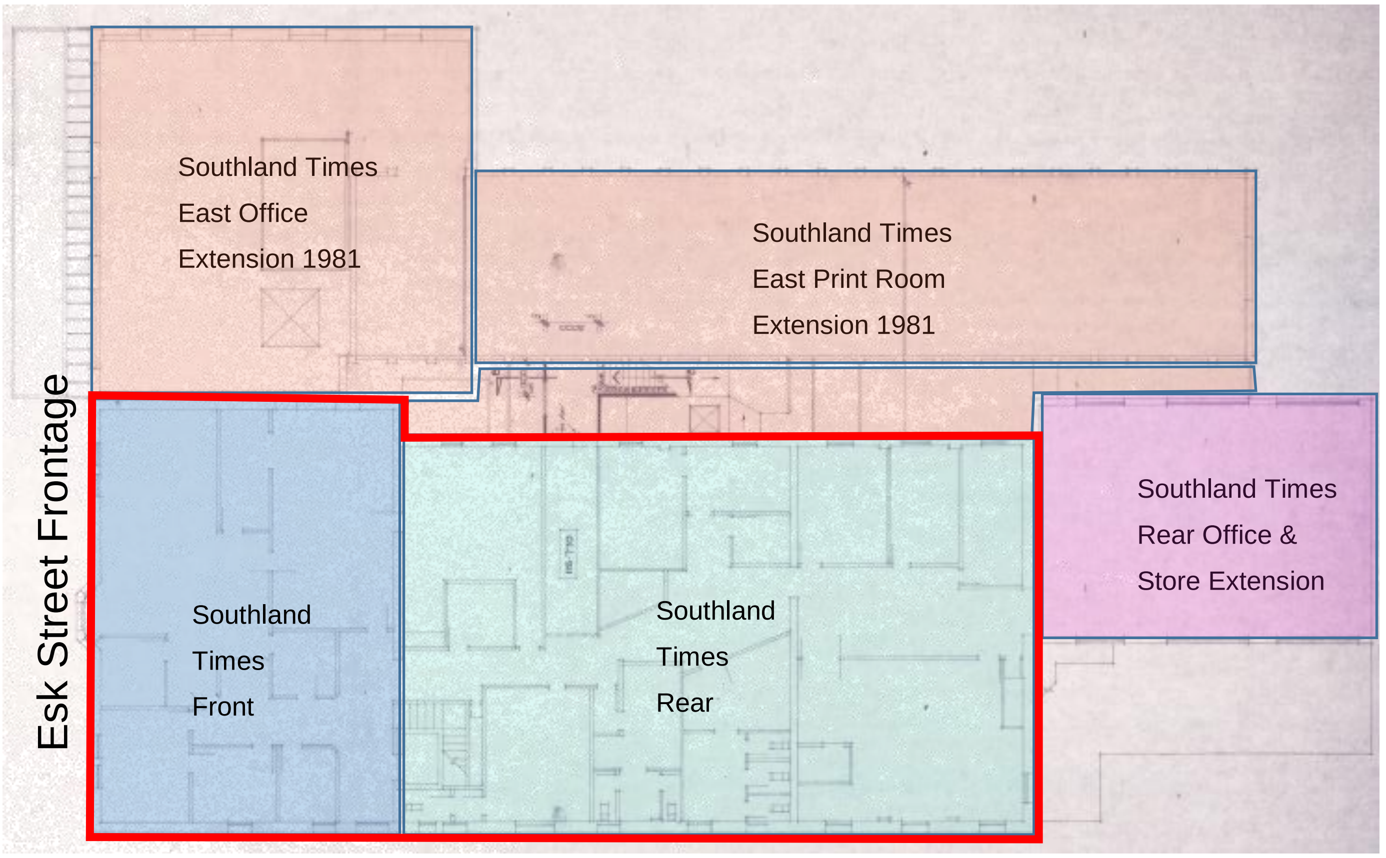

Figure 5 – Government Life building general arrangement plan (NTS). Heritage Category 2 listing building extents in red

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 Stairswells

The building incorporates a stairwell of concrete construction located centrally within the structure at the rear elements interface with the front element. This stair wraps around a URM lift shaft

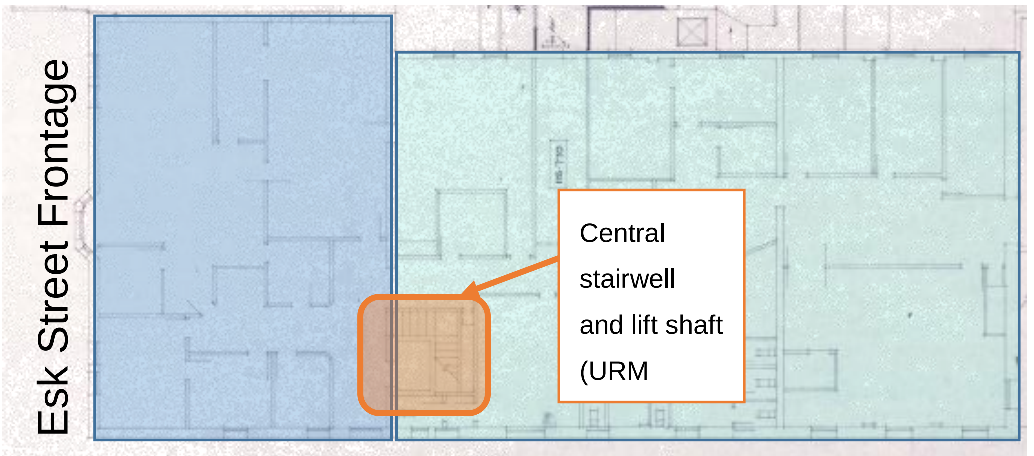

Figure 6 - Location of secondary features (NTS).

6.0 The Structure

6.1 Gravity load resisting system

The structural systems to both the front and rear elements are of an identical nature with only the plan size, roof orientation and façade embellishments forming the differences between the two

6.1.1 Southland Times General

Roof Level

- → URM parapets 350mm thick at the façade and 240mm elsewhere.
- → Steel angle trusses flat soffit to front elevation and raised tie to the rear.

Second Floor

- → 300 x 50 mm deep @ 400mm crs timber joist floor with timber straight boarding and wet plaster ceiling.
- → 350mm thick URM external walls (solid – no cavity).
- → 470mm thick URM internal walls (solid – no cavity).
- → Inset Reinforced concrete wall opening frames to internal walls.
- → Some localised steel strongback reinforcement to walls.

First Floor

- → 300 x 50 mm deep @ 400mm crs timber joist floor with timber straight boarding and wet plaster ceiling.
- → 350mm thick URM external walls (solid – no cavity).
- → 470mm thick URM internal walls (solid – no cavity).
- → Inset Reinforced concrete wall opening frames to internal walls.
- → Steel UB floor beams.

Ground Floor

- → 100mm thick reinforced ground bearing slab with a concrete metal decking suspended slab over the basement area
- → 500mm thick façade, 470mm West and Rear and 350mm thick East URM external walls (solid – no cavity).
- → 470mm thick URM internal walls (solid – no cavity).
- → Inset Reinforced concrete wall opening frames to internal walls
- → Localised steel strongback reinforcement to walls
- → Steel SHS posts/columns and UB floor beams

Basement

- → Profiled metal decking with insitu reinforced concrete topping
- → concrete Brickwork URM walls
- → Insitu concrete floor slab

6.1.2 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.

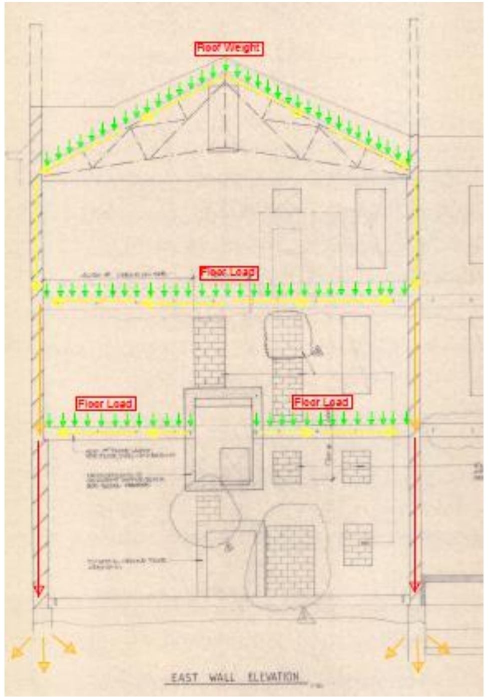

Figure 7 – Southland Times West elevation gravity load path.

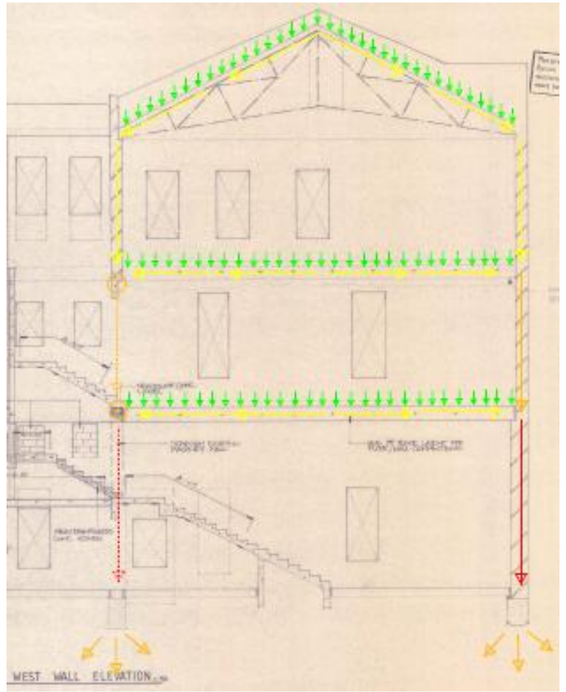

Figure 8 – Southland Times East elevation typical gravity load path.

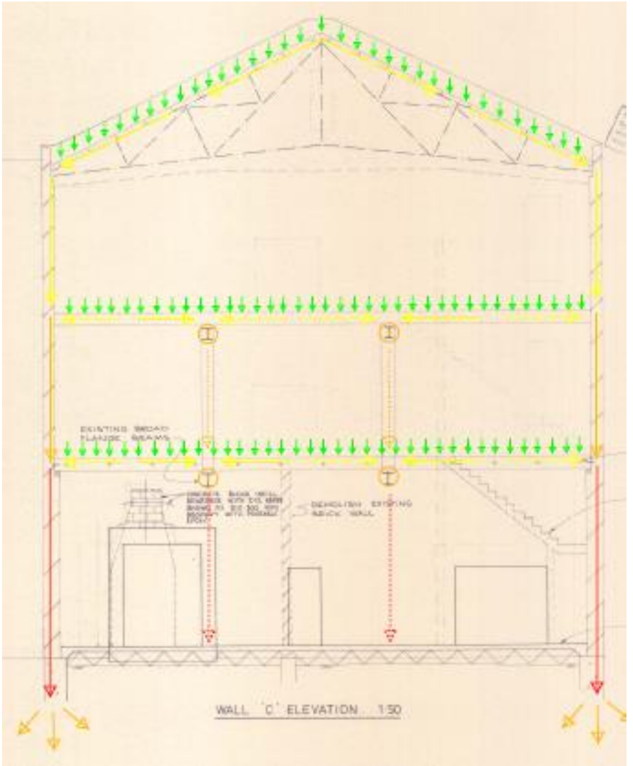

Figure 9 – Southland Times internal wall C gravity load path

6.2 Lateral load resisting system

6.2.1 Southland Times General

6.2.1.1 Transverse direction

Roof Level

- → Steel angle hips, valleys and cross bracing
- → Plasterboard ceiling diaphragms
- → URM parapets 350mm thick front and 240mm sides and rear and gable walls Out of Plane (OOP)

Second Floor

- → Flexible timber diaphragm formed by flooring and plaster / GIB ceiling
- → Reinforced Concrete moment frames inset locally into the URM walls to openings
- → 350mm thick external and 470mm thick internal Shear walls comprising URM
- → Side walls OOP with localised strongback reinforcement.

First Floor

- → Flexible timber diaphragm formed by flooring and plaster / GIB ceiling
- → Reinforced Concrete moment frames inset locally into the URM walls to openings
- → 350mm thick external and 470mm thick internal Shear walls comprising URM
- → Side walls OOP.

Ground Floor

- → Reinforced Concrete moment frames inset locally into the URM walls to openings
- → 500mm façade, 470mm thick West and Rear and 350mm thick East thick external and 470mm internal Shear walls comprising URM
- → Side walls OOP

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 reinforced concrete moment frames inset into the rear elements East side wall. The URM shear walls provide significant lateral resistance in the longitudinal direction.

The roof plane steel bracing (part of the 1986 strengthening works) provides Out-of-plane support to end walls and the roof seismic mass, transferring loads to the longitudinal side walls.

Floor diaphragms at 1st and 2nd floors similarly transfer OOP loads to longitudinal side walls.

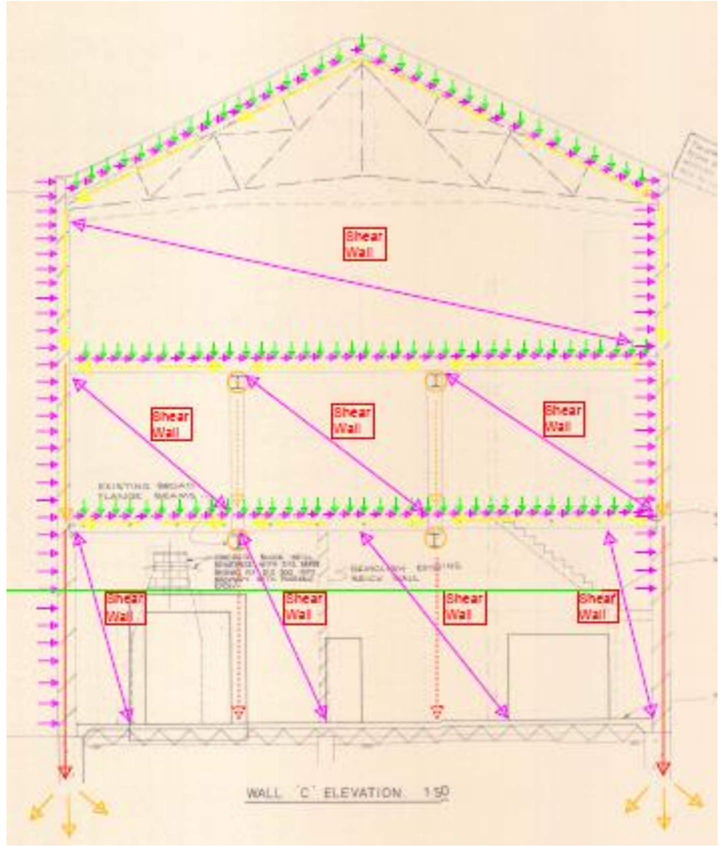

Figure 10 – Southland Times Internal cross wall C typical transverse lateral load path

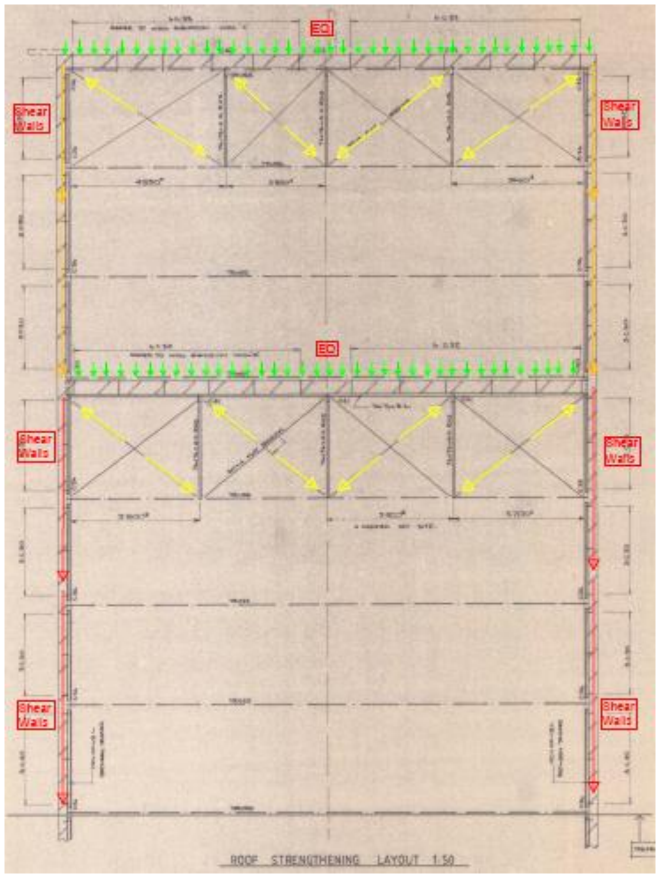

Figure 11 – Southland Times rear element roof bracing longitudinal lateral load path

7.0 Geotechnical Considerations

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

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 b.g.l 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 Southland Times 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 are expected to have a higher geotechnical ultimate bearing capacity of 300 kPa. Figure and Figure outline these bearing capacities. Note low allowable bearing for larger footings.

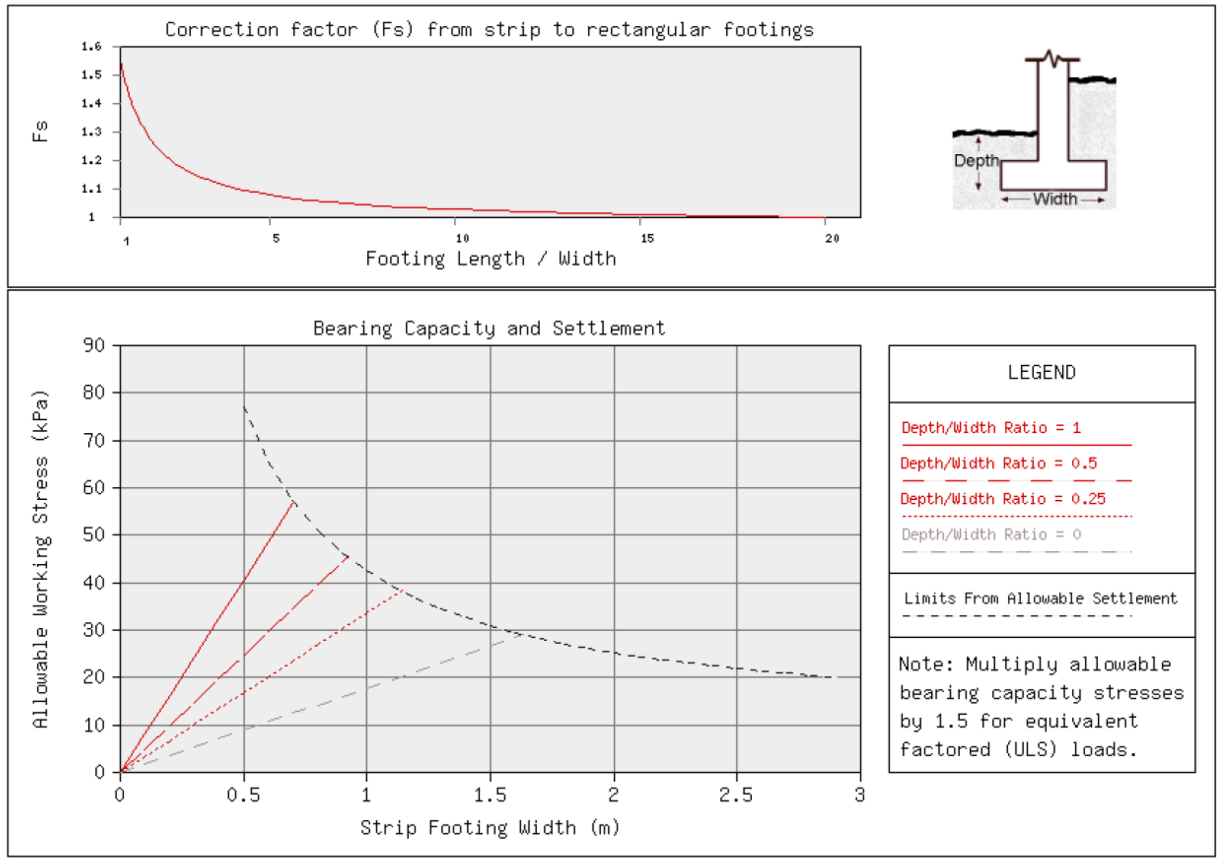

Figure 12 – Typical bearing for shallow footings on alluvial silt (extract from Geosolve Report).

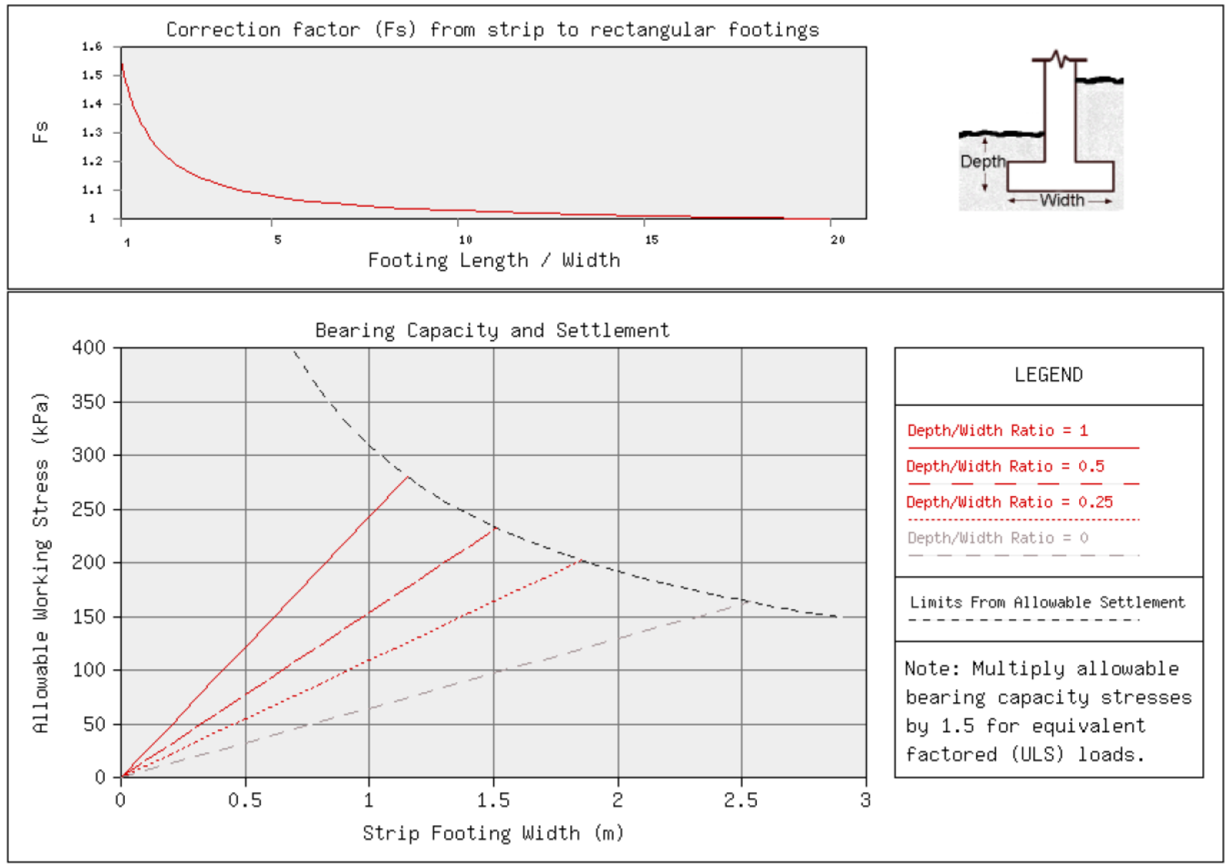

Figure 13- Recommended bearing for shallow footings on alluvial gravel (extract from Geosolve Report).

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
Unreinforced Masonry –	Concrete Bricks	compressive strength, f'b	14 MPa	OPUS Concrete compression test report, ref CH3687, and dated 22/01/2018.
	Mortar	compressive strength, f'j	1.5 MPa	SAEB Part C Section C8.7.2 table C8.4.
	Wall elements	Cohesion C	0.3 MPa	SAEB Part C Section C8.7.2 table C8.4.
	Spandrel elements	Coefficient of friction μf	0.3	SAEB Part C Section C8.7.2 table C8.4.
	All concrete members	Modulus of elasticity	1.64 GPa	NZS 3101: Part 1:2006 clause 5.2.3
Concrete	Frame members	probable compressive strength, f'cm	40 MPa	SAEB Part C Section C5.4.2.2- table C5.3.
	All concrete members	Modulus of elasticity	13-20 GPa	NZS 3101: Part 1:2006 clause 5.2.3
Steel Reinforcement	D bars	Lower characteristic yield strength	275 MPa	SAEB Part C Appendix C5-22 table C5C.4
		Probable yield strength	340 MPa	SAEB Part C Section 5 C5.4.3.2
	HD bars	Lower characteristic yield strength	410 MPa	SAEB Part C Appendix C5-22 table C5C.4
		Probable yield strength	465 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	276 MPa	SAEB Part C Appendix C6-10 table C6B.10
		Probable yield strength	317 MPa	SAEB Part C Section 6 table C6.2
	All structural steel	Modulus of elasticity	205 GPa	NZS 3404:Part 1:1997

8.1.1 Importance Level

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

8.1.2 Design Working Life

The Southland Times 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
CdT	Varies	Modified by SAEB C8 Guidance Documents

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)
Southland Times (Cat 2 Heritage building only - see figure 5)	14078	602	23.4

Please Note: The performance of the building under 'Serviceability' (SLS) seismic loads has not been addressed.

A review of the GNS strong motion data for Invercargill (earliest record 1994) shows a Peak ground Acceleration (PGA) of 0.03g and 20mm displacement in the 2009 Milford Quake, that is significantly less than the expected ULS event. Recent research has suggested under a ULS Alpine Fault event the expected strong motion shaking is to be approximately 45 seconds in duration.

9.0 Seismic Assessment Procedures

9.1 Analysis Techniques

The URM wall elements have been analysed using the procedures within the NZSEE Guidance document "The Seismic Assessment of Existing Buildings" July 2017 Section C8 - Unreinforced Masonry Buildings. This follows a step by step assessment procedure of the elements determined, through testing and experience, to the most critical elements in the building. The URM element assessment is essentially a linear elastic analysis technique. There are potentially elements within the structure which may form part of the primary structure which may be analysed using the Nonlinear Static Pushover Analysis which is a form of nonlinear analysis if the linear elastic elements are found to be deficient in their initial assessment. However we have not needed to resort to these techniques for these elements as they have found to be >100%NBS under linear elastic techniques.

A short description of analysis techniques is provided below.

Please note that the SAEB C8 – URM assessment process is cited in the Building Act as the approved procedure to be used in a Detailed Seismic Assessment.

9.1.1 Elastic analysis techniques

Elastic analysis techniques rely on linear-elastic assumptions and maintains the use of linear stress-strain and force-displacement relationships. Implicit material non-linearity (e.g. cracked section) and geometric nonlinearity may be included. Includes equivalent static analysis and modal response spectrum dynamic analysis. Under the equivalent static method, 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. In URM element analysis the lateral load distribution method of NZS1170.5:2004 is ignored for a tributary height distribution of the load as specified in SAEB C8.

9.1.2 Nonlinear analysis techniques

Nonlinear structural analysis techniques incorporate material nonlinearity (strength, stiffness and hysteretic behaviour) or plastic response of building materials after initial elastic capacity is exceeded (that is it bends or yields). This analysis technique may be used for the inset reinforced concrete frames, however these elements were found to be >100%NBS under an elastic analysis.

Typically brick buildings do not exhibit this behaviour.

9.2 Analysis Procedure overview

9.2.1 Southland Times

The structural analysis was generally completed in accordance with C8 of the technical guidelines, section C8.5.2 Illustrates the Assessment process. Steps 1 (Gather Documentation), Step 2 (Decide on level of assessment based on building complexity), Step 3 (On-site Investigations) and Step 4 (Assess material properties) have been completed.

9.2.1.1 Step 5 – Identify potential structural weaknesses and rank in order of vulnerability

The assessed vulnerabilities would rank the structural weaknesses as follows:-

1. Parapet, ornamentation, façade collapse under Out of Plane actions.
2. Wall / diaphragm connections (for both in-plane shear and out-of-plane tension loads).
3. Out–of-Plane Instability of face loaded URM walls second floor to ground floor for each wall.
4. Timber Diaphragm strength and stability.
5. In-plane capacity of URM walls.
6. In-plane capacity of reinforced concrete frames embedded in URM walls.

9.2.1.2 Step 6 – Assess member / element capacities

- Timber Diaphragm strength and stability Section - C8.8.3 – This will utilise the depth and width of the various diaphragms using their joist span direction, the inspected decking and ceiling construction type, condition and fixings to determine their deflection under the loading from self-weight and the out of plane wall loads. It is considered from this assessment that the diaphragms are deemed as being flexible and thus the in-plane wall demands will be derived from tributary width loads independent of any torsional effects due to the relative stiffness's of the other bracing walls in the building. Assessment has found these to all be >100%NBS capacity.
- Wall / diaphragm connections - Section C8.8.4 - The diaphragm reactions derived from the diaphragm strength assessment above will determine the shear loading to the diaphragm fixings both original and as part of the strengthening works of 1986. This includes the bearing capacity of embedded timber joists where they are built into the wall construction (second floor) in the loaded direction and manufacturer's data from equivalent anchors / connections identified from archive drawing and verified by site inspections. Critical shear capacity under diaphragm end shears is 19%NBS. Out of plane wall assessment reactions to the restraining floor and ceiling diaphragms were checked against the available capacity of the fixings as verified above and any frictional resistance to the joists ignored to cater for the potential of vertical seismic loadings. Critical Pull-out capacity under OOP forces is 61%NBS.
- Out–of-Plane Instability of face loaded URM walls and parapets – Section C8.8.5. The capacity of parapets with a base only restraint and the walls restrained by floors /ceiling diaphragms to their top and bottom are assessed under the virtual work based assessment recommendations utilising their head and base support conditions. The degree of horizontal deformation by the diaphragm as assessed above is utilised to determine the elements capacities. Axial loading has a positive effect on the capacities and as this

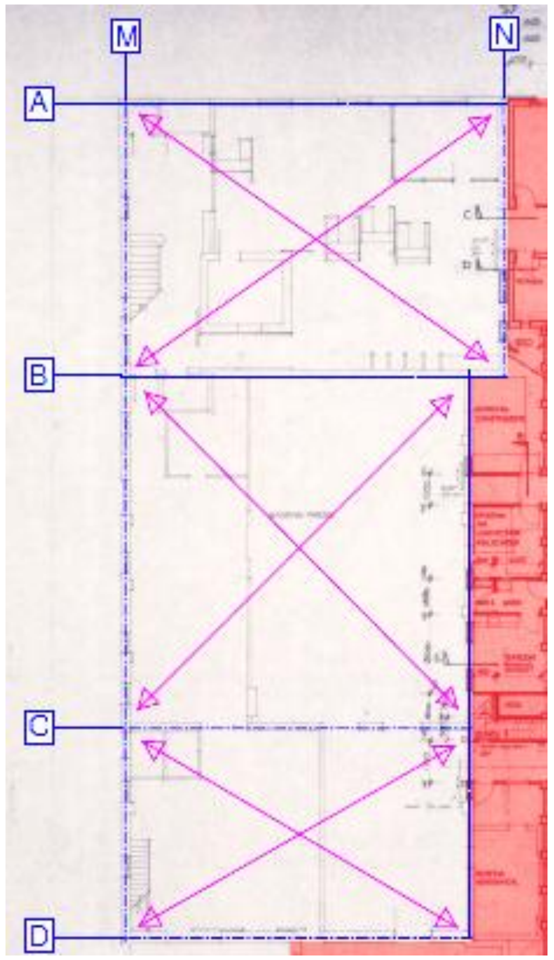

Figure 14 – Southland Times Bracing lines of Primary Lateral Structure and floor diaphragm extents plan

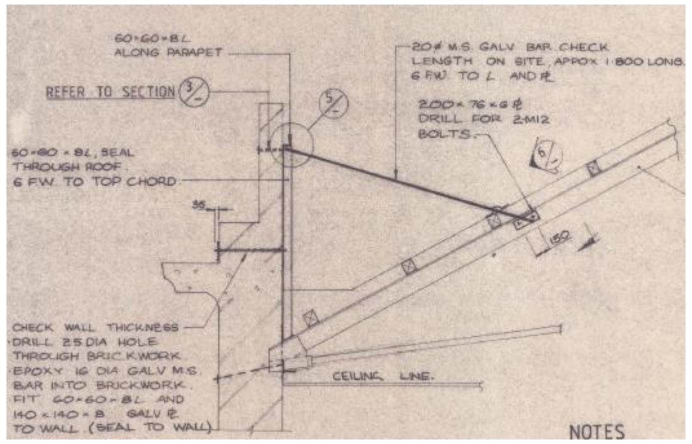

Figure 15: Façade parapet current restraint system

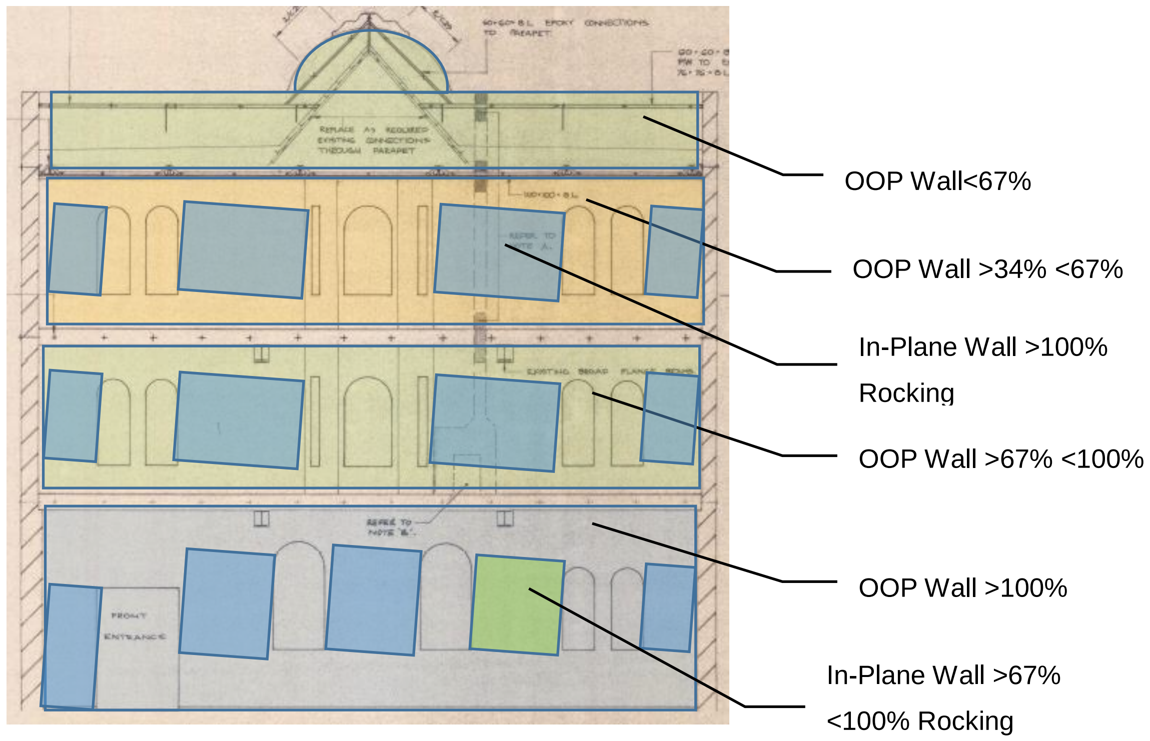

Figure 16 – Sketch of Front elevation URM element Outputs and Out of Plane Capacities

increases towards ground level the wall capacities are found to increase accordingly toward ground level. Capacities were found to be limited to 33%NBS to the critical parapets and second floor walls. The façade parapet has received a strengthening restraint as part of the 1987 work which is deemed, by inspection, to raise its capapcity to >67%NBS. See figure 2 for the strengthening provision detail

- In-plane capacity of URM walls. – Section C8.8.6 – In-plane wall capacities are derived by assessing the critical capacity for each wall and spandrel element (horizontal element of the wall above or below an opening) within the wall at each level. Given the deep spandrels the wall pier / panels were found to have failure mode either by rocking or bed joint sliding with a limiting capacity of 76%NBS. The modes of failure from each element are derived to determine the critical loading condition.
- In-plane capacity of reinforced concrete frames embedded in URM walls – Section C5.6 – The concrete frames constitute a flexible lateral load resisting system within the wall and as such the deformation compatibility between the stiff URM structure and the frame needs to be considered. Hence the mobilised capacity of the inbuilt frame is limited to that mobilised at the deformation limit of the URM wall. It is feasible to remove the URM wall elements from the assessment, due to their short lengths, as long as this does not present a life safety hazard due to the masonry element failing then the full capacity of the frame can be used using a simple lateral mechanism analysis or a nonlinear static pushover analysis.

9.2.1.3 Step 7 – Analyse the Structure to determine relationship between member / element actions and global capacity

The primary lateral load resisting structures, diaphragms and shear bracing walls were assessed as a series of various contributing elements to determine the global capacity of the whole wall length at each level and bracing line. The rocking failure of slender piers is unlikely to cause a life safety hazard in an isolated case and if this occurs the elements capacity at the point of rocking will be isolated and the loading increased to the other elements until a global capacity is determined.

9.2.1.4 Step 8 – Assess global capacity

This is determined from the results of the various structural weaknesses assessments and the point at which the critical member / element of the primary lateral load resisting system reaches its determined capacity.

9.2.1.5 Step 9 Determine demands and %NBS

The demands arise from the seismic load of the building under the elastic loading, i.e. no ductility, the seismic load is modified by a Kr factor dependent upon the mode of failure of the element under consideration. In this case the Kr for the shear walls will be taken as = 3 (all in accordance with SEAB C8).

The demands are compared to the global element capacities and the %NBS rating determined for each element and then globally as the rating of the critical structural weakness (CSW) which cause a significant life safety hazard.

9.2.1.6 Modelling assumptions

The following assumptions have been made when modelling the building:

1. The diaphragms at each floor are considered as being flexible with sufficient internal capacity to transfer the imposed loads.
2. The reinforced concrete frames and masonry wall elements are in good condition (i.e. no cracking). This assumption is not entirely valid with some isolated minor cracking evident but it is assumed no yielding of reinforcement has occurred.
3. The soils which the walls are founded on have adequate bearing capacity to resist over turning. This assumption would need to be verified by a site specific Geotechnical Engineering assessment of soil bearing capacity but at this stage it is considered a reasonable assumption.

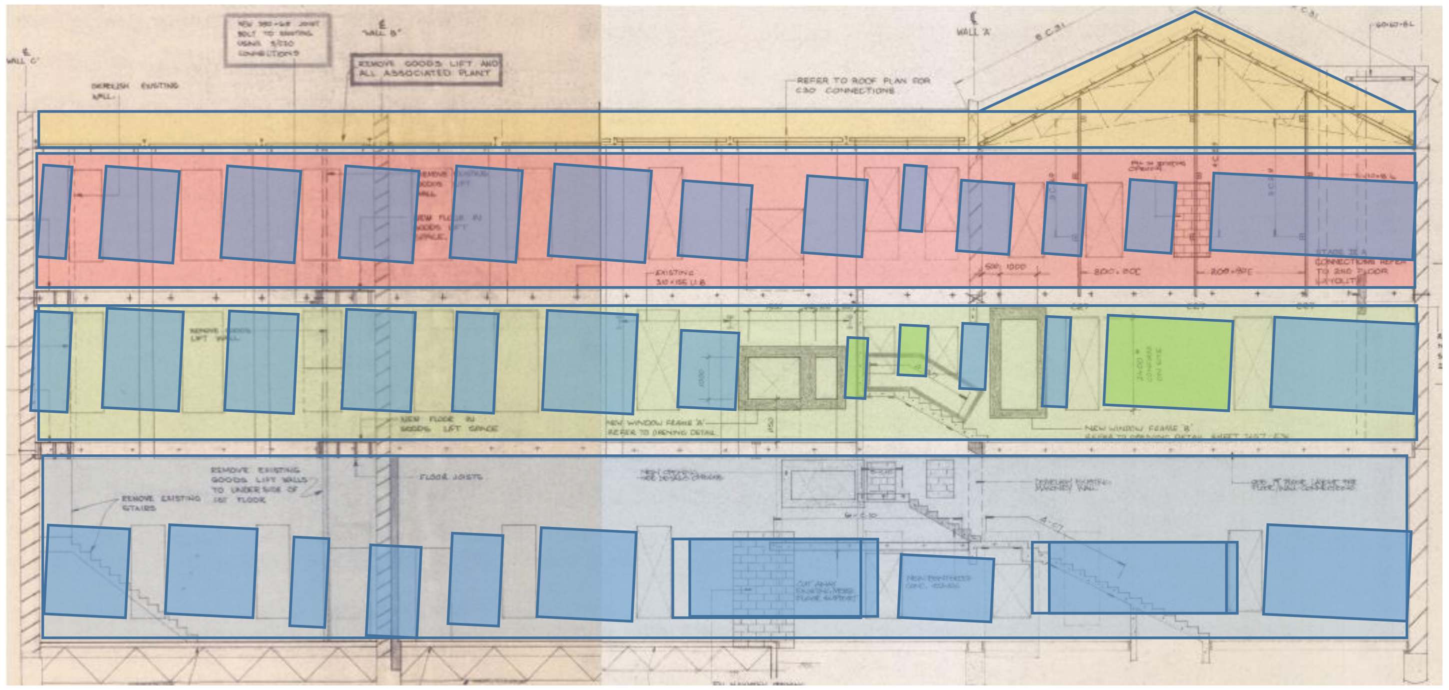

Figure 17 - Sketch of Side elevation URM element Outputs and Out of Plane Capacities (for key of shading see figure 15 above).

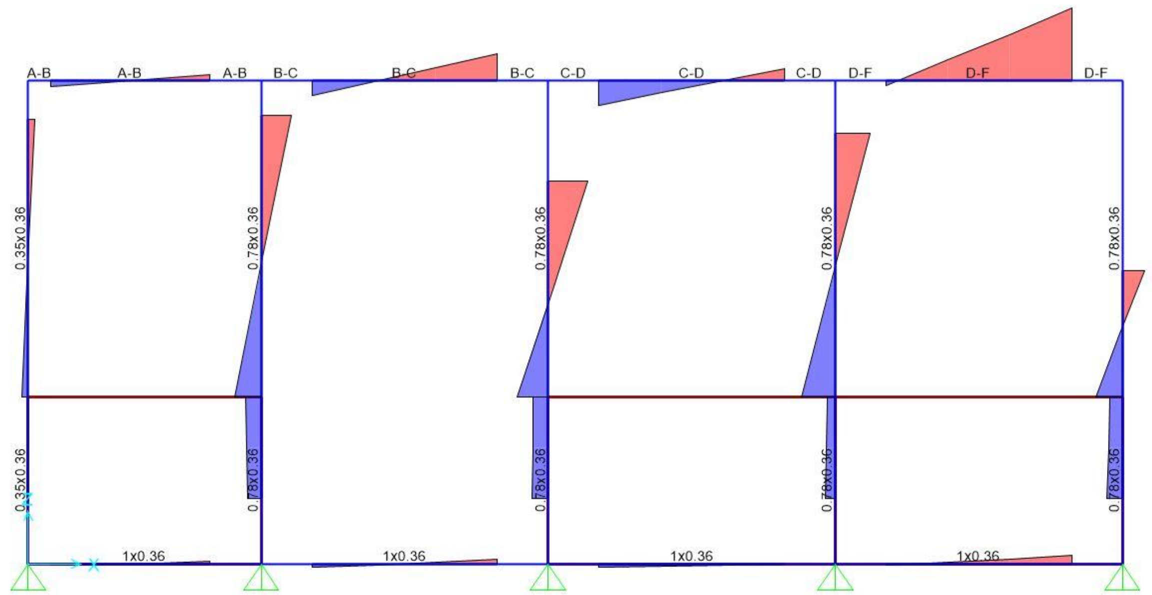

Figure 18 – SAP2000 output of Wall B ground floor RC bracing frame G+E Moment demands.

10.0 Quantitative Results Summary

A summary of the results from the quantitative assessment is provided in the table below.

Building area	Loading direction	Specific review element	%NBS	Notes/Description of limiting criteria
Southland Times	Transverse (E-W)	Side walls – Parapet OOP	40% >100%	URM Wall - Rocking Stability * Connections - N/A
		Side walls – Second Floor OOP	33% 62%	URM Wall - Rocking Stability * Connections Floor - Anchor Pull-out
		Side walls – First Floor OOP	72% 64%	URM Wall - Rocking Stability * Connections Floor – Anchor Pull-out
		Side Walls – Ground Floor OOP	>100%	URM Wall – Stability – N/A *
		Roof Diaphragm - Front	>100% 64%	GIB Sheathing - Deformation - N/A Connections – Anchor Shear
		Second Floor Diaphragm - Front	>100% 34%	Timber Boarding - Deformation - N/A Connections – Anchor Shear
		First Floor Diaphragm - Front	>100% 31%	Timber Boarding - Deformation - N/A Connections – Anchor Shear
		Roof Diaphragm - Rear	>100% 49%	GIB Sheathing - Deformation - N/A Connections Ceiling - Anchor Shear
		Second Floor Diaphragm - Rear	>100% 22%	Timber Boarding - Deformation - N/A Connections Floor - Anchor Shear
		First Floor Diaphragm - Rear	>100% 19%	Timber Boarding - Deformation - N/A Connections Floor - Anchor Shear
		In-plane Façade A – Second Floor	>100%	URM Wall - N/A
		In-plane Façade A - First Floor	>100%	URM Wall - N/A
		In-plane Façade A – Ground Floor	76%	URM Wall - URM Rocking
		In-plane Internal Wall B – Second Floor	>100%	URM with RC Frame - N/A
		In-plane Internal Wall B – First	>100%	URM with RC Frame - N/A
		In-plane Internal Wall B - Ground	>100%	RC Frame - N/A
		In-plane Internal Wall C - Second	>100%	URM with RC Frame - N/A
		In-plane Internal Wall C– First	>100%	URM with RC Frame - N/A
		In-plane Internal Wall C - Ground	93%	URM with RC Frame - URM Sliding
		In-plane Rear Wall D - Second	>100%	URM – N/A
		In-plane Rear Wall D– First	>100	URM – N/A
		In-plane Rear Wall D - Ground	>100	URM with RC Frame - N/A

Building area	Loading direction	Specific review element	%NBS	Notes/Description of limiting criteria
Southland Times	Longitudinal (N-S)	Facade Wall – Parapet OOP	>67% >100%	URM Wall - Rocking Stability * Connections - N/A
		Facade Wall – Second Floor OOP	43% 80%	URM Wall - Rocking Stability * Connections Floor - Anchor Pull-out
		Facade Wall – First Floor OOP	81% 61%	URM Wall - Rocking Stability * Connections Floor - Anchor Pull-out
		Facade Wall – Ground Floor OOP	>100%	URM Wall – Stability – N/A *
		Internal Walls – Second Floor OOP	>100%	URM Wall – Stability – N/A *
		Internal Walls – First Floor OOP	>100%	URM Wall – Stability – N/A *
		Internal Walls – Ground Floor OOP	>100%	URM Wall – Stability – N/A *
		Rear Wall – Parapet OOP	55% 97%	URM Wall - Rocking Stability * Connections Ceiling- Anchor Pull-out
		Rear Wall – Second Floor OOP	33% 92%	URM Wall - Rocking Stability * Connections Floor - Anchor Pull-out
		Rear Wall – First Floor OOP	72% 64%	URM Wall - Rocking Stability * Connections Floor - Anchor Pull-out
		Rear Wall – Ground Floor OOP	>100%	URM Wall – Stability – N/A *
		Roof Diaphragm - Front	>100% 48%	GIB Sheathing - Deformation - N/A Connections – Anchor Shear
		Second Floor Diaphragm - Front	>100% 100%	Timber Boarding - Deformation - N/A Connections – N/A Embedded Joists
		First Floor Diaphragm - Front	>100% 32%	Timber Boarding - Deformation - N/A Connections – Anchor Shear
		Roof Diaphragm - Rear	>100% 55%	GIB Sheathing - Deformation - N/A Connections – Anchor Shear
		Second Floor Diaphragm - Rear	>100% >100%	Timber Boarding - Deformation - N/A Connections – N/A Embedded Joists
		First Floor Diaphragm - Rear	>100% 22%	Timber Boarding - Deformation - N/A Connections – Anchor Shear
		In-plane Wall West M – Second Floor	>100%	URM with minor masonry infill – N/A
		In-plane Wall West M – First Floor	83%	URM with minor RC Frame – URM Rocking
		In-plane Wall West M – Ground Floor	>100%	URM with minor masonry infill – N/A
		In-plane Wall East N Front – Second Floor	86%	URM with RC Frame – URM Rocking

		In-plane Wall East N Front – First Floor	>100%	URM with minor RC Frame – N/A
		In-plane Wall East N Front – Ground Floor	>100%	URM with minor RC Frame – N/A
		In-plane Wall East N Rear – Second Floor	97%	URM with RC Frame – URM Rocking
		In-plane Wall East N Rear – First Floor	>100%	URM with RC Frame – N/A
		In-plane Wall East N Rear– Ground Floor	>100%	URM – N/A

* The capacity is determined assuming that the connections are adequate to restrain the walls.

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) other than opening up of the floor structure to the North East and South West corners of the first and second floor and obtaining representative samples of the concrete bricks for strength.

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

- Basement
- Levels 1-3 (site measure of level 1, 2 and part 3)
- Site measure of critical wall 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:

- Level 3 (conclude site measure of level 3)
- Roof and gables (front element only accessible)
- Exterior elevations.

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.

Figure 19 – Concrete Masonry Brickwork with RC window infill and RC stairs tied into West side wall.

Figure 20 – Ground Floor North West corner evidence of efflorescence to the brickwork internal leaf.

Concrete brick samples were obtained from the areas shown on the plan in Appendix A and sent to Opus Laboratories in Christchurch for testing. The results of the testing are attached in appendix A.

It is believed that the basement and levels 1 through 3 of the building have been unoccupied for approximately two years. The building has been maintained to a good standard during the period it has not been occupied and currently does not appear to suffer from significant moisture ingress through the roof and exterior walls.

Figure 21 – North Elevation Façade exhibit cracks to the cornice and parapet cladding.

Figure 22 – North and East Elevation ties into floor diaphragm at first floor. Façade tie local to trimming joist only, minimal transfer path. East elevation fixings every second joist

Figure 23 – East Gable front element strengthening girt has no effect on outwards movement as no fixing to wall and only at one location.

Figure 24 – West Gable front element strengthening girt has no effect on outwards movement as no fixing to wall and only at one location.

11.2 Site Observations

Structurally the main observation was the number of penetrations to the internal shear walls as a result of the historic refurbishments which have removed areas of URM shear walls and replaced them with reinforced concrete frames. The nature of the strengthening works to the 1986 refurbishments have in locations, been found to be below the level of provision typically used in today's seismic strengthening regimes and this may well be the result of more robust assessment procedures which have increased the seismic demands for strengthening works.

Figure 25 – South Elevation inset RC frame

Figure 26 – Internal Grid B RC wall/frame

11.3 Building Condition Discussion

The Southland Times building is 110 years old and was strengthened and refurbished in 1984-6. It has minimal signs of deterioration that would affect its structural capacity although its age and construction technique is such that there are inherent capacity issues to the higher storey walls and parapets. New structures are normally designed using the current material codes for a durability of 50 years so the building has far exceeded its normal lifespan.

The façade remains the buildings primary feature of architectural merit. It is feasible that this element could be retained in any redevelopment of the remaining building footprint. A steel grillage would be fixed to the inner face of the wall with ties back to any new structure behind. The inner face of the masonry wall could then be stabilised by a spray concrete reinforced wall incorporating resin anchor dowels into the current wall construction if required.

The temporary stabilisation works during demolition and construction and subsequent additional load imposed on any new construction behind would impose considerable cost (over and above that for total demolition).

12.0 Seismic Concept Remedial Strategy

12.1 Primary Structure

The primary structure comprises the URM in-plane walls and the roof and ceiling diaphragms and as this is the case in both the transverse and longitudinal directions all the walls are considered primary elements although dependent upon the load direction they will switch from primary to secondary elements.

The findings of the assessment have determined that it is the wall to diaphragm connections that are deficient throughout the building which are limiting the buildings overall capacity initially.

The remedial action is to install additional fixings to the interface and detail to ensure the diaphragm can adequately transfer its load into the in-plane walls, which is currently not being provided by the original or the strengthening provisions.

Typical details for this work are shown in figures 27-28 adjacent.

12.2 Secondary Structure

The secondary structure elements requiring strengthening are the URM parapets and the upper storey URM walls under out of plane loading.

The URM parapet could be either reduced in height until its capacity is deemed adequate or install parapet tie backs along the back of all parapets. This will vary depending on the height of the parapet but will typically consist of a PFC strongback with a CHS tube strut to the roof line and fixed through the roofing to the steel trusses below. Vertical strapping will also be required at strut locations to tie the parapet down to the wall below the parapet base to resist uplift forces. All elements will require Chemical anchor fixings into the brickwork with the anchors fixed through to 50mm from the outer face of the brickwork wall in order to restrain or mobilise the full wall section.

The URM walls to the second floor will require an internal timber framework with 150x45 hySPAN studs @400mm crs to span from floor to ceiling and, adequately fixed to the same, with wall tie fixings into the URM wall picking up each wythe at regular vertical centres, as determined by detailed design, to each stud.

Typical details for this work are shown in figures 29 adjacent.

12.3 Façade Retention

The potential retention of the Esk Street façade can be engineered, if considered necessary / appropriate as discussed in section 11.3.

A concept of the steel grillage system to enable its retention is shown in Figure 30.

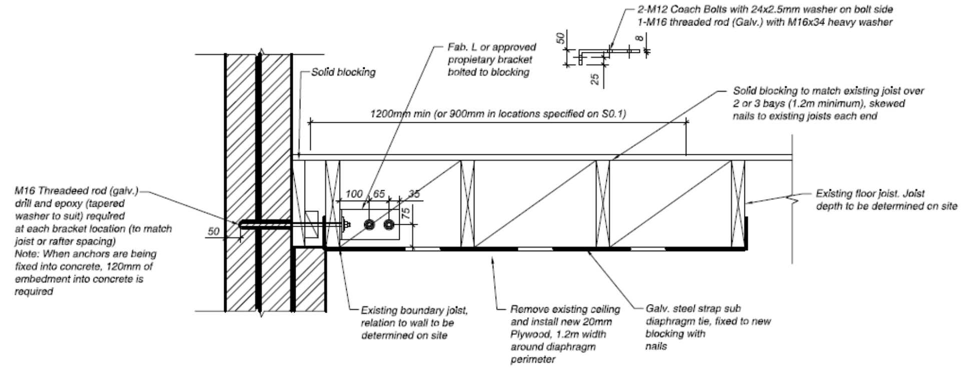

Figure 27 – Diaphragm to Wall typical remedial detail - Joist Parallel

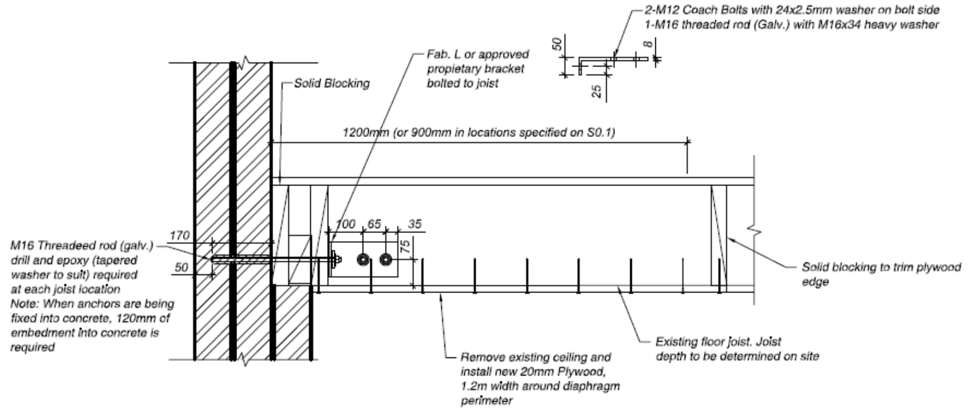

Figure 28 – Diaphragm to Wall typical remedial detail - Joist Perpendicular

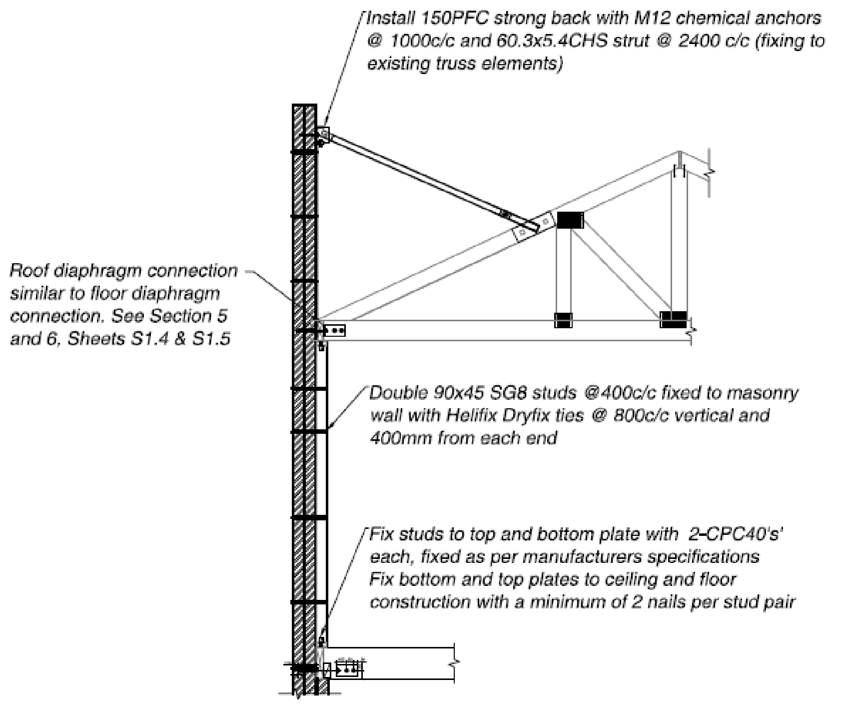

Figure 29 – Parapet Tie back and URM Out of plane restraint typical remedial detail

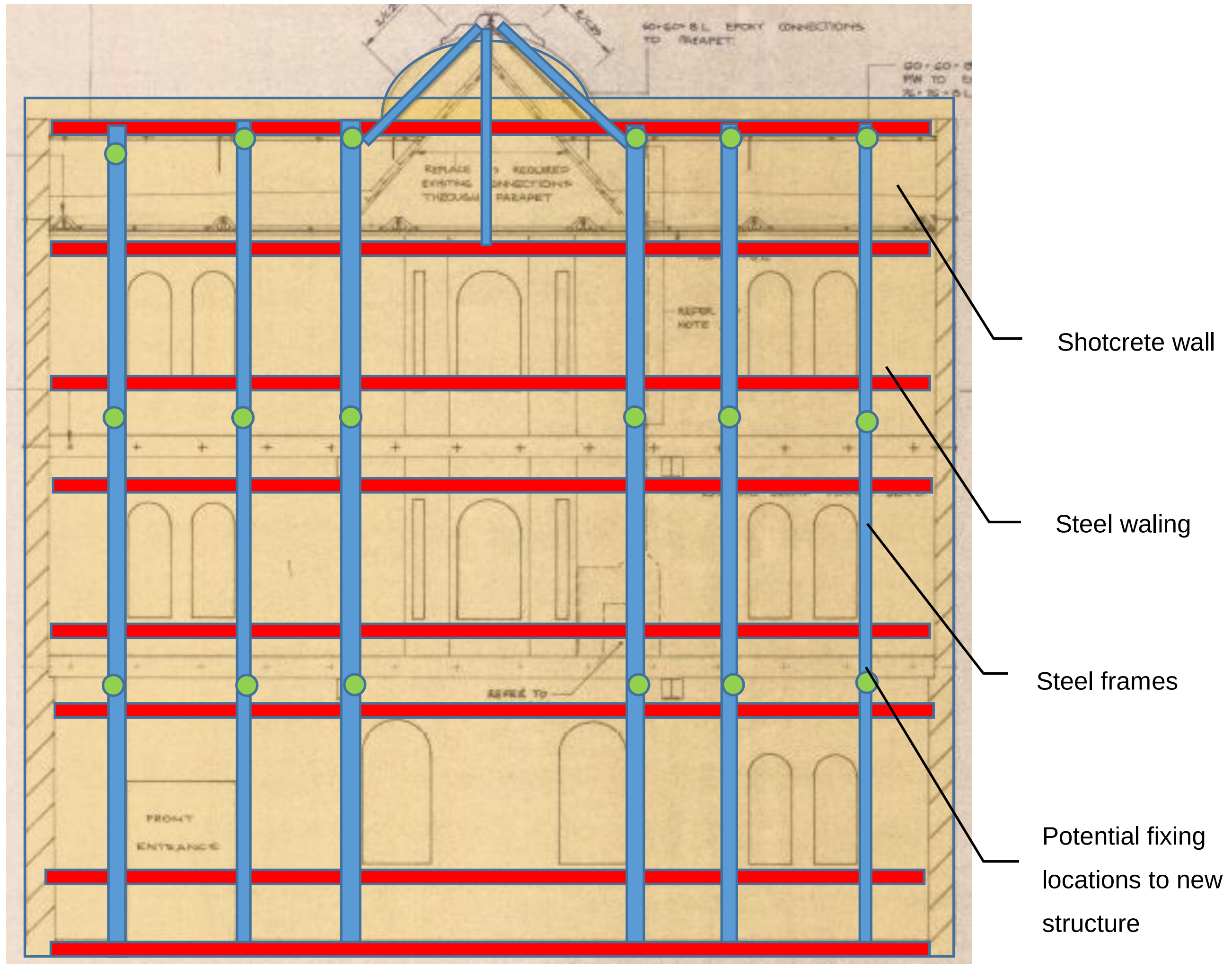

Figure 30 – Heritage Façade retention system Concept Sketch

13.0 Conclusions

The Southland Times building is currently 110 years old, it is three stories high with a small additional basement area and is constructed in unreinforced concrete masonry with timber floors and steel truss and purlin roofs in two intersecting roof forms (herein designated the front and rear elements). The building is essentially rectangular in plan with gables to the rear elevation and the front element side elevations. The façade to the North elevation is of particular architectural merit and the building has been awarded a category 2 heritage building status for this and its part in the development of Invercargill and Southland regional history.

The building is of a form of construction which is inherently at risk under seismic loads such that its construction methods no longer meet the building code guidance for new buildings. The building has had a number of extensions during its lifespan and a refurbishment in 1984-6 which included removal of some areas of the URM bracing walls with reinforced concrete frames being installed in their stead and installation of some strengthening works.

The building itself has shown no evidence of structural distress or defects which would cause a reduction in the assessed element capacities. Some of these capacities however are<33%NBS.

There are a number of issues with regards the diaphragm / wall connections being inadequate, URM parapets and upper walls being of a low seismic capacity under out of plane loads all of which have placed the buildings rating within the “Earthquake Prone” Building Classification (‘Grade D’ building).

There are established seismic strengthening details that can be adapted to raise the buildings capacity to the level required to satisfy the recommended strengthening requirements (>67%NBS).

The building form and large size is particular to its former use and its use as a standalone building in the current office and commercial market is limited evidenced by the fact it has been untenanted for the last 18 months despite significant marketing. It is also acknowledged that given the rate of development of assessment analysis techniques and research unto URM buildings its retention in the future it may become more and more onerous for its owners. It is possible the building façade can be retained and the remaining structure demolished for a new development on the site more suitable for the market and using more modern construction techniques to provide a more resilient structure.

14.0 Recommended Next Steps

Consideration of the following options needs to be explored by the relevant parties:-

Option 1 - Strengthen of the Heritage building and integration into the new development.
Option 2 - Demolition of the building in total.
Option 3 - Retention of the façade for incorporation into the master plan design and specific building to be constructed behind. Significant cost for option 3 would likely be incurred and specific funding from Government or Heritage NZ could be sought in this case.

A - Appendix A – Concrete Brick Compression Testing

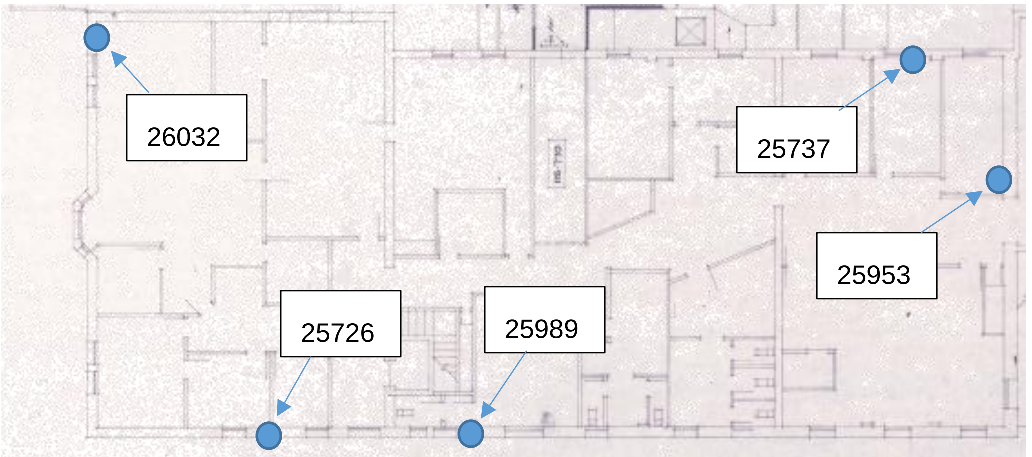

Figure 31 Location of Concrete Brick samples taken for compression testing (indicative only).

OPUS

RECYCLED BRICK COMPRESSIVE STRENGTH TEST REPORT

Project :	**Quality Assurance Testing**
Location :	**67 Esk Street, Invercargill**
Client :	**Batchelar McDougall Consulting Limited**
Contractor :	**Batchelar McDougall Consulting Limited**
Sampled by :	**Andrew Marriott**
Date sampled :	**22 January 2018**
Sampling method :	**Random Selection**
Sample description :	**Recycled Limestone Brick**
Sample condition :	**Air Dry as Received**

Project No :	**6-JBMCL.16/6LC**
Lab Ref No :	**CH3687**
Client Ref No :	**Job 1711-2266**

Test Results

Compressive Strength

Brick:	Ground Sth Wall Load Bay	Ground West Wall Storage	First Floor West Wall Office	First Floor Northwall Office	Ground East Wall Loading
Available Area (mm^2):	25953	25989	25726	26032	25737
Capping Material:			12mm Fibreboard		
Face's Capped:	Yes	Yes	Yes	Yes	Yes
Cavities Filled:			Fujirock Compound		
Density (kg/m^3):	2012	1905	1832	1935	1867.0
Load (kN):	349.6	427.0	289.1	454.6	**274.0**
Stress (MPa):	**13.5**	**16.4**	**11.2**	**17.5**	**10.6**
Average Stress (MPa):			**13.8**		
Standard Deviation:			**3.0**		

Test Methods: AS 3700:2001 Determination of Compressive Strength of Masonry Units

Date tested : 24 January 2018
Date reported : 25 January 2018

This report may only be reproduced in full

Approved

Designation : *Laboratory Manager*
Date : 25 January 2018

CSF 2004 (13/09/2006) Page 1 of 1

Opus International Consultants Limited
Christchurch Laboratory
Quality Management Systems Certified to ISO 9001

52C Hayton Road
Wigram, Christchurch 8042,
New Zealand

Telephone +64 3 343 0739
Facsimile +64 3 343 0737
Website www.opus.co.nz

B – Appendix B - Geotechnical Desktop Study

By Geosolve Limited Ref: 171019 - February 2018

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.

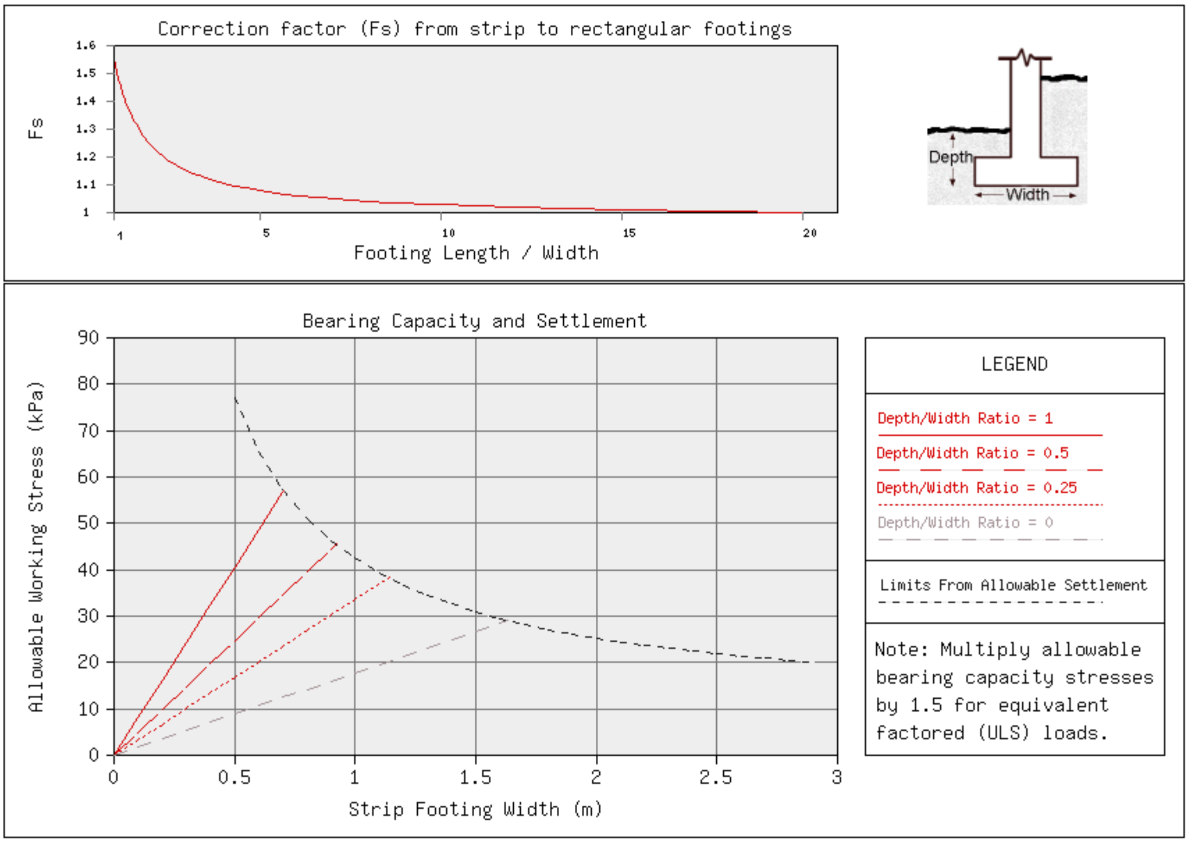

Figure 2: Typical Bearing for Shallow Footings on Alluvial Silt

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.

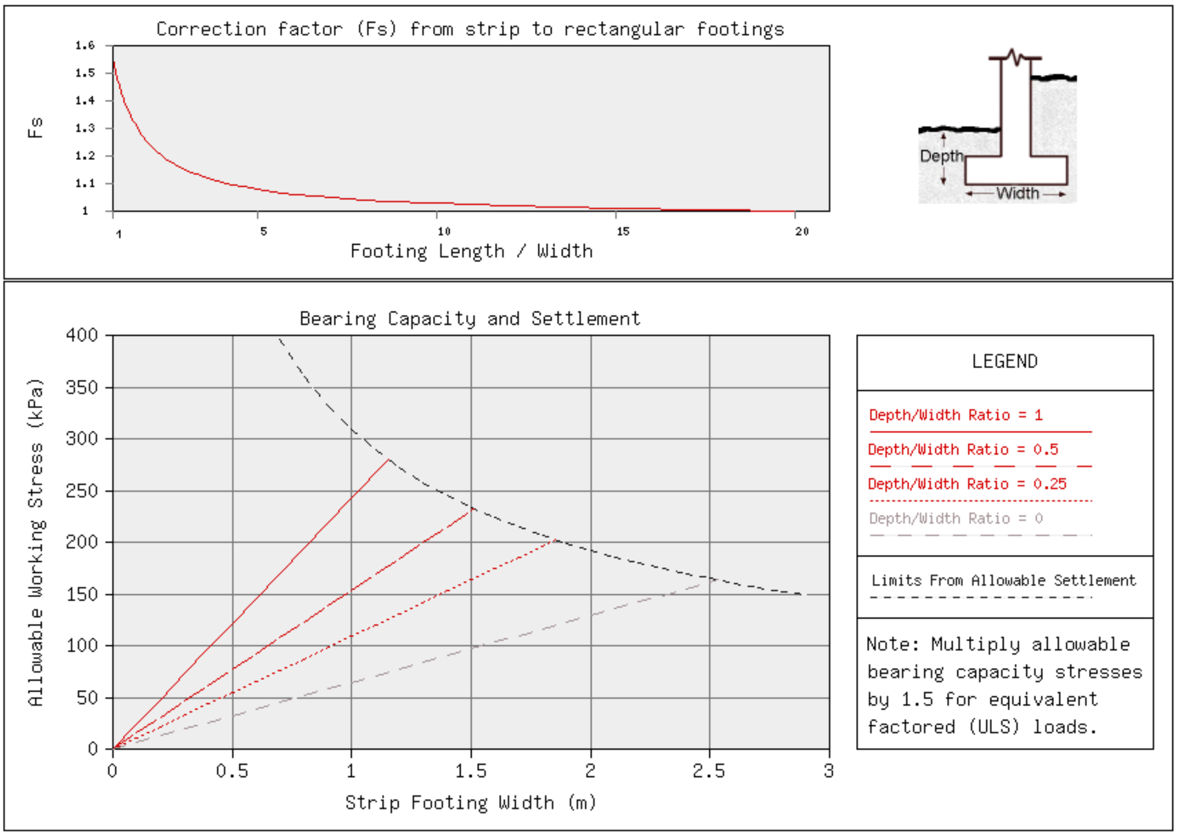

Figure 3: Recommended Bearing for Shallow Footings on Alluvial Gravel

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:	Reviewed for GeoSolve Ltd by:
....	....
Mike Plunket	Fraser Wilson
Geotechnical Engineer	Senior Engineering Geologist

Reviewed for GeoSolve Ltd by:

....

Colin Macdiarmid

Senior Geotechnical Engineer